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Effects of traffic-induced vibrations on bridge-mounted overhead sign structures

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Abstract. Large-amplitude vibration of overhead sign structures can cause unfavorable psychological responses in motorists, interfere with readability of the signs, and lead to fatigue cracking in the sign structures. Field experience in Texas suggests that an overhead sign structure can vibrate excessively when supported within the span of a highway bridge instead of at a bent. This study used finite element modeling to analyze the dynamic displacement response of three hypothetical sign structures subjected to truck-passage-induced vertical oscillations recorded for the girders from four actual bridges. The modeled sign bridge structures included several span lengths based on standard design practices in Texas and were mounted on precast concrete I-girder bridges. Results revealed that resonance with bridge girder vertical vibrations can amplify the dynamic displacement of sign structures, and a specific range of frequency ratios subject to undesirable amplification was identified. Based on these findings, it is suggested that this type of sign structure be located at a bridge bent if its vertical motion frequency is within the identified range of bridge structure excitation frequencies. Several alternatives are investigated for cases where this is not possible, including increasing sign structure stiffness, reducing sign mass, and installing mechanical dampers.

Keywords: overhead sign structure; sign structure vibration; bridge vibration; dynamic displacement

1. Introduction

An overhead sign structure, also called an overhead sign bridge, is typically mounted on the abutment or the bent cap of a highway bridge. However, there is a need to install the sign structure within the highway bridge span for effective traffic operation purposes. When the sign structure is mounted within the bridge span, its dynamic motion can be affected by bridge vibration. Schell *et al*. (2006) reported that traffic-induced highway bridge vibrations could cause excessive dynamic displacements of sign structures, invoking unfavorable psychological responses from motorists.

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Table 1 Geometries and fundamental frequencies of instrumented bridges

Site	Span (m)	Width (m)	Lanes	Skew (degrees)	Num. of samples	Fundamental frequency(Hz)	Mode shape
1	33.53	21.34	4	0	60	3.40	Torsion
2	36.58	21.34	4	0	50	2.85	Bending
3	39.62	15.85	3	0	32	2.48	Bending
4	32.31	21.34	4	20	41	3.95	Torsion

Furthermore, the large-amplitude vibration of sign support structures not only hinders motorists from reading signs easily and quickly, but also causes fatigue damage (Hosch and Fouad 2010). Unfortunately, current design specifications for support structures do not provide practical limits on live load displacement, whereas there are provisions on the limit for dead load deflection to avoid wind-induced vortex shedding (AASHTO 2013).

In this study, vibrations of overhead sign structures mounted on highway bridges are investigated and a design method is suggested to avoid excessive vibrations by heavy trucks. Three-dimensional, truss-type sign structures following the standard design practice in Texas are covered in this investigation. Span lengths of the sign structure models considered are 15.24 m, 22.86 m, 30.48 m, and 36.58 m, with various sign types including a regular sign, an LED sign, and a fiber optic sign (FOS). Vertical accelerations measured at the exterior girders of four highway bridges are utilized as the boundary conditions in models simulating the dynamic motion of the sign structure mounted on bridge spans.

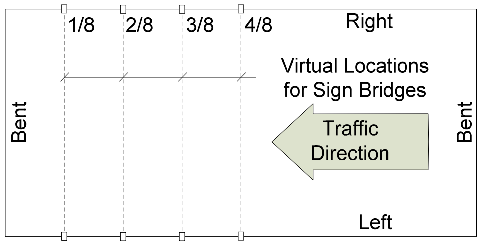
Results from the models allow calculation of maximum displacement ranges for the sign structures, which are compared with displacement limits proposed by Ghosn and Moses (1998). Where displacement results are undesirable, analysis of bridge vibration modes provides insights as to the effect of mounting location. Where sign location cannot be modified (e.g., for reasons of visibility to motorists), the sign structure may be retrofitted to achieve better sign displacement properties given the type of bridge structure present.

2. Field test and data acquisition

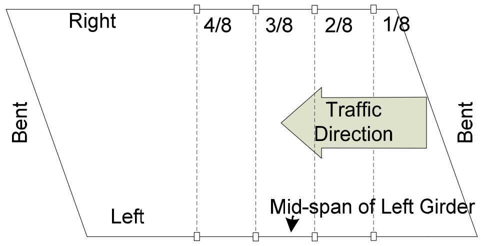
Four precast concrete I-girder bridges located in Austin, Texas, were selected as highway bridges on which sign structure models were mounted virtually. These were chosen to include a variety of span lengths, widths, and skews. Their spans varied from 32.31 m to 39.62 m, covering the commonly used range for highway bridge construction. The geometries of the four instrumented bridges, indicated as Sites 1, 2, 3, and 4, are summarized in Table 1. The bridges were instrumented to measure traffic-induced vibrations along their spans.

Accelerometers were installed on the bottom of the outer girders of these bridges. Vertical accelerations were measured using a data acquisition system with a 500-Hz sampling rate. Data were recorded for 20 to 25 seconds after the passage of heavy trucks. Figs. 1(a) and 1(b) show the instrument configuration of the highway bridges. Accelerometers were installed at four positions (i.e., 1/8, 2/8, 3/8, and 4/8 of the highway bridge span) along the outermost girders.

Figs. 2(a) and 2(b) show an acceleration time history measured at Site 1 with its frequency spectrum. More than 30 data samples were collected for each highway bridge. The lowest peak frequencies analyzed from these samples were averaged to determine the fundamental frequency

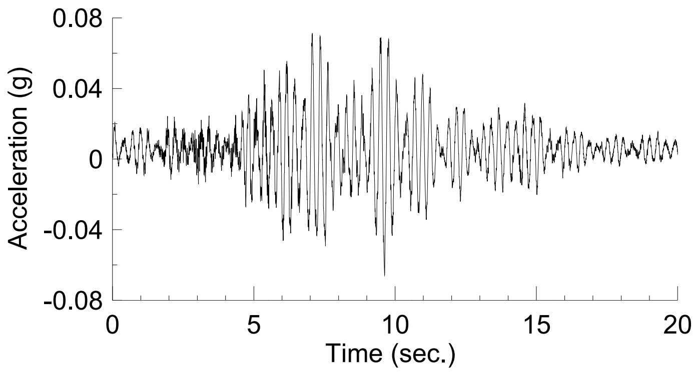


(a) Instrument locations for Sites 1–3

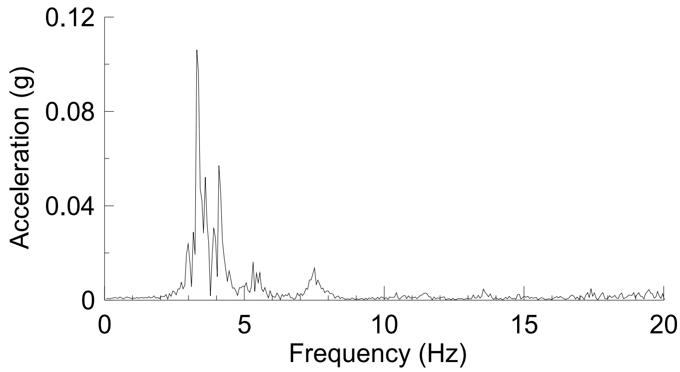


(b) Instrument locations for Site 4

Fig. 1 Accelerometer layout on sign bridge structures

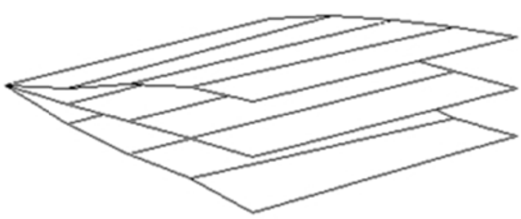


(a) Acceleration event

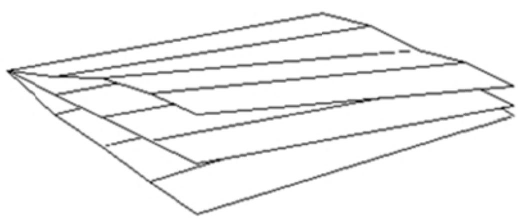


(b) Frequency response

Fig. 2 Acceleration and its frequency spectrum at Site 1



(a) Bending mode



(b) Torsional mode

Fig. 3 Mode shapes of the bridges

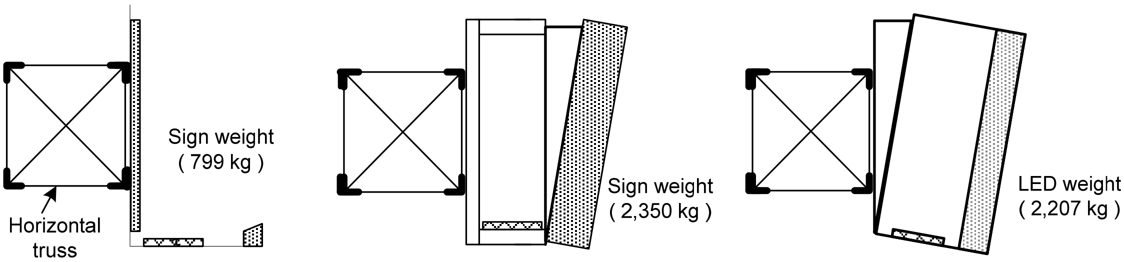
of each highway bridge. Spectral analysis results are summarized in Table 1, where fundamental frequencies and the corresponding mode shapes are presented for each highway bridge. Mode shapes were computed by integrating measured accelerations twice with respect to time. As shown in Fig. 3, the highway bridges show flexural or torsional modes in response to traffic loads. The mode shapes are cut at mid-span and the left edges represent pier locations. Middle surfaces in the plot represent undeformed bridge shapes. Among the collected data, an acceleration set showing the largest amplitude oscillation for each highway bridge was selected as the input to the overhead sign structure models.

At Sites 1 and 4 the torsional mode was dominant while the flexural mode was foremost at Sites 2 and 3. The dominant mode of bridge vibration (i.e., the mode corresponding to the lowest peak frequency in the response spectrum) can be affected by the geometry and the live load configuration of the bridge. When one side of a multi-lane bridge is loaded by traveling trucks at the time of data acquisition, the bridge can develop a torsional mode as shown in Fig. 3. Although the bridge is loaded concentrically by the live load, the torsional mode may dominate vibration when the bridge is skewed. For example, the bridge at Site 4 has a 20° skew. Even if the sign structure is mounted perpendicular to the roadway (not parallel to the pier) as indicated in Fig. 1(b), the two ends of the sign structure will be located at different positions along the two outside girders. These two positions are prone to move out-of-phase due to geometrical asymmetry, resulting in the torsional mode of bridge vibration.

3. Dynamic analysis of sign structure models

This study investigates the dynamic displacement responses of truss-type overhead sign structures subjected to the vibrations of highway bridges. The sign structures considered vary in span length and in the sign type attached to them: four span lengths (i.e., 15.24, 22.86, 30.48, and 36.58 m) and three types of signs (i.e., regular, FOS, and LED signs) are selected. It is assumed that each sign is mounted at the center of the sign structure span. The bare sign structures with no sign are also modeled and analyzed for the purpose of reference. Fig. 4 illustrates schematically the configurations of the sign types considered.

ABAQUS 12 (Dassault Systemes 2012a) is utilized to develop the sign structure models. The sign structure members are modeled using two-node beam elements (B31) and eight-node shell elements (S8R). Vertical accelerations measured in the field test are applied as the boundary conditions for the sign structures. Fig. 5 shows one of the sign structure models with a regular



(a) Regular sign (b) Fiber optic sign (c) LED sign

Fig. 4 Cross-sectional views of signs

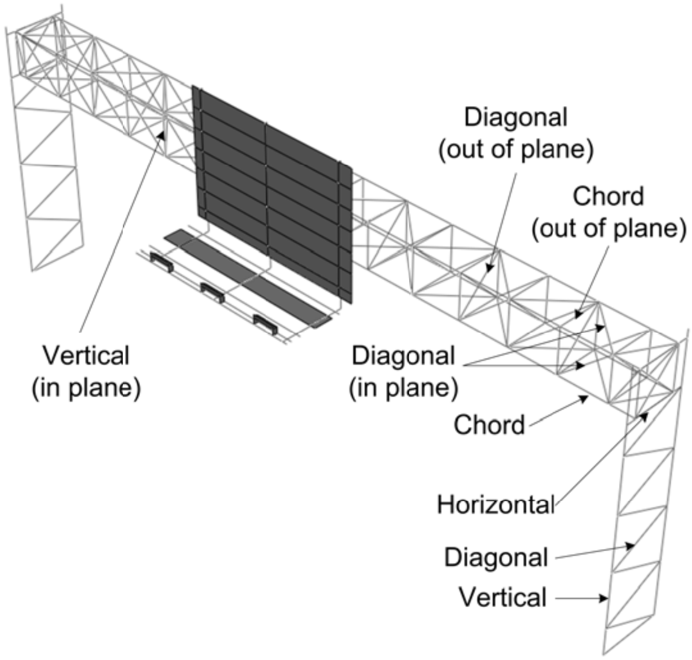


Fig. 5 Sign structure model with a regular sign

Table 2 Configurations of sign structure models

Column members					
Model	Vertical		Diagonal	Horizontal	
15.24 m	W10×22		2Ls2.5×1.5×0.188	L4×4×0.313	
22.86 m	W16×36		2Ls3×2.5×0.25	L5×5×0.375	
30.48 m	W14×30		2Ls3×3×0.188	L5×5×0.5	
36.58 m	W14×34		2Ls3×2.5×0.25	L5×5×0.5	
Beam members					
Model	Chord	In-plane diagonal	In-plane vertical	Out-of-plane diagonal	Out-of-plane chord
15.24 m	L3×3×0.188	L2.5×1.5×0.188	L2.5×1.5×0.188	L2.5×2.5×0.188	L2×2×0.188
22.86 m	L3.5×3.5×0.313	L2.5×2.5×0.188	L2.5×2.5×0.188	L3×3×0.188	L2×2×0.188
30.48 m	L3.5×3.5×0.375	L3×2×0.188	L3×2×0.188	L3×2.5×0.25	L2.5×2.5×0.188
36.58 m	L4×4×0.438	L3×2.5×0.188	L3×2×0.188	L3×3×0.25	L2.5×2.5×0.188

sign. Detailed information on the sign structure members is summarized in Table 2.

3.1 Dynamic displacement response

A total of 256 cases are numerically simulated; these vary in sign type (i.e., no sign, regular, FOS, and LED), span length (i.e., 15.24, 22.86, 30.48, and 36.58 m), applied highway bridge acceleration (i.e., Sites 1, 2, 3, and 4), and mounting location (i.e., 1/8, 2/8, 3/8, and 4/8 of the

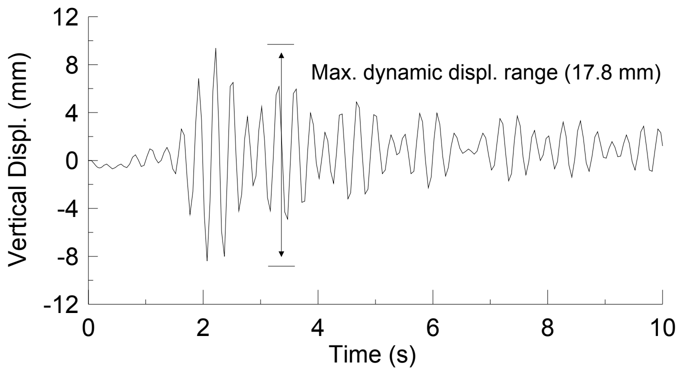


Fig. 6 Vertical displacement at the mid-span of a 22.86-m span sign structure

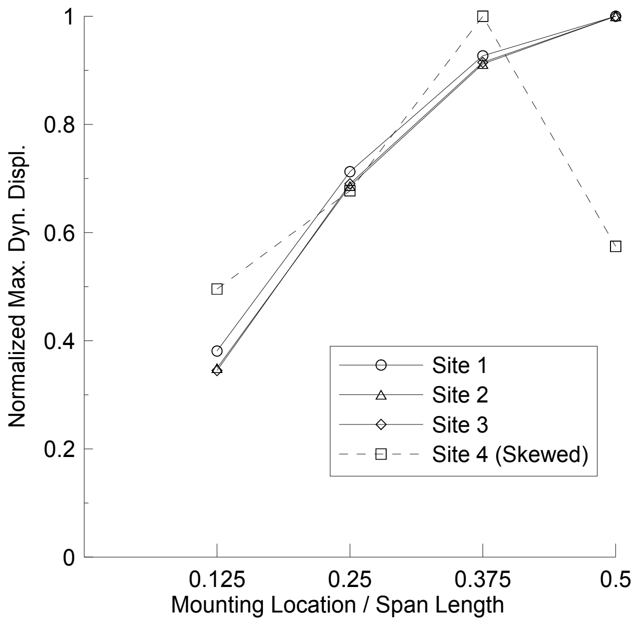
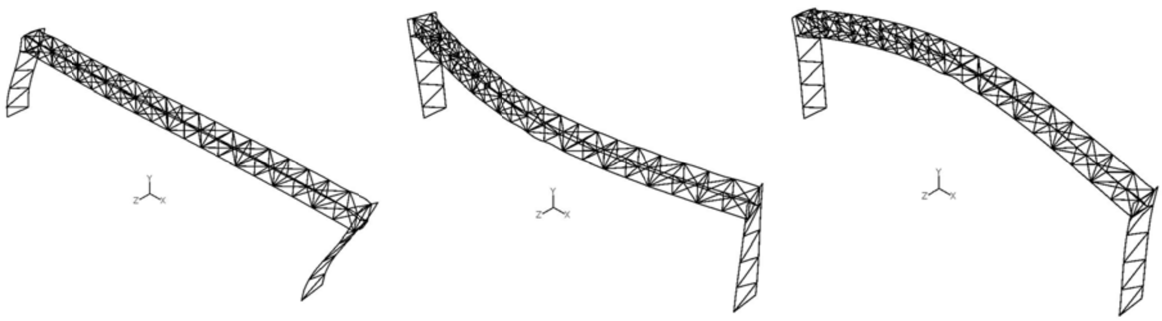


Fig. 7 Effect of mounting location along bridge span

bridge span from the bent). The acceleration data set with the largest amplitude measured at each bridge is utilized as input to the supports of the sign structure models. Dynamic analysis is conducted using an implicit time integration method (Dassault Systemes 2012b).

Vertical displacement responses at the mid-spans of the sign structures are calculated from dynamic analysis. Fig. 6 shows the vertical displacement response of a 22.86-m long sign structure with a regular sign. The model is excited by the mid-span acceleration of the bridge at Site 2. As indicated in Fig. 6, the maximum range of dynamic vertical displacement is 17.8 mm in this case. In fact, the maximum range of displacement depends on the mounting location of a sign structure because the magnitude of a bridge's vibration varies along its span. In Fig. 7, the displacement variation of the sign structure is shown along with its mounting location on the bridge. The displacement range is normalized with respect to the maximum displacement range of the sign structure. The displacement range of the 22.86-m long sign structure shown in Fig. 7 decreases by



(a) Transverse mode (b) Horizontal bending mode (c) Vertical bending mode

Fig. 8 Representative mode shapes of the truss-type sign structure (no sign is attached)

Table 3 Natural frequencies and mode shapes of sign structure models

Sign structure model	Mode	Frequency (no sign, Hz)	Mode shape	Frequency (Regular, Hz)	Mode shape	Frequency (FOS, Hz)	Mode shape	Frequency (LED, Hz)	Mode shape
15.24 m	1	3.14	T	2.53	T	1.94	T	2.03	T
	2	6.48	V+H	4.45	V+H	3.98	V+H	3.89	V+H
	3	6.64	V-H	6.14	V-H	4.72	V-H	4.71	V-H
22.86 m	1	4.28	T	3.75	T	2.99	T	3.19	T
	2	5.91	V-H	4.17	V+H	3.19	V+H	3.37	V+H
	3	6.4	V+H	4.96	V-H	3.65	V-H	3.77	V-H
30.48 m	1	2.86	T	2.6	T	2.2	T	2.27	T
	2	3.54	H	2.9	V+H	2.31	V+H	2.37	V+H
	3	4.04	V	3.35	V-H	2.48	V-H	2.61	V-H
36.58 m	1	2.78	T	2.52	V+H	2.06	V+H	2.11	V+H
	2	2.88	H	2.6	T	2.22	V-H	2.32	V-T
	3	3.27	V	2.90	V-H	2.27	T	2.33	V+T

over 60% if the mounting location is changed from the mid-span to the 1/8-span of the bridge at Site 2. The same tendency is observed for the other highway bridges investigated except for the bridge at Site 4, where the maximum response occurs when the sign structure is mounted at the 3/8-span location. This inconsistent result can be attributed to the bridge skew unique to Site 4. In other words, even if the sign structure is mounted with its right column at the mid-span of the right girder, its left-side column is mounted between the 1/4- and 3/8-span of the left girder, as indicated in Fig. 1(b). When the mounting location of the right-side column is moved to the 3/8-span of the right girder, the location of the left-side column comes closer to the mid-span than in the former case. If the magnitude of bridge vibration transmitted from the left- and right-side columns increases compared to the former case, the displacement response of the sign structure also increases.

3.2 Dynamic characteristics and displacement amplification

Natural frequencies and mode shapes of the sign structures are obtained using modal analysis.

The sign structures showed transverse, horizontal, and vertical bending shapes primarily in the lowest three modes of vibration, as shown in Fig. 8. These pure mode shapes, however, are not always distinct; rather, combined shapes are observed in many cases, as indicated in Table 3. In this table, the mode shapes are denoted by letters T, H, and V, indicating transverse, horizontal, and vertical bending modes, respectively. The combined mode shapes are denoted by V+H, V-H, V+T, and V-T. A positive sign in front of H or T indicates that the overall deflected direction of the mode shape follows the positive coordinate axis in the Cartesian coordinate system shown in Fig. 8, while a negative sign implies the reverse direction.

A displacement amplification factor (DAF) is used to analyze the vertical displacement response for each sign structure. The amplification factor is computed by dividing the maximum vertical displacement range of the sign structure by that of the bridge. It is assumed that the sign structure is mounted at the mid-span of the bridge. The maximum displacement range of the bridge is computed using the double-integration method from measured acceleration records as described earlier. Fig. 9 shows the amplification factor computed for the sign structures mounted at the mid-spans of the bridges at Sites 2 and 3. The normalized frequency is the ratio of the lowest frequency of vertical motion of the sign structure model, f_s (i.e., the modes denoted by V, V+H, or V-H in Table 3) to the fundamental frequency of the bridge, f_b. As shown in this figure, the amplification factor surges when the frequency ratio is between about 0.9 and 1.5. This implies that relatively large displacement can be induced by resonance between the sign structure and the bridge.

Contrary to the DAF results for Sites 2 and 3, the vertical motion of sign structures mounted on the bridges of Sites 1 and 4 are not as significantly amplified in this frequency range, as shown in Fig. 10. This can be attributed to the nature of the bridge vibration mode at these sites. At Sites 1 and 4, the torsional mode governs bridge vibration and sign structure vertical motion is less sensitive to the torsional vibration mode of a bridge than the bending vibration mode.

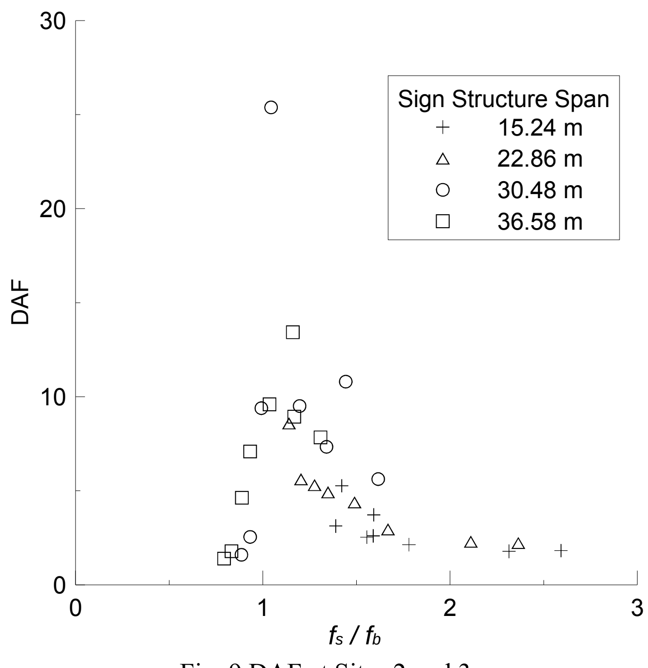


Fig. 9 DAF at Sites 2 and 3

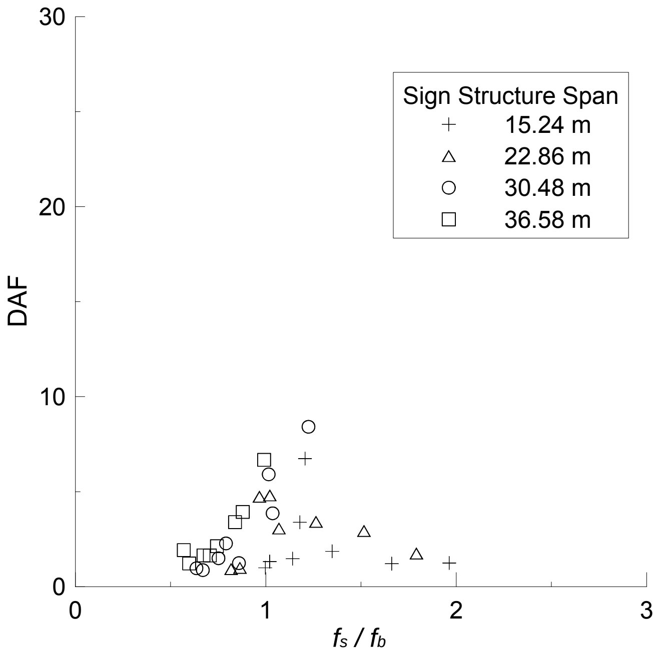


Fig. 10 DAF at Sites 1 and 4

3.3 Maximum dynamic displacement range

The maximum displacement range of a sign structure depends on the acceleration magnitude of the highway bridge on which the sign structure is mounted. The accelerations utilized in this study, however, were measured during a limited time period. Considering the uncertainty involved in traffic loads during the service life of the bridge, it is difficult to estimate the real maximum acceleration experienced by the bridge. For this reason, a code-specified deflection limit owing to vehicular loads is utilized to estimate the maximum displacement range of the bridge. AASHTO Bridge Design Specifications (2014) provide the deflection limit, as a function of the span/1000, needed to avoid undesirable structural and psychological effects when the bridge is subjected to live loads. This displacement limit consists of dynamic and static components imposed by vehicular loads. To account for dynamic effects, AASHTO Bridge Design Specifications consider a dynamic load allowance of 0.33, resulting in the increased load factor of 1.33. This load factor is multiplied by vehicular loads to compute displacement accounting for dynamic effects. In other words, the dynamic component of the displacement limit determined by the increased portion of the load factor corresponds to about 25% of the specified displacement limit. This dynamic component (i.e., 25% of span/1000) in line with the design specification's displacement limit provision is the amplitude of dynamic displacement. Therefore, the estimated displacement range of the bridge (EDB) is calculated by multiplying the amplitude of the dynamic displacement by a factor of two as indicated in Eq. (1).

$$EDB = 2 \times 0.25 \times \frac{span}{1,000} \tag{1}$$

$$EDS = DAF \times EDB \tag{2}$$

The expected vertical displacement range of the sign structure (EDS) is then computed by multiplying EDB by the displacement amplification factor (DAF) as in Eq. (2). The computed EDS results for the investigated sign structure models at the investigated sites are shown in Fig. 11.

4. Allowable dynamic displacement range

When a sign structure is installed on a highway bridge span, dynamic displacement of the sign structure should be carefully examined. If the natural frequency related to the vertical motion of the sign structure is close to the fundamental frequency of the bridge, then vertical displacement of the sign structure can be amplified excessively by the resonance effect. Such excessive dynamic displacement may not only cause motorists unfavorable psychological reactions, but also induces structural problems such as fatigue cracking. However, there is little guidance to limit the dynamic displacement of a sign structure.

To date, there has been little research on the displacement limit of sign structures in relation to unfavorable psychological effects. For this reason, a tolerable displacement limit proposed by Ghosn and Moses (1998) for a highway bridge is utilized as the allowable vertical displacement range for the sign structures in this investigation. They suggested 1% of the bridge span length as a visible and tolerable limit to a highway bridge user or an observer. The allowable displacement ranges according to this limit are indicated in Fig. 11 for the four different span lengths of sign structure models. As shown in this figure, EDS exceeds the allowable limit for at least one span length among the investigated span lengths of the sign structure when the frequency ratio of the sign structure is between 0.99 and 1.44.

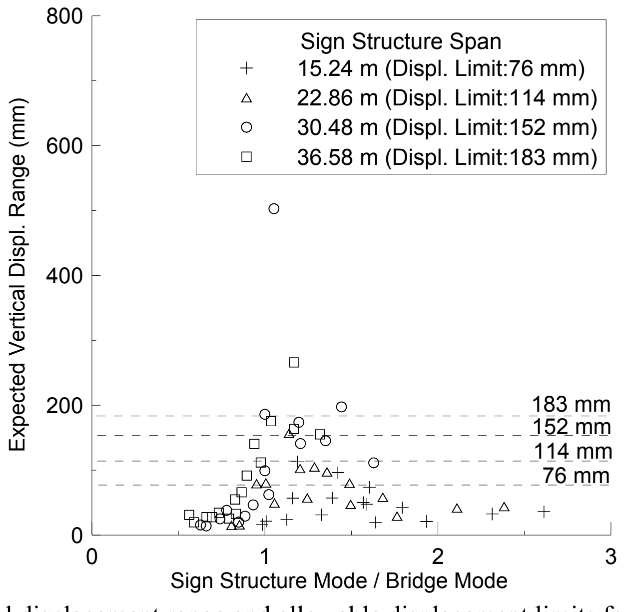


Fig. 11 Estimated vertical displacement range and allowable displacement limits for the bridges at Sites 1-4

Table 4 Effect of sign mass modification

Model	Regular sign		Modified sign	
	Frequency ratio	Displacement range (mm)	Frequency ratio	Displacement range (mm)
22.86	1.49	0.70	2.12	0.43
30.48	1.20	1.52	1.33	1.91

5. Retrofit of sign structures

Various mitigation methods should be considered when excessive vibrations are expected for a sign structure mounted on a bridge span. One of the possible approaches is to modify the mounting location of the sign structure. By mounting the sign structure near a bent cap, the dynamic displacement response can be reduced owing to the reduction of input vibration as shown in Fig. 7. Another approach is structural retrofitting such as modifying dynamic characteristics, including the stiffness, mass, and/or mechanical damping, of the sign structure. The first method, however, may not be cost-effective. The stiffness of the sign structure typically needs to be increased to meet strength limit criteria; this may raise construction cost excessively. Therefore, the other retrofit methods are considered below.

Extremely low damping (e.g., lower than 1% of the critical damping for cantilever-type sign structures) can make sign structures susceptible to large-amplitude vibrations (Dexter and Ricker 2002, Hosch and Fouad 2009). Tuned mass dampers have been utilized to control the vibrations of civil structures, and their effectiveness in vibration control has been reported by many researchers (Matta 2011, Lu *et al*. 2012, Roffel *et al*. 2013). A Stockbridge-type damper, which is a class of tuned mass dampers, has been widely used to control wind-induced vibration of sign structures owing to its relatively low cost and installation simplicity. Rice *et al*. (2012) investigated the effectiveness of these dampers in cantilevered sign support structures with sign panels and found that Stockbridge-type dampers could reduce vibrations caused by wind acting horizontally on the sign panels. Similarly, the traffic-induced vibration of an overhead sign structure mounted on a highway bridge span might be reduced by applying these dampers. Further research will be needed, however, to clarify the effectiveness of these dampers on the vibration of bridge-mounted sign structures.

Large-amplitude vibration can also be mitigated by changing the mass of the sign panels. For regular signs, the mass can be modified simply by removing appurtenances used for illumination such as the horizontal work platform, railing, and lights in front of the signs. AASHTO Specifications for Structural Supports (2013), however, mandate that all overhead signs be illuminated where a significant amount of traffic is expected at night because the amount of vehicle headlight illumination incident on an overhead sign display is small. For this reason, a supplementary method may be needed to compensate for the lack of illumination when sign-mounted lighting is removed. Zwahlen *et al*. (2003) suggested that highly reflective paints, currently available in industry, could be a possible solution for unlighted signs. They found that unlighted signs with white microprismatic legends on a green-beaded background could provide adequate readability.

In this study, the effectiveness of mass reduction is investigated with two different model spans: 22.86 m and 30.48 m. The appurtenances are removed in these models and the bridge vibrations of Site 2 showing the largest amplitude within the collected data are applied. Table 4 summarizes vertical displacement variations of the two sign structure models when the sign mass is modified. According to these simulations, the vertical displacement range is reduced for the 22.86-m-span model, but this reduction is not observed for the 30.48-m-span model. This inconsistent result can be attributed to the fact that the sign modification increases the frequency ratio of the 22.86 m model beyond the frequency range susceptible to resonance with bridge vibration (i.e., between 0.99 and 1.44) while sign modification in the 30.48 m model fails to shift the frequency ratio outside of this range. Therefore, reducing sign mass is not always effective for reducing the dynamic displacement of a sign structure.

6. Conclusions

This paper presents the results of an investigation into the vertical dynamic displacement response of overhead sign structures installed on highway bridge spans using numerical simulations. Highway bridge accelerations measured in the field are utilized as input to excite sign structure models. Natural frequencies of the sign structure models and the highway bridges are evaluated using modal and spectral analysis, respectively. According to the results, the natural frequencies of sign structures related to vertical motion are close to the fundamental frequencies of the bridges in many cases. Vertical displacement of a sign structure is significantly amplified by bridge vibration where bending is the dominant bridge vibration mode and the ratio of natural frequency of the sign structure to that of the bridge is within the range 0.99–1.44. Changing the mounting location of the sign structure from the mid-span to the 1/8-span of a straight bridge results in a vertical displacement reduction of over 60% owing to the smaller amplitude of bridge vibrations at the new location. Mass tuning on a sign can be effective in reducing the vertical displacement of a sign structure once its natural frequency related to vertical motion is shifted away from the fundamental frequency of the highway bridge on which the sign structure is mounted.

Therefore, it is recommended that overhead sign structures be mounted on pier caps wherever possible if the frequency ratio is in the range 0.99–1.44. If a sign structure needs to be mounted instead within a bridge span for effective traffic operation, it should be placed as closely as possible to the pier cap and/or installed using appropriate vibration mitigation methods. When mass modification of a sign is considered, the resulting natural frequency of the sign structure should be investigated to ensure that the modification shifts it such that the frequency ratio is outside of the range 0.99–1.44.

This study focuses only on the vertical motion of sign structures because the majority of highway bridges have fundamental modes of vibration with bending deformation. For skewed highway bridges, however, a torsional mode may be significant, and therefore the transverse displacement of sign structures needs to be investigated to study the dynamic interaction between sign structures and highway bridges.

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