Deflection Criteria for Sign Support Structures

By Mohammad R. Ehsani,[1] A. M. ASCE, and Reidar Bjorhovde,[2] F. ASCE

Abstract: The criterion set by the American Association of State Highway and Transportation Officials (AASHTO) for the design of highway sign support structures is examined. It is shown that this requirement, limiting the maximum dead load deflection of the structure in terms of the depth of the sign, is not a good indicator of the performance of the structure. A design procedure is suggested to replace the existing AASHTO criterion.

Introduction

Considerable progress has been made in recent years in improving the safety and aesthetic standards of modern highways. Whether in the form of roadway alignment, appearance and location of bridges, or the use of better highway sign structures, the current system reflects the advances that have been made.

In order to facilitate travel, the method and extent of highway signing are important considerations. Thus, the annual expenditure for signs and sign structures amounts to some $3,000,000 in the State of Arizona alone; the national figure may be $200,000,000 or more. The industry that supports these construction activities is therefore of economical significance in the overall transportation picture.

In the past, most of the overhead sign support structures utilized three-dimensional truss spans and frames, as shown in Fig. 1. These were constructed of steel or aluminum angles, channels, plates, and tubes. Although in most cases the sign-supporting portion of these structures would be a quadrichord truss, trichord trusses also have been common.

Over the past few years, however, the need for more economical and aesthetically pleasing structures led to the development of monotube systems. Fig. 1(*c*) shows a monotube frame structure, and Fig. 1(*d*) depicts what is known as a monotube span-type sign structure. Both of these types have seen increasing usage in North America, due to the relatively low construction cost, as well as the ease with which they may be dismantled and used at other highway locations.

The monotube structure is typically built with linearly tapered tubes of constant wall thickness. A splice is used at the midspan of the beam to allow easier erection, as well as to provide a symmetrically tapered member. The beam-to-column connection is a simple fastening detail that provides for some restraint of in-plane bending and essentially free out-

[1]Asst. Prof. of Civ. Engrg. and Engrg. Mechanics, Univ. of Arizona, Tucson, AZ 85721.

[2]Prof. of Civ. Engrg. and Engrg. Mechanics, Univ. of Arizona, Tucson, AZ 85721.

Note.—Discussion open until March 1, 1987. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on March 13, 1985. This paper is part of the *Journal of Structural Engineering*, Vol. 112, No. 10, October, 1986. ©ASCE, ISSN 0733-9445/86/0010-2332/$01.00. Paper No. 21006.

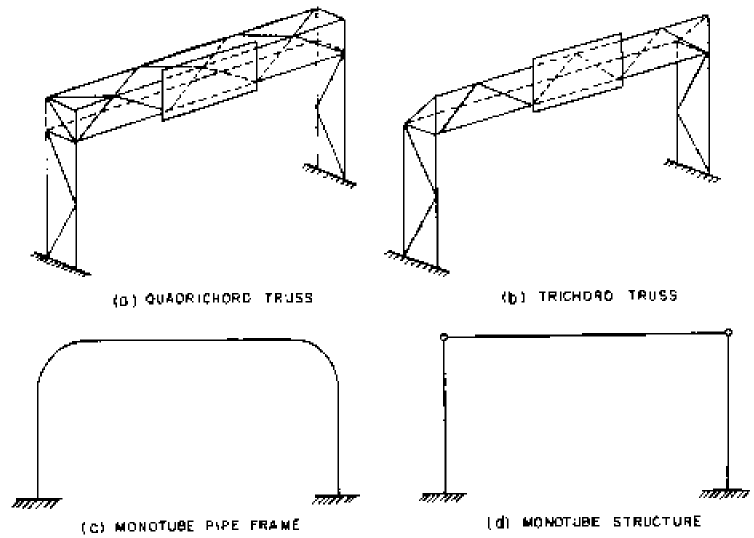

FIG. 1.—Configuration of Commonly Used Span-Type Sign Support Structures

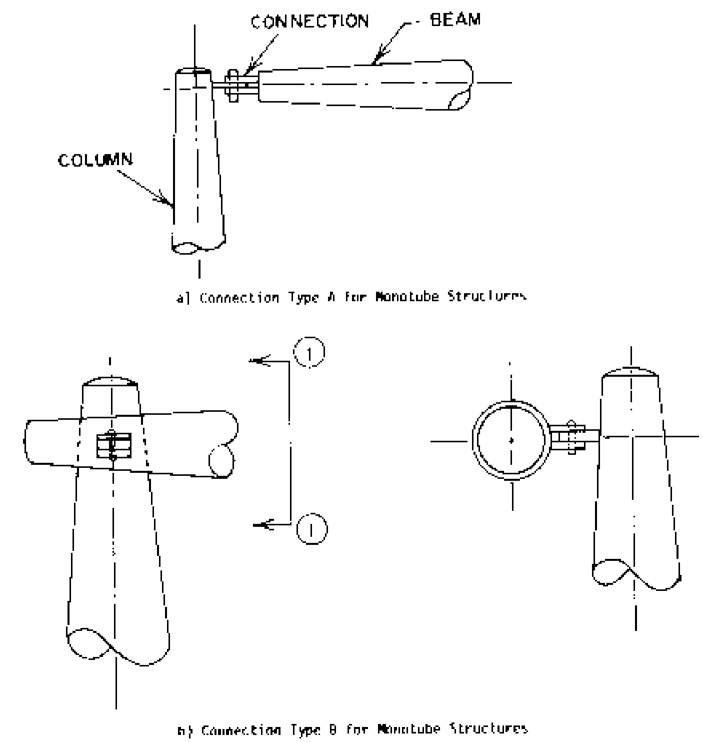

FIG. 2.—Some Typical Beam-to-Column Connection Details for Monotube Structures: (*a*) Connection Type A; (*b*) Connection Type B

of-plane rotation. Fig. 2 shows some typical connection details (schematic). Because of the simplicity of the monotube span-type structure, its design, fabrication, and erection tend to be easy to accomplish, thus permitting economy of construction. This type of sign structure therefore has many advantages over the truss types, as well as over the monotubular frames.

The design of highway sign structures is currently governed by the American Association of State Highway and Transportation Officials (AASHTO) *Specifications* (AASHTO 1975). This document details loading and member strength criteria, in addition to placing a limit on the dead load deflection of the beam (or sign bridge). It is the intent of this paper to examine the behavior of various types of sign support structures, and to correlate their strength and serviceability with those required by the *Specifications*. Particular emphasis will be placed on the performance of monotube structures, and the applicability of the AASHTO deflection criteria in general.

Background of Current Criteria

The gravity loading and general strength design criteria of the AASHTO *Specifications* are well documented, and in agreement with those of other codes. For example, the member strength requirements for sign support structures in steel parallel those of the American Institute of Steel Construction (AISC) (AISC 1978) very closely. In brief, therefore, with the typical span lengths and loadings that are used, the sign support structures normally do not experience any difficulties in satisfying the strength design criteria of the *Specifications*.

The AASHTO *Specifications* also give serviceability requirements for the sign support structures, specifically by limiting their dead load deflection. The design criterion is given as

$$\Delta_{DL} \le \frac{d^2}{400} \qquad (1)$$

where Δ_{DL} = dead load deflection (in feet); and d = vertical dimension (depth) of the traffic sign(s) (in feet). (In SI units the equation becomes $\Delta_{DL} \le d^2/37$, with Δ_{DL} and d both expressed in meters.)

A survey of the available literature shows that the origin of the $d^2/400$ requirement is insufficiently documented. In the commentary on the AASHTO *Specifications*, it is stated that models of frame-type overhead sign support structures were tested in a wind tunnel, and a tentative criterion was established to ensure that the structure did not suffer resonance due to wind-induced vortex shedding. It is emphasized here that the vortex shedding takes place in the vertical (in-plane of sign structure) direction, perpendicular to the direction of the wind. However, it is not clearly stated whether the wind tunnel tests contributed at all to the development of the $d^2/400$ criterion, or in what form.

The AASHTO commentary also notes that "For most structures, the frequency of the structure is very nearly one over the square root of the dead load deflection, in feet." As will be discussed in the following section of this paper, a review of the available literature indicates that this

assumption is not generally correct for all categories of structures. In any case, it is not documented to what extent the above relationship contributed to the development of the $d^2/400$ requirement.

Another account of the origin of the $d^2/400$ is given by Pelkey (Pelkey 1971). According to this evaluation, $d^2/400$ was obtained by equating the vortex shedding frequency of a sign panel, given as

$$f_v = \frac{SV}{d} \qquad (2)$$

to the fundamental frequency of a simple span beam of uniformly distributed mass and stiffness, given by the expression

$$f_0 = \frac{\pi}{2}\sqrt{\frac{EIg}{wl^4}} \qquad (3)$$

where f_v = vortex shedding frequency; S = Strouhal Number; V = wind velocity; d = depth of sign; f_0 = fundamental frequency of a simple span beam of uniformly distributed mass and stiffness; E = modulus of elasticity; I = moment of inertia; w = weight per unit length of the vibrating system; l = span length; and g = acceleration of gravity. Using $\Delta_{max} = 5(wl^4/384)/EI$ for the maximum deflection of a simply supported, uniformly loaded beam, it can be shown that Eq. 3 may be written as

$$f_0 = \frac{1.017}{\sqrt{\Delta_{max}}} \approx \frac{1}{\sqrt{\Delta_{max}}} \qquad (4)$$

in which Δ_{max} is expressed in units of feet. For a wind velocity of 80 mph (117.33 ft/sec), and a Strouhal Number of 0.2, which is applicable for most plates (Fung 1955), equating Eqs. 2 and 4 gives

$$\frac{(0.2)(117.33)}{d} = \frac{1}{\sqrt{\Delta_{max}}} \qquad (5)$$

which becomes

$$\Delta_{max} = \frac{d^2}{551} \qquad (6)$$

Although no reference is made to this effect in the AASHTO code (AASHTO 1975), according to Pelkey (Pelkey 1971) the $d^2/400$ criterion was developed from calculations similar to those shown above.

Evaluation of Current Criteria

At present, the only design criteria for sign support structures are those of AASHTO. As noted previously, the strength criteria governing the design of these structures are very similar to those provided by many other codes and are generally easily satisfied. Additional criteria, such as those detailing the fatigue performance of the members and connections, are also needed, but are beyond the scope of this article.

As an illustration of the characteristics of the response of monotube structures, a recently computed study at the University of Arizona (Ehsani, et al. 1985) indicated maximum combined stresses due to dead, ice, and wind loads for a typical monotube sign support structure with a span of 30.48 m (100 feet) of 119.3 MPa (17.3 ksi) at the midspan of the beam, and 128.6 MPa (18.65 ksi) at the base of the column. In a similar study at North Carolina State University (Mirza, et al. 1975) dealing with trichord sign support structures, the maximum stresses were calculated as 99.3 MPa (14.4 ksi) at the midspan and 138.6 MPa (20.1 ksi) at the base of the column. Both of these investigations showed that the stresses were well below the allowable values.

In contrast to the strength requirements, the AASHTO serviceability criterion, which limits the maximum dead load deflection to $d^2/400$, is very difficult, if not impossible, to satisfy for many sign structures. Fig. 3 gives the values of the maximum calculated dead load deflection as functions of the span length for three monotube (Ehsani, et al. 1985) and two trichord sign support structures (Mirza, et al. 1975). The limiting AASHTO deflection, based on a commonly used sign depth of 1.5 m (5 ft) is included in the figure for comparison. It is clear that although for short spans the AASHTO requirement can be satisfied, longer span sign structures of diverse types violate the criterion significantly. Thus, the calculated dead load deflection for a 45.72-m (150-ft) span trichord truss is roughly four times larger than the AASHTO limit of 19 mm (0.75 in.), using a 1.5-m (5-ft) sign depth. Similarly, for a monotube structure with a span of 36.58 m (120 ft), the maximum deflection is more than an order of magnitude larger than the AASHTO requirement.

In cases such as the preceding, the only option available to the designer is to increase the sign depth to produce an allowable deflection

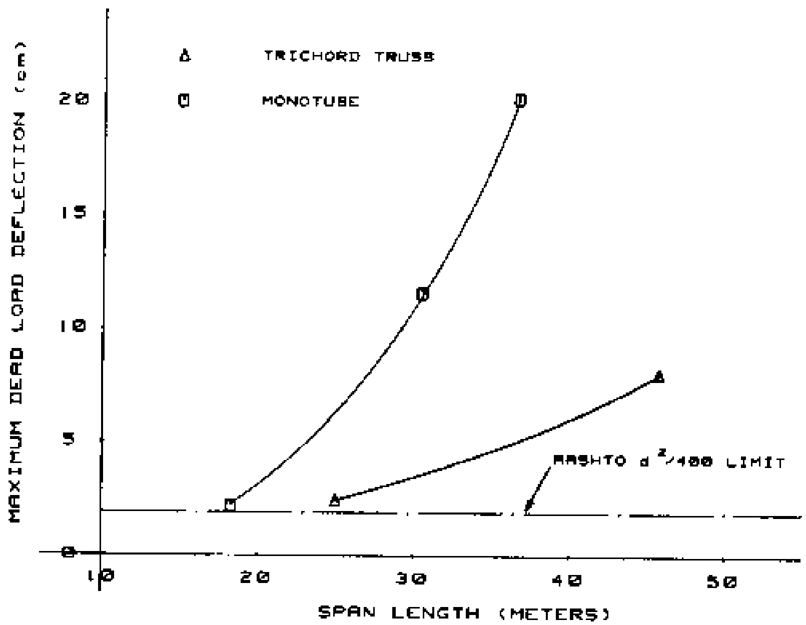

FIG. 3.—Comparison of the Dead Load Deflections with AASHTO Limiting Value

of $d^2/400$ that is larger than the computed one. For example, for a 30.48-m (100-ft) span monotube structure with a dead load deflection of approximately 117 mm (4.6 in.), a minimum sign depth of 3.68 m (12.25 ft) is needed to satisfy the AASHTO requirement. Seemingly acceptable in itself, it is an unusual criterion that ties the structural deflection to the depth of the traffic signs.

As was noted earlier, the $d^2/400$ criterion was developed by equating the first mode frequency of a simply supported, uniformly loaded beam to the frequency of vibration of a sign panel subjected to a wind speed of 128.8 kmh (80 mph). This implies that at this wind speed, the frequency of vortex shedding of the sign and the fundamental frequency of the structure are equal, producing resonance. Several questions have already been raised about the validity of the above assumptions; these will be discussed in detail in the following.

First, although the term $1/\sqrt{\Delta_{DL}}$ is a good approximation of the first mode frequency of a simply supported beam of uniform load and mass, it has limited application for practical sign support structures. The most common structures utilize trichord or quadrichord trusses or nonprismatic beams, as in monotube structures. In addition, the weight of the signs which are usually placed at arbitrary locations along the span, contribute to the nonuniformity of the mass of the beam element. Moreover, the typical beam-to-column connections provide at least some degree of fixity against in-plane rotation of the beam or truss element.

The calculated first mode frequencies of three monotube structures (Ehsani, et al. 1985) and two trichord trusses (Mirza, et al. 1975) are

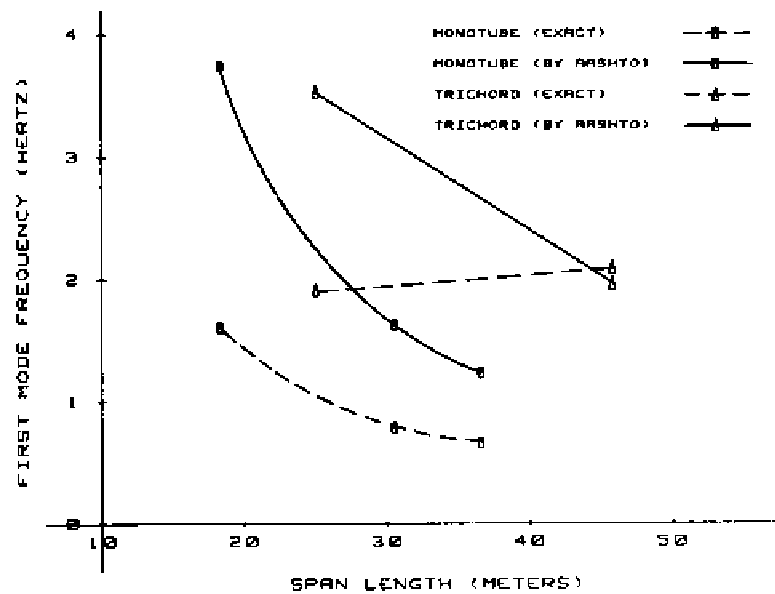

FIG. 4.—Comparison of the Exact First Mode Frequencies with AASHTO Suggested Values

indicated in Fig. 4. For comparison, the first mode frequencies as approximated by $1/\sqrt{\Delta_{DL}}$ are also plotted. Good agreement between the exact frequency and the approximation given by $1/\sqrt{\Delta_{DL}}$ is found only for very long span trichord trusses. For the monotube structures, the AASHTO approximation is between 1.86 and 2.33 times larger than the first mode frequencies, calculated by exact analysis for the spans investigated. Clearly, the applicability of the assumption that the first mode frequency of the beam can be approximated by the inverse of the square root of the maximum dead load deflection is questionable, and throws further doubt as to the use of the $d^2/400$ criterion.

Secondly, the assumption of the code (AASHTO 1975) that the vortex-shedding frequency of the structure is equal to the frequency of the first mode of vibration has not been verified by any published investigations. Studies at the University of Arizona (Ehsani, et al. 1985) and at North Carolina State University (Mirza, et al. 1975) indicate that the vortex-shedding frequency is developed only over a very narrow range of wind speeds (usually less than 1.6 kmh or 1 mph), and is close to the third or higher mode frequencies of the structure. It is therefore imperative that exact vibration characteristics of each structure be determined if a realistic, dynamic structural response is needed. However, as will be shown later, the latter is not needed in most cases, but it is important to recognize that "simple" relationships such as that of Eq. 4 can lead to extremely conservative design approximations.

Thirdly, the use of a maximum wind speed of 128.8 kmh (80 mph) for resonant conditions cannot be substantiated. Apparently, this wind speed was selected to represent the worst case of loading on the sign and the structure. While the vibration of a plate, i.e., the sign panel in this case, may be analyzed for high wind speeds (Hoerner 1965), the dynamic behavior of the sign support structure is well defined only up to a certain wind velocity. According to Fung (Fung 1955), for the purpose of determining the response of an elastic system with a circular cross section, such as that used in monotube or trichord truss structures, the forcing function is sinusoidal with a deterministic frequency but a random amplitude only for Reynolds Numbers R between 300 and 3×10^5. For R-values larger than 3×10^5, the forcing function is still sinusoidal, but with random frequency and random amplitude. Although certainly of interest, mathematical models are currently only available to determine structural behavior in the lower R-range.

It has been shown that for the monotube sign support structures, the maximum wind speed producing vortex shedding in the deterministic range is approximately 43.5–46.7 kmh (27–29 mph) (Ehsani, et al. 1985). Similarly, the maximum wind speed for trichord trusses was calculated as roughly 27.4 kmh (17 mph) (Mirza, et al. 1975). Although higher wind velocities may cause vortex shedding of the structure, the exact magnitude of the wind speed and the displacement of the structure cannot be determined analytically by currently available methods. It is therefore noted that while higher wind velocities such as 128.8 kmh (80 mph) may be of significance in determining the maximum stresses and deflections of a structure due to the equivalent static load, the distress caused by vibration often occurs at much lower wind speeds. As a result, extrapolation of the results into higher wind velocities for this category of

problems will only produce solutions which have limited analytical and practical basis, in addition to implying that the deterministic modeling is correct beyond R = 3×10^5. The extrapolation is therefore not suitable.

Suggested Design Considerations

While the performance of the sign support structures in the past has been generally satisfactory, substantial savings can be realized as a result of relaxing the current AASHTO code requirements. The use of a single, simple design criterion often results in structures which are overdesigned and thus uneconomical. Although the limitation of the dead load deflection to $d^2/400$ reflects concerns for structural stiffness and dynamic characteristics, its application is neither uniform nor rational in all instances. As an example, the significance of the span length is ignored in the $d^2/400$ criterion. On the other hand, if the dead load deflection of a particular structure is found to be too large, the code requirement can be easily satisfied by specifying an unrealistically deep sign to be mounted on the structure.

It is recommended that the design of the sign support structures should include the following as a replacement for the current $d^2/400$ requirement:

1. Static stresses and in-plane and out-of-plane deflections must be calculated for all realistic combinations of dead, ice, and wind loads. It is noted that the one-third increase in allowable stresses are permitted for the loading cases which include wind loads.
2. Dynamic stresses and in-plane deflections due to dead load and wind loads causing possible resonance of the structure must be investigated. The dynamic analysis can be limited to the beam portion of the structure, ignoring the deformations of the columns. In particular, the narrow ranges of wind speed which cause resonance of the structure must be determined.
3. Large deflections under dead load and ice load are unavoidable, especially for the more flexible monotube structures. In order to improve the appearance of the structures, the dead load deflections can at least be partially corrected by proper cambering of the beam.
4. Very limited data are available on the fatigue performance of the structures. However, the stress ranges appear to be very low.

The foregoing procedure should result in more economical structures, compared to those designed on the basis of the current code. It is expected that considering the similarity of the structure and their loading conditions, design charts can be developed for each category of structures such as monotubes, trichords, etc. These can reflect the effects of such variables as span length, and the dimension and location of the signs.

Summary

One of the current design criteria for highway sign support structures, given by the American Association of State Highway and Transportation

Officials (AASHTO), requires that the maximum dead load deflection of the structures (in feet) be limited to $d^2/400$, when d is the sign depth in feet. Due to the long span and flexibility of most such structures, this limit cannot be satisfied in many cases.

The deflection criterion was developed in an attempt to eliminate the vortex shedding of the sign panels mounted on a structure. In doing so, it was assumed that the mass of the beam element is uniform, and that vortex shedding may occur at a wind speed of 128.8 kmh (80 mph). However, these assumptions are incorrect for a large number of structures. The wind speed assumption can be particularly misleading, as it requires extrapolating results obtained using low wind speeds into a wind speed range where no analytical solution exists.

Considering the importance of these structures, it is recommended that a more thorough static and dynamic analysis be substituted for the current, simple $d^2/400$ limitation on the dead load deflection. It is suggested that design charts can be developed for different categories of structures, which would include the effect of such parameters as span length, stiffness, size, and location of the signs. This design procedure will not be much more complicated than what has been used in the past. However, it is clear that it will lead to construction economy in an important area of the transportation infrastructure.

Acknowledgments

This study was part of a larger research project sponsored by the Arizona Department of Transportation (ADOT). The interest and support provided by Frank R. McCullagh, Mumtaz Sarsam, and James R. Pyne are very much appreciated.

Appendix I.—References

American Association of State Highway and Transportation Officials (AASHTO). (1975). *Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*. AASHTO, Washington, D.C.

American Institute of Steel Construction (AISC). (1978). *Specifications for the Design, Fabrication and Erection of Steel Structures for Buildings*. AISC, Chicago, Ill.

Ehsani, M. R., Chakrabarti, S. K., and Bjorhovde, R. (1985). "Static and Dynamic Behavior of Monotube Span-Type Sign Structures." *Report No. FHWA/AZ 85/194-I and -II* (2 Vol.). Arizona Department of Transportation, Phoenix, Ariz.

Fung, Y. C. (1955). *An Introduction to the Theory of Aeroelasticity*. John Wiley and Sons, Inc., New York, N.Y.

Hoerner, S. F. (1965). *Fluid-Dynamic Drag*. Hoerner Fluid Mechanics, Brick Town, N.J.

Mirza, J. F., Tung, C. C., and Smith, J. C. (1975). *Static and Dynamic Behavior of Trichord Truss Overhead Sign Support Structures*. Highway Research Prog., North Carolina State Univ., Raleigh, N.C.

Pelkey, R. E. (1971). "New Design Approach to Long-Span Overhead Sign Structures." *Highway Research Record*, No. 346, Highway Research Board of National Research Council, Washington, D.C., 1–10.

Appendix II.—Notation

The following symbols are used in this paper:

d = depth of sign;
E = modulus of elasticity;
f_v = vortex shedding frequency;
f_0 = fundamental frequency of a simple span beam of uniformly distributed mass and stiffness;
g = acceleration of gravity;
I = moment of inertia;
l = span length;
R = Reynolds Number;
S = Strouhal Number;
V = wind velocity;
w = weight of the vibrating system; and
Δ_{DL} = vertical deflection of a beam at midspan due to dead load.