Section 1 **End-Cross Beam Reinforcement on Reducing Traffic-Induced Vibration of Steel Two-Girder Bridge**

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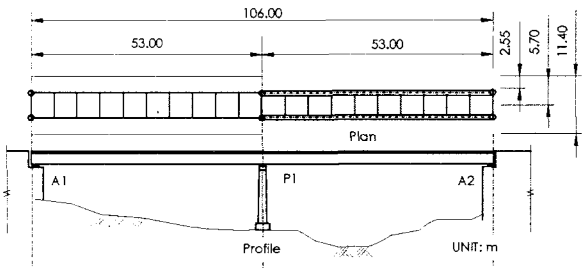
1. Purpose: The wider girder spacing and more simplified structural system of two-girder bridges than conventional multi-girder bridges make bridges to be easily vibrated due to external dynamic loads like wind, vehicle loads, etc. Thus, in order to enhance the vibration serviceability of steel two-girder bridges, the end-cross beam reinforcement[1,2)] is applied as a countermeasure against traffic-induced vibration, and analytical investigations are carried out in this study. The effect of removing bumps at expansion joints, so called a layover bump, on reducing the traffic-induced vibration is also investigated.

2. Analytical Procedures: The finite element (FE) method and modal analysis are adopted for modeling and analyzing a bridge. Lagrange equation of motion is adopted to develop governing dynamic differential equations for the bridge-vehicle interaction system. Newmark-β method is applied to solve the derived system governing equations of motion as a direct integration method. Vertical accelerations are estimated by superposing up to 120th modes.

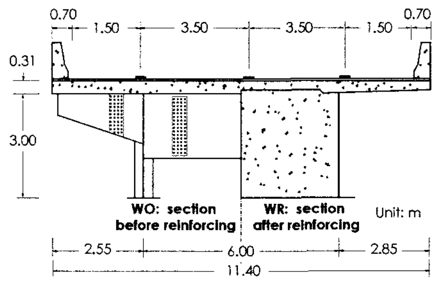
3. Analytical Models: **3.1 Bridge** A two-span continuous steel two-girder bridge in service (**Fig. 1a)**) with total span length of 106m (53m+53m), 6.0m girder spacing and 11.4m width is adopted for an analytical example. The damping constant of the bridge is assumed to have 0.7 % for the first and second modes based on field test results. Typical cross section and reinforced section of the bridge are shown in **Fig. 1b)**. The deck slab is made of a prestressed concrete of 31cm thick, and is assumed to act compositely with main girders. The symbols WO and WR in **Fig. 1b)** indicate the sections in accordance with existence of reinforcement at the end-cross beams and the intermediate-cross beam on the pier P1; WO is the section before reinforcing; WR, the section after reinforcing with thickness of 50cm. The reinforced concrete blocks at the cross beams located at abutments A1 and A2 and the pier P1 are assumed to completely link with deck slabs. A FE model of the bridge for dynamic response analysis appeared in **Fig. 2** consists of 231 nodes, 192 flat elements and 159 (163 for the bridge model with WR section) beam elements.

3.2 Vehicle A rear-tandem dump truck idealized as 8DOF model with gross weight of 196kN is adopted as a heavy vehicle running on the bridge. Details of the vehicle model are in **Table 1**.

3.3 Roadway roughness Roadway surface profiles used in the dynamic response analysis are obtained by Monte-Carlo simulation method based on power spectral density (PSD) functions to fit the measured PSD of Meishin expressway in Japan. In dynamic response analysis, measured bumps at expansion joint on A1 abutment under the vehicle path are considered; 16mm with 780mm width for left wheel path; 14mm with



a) General layout of steel two-girder bridge



b) Cross section before and after reinforcing

Fig. 1 Cross section and general layout of two-girder bridge

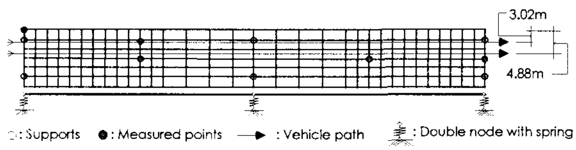


Fig. 2 FE model of two-girder bridge

Table 1 Details of vehicle model

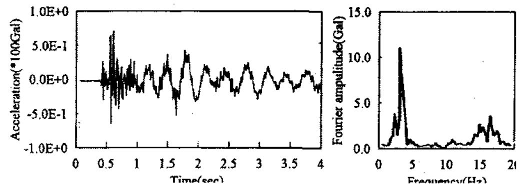
Total weight (KN)	Axle weight (KN)		Tread (m)	Axle distance(m)		Natural frequency(Hz)	
	Front	Rear		Front-rear	Tandem	Front axle	Rear axle
196.03	42.95	153.08	1.86	3.86	1.30	2.20	3.40

780mm width for right wheel path.

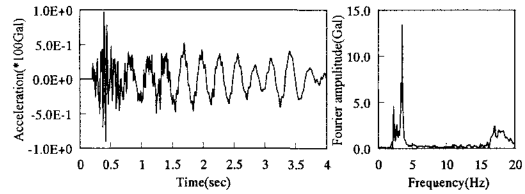
4. Analytical Results: 4.1 Natural frequencies To verify the validity of the analytical results, natural frequencies and acceleration responses taken from field test are compared with analytical ones. The natural frequencies of field test for the first and second bending modes are 2.26-2.30Hz and 3.38-3.42Hz, respectively. Those analytical frequencies are 2.26Hz and 3.37Hz for 1st and 2nd bending modes, respectively. For 1st torsional mode, the experimental and analytical results are 3.38-3.42Hz and 3.37Hz, respectively. It indicates the validity of the bridge modeling for dynamic response analysis.

4.2 Acceleration response Acceleration responses and those Fourier spectra of the deck slab at the center of the 1st span (between A1 and P1) taken from experiment and analysis are shown in **Fig. 3**. It can be observed that the wave profile and Fourier spectrum of experimental result are simulated well by the analysis. However amplitudes of acceleration responses from analysis are greater than those of experimental ones because of the difference of roadway profiles used in analysis and field test. It is possible to see from **Fig. 4** that the reduction on amplitude of acceleration responses of the slab center at A1 abutment can be obtained by end-cross beam reinforcement.

4.3 Vibration level To assess the reduction effect quantitatively, the vibration level (VL) is considered as a measure. Results of 1/3 octave band spectral analysis of the deck at A1 abutment and the deck located 2.65m away from A1 abutment with respect to the reinforcement and removing bumps are shown in **Fig. 5**. The symbol B indicates consideration of measured bumps in dynamic response analysis, on the other hand NB means the result without considering the bumps. The reducing effect for the deck at the end-cross beam (see, **Fig. 5a)**) can be expected covering most of the considered frequency range by end-cross beam reinforcement as well as removing bumps. On the other hand, for the deck located apart from the A1 abutment **(Fig 5b))**, the reinforcing end-cross beam can only suppress the vibration level in high frequencies, while removing the bumps reducing the vibration covering the entire frequency range in considering. In addition, the over all pass vibration level with respect to each node of the bridge model is summarized in **Fig. 6**. It can be observed that the effect of end-cross beam reinforcement is restricted for members near the end-cross beam. Thus the end-cross beam reinforcement in combination of removing bumps can give effective reduction against traffic-induced vibrations.

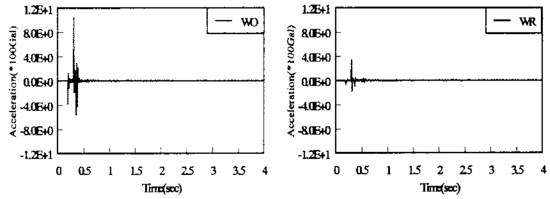


a) Experimental results



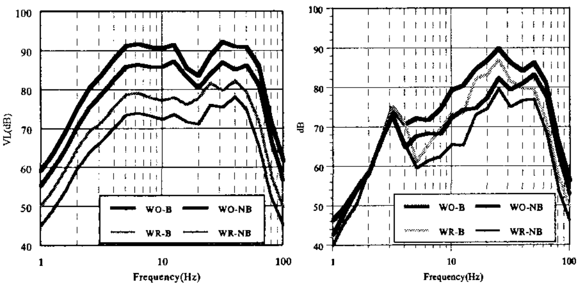
b) Analytical results

Fig. 3 Accelerations; slab center at the center of 1st span



a) before reinforcing b) after reinforcing

Fig. 4 Acceleration responses of slab center at A1 abutment



a) Slab center at A1 joint b) Slab 2.65m apart from A1 joint

Fig. 5 Typical 1/3 octave band spectra

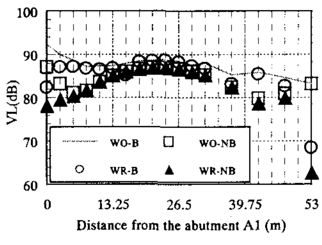


Fig. 6 All pass Vibration Level w.r.t. distance from A1 joint

[References]

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