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Study of dynamic impact factors of two-track continuous and integral railway bridge subjected to high-speed loads

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Abstract

The impact factor (IF) assessment of a four-span non-prismatic continuous and an equivalent integral railway bridge under the action of high-speed moving train loads is conducted in this study. Critical analysis of the full-scale three-dimensional finite element (FE) bridge models is done to investigate the differences in the IFs of all the spans of the two-track railway bridge under various loading conditions. A simplified approach is proposed to identify the dynamic IF values. The results show that for a continuous bridge, with the increase in the load on the bridge, IF coefficients reduce from 0.195 to 0.102. However, for the integral bridge, and considered loading conditions, almost similar IF coefficients (0.100) are obtained. For the intermediate spans, the resonance phenomenon for the integral bridge is achieved at lower speeds compared to the equivalent continuous bridge.

Keywords

Impact factor, FE model, Continuous bridge, Integral bridge, Two-track loading

1. Introduction

In the recent past, several researchers have been studying the cause of the impact induced due to the dynamic action of moving vehicles on various bridges (Green et al. 1995; Yang et al. 1995; Ichikawa et al. 2000; Brady and O'Brien 2006; Miguel et al. 2016; Gharad and Sonparote 2021). These authors termed this dynamic impact as the Dynamic Amplification Factor (DAF), Impact Factor (IF), Dynamic Load Factor (DLF), and Dynamic Increment Factor (DIF).

Various researchers studied the numerical and analytical estimation of the DIF values for different types of railway bridges under the action of high-speed trains. Yang et al. (1997) highlighted the importance of dynamic IF identification to study the resonance phenomenon induced in small and long-span simple bridges, under the action of high-speed moving loads. Gu et al. (2008) presented an economical numerical study to assess the dynamic IF of various railway bridges. Hamidi and Danshjoo (2010) evaluated the DIF values of various steel bridges to compare them with the different international codes. The IFs thus obtained were more than the values prescribed in these codes. Mu and Choi (2014) studied two railway bridges viz. continuous and simply supported to compare their IF values at resonance.

Youliang and Gaoxin (2016) investigated the experimental and numerical DLFs of different truss arch bridge members subjected to a high-speed railway and reported a linear relationship between the train speed and DLF. In their recent work, Gou et al. (2018) identified the DFs of an asymmetrical arch railway bridge under high-speed train loads. The authors concluded that the bridge's vertical amplitude is magnified by the sudden braking of the moving train.

Many researchers have put efforts to mitigate the impact effects on railway bridges under dynamic loading using energy dissipation devices. Soneji and Jangid (2005) considered various seismic isolation systems to study the reduction in the dynamic response of cable-stayed bridge structure under seismic loads. Pisal and Jangid (2016) studied various arrangements of tuned mass friction dampers (TMFD) responsible to reducing the resonance response of railway bridge. Chang (2020) developed an active mass damper to assess the reduction experimentally and numerically in the vertical vibration response of long-period bridges under dynamic loads.

Nevertheless, the IF values under high-speed moving trains had been evaluated for the simply supported and continuous railway bridges, as per the previous studies (analytical and experimental) no substantial work is reported to assess the dynamic IF values of an integral railway bridge under the high-speed train loads considering the effects of soil-structure interaction (SSI). However, recently the dynamic SSI analysis to assess IFs for prismatic bridges can be found (Gharad and Sonparote 2021), a comparative analysis of the integral and continuous bridges' IF values under high-speed moving load including the SSI effects for non-prismatic bridges is difficult to find.

Thus, in the present study, two types of four-span non-prismatic railway bridges (viz. continuous and integral) are considered to identify the IF values of each span. Two different loading cases of a real train are considered to assess the dynamic responses of these two railway bridges (of same configuration and boundary conditions) using finite element method (FEM). The effect of soil-bridge interaction on the IF values under the action of high-speed moving loads is also studied. In this work, the maximum midpoint vertical displacement of the railway bridge structure under the moving loads is assessed. The results so obtained are compared and the differences in the dynamic IFs of these equivalent bridges are studied. Also, suggestions to mitigate the impact effects on railway bridges under dynamic loading using different energy dissipation devices / systems are discussed in the concluding section.

2. Bridge-soil interaction models

The bridge models adopted by Gharad and Sonparote (2020) are considered in the present study. Figure 1 (a) shows a non-prismatic three-dimensional (3D), double-tracked, full-scale bridge model. The cross-sectional details (A-A, B-B, and C-C) of the continuous and integral bridges (Figure 2) at various locations can be seen in Figure 1 (b, c, and d). The various bridge parameters, finite element model in SAP2000, dynamic soil-bridge interaction analysis, and other related details can be read in Gharad and Sonparote (2019) and Gharad and Sonparote (2020).

a)

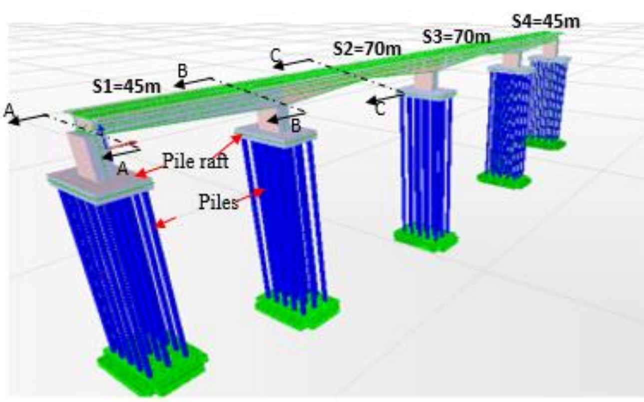


Fig 1. Model of (a) two-track ballast less bridge model

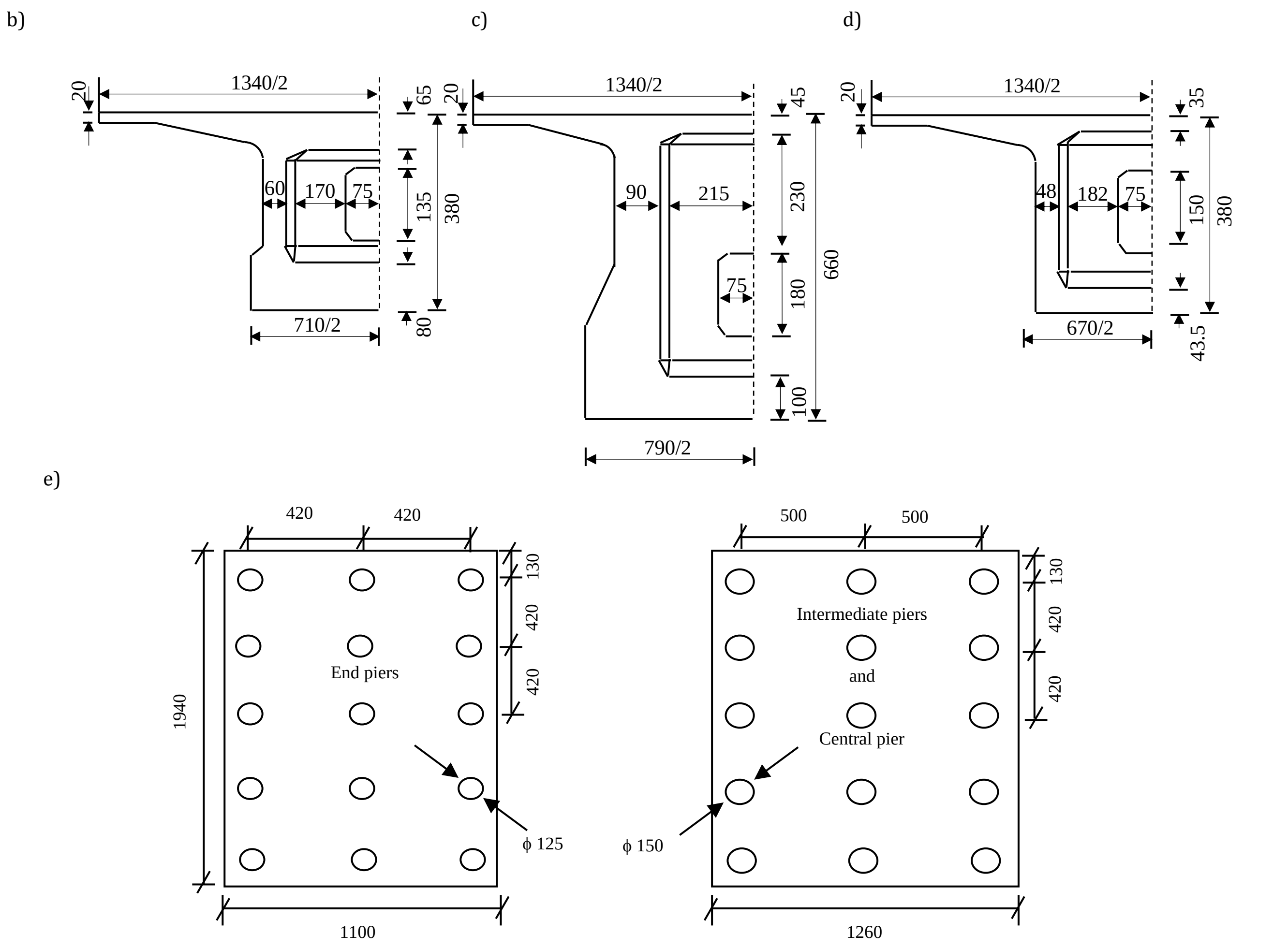


Fig 1. (b) cross-section of A-A (c) cross-section of B-B, (d) cross-section of C-C, and (e) arrangement of piles below end piers and intermediate and central piers (all units are in cm) (Gharad and Sonparote 2020).

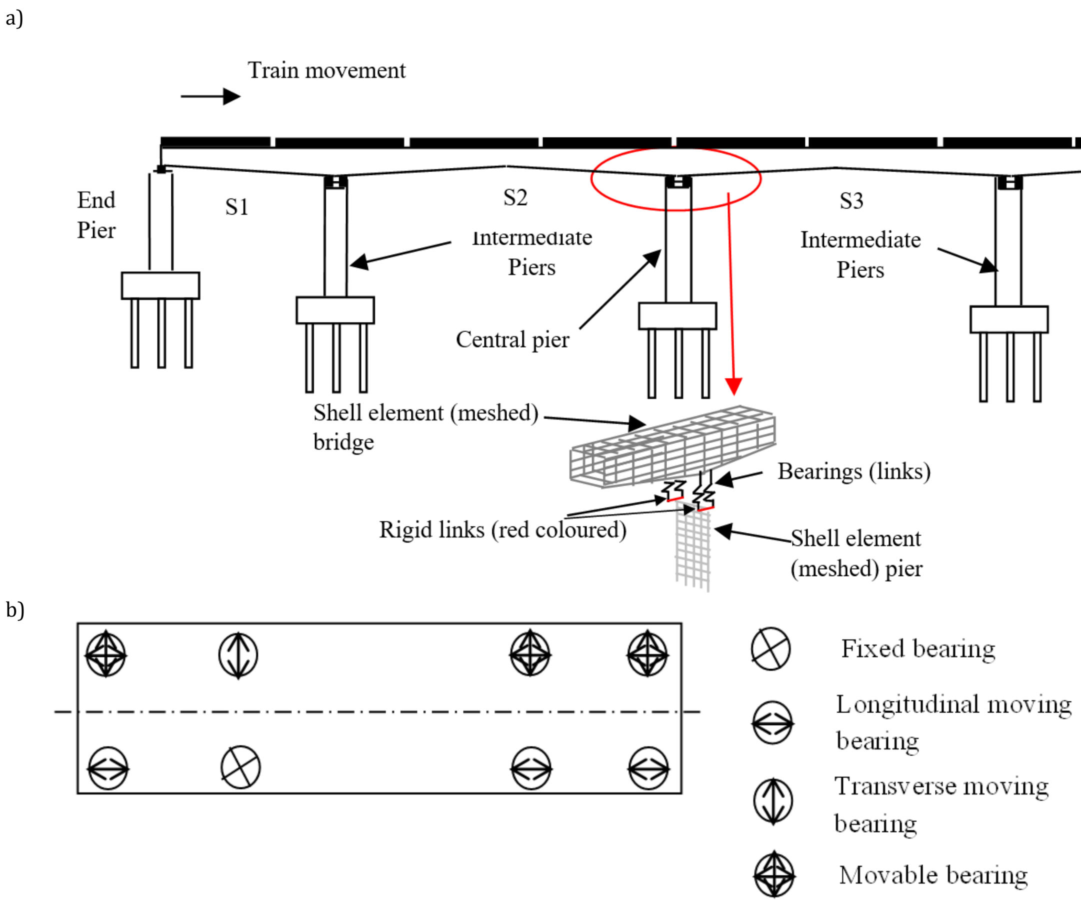


Fig 2. Models of (a) continuous bridge, (b) layout of bearings

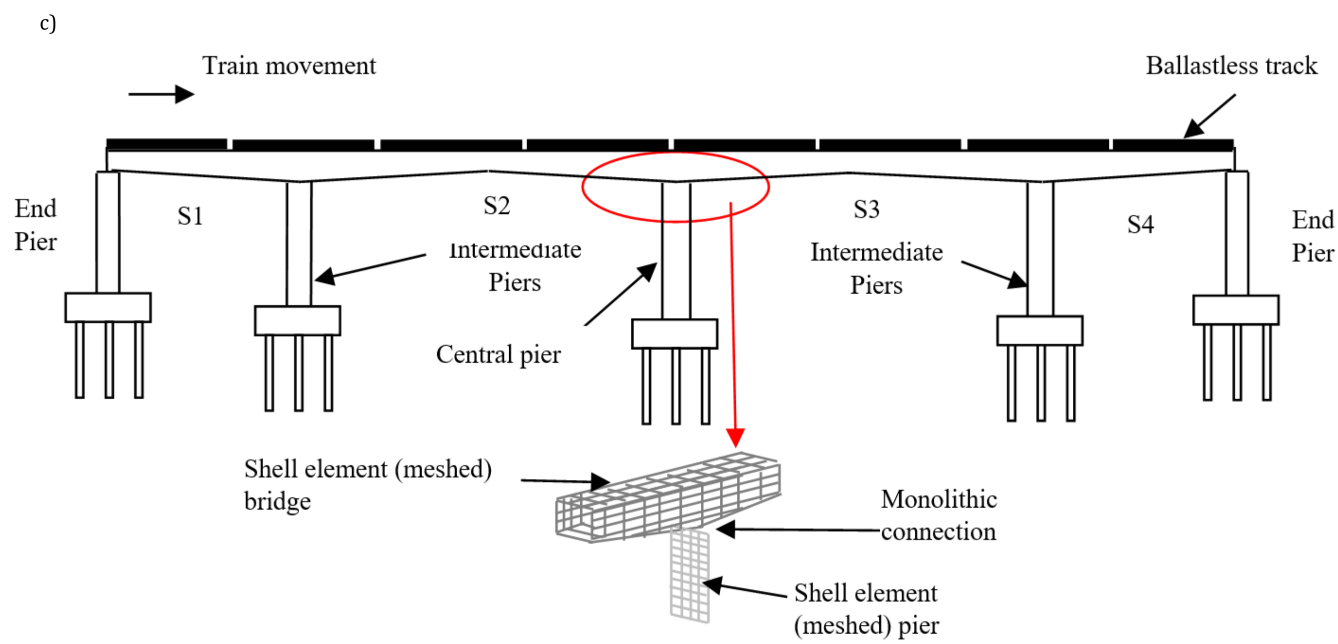


Fig 2. (c) integral bridge (Gharad and Sonparote 2019)

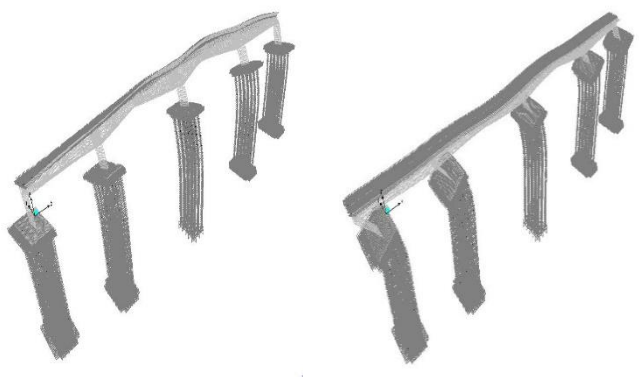
3. Modal analysis

To understand the dynamic characteristics of the bridge super-structure coupled with the raft-pile-soil sub-structure, modal analysis is conducted. Figures 3 and 4 show the first four natural frequencies and the mode shapes of the continuous bridge and integral bridges, respectively. Table 1 provides the details of the frequencies and their corresponding modal characteristics for both the bridges considered here.

This study concentrates on evaluating the maximum displacement response of the bridge structure under the moving loads only in the vertical direction. Thus, the contribution of the first four modes is considered in the dynamic analysis (Table 1) (Gharad and Sonparote 2019).

Table 1. First four natural frequencies and their modal characteristics of the bridges.

Mode #	Continuous bridge		Integral bridge	
	Frequency (Hz)	Modal characteristics	Frequency (Hz)	Modal characteristics
1	2.033	Lateral floating	2.142	Lateral floating
2	2.239	Longitudinal floating	2.296	Longitudinal floating
3	2.384	Twisting	2.474	Twisting
4	2.683	Vertical antisymmetric bending	2.773	Vertical antisymmetric bending



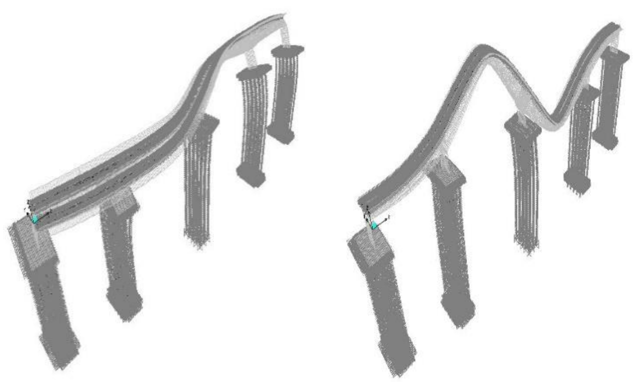


Fig 3. The first four mode shapes of the continuous bridge.

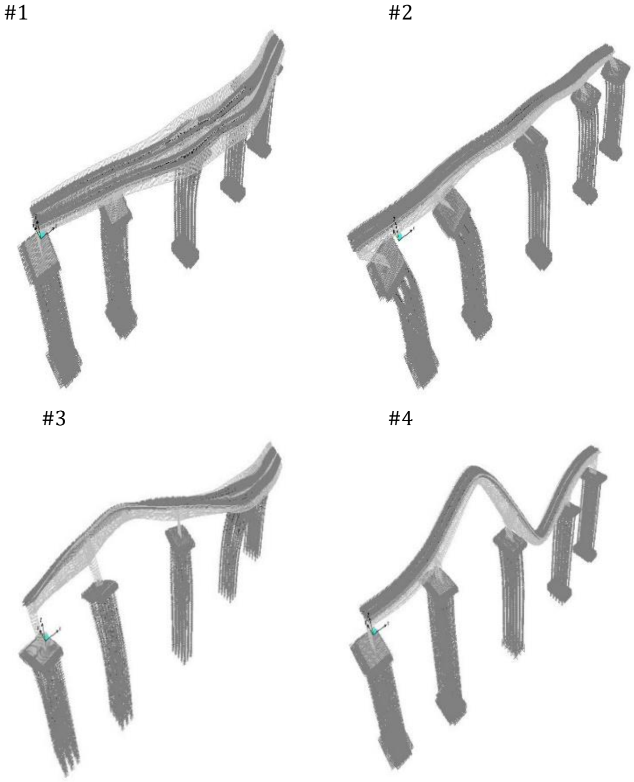


Fig 4. The first four mode shapes of the integral bridge.

4. Train load model

A simple moving load model which ignores the vehicle's inertia effect is adopted to study the soil-bridge interaction analysis. In the present work, the dynamic response of the bridge is calculated in the vertical direction. Thus, the train's inertial effects may be neglected (Liu et al. 2009). China Railway High-speed train (CRH3) real train load is considered for the present work. The axle loads for two motor cars and six passenger cars are taken as 160 kN and 146 kN, respectively. The total length of the CRH3 train is considered 200 m. The length of two motor cars and passenger cars is taken as 15 m. The distances between the axle loads, and the motor cars and passenger cars are 2.5 m and 5 m, respectively (Figure 5 c).

5. Dynamic Finite Element Analysis (DFEA)

The dynamic equations of the bridge system subjected to moving forces can be represented as:

$$\mathbf{M\ddot{U}_b} + \mathbf{C\dot{U}_b} + \mathbf{KU_b} = \mathbf{F_b} \tag{1}$$

where, **M, C, K,** respectively, is the mass, damping, and stiffness of the bridge system, $\mathbf{U_b}$ the displacement, $\mathbf{\dot{U}_b}$ the velocity, $\mathbf{\ddot{U}_b}$ the acceleration of the bridge, and $\mathbf{F_b}$ the external loads moving on the bridge. The damping matrix **C** considered here is proportional to the mass matrix **M** and the stiffness matrix **K** as:

$$\mathbf{C} = a_0\mathbf{M} + a_1\mathbf{K} \tag{2}$$

The damping ratio for the nth mode can be calculated using the following equation as:

$$\zeta_n = \frac{a_0}{2}\frac{1}{\omega_n} + \frac{a_1}{2}\omega_n \tag{3}$$

The coefficients a_0 and a_1 can be determined from specified damping ratios ζ_i and ζ_j for the ith and jth modes, respectively. For the same damping ratio ζ, the damping coefficients are as:

$$a_0 = \zeta\frac{2\omega_i\omega_j}{\omega_i+\omega_j};\ a_1 = \zeta\frac{2}{\omega_i+\omega_j} \tag{4}$$

The damping is evaluated from Eq. (2) and the damping ratio for any other mode, given by Eq. (3), varies with natural frequency. To determine the dynamic response of the bridge, Eq. (1) is solved in the time domain using Newmark-β method (Chopra 2008) with Newmark's parameters a_0 = 1/4 and a_1 = 1/2. Rayleigh damping coefficients are evaluated using the modal damping ratio of 0.037 (Gharad and Sonparote 2020). For the continuous bridge, the mass and stiffness proportional coefficients are 0.548 s^{-1} and 2.545×10^{-3} s; whereas, for the integral bridge these are 0.572 s^{-1} and 2.443×10^{-3} s.

6. Impact Factor (IF) assessment

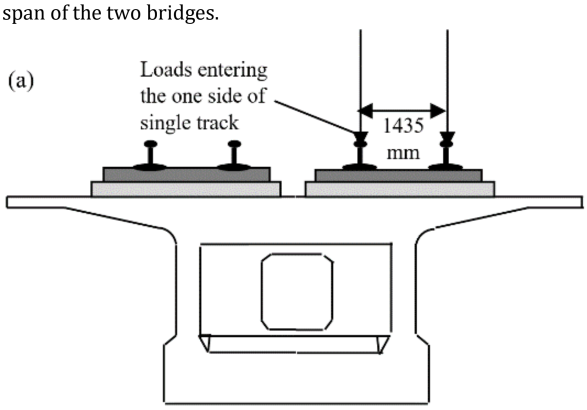
The calculation of impact factor (IF) is based on the dynamic and static responses of bridge structure, defined as:

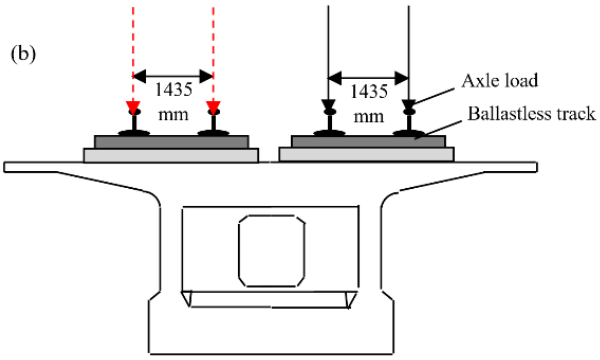
$$\mathrm{IF} = \frac{R_{dy} - R_{st}}{R_{st}} \tag{5}$$

where, R_{dy} = maximum dynamic response, and R_{st} = maximum static response of the bridge for the loads moving on the bridge at a speed of 0.1 m/s (0.36 km/h).

Two real train loading cases are assumed in the present study: (i) single track (1-track) is loaded with the train loads, and (ii) both the tracks (2-track) are loaded with the train loads facing each other and moving at the same instant (Figure 5a and 5b). Figure 6 shows the comparison of time history curves of maximum static and dynamic deflection when the loads are moving on a single track.

The impact factor (IF) values for the present work are identified using Eq. (5). All four spans of both the continuous and integral bridges are investigated for two loading cases. High-speed moving loads (variation from 60 km/h to 380 km/h, at 10 km/h interval) are considered in the dynamic analysis. The IFs are identified at the midpoint (displacements) of each span of the two bridges.





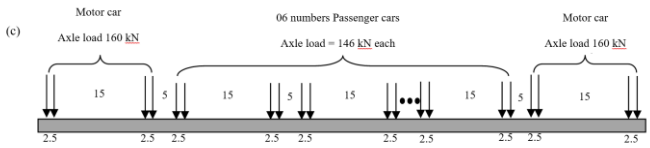


Fig 5. Railway track loading cases (a) 1-track, (b) 2-track, and (c) train load model (unit: m).

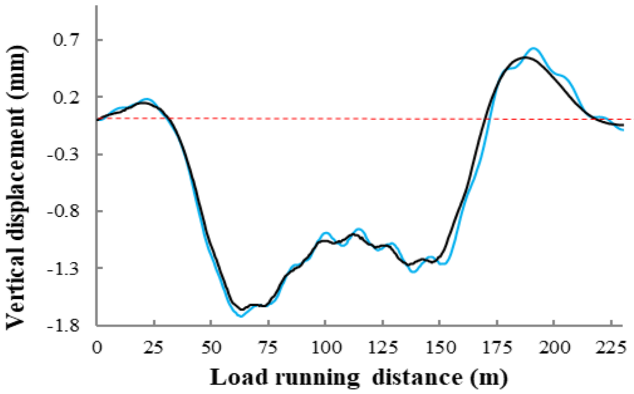


Fig 6. Comparison of time history curves of maximum static and dynamic deflection when the loads are moving on the single track.

6.1 Dimensionless speed parameter

Various parameters as the trains' speed, length of the span, mass of the structure, mode shape, natural frequencies of vibration, number of train axles, spacing of axles, damping value of the structures, characteristics of vehicle, soil conditions, track irregularities, etc. affect the dynamic behavior of bridge structure. It is sensible to equate the IF with a single term that will have an amalgamation of the aforesaid parameters. This may be denoted by a dimensionless speed parameter (**s**) represented by Eq. (6)

$$\mathbf{s} = \frac{\pi v}{\omega L} \tag{6}$$

where, v is the velocity of the train, ω is the fundamental frequency of the bridge, and L is the characteristic length which is the distance between the two inflection points of the first vertical bending mode of vibration of a continuous bridge (Yang 1995). Here, the trains' speed, length of the span, mass of the structure, mode shape, natural frequencies of vibration, number of train axles, spacing of axles, damping value of the structures, characteristics of vehicle, and soil conditions are considered to evaluate **s**.

6.2 Linear variation of IF

A linear variation is assumed to evaluate the conservative IFs for both bridges. This consideration can be justified in the following manner:

The speed parameter can be related to the ratio of the IF to the frequency ratio of a SDOF system. The IF increases with the value **s** ≤ 1. Also, this variation becomes almost linear when the damping ratio nears the zero value. Thus, the IF and the speed parameter **s** relation can be conservatively treated as linear and can be given by Eq. (7):

$$\mathrm{IF} = \mathrm{a}\mathbf{s} \tag{7}$$

where, a is the impact coefficient.

6.3 Continuous bridge's IF

The characteristic length of the adopted continuous bridge is identified as 36.8 m. The frequency considered here to determine the speed parameter **s** is 16.86 rad/s (mode #4). Based on these values, the IFs for each span of the assumed continuous bridge are shown in Fig. 7. From this figure it can be noted that the **s** value corresponding to the highest considered speed is well below 1. This information can be utilized to propose the IF formulation for the adopted continuous bridge. Thus, an attempt is made to identify a simplified IF value for the present bridge type. Figs. 7 a, b, c, and d are considered to identify the impact coefficients for both 1-track and 2-track loadings.

For the continuous bridge, the maximum impact coefficient values obtained for 1-track and 2-track loadings are 0.195 and 0.102, respectively (Fig. 7). The distinction between the IFs of these two tracks demonstrates the variations in the dynamic effects of the moving train loads entering the continuous bridge from different directions. Also, for 1-track loading, the maximum IFs of spans S1 and S4 are higher than the spans S2 and S3, respectively. In case of 2-track loading, there is an obvious increase of overall weight on the bridge. Hence, it can be observed that with the increase in loads on the bridge, the IF reduces.

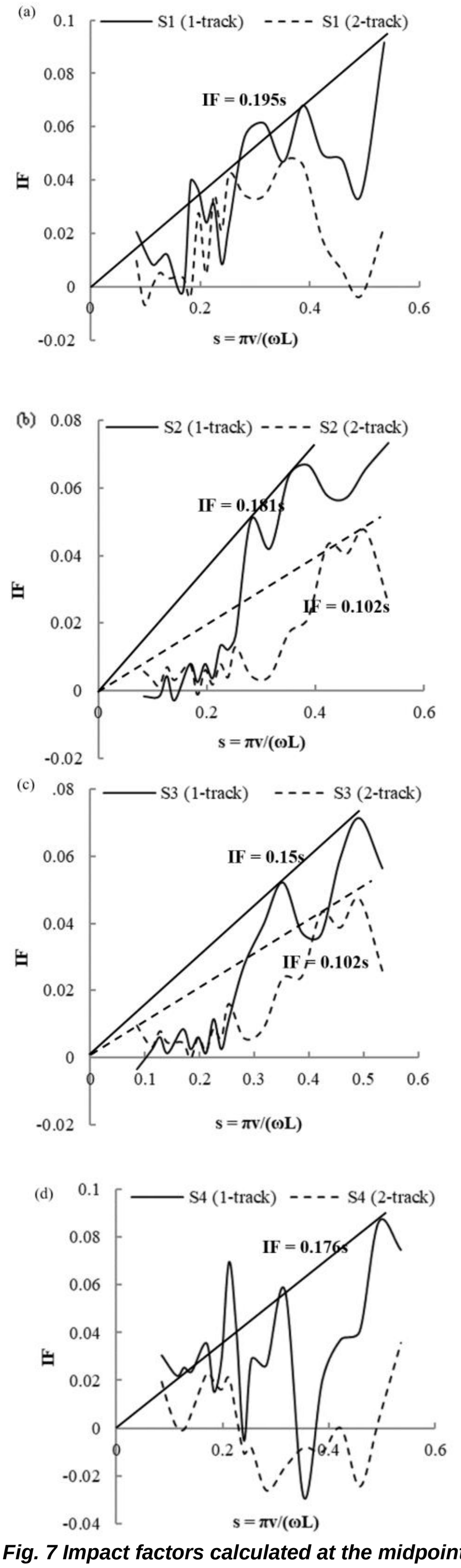


Fig. 7 Impact factors calculated at the midpoint of each span of the four-span continuous bridge.

6.4 Integral bridge's IF

The characteristic length of the adopted integral bridge is identified as 36.8 m. The frequency considered here to identify the speed parameter **s** is 17.02 rad/s (mode #4). Based on these values, the IFs for each span of the assumed integral bridge are shown in Fig. 8 (a, b, c, and d).

The IFs for integral bridge, however, show an interesting change in their variation with the speed parameter **s**. Here, IF values for both loading cases are almost similar. For spans S1 and S4, the IFs are observed to be increasing with **s**. However, for the spans S2 and S3 IFs are not observed to be increasing with **s**. Thus, average IF values are evaluated to decide the impact coefficients of these two intermediate spans. Figs. 8 (b) and 8 (c) represent the impact coefficients as 0.102 for span S2 and 0.100 for span S3, respectively.

From Figs. 7 and 8, the IFs for all the spans of the continuous bridge are more than that of the integral bridge. Since the integral bridge is stiffer than the continuous one, the lesser impact coefficient for all the spans is obvious.

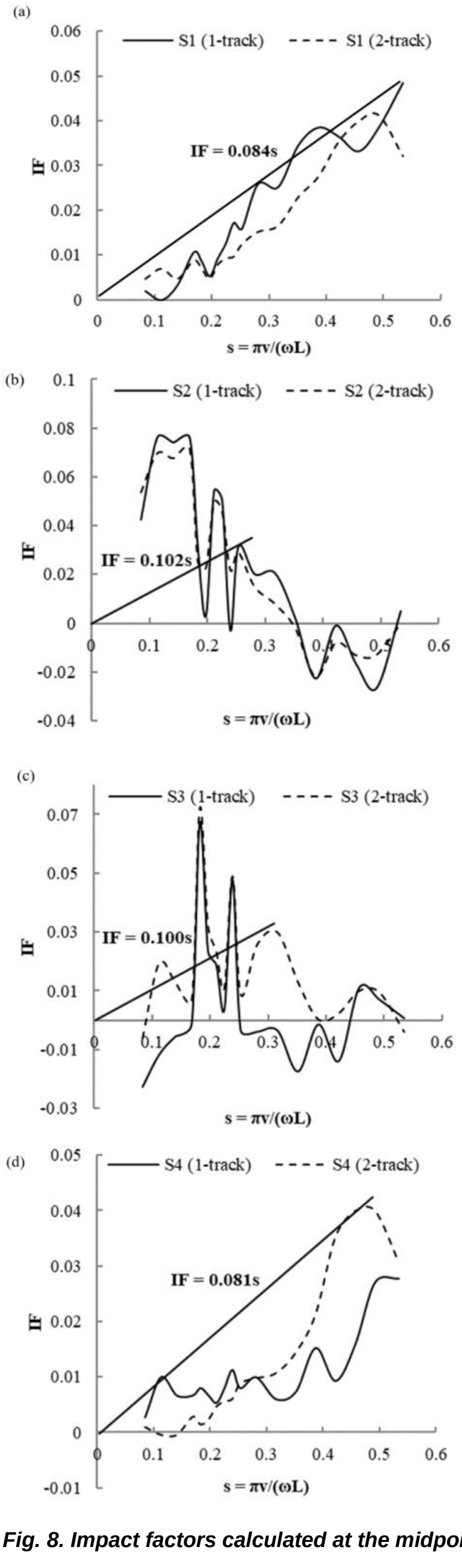


Fig. 8. Impact factors calculated at the midpoint of each span of the four-span integral bridge.

7. Codal provision for Dynamic Factor (DF)

The minimum DF value suggested by the Eurocode (2003) is 1.0, which is significantly on the higher side. Also, for this case, the following equation is adopted to evaluate the actual DF value denoted by Φ_2:

$$\Phi_2 = \frac{1.44}{\sqrt{L_\Phi} - 0.2} + 0.82 \qquad 1.0 \le \Phi_2 \le 1.67 \tag{8}$$

where, L_Φ = equivalent span length of the bridge, in meters, defined as follows:

$$L_\Phi = k\frac{\sum_{i=1}^{n} L_i}{n} \tag{8.1}$$

k = constant value determined as 1.2, 1.3, 1.4 and 1.5 for the number of spans n= 2, 3, 4 and ≥ 5, respectively.

Thus, from Eqs. 5.1 and 5, the values for L_Φ and Φ_2 are 80.5 m and 1.0 (since the obtained result of 0.984 is less than the minimum value), respectively. Thus, from Eq. 9, the IF will be zero.

$$DF = IF + 1 \tag{9}$$

In the present study, train speeds of more than 200 km/h are also considered to assess the IF coefficients. The resonance phenomenon is evident at these higher speeds (Figs. 7 and 8). Thus, non-zero IF coefficients are obtained, justifying that Eurocode 1 does not consider the resonance effect. Using a single bridge model and train loading case cannot provide a clear idea related to the IF values, and thus, without considering comprehensive parameters, the impact coefficient cannot be proposed.

8. Discussion and Conclusions

A four-span non-prismatic double-tracked continuous bridge and an equivalent integral bridge are evaluated under the action of the high-speed real trainload to identify the impact coefficients for each span. Two different train loading cases are assessed to find the variations in the dynamic impact factor (IF) values of the two bridge types. A non-dimensional speed parameter is adopted to decide the impact coefficients. The IF values are obtained for the midpoint displacements of each span of the two bridges and are compared. Based on the comparison, the following conclusions can be stated:

For the considered continuous bridge, the distinction between the IFs of the two tracks demonstrates the variations in the dynamic effects of the moving train loads entering the bridge from different directions. The intermediate spans show less IF values (0.181s, 0.15s) compared to the end spans (0.195s, 0.176s). In the case of 2-track loading, there is an obvious increase in overall weight on the bridge. Hence, with the increase in loads on the bridge, the IF reduces.

Due to its monolithic construction, the considered integral bridge is stiffer than the continuous bridge. Thus, the integral bridge has reduced impact coefficients (maximum value = 0.102s) than the continuous bridge (maximum value = 0.195s).

This study considered a four-span non-prismatic continuous and an equivalent integral railway bridge under the action of high-speed moving train loads to evaluate the impact coefficients. However, parameters like the track irregularities, variation in the bridge dimensions, its length, different soil types, and vehicle-bridge interaction effects were neglected. These parameters may be considered in future studies to verify the changes, if any, in the proposed IF values for the integral and continuous high-speed railway bridges. Also, various vibration controlling measures / systems such as tuned mass dampers, base isolators, actuators, etc. can be used to further assess the dynamic response of these bridges under different soil conditions.

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