Structural Engineering and Mechanics, Vol. 15, No. 1 (2003) 21-38
DOI: http://dx.doi.org/10.12989/sem.2003.15.1.021

A new bridge-vehicle system part II: Parametric study

Tommy H.T. Chan†

Department of Civil and Structural Engineering, The Hong Kong Polytechnic University, Hunghom, Kolwoon, Hong Kong

Ling Yu‡

Blasting and Vibration Research Department, Yangtze River Scientific Research Institute, 23 Huangpu Street, Wuhan, Hubei, 430010, P.R. China

T.H. Yung‡†

T.Y. Lin, Hong Kong, Consulting Engineers Ltd., Hong Kong

Jeffrey H.F. Chan‡‡

Wong and Ouyang (Civil & Structural Engineering) Ltd., Hong Kong

(Received May 22, 2000, Revised March 22, 2002, Accepted December 6, 2002)

Abstract. The formulation of a new bridge-vehicle system using shell with eccentric beam elements has been introduced in a companion paper (Part I). The new system takes into account of the contribution of the twisting and pitching modes of vehicles to the bridge responses. It can also be used to study the dynamic transverse load distribution of a bridge. This paper presents a parametric study on the impact induced by one vehicle or multi-vehicle running across a bridge using the proposed model. Several parameters were considered as variables including the mass ratio, the speed parameter, the frequency ratio and the axle spacing parameter to investigate their effects on the impact factor. A total number of 189 cases were carried out in this parametric study. Within the realistic range of vehicle considered, the maximum impact factors could be 2.24, 1.78 and 1.49 for bridges with spans 10 m, 20 m and 30 m respectively.

Key words: axle spacing parameter; bridge-vehicle interaction; dynamic transverse load distribution; frequency ratio; impact factor; mass ratio; speed parameter; transverse impact factor.

---

† Associate Professor
‡ Senior Engineer
‡† Assistant Structural Engineer
‡‡ Civil Engineer

1. Introduction

The dynamic response of a bridge structure when loaded by a moving load has been a topic of interest for over a century. In general, the increase in stress or deflection due to the dynamic nature of traffic loads is expressed by the term *Impact Fraction* I and the *Impact Factor* $I + 1$ or the *dynamic factor* that is defined as the ratio of total dynamic response to maximum static response (Chan and O'Connor 1990a). It was found that the impact factor could be much higher than the values given in the design codes. For example, previous studies on a small composite highway bridge reported the values of I to be –0.19 to 1.25, compared with the AASHTO code value of 0.30 for this bridge (Chan and O'Connor 1990b). Theoretical studies also suggest that high impact factors could occur occasionally (Wang and Huang 1992, Wang and Huang 1993). These all prompt the need to study the causes of high impact.

Previous literature indicated that impact factors of a bridge depend on many factors, e.g., bridge characteristics such as geometry, natural frequency, damping, and mass of the bridge; vehicle characteristics such as tire-suspension system, axle weights, natural frequency, and axle spacing of the vehicle; other factors include speed of the vehicle, and road surface roughness.

A number of analytical investigations have been carried out in the past to study dynamic behavior of bridge decks traversed by moving vehicles. Various degrees of modeling sophistication of bridge-vehicle systems have been used. These analyses indicate that the impact of a bridge depends on the characteristics of the bridge and the vehicle. It is convenient to put these factors in the form of dimensionless parameters as stated by Fleming and Romauldi (Fleming and Romauldi 1961). However, it is seldom found in the literature, especially for a three-dimensional modeling, that the parametric studies were thorough and practical, e.g., neglecting the effect of road roughness (Wang and Huang 1992, Wang and Huang 1993) or not varying the axle spacing or mass of the vehicle (Hutton and Cheung 1979). The most common vehicle model used in the literature is the model by Veletsos and Huang (1970). However, this vehicle model considers the pitching motion only, which is not suitable for 3-D modeling. Besides, some researchers modeled the bridge as a simple beam, e.g., Veletsos and Huang (1970), which is obviously inadequate and not suitable for bridges with a large width to length ratio. In addition, Wang and Huang (1992) and Wang and Huang (1993), who adopted a grillage analogy method in their studies, have shown that the impact on each girder is not the same. On the other hand, owing to the complexity nature of dynamic analysis in bridge-vehicle interaction, researchers seldom use vigorous method like a 3-D finite element method for their studies, particularly for the analysis of slab-on-girder bridges.

Chan and Chan (1999) demonstrated an efficient finite element method using quadrilateral flat shell elements with eccentric beam stiffeners to model slab-on-girder bridge structures. It is shown that this modeling saves much time and effort in the preparation of the input data computation and the analysis of the output data, when compared with other finite element approaches. Chan *et al*. (2002) introduced a new bridge-vehicle system in a companion paper. The proposed system has been validated by one theoretical study and by two field tests. It can be used to further study on the bridge dynamics. This paper aims to adopt this proposed system to study systematically the possible causes of high impact of slab-on-girder bridges under the passage of a vehicle or multi-vehicles based on an extensive parametric study.

2. Dimensionless parameters

As mentioned earlier, the impact values of bridges depend on many factors. A self-developed computer program BRVEAN which is an acronym derived from BRidge-VEhicle system ANalysis was used to study the effect of these factors on the impact values under the passage of a vehicle or multi-vehicles. As each factor may have a correlation to one another, in order to give a clear understanding of the effect of these factors more systematically, some dimensionless parameters were introduced. These parameters include the characteristics of the bridge and the vehicle. The virtual bridge models to be considered are within practical ranges of designed bridges. The vehicle characteristics are varied aiming to concentrate on realistic range of vehicle as well. The following dimensionless parameters will be adopted in the parametric study.

2.1 Mass ratio (MR)

Mass Ratio (*MR*) is defined as follows:

$$MR = \frac{M_{vt}}{M_b} \tag{1}$$

where M_{vt} and M_b are the total vehicle mass and the total bridge mass respectively.

This parameter was adopted many times in the past literature (Hutchinson and Al-Hussaini 1986). The realistic range of *MR* varies from almost zero to approximately one. The cases of *MR* equal to zero represent a massless force crossing the bridge.

2.2 Speed parameter (γ)

Speed parameter (γ) is defined as follows:

$$\gamma = \frac{V}{2Lf_b} \tag{2}$$

where V is the speed of the vehicle, L is the span length of the bridge and f_b is the fundamental frequency of the bridge.

γ can be found in the past literature, e.g., Chatterjee *et al*. (1994). It will be noted later that, the product Lf_b for all virtual bridge models is constant and equals to 100 in this study. Therefore, γ varies only with V and can be written as $V/200$. For the common range of highway speed 10 m/s (36 km/hr) to 40 m/s (144 km/hr), γ ranges from 0.05-0.2.

2.3 Frequency ratio (FR)

Frequency ratio (*FR*) is defined as Chatterjee *et al*. (1994):

$$FR = \frac{\text{minimum of } f_{v,i}}{f_b} \tag{3}$$

in which

$$f_{v,\,i} = \frac{1}{2\pi}\sqrt{\frac{k_t}{\frac{P_{st,\,i}}{g}}} \tag{4}$$

where $P_{st,\,i}$ is the static load under the i^{th} axle of vehicle and k_t is the tire spring stiffness as mentioned before. g is the gravitational constant (9.8 m/s^2).

For a vehicle with the center of mass at its center, such that $l_1 = l_2$ (l_1 is the ratio between the rear axle and the centroid and the axle spacing; l_2 is the ratio between the front axle and the centroid and the axle spacing) and $m_1 = m_2$ (m_1 is the total mass of the front axle; m_2 is the total mass of the rear axle), the fundamental frequency of the vehicle can be written as:

$$f_v = \frac{1}{2\pi}\sqrt{\frac{4k_t}{M_{vt}}} \tag{5}$$

It can be seen that the main advantage of using *FR* is that it includes the effect of the tire spring stiffness.

2.4 Axle spacing parameter (ASP)

The axle spacing parameter (*ASP*) is defined as:

$$ASP = \frac{S}{VT_b} \tag{6}$$

where S is the axle spacing of the vehicle and T_b is the fundamental period of the bridge.

Actually, the physical meaning of the parameter *ASP* is:

$$ASP = \frac{time\ taken\ for\ vehicle\ traveling\ one\ axle\ spacing}{fundamental\ period\ of\ dridge} = \frac{S/V}{T_b} \tag{7}$$

Besides, *ASP* can be related to the following factor, T_N:

$$T_N = \frac{time\ taken\ for\ the\ vehicle\ to\ cross\ the\ bridge}{fundamental\ period\ of\ of\ the\ bridge}$$

$$= \frac{(L+S)/V}{T_b} = \frac{L+S}{VT_b} = \frac{L}{VT_b} + ASP = \frac{1}{2\gamma} + ASP \tag{8}$$

where γ is the speed parameter.

3. Description of virtual bridge models

Three virtual bridge models of span lengths of 10 m, 20 m and 30 m are used for the parametric study. All bridges supported simply, have an overall width of 11.25 m and consist of five identical girders. The typical cross sections of the bridges are shown in Fig. 1 and the overall sectional properties of the bridges together with their corresponding characterizing parameters, α and θ (Bakht and Moses 1988), are tabulated in Table 1. The 10 m span bridge is assumed to be a concrete slab on five steel girders, and the other two spans are assumed to be concrete bridges. The

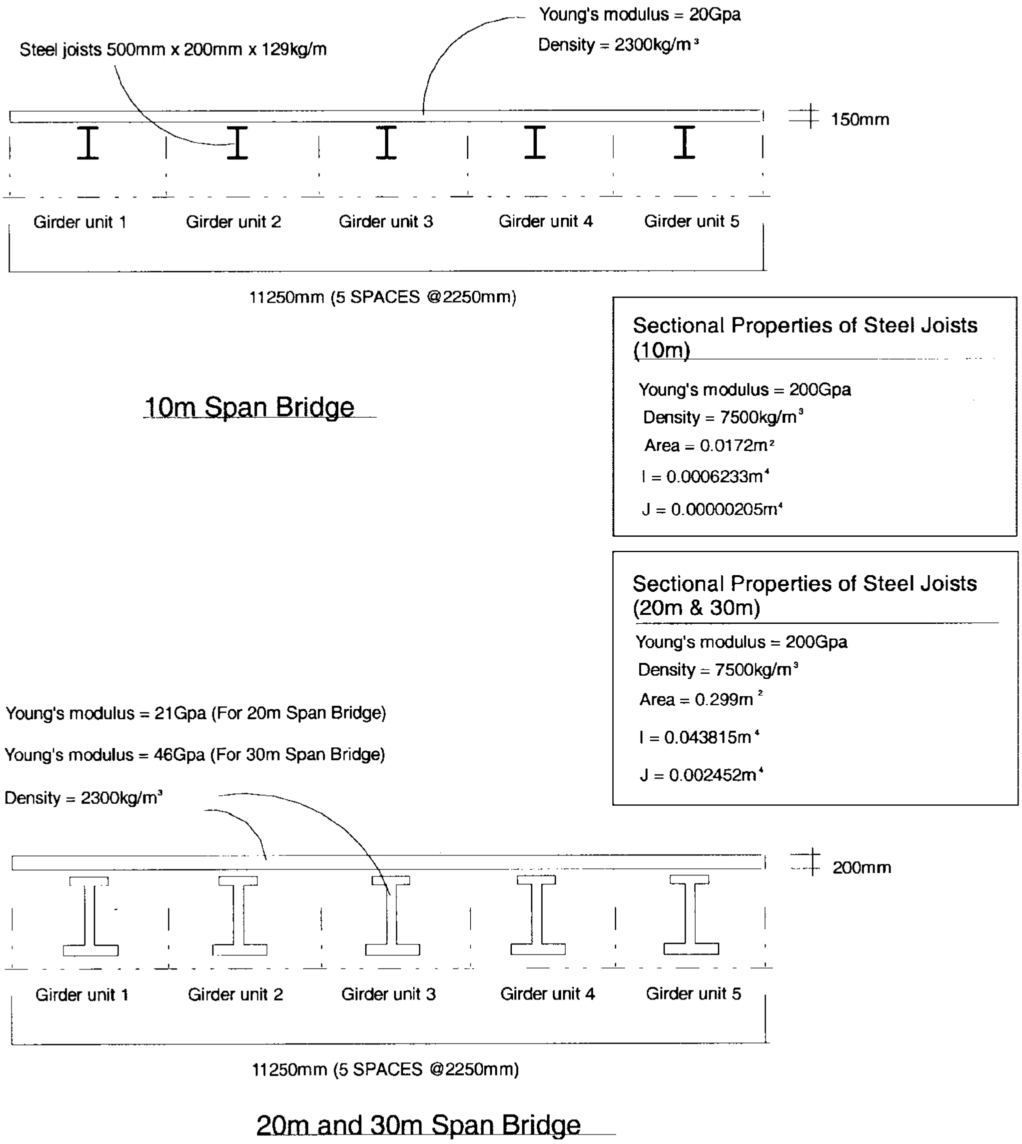


Fig. 1 Typical cross sections of virtual bridges

physical properties of the virtual bridge models are well compared to the bridges of real cases selected by Chan and O'Connor (1990a). It can be seen that the characterizing parameters of the virtual bridge model are within the practical ranges as stated by Bakht and Moses (1988).

Although the three virtual bridge models represent different types of bridges, the bridge models are adopted in such a way that the product Lf_b for them is constant and equals to 100, where L is the span length and f_b is the fundamental frequency of the bridge. Cantieni (1984) related the frequency of a bridge to its span by the formula $f_b = 95.4\ L^{-0.933}$ and Chan and O'Connor (1990a) suggested that $Lf_b = 120$ is a reasonable approximation for most bridges. Therefore $Lf_b = 100$ is an acceptable value for the virtual bridge models.

Table 1 Overall sectional properties of bridges

Span (m)	Mass per unit length (kg/m)	Young's modulus (GPa)	Second moment of area (m^4)	Fundamental frequency (Hz)	α	θ
10	4526.25	200	0.00945	10.15	0.1847	1.3150
20	8613.5	21	0.6882	5.09	0.1453	0.8705
30	8613.5	46	0.6882	3.35	0.1453	0.5803

Table 2 Masses of vehicles for various mass ratios and spans

	Masses of vehicles (kg)		
MR	10 m span	20 m span	30 m span
0.05	*	*	12920.25
0.10	*	17227	25840.5
0.15	*	25840.5	38760.75
0.20	*	34454	51681
0.25	11315.62	43067.5	64601.25
0.30	13578.75	51681	77521.5
0.35	15841.87	60294.5	90441.75

Note: * indicates the value belongs in the unrealistic range of vehicles.

4. Parametric study on general cases

It is important to have a good planning for the various parameters such that a clear understanding of their effects on the dynamic impact of a bridge can be demonstrated. The parameters for the study on general cases are the mass ratio (*MR*), frequency ratio (*FR*) and axle spacing parameter (*ASP*). In each case a single vehicle moves across the span on the central girder (transversely symmetrical) with a constant speed of 20 m/s (72 km/hr). As mentioned earlier, the values of the product Lf_b for each span are equal to 100 which implies speed parameter (γ) = 0.1 for all cases. *MR* varies from 0.05 to 0.35 by 0.05 increments so that the vehicle mass is determined by *MR*. The masses chosen for every *MR* are tabulated in Table 2.

It should be noted that, for the same mass ratio, the actual vehicle mass is different for various bridge spans as the masses of the three virtual bridge models are different. For comparison, the data of some standard trucks in past literature were adopted in this study. The mass of the standard truck used by Hutton and Cheung (1979) is 27250 kg and that by Hwang and Nowak (1991) is 18144 kg. The lower limit of the vehicle mass is set to 10000 kg. With this lower limit, the realistic ranges of mass ratio are respectively 0.25-0.35, 0.10-0.35 and 0.05-0.35 for bridges of 10 m, 20 m and 30 m spans.

Moreover, three values of frequency ratio (*FR* = 0.25, 0.50 and 0.75) are adopted for each mass ratio. In order to vary the frequency ratio, the frequency of vehicle vibrating upon its tire springs is changed by modifying the stiffness of tire springs. Table 3 shows the tire spring stiffness for various spans, mass ratios and frequency ratios. The suspension spring stiffness is constant and equals to 3939000 N/m and the coefficient of interleaf friction is equal to 0.15. These values have been adopted by Hutton and Cheung (1979) and Gupta (1980).

Table 3 Tire spring stiffness for various spans, mass ratios and frequency ratios

MR	Tire spring stiffness of vehicles (N/m)								
	10 m span			20 m span			30 m span		
	FR								
	0.25	0.50	0.75	0.25	0.50	0.75	0.25	0.50	0.75
0.05	*	*	*	*	*	*	88375	353498	795371
0.10	*	*	*	265662	1062647	2390958	176749	706996	1590741
0.15	*	*	*	398493	1593971	3586437	265124	1060494	2386112
0.20	*	*	*	531324	2125295	4781916	353498	1413992	3181482
0.25	695300	2781200	*	664155	2656619	5977395	441873	1767490	3796853
0.30	834360	3337439	*	796986	3187942	7172874	530247	2120988	4772224
0.35	973420	3893679	*	929817	3719266	8368353	618622	2474486	5567594

Note: * indicates the value belongs in the unrealistic range of vehicles.

Table 4 Axle distance of vehicles for various spans and *ASP*s

Axle distance of vehicles (m)			
Span (m)	*ASP* = 1.0	*ASP* = 1.5	*ASP* = 2.0
10	2	3	4
20	4	6	8
30	6	9	12

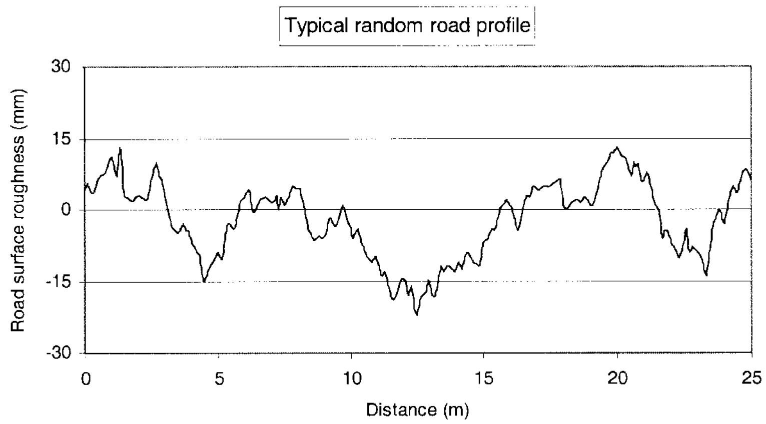


Fig. 2 Typical random road profile

Furthermore, three values of axle spacing parameter *ASP* = 1.0, 1.5 and 2.0 are adopted for the above mentioned cases. The axle distance of vehicles for each span and *ASP* are tabulated in Table 4.

As a result, there are totally $3 \times 7 \times 3 \times 3 = 189$ cases to be carried out for the general case parametric study, which include 135 realistic cases that will be main reported in this paper. In addition, the following assumptions have been made in the parametric study unless otherwise stated:

1) The same class of surface roughness for bridge deck is adopted for every case. Spectral shape

Table 5 Fundamental frequency and period of virtual bridge models

Span (m)	f_b (Hz)	T_b (sec)
10	9.98	0.1
20	5.00	0.2
30	3.33	0.3

index (n) = 2 and spectral roughness coefficient (A_r) = 0.64×10^{-6} m/cycle adopted by Hwang and Nowak (1991) are also assumed in this study for generation of road roughness. The values of n and A_r used in this study are the minimum values for highways. A typical random road profile is shown in Fig. 2. The random roughness field is assumed to be fully correlated over the width of the deck (Chatterjee *et al.* 1994).

2) All bridges have damping characteristics and 1% of critical damping is adopted for the first and second modes. This value of critical damping is also adopted by Cantieni (1984). The damping coefficients were determined by using an approach as described in the road surface roughness.
3) All initial conditions of vehicle and bridge for various degrees of freedom are assumed zero. The vehicle starts the motion when its front axle enters the bridge from the left end of the bridge and continues moving until the entire vehicle clears the right end of the bridge.
4) The time step (Δt) used in the direct integration for all studied is taken as $T_b/40$, where T_b is the fundamental period of the bridge as shown in Table 5. The Δt suggested by Bathe (1982) is $T_b/10$ which is larger than $T_b/40$ used in this study.

4.1 Effects of various parameters

The virtual bridge models considered in the parametric study are multi-girder (5 girders) slab-on-girder bridges. The impact characteristics of each individual girder unit ($I_{\mathrm{BI}} + 1$) as well as the total transverse section ($I_{\mathrm{BT}} + 1$) are considered. The effects of various parameters are described below.

4.1.1 Effect of span length

The results show that impact factors decrease as the span length increases provided that other parameters remain constant. The $I_{\mathrm{BT}} + 1$ for 10 m span bridge are much higher than those of the 20 m span bridge and 30 m span bridge especially for the case of high *FR* and low *MR*. The maximum calculated impact factors for each span are 2.42 (10 m), 2.02 (20 m) and 1.63 (30 m); and all are corresponding to the combination of the lowest *MR* and the highest *FR* and *ASP* = 2.0 in the study.

4.1.2 Effect of mass ratio (MR)

Previous theoretical study in the past literature (Wang and Huang 1992, Hwang and Nowak 1991) showed that the mass ratio was an important parameter influencing impact of bridges. The impact factor decreases as the weight increases. In this study, Figs. 3 to 5 show that the impact factor generally decreases as *MR* increases. However, the relation between impact factor and *MR* is related to *FR* and the span length. The higher *FR*, the more rapid the impact factor will decrease with increasing *MR*. For low *FR* of 0.25, the impact factor keeps almost constant with *MR*. In addition, the shorter the span length, the more rapid the impact factor will decrease with increasing *MR*.

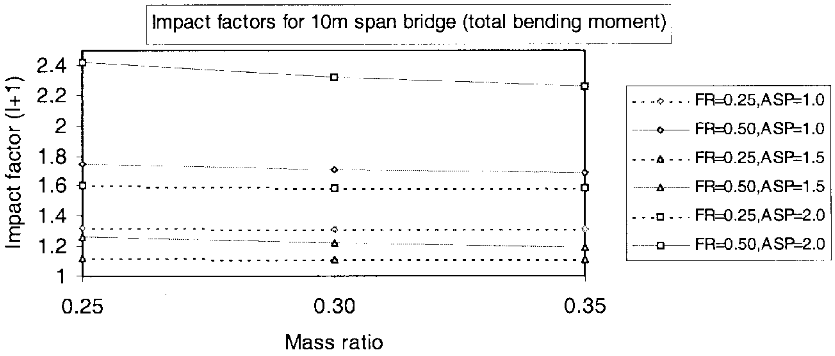


Fig. 3 Impact factors for 10 m span bridge (total bending moment)

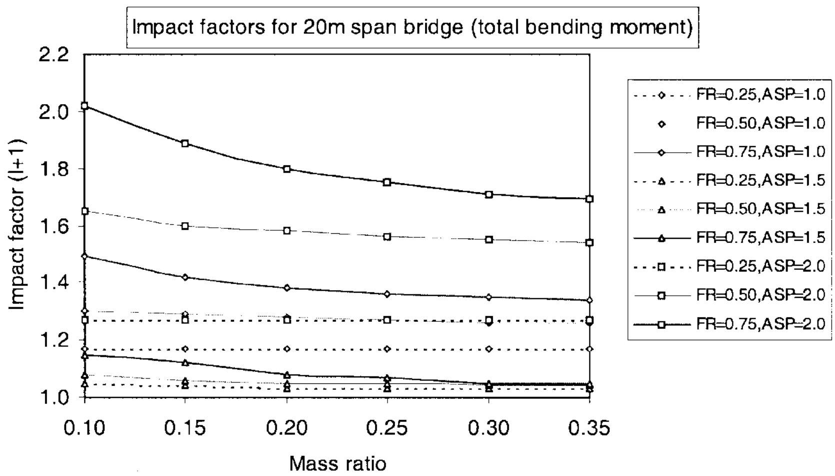


Fig. 4 Impact factors for 20 m span bridge (total bending moment)

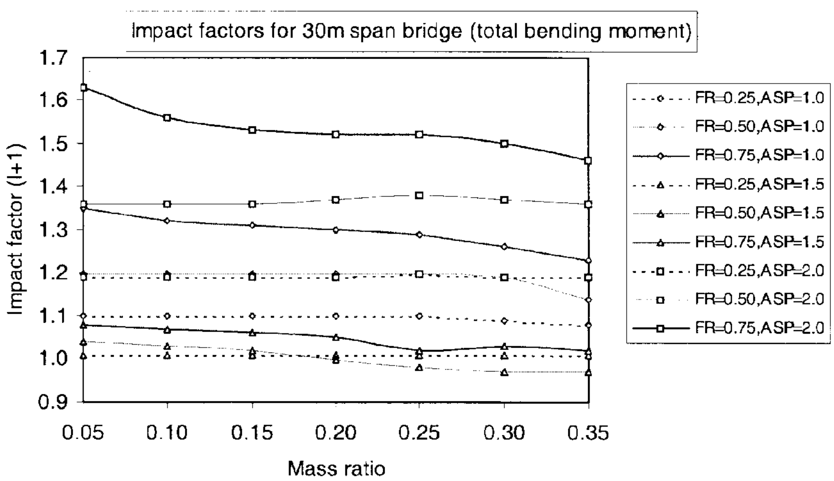


Fig. 5 Impact factors for 30 m span bridge (total bending moment)

4.1.3 Effect of frequency ratio (FR)

The impact factor increases with the *FR*. Figs. 3 to 5 show that the impact factors for higher *FR* are larger than that of the lower *FR*. For a instance, the maximum $I_{BT} + 1$ for $FR = 0.25$, 0.50 and 0.75 are 1.27, 1.65 and 2.02 for the 20 m span bridge respectively (with $ASP = 2.0$, $MR = 0.10$); and for the case $ASP = 1.5$, with the same span length and *MR*, the maximum $I_{BT} + 1$ for $FR = 0.25$, 0.50 and 0.75 are 1.05, 1.08 and 1.15 respectively. However, the difference of impact factors between those of the higher *FR* and lower *FR* will decrease with the increase of *MR* or the span length.

4.1.4 Effect of axle spacing parameter (ASP)

Figs. 3 to 5 show that *ASP* has significant effect on the impact factors of the bridges and give the following relationship: $I_{BT} + 1$ ($ASP = 2.0$) > $I_{BT} + 1$ ($ASP = 1.0$) > $I_{BT} + 1$ ($ASP = 1.5$). For example, for the 20 m span bridge, the maximum $I_{BT} + 1$ for $ASP = 2.0$, 1.0 and 1.5 are 2.02, 1.49 and 1.15 respectively (with $FR = 0.75$, $MR = 0.10$); and for the case $FR = 0.25$, with the same span length and *MR*, the maximum $I_{BT} + 1$ for $ASP = 2.0$, 1.0 and 1.5 are 1.27, 1.17 and 1.05 respectively.

4.2 Discussions on results for the study on general cases

4.2.1 Causes of high impact

It should be noted that $I_{BT} + 1$ presented in Figs. 3 to 5 just cover the realistic range of vehicle data in the general cases studies. The calculated results show that the causes of high impact is due to the combinations of low *MR*, high *FR*, high *ASP* and short span length. Although most of the calculated impact factors of the general cases are below the value of 2.00 for 20 m span bridge and 30 m span bridge. It can be concluded that bridges of short span are easier to cause high impact than bridges of long span.

4.2.2 Impact factors for individual girder unit

For the relationship between the impact factors of individual girder unit and the three parameters: *FR*, *ASP* and *MR* of each girder unit, it can be found that they follow a similar trend as that for the impact factors of the whole transverse section. However, it is interesting to observe that the impact factors for the exterior girder unit ($I_{B1} + 1$) and central girder unit ($I_{B3} + 1$) are quite different, especially for short span bridge (10 m). $I_{B1} + 1$ increase much faster than $I_{B3} + 1$ as *MR* decreases. Besides, it shows $I_{B1} + 1 > I_{B2} + 1 > I_{B3} + 1$. However, the difference between $I_{B1} + 1$, $I_{B2} + 1$ and $I_{B3} + 1$ will decrease when the span length increases. In addition, it should be noted $I_{B3} + 1 < I_{B2} + 1 < I_{B1} + 1$. The difference of impact factors between individual girder units shows that the lateral static and dynamic load distributions are quite different, especially for the short span bridge (10 m). On the other hand, if the lateral static and dynamic load distributions were equal, the impact factors of each individual girder should be equal. It should be noted that the above discussions are based on vehicle loading position on central girder unit (girder unit 3). In fact, both lateral static and dynamic load distributions would be changed if the lateral vehicle loading position were changed.

4.2.3 Impact factors for central girder unit

In this study, the impact factor of the central girder unit ($I_{B3} + 1$) is of primary concern. It is because the vehicles are moving in the central girder and it takes the largest proportion of dynamic load for all cases. The graphs of $I_{B3} + 1$ against *MR* for different span lengths are shown in Figs. 6 to 8. It can be seen that the maximum $I_{B3} + 1$ are 2.24, 1.78 and 1.49 for the 10 m, 20 m and 30 m

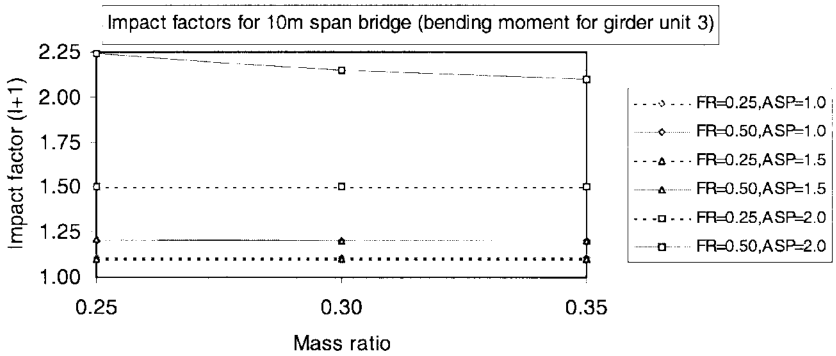


Fig. 6 Impact factors for 10 m span bridge (bending moment for girder unit 3)

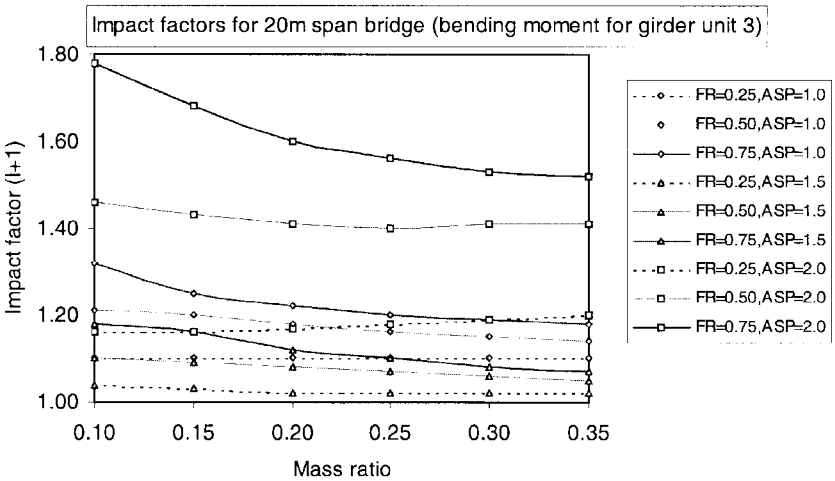


Fig. 7 Impact factors for 20 m span bridge (bending moment for girder unit 3)

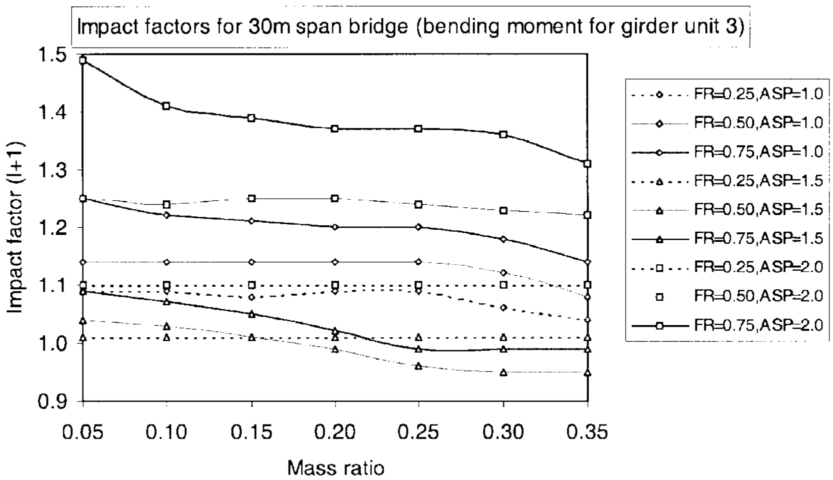


Fig. 8 Impact factors for 30 m span bridge (bending moment for girder unit 3)

Table 6 Effect of various dimensionless parameters and span length on $I + 1$

Dimensionless parameters and span length		$I + 1$
	FR = 0.75	↓ rapidly
MR ↑ (0.05 – 0.35)	*FR* = 0.50	↓ gently
	FR = 0.25	keep almost constant
FR ↑ (0.25 – 0.75)		↑ significantly
ASP (1.0, 1.5 and 2.0)		$I + 1_{(ASP=2.0)} > I + 1_{(ASP=1.0)} > I + 1_{(ASP=1.5)}$
Span length ↑ (10 m – 30 m)		↓ significantly

span of bridges respectively, and all are corresponding to the combination of the lowest *MR*, the highest *FR* and *ASP* for each span bridges in the study. In fact, for the 135 realistic cases of $I_{B3} + 1$ being considered, there are only 12 cases of them exceed 1.50. Half of them occur in two cases of *FR* for the 10 m span bridge but in one case of *FR* for the 20 m span bridge respectively. It can be concluded that most of high impact factors occurs in short span bridges.

4.2.4 Impact factors of deflection

The impact factors for deflection show a similar trend to that for bending moment. Generally, the impact factors of deflection are less than that of bending moment.

4.2.5 Summary for general cases studies

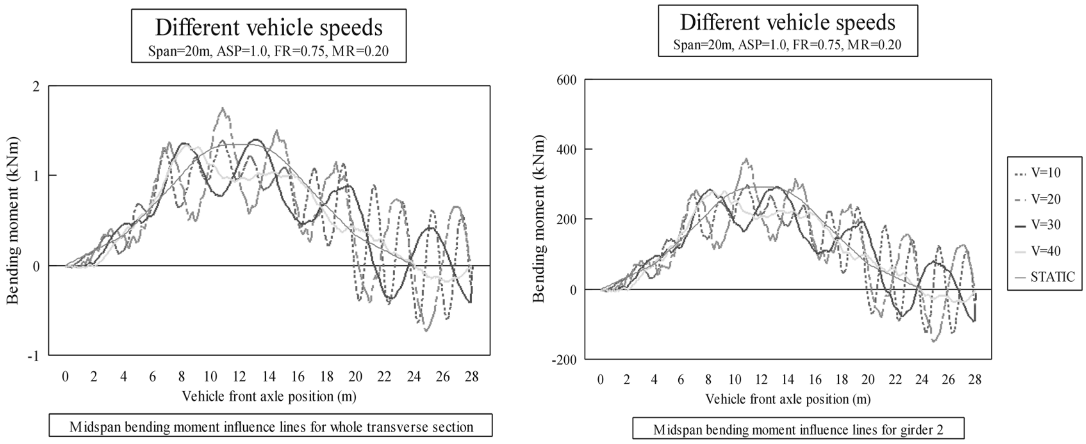
The effect of the various dimensionless parameters and span length on $I + 1$ for the general cases study are summarized below in Table 6.

5. Further parametric studies

In addition to the general cases described above, additional parametric studies were carried out for some special cases, e.g., cases with large impact factors, to study further the behavior of bridge-vehicle problem. Because there are few realistic cases for the 10 m span bridge, only 20 m and 30 m span bridges are chosen for this study. Similarly, a single vehicle moves across the span on the central girder (transversely symmetrical) in each case.

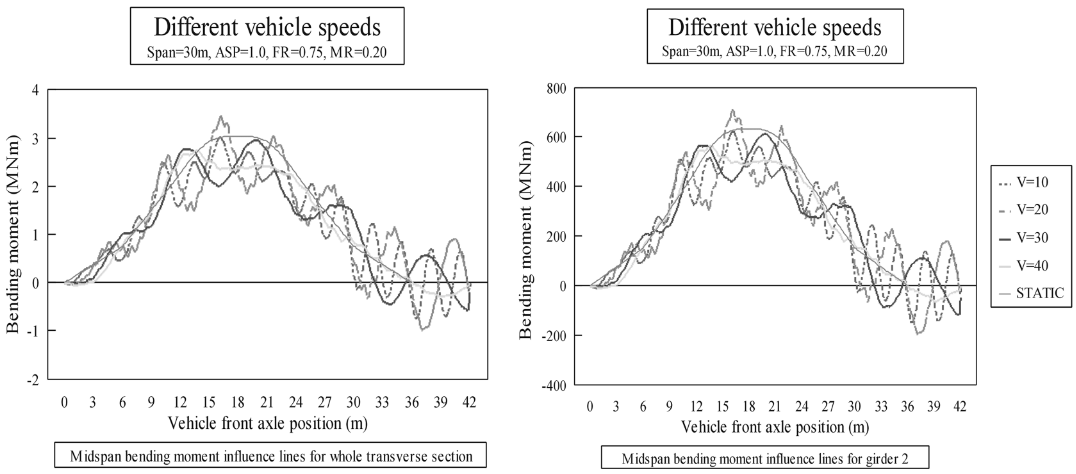
5.1 Vehicle speed

The influence of vehicle speed on the dynamic response of the bridge is investigated. The magnitude of vehicle speed varies from 10 m/s (36 km/hr) to 40 m/s (144 km/hr). The cases of *MR* = 0.20 and *FR* = 0.75 are chosen for the study. With this mass ratio (*MR* = 0.20), most vehicles considered here are within the practical range. Therefore, three different axle spacing are considered for the three chosen span lengths and a total of 36 different cases are considered in this study. Figs. 9 to 10 present some typical results of mid-span bending moment influence lines for various spans and vehicle speeds. Table 7 presents $I_{BT} + 1$ with various speed parameters. It can be seen from Table 7 that: (1) for a fixed axle distance, the impact factor varies with vehicle speed, but it



(a) Total bending moment at mid-span (b) Bending moment at girder 2

Fig. 9 Bending moments caused by a vehicle running across the 20 m span bridge with different speeds



(a) Total bending moment at mid-span (b) Bending moment at girder 2

Fig. 10 Bending moments caused by a vehicle running across the 30 m span bridge with different speeds

Table 7 Impact factors for different vehicle speeds

Span (m)	Axle Distance (m)	Impact factors for total bending moment			
		Vehicle speed (m/s)			
		10	20	30	40
20	4	1.34 (2.0)	1.85 (1.0)	1.35 (0.67)	1.29 (0.5)
20	6	1.97 (3.0)	1.67 (1.5)	2.16 (1.0)	1.95 (0.75)
20	8	2.06 (4.0)	2.38 (2.0)	1.75 (1.33)	2.24 (1.0)
30	6	1.30 (2.0)	1.68 (1.0)	1.28 (0.67)	1.18 (0.5)
30	9	1.77 (3.0)	1.61 (1.5)	1.89 (1.0)	1.67 (0.75)
30	12	1.71 (4.0)	2.04 (2.0)	1.54 (1.33)	1.79 (1.0)

Notes: Values inside the parentheses are the corresponding axle spacing parameter (*ASP*)

Table 8 Axle distance of vehicles for various spans and axle spacing parameters

Span (m)	Axle distance of vehicles (m)								
	ASP								
	0.5	0.75	1.0	1.25	1.5	1.75	2.0	2.25	2.5
20	2	3	4	5	6	7	8	9	10
30	3	4.5	6	7.5	9	10.5	12	13.5	15

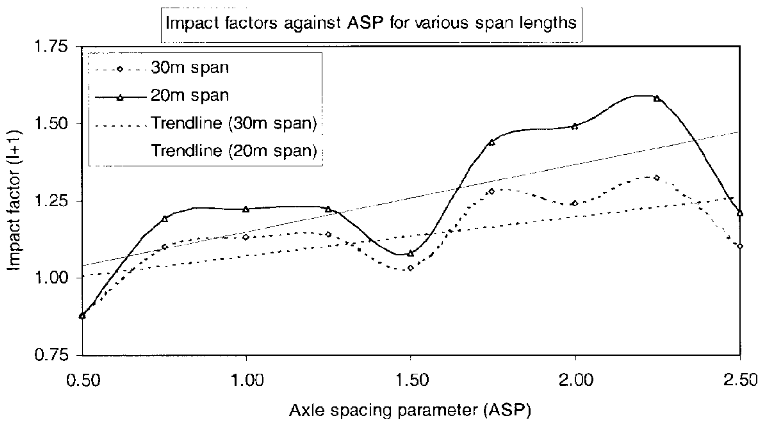


Fig. 11 Impact factor against *ASP* for various span lengths

does not show any obvious trend; (2) the impact factors for the short span bridge are higher than those of the long span bridge; (3) the impact factors for the cases of *ASP* = 0.5 or 1.5 give local minimum values; (4) when based on the same axle spacing parameter (*ASP* = 1.0), the impact factors increases with vehicle speed.

5.2 Axle spacing of vehicles

The *ASP*s of vehicles chosen for general cases in the parametric study are 1.0, 1.5 and 2.0. In order to investigate the effect of axle spacing more comprehensively, a total of nine values of *ASP* are adopted for further study for the case of *MR* = 0.20 and *FR* = 0.50 (with this value of *MR*, most vehicles considered here are within practical range). *ASP* ranges from 0.5 to 2.5 by 0.25 increments and the corresponding axle distance of vehicles for each span is tabulated in Table 8.

The I_{BT} + 1 of various *ASP*s for each span are shown in Fig. 11 as well as their trend lines. The results show that the impact factor generally increases as *ASP* increases but give a local minimum at *ASP* = 0.5, 1.5 and 2.5, and the variation of impact factor for the 20 m span bridges is higher than that of the 30 m span bridge.

5.3 Plank on bridge surface

Occasionally, there may be a plank on the bridge surface such that the passage of a vehicle will cause significant vibration on the bridge. The dynamic responses of bridges of various span lengths

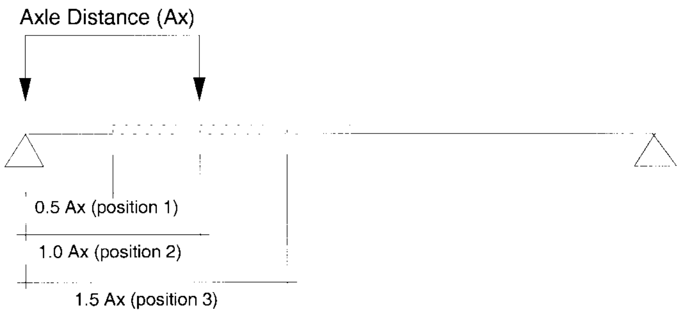


Fig. 12 Positions of plank on bridge surface

Table 9 Impact factors for total bending moment of plank at different positions

Span (m)	Axle spacing parameter	Impact factors for total bending moment (Plank position)		
		1	2	3
20	1.0	1.15	1.51	1.55
20	1.5	1.00	1.23	1.43
30	1.0	1.00	1.26	1.20
30	1.5	0.97	1.03	1.18

Table 10 Impact factors for total bending moment of plank at different positions

Span (m)	Axle spacing parameter	Impact factors for bending moment of plank at different position: girder unit 1			girder unit 2			girder unit 3		
		position of plank								
		1	2	3	1	2	3	1	2	3
20	1.0	1.30	1.72	1.73	1.12	1.48	1.52	1.05	1.38	1.40
20	1.5	1.19	1.39	1.61	1.02	1.23	1.46	1.02	1.19	1.38
30	1.0	1.01	1.30	1.24	1.00	1.24	1.19	0.99	1.21	1.16
30	1.5	0.96	1.01	1.20	0.96	1.02	1.18	0.90	0.99	1.09

with a plank at different positions of bridge surface subjected to the passage of a vehicle are studied. The cases with *MR* = 0.20 and *FR* = 0.50 *ASP* = 1.0 and 1.5 are chosen for this study. It was found that impact factors would not be affected much by varying the thickness of the plank from 30 mm to 50 mm. Therefore the thickness of the plank was chosen to be 40 mm. The positions of the 40 mm plank vary for 0.5, 1.0 and 1.5 times vehicle axle distance from bridge entry, and as shown in Fig. 12. Table 9 shows I_{BT} + 1 for different positions of plank on bridge surface. The results show that the positions of the plank have significant effect on the dynamic impact of bridges especially for the bridge of short span (20 m). It is seen that the I_{BT} + 1 can be as high as 1.55 for plank position at 1.5 times vehicle axle distance from the bridge entry. The impact factor increases as the plank position moves away from the bridge entry. It should also be pointed out that the farthest of plank dose not exceed the mid-span of bridge. In addition, the impact factors for *ASP* = 1.0 are higher than those of *ASP* = 1.5. Table 10 presents the impact factors for individual girder. The results show that the impact factors for exterior girders are much higher than the impact factors of center girder because the vehicle is moving in the central girder.

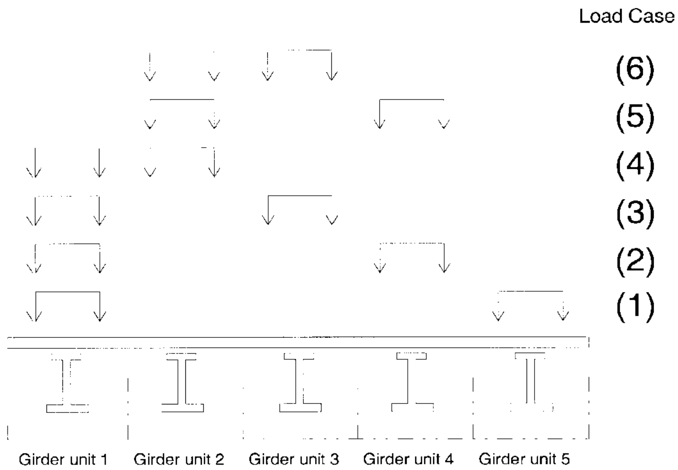


Fig. 13 Transverse positions of vehicles

Table 11 Impact factors for bending moment of 20 m span bridge for different positions of vehicles

Vehicle position	Impact factors for bending moment of 20 m span bridge					
	girder 1	girder 2	girder 3	girder 4	girder 5	total
1	1.18	1.34	1.47	1.34	1.15	1.28
2	1.13	1.25	1.34	1.34	1.47	1.28
3	1.06	1.15	1.25	1.51	2.09	1.26
4	0.97	1.05	1.28	1.86	3.36	1.22
5	1.33	1.26	1.25	1.26	1.33	1.28
6	1.21	1.16	1.19	1.39	1.73	1.27

Table 12 Impact factors for bending moment of 30 m span bridge for different positions of vehicles

Vehicle position	Impact factors for bending moment of 30 m span bridge					
	girder 1	girder 2	girder 3	girder 4	girder 5	total
1	1.05	1.10	1.13	1.10	1.05	1.09
2	1.03	1.08	1.11	1.13	1.17	1.10
3	0.99	1.03	1.08	1.16	1.29	1.07
4	0.93	0.97	1.04	1.18	1.48	1.03
5	1.11	1.09	1.08	1.09	1.11	1.10
6	1.06	1.04	1.06	1.11	1.20	1.08

5.4 Multi-presence of vehicles

In practice, it is often that more than one vehicle moves across a bridge in different lanes simultaneously. Hence, six combinations (symmetric and asymmetric) of two vehicles with different transverse positions are considered, as shown in Fig. 13. It is assumed that the vehicles are moving in the same direction and each vehicle has the same road roughness and under the conditions of 20 m/s (72 km/hr) vehicle speed, *FR* = 0.5, *ASP* = 1.0 and *MR* = 0.2. Tables 11 to 12 tabulate I_{BT} + 1, I_{B1} + 1, I_{B2} + 1 and I_{B3} + 1 for bridges with span 20 m and 30 m respectively.

The results show that (1) the impact factors of each girder change significantly with different vehicle transverse positions, especially for the short span bridge. However, values of I_{BT} + 1 do not change significantly with different vehicle transverse positions; (2) the impact factors of the girder beneath the wheel load (the loaded girder) are much smaller than those of other girders; (3) the variation of impact factors for the short span bridge (20 m) is much larger than those of the long span bridges (30 m); (4) the impact factors of the girders which are not under the wheel load for the cases of asymmetric vehicle positions (load case 4 of Fig. 13) are much higher than those of the cases of symmetric vehicle positions.

6. Conclusions

1) Generally, the impact factor decreases as the mass ratio increases but their relationship depends on the frequency ratio and span length. For a high frequency ratio (*FR* = 0.75), the impact factor will decrease rapidly with increasing the mass ratio. For a low frequency ratio (*FR* = 0.25), the impact factor keeps almost constant with the mass ratio.
2) The impact factor increases as the frequency ratio increases.
3) The impact factor decreases as the span length increases.
4) The causes of high impact are due to the combinations of a low mass ratio, a high frequency ratio and a large axle spacing parameter for each span case. For the realistic range of vehicles, the maximum impact factors I_{B3} + 1 are 2.24, 1.78 and 1.49 for the 10 m, 20 m and 30 m span bridges respectively. For the 135 realistic cases of I_{BT} + 1 being considered, there existed only 12 cases where the impact factors exceed 1.50. Six of them corresponding to the 10 m span bridge under the two cases of different frequency ratios. It can be concluded that most of high impact factor occurs in short span bridges.
5) The impact factors for the exterior girder unit and interior girder unit are quite different, especially for short bridge (10 m). For vehicle position on central girder, the impact factors for the exterior girder unit are larger than that of interior girder unit. The difference of impact factors between exterior and interior girder units will decrease with the increase of span length.
6) In general, the impact factors of deflection are less than the impact factors of bending moment.
7) For fixed axle distance, the impact factor varies with the vehicle speed, but it does not show any obvious trend. For the same axle spacing parameter (*ASP* = 1.0), the impact factors increases with the vehicle speed.
8) Generally, the impact factor increases as *ASP* increases but will give a local minimum at *ASP* = 0.5, 1.5 and 2.5. The variation of impact factor for the 20 m span bridges is higher

than that of the 10 m span bridge.

9) The position of the plank has significant effect on the dynamic impact of bridges. The impact factor increases as the plank position moves away from the entry provided that the position of plank does not exceed the mid-span of bridge.
10) The impact factors of each girder change significantly with different lateral vehicle positions. The impact factors of the girder under the wheel load are much smaller than that of other girders. The impact factors of the girders that are not under the wheel load for the cases of asymmetric vehicle positions are much higher than that of the cases of symmetric vehicle positions.

Acknowledgements

This project is funded by the Hong Kong Polytechnic University Research Grants and the Research Grants Council of the Hong Kong SAR Government.

References

Bakht, B. and Moses, F. (1988), “Lateral distribution factors for highway bridges”, *J. Struct. Eng.*, ASCE, **114**, 1785-1803.

Bathe, K.J. (1982), *Finite Element Procedures in Engineering Analysis*, Prentice-Hall Inc., Englewood Cliffs.

Cantieni, R. (1984), “Dynamic load testing of highway bridges”, *Proceedings IABSE*, P-75/84.

Chan, T.H.T. and O’Connor, C. (1990a), “Wheel loads from highway bridge strains : field studies”, *J. Struct. Eng.*, ASCE, **116**, 1751-1771.

Chan, T.H.T. and O’Connor, C. (1990b), “Vehicle model for highway bridge impact”, *J. Struct. Eng.*, ASCE, **116**, 1772-1793.

Chan, T.H.T. and Chan, J.H.F. (1999), “The use of eccentric beam elements in the analysis of slab-on-girder bridges”, *Struct. Eng. Mech., An Int. J.*, **8**(1), 85-102.

Chan, T.H.T., Yu, L., Yung, T.H. and Chan, J.H.F. (2003), “A new bridge-vehicle system part I : formation and validation”, *Struct. Eng. Mech., An Int. J.*, **15**(1), 1-19.

Chatterjee, P.K., Datta, T.K. and Surana, C.S. (1994), “Vibration of suspension bridges under vehicular movement”, *J. Struct. Eng.*, ASCE, **120**, 681-703.

Fleming, J.F. and Romauldi, J.P. (1961), “Dynamic response of highway bridges”, *J. Struct. Div.*, ASCE, **87**, 31-61.

Gupta, R.K. (1980), “Bridge dynamic loading due to road surface irregularities and braking of vehicle”, *Earthq. Eng. Struct. Dyn.*, **8**, 83-96.

Hutchinson, G. and Al-Hussaini, A. (1986), “Interaction of load speed and mass on the dynamic response of damped beams”, *IABSE*, 44-56.

Hutton, S.G. and Cheung, Y.K. (1979), “Dynamic response of single span highway bridges”, *Earthq. Eng. Struct. Dyn.*, **7**, 543-553.

Hwang, E. and Nowak, A.S. (1991), “Simulation of dynamic load for bridges”, *J. Struct. Eng.*, ASCE, **117**, 1413-1434.

Veletsos, A.S. and Huang, T. (1970), “Analysis of dynamic response of highway bridges”, *J. Eng. Mech. Div.*, ASCE, **96**, 593-620.

Wang, T.L. and Huang, D. (1992), “Dynamic response of multigirder bridges”, *J. Struct. Eng.*, ASCE, **118**, 2222-2238.

Wang, T.L. and Huang, D. (1993), “Impact studies of multigirder concrete bridges”, *J. Struct. Eng.*, ASCE, **119**, 2387-2402.