FIELD TESTING OF HIGHWAY SIGN SUPPORT STRUCTURES

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ABSTRACT: The results of field tests for two monotube highway sign support structures, each with spans of 60 and 100 ft, are presented. The intent of the study was to compare field measurements with previously conducted analytical results, including the possibility of resonance due to vortex shedding. Wind speeds of up to 25.2 mph were recorded during the tests. Although the analytical studies predicted resonance tendencies at several lower wind speeds, this did not take place during the field tests due to the inherent damping of the structures. Measured stresses were analyzed statistically to account for the cyclic nature of the loads to assure a high level of confidence in the maximum stress levels. There was good correlation between computed and measured stresses, all of which were found to be well below allowable values. The adequacy of previous design recommendations was verified.

INTRODUCTION

As the width of highways has grown, so have the sizes of the necessary sign support structures, until today spans in excess of 100 ft are not uncommon. To support signs over these large spans, truss-type structures have traditionally been used. These typically consist of two columns supporting a truss or trichord element, and the traffic signs are arranged in the desired locations and bolted in place.

The design of sign support structures is based on the American Association of State Highway and Transportation Officials's (AASHTO) 1975 "Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals." For convenience, this document will be referred to in the following as the Specifications.

The Specifications set minimum performance guidelines. Among these are criteria governing deflections. Essentially, the maximum static dead load deflection, in units of feet, is limited to the empirical value of 0.006/400, where / = the depth of the sign in feet. The rationale and consequences of this approach have been discussed by Ehsani and Bjorhovde (1986).

Over the years, the performance of the truss structures has generally been satisfactory. However, there are some drawbacks to their use. They are expensive to fabricate, and for that reason are not as economical as some of the pre-engineered structures that are available. One of the latter types that has seen increased use is the monotube sign support structure, which is significantly lighter as well as more attractive than most truss structures.

FIG. 1.—60-ft Monotube Structure Located in Tucson, Arizona

FIG. 2.—100-ft Monotube Structure Located in Phoenix, Arizona

Monotube structures are constructed of linearly tapering elements that have a constant wall thickness. The columns are one-piece tapered members with the largest diameter at the base. The beam normally consists of two tapered pieces that are joined with the largest diameter at mid-span. Beams of longer spans may consist of three pieces, with the middle one having a constant diameter. Figs. 1 and 2 show the structures that were tested as part of this study.

Unfortunately, the Specifications do not provide much information on the design of monotube structures. The absence of adequate design guidelines can be partly attributed to the sparsity of research and engineering data on the strength and behavior of such structures. The first major work in this area was a project conducted by Ehsani et al. (1985) in 1984 at the University of Arizona. It was found that the d/400 deflection criterion as inappropriate for monotube structures. Deflection deflections in excess of the d/400 limit were calculated, although the stresses associated with these deflections were well below the allowable levels.

The first monotube study was purely analytical and the accuracy of the results obtained is a function of the assumptions that were made to
model the structure. It is important to compare such theoretical results
with actual performance data for a real structure to verify the modeling,
as well as the responses that have been found. The latter should be
obtained from testing, preferably using a full-scale structure being
subjected to a variety of service conditions. With such test data and
correlations in hand, improved design guidelines can be developed.

**Scope**

Before the validity of any analytical study can be fully accepted, its
results should be compared to the actual behavior of the subject in ques-
tion. While the study by Ehsani, et al. (1985), provided detailed data on
the behavior of monotube structures, it lacked comparison with the per-
formance of an actual structure.

The study presented here was conducted in three parts. In the first,
two actual structures were modeled for computer analysis. Part two in-
volved field testing of the same two structures. By testing the structures
under service conditions, the true response was obtained. The final part
of the study was aimed at comparing and evaluating the results obtained
in the first two parts. Through this comparison, it was possible to judge
the validity of the computer model, as well as to determine whether
problem conditions such as resonance were developing. Details of the
analytical and experimental work that is reported in the following are

**Structural Response under Wind Loads**

For most sign structures, the only loads acting on the structure are
gravity and wind. The forces due to gravity are simply the self-weight
of the structure as well as the weight of the ice that may attach to the
structure during certain times of the year in many geographical areas.
The magnitude and effect of these loads on the structure are relatively
easy to determine.

In contrast to the gravity loads, which are static, wind loads are dy-
namic. The wind blowing on a monotube structure acts primarily in two
ways. The first is drag on the signs and tubes of the structure, which
is a static force. Due to the low coefficient of drag for cylindrical objects,
the drag on the tubular elements of a monotube structure is negligible.
An approximation of the drag on the signs is incorporated in the com-
putation of the static equivalent of the wind forces in the direction per-
pendicular to the plane of the structure.

The second action of the wind is a phenomenon known as vortex
shedding. Air flowing around an object will develop vortices in the wake
which will alternate from one side of the object to the other. These
alternating vortices cause the object to vibrate in the plane perpendicular
to the direction of the wind.

The magnitude of the vortex forces depends on three major factors,
namely the wind velocity, the diameter of the tubular elements of the
structure, and the viscosity of the air. While vortex shedding may occur
for all values of the Reynolds Number $R$ a deterministic solution can be
obtained in the range of $300 < R < 3 \times 10^6$ only. The frequency of the

sheding is a function the Strouhal Number $S$, which in turn is related
to the Reynolds Number (Fung 1955; Hoerner 1965).

To account for the random force amplitude, Weaver (1961) experi-
mentally determined that root mean square (rms) values of the force
coefficient, $C_L$, denoted as $C_L$, and obtained an expression for the vor-

text shedding forces as

$$ F(t) = \frac{1}{2} \rho V^2 A \omega C_L \sin \Omega t $$

where $\rho$ = density of dry air, $V$ = wind velocity, $A = \text{projected area of the cylinder}$, $\omega = \text{frequency of the vortex shedding}$, and $t = \text{time}$.

The determination of $\omega$ is critical in the use of this equation, and can
be determined from the equation (Fung 1955):

$$ \omega = \frac{S \sqrt{V}}{D} $$

where $S$ = Strouhal number, and $D$ = the cylinder diameter. For a com-
plete derivation of the vortex shedding force (see Ehsani, et al. 1985).

**Analysis of Monotube Structures**

To help in developing the design guidelines, as well as to provide data
for a detailed comparison with the results of full-scale testing, extensive
structural analyses were performed on the two structures that had been
selected for the field testing. The response of the models under static
and dynamic loading was determined, including details on stresses and
deflections.

For this study, the Arizona Department of Transportation provided
shop drawings and site plans for two sign structures. Both of these had
been designed in accordance with the AASHO Specifications (1975).
The structures had spans of 60 and 100 ft as shown in Figs. 1 and 2.
Detailed dimensions are shown in Figs. 3 and 4. It is noted that the 100-
ft structure used a three-segment beam where the center segment had
a constant diameter. The longer span structure was located at University
Drive and Haydenkam Expressway in Phoenix, Arizona; at the time, apart
from the freeways, the surrounding area was undeveloped. The shorter
span structure was located across North Oracle Road at Miracle Mile in
Tucson, Arizona; the surrounding area consisted primarily of one- and
two-story residential and commercial buildings.

The beam-to-column connection provides some moment resistance to
vertical loads and essentially free rotation under horizontal loads. A more
detailed discussion of the connection is given in by Ehsani, et al. (1985).
The details of the column base are such that it can reasonably be as-
sumed to be fixed.

For each structure, a finite element model was developed following
generally accepted guidelines. The elements in both structures were
modeled as beam elements with three translational and three rotational
degrees of freedom at each node. Hollow circular sections were used for
the beam and column elements, while an I-shape was used for the beam-
to-column connection elements. It was not possible to model the struc-

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TABLE 1.—Natural Frequencies for 60-ft and 100-ft Structures

<table>
<thead>
<tr>
<th>Mode (1)</th>
<th>60-ft Structure</th>
<th>100-ft Structure</th>
</tr>
</thead>
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<td>2D, cps (2)</td>
<td>3D, cps (3)</td>
</tr>
<tr>
<td>1</td>
<td>2.834</td>
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<td>5</td>
<td>31.760</td>
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<td>92.693</td>
<td>29.307</td>
</tr>
<tr>
<td>10</td>
<td>102.733</td>
<td>31.934</td>
</tr>
</tbody>
</table>

In selecting the nodal points for each structure, a node was placed at the actual attachment points of each traffic sign for signs wider than four ft, and at the center of the sign for signs four ft wide or less. In this manner, the mass of the signs would be applied at a node. A node was also placed at the midspan of the beam to be able to determine deflections and stresses at this point.

All loads were applied to the models as nodal loads. The magnitudes of these loads were computed as discussed previously. Each structure was analyzed for various combinations of static and dynamic loads for wind speeds between 2.5 mph and 20.0 mph, in increments of 2.5 mph. Additional runs were made for 22.5 mph and 23.2 mph for the 60 ft structure and 21.3 mph for the 100 ft structure. Above the largest wind speed for each structure, the vortex shedding becomes random, and the analytical solution is no longer applicable (Ehssani and Bjorhovde 1966).

In addition to the static and dynamic load analysis, the first ten two-dimensional (2D) and three-dimensional (3D) natural frequencies were also determined, assuming zero damping. These are given in Table 1.

Analytical Results.—For the wind speeds considered, the gravity loads due to the weight of the structure govern in all cases. The stresses at midspan are principally due to the weight of the structure. The magnitudes of the in-plane bending stresses are independent of the wind speed. For the 60-ft structure, the maximum stress was 5.11 ksf, which is about 15% of the yield stress of 34 ksf. The magnitude of the maximum stress for the 100-ft structure was 10.92 ksf, which is about 32% of the yield stress. The maximum normal stress at the column base for the 60-ft structure was 2.64 ksf, and for the 100-ft structure it was 3.25 ksf.

At the connection between the column and the beam, shear stress is the governing factor. For both structures, the finite element model showed some normal stress, but this is largely due to the way the joint was modeled rather than any actual stress. The shear stress at the joint was close to constant for both structures, and the gravity loads appear to be...
govern. For the 60-ft structure, the maximum shear stress was 2.64 ksf, and for the 100-ft structure, it was 4.53 ksf. Both are well below the shear yield stress of the steel, which is 19.4 ksf.

As has been seen, the stresses at the three critical locations on each structure are significantly below the representative yield values of the steel. However, neither structure was able to meet the $d^2/400$ deflection criterion. For the 60-ft structure, which has a sign depth of seven ft, the maximum allowable deflection according to Specifications (AASHTO 1975) is: $d^2/400 = 7^2/400 = 0.123$ ft = 1.48 in. For the 100-ft structure with a sign depth of 5 ft, the allowable deflection is: $d^2/400 = 5^2/400 = 0.0625$ ft = 0.75 in.

These compare to the computed deflections of 1.12 and 5.51 in. for the 60- and 100-ft structures, respectively. Since the stresses are very low in both cases, this illustrates why the $d^2/400$ deflection criterion is unrealistic for monohube structures.

**FIELD TESTING OF FULL-SCALE STRUCTURES**

The second phase of the study consisted of the testing of actual sign support structures under service conditions. This was accomplished by instrumenting the Phoenix and Tucson structures with electrical resistance strain gages, as well as an anemometer to determine wind velocity and direction. The data were used to determine the stresses and strains at a number of important locations in the structures, and subsequently to evaluate the correlation between theoretical and actual structural performance. The data collection for the Tucson structure was carried out during the months of February—April, and for the Phoenix structure during the months of May—July. Due to the high ambient temperatures in central and southern Arizona during the summer months, it was necessary to calibrate the instruments and the strain gage readings to eliminate the effects of high temperatures.

**Experimental Results.**—The portable data collection equipment was capable of recording the anemometer and strain gage approximately every 34 sec (Martin et al. 1985). The data acquisition unit first read and stored the wind speed and direction. Using these values, the magnitude of the wind component perpendicular to the plane of the structure was then computed and stored. It is noted that while the gage readings were accomplished quickly, most of the time that was needed was used to retrieve and store the data.

Due to the vibrating nature of the structures, the strain gage readings were not necessarily always made at peak. The readings might have been taken at any point in the cycle, and it was therefore determined that a statistical evaluation of the data was the only way in which logical explanations of the results could be provided.

The analysis was conducted for each gage for all wind speeds using increments of 1 mph. Each nominal wind speed covered a range of 0.5 mph on either side of the nominal value. Therefore, actual wind speed values exactly halfway between two nominal wind speed increments were rounded up to the higher value.

For the 60-ft structure, a total of 1,244 readings were made by each gage. For the 100-ft structure, 1,133 readings were taken per gage. For each nominal wind speed, the maximum positive and negative stresses were found, and the average stress and standard deviation were computed. The average positive and negative stresses along with their respective deviations were also determined.

The two locations of primary interest for each structure were the midspan of the beam and the base of the column. A stress envelope was determined for each of these by plotting the average stresses, along with points identifying values of ±3 standard deviations to either side of the average. This envelope included 99.5% of all possible stress levels, assuming that the readings are normally distributed. The envelopes for the midspan of the 60-ft and 100-ft structures are shown in Figs. 5 and 6, respectively.

It can be seen that the maximum values of the stress envelopes are well within the safe range. For the 100-ft span structure, the maximum value given by the envelope is 18 ksf, which is only 53% of the yield stress of 34 ksf. For the 60-ft structure, the maximum value given by the
envelope is 8.2 ksi, or 24% of the yield stress. Furthermore, it is emphasized that these values are extremes. The large number of tests that were made lend confidence to the statistical evaluations. Further data are likely to alter neither the averages nor the ±3 standard deviations to a significant degree. It is therefore clear that the low level of service load stress that was predicted by the analytical study has been substantiated.

It is interesting to note that both structures exhibit local maxima in the envelopes at wind speeds of approximately 2 mph, and again at 14–15 mph. The frequency of oscillations at 2 mph is well below any natural frequency for both structures. However, between wind speeds of 15 and 16 mph, both structures are near a natural frequency. For the 60-ft structure, the frequency corresponds to the third 3D mode at 3.26 cps. For the 100-ft structure, the mode is also the third 3D mode, at a frequency of 3.06 cps. It is believed that the maxima observed in the stress envelopes at this wind speed indicate that the structure is tending towards resonance at these points. However, due to the structural damping and the gusting of the wind, the resonance condition is not achieved. This is the same finding that was made in the original monotube study (Ehman, et al. 1985).

The bending stresses at the column bases are not as large as their midspan counterparts. The envelopes for the out-of-plane stresses for both structures are shown in Figs. 7 and 8. The maximum values are 7.7 ksi for the 60-ft structure and 17.0 ksi for the 100-ft structure. This demonstrates that the span length has a greater influence on the column base stresses than does the size (i.e., it is noted that the slope on the 60-ft structure were about twice as large as those on the 100-ft structure, but the latter has a span that is 67% longer). The column base stresses also exhibit local maxima at approximately the same wind speeds as were found for the beam.

The maximum stresses discussed so far were not the actual maximum stresses recorded, but the maximum values that are likely to occur given the statistical distribution of the data. The recorded stresses were less than those presented. For example, at 16 mph, the wind speed for which the stress envelope is the widest for the 60-ft structure, the maximum recorded stress was 7.3 ksi, as compared to the 8.7 ksi of the envelope. It is therefore clear that the use of a statistical approach has made it possible to include essentially all of the possible stress levels in the analysis. This also reflects the cyclic nature of the structural behavior, as well as the influence of the time lapse involved in the reading of the gages.

**Comparison of Analytical and Experimental Results**

The full-scale test results that have been obtained represent an important addition to the pool of data that previously provided only theoretical results on the response of monotube sign support structures. In the following, a detailed evaluation of the data will be given, affording comparisons between actual in-service behavior of the structures and the...
Theoretical studies that have been made. In addition, the results will also be used to examine and verify the design recommendations that were made earlier (Ehsani, et al. 1985). It is noted that this earlier study examined the response of a variety of monotube structures, including the effects of high wind, sign placement, span, and stiffness of the structural components.

For the two structures, the analytical study and the field testing both showed the point of maximum in-plane stress to be at the midspan of the beam. The results are given in Table 2. It is interesting to note that for higher wind speeds, the in-plane stresses predicted in the analytical study were consistently higher than those measured in the field. Although the differences are generally acceptable, some discrepancies are to be expected due to modeling and loading assumptions.

In the out-of-plane direction, the point of maximum stress is the column base for both structures, and the results are shown in Table 2. It is seen that the analytical model predicts lower stresses than those that were measured. This is most likely due to the method used to model the beam-to-column connection.

Generally, good correlation was obtained between the analytical and experimental results. This is further shown in Figs. 9 and 10, which compare the computed and measured live load stresses for the two structures. It should be remembered that Figs. 9 and 10 show only live load stresses. It was not possible to measure the dead load stresses in the field, since both structures had been erected earlier. To obtain an indication of the total stress level, the dead load stresses from the analytical study were added to the measured live load stresses. These results are shown in Figs. 11 and 12. It can be seen that for both structures, the stresses are well below the yield stress of 34 ksi and the applicable allowable stress.

It should be noted that while the stresses for the structures are in the acceptable range, the deflections are well beyond the currently prescribed limits. This further indicates that the $d^2/400$ criterion is not suitable for monotube structures. If these had been designed to limit deflections to the $d^2/400$ limit, they would be overdesigned and probably uneconomical. A new deflection criterion, such as the one recommended by Ehsani, et al. (1985), needs to be adopted for use with monotube structures.

**Summary, Conclusions, and Recommendations**

The purpose of this study was to gather data on the performance of monotube sign support structures under service conditions. Through the use of field testing and computer modeling, data were collected to determine the service load response characteristics of the monotube structures.

On the basis of the two full-scale structures that have been tested and analyzed, the following conclusions can be made:

1. The service load stresses can be accurately predicted by the use of finite element modeling. The computer models in this study correlate very well with the field measurements, as well as with earlier analytical studies.
2. Due to the correlation between past and present results, the recommendations that have been made previously are well-founded and should be considered for adoption. These recommendations include...
suggested methods of analysis, new performance criteria, and topics in need of further study.

3. The stress levels associated with the actual deflections are well below the magnitudes of the allowable stresses, even though the structures do not meet the $d^2/400$ deflection criterion.

4. The stress level at any point can be found by superimposing the stresses due to static loads and those due to dynamic loads.

5. The maximum in-plane stresses occur at midspan of the beam.

6. The maximum out-of-plane stresses occur at the column base.

7. Resonance did not occur in the field testing, even when vortices were shed at frequencies equal to the natural frequencies of the structures. This can be attributed to the inherent damping of the structure, as well as to the gusting nature of the wind.

8. Monotube structures of moderate to long spans ($>60$ ft) cannot meet the $d^2/400$ dead load deflection requirement of AASHTO. In most cases, it would prove to be very uneconomical to design such a structure to meet this requirement. It is recommended that the previously suggested deflection criterion should be adopted. This limits the dead load deflection-to-span ratio to a value of 0.015.

9. Since the maximum wind speeds were relatively low (23.2 mph), it is recommended that wind tunnel tests should be performed to evaluate the behavior of the structures under high wind conditions.

Acknowledgments

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Appendix I.—Metric Conversion Table

The following is provided for the conversion of units from U.S. Customary to S.I. Units:

1 in. = 25.4 mm
1 ft = 304.8 mm
1 ksi = 6.89 MPa
1 mph = 1.609 km/h

Appendix II.—References


