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Development of dynamic impact factor for performance evaluation of existing multi-girder concrete bridges

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ABSTRACT

The dynamic effect of moving vehicles on bridges is generally treated as a dynamic load allowance (or dynamic impact factor) in many design codes. Due to the road surface deterioration of existing bridges, studies have shown that the calculated impact factors from field measurements could be higher than the values specified in design codes that mainly target at new bridge designs. This paper develops a 3D vehicle–bridge coupled model to simulate the interaction between a bridge and vehicles and investigates the impact factor for multi-girder concrete bridges. The effects of bridge span length, vehicle speed, and road surface condition on the impact factor are examined. Chi-square tests are then performed on the impact factors and it is found that the impact factors obtained under the same road surface condition follow the Extreme-I type distribution. Finally, simple expressions for calculating the impact factors are suggested applicable to both new and existing bridges. Corresponding confidence levels with the proposed impact factors for the five studied bridges indicate that the proposed expressions can be used with considerable confidence. The proposed expressions for impact factor can be used as a modification of the AASHTO specifications when dealing with short bridges and old bridges with poor road surface condition for which the AASHTO specifications may underestimate the impact factor.

1. Introduction

The dynamic effect of moving vehicles on bridges is generally treated as a dynamic load allowance (or dynamic impact factor) in many design codes. For example, a value of 0.33 is suggested for the dynamic impact factor by the AASHTO LRFD specifications [1]. In AASHTO standard specifications [2], it is expressed as a function of the bridge length. In other codes, like Canada's Ontario Bridge Design Code [3] and Australia's NAASRA Code [4], the impact factor is defined as a function of the first flexural frequency of the bridge. A review of various impact factors for highway bridges implemented by various countries around the world can be found in GangaRao [5].

In the past two decades, significant efforts have been made to investigate the dynamic effect caused by dynamic vehicle loads using different analytical bridge–vehicle models [6–13]. Field testing has also been carried out to verify the impact factors specified in the design codes [11,14–16]. However, it has been demonstrated through both analytical studies and field testing that the design codes may underestimate the impact factor under poor road surface conditions [15–17].

One of the reasons for the underestimation of the impact factor could be that design codes, like the AASHTO specifications, are mainly providing guidelines for designing new bridges with good road surface condition. Therefore, the code-specified impact factors may not be a problem for bridges with good surface condition. However, for a large majority of old bridges whose road surface conditions have deteriorated due to factors like aging, corrosion, increased gross vehicle weight and so on, caution should be taken when using the code-specified impact factor. As a matter of fact, the average age of bridges in the United States has reached 43 years according to a recent AASHTO report [18]. Therefore, for safety purposes more appropriate impact factors should be provided for performance evaluation of these old bridges. Chang and Lee [8] proposed a function of impact factor for simple-span girder bridges with respect to bridge span length, vehicle traveling speed, and maximum magnitude of surface roughness; however, their study was based on simplified bridge and vehicle models, and more theoretical support was also needed for the proposed impact factor functions.

In this paper a 3D vehicle–bridge coupled model is used to analyze the impact factor for multi-girder bridges. The relationship between three parameters, which include the bridge span length, road surface condition, and vehicle speed, and the impact factor is examined by numerical simulations. Chi-square tests are then performed to examine the distribution of the impact factors under the same road surface condition for all the five road surface conditions considered. Based on the results from this study, reasonable

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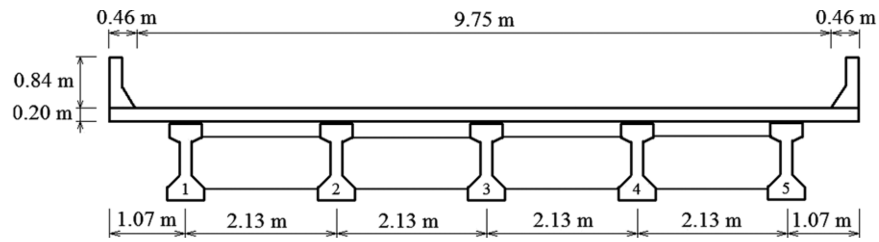

Fig. 1. Typical cross section of bridges.

Table 1
Detailed properties of the five bridges.

Bridge number	Span length (m)	Fundamental natural frequency (Hz)	Girder			Number of Intermediate Diaphragm
			AASHTO type	Cross-sectional area (m^2)	Inertia moment of cross section (10^{-2} m^4)	
1	9.14	15.51	II	0.238	2.122	0
2	16.76	6.58	II	0.238	2.122	1
3	24.38	4.60	III	0.361	5.219	1
4	32.00	3.20	IV	0.509	10.853	2
5	39.62	2.66	V	0.753	32.859	2

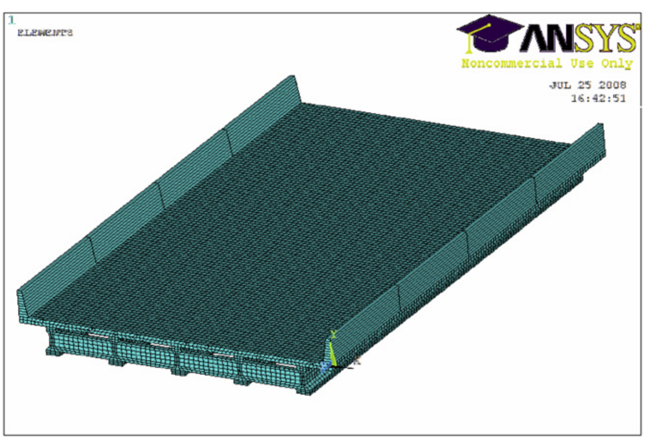

Fig. 2. A finite element model for Bridge 2.

expressions for calculating the impact factor are suggested applicable to both new and existing bridges. Corresponding confidence levels with the proposed impact factors for the five studied bridges are provided along with the determined distributions of the impact factors. The proposed expressions can be used as a modification of the AASHTO specifications when dealing with short bridges and old bridges with poor road surface condition for which the AASHTO specifications may underestimate the impact factor.

2. Analytical bridges

The bridges used in this study are good representatives of the majority of concrete slab-on-girder bridges in the United States. Five typical prestressed concrete girder bridges with a span length ranging from 9.14 m (30 ft) to 39.62 m (130 ft) were designed according to the AASHTO standard specifications [2]. All five bridges, consisting of five identical girders with a girder spacing of 2.13 m (7 ft), are simply supported and have a roadway width of 9.75 m (32 ft) and a bridge deck thickness of 0.20 m (8 in). A typical cross section of the bridges is shown in Fig. 1. Besides the end diaphragms, which are used for all five bridges, intermediate diaphragms are also used to connect the five girders depending on their span lengths as shown in Table 1.

In the present study, the concrete bridges were modeled with the ANSYS© program using solid elements (with three translational DOFs at each node). Fig. 2 shows the finite element model of Bridge 2. A summary of the detailed properties and the fundamental frequencies of the five bridges obtained from the finite element analysis are shown in Table 1.

Table 2
Major parameters of the vehicle under study (HS20).

Mass of truck body 1	2612 (kg)
Pitching moment of inertia of truck body 1	2022 (kg m^2)
Rolling moment of inertia of tuck body 1	8544 (kg m$^{2)}$
Mass of truck body 2	26 113 (kg)
Pitching moment of inertia of truck body 2	33153 (kg m^2)
Rolling moment of inertia of tuck body 2	181 216 (kg m^2)
Mass of the first axle suspension	490 (kg)
Upper spring stiffness of the first axle	242 604 (N/m)
Upper damper coefficient of the first axle	2190 (N s/m)
Lower spring stiffness of the first axle	875 082 (N/m)
Lower damper coefficient of the first axle	2000 (N s/m)
Mass of the second axle suspension	808 (kg)
Upper spring stiffness of the second axle	1 903 172 (N/m)
Upper damper coefficient of the second axle	7882 (N s/m)
Lower spring stiffness of the second axle	3 503 307 (N/m)
Lower damper coefficient of the second axle	2000 (N s/m)
Mass of the third axle suspension	653 (kg)
Upper spring stiffness of the third axle	1 969 034 (N/m)
Upper damper coefficient of the third axle	7182 (N s/m)
Lower spring stiffness of the third axle	3 507 429 (N/m)
Lower damper coefficient of the third axle	2000 (N s/m)
L1	1.698 (m)
L2	2.569 (m)
L3	1.984 (m)
L4	2.283 (m)
L5	2.215 (m)
L6	2.338 (m)
B	1.1 (m)

3. Analytical vehicle model

An AASHTO HS20-44 truck, which is a major design vehicle in the AASHTO bridge design specifications, was used for the vehicle loading for the five bridges. The analytical model for this truck is illustrated in Fig. 3, and the properties of the truck including the

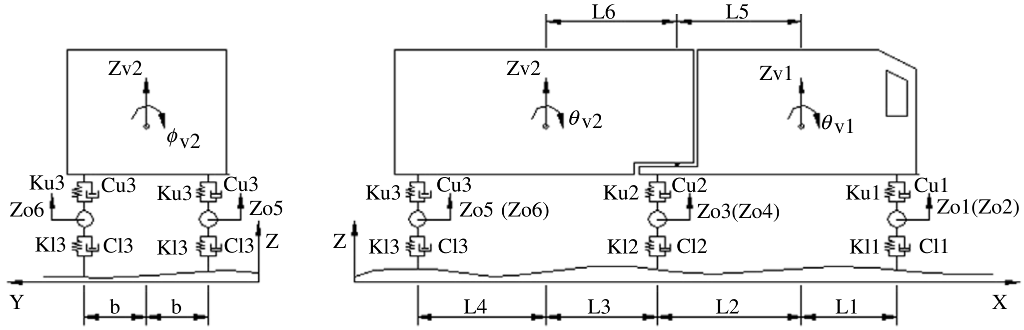

Fig. 3. Analytical model of the HS20-44 truck.

a b

Fig. 4. Deterioration of bridge road surface at (a) bridge deck [20] and (b) bridge joint [21].

geometry, mass distribution, damping, and stiffness of the tires and suspension systems are shown in Table 2 [16,19].

4. Vehicle–bridge coupled system

4.1. Equation of motion of the vehicle

The equation of motion for a vehicle can be expressed as follows:

$$[M_v]\{\ddot{d}_v\}+[C_v]\{\dot{d}_v\}+[K_v]\{d_v\}=\{F_G\}+\{F_v\} \tag{1}$$

where $[M_v]$, $[C_v]$, and $[K_v]$ = the mass, damping, and stiffness matrices of the vehicle, respectively; $\{d_v\}$ = the displacement vector of the vehicle; $\{F_G\}$ = gravity force vector of the vehicle; and $\{F_v\}$ = vector of the wheel–road contact forces acting on the vehicle.

4.2. Equation of motion of the bridge

The equation of motion for a bridge under vehicle loading can be written as follows:

$$[M_b]\{\ddot{d}_b\}+[C_b]\{\dot{d}_b\}+[K_b]\{d_b\}=\{F_b\} \tag{2}$$

where $[M_b]$, $[C_b]$, and $[K_b]$ = the mass, damping, and stiffness matrices of the bridge, respectively; $\{d_b\}$ = the displacement vector of the bridge; and $\{F_b\}$ = vector of the wheel–road contact forces acting on the bridge.

4.3. Road surface condition

Road surface condition is a very important factor that affects the dynamic responses of both the bridge and vehicles. Deterioration of bridge road surfaces can occur at both the bridge deck and joints due to factors like aging, varying environmental conditions, corrosion, increased gross vehicle weight, etc. Fig. 4 shows two examples of degraded bridge road surface.

A road surface profile is usually assumed to be a zero-mean stationary Gaussian random process and can be generated through an inverse Fourier transformation based on a power spectral density (PSD) function [22] such as:

$$r(X)=\sum_{k=1}^{N}\sqrt{2\varphi(n_k)\Delta n}\cos(2\pi n_k X+\theta_k) \tag{3}$$

where θ_k is the random phase angle uniformly distributed from 0 to 2π; $\varphi()$ is the PSD function (m^3/cycle) for the road surface elevation; and n_k is the wave number (cycle/m). In the present study, the following PSD function [23] was used:

$$\varphi(n)=\varphi(n_0)\left(\frac{n}{n_0}\right)^{-2}\quad(n_1<n<n_2) \tag{4}$$

where n is the spatial frequency (cycle/m); n_0 is the discontinuity frequency of $1/2\pi$ (cycle/m); $\varphi(n_0)$ is the roughness coefficient (m^3/cycle) whose value is chosen depending on the road condition; and n_1 and n_2 are the lower and upper cut-off frequencies, respectively.