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Determination of a reasonable impact factor for fatigue investigation of simple steel plate girder railway bridges

Hee-Hyun Lee [a,*], Jun-Chang Jeon [a], Kab-Soo Kyung [b]

[a] Construction Technology Consultant Co. Ltd., 1-40, Geumjeong-dong, Gunpo-si, Gyeonggi-do 435-050, Republic of Korea
[b] Department of Civil Engineering, Korea Maritime University, 1, Dongsam-dong, Youngdo-ku, Busan 606-791, Republic of Korea

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ABSTRACT

In Korea, there are approximately 1000 ballastless steel plate girder railway bridges that are short span and simply supported and have been in service for over 30 years. It is well known that one of the main factors affecting the durability of such bridges is the fatigue phenomenon. This study aimed to determine a reasonable impact factor for fatigue investigation of these bridges. The impact factor was obtained from a dynamic analysis modeling a bridge and a train as beam and moving masses, respectively. Train speed and the type, span length and stiffness of the bridge were considered as the main variables in the analysis. The validity of the presented impact factor was confirmed through comparison with measured values. The results of this work show that the present Korean code has a tendency to overestimate the fatigue impact factor, and a value only half of that used in ordinary static design can be used for fatigue investigation of simple steel plate girder railway bridges.

1. Introduction

In Korea, there are approximately 2600 railway bridges in service, and 1000 of these are ballastless, short span (a span length less than 26 m), and simply supported steel plate girder bridges and have been used for over 30 years [1]. Such short span bridges are known to be more vulnerable to fatigue compared to long-span bridges [2]. Therefore, it can be readily understood that the fatigue phenomenon, which is caused by accumulation of live load stress over a long period, is one of main factors controlling the life and increasing maintenance costs of such bridges.

Fatigue investigation of existing railway bridges is, in general, performed at the suspicious joints using the stress histories calculated by the actual trains crossing the bridges. The impact factor should thus be considered in these calculations. However, noting that the impact factor for general static design may develop an overly-safe design in terms of fatigue, as the fatigue phenomenon is caused by the accumulation of live load stress for a long time under normal conditions, rather than in extreme conditions, it is commonly recommended to reduce this factor for fatigue investigation [3–6,8]. However, the reduction ratio differs nationally. For example, the Japanese Fatigue Design Guide for Steel Highway Bridges [3] and the AASHTO LRFD [4] for the fatigue limit state recommend 50% of the value used in normal static design. The Korean Railway Bridge Design Code [5] and AREA [6] specify the fatigue factor as 65% and 100% of the static values for bridges longer and shorter than 9 m, respectively, and the Japanese Railway Standards for Steel Bridges [7] specify 75%, regardless of span length.

The factors differ by country and bridge function. The authors have performed similar studies [8,9] on highway bridges and have suggested a value of 50% of the static factor if the roadway surface is properly maintained. However, as far as we know, research related to the fatigue impact factor has not been performed until now in the field of the railway bridges, although there are many papers reporting their dynamic behavior and the distribution of impact magnitudes [10–18].

This study investigated the determination of a reasonable impact factor for the fatigue investigation of existing short span and simply supported ballastless steel plate girder railway bridges. The impact factor for fatigue investigation was newly defined using the maximum dynamic stress range rather than the maximum dynamic stress amplitude in this paper and then obtained from a dynamic analysis, modeling the bridge and axle loads of the train as beam and moving masses, respectively. Train speed and the type, length and stiffness of the bridge were considered as the main variables in the analysis. The validity of the presented impact factor was confirmed by comparison with measured values.

2. Definition of impact factor for fatigue investigation

In general, a running train develops a larger response in the bridge members than that due to a normal static load owing to vibration and the interaction of bridge and train. The magnitude

* Corresponding author. Tel.: +82 10 5240 0966; fax: +82 31 477 5185.
E-mail address: lhh@ctceng.co.kr (H.-H. Lee).

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of the response is influenced by various factors such as running speed and the characteristics of the bridge and the train. Such an effect is considered in the bridge design codes as an impact factor. The Korean Railway Bridge Design Code [5] specifies the design impact factor to be used in the ordinary static design of steel and composite bridges as Eq. (1). However, for fatigue design, the regulation specifies the use of only 65% of this value for a bridge longer than 9 m. Such an impact factor is also used in the fatigue investigation of existing railway bridges.

$$i = 50 - \frac{L^2}{48} \quad \text{for} \quad L \leqslant 24\ m \tag{1a}$$

$$i = \frac{180}{L-9} + 26 \quad \text{for} \quad L > 24\ m \tag{1b}$$

where L is the span length (m) of the bridge.

If a dynamic bending stress time history at the center of a simple span bridge due to a single-axle moving load is obtained as the solid line in Fig. 1, then the impact factor is normally calculated using Eq. (2). The static moment is given as the dotted line. However, in fatigue design, we are interested in the maximum stress range rather than the maximum stress. Therefore, in this work, we defined the impact factor for fatigue investigation as Eq. (3). Furthermore, Eq. (4) was used to derive a representative average value for statistical analysis because fatigue damage occurs by accumulation of live load stress for a long time, as mentioned above. If we assume that the impact factors follow the normal distribution, the probability that the exact unknown mean is less than or equal to $i_{95\%}$ is 95% [19]. Byers [13] indicated from the test data on 37 spans that the distribution of impact magnitudes on railway girder bridges can be approximated by a normal distribution if span length is constant.

$$i_{static\ \ design} = \frac{f_{dyn}}{f_{stat}} - 1 \tag{2}$$

$$i_{fatigue\ \ design} = \frac{f_{dyn,r}}{f_{stat}} - 1 \tag{3}$$

in which: f_{dyn}, maximum dynamic stress; f_{stat}, maximum static stress; $f_{dyn,r}$, maximum dynamic stress range

$$i_{95\%} = \mu + 1.96\frac{\sigma}{\sqrt{N}}. \tag{4}$$

where μ, sample mean; σ, standard deviation; N, number of samples.

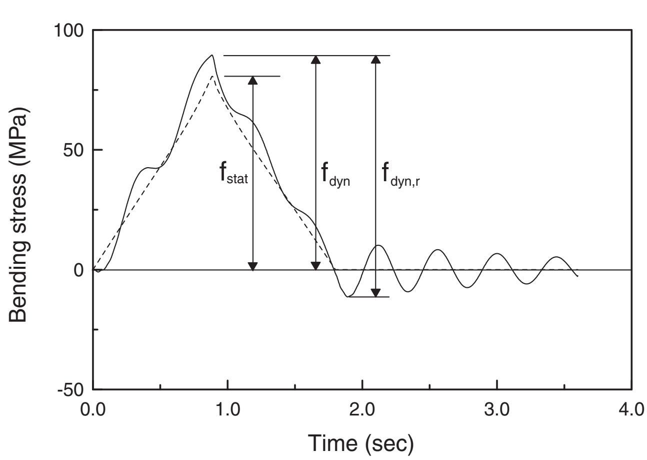

Fig. 1. Example of bending stress time history at center of simple span bridge.

3. Dynamic analysis

3.1. Bridge properties

The cross-section of a typical bridge in this study is shown in Fig. 2. The bridges are ballastless, single span, and relatively lightly weighted, as shown in Table 1. The detailed dimensions of the bridges are given in the 'Standard Design Documents' published by the Korea National Railroad [20].

3.2. Analysis variables

Because the dynamic behavior of the railway bridge is influenced by various factors, such as speed, the type and model of train, and the length and stiffness of the bridge, the variables described in Table 2 were considered in the parametric dynamic analysis. One conventional passenger train (Saemaeul), one freight train and one high-speed train (KTX) were considered, and their weights and axle spacings are given in Fig. 3.

Two train-models were used, namely, the multi-axle-moving-force and multi-axle-moving-mass models. The axle-loads are constant in the force model but are not in the mass model due to the inertia forces caused by bridge vibrations, as shown in Fig. 4 [10–12,21–24]. The force model was used to investigate the dynamic behavior of the Korea High Speed Railway Bridges [17,18]. However, as the bridge weight is small compared to the axle-loads applied to the bridge, it was assumed that the effects of the inertia forces could not be neglected. Therefore, the results obtained using two different models were compared, and the appropriate train model was suggested. The wheel-suspension system of train and track irregularities can also affect the dynamic behavior of the railway bridges [15]. However, they were not included in the analysis model because the aim of this study was not to find a complicated train–structure interactive model for the derivation of an extreme impact factor but rather to obtain representative values for fatigue investigation.

Train speed was taken into account at speeds up to 80 km/h for the freight train, and 150 km/h for the Saemaeul and KTX trains, which are their usual operating speeds. Speed increments of 2 km/h were applied to determine whether there exists a critical speed due to the successive passage of axle-loads. A resonance phenomenon may occur when the passing frequency of axles of the trains given in Fig. 3 is approximately equal to a modal frequency of the bridge. Such speed is, in this paper, called as the critical speed, and can be calculated according to Eq. (6) [17] given in Section 4.

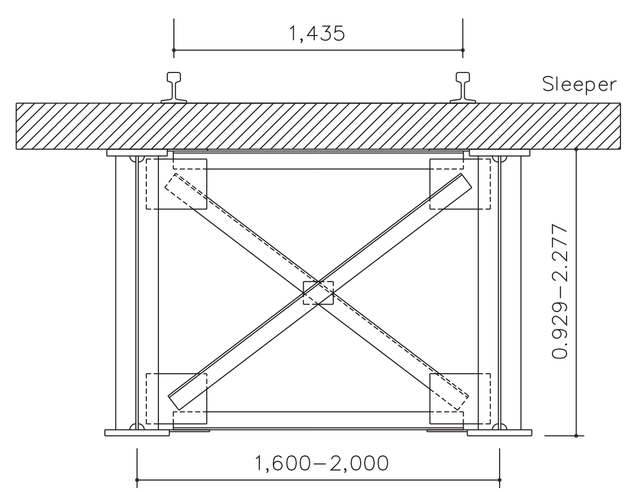

Fig. 2. Typical cross-section of a study bridge (unit: mm).