NEW DESIGN APPROACH TO LONG-SPAN OVERHEAD SIGN STRUCTURES

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The use of long-span sign structures of improved appearance is required to meet the higher safety and alignment standards for modern highways. Current specifications impose restrictive limitations on allowable dead-load deflections for such structures. The basis for these limitations involves the assumption that both the fundamental frequency of vibration of the structure and the frequency of vortex shedding are in harmony at a design wind speed of 80 mph.

A study was conducted to examine this assumption. The results show that it does not hold for the basic equations applying specific parameters from actual structural designs. An analogy is made with roadway lighting poles. A deflection-limiting relationship developed on the same basis as that developed from the current specifications shows that standard light-gage steel poles, as we know them, would not have been developed had such a limit on their deflection been required. Therefore, a consistent philosophy with respect to the design of both roadway lighting poles and overhead sign structures is desirable.

As a part of this study, designs for monotubular structures have been completed for clear spans up to 200 ft, sufficient to span 10 lanes of divided traffic with adequate safe outside shoulder clearance and capable of supporting as many as four full-size freeway signs. Also, several such structures involving spans as long as 156 feet have been built and have been in service for 2 to 3 years. The performance of these structures in winds of gust speeds as high as 65 mph has been excellent. It is concluded that more liberal allowable dead-load deflections are permissible for this type of structure. This new freedom in design makes possible simple, efficient, safe, and attractive long-span highway sign structures.

•A GREAT deal of progress has been made in recent years in improving safety features and aesthetic standards of modern highways. On controlled-access highways in particular, higher standards are being applied to geometric design and cross-sectional features. Research has been extended into almost every area that has application to highway engineering. The results have been impressive. Much work remains to be done, however.

The development of long-span sign structures to meet new safety and aesthetic standards has received little attention until recently. In an attempt to overcome this situation this study has been carried out to examine present design criteria and to explore possibilities for developing new types of long-span sign structures. The study revealed that deflection limitations set by existing design specifications (1) represent a most severe restraint on the selection of structural types and on the creation of aesthetic designs.

As a result of the study several long-span designs were evolved utilizing hollow thin-walled single tubular members. Designs have been completed for spans ranging up to 200 ft in length. Several have been built including spans as long as 156 ft. The shape of the sign structure was chosen to complement that of the single-davit curved light

Sponsored by Committee on Traffic Safety Barriers and Sign, Signal and Lighting Supports and presented at the 50th Annual Meeting.

poles used. Both poles and sign structure support columns are located at the same offset dimension from the edge of pavement. Both are finished in the same color of field paint. The result is seen as a simple, efficient, safe, and attractive highway sign structure. Symbols used in this paper are listed in an appendix.

CONSIDERATION OF SIMPLICITY, SAFETY, AND AESTHETICS

The need for overhead sign supports has been met in the past by the development of three-dimensional truss spans and frames. These have been made up of small member aluminum or steel angles, channels, plates, and tubes. Figure 1 shows an example of this type of installation. A column support has been located in the gore area and another in the median. These are familiar structures. They appear to have developed as a direct result of the application of the AASHO Specifications (1).

The truss configuration was, at one time, almost a universal choice for all but very short-span highway bridges. It became recognized that this type of structure was not aesthetically pleasing. The modern welded girder and prestressed concrete long-span structures were developed to replace the truss. The resulting structures were more slender and presented a solid profile. Deflection and span-depth-ratio limitations continue in most present day highway bridge design specifications as an impediment to further improvement. These rules are rather arbitrary and, while they may have served suitably in the past, it is now recognized that relaxation is possible by means of more liberal regulations (2). Similar changes can be applied to overhead sign structures resulting in greater simplicity and improved appearance. An example of where this has been done is shown in Figure 2.

New highway safety practices (3) require the elimination of hazardous fixed objects from the roadside. Where this is impractical, adequate protective barriers are to be used. Overhead sign supports are not to be located in the gore area. Such structures are required to span the ramp exit pavement as well as the through lanes. Where flexible barriers are used, the location of overhead sign supports in narrow medians is undesirable, and spans clearing the roadways for both directions of traffic are necessary. Under these circumstances spans as long as 200 ft may be required to clear as many as 10 lanes of divided traffic and exit pavement, with adequate safe outside shoulder clearance, and capable of supporting up to four full-size freeway signs. The use of long-span monotubular sign structures as reported herein, allows these new highway safety practices to be met (Fig. 3).

A NEW DESIGN PHILOSOPHY

Beginning in 1967, a review of the AASHO Specifications (1) was carried out in conjunction with the design of several monotubular pipe frames for overhead spans. A detailed structural analysis of the behavior of a typical roadway lighting pole under specified loadings was also completed.

Figure 1. "Old style" overhead sign structure.

Figure 2. "New style" overhead sign structure (85-ft span; 14-in. pipe diameter).

The possibility of using large-diameter, thin-walled, single pipe sections in sign structures was enhanced in 1966 with the publication of the Second Edition of the Guide to Design Criteria for Metal Compression Members (4). The new Guide contained greatly increased slenderness ratios for local buckling of circular tubular members. The Revised Edition of AASHO (5) recognizes this information by including new limits on pipe radius/wall thickness, slenderness ratios. However, both the original and revised editions of AASHO require a limit on dead-load deflection of $d^2/400$ (in ft).

Figure 3. Long span provides clear gore area and flexibility of structure location (140-ft span; 20-in. pipe diameter).

It was found that adherence to this limit was not practical using reasonable pipe sizes. For example, if a typical sign panel has a vertical dimension of 10 ft ($d = 10$ ft), the limiting dead-load deflection would be $10^2/400 = 0.25$ ft, or 3 in. This applies irrespective of span length and requires that very deep and stiff structures be used, especially in long spans. The calculated dead-load deflections for the pipe-structure designs were found to increase with span length. Values exceeding 3 in. were calculated for all spans over 100 ft.

The AASHO deflection relationship $\Delta_{max} = d^2/400$ is obtained by equating the frequency of vortex shedding from a sign panel, $f_V = SV/d$, to the fundamental frequency of a simple span beam of uniformly distributed mass and stiffness, $f_O = \pi/2\sqrt{EIg/wl^4}$. The similarity of the expression for dead-load deflection at midspan, $\Delta_{max} = 5wl^4/384\ EI$, to the inverse of that under the radical sign, has led to substitution and evaluation, with a wind velocity of 80 mph, to obtain the AASHO relationship. The procedure implies that at a wind speed of 80 mph both the frequency of vortex shedding and the fundamental frequency of the structure are resonant. Evaluation of the two equations separately will show that this is most unlikely.

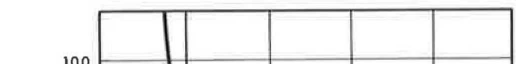


Figure 4. Critical wind-speed chart.

By substituting $f_O = f_V$, the so-called condition of resonance, and evaluating the equation for vortex shedding on each structure designed, a set of values of critical wind speed and span lengths was obtained. A curve representing these values is shown in Figure 4 which shows the critical wind speeds compared with the AASHO value of 80 mph. The suggestion is that the Specification grossly overestimates critical wind velocities for long-span sign structures.

In a similar manner, if the derived critical wind speeds for the condition of resonance are used for each span length, rather than 80 mph, a new set of deflection limits may be obtained. Figure 5 shows a curve representing these values and compares them to the AASHO limit of $d^2/400$. The observation made is that the Specification imposes too rigid a limitation on dead-load deflection for long-span sign structures.

The expression, $f_V = SV/d$, is also applicable to light poles if the diameter is used for the representative dimension d. The fundamental frequency of a cantilever beam of uniformly distributed mass and stiffness is given by $f_0 = 7/4\pi\sqrt{EIg/wl^4}$. The equation for deflection at the free end of a cantilever beam under uniform transverse loading is $\Delta_{max} = wl^4/8EI$. Substitition of this, as has been done in the AASHO Specifications, leads to a limit of $\Delta_{max} = d^2/440$ (in ft), where d is the pole diameter in ft.

Evaluation of this expression for a typical steel light pole erected horizontally (representative diameter = 0.5 ft), results in a dead-load deflection of a $(0.5)^2/440 = 0.00056$ ft, or just 0.0068 in. The pole, not including the heavy luminaire, erected in a horizontal position would deflect several inches. Presumably, light poles as presently known would not have evolved had they been required to meet such a severe limitation. An analysis has indicated dead-load deflections at the tip of the mast arm for the standard 36-foot-high, 11-gage steel pole in the normal vertical position, with luminaire in place to be 3.06 in. vertical and 4.61 in. horizontal. Under design wind load the tip deflection normal to the plane of the pole was found to be 21.10 in. A recent research report (6) on similar light poles is of interest in that it shows that maximum dynamic-response effects may not be associated with design wind velocities, but rather with some value substantially less and perhaps less than half the design velocity.

On the basis of this investigation it was concluded that the AASHO limitation on deflection was not representative for design of long-span monotubular sign supports. It was also felt that a consistent philosophy should be reflected in the design of both light poles and sign supports. Similar, it not identical, materials are used. Foundations and methods of fixing structures to them may also be similar. The conditions of service and consequences of failure are quite alike. Neither structure is a particularly costly element in comparison to the total cost of the facility.

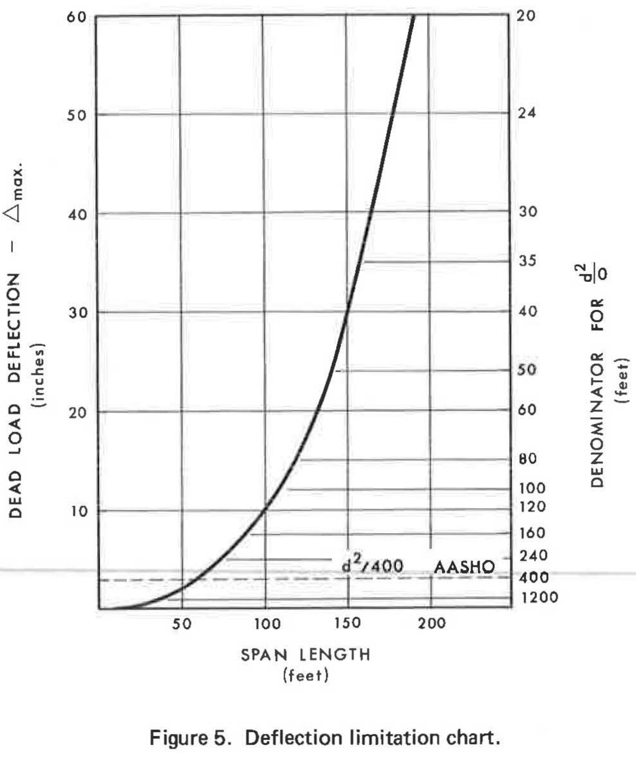


Figure 5. Deflection limitation chart.

APPLICATION AND OBSERVATION

The designs for several overhead pipe sign structures were completed in detail using AASHO group loadings and stress levels, and plans prepared for their construction. The spans varied in length from 85 to 200 ft. Frame analyses for loads and deflections were reviewed and found to be in excess of $d^2/400$, but very much less than the limits suggested in Figure 5. The calculated dead-load deflection at midspan was $7\frac{1}{2}$ in., and for a 200-ft span, $12\frac{1}{2}$ in. The calculated maximum horizontal deflection under design wind load (maximum gust speed 104 mph) for these structures is $24\frac{1}{2}$ in., $18\frac{1}{2}$ in., and $25\frac{1}{4}$., respectively. The 156-ft span is the longest built to date.

Details are shown on the accompanying Plan Sheets, Figures 6 and 7.

The effects of dynamic wind loading and structural damping on stress levels and deflections were not evaluated in the analysis. Essential to design that does not evaluate all stresses which may occur is the use of member and connection details leading to high-fatigue resistance. The pipe splices were detailed as flush butt, welds. High-strength, bolted field connections were located near the dead-load points of contraflexure. A

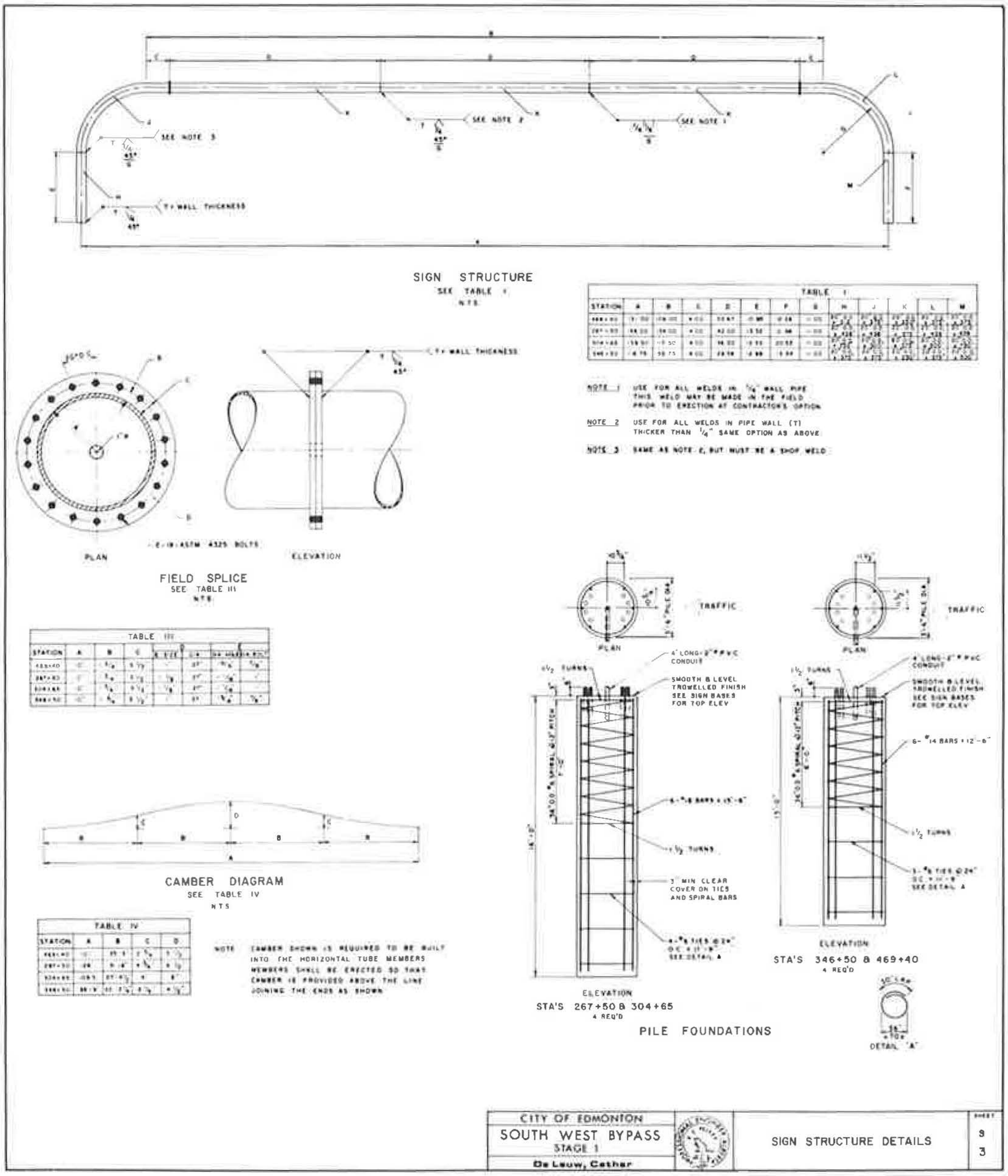
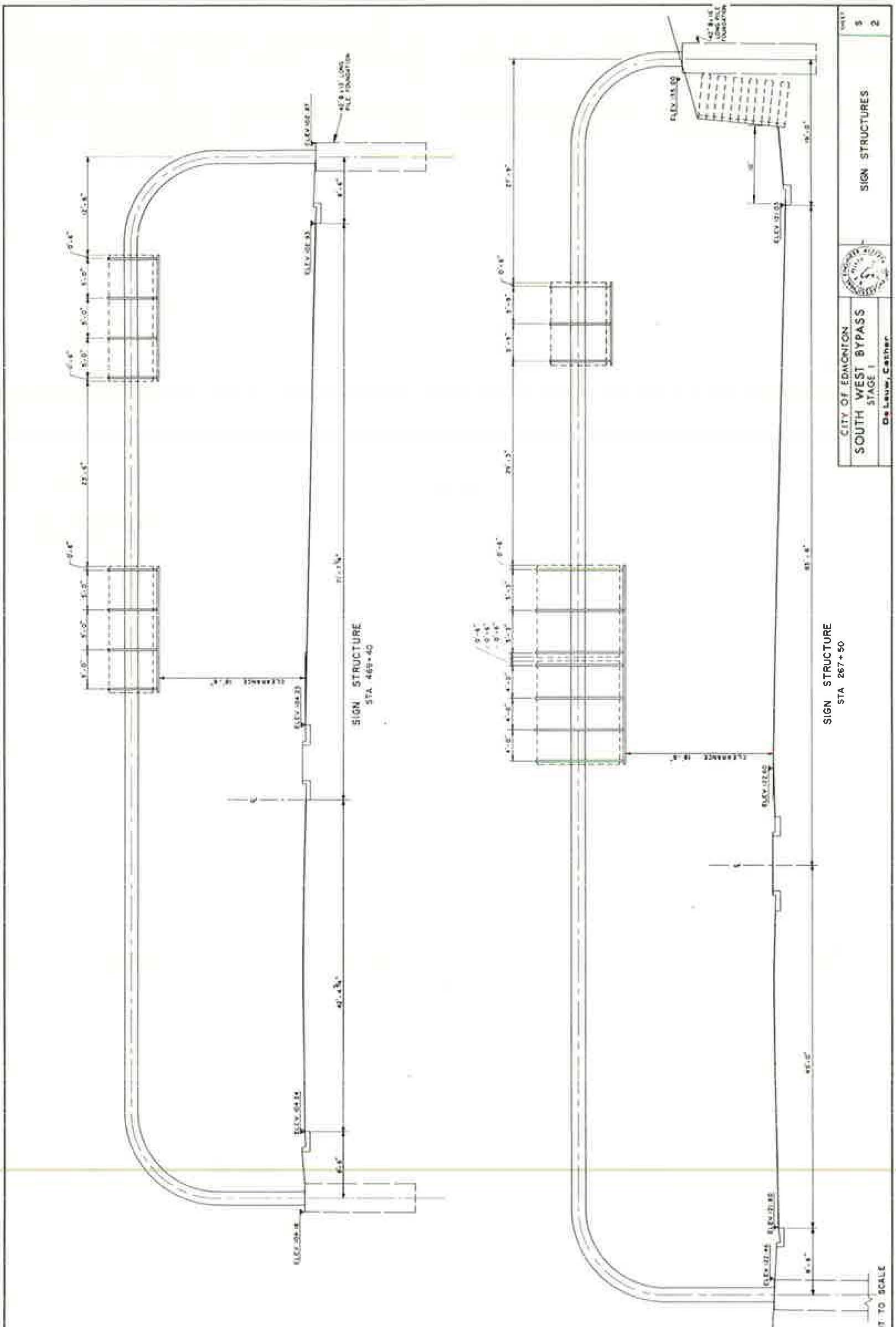


Figure 6.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SIGN STRUCTURES
NOT TO SCALE

Figure 7.

circular pipe section was selected for design because it was readily available and was similar in shape to the octagonal light poles used. A pipe was also a desirable section because, when used as a very slender flexural member, lateral-torsional buckling is not a problem. A pipe shape has a streamlined profile that results in lower wind-drag forces than for any other available section. The members are subject to high stresses due to loading in more than one plane. These stresses are effectively accommodated in the pipe section. The outside pipe diameter was uniform in any one structure. The pipe material conforms to ASTM Specification A53—Grade B. ASTM A36 material was used for all other shapes and plates.

The sign structures are supported by a single bored concrete pile beneath each vertical column. The AASHO Specifications (5) provide a chart intended for use in the design of embedded posts of this type. Unfortunately the loadings involved exceed the limits of the chart. A suitable method of design was found in the procedure suggested by Broms (7) for clay-type soils such as may occur at the site. A further paper by Broms provides a means for design in sandy and other cohesionless-type soils (8).

A structure combining sign panels over limited length and exposed main tubular member elsewhere, presents a highly complex system for vortex shedding and oscillation normal to the wind direction. Apparently on long circular cylindrical members a spanwise nonuniformity of vortex shedding also occurs over a wide range of Reynolds Numbers (9). It is likely that any mathematical analyses attempting to simulate actual behavior under these conditions would be suspect. More promising solutions are likely to be found in wind-tunnel tests using scale models and on field tests using full-scale structures.

Visual observations of all sign structures since installation have shown no unusual behavior. Vibrations have been nominal and noticeable only under close inspection. It is interesting to note that meteorological records, taken by the Canadian Department of Transport in the vicinity of the installations, show that since being erected in 1967 and 1968, wind-gust speeds in a direction approximately normal to the spans have reached a maximum value of a least 65 mph. Over the period of record available (nearly 30 years) the maximum recorded value of wind-gust speed is 82 mph. It is felt that this evidence establishes the acceptability of the aerodynamic performance of the structures.

Future development of long-span monotubular sign structures may benefit greatly from research presently being carried out under HRB sponsorship. Investigations are being continued on wind loads on louvered signs at the Texas Transportation Institute. An earlier report (10) has shown that wind-drag forces on nonsolid signs may be reduced 50 percent or more without impairing readability or target value. A significant recommendation is also made in favor of lower design wind speeds for highway sign loading. The continuation of this research, including testing of full-size installations, may show that a particular type and arrangement of louvers results in a "shroud effect" or "spoiler effect" preventing the formation and shedding of vortices as well as substantially reducing wind forces on sign panels. The results will be anticipated with enthusiasm since the AASHO Group 2 loading, combining dead-load and wind-load effects, governs the design of long-span sign structures.

Equally important research is being directed toward the adaptation of breakaway behavior to large supporting members of long-span overhead sign structures (11). Such members will presumably be adaptable to all overhead monotubular spans supported on three or more columns. In instances where support columns are insufficiently offset from the edge of pavement, their use will eliminate the need for protective barriers and the hazard they represent.

FABRICATION, ERECTION, AND COSTS

The sign-support structures were shop welded by manual procedures. All main pipe welds were full-penetration butt welds. Low-hydrogen electrodes of Class E7018 were specified for this purpose. Visual inspection of the joints were made after fit-up was complete, during the welding operation, and upon completion. No other form of weld inspection was performed. Some weld splices were made on site in order to reduce shipping lengths of members.

The 90-deg bends were manufactured by heat curving originally straight lengths of pipe. Experience with these bends has shown that wall thicknesses obtained from structural design may not be adequate for fabrication. For each pipe diameter and wall thickness there is a minimum radius of bend which can be successfully made. Smaller radii result in the development of buckles in the pipe wall on the inside of the bend. To avoid this condition the design thickness and radius of bend should be reviewed and an acceptable combination chosen to satisfy the relationship $t_{min} = 2.5\ d/R$.

Figure 8. Simple three-piece erection—fast bolting-up operation (120-ft span; 20-in. pipe diameter).

Erection of the sign supports was easily and quickly carried out. As shown in Figure 8, vertical members including the 90-deg bends were first erected on the foundation piles. The anchor bolts had previously been accurately located in concrete piles by means of precise surveys. The horizontal member was then hoisted into position and the bolted splice made. ASTM A325 high-strength bolts in $^7/_8$-in.-and 1-in.-diameter size were used. Field tightening was by the turn-of-the-nut method producing the required tension in the bolts. The entire erection operation for each sign structure was accomplished in less than one hour.

Painting for both shop and field coats was a simple operation because of the clean and continuous surface of the pipe. Internal surfaces were left unpainted except for those within reach of a handhole at the base of each leg. A 1-in.-radius hole in the splice plates provides for the electrical wiring to pass through the verticals and bend into the horizontal members. A small hole, field cut in the bottom of the horizontal member, leads the wiring to the luminaires. The concept of using an unpainted interior is similar to that employed with light poles and is felt to be quite acceptable for this type of structure. Experience with light poles has shown that metal oxidation is a problem only in the region of the base-to-pole connection. This part of the sign support is open to inspection and repainting if necessary.

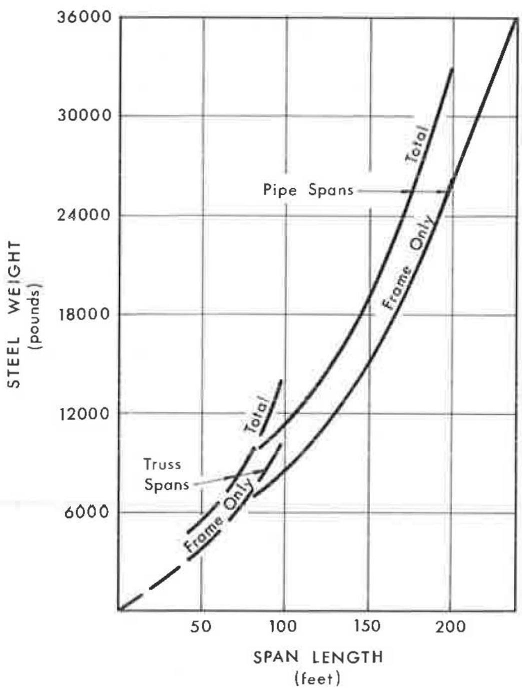

Figure 9. Steel weight chart.

The chart of steel weight plotted against span length shown in Figure 9 compares the pipe support structures to available information on truss span sign supports of similar design and details. The curves indicate that the weight of steel required in pipe supports is not excessive when compared to truss spans. Some advantages in terms of weight may be indicated in spans over 100 ft in length.

Included in the weight curves shown are the panel stiffeners and luminaire supports. In all cases these members were designed to support a temporary walkway and live-load forces as called for in the AASHO Specifications. A permanent walkway has not been installed as the intention is to carry out routine maintenance from a basket arm or snorkle truck operated from the shoulder area. For the infrequent occasions when more extended work or reaches exceeding the boom length of the truck are required, a portable walkway and light tubular railing will be clamped into place on the permanent supports to provide a safe work platform.

The construction of the sign support structures was carried out under contract. The contract included the construction of foundations, the supply, erection, and both shop and field painting of the steel frames, panel stiffeners and luminaire supports. The total contract price for the 85-ft-span pipe structures built during 1967 was $6,170.00 per span. The 1968 contract price of the four longer pipe span structures was $46,643.00 in total. Not included in the contract were items of supply and installation of sign panels, luminaires, and wiring.

CONCLUSIONS

The results of this study and the subsequent full-scale construction have shown the use of closed-section, single-member frames for long-span sign structures to be both economical and attractive. The service performance during the two and three years since installation has been excellent. The design of these frames would not be possible if the specified dead-load deflection limitation in AASHO had been met. Reliable new limitations have not been defined, but it has been demonstrated that existing limitations can be greatly relaxed.

The behavior of highway-sign support structures under dynamic wind loading appears highly complex, especially at the very high Reynolds Numbers involved. Further progress in evaluating the oscillating forces accompanying such loading and their effects on the support structures may be facilitated by wind tunnel model studies and field research on full-scale installations. In the meantime, monotubular frames of characteristics similar to those shown here may be used to achieve simplicity and safety in aesthetically pleasing structures for spans as long as 200 ft.

ACKNOWLEDGMENT

As a part of this study, use is made of information taken from several designs completed for the City of Edmonton, Alberta, by De Leuw, Cather and Company of Canada Limited. The author wishes to thank Mr. William E. Gillespie, Roadway Design Engineer, City of Edmonton, for permission to include this material.

The author also wishes to express his appreciation to Mr. Jack E. Leisch, Vice-President, De Leuw, Cather and Company of Canada Limited, and President of Jack E. Leisch and Associates, Evanston, Illinois, for helpful suggestions and encouragement during the preparation of this report.

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Appendix

SYMBOLS USED IN THIS PAPER

d = outside diameter of pole or pipe (feet), also sign panel height (feet);
E = modulus of elasticity (pounds/inch^2);
f_0 = fundamental frequency of vibrating system (cycles/second);
f_v = vortex shedding frequency (cycles/second);
g = acceleration due to gravity (386.4 inches/second);
I = moment of inertia (inches^4);
l = span length (inches);
R = centerline radius of pipe bend (feet);
S = Strouhal number (usually 0.2);
t_{min} = minimum wall thickness of pipe (inches);
V = wind velocity (feet/second);
w = weight of vibrating system per unit length (pounds/inch); and
Δ_{max} = maximum static (dead-load) deflection (inches).