WIND-EXCITED VIBRATIONS OF TRI-CHORD OVERHEAD SIGN SUPPORT STRUCTURES

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ABRIDGMENT

The dynamic response of two tri-chord truss overhead sign support structures under the action of wind loadings is examined analytically. Two structures, an 82-ft (24.9-m) steel frame and a 150-ft (45.6-m) aluminum frame, are used for this purpose. The wind loadings considered are the harmonic vortex-shedding excitations under moderate conditions and the random gusty drag force under extreme design conditions. The effectiveness of stockbridge dampers in reducing vortex excitation vibration of the aluminum frame is also investigated. The response of the two structures to random drag force is used to verify the adequacy of the gust factor recommended by the AASHO Specifications governing the design of sign support structures. For dynamic analysis, the structures are idealized as space frames with rigid joints. The masses are lumped at certain joints. The response of the frames to the two types of wind forces was determined by using the classical modal superposition method. Results of the study indicate that the stockbridge damper is effective in reducing vortex-excited vibration of the aluminum frame. The gust factor specified by the AASHO Specifications, however, appears to be insufficient.

•THE RESPONSE of two highway overhead sign support structures, one made of steel and one made of aluminum, of the tri-chord type to wind loadings is considered in this study. The wind loadings are considered to consist of a sinusoidal vortex-shedding force, transverse to the direction of wind velocity, and a drag force, in the direction of wind velocity. Under extreme design wind conditions, the drag force is accompanied by randomly fluctuating gusts that may induce vibratory motion in the structures. The gustiness of drag force and its dynamic effects are recognized by the AASHO Specifications (1) as gust factor in the computation of design wind loading. An attempt is made to verify, analytically, the adequacy of the value of the gust factor. Vortex-shedding forces have been observed to cause sustained vibrations of structures when the frequency of vortex shedding is close to that of one of the modes of vibration of the structure. This phenomenon occurs commonly at normal wind speeds that predominate during most of the life of the structures. To prevent large-amplitude vortex-shedding vibrations, AASHO Specifications (1) require the installation of damping devices (stockbridge damper) on all aluminum overhead sign support structures. In this study, the degree of effectiveness of stockbridge dampers in reducing vibrations of tri-chord overhead sign support structures is investigated analytically.

STRUCTURAL CHARACTERISTICS

A portion of the typical tri-chord overhead sign support structure is shown in Figure 1. Structure A is steel, and structure B is aluminum; their dimensions are shown in the figure. For dynamic response computations, the normal mode superposition method is employed. The structures are idealized as lumped-mass systems. For each structure, there are 18 lumped masses. The frequencies and mode shapes of these structures are computed by using the ICES STRUDL program. The first five natural frequencies are given in Table 1.

RESPONSE TO VORTEX SHEDDING

The vortex-shedding force exerted on the cylindrical members of the structures is considered (8) to be given by

$$F_1(t) = \tfrac{1}{2}\rho V^2 A \bar{C}_L \sin \Omega t \qquad (1)$$

where

ρ = density of air,
V = wind speed,
A_p = projected area of member,
$\bar{C}_L$ = root mean square value of random force coefficient C_L,
t = time,
Ω = SV/D = the vortex-shedding frequency,
D = diameter of the cylindrical member, and
S = the stouhal member.

In the present study, the maximum displacement amplitudes D_m of the two structures are computed both with and without signs mounted. Values of $\bar{C}_L$ = 1.0, S = 1.12 (5), and structural damping coefficient λ = 0.1 percent of critical damping for all modes are used. For structure A, D_m = 0.16 in. (4.06 mm) with signs mounted and 1.1 in. (27.8 mm) without signs at 7-mph (3.13-m/s) and 6-mph (2.69-m/s) wind speeds respectively. For structure B, D_m = 3.2 in. (81.2 mm) with signs, and 5.9 in. (150.0 mm) without signs at wind speeds of 6 mph (2.69 m/s) and 11 mph (4.92 m/s) respectively. That signs shield off vortex-shedding forces and reduce structural response is clearly noted.

The effects of 31-lbm (14.1-kg) stockbridge damper on the response of the aluminum structure are also examined analytically. When no signs are mounted, D_m reduces from 5.9 in. (150.0 mm) with no damper attached to about 0.5 in. (12.7 mm) when damper is used. When signs are mounted, D_m diminishes from 3.2 in. (81.2 mm) without damper to 0.3 in. (7.6 mm) with damper, indicating that the stockbridge damper is in fact an effective means of preventing excessive vibrations due to vortex-shedding excitations.

RESPONSE TO RANDOM DRAG FORCE

The random drag force exerted on members of the structure is considered (3) to be given by

$$F_2(t) = \tfrac{1}{2}\rho C_D V(t)\,|\,V(t)\,| + C_M \rho \frac{A_o}{D} A(t) \qquad (2)$$

where

C_D = coefficient of drag,
C_M = coefficient of virtual mass,
A_o = reference area for virtual mass,
$A(t)$ = wind acceleration,
$V(t)$ = $\bar{V} + v(t)$ = wind velocity,
$\bar{V}$ = mean wind speed, and
$v(t)$ = randomly fluctuating part.

In this study, the mean and standard deviation of the maximum displacement response of the two structures with and without the signs mounted are computed by using random vibrational analysis techniques. The values of C_D = 1.73 for cylinders, C_D = 1.15 for signs, and C_M = 1.0 are used (2). A value of 5 percent of critical damping, typical of the damping in these structures oscillating in high wind, is assumed for all modes. The spectrum for horizontal wind gusts is taken to be that proposed by Davenport (4). For an 80-mph (35.8-m/s) mean wind speed, the mean of maximum displacement $\bar{q}$, treated as the maximum static displacement under mean wind load, and the standard deviation σ_q of maximum displacement are computed and given in Table 2. The sufficiency of the gust factor recommended by the AASHO Specifications was examined (6) by using a peak

Figure 1. Dimensions of sign structures.

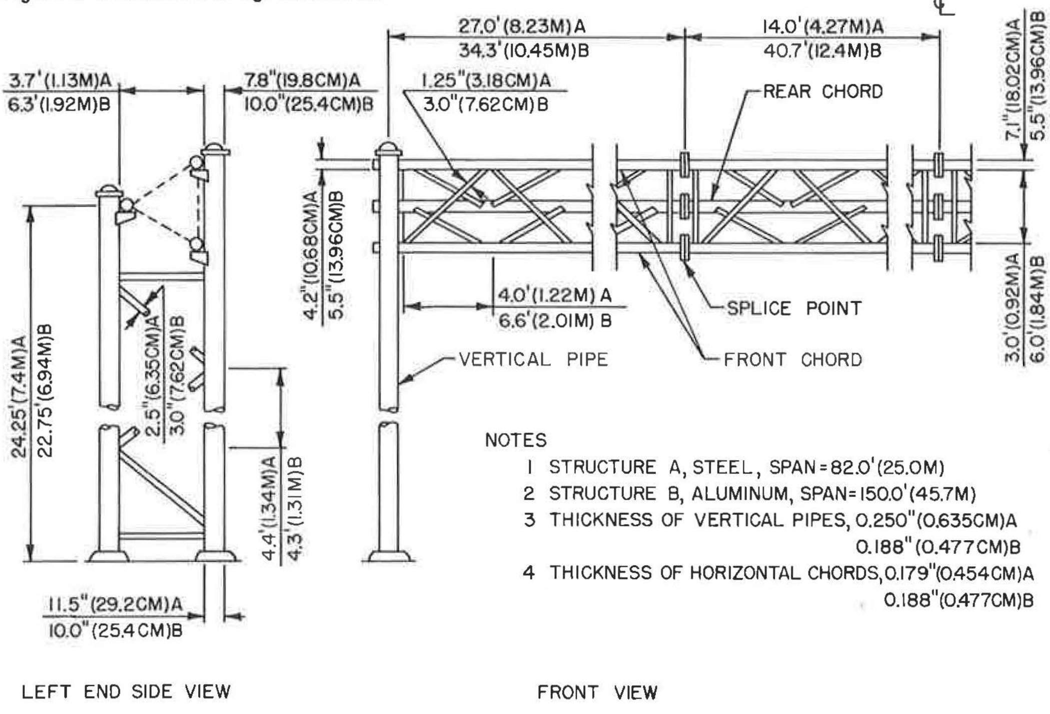

Table 1. Natural frequencies (in H_z).

	Structure A		Structure B	
Mode	Without Signs	With Signs	Without Signs	With Signs
1	2.254	1.913	2.638	2.094
2	3.479	2.909	3.229	2.496
3	3.833	3.072	3.432	2.574
4	7.749	5.681	9.787	5.966
5	11.857	7.806	10.501	7.837

Table 2. Maximum displacement (in inches) of structures to 80-mph wind.

	Structure A		Structure B	
Displacement	Without Signs	With Signs	Without Signs	With Signs
$\bar{q}$	0.59	1.94	5.04	6.78
σ_p	0.35	1.05	2.43	3.05
q_p	1.63	5.09	12.33	15.92
q_s	1.67	5.50	14.40	19.25
q_p/q_s	0.976	0.925	0.858	0.828

Note: 1 in. = 25.4 mm; 1 mph = 4.47 m/s.

displacement $q_p = \overline{q} + 3.0\sigma_q$. The quantity q_p so determined is compared with the quantity q_s (Table 2), the maximum displacement under statically applied wind drag force using a gust factor of 1.3 applied over the fastest mile wind speed (7) as specified by the AASHO Specifications. The ratio q_p/q_s, given in Table 2, ranges from 0.828 to 0.976, suggesting that the specified gust factor of 1.3 can be considered adequate for the type of structures examined.

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