Structural Engineering and Mechanics, *Vol. 53, No. 1 (2015) 89-104*
DOI: http://dx.doi.org/10.12989/sem.2015.53.1.089

A new look at the restrictions on the speed and magnitude of train loads for bridge management

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(Received August 12, 2013, Revised June 24, 2014, Accepted October 2, 2014)

Abstract. In current bridge management systems (BMSs), load and speed restrictions are applied on unhealthy bridges to keep the structure safe and serviceable for as long as possible. But the question is, whether applying these restrictions will always decrease the internal forces in critical components of the bridge and enhance the safety of the unhealthy bridges. To find the answer, this paper for the first time in literature, looks into the design aspects through studying the changes in demand by capacity ratios of the critical components of a bridge under the train loads. For this purpose, a structural model of a simply supported bridge, whose dynamic behaviour is similar to a group of real railway bridges, is developed. Demand by capacity ratios of the critical components of the bridge are calculated, to identify their sensitivity to increase of speed and magnitude of live load. The outcomes of this study are very significant as they show that, on the contrary to what is expected, by applying restriction on speed, the demand by capacity ratio of components may increase and make the bridge unsafe for carrying live load. Suggestions are made to solve the problem.

Keywords: rating bridges; train loads; axle spacing; criticality; speed restriction; bridge management

1. Introduction

Rating bridges based on their structural condition is one of the most important parts of Bridge Management Systems (BMS). In every bridge management system, one of the most important aims is to determine whether the bridge is safe and serviceable to credible live load, as bridges conditions deteriorate with age. Therefore, continuously monitoring the condition of its critical structural components and rating them, in order to evaluate the health of a railway bridge is essential, and it is one of the significant tasks of managers and engineers. In addition, engineers try to predict the condition of the critical components through different methods such as Markov chains and Weibull-distribution and improving them (Agrawal *et al*. 2010). Because the structural behaviour of a bridge and its ultimate capacity is mainly dependant on the condition of its critical components (Austroads 2004, AS5100. 7, 2006). Here, critical components denote those components in which any failure can cause the failure of a portion or the collapse of the whole

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http://www.techno-press.org/?journal=sem&subpage=8 ISSN: 1225-4568 (Print), 1598-6217 (Online)

structure (Catbas *et al*. 2008, Bridge Inspection Committee 2010). Depending on the type of loads applied to the structure including Live Load, Earthquake, Wind, Collision, or Flood, the criticality of components will change (Aflatooni *et al*. 2012, 2013).

Live load is of the great interest, as the purpose of a bridge is to carry live load with the acceptable level of safety. Live load changes over time, as new types of trains with different loads and speeds are used in railway systems regularly. To keep an old bridge serviceable as long as possible, sometimes load and/or speed restrictions are applied. However, sound dynamic analyses are required to determine the effect of changes in load and speed on the demand by capacity ratios of the critical components of a bridge.

In this paper, the term demand refers to the internal stresses in different components of the bridge induced due to the peak excitation caused by the loads applied to the structure including dead and live loads. Capacity means the strength of different components subjected to different types of the loads. For instance, simply supported beams are prone to bending moment forces, therefore their capacity will be their strength towards bending moments, but for columns that are subjected to axial and bending forces, their capacity will be calculated based on the combined effects of these forces.

Many researchers have studied the dynamic behaviour of bridges to live load (e.g., Chan and O'Connor 1990, Memory *et al*. 1995, Kwark *et al*. 2004, Sieffert *et al*. 2006). Chan *et al*. (2003b, a) investigated the bridge responses to twisting and pitching modes. Xia *et al*. (2000) investigated the dynamic behaviour of the suspension bridges under train loads. Their studies showed that the dynamic interaction between the bridge and train is not significant. Fryba (1996, 1999) thoroughly explained the vibration of structures and dynamics of railway bridges. Xia *et al*. (2000) developed formulations for a three dimensional model of a suspension bridge and applied it to an existing long span suspension bridge. The results do not show any significant interaction between train and the real bridge. Kim (2011) conducted experimental studies to investigate the influence of track structure including rail, sleeper, ballast on the railway bridge.

Lee *et al*. (2006) evaluated the dynamic response of a monorail bridge by establishing a procedure, including analytical, experimental and field test. According to their investigations, the reason for the lateral displacement of the monorail bridge is that torsional loads are applied to the bridge due to the eccentricity between the vertical load of the train and the shear centre of the bridge. The focus of the all the above studies was on some particular modes or only on some specific response. The effects of the increase of the speed or load of the train considering the ultimate capacity of the critical components of the bridge have not been investigated.

The analytical and experimental investigations of Senthilvasan *et al*. (2002) on curved bridge depicted the effect of speed of a moving vehicle on the Dynamic Amplifications Factor (DAF). This study shows that DAFs will not necessarily increase with the speed of vehicle. DAF indicates the increase in response of a bridge due to the dynamic effect of the motion of a single moving load and it does not consider the resonance effect of a moving load with multiple axles (Liu *et al*. 2009).

The resonant vibration of railway bridges was investigated by Xia *et al*. (2006). The outcome of their research identifies the natural frequencies of the train motion, the train shape and the axle spacings, the span length and the stiffness of the bridge in lateral and vertical directions, as the main parameters for resonant vibration of railway bridges. The studies of Liu *et al*. (2009) identified the speed of the train, the bridge damping ratio, the vehicle by bridge mass ratio, and the vehicle by bridge natural frequency ratio as the factors which have significant impact on the dynamic behaviour of the bridge.

The investigations of Majka and Hartnett (2008) show that, damping of the vehicle does not have a considerable impact on the response of the bridge. According to the studies mentioned above, the parameters, which have significant impact on the dynamic behaviour of the bridge and resonance in vibration, were identified, but the impact of this resonant vibration on the critical components of the bridge is still required to be investigated. Therefore, it can be concluded that the focus of the past research was on evaluating the dynamic response of the bridge when it is subjected to train loads. The effect on internal forces such as moment, axial, shear or the combination of them induced by train loads, with respect to the capacity of different components has not been taken into consideration. In other words, the susceptibility of the different critical structural components of the bridge to the changing of the magnitude of the train load and/or speed of the train has not been taken into account.

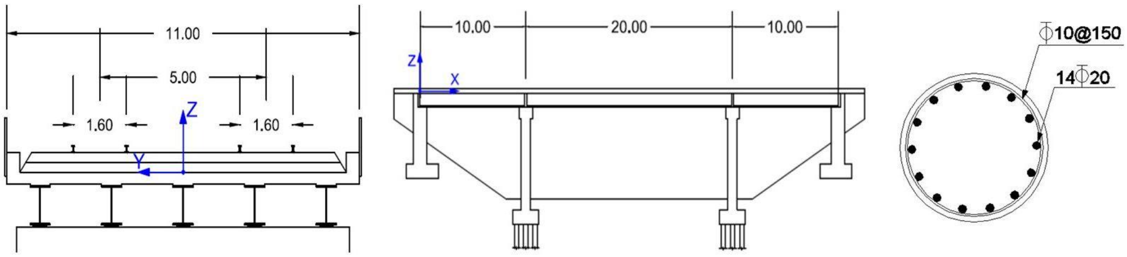
In recent years, criticality and vulnerability analyses have been conducted to much more reliably evaluate the condition of a bridge. In this method, different criticality and vulnerability factors are considered (Wong 2006). Criticality and vulnerability analyses are based on quantifying the criticality of the components of the bridge as well as identifying the critical factors which can have significant impact on the health of the structure including the effects of loads and environmental factors. The technical term vulnerability is related to the environmental impacts on the condition of the bridge.

As can be observed, although in the past there are numerous literature on the study of structural behaviour of bridges under moving train loads, the capacity of the different components such as columns and beams have not been taken into account. This research for the first time in literature will focus on evaluating the sensitivity of the critical components of a bridge to the train loads through calculating the demand/capacity ratios. The effect of the track structure including the ballast, track, etc on the dynamic responses of the bridge has been identified to be insignificant according to the study reported by Cheng *et al*. (2001). Since the present study considers a full scale bridge (similar to real bridges) details such as rail, sleepers, and ballast are not taken into account in the scope of this research. The results can be used for quantifying the criticality of the components in the synthetic rating procedures developed by Aflatooni *et al*. (2014). Demand means the internal stresses generated in components due to live and dead load. The capacities of the different components are the combined strength capacities for carrying internal axial forces and moments and are calculated, based on properties of the structural member, e.g., beams, columns and diaphragms.

The unique, important outcome of this research will be its anticipated influence on the decisions made by engineers and managers for applying load and speed restrictions on vulnerable railway bridges. Moreover, the results can be used for the interpretation of the data collected from Structural Heath Monitoring (SHM) systems. SHM systems are the advanced methods of monitoring the behaviour of the structure. More information on the development of these methods in Australia can be seen in a book edited by Chan and Thambiratnam (2011).

2. Modelling

To investigate the impact of the increase of load and speed of the train on the critical structural components of the railway bridge, a 3D finite element model of the bridge is created by using CSI



(a) Deck (b) Spans (c) Column Section

Fig. 1 The geometry of the structure, and the cross section of the columns

Bridge Software[1]. Fig. 1 shows the geometry of the bridge under consideration. The bridge has been designed and checked for different load combinations based on AASHTO LRFD (2007), ACI 318-05/IBC (2005) and AISC360-05/IBC (2005). All the above codes' requirements including, stress ratio limits, deflection limits, stress reduction factors, and other specifications have been taken into account.

Two bents and two abutments support the whole deck, and three columns transfer the loads of each bent (Figs. 1 and 4). Circular column e.g., C1 and C2 as shown in Figs. 1(c) and 4, with 7000 mm clear height and 700 mm diameter are considered. Clear height means the length of the column from the top of foundation to the bottom of the bent beam. Fourteen Nos. of 20Ø steel bars are used in columns, as longitudinal reinforcement while 10Φ bars at 150mm spacing are provided for confinement. L100×100×10 are utilized for diaphragms D1 to D3, as shown in Fig. 4. The space between diaphragms is 5 meters.

The composite deck with I steel girder (e.g., P1 to P6 as shown in Figs. 1(a) and 4) are used. The height of the I section is 1170 mm, thickness of flange is 30 mm, thickness of web is 16 mm. The thickness of the concrete slab is 300mm and it is modelled with shell elements. In order to take into account the interaction between the I section and the concrete slab of the composite deck, the deformation of the top flange of I section is constrained to the deformation of the concrete slab at their connection surface. Table 1 shows the section properties of each individual component of the bridge.

The two side spans are 10 m long and the middle span is 20 m. The spans are simply supported structures. The reason for conducting this research on a simply supported railway bridge is that, these types of bridges are widely used in Australia and therefore, their maintenance cost is high. Another reason for modelling a three span bridge is, to take into account different load conditions. Because, the train load can be on one, two or all three spans at a time.

Although the spans are simply supported, because of the continuity of the trainload, the deflection of columns can change the supporting condition of the middle span and the vibration of whole bridge. At bent supports, translations in all directions are fixed and all the three rotations about their local axes are free. At the abutments, translation in vertical direction and rotation about longitudinal axis are fixed and all other degrees of freedom are free.

Fig. 2 shows the train load applied to the bridge. Two trains move across the bridge in opposite directions with the same speed, and enter the bridge at the same time. For different speeds and

[1]CSI Bridge is a structural and earthquake engineering software, developed by Computers and Structures, INC. 1995 University Avenue Berkeley, California 94704 USA.

Table 1 Frame section properties

Section Name	Material	Shape	Area	I33	I22	AS2	AS3
			m2	m4	m4	m2	m2
ABUT	Concrete	Rectangular	4.730000	8.818651	0.394167	3.941667	3.941667
Cap Beam	Concrete	Rectangular	1.300000	0.108333	0.183083	1.083333	1.083333
Column	Concrete	Circle	0.384845	0.011786	0.011786	0.346361	0.346361
L100	Steel	Angle	0.001900	1.800E-06	1.800E-06	0.001000	0.001000
PG1	Steel	I/Wide Flange	0.038760	0.008648	0.000215	0.018720	0.017500

Table 2 Integration parameters

Gamma	Beta	Alpha
0.50	0.25	0.0

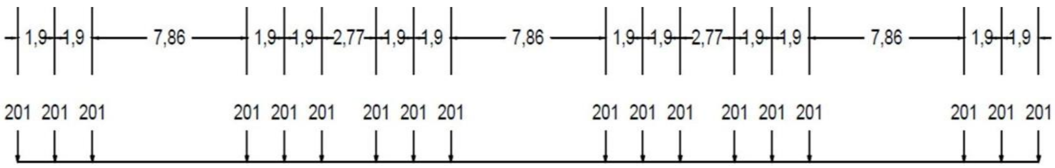


Fig. 2 Moving load (forces are in KN and distances are in meter)

loads, linear dynamic structural analysis is conducted. Eq. (1) shows the dynamic equation of motion. In order to capture dynamic effects, time history (direct integration) load case has been selected instead of static moving load case. Hilber-Hughes-Taylor (HHT) dynamic time integration method, which is an implicit method for solving transient problem, is used. HHT is unconditionally stable for linear problems (Hilber *et al*. 1977). In direct integration method, unlike mode superposition method, the dynamic equations of motion (e.g., Eq. (1)) are integrated through numerical method and prior to any transformation of the equations to any other forms (Bathe, 1982).

$$M\ddot{U} + C\dot{U} + KU = R \tag{1}$$

In Eq. (1) *M*, *C* and *K* are respectively mass, damping and stiffness matrices. Vector *R* is external load and time dependent, $\ddot{U}$, $\dot{U}$ and U are respectively, acceleration, velocity and displacement vectors.

Time integration parameters are shown in Table 2. The time step size considered is 0.05 second. A time function and a transient time history motion type have been considered. The time function increases the load (e.g., Fig. 2) from zero to its full value in one time step and then it decreases to zero immediately for next time step. By using this time function, at any given time within a time step, the applied load will be determined through a linear interpolation of the load pattern at the beginning and the end of the time step.

The Dead Loads applied to this bridge are, the weight of the structural components calculated by CSI Bridge Software. The magnitude of the Superimposed Dead Load on the deck, due to weight of ballast, rail, sleepers, and non-structural components etc calculated to be 10 KN/m^2, and applied to the bridge.

In order to show that the dynamic behaviour of the model is similar to real bridges, the Natural

Table 3 Summary of natural frequencies of simply supported bridges (Chan and O'Connor 1990)

Bridge	**Span *L* (m)**	***f* (Hz)**	**Bridge**	**Span *L* (m)**	***f* (Hz)**
Six Mile Creek (1^{st})	11.28	10.8	Beatrice Creek	9.094	8.0
Six Mile Creek (2^{nd})	13.72	8.0	George Creek	14.95	4.5
Bremer River (1^{st})	11.43	10.3	Coomera Overpass	20.95	2.2
Bremer River (2^{st})	13.72	8.1	Basin Creek	20.75	5.7
Goodbye Creek	13.38	7.9	Pioneer River	25.0	4.0
Sandy Creek	11.276	12.2	Currumbin	27.95	4.4
St. Aranadus Creek	11.4	10.3	Black River	23.95	4.7
Deebing Creek	15.0	3.9	Coochin Creek	28.75	4.2
Armstrong Creek	13.95	4.9	Rollingstone Creek	22.95	5.2
Emerald Creek	16.95	3.9	Plane Creek	25.8	3.9

L: Span length
f: Natural frequency of Bridge

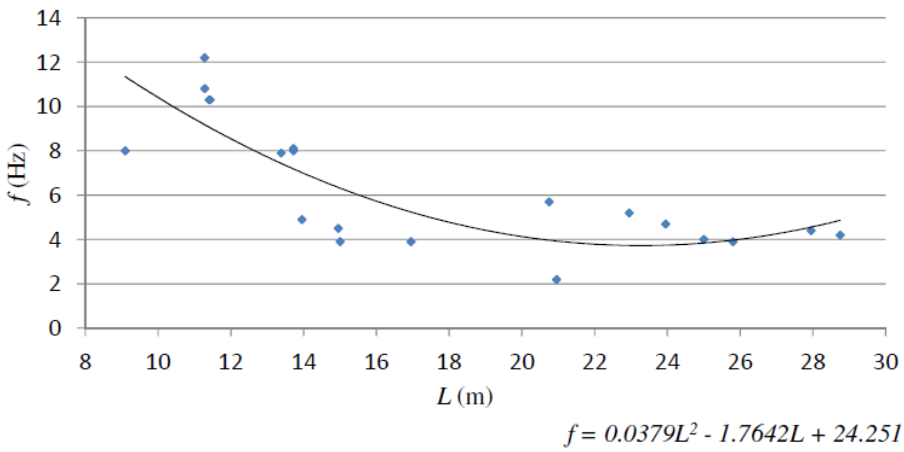


Fig. 3 Relationship between span length and natural frequency of the above bridges

Frequencies of the twenty real bridges with respect to their span lengths investigated by Chan and O'Connor (1990) and shown in Table 3 have been used. Similar to the model developed in this research, these bridges are simply supported bridge. As observed, the span lengths of the bridges mentioned in Table 3 are between 9.094 m to 28.75 m and the span length of the middle span of the model developed in this research is 20 m. Therefore, in order to validate whether the dynamic behaviour of this model is similar to real bridges, it is necessary to find an equation, which estimates the relation between span length of a real simply supported bridge with its dominant natural vertical frequency. Fig. 3 shows the relation between span length and natural frequencies of the bridges mentioned in Table 3. According to this Figure, by increasing the span length of the bridge the natural frequency will decrease.

Based on Fig. 3, Eq. (2) can be formulated, which may represent the relationship between span and natural frequency of real bridges of the type considered here. A second order function is selected for Eq. (2), considering that in simply supported beams, the frequency is related to the second order of the span length.

$$f = 0.0379L^2 - 1.7642L + 24.251 \quad (2)$$

From Eq. (2), for the span L=20 (m), the frequency can be calculated as 4.127 Hz, which is close to the natural frequency of the developed model in this research (e.g., 3.97 Hz). The 20 m span is equal to the span length of the middle span of this model. Although it is obvious that the natural frequency changes based on the stiffness and mass of the bridge components; here this comparison has been conducted to show that, the dynamic behaviour of this model is similar to the dynamic behaviour of a group of real bridges shown in Table 3.

3. Results

Structural analyses are conducted on this model, considering moving loads with different speeds and magnitudes. Speeds from 20 to 300 km/hr, and different magnitudes of live load by multiplying the moving loads shown in Fig. 2 by coefficients from 0.8 to 1.8. This increase in load is considered to include a variety of train types with different load configurations. For the present studies, load combinations of Dead + (coefficients from 0.8 to 1.8) × Live have been used for analysis and design of this bridge. The same load combinations have also been used with different speeds. Demand/capacity ratio for different components and different load magnitude and train speed are calculated. Based on ACI 318-05/IBC (2005), columns have been checked for axial force and biaxial moments. In addition, they have been checked for shear in both directions. Beams and Diaphragm members have been checked for stresses due to axial and shear forces and biaxial moments according to AISC360-05/IBC (2005). The calculations have been classified in 3 cases as follows.

- Case 1: Increasing the speed of the train from 20 to 300 km/hr, without increasing the magnitude of load. The load factor is considered 1.0 in this case.
- Case 2: Increasing the load by multiplying the train load by the coefficients from 0.8 to 1.8 and without changing the speed. The speed is considered 100 km/hr.
- Case 3: Increasing the speed form 60 to 140 km/hr and increasing the magnitude of load by multiplying the train load by the coefficients from 1.0 to 1.8.

3.1 Case 1

Figs. 5(a)-(d) show the demand/capacity ratios of the components: C1 and C2, P1 to P6, and D1 to D3. In this section of the study, the focus is on the effect of changing speed on the demand/capacity ratio of the structural components. To more reliably study the resonance in vibration at different speeds, a wide range of speeds is taken into consideration. Speeds higher than 140 km/hr are applied to only show the significant excitation of the bridge, and the behaviour of the bridges at high speeds is not studied in this research.

As can be observed in Fig. 5, distinctive peaks appeared at certain speeds in the middle span. These occur at about 124 and 258 km/hr in the columns, and about 65 and 258 km/hr in the girders. These peaks mean larger forces will be applied on those components. In order to investigate the reason of this phenomenon, modal analysis has been conducted.

To calculate the natural frequencies and mode shapes of the bridge, the eigenvalue and eigenvector method has been adopted. The dominant vertical natural mode of the structure is shown in Fig. 6. Fig. 7 shows the train resultant loads from loads shown in Fig. 2 applied on the

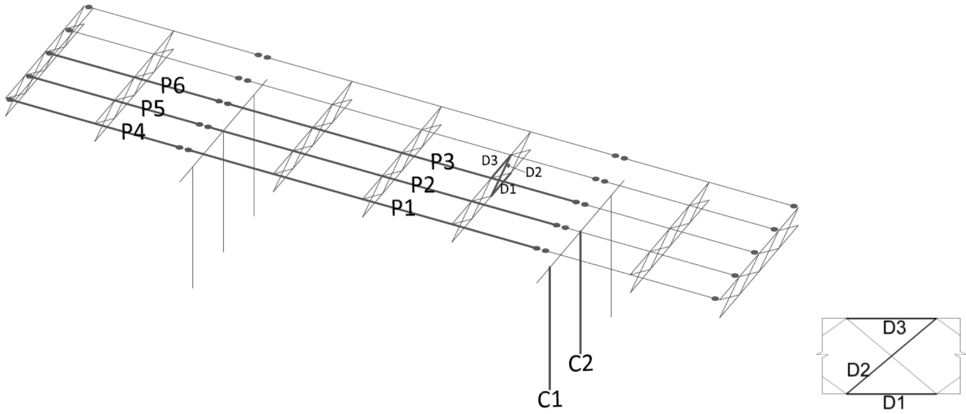


Fig. 4 Critical component (Dots denote simply-supported boundary conditions of beams)

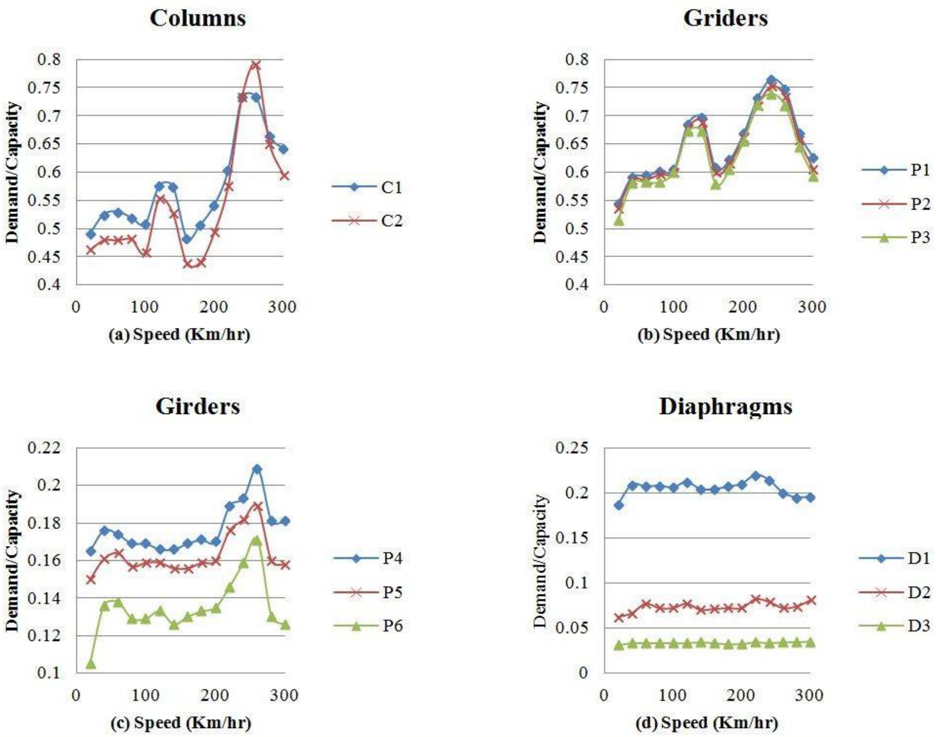


Fig. 5 Demand/capacity ratio of the bridge structural components Vs speed of the train

structure with different speeds. The resultant loads considered here, are the summation of forces applied by a group of axles of one bogie. By taking into account the resultant forces, the frequency of the load can be more easily calculated. The resultant forces shown in Fig. 7 were only used to explain that the dynamic behaviour of the model is similar of that of real bridges shown in Table 3, and the reason for resonance. For all the analyses and designs and demand by capacity ratios calculations including all the results shown in Fig. 5 and Figs. 8 to 11, the real loads shown in Fig. 2 was taken into account.

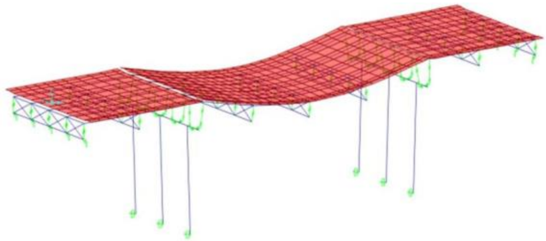


Fig. 6 Natural dominant mode shape (5th) of the bridge (frequency: 3.97 Hz, Period: 0.252 Sec)

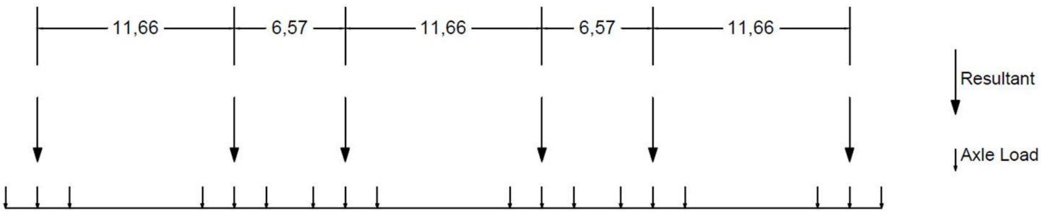


Fig. 7 The resultant forces of each 3 close axles

From the Fig. 7, the average distance between the resultant loads is 9.115 m. The frequency of the vehicle load (f_t) can be obtained from Eq. (3)

$$f_t = \frac{v}{x} \tag{3}$$

Where:
v: Velocity of the train
x: Average distant between the resultant forces

Using the speeds at the peak values in Figs. 5(a) and 5(b), the maximum Demand by Capacity ratios are calculated at the speeds 124 and 258 km/hr. From Eq. (3) the frequencies of the loads (f_t) at the above speeds were calculated and shown in Table 4:

Table 4 The speed and frequency of the moving load

v (km/hr)	f_t (Hz)
124	3.8
258	7.9

By comparing the frequency of the vehicle with the natural frequency of the vertical mode of the bridge, which is the dominant one, and equal to 3.97 Hz, the reason for occurrence of resonance at peak points can be explained. As can be observed in Fig. 5, the only peaks occur when the frequency of the load is equal to the dominant natural vertical frequency of the bridge (as shown in Fig. 6) multiplied by an integer.

In order to investigate the effect of speed restrictions on simply supported railway bridges based on real condition, the load shown in Fig. 2 and the speeds between 20 km/hr to 160 km/hr are taken into account. Fig. 5(a), shows that when the train used in this research passes over the bridge with the speed about 160 km/hr the minimum demand/capacity will be in C1. It means that applying any speed restrictions will increase this ratio and make the condition worse. Fig. 8 shows

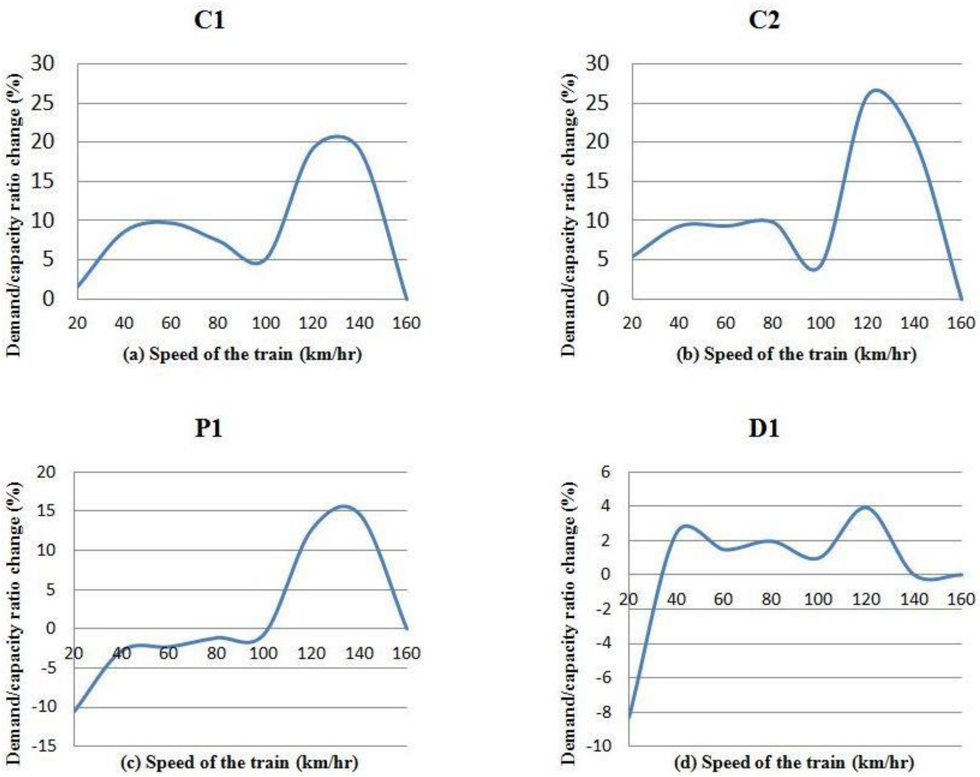


Fig. 8 Increase of the demand/capacity ratios of the critical components, when the speed of the train reduces from 160 to 20 km/hr

Table 5 The maximum increase in demand/capacity capacity ratio in percent due to the changes in speed of the train from 20 to 300 km/hr

Component	**Demand/Capacity Changes**	**Component**	**Demand/Capacity Changes**
C1	52%	P5	26%
C2	80%	P6	63%
P1	41%	D1	18%
P2	41%	D2	33%
P3	44%	D3	13%
P4	27%		

the changes in demand by capacity ratios of the components C1, C2, P1 and D1, when the speed of the train reduces from 160 km/hr to 20 km/hr. These figures shows that, if the speed limit decreased from 160km/hr to 120 km/hr, the ratio of demand/ capacity will increase about 20% for C1 and 26% for C2. If the current speed limit is 100 km/hr and this speed limit reduces to 60 km/hr, the above ratio will increase about 4% for C1 and 5% for C2.

Fig. 8 also shows that when the speed reduces from 160 to 140 km/hr the demand/capacity ratio of P1 will increase by about 15%. When the speed reduces form 140 to 100 km/hr, this ratio will also decrease by about 15% and almost equal to the time that this train will pass over the

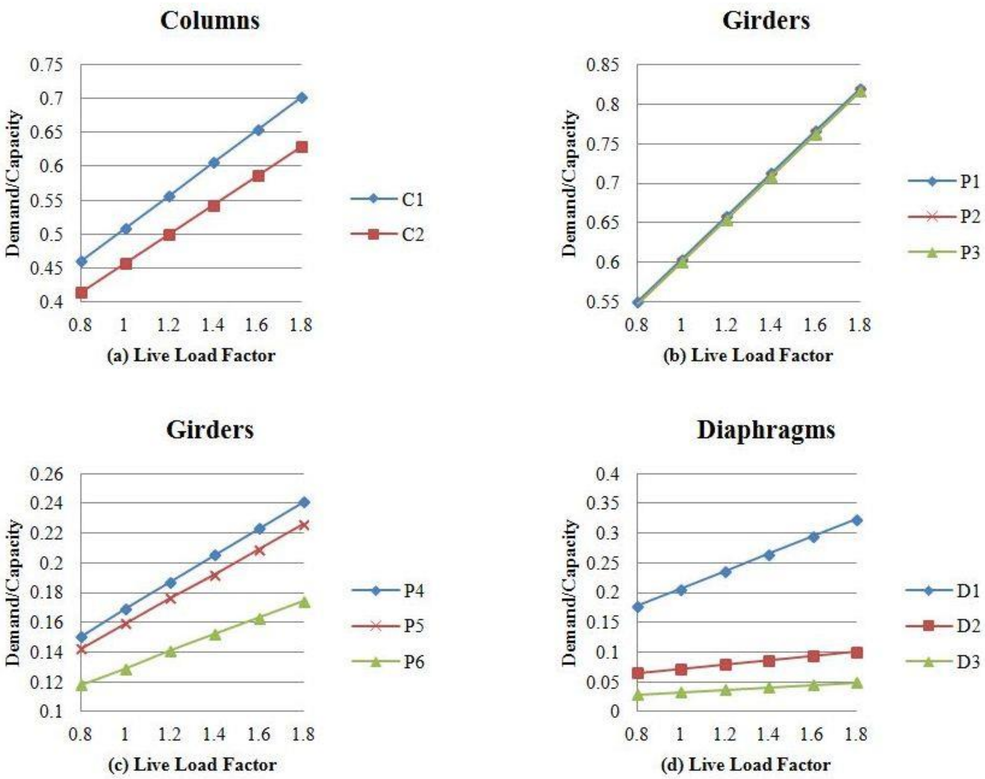


Fig. 9 Demand/capacity ratio of different structural components with respect to the increase of live load when the speed is constant and equal to 100 km/hr

bridge at 160 km/hr. However, from 100 to 40 km/hr this ratio will not change significantly. This means that by applying speed restriction form 100 to 40 km/hr, the above ratio for girder P1 will not considerably change. The significant decrease in the demand/ capacity ratio can be seen in almost all components, when the speed reduces beyond 40 km/hr. For high speed trains the increase or decrease of speed can have a huge effect on the demand/capacity ratios. For the model developed here as can be observed from Table 5, if the speed of the train decreases from 260 to 160 km/hr, the demand by capacity ratios will increase by 80%. This increase in load can cause catastrophic collapse of the structure.

Table 5 shows the columns are more sensitive to the increase of speed than girders, especially the middle column. Changes in demand/capacity ratio in the middle column due to the increase of speed within the range of 20 km/hr to 300 km/hr is about 80% which is almost twice more than each girder in the middle span which is about 41%. The diaphragm components are less sensitive to the increase of speed compared to girders and columns. The results also show that the sensitivity of different components of the same type (e.g., girders) are different and it depends on their position in the structure.

3.2 Case 2

In case two as mentioned before, the speed of the trains does not change, but the magnitude of

Table 6 The maximum increase in demand/capacity capacity ratio in percent due to the increase of the train live load factor from 0.8 to 1.8

Component	Demand/Capacity Changes	Component	Demand/Capacity Changes
C1	52%	P5	59%
C2	52%	P6	47%
P1	49%	D1	82%
P2	49%	D2	54%
P3	49%	D3	69%
P4	61%		

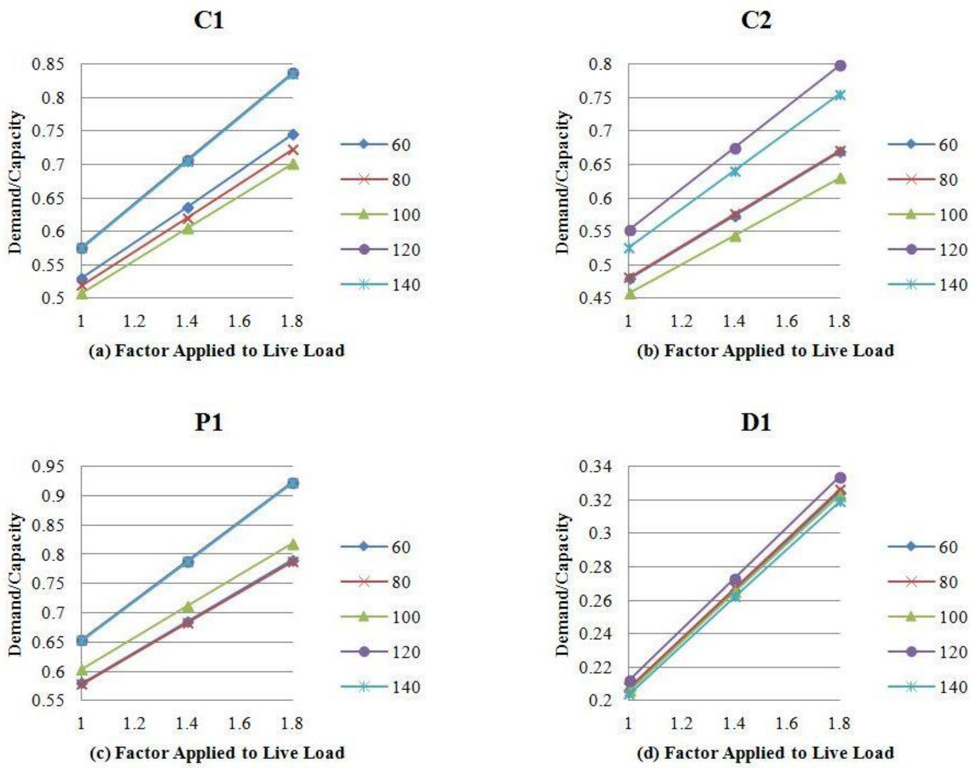


Fig. 10 Demand/capacity ratio of the bridge structural components with respect to the increase of live load and speed

the trains loads (e.g., Fig. 2) are increased from 0.8×train load to 1.8×train load. Fig. 9 shows the demand by capacity ratios of the different components of the bridge. As can be observed in Fig. 9, by increasing the load, the demand by capacity ratios of all the different components will increase in linear form. However, the rates of increase are different for the different components (columns, girders, and diaphragm).

Table 6 shows the maximum increase in demand/ capacity ratio in percent due to the changes of the load from 0.8×train load to 1.8×train load. The results also show that, except diaphragm components, the sensitivities of components of the same type to the increase in load, are not considerably different. For instance, by increasing live load from 0.8×train load to 1.8×train load,

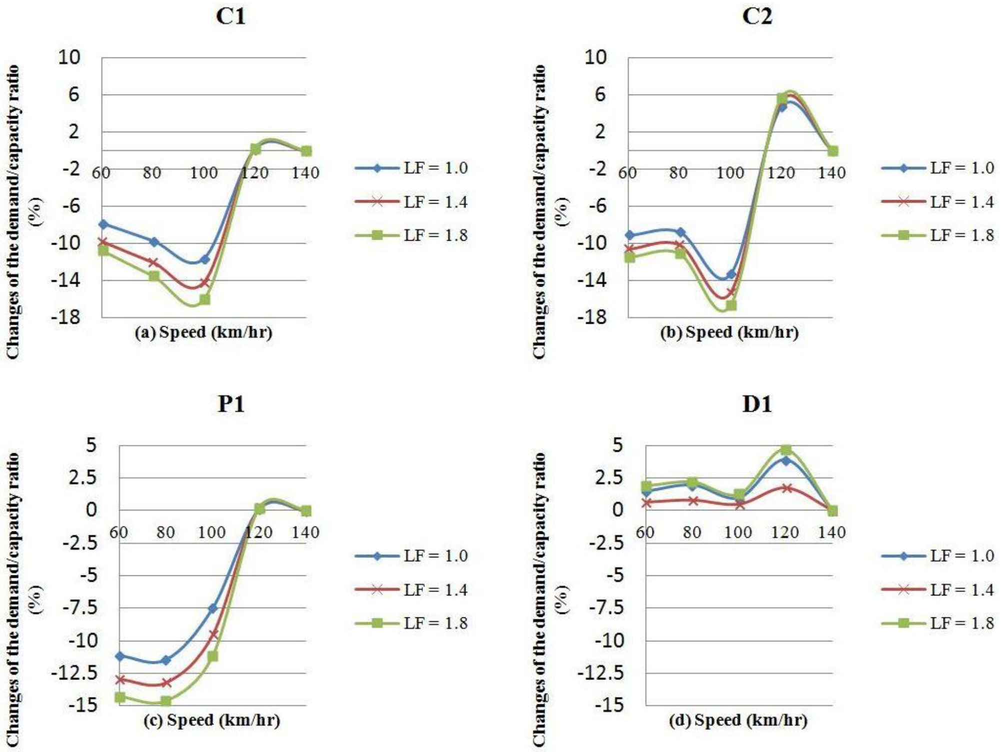


Fig. 11 Changes of the Demand/capacity ratio of different structural components in percent with respect to the increase of live load and speed

the increase of demand by capacity ratio of all columns are almost identical and equal to 52% and all girders are almost 49%. However, it can be observed that the Demand by Capacity ratio of the diaphragm component D1 increases about 82% when the load increases by 125%, which is about 67% more than girders and 58% more than columns.

3.3 Case 3

In case 3, the effect of the both increase of load and speed will be studied. Fig. 10 shows the effect of both increase of loads and speeds. The train load (e.g., Fig. 2) increased from 1.0×train load to 1.8×train load and the speed increased from 60 km/hr to 140 km/hr. It can be observed that the dynamic effect of the speed of the train on vertical vibration response of the bridge can have a high impact on the response of the structure including internal forces and displacements in critical components.

For example according to Fig. 10, for column C1, the demand by capacity ratio when trains with 60 km/hr speed pass over the bridge are higher than when trains cross at 80 or 100 km/hr. Therefore, by applying speed restriction on bridges, without detailed investigations, can lead to catastrophic failures rather than fulfilling its intended purpose. In addition, by applying speed restriction on damaged railway bridges that have lost some of their capacities, the effect of fatigue may become more severe, as a result of likely increase in magnitude of internal forces in critical components. Increase in the demand (internal stresses) have affect on fatigue damage (Polepeddi and Mohammadi 2000). According to Imam *et al*. (2008) investigations, the increase in loads significantly affect the remaining fatigue life of railway bridges.

As seen from Fig. 10, by decreasing the speed from 100 to 60 km/hr, the demand/capacity ratios increase by up to 6% in columns for a LF of 1.8. The changes in demand/capacity ratio of different component have been calculated and shown in Fig. 11. This figure shows, for C1 and C2 and P1, at each specific speed, when the load increases, changes in demand/capacity ratio will increase. For D1 these changes are small and do not increase with respect to the increase of the load.

4. Conclusions

Rating railway bridges are normally carried out through evaluating the structural condition of the critical components of the bridge. Based on the criticality of these components and the magnitude and speed of applied loads, the overall condition of the bridge can be evaluated. To keep an old bridge serviceable for as long as possible sometimes load and speed restrictions are applied to some bridges. In this paper the effect of increase of load and speed on critical components have been investigated though performing dynamic analysis. Demand by capacity ratios of the critical components are calculated to evaluate their sensitivity to these two important parameters.

The results show the significant effects of increasing speed on demand by capacity ratios of the critical components. Some components are more sensitive to this increase than others. The outcomes depict that, by applying restrictions on speed, internal forces may unexpectedly increase. This means that reducing speed may subject the bridges to danger more than before, especially by increasing the effect of fatigue in the long run. It is identified that, the resonance of responses can occur as a result of equality of natural vertical frequency of the simply supported bridge with the frequency of the live load at certain speeds.

The outcome of this research is very significant as it shows the strategies for applying speed restrictions may need to be revised. According to this study, to avoid resonance of the responses, applying speed restrictions should be based on the frequency of the moving load which depends on the speed of the train and the configuration of its axles as well as the natural vertical frequency of the simply supported bridge. Therefore, it is suggested to apply different speed limits based on the structural configuration of the railway bridge and train specifications including, train loads and axle spacings, and applying one speed limit to different types of trains will not be an appropriate strategy for decreasing the internal forces in critical components of the bridge. To evaluate the sensitivity of different components to changes of live load, demand by capacity calculations similar to that conducted in this research need to be conducted on each specific bridge and for each type of train.

Acknowledgements

The authors are grateful to the CRC for Rail Innovation (established and supported under the Australian Government's Cooperative Research Centres program) for the funding of this research. Project No. R3.118 and Project Title: Life Cycle Management of Railway Bridges.

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