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Field tests for estimating traffic induced vibration of an orthotropic deck steel bridge

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ABSTRACT: Controlling of traffic induced vibrations, is one of the most important issues in maintenance and rehabilitation of existing bridges. Lowering traffic induced vibrations in seven multi span flyover orthotropic deck steel bridges, which were constructed in 1970s as temporary solution for growing Tehran traffic and are still at service, is a main part of retrofit project introduced by the Tehran Municipality. There are a number of parameters which can affect the traffic induced vibrations of bridges, i.e. bridge geometry, speed, weight and suspension system properties of passing vehicles, surface roughness and specifications of the bearings. Modelling of all effective parameters is too complicated to be carried out; therefore, identification of dynamic behaviour through numerical modelling may associated with some errors. Field tests under specified dynamic loads are reliable means for evaluation of dynamic characteristics of bridges and help to verify numerical models. In this paper, results of field tests on a typical 24-meter simple span of an orthotropic deck bridge are presented. Verified models later are used to find an optimal method for control of traffic induced vibrations.

1 INTRODUCTION

Vibration control is a matter of importance in a wide range of applications from small structures like a building floor to large scale complex ones such as high rise buildings and bridges. All the structures are prone to different types of disturbances and excitations; for example in bridges wind, traffic and earthquake excitations may vibrate the structure. Traffic-induced vibrations as the main subject of this paper have two important effects on bridge structures. The first, stresses are increased compared with static (non-vibrating) load applications and this is normally accounted for by the “impact factor” in bridge codes. Impact factors provide acceptable safety margins practically, in order that dynamic increase of stresses cannot endanger bridge structural stability. The second effect is annoying oscillations which may be noticeable for pedestrians and vehicle passengers passing the bridge. This issue commonly does not affect the structural safety however may have the psychological effects of impairing public confidence for the structure. The later effect cause structural engineers and bridge owners to have a great interest in lowering traffic-induced vibrations. Theoretically increment of mass, stiffness and damping of structures may decrease vibrations. Nevertheless additional mass may cause other structural deficiencies and arouse economic issues. Therefore usually increasing of structural stiffness (i.e. application of cross-frames or diaphragms) and/or enhancing structural damping (i.e. application of Tuned Mass Dampers (TMDs)) are utilized to suppress traffic-induced vibration in bridges.

Cross-frames or diaphragms are one the most important components of bridges structure. Traditionally cross-frames have three main functions: transfering lateral wind or inertia loads from the deck to the bearings at end supports, distribution of gravitional dead weights and live loads between the longitudinal girders and providing a stable bracing point for girder compresive flanges during erection and placement of the deck in composite deck bridges. In this paper role and capability of cross-frames in decreasing traffic-induced vibrations are evaluated. In most of bridges, these elements have been considered in initial design, but in a few old bridges like Tehran orthotropic steel deck bridges and some new ones aiming more practical construction, cross-frames or trusses between girders as diaphragms have not been contrived. The effects of cross-frames on the seismic performance bridges have been investigated by many researchers (e.g. Azizinamini (1996)). However limited researches have examined the influence of cross-frames on bridge vibration response under passing traffic loads. Yoon and Kang (1998) investigated cross-frame effects on free vibration response of horizontally curved I-girder bridges with varying radii of curvatures, cross-sections and number of cross-frames. Keller (1994:97) investigated the dynamic response of a system of horizontally curved steel I-girders for non-composite dead load and composite live load conditions. The effects of span length, girder depth, number of girders, flange width, degree of curvature and cross-frame spacing were studied. Kim et al. (2004) investigated on the effect of reinforced diaphragms at bridge piers to reduce traffic induced vibrations. They indicated that their special proposed reinforced diaphragm can enhance the deck transverse integrity near the end expansion joints at supports and suppress traffic-induced vibrations.

One of the other methods for reduction of structural vibrations could be addition of an energy dissipative system to the existing structure to control the dynamic response. Application of Tuned Mass Dampers (TMDs), as a secondary added dynamic system which is connected to the primary structure at suitable points, is a classical passive solution to dissipate vibration energy of the main structure. TMDs are installed to absorb the energy transmitted from the primary structure by tuning its natural frequency to the structural frequency without additional power. The modern concept of TMDs for structural applications is originated from dynamic vibration absorbers which were primarily studied by Fraham (1909). A typical TMD generally consists of a mass, a spring and a dashpot. Den Hartog (1956) first investigated the optimal values of TMD parameters using a two-degree of freedom model. TMDs have been extensively studied and applied to suppress vibrations of high rise buildings and bridges. It is well known that the TMD can be effective in suppressing the single-mode resonant vibration when its frequency is tuned to the modal frequency of the structure (Igusa and Xu, 1991). Most of the previous research efforts were focused on developing the design procedure and optimizing the TMD parameters for various dynamic loads applications. Chen and Cai (2004) have studied on TMDs for suppressing vibrations of structures under wind and seismic loads. TMD’s success in reducing wind-excited structural responses is now well established. However, because of the complex characteristics of earthquake excitations, there still has not been a general agreement on the effectiveness of TMD systems. Relatively little research has been done about application of TMDs to control vehicle-induced bridge vibration and the TMD effectiveness for suppressing traffic vibrations has also not been clearly investigated and confirmed. Kajikawa et al. (1989) utilized a single TMD on highway bridges and concluded that this passive control device could not completely suppress traffic-induced vibrations since the dynamic responses of a bridge are frequency-variant due to vehicle motion. Kwon et al. (1998) inspected the TMD effectiveness on a high-speed railway continuous bridge with three spans in Korea. In their study, a single TMD with the parameters proposed by Den Hartog (1956) was used. The numerical results for the bridge subjected to the French T.G.V. train with an arbitrary train speed of 300 km/h showed that the TMD with mass ratio of 1% would be able to reduce the bridge vertical free displacement response for about 21% but is less effective in suppressing vertical acceleration due to passenger car since the vehicle passage time on the bridge is too short. Wang et al. (2001) focused on vibration suppression for high-speed railway bridges using tuned mass dampers. They concluded that for simply supported bridges of Taiwan High-Speed Railway (THSR) under German I.C.E., Japanese S.K.S. and French T.G.V. trains, numerical results show that the TMD is a useful vibration control device in reduction of bridge vertical displacements, absolute accelerations, end rotations and train accelerations during resonant speeds of trains. Xiaomin and Cai (2008) studied analytically the suppression of vehicle-induced vibration using tuned mass dampers in short and medium span highway bridges. They found that the additional damping provided by the TMDs, results in less reduction of the maximum dynamic displacement during the forced vibration period than during the free vibration, due to the fact that the forced vibration period is too short and the TMD does not have enough time to respond.

2 CASE STUDY

In early 1970's a series of typical orthotropic steel deck flyover bridges were installed in several key locations of Tehran, capital city of Iran. These bridges were considered as a temporary traffic solution. Nevertheless with rapid expansion of the Tehran City, they have never been replaced and are still under service. Occasionally, Tehran Municipality has received some complaints from residents living in the vicinity of these bridges and as well some vehicle passengers about annoying or even structurally-damaging traffic-induced vibration. However, the severity of the case matter and its resolution couldn't be proved unless field tests were performed. The main focus of this paper is on the measurement of traffic-induced vibration during field tests and finding effective methods to reduce them to acceptable limits if proved to be excessive. Since orthotropic decks have low inherent damping relative to other types of bridges, even small amplitude vibrations may lead to resonance. Application of diaphragm cross-frames and TMDs were proposed as two solutions to increase stiffness and damping of the bridges by TAZAND Consulting Engineers. However the effectiveness of solutions couldn't be investigated through unverified structural models. Thus field tests were planned to check and verify the dynamic models through which assessment of solutions would be more accurate.

Orthotropic flyover bridges of this study are composed of fifteen 24-meter simple supported spans which were separated by the expansion joints. Field tests were carried out on one of the typical spans. Each span of the orthotropic deck is seated on two steel hammer head-shape piers with I cross section. There is no sidewalk on the bridge deck and four traffic lanes are passing over 10.5-meter width of the deck. The steel used for the piers and the deck is DIN St-52. The deck is composed of six girders which 4 longitudinal welded HEA200 ribs stiffen the 10 mm deck plate of each girder. There aren't any cross-frames or trusses between girders as a diaphragm and just one tie with a channel section connects girders transversely, consequently girders are almost independent in gravity load carrying of the bridge. Deck weight of each span is about 74 tons. Fig 1 shows the deck's cross section of typical orthotropic steel deck flyover bridges of Tehran.

3 FIELD TESTS

For the purpose of field testing of the bridge, ambient vibration tests were carried out on one span of the bridge with passing of a specific truck as a source of excitation. Acceleration and deflection responses of girders were measured by five Micro Electro Mechanical System (MEMS) accelerometers and one Linear Variable Differential Transformer (LVDT). All sensors were installed at mid-spans of girders. For exciting the bridge, a three-axle truck with 41 tons weight passed at four different speeds i.e. 8, 20, 40 and 60 Km/h on two specified lanes. Fig 2 shows truck's transverse position on two different lanes on the deck during tests. By changing the transverse position of truck and measuring the deflection of G1 girder, the level of continuity of girders in transverse direction can be assessed. One static test was performed additionally that truck stayed at mid-span and LVDTs measured static deflection of underlying girder to identify the deck bending stiffness.

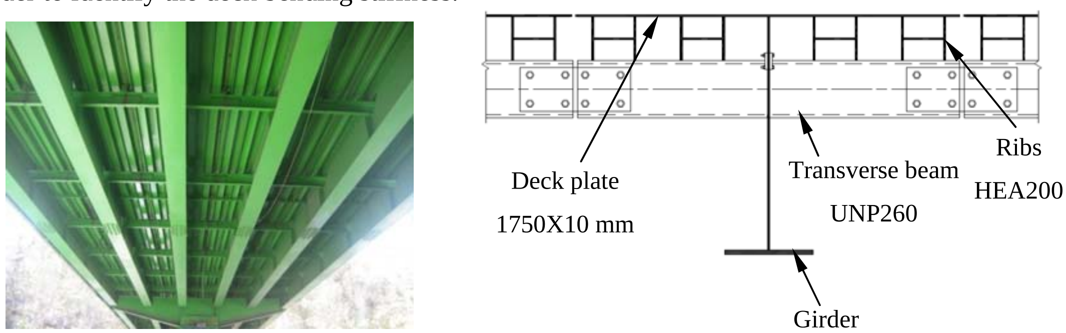


Figure 1. Details of typical orthotropic steel deck flyover bridges of Tehran.

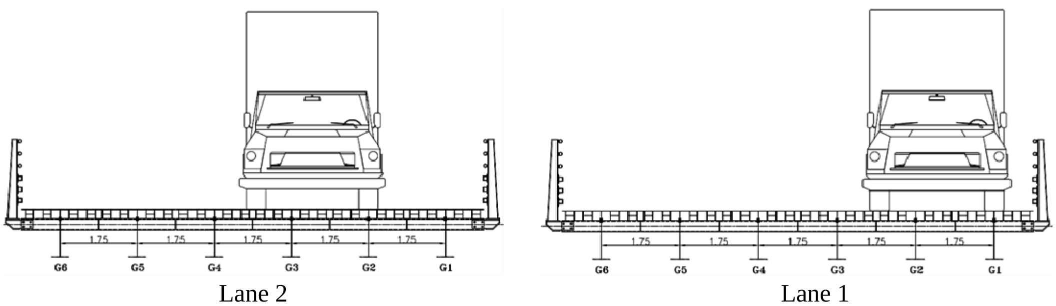


Figure 2. Transverse positioning of truck during the test.

It is possible to assume the bridge response under passing truck at 8 Km/h as pseudo dynamic response and at other speeds as dynamic ones. Consequently according to equation 1, Acceleration Increase Factor (AIF) can be defined as maximum acceleration of spectrum at each speed divided to maximum acceleration of spectrum at 8 Km/h speed.

$$\mathrm{AIF} = \frac{\mathrm{Max}\,(\ddot{y}_{(20,40,60\,\mathrm{Km/h})}(T))}{\mathrm{Max}\,(\ddot{y}_{(8\mathrm{Km/h})}(T))} \tag{1}$$

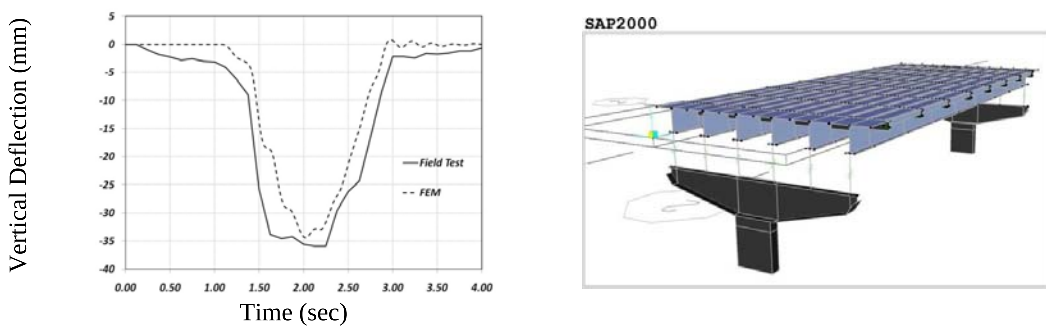
Table 1 shows maximum values of spectrum accelerations and associated AIF values at different speeds. AIF values increase by increasing the speed from 8 to 40 Km/h, however unexpectedly by increasing speed from 40 to 60 Km/h, AIF value is decreased. This effect probably is the consequence of surface roughness effects. Increasing vehicles speed lead to amplification of dynamic response and bridge deck acceleration and on the other hand helps with attenuation of surface roughness effects and causes bridge acceleration decrease. At specific speeds resultant of these effects may end up in reduction of bridge accelerations. Maximum deflections at the mid-span of G1 girder during the passing of truck with different speeds in different lanes are shown in Table 1 as well. Deflection of G1 girder decreases about 55% with change of truck transverse position from Lane 1 to Lane 2. This may shows low degree of transverse connectivity between the girders. Note that as the results of LVDT during passing of truck with 8 and 20 Km/h speed encountered with some errors were not reported in Table 1.

Table 1. AIF and maximum deflection of G1 girder at different truck speeds

Truck Speed (Km/h)	Maximum of spectrum accelerations (g)	AIF	Maximum deflections (mm)	
			Lane 1	Lane 2
8	0.193	1	---	---
20	0.323	1.67	---	15.77
40	0.543	2.81	36.57	14.72
60	0.412	2.14	35.98	17.51

4 FINITE ELEMET MODEL

Finite Element Method (FEM) was employed for parametric study through modeling in SAP 2000 software. All geometric, boundary and loading condition were defined in a Cartesian coordinate system. The model consisted of approximately 1250 elements. Rectangular 2d Shell elements were used to model the webs and flanges of girders and ribs. Transverse beams, piers and pier caps which have less effect on the deck vibrations, were modeled with linear 1d Beam elements. Finally characteristics of elastomeric bearings were assigned to link elements. Pier to the foundation connection was modeled as fixed. Vehicle was modeled as a moving load. Fig 5 shows geometry of the bridge model. To calibrate the model, results of field tests and FEM model are compared. Deflection of G1 girder under moving truck from field tests and FEM model are shown in Fig 5a.



a b

Figure 5. a) Comparison of results, b) Bridge model

Another calibration was performed by comparing analytical natural frequency versus first frequency obtained from the FEM. The analytical first natural frequency from Equation 2 as closed form estimation would be calculated 5.1 Hz;

$$f = \frac{\pi}{2l^2}\left(\frac{EI}{m}\right)^{1/2} \tag{2}$$

However FEM presents the first natural frequency of 5.04 Hz which differs from closed form results by 1 percent only. Note that in Equation 2, E, I, l, m and f are Young’s modulus, constant moment of inertia, span length, mass per unit length and first frequency of the bridge respectively.

5 PARAMETRIC STUDY

To assess efficiency of the proposed solutions, maximum accelerations and deflections of deck, with and without cross-frame and TMD are compared in the following.

5.1 Cross frame

As mentioned before girders are transversely connected only by ties made of UNP260 profiles. The cross-frame which proposed to increase transverse stiffness is an inverted V brace at the existing tie locations spaced 3 meters apart. Members and brace scheme are shown in Figure 6. Addition of cross-frames is deemed to improve load distribution and therefore it is expected to decrease deflections. Maximum dynamic deflections and accelerations of girders with and without cross frame calculated analytically are shown in Table 3. In this calculation it is assumed that truck passing over deck center line at 60 Km/h speed. Maximum deflection of girders G3 and G4 which were located exactly under truck were reduced by 28.7%; however deflections of G2 and G5 girders were approximately gained values like before cross-frame addition and deflections G1 and G6 girders increased. The values show that the deck after addition of cross-frames would be more integrated transversely and all girders participate in load bearing effectively. However since deck bending stiffness in longitudinal direction after cross-frame almost would not change, first bending frequency increase would be only 1 percent. In fact since maximum deflections of girders are consequences of load share values and bending stiffness, newly added cross-frame reduce load share of under truck girders even cannot increase longitudinal bending stiffness significantly. Consequently maximum deflection of under truck girders are reduced about 30 percent although other girders tolerate more deflections. Note that addition of new cross-frames even can suppress maximum acceleration of deck about 40% which may be accounted a significant benefit.

Table 3. maximum values of deflections and accelerations of girders with and without cross-frames

		G1	G2	G3	G4	G5	G6
Deflection (mm)	Without corss-frame	3.30	11.66	17.85	17.87	11.74	3.42
	With corss-frame	9.18	11.07	12.71	12.73	11.15	9.31
	Change (%)	+178	-5	-28.7	-28.7	-5	+172
Acceleration (m/s^2)	Without corss-frame	0.176	0.323	0.526	0.526	0.322	0.2
	With corss-frame	0.313	0.317	0.309	0.313	0.313	0.316
	Change (%)	+77	-1.8	-41	-40	-2.7	+58

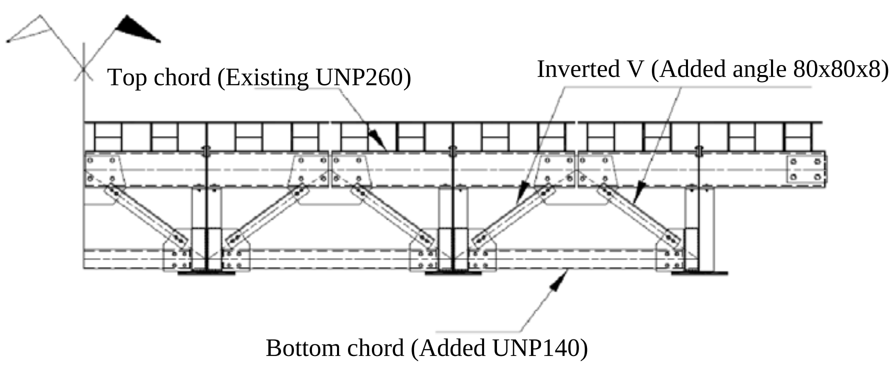


Figure 6. Newly added inverted V braces and bottom chords

5.2 Tuned mass damper

Since existing girders have a little degree of transverse connectivity, one TMD cannot affect on vibration of all girders. Thus application of three TMDs is proposed between each couple of girders. Mass of TMD is considered 5 percent of total deck mass. Optimum absorber parameters are calculated according to Warburton (1982) studies with the purpose of minimizing accelerations. Equations 3 express optimal parameters of proposed TMD. In these Equations μ, m, M, $\alpha_{opt}, \zeta_{opt}, \omega_s$, ω_a, k_{opt} and c_{opt} are mass ratio, TMD mass, deck mass, optimal frequency ratio, optimal damping ratio of TMD, natural frequency of deck, natural frequency of TMD, optimal stiffness of TMD and optimal damping of TMD respectively.

$$\mu = \frac{m}{M} = 0.05 \quad , \quad \alpha_{opt} = \left(\frac{1}{1+\mu}\right)^{1/2} = 0.976 \ , \ \zeta_{opt} = \sqrt{\frac{3\mu}{8(1+\mu/2)}} = 0.135$$
$$k_{opt} = \alpha_{opt}^2 \omega_s^2 m = 30.25 \ \text{ton/m} \ , \ c_{opt} = 2\zeta_{opt}\omega_a m = 1.66 \ \text{ton.s/m} \tag{3}$$

In FEM models, solid elements are used to model TMDs masses and stiffness and damping of TMDs is assigned to link elements. Natural frequency of deck with TMD is reduced 3 percent, which is due to TMDs masses. Maximum deflections and accelerations of girders with and without TMD are presented in Table 4.

Table 4. maximum values of deflections and accelerations of girders with and without TMD

		G1	G2	G3	G4	G5	G6
Deflection (mm)	Without TMD	3.30	11.66	17.85	17.87	11.74	3.42
	With TMD	3.6	11.7	17.5	17.58	11.89	3.7
	Change (%)	+8.3	+0.8	-2	-1.6	+1.2	+8.1
Acceleration (m/s^2)	Without TMD	0.176	0.323	0.526	0.526	0.322	0.2
	With TMD	0.158	0.305	0.489	0.488	0.304	0.18
	Change (%)	-10	-5.9	-7	-7	-5.6	-10

As Table 4 implies, TMD cannot reduce maximum deflection values effectively and suppression of accelerations are below 10 percent. In fact TMDs are passive devices that are involved in equation of motions after vibrating main structure. Thus if time of vibration be not enough, TMD could not affect properly. In these short span bridges, force vibration period (i.e. when the vehicle is on the bridge) is about 2 sec and maximum response occur before first second that is a little time for activation of TMD response. Consequently TMD declines maximum acceleration values only around 7 percent.

Another important issue in TMD application could be the vehicle mass effect on modal frequencies of bridge-vehicle system. It is apparent that the vehicle mass could change bridge frequencies. Moreover vehicle location on the bridge alters these frequencies. In Fig. 7 the dependence of the first bending natural frequency of a beam under moving mass against moving load $\bar{f}_1/f_1$ are shown. The values were plotted versus the point of action of mass x_0/l and for several values of the P/W ratio where x_0, l, P and W are location of vehicle, span length, weight of vehicle and deck, respectively. In this case study, P/W ratio is about 1.66, therefore when truck arrives in mid-span, $\bar{f}_1/f_1$ will be 0.48 and it means first natural frequency of deck change more than 50 percent due to vehicle mass. Consequently TMD which has been tuned for deck frequency would be detuned. As a conclusion, application of TMDs would not be the best solution for low weight decks because vehicle masses change deck frequency and TMD will be detune and its performance will decrease.

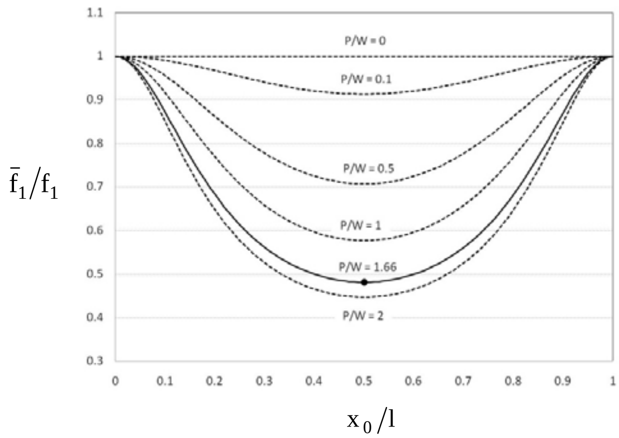


Figure 7. Dependence of first natural bending frequency of a loaded beam to mass position along the span length

6 CONCLUSION

Field tests were carried out on one of the typical orthotropic deck steel bridges of Tehran city using ambient vibration excitation of a truck. Test results show that level of transverse connectivity between the girders is relatively low. Performance of cross-frames and TMDs were studied as two solutions to increase stiffness and damping of the deck, respectively. Addition of Cross-frames improved load distribution between the girders and therefore load share of under truck girders reduced significantly up to 30%, as well cross-frames decrease maximum accelerations about 40%. Application of TMDs cannot reduce deck deflections significantly. Deck maximum accelerations in case of TMD application reduce about 7%. This may be due to the fact that the forced vibration period is too short and TMDs have not enough time to respond. On the other hand, passing vehicle mass can change the natural frequency of vehicle-bridge system and as a consequence, it can be said that application of TMDs may not be the best solution for low weight decks because each vehicle mass can change significantly the frequency of combined vehicle-bridge vibrating system. Therefore TMDs would be detuned and their performance would be decreased.

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