

**NYS
DOT**

DESIGN SPECIFICATIONS

FOR

OVERHEAD SIGN STRUCTURES

CARRYING

VARIABLE MESSAGE SIGNS

October 1998

With 1999 Updates

INTRODUCTION

These specifications are based on the following documents:

1. Proposed AASHTO Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals
2. Research report titled "*Fatigue Related Wind Loads on Highway Support Structures*" by Kevin Johns and Robert Dexter.
3. NCHRP Report 411 titled "*Structural Supports for Highway Signs, Luminaires, and Traffic Signals*".
4. NCHRP Report 412 titled "*Fatigue - Resistant Design of Cantilevered Signal, Sign and Light Supports*".
5. Report titled "*Unanticipated Loading Causes Highway Sign Failure*" by Philip V. DeSantis, Ph.D. and Paul Haig, P.E.
6. Research report titled "*Wind Forces On Structures*" published in "*Transactions of the ASCE Volume 126, Part II 1961*".



In addition, it is also based on discussions with persons involved in developing state of the art specifications for designing overhead sign structures.

These specifications shall not be used to design single arm cantilever structures that support a variable message sign.

The basis for this specification is the "*Proposed AASHTO Specifications for Structural Supports for Highway, Signs, Luminaires and Traffic Signals*". It has been significantly "chopped up" and its approach to fatigue modified. We purposely did not change section and subsection numbers. The "old" section, numbers, etc. were used so that interested persons could easily refer to the original specifications for comparison.

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To:		New York State Department of <i>Transportation</i> ENGINEERING BULLETIN EB 99-063
Title: REVISIONS / ADDITIONS TO THE NYSDOT DESIGN SPECIFICATION FOR OVERHEAD SIGN STRUCTURES CARRYING VARIABLE MESSAGE SIGNS, OCTOBER 1998		
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Approved:  JAMES M. O'CONNELL Deputy Chief Engineer (Structures)		824.99 Date

ADMINISTRATIVE:

This Engineering Bulletin transmits revisions and additions to the NYSDOT Design Specifications for Overhead Sign Structures Carrying Variable Message Signs. It is effective immediately.

This Engineering Bulletin Supersedes Update U 98-01, issued 12/5/98 by the Sign Structure Management Unit.. The revisions described in U 98-01 are incorporated in the pages transmitted with this Bulletin.

TRANSMITTED MATERIALS:

This Engineering Bulletin transmits new pages, 3-1, 3-2, 3-4, 11-i, 11-2, 11-4, 11-4a, 11-4b, 11-5, and 11-6, dated 6/22/99. Please replace pages 3-1, 3-2, 3-4, 11-i, 11-2, 11-4, 11-5, and 11-6, in the subject Specification, with the pages transmitted by this Bulletin. (Revisions are annotated with a vertical side-bar located in the right margin of the revised text.)

SUMMARY OF CHANGES:

- 1) Pages 3-2, 3-4, and 11-5 incorporate the changes from U 98-01.
- 2) On Page 3-1; Section 3.2 DEFINITIONS, the definition for *Basic Wind Speed, V*, was revised to make it consistent with the 1994 AASHTO Isotach charts, that are being used in the NYSDOT Specification at this time.
- 3) Section 11.7.1 Galloping, was added. Pages 11-i, 11-2, 11-4 to 11-5 were revised to include the requirement to consider Galloping effects in the fatigue analysis of cantilever structures, by applying an equivalent static load independently from the Gust Loads.
- 4) On Page 11-6; Section 11.7.4 Truck-Induced Gust, in the second sentence of the second paragraph, the phrase, "...outer 3.7m (12 ft) of the sign.", was deleted and replaced with language to clarify where the upward load should be placed along the length

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1.1 SCOPE

The provisions of these NYSDOT Design Specifications For Designing Overhead Structures Supporting Variable Message Signs, after herein referred to as the Specifications, are applicable to the structural design of supports for variable message sign structures. The Specifications are intended to serve as a standard and guide for the design of these types of supports.

These Specifications are not intended to supplant proper training or the exercise of judgment by the Designer, and includes only the minimum requirements necessary to provide for public safety. The Owner or the Designer may require the design and quality of materials and construction to be higher than the minimum requirements.

The commentary discusses some provisions of the Specifications with emphasis given to the explanation of new or revised provisions that may be unfamiliar to users of the Specifications. The commentary is not intended to provide a complete historical background concerning the development of this and previous Specifications, nor is it intended to provide a detailed summary of the studies and research data reviewed in formulating the provisions of the Specifications. However, references to some of the research data are provided for those who wish to study the background material in depth.

The commentary directs attention to other documents that provide suggestions for carrying out the requirements and intent of these Specifications. However, those documents and the commentary are not intended to be a part of the Specifications.

1.2 DEFINITIONS

Arm -- a cantilevered support, horizontal or sloped.

Bridge support -- also known as span-type support; a horizontal or sloped member or truss supported by at least two vertical supports.

Cantilever -- a support, either horizontal or vertical, supported at one end only.

Designer -- person responsible for design of the structural support.

Mast arm -- a supporting arm designed to hold a sign, in an approximately horizontal position.

Overhead sign -- a sign suspended above the roadway.

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Owner -- person or agency having jurisdiction for the design, construction, and maintenance of the structural support.

Pole -- a vertical support that is long, relatively slender, and generally rounded or multi-sided.

Pole-top -- a descriptive term indicating that an attachment is mounted at the top of a structural support, usually pertaining to one luminaire or traffic signal mounted at the top of a pole.

Sign -- a device conveying a specific message by means of words or symbols, erected for the purpose of regulating, warning, or guiding traffic.

Truss -- a structural support, usually vertical or horizontal, composed of framework that is often arranged in triangles.

1.3 APPLICABLE SPECIFICATIONS

The following specification documents may be referenced for additional information on design, materials, fabrication, and construction:

- a). AASHTO *Standard Specifications for Highway Bridges* (AASHTO, 1996).
- b). AASHTO LRFD *Bridge Design Specifications* (AASHTO, 1994).
- c). *Standard Specifications for Transportation Materials and Methods of Sampling and Testing* (AASHTO, 1996).
- d). *Book of ASTM Standards* (American Society for Testing and Materials).

1.4. TYPES OF STRUCTURAL SUPPORTS

Support structures for signs.

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Section 2 - General Features of Design

SPECIFICATIONS

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2.1 SCOPE

Minimum requirements are provided or referenced for aesthetics, clearances, constructibility, inspectability, and maintainability of structural supports. Guidelines for determining vertical and lateral clearances, use of breakaway supports, use of guardrails, illumination of the roadway, sizes of signs, illumination and reflectorization of signs, and maintenance are found in the following references:

- a). *A Policy on Geometric Design of Highways and Streets.*
- b). *AASHTO Roadside Design Guide*
- c). *AASHTO Maintenance Manual*

This section is intended to provide the Designer with information and references to determine the configuration, overall dimensions, and location of structural supports for highway signs. The material in this section is broad in nature. No attempt has been made to establish rigid criteria in such areas as vertical heights. This section provides references and considerations for the different aspects of design that should be considered in the preliminary stages of a project. In addition to the requirements provided within this section, many Owners have their own requirements.

2.2 DEFINITIONS

Barrier -- a longitudinal traffic barrier, usually rigid, used to shield roadside obstacles or non-traversable terrain features. It may occasionally be used to protect pedestrians from vehicle traffic.

Breakaway -- a design feature that allows a sign, luminaire, or pole-top mounted traffic signal support to yield, fracture, or separate near ground level upon impact.

Clear zone -- the total roadside border area, starting at the edge of the traveled way, available for unobstructed use by errant vehicles.

Clearance -- horizontal or vertical dimension to an obstruction.

Curb -- a vertical or sloping surface, generally along and defining the edge of a roadway or roadway shoulder.

Gore -- the center area immediately past the point where two roadways divide at an acute angle; usually where a ramp leaves a roadway.

Guardrail -- a type of longitudinal traffic barrier, usually flexible.

Mounting height -- minimum vertical distance to the bottom of a sign or traffic signal, or to the center of gravity of a luminaire, relative to the pavement surface.

Pedestal pole -- a relatively short pole supporting a traffic signal head attached directly to the pole.

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Roadside -- the area between the shoulder edge and the right-of-way limits, or the area between roadways of a divided highway.

2.3 AESTHETICS

The structural support should complement its surroundings, be graceful yet functional in form, and present an appearance of adequate strength. The support should have a pleasing appearance that is consistent with the aesthetic effect of the highway's other physical features. Supports should have clean, simple lines, which will present minimum hazard to motorists.

The appearance of ordinary structural supports should consider aesthetics and function. Combination poles, which serve multiple functions for lighting, traffic control, and electrical power, should be taken under consideration to reduce the number of different poles along the highway.

Structural supports should be designed and located so as not to distract the motorist's attention or obstruct the view of the highway. Supports should be placed so they do not obstruct the view of other signs or important roadway features. The effect that signing or lighting installations have on the surrounding environment should be evaluated.

2.4 FUNCTIONAL REQUIREMENTS

2.4.2 Structural Supports for Signs

2.4.2.1 Vertical Clearances

Over head sign structures shall provide a vertical clearance over the entire width of the pavement and shoulders of at least 300 mm (1.0 ft.) greater than the required minimum vertical clearance of overpass structures on the route. The vertical clearance shall be either in conformance with *A Policy on Geometric Design of Highways and Streets* for the Functional Classification of the Highway, or exceptions thereto shall be justified.

The minimum clearance should include an allowance for possible future overlays. The additional 300 mm (1.0 ft) vertical clearance is required so that high vehicles will strike the stronger overpass structures first, thereby lessening the chance of major collision damage to the structurally weaker overhead sign support structures.

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Additional guidance on vertical clearances may be found in the *Manual on Uniform Traffic Control Devices for Streets and Highways*.

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2.5 ROADSIDE REQUIREMENTS FOR STRUCTURAL SUPPORTS

Consideration shall be given to safe passage of vehicles adjacent to or under a structural support. The hazard to errant vehicles within the clear zone distance, defined in Article 2.5.1, should be minimized by locating obstacles a safe distance away from the travel lanes. Roadside requirements and location of structural supports for highway signs, should generally adhere to the principles given in Articles 2.5.1 through 2.5.9.

Where possible, a single support should be used for dual purposes (i.e., signals and lighting). Consideration should also be given to locating luminaire supports to minimize the necessity of encroaching on the traveled way during routine maintenance.

2.5.1 Clear Zone Distance

Structural supports should be located in conformance with the clear zone concept as contained in Chapter 3 "*Roadside Topography and Drainage Features*" of the AASHTO Roadside Design Guide, or other clear zone policy accepted by the FHWA. Where the practical limits of structure costs, type of structures, volume and design speed of through traffic, and structure arrangement make conformance with the *Roadside Design Guide* impractical, the structural support should be provided with a breakaway device or protected by the use of guardrail or other barrier.

The clear zone is the roadside border area beyond the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry.

Suggested minimum clear zone distance are provided in the *AASHTO Roadside Design Guide* (AASHTO, 1996) and are dependent on average daily traffic, slope of roadside, and design vehicle speed. Additional discussions of clear zone distances and lateral placement of structural support may be found in the *Manual on Uniform Traffic Control Devices* (FHWA, 1993) and *A Policy on Geometric Design of Highway and Streets* (AASHTO, 1994).

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2.5.3 Guardrails and Other Barriers

The location of roadside sign and luminaire supports behind a guardrail should provide clearance between the back of the rail and the face of the support to insure that the rail will deflect properly when struck by a vehicle. Continuity of the railing on rigid highway structures should not be interrupted by sign or luminaire supports.

Guardrails are provided to protect fixed objects, such as overhead sign supports. The *AASHTO Roadside Design Guide* provides guidelines for the provision of roadside barriers for fixed objects.

The clearance between the back of the barrier and the face of the support may vary, depending on type of barrier system utilized. The *AASHTO Roadside Design Guide* may be used to determine the proper clearance.

The clearance between the edge of a sign panel, which could present a hazard if struck, and the back of a barrier should also take into consideration the deflection of the rail. The edge of a sign shall not extend inside the face of railing.

2.5.5 Overhead Sign Supports

Overhead sign structures should be placed outside the clear zone distance; otherwise, they should be protected with a proper guardrail or other barrier.

Overhead sign and high-level lighting supports are considered fixed-base support systems that do not yield or break away upon impact. The large mass of these support systems and the potential safety consequences of the system falling to the ground necessitate a fixed-based design. Fixed-base systems are rigid obstacles and should not be used in the clear zone area unless shielded by a barrier. In some cases, it may be cost effective to place overhead sign supports outside the clear zone with no barrier protection when the added cost of the greater span structure is compared to the long-term costs of guardrail and vegetation maintenance. Some structures can sometimes be located in combination with traffic barriers protecting other hazards such as culverts, bridge ends, and embankments.

2.5.7. Gores

Where obstruction in the gore is unavoidable within the clear zone, protection

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should be provided by an adequate crash cushion or the structure should be provided with a breakaway device.

2.5.8 Urban Areas

For sign structures located in working urban areas, the minimum lateral clearance from a barrier curb to the support is 500 mm (20 in). Where no curb exists, the horizontal clearance to the support should be as much as reasonably possible.

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The 500 mm offset is not an urban "clear zone," rather it was established to avoid interference with truck mirrors, open doors, etc. The preferred location of support structures is on the "house side" of the sidewalk.

2.6 CORRELATION OF STRUCTURAL SUPPORT DESIGN WITH ROADWAY AND BRIDGE DESIGN

2.6.1 Signs

Sign panels may be supported on existing or proposed grade separation structures. In these cases, the minimum vertical clearance requirements for overhead signs do not apply. A specifically designed frame shall be required to attach the sign panel to the existing structure. The overhead sign should be located as near to the most advantageous position for traffic operation as possible, but where structurally adequate support details can be provided.

Sign installation on grade separation structures is generally acceptable aesthetically when the sign panels do not extend below the girders or above the railing. The sign panel should be placed slightly above the minimum vertical clearance specified for the grade separation structure. Close liaison between Bridge and Traffic Engineers is essential for signs mounted on grade separation structures.

The placement of overhead signs must be considered in the preliminary design stages to avoid possibly restricting the driver's view of sign messages by other signs or structures. Signing is an integral part of the highway environment and must be developed along with the roadway and bridge designs.

2.7 MAINTENANCE

A regular maintenance program should be established that reflects periodic inspection, maintenance, and repair of structural supports.

Provisions to perform maintenance and inspection of structural supports should include the following:

- a). Inspection ladders, walkways, and

The AASHTO Maintenance Manual (AASHTO, 1987) includes information for scheduling, inspecting, and maintaining structures.

All structural supports should be inspected for the effects of corrosion and fatigue. Some connections, such as the slip-fit breakaway connection on some roadside sign

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covered access holes, if necessary, where other means of inspection is not practical.

- b). Means to perform inspection, maintenance, and repair of overhead sign structural supports, without obstructing the traveled way on all except low volume highways.

In all cases, safety from vandalism and the provision of an attractive hazard shall be considered.

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supports, may require adjustment in fasteners to a proper torque to work properly. Steel poles and brackets that are not galvanized should be painted as frequently as required by local conditions.

Maintenance and repair of overhead sign structural supports may be done either by special maintenance equipment operated from the shoulder or by construction of a maintenance walkway on the sign structural support.

2.8 REFERENCES

- A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, 1994.
- AASHTO Maintenance Manual*, American Association of State Highway and Transportation Officials, Washington, D.C., 1987.
- Manual on Uniform Traffic Control Devices*, 1988 Edition of MUTCD, Revision 3, U.S. Department of Transportation, Federal Highway Administration, 1993.
- Roadside Design Guide*, American Association of State Highway and Transportation Officials, Washington, DC, 1996.

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SECTION 3 LOADS

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3.1 SCOPE

This section specifies minimum requirements for loads and forces, the limits of their application, load factors, and load combinations that are used for the design or structural evaluation of highway sign structures.

The section includes specifications for the dead load, live load, ice load, and wind load.

Where different mean recurrence intervals may be used in specifying the loads, the selection of the proper mean recurrence interval is the responsibility of the Owner.

3.2 DEFINITIONS

Basic wind speed, V -- the "Fastest-mile wind speed" (in kph or mph) at 10 m (32.8 ft) above the ground associated with a 50-year mean recurrence interval.

Design wind pressure, P_z -- the pressure exerted on a member or attachment by the effects of wind. The pressure is calculated using appropriate design values for all variables in the wind pressure formula.

Drag coefficient, C_d -- a dimensionless coefficient that corrects velocity pressure for the effects of the geometry of the structural element and the Reynolds number.

Fastest-mile wind speed -- the peak wind speed averaged over 1.609 km (1 mile) of wind passing a point.

Gust coefficient factor, G -- a dimensionless coefficient that corrects the wind pressure to account for the dynamic interaction of the wind and the structure.

Height and exposure factor, K_z -- a dimensionless coefficient that corrects the magnitude of wind pressure referenced to a height above the ground of 10 m (32.8 ft) for the variation of wind speed with height.

Importance factor, I_r -- a factor that converts wind pressures associated with a 50-year mean recurrence interval to wind pressures associated with other mean recurrence intervals.

Mean recurrence interval, r -- the inverse of the probability of occurrence of a specific event in a one year period. (If an event has a 0.02 probability of occurrence in one year, it has a mean recurrence interval of 50 yr. = $1/0.02$.)

Service life -- the period of time that the structure is expected to be in operation.

Solidity -- the solid elevation area divided by the total enclosed elevation area for a truss.

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3.3 NOTATION

b	= overall width (m, ft)
BL	= basic load
C_d	= drag coefficient, as defined in Table 3-5 (Article 3.8.6)
C_{ad}	= drag coefficient for round cylinder of diameter D
C_{ad}	= drag coefficient for round cylinder of diameter d_o
d	= depth (diameter) of member (m, ft)
D	= major diameter of ellipse (m, ft)
DL	= dead load (N, lb)
d_o	= minor diameter of ellipse (m, ft)
G	= gust coefficient factor, which is taken to be 1.3 (Article 3.8.5)
I_{ce}	= ice load (N, lb)
K_z	= height and exposure factor, as presented in Table 3-6 (Article 3.8.4)
n_c	= normal component of wind force (N, lb)
P_z	= design wind pressure (Pa, psf)
r	= mean recurrence interval in year interval
r	= ratio of corner radius to inscribed circle
t_c	= transverse component of wind force (N, lb)
V	= basic wind speed, (kmph, mph)
W	= wind load (N, lb)
W_h	= wind load on exposed horizontal support (N, lb)
W_p	= wind load on sign panel
W_s	= wind load on sign (N, lb)
W_v	= wind load on exposed vertical supports (N, lb)
Z	= height at which wind pressure is calculated (m, ft)
Z_g	= constant for calculating the height and exposure factor
a	= constant for calculating the height and exposure factor

3.4 GROUP LOAD COMBINATIONS

The loads described in Articles 3.5 through 3.8 shall be combined into appropriate group load combinations as stipulated in Table 3-1. Each part of the structure shall be proportioned for the combination producing the maximum effect, using allowable stresses increased as indicated for the material and group load.

Table 3-1 Group Load Combinations has been modified from the 1994 edition of the Specifications. The percent increase for group load combinations II and III has changed from 40% to 33%. The main reason for the change is to ensure consistency with major codes in the United States.

An additional load combination for fatigue has been added based on the work of NCHRP Project 10-38.

The intent of the Specifications is to

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provide an adequate margin of safety against failure. For example, the minimum safety factors for bending for a steel tubular section are approximately 1.92 for Group I loading and 1.45 for Group II and Group III loading. The safety factor may vary somewhat depending upon the material and cross section used; however, consideration has been given to ensure the equity in safety factors among different materials covered by these Specifications. Some materials and structural shapes may warrant a higher safety factor because of inherent variability in the material or the manufacturing process.

3.5 DEAD LOAD

The dead load shall consist of the weight of the structural support and signs. Temporary loads during maintenance shall also be considered as part of the dead loads. The points of application of the weights of the individual items shall be their respective centers of gravity.

Dead Load is to include all permanently attached fixtures, including hoisting and walkways provided for servicing of luminaires or signs.

3.6 LIVE LOAD

A live load consisting of a single load of 2200 N (550 lb) distributed over 0.6m (2.0 ft.) transversely to the member shall be used for designing members for walkways and service platforms.

The specified live load represents the weight of a man and equipment during servicing of the structure. The load need not be applied to the structural support. Only the members of walkways and service platforms are designed for the live load. Any structural member designed for the group loadings in Article 3.4 will be adequately proportioned for this temporary live load application.

3.7 ICE LOAD

Ice load shall be a load of 145 Pa (3.0 psf) applied around the surfaces of the structural supports, traffic signals, horizontal supports, and luminaires, but shall be considered only on one face of sign panels. In addition, the ice load shall be applied to the top surface of a VMS.

The ice loading is applicable to those areas shown in Figure 3-1. It is based on a 15 mm (0.60 in) radial thickness of ice at a unit weight of 960 kg/m³ (60 pcf) applied uniformly over the exposed surface of the member.

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Figure 3-1 shows the locations within the contiguous United States where an ice load should be considered.

More or less severe ice loads may be used provided historical ice accretion data is available for the region of interest.

3.7.1 Snow Load

Snow load shall be a load of 140 Pa (40 psf) applied to the top surface of a VMS.

More or less severe snow loads may be used provided historical snow assertion data is available for the region of interest.

3.8 WIND LOAD

Wind load shall be the pressure of the wind acting horizontally on the horizontal and vertical supports and signs computed in accordance to Articles 3.8.1 through 3.8.6.

3.8.1. Wind Pressure Formula

The design wind pressure shall be computed using the following formula:

$$P_z = 0.0473K_z (1.3V)^2 C_d \quad (Pa)$$

$$P_z = 0.00256K_z (1.3V)^2 C_d \quad (psf)$$

Eq. 3-1

3.8.2. Basic Wind Speed

The basic wind speed V used in the determination of the design wind pressure shall be as given in AASHTO's "Standard Specification For Structural Supports For Signs, Luminaires and Traffic Signals" 1994, Figure 1.2.4C page 13.

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3.8.4. Height and Exposure Factor K_z

The height and exposure factor K_z shall be either determined from Table 3-6 or calculated using Eq. C-3-1 in the commentary.

K_z is a height and exposure factor that varies with height above the ground depending on the local exposure conditions. The variation is caused by the frictional drag offered by various types of terrain. Since 1982, widely accepted wind design procedures have involved the use of four different terrain exposure conditions, which are designated as exposures A, B, C, and D (ANSI, 1982). The conditions associated with these four different exposure conditions are defined by ANSI/ASCE 7-95. For a specified set of conditions, the wind pressures associated with the different exposures increase as the exposure conditions progress from A to D, with exposure A resulting in the least pressure and exposure D resulting in the greatest pressure. Exposure C has been adopted for use in this Specification as it provides a conservative approach for the design of structural supports. It represents open terrain with scattered obstructions.

Once the terrain exposure conditions are established, the height and exposure factor, K_z , is calculated using the following relationship that is presented in ANSI/ASCE 7-95:

$$K_z = 2.01 \left[\frac{Z}{Z_g} \right]^{2/\alpha}$$

Eq. C-3-1

where z is height above the ground or 4.57 m (15 ft) whichever is greater, and z_g and α are constants that vary with the exposure condition (1996). Based on information presented in ANSI/ASCE 7-95, α should be taken to be 9.5 and z_g should be taken to be 274.3 m (900 ft) for exposure C. These values are for three-second gust wind speeds and are different from similar constants that have been used for fastest-mile wind speeds. Table 3-6 presents the variation of the height and exposure factor, K_z , as a function of height based on the above relation.

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3.8.5. Gust Coefficient Factor G

The gust coefficient factor G shall be taken as 1.3.

3.8.6. Drag Coefficients C_d

Wind drag coefficient C_d shall be determined from Table 1.2.5C "Wind Drag Coefficients (C_d)" in ASHTO "Standard Specifications For Structural Supports For Highway Signs, Luminaires and Traffic Signals - 1994, except that the drag coefficient for a variable message sign shall be 2.

A method of determining C_d values for cylinders, dodecagons, and elliptical shapes, by which varying diameters and Reynolds numbers may be taken into account, is detailed in the 1994 edition of the Specifications.

The equation for C_d for an elliptical member with the narrow side facing the wind was empirically derived to fit wind tunnel test data.

The C_d for VMS's came from the research report titled "*Wind Forces on Structures*" published in "*Transactions of the ASCE Volume 126, Part II 1961*". Specifically, it came from "TABLE 2. - DRAG COEFFICIENTS", on page 1145 of that report. A VMS is a paralleloiped.

Values of C_d for octagonal shapes vary with wind velocity and orientation according to tests, but a value of 1.20 shows to be conservative over a wide range. Values of C_d for square tubing commonly used in lighting structures has been included in the Specifications.

When three members are used to form a triangular truss, the wind load shall be applied to all of the members. Even though all of the members are not in the same plane of reference, they may be seen in a normal elevation.

3.9 DESIGN WIND LOADS ON STRUCTURES

3.9.1. Load Application

The wind loads acting horizontally on a structure shall be determined by the areas of horizontal and vertical supports, and signs, and shall be applied to the surface area as viewed in normal elevation.

The effective projected area is the actual area multiplied by the appropriate drag coefficient.

3.9.2. Design Loads for Horizontal Supports

Rigid horizontal supports of sign structures (Cantilevered or bridge type) shall be designed for wind loads, W_n and W_p , applied normal to the support at the centers of pressure of the respective areas.

3.9.3. Design Loads for Vertical Supports

The vertical supports for sign support structures shall be designed for the effects of wind from any direction. An acceptable method to design for the effects of wind from any direction is by applying the following two load cases of normal and transverse wind loads acting simultaneously. This method is applicable where all signs on the structure are approximately in one plane and is not applicable for structures with arms in two or more planes.

Load Case	Normal Component (n_s)	Transverse Component (t_s)
1	1.0 (BL)	0.2 (BL)
2	0.6 (BL)	0.3 (BL)

The basic load (BL) for the structures with rigid horizontal supports shall be the effects from the wind loads, W_v , W_p and W_n , applied at the centers of the pressure of the respective areas of the structures and normal to

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the sign faces. The transverse components may be distributed in proportion to the relative lateral stiffness and conditions of restraint of the supports.

3.9.4. Unsymmetrical Wind Loading

To allow for unsymmetrical wind loading use the loading conditions stipulated in Article 3.9.4.1.

3.9.4.1. Overhead Cantilevered Supports

For vertical supports with balanced double cantilevers (i.e., equal length arms), the normal wind load shall be applied to the one arm only, neglecting the force on the other arm. For unbalanced double cantilevers, the normal wind loads shall be applied to the longer arm only.

When vertical supports have more than two arms, and the arms are mounted opposite or at diverging angles from one another, the wind load shall be applied to one arm when balanced or the longer arm when unbalanced.

3.10 REFERENCES

"Wind Forces on Structures," by the Task Committee on Wind Forces of the Committee on Loads and Stresses of the Structural Division, *Transactions*, ASCE, Part II, Vol. 126, 1961.

Section 3 - Loads

Table 3-1. Group Load Combinations

Group Load	Load Combination	Percent of Allowable Stress (see Note 1)
I	DL	100
II	$DL + W$	133
III	$DL + \text{Ice} + \text{Snow} + 1/2(W)$ (See Note 2)	133
IV	Fatigue	(see Note 3)

1. No load reduction factors shall be applied in conjunction with these increased allowable stresses.
2. W shall be computed on the basis of the wind pressure formula. A minimum value of 1200 Pa (25 psf) shall be used for W in Group Load III.
3. See Section 11 for fatigue loads and stress range limits.
4. See Article 3.6 regarding application of live load.

Section 3 - Loads

Table 3-6. Height and Exposure Factor, K_z

z , Height, m(ft)	K_z
5.0 (16.4) or less	0.87
7.5 (24.5)	0.94
10.0 (32.8)	1.00
12.5 (41.0)	1.05
15.0 (49.2)	1.09
17.5 (57.4)	1.13
20.0 (65.6)	1.16
22.5 (73.8)	1.19
25.0 (82.0)	1.21
27.5 (90.2)	1.24
30.0 (98.4)	1.26
35.0 (114.8)	1.30
40.0 (131.2)	1.34
45.0 (147.6)	1.37
50.0 (164.0)	1.40
55.0 (180.5)	1.43
60.0 (196.9)	1.46
70.0 (229.7)	1.51
80.0 (262.5)	1.55
90.0 (295.3)	1.59
100.0 (328.1)	1.63

NOTE: Height and exposure factor may be interpolated.

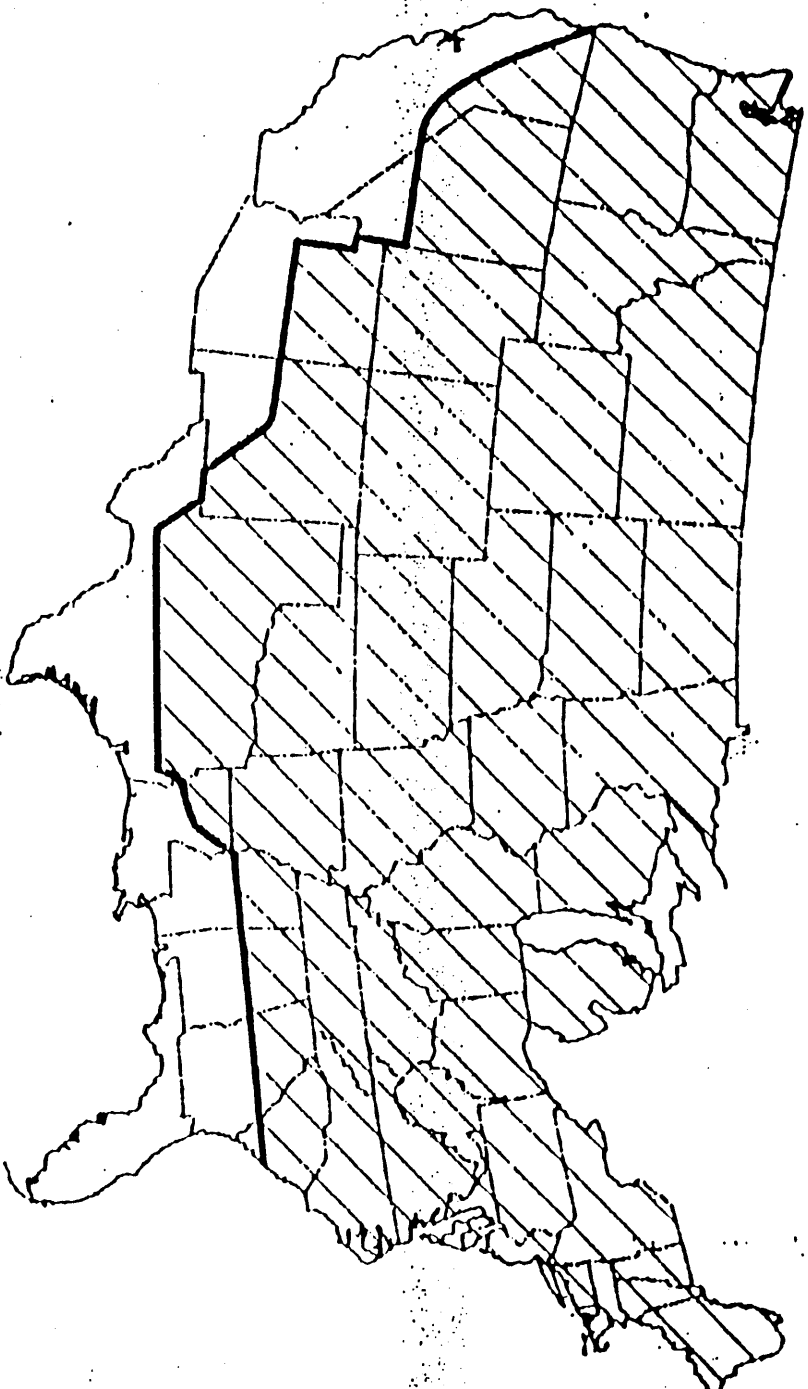


Figure 3-1. Ice Load Map

Section 3 - Loads

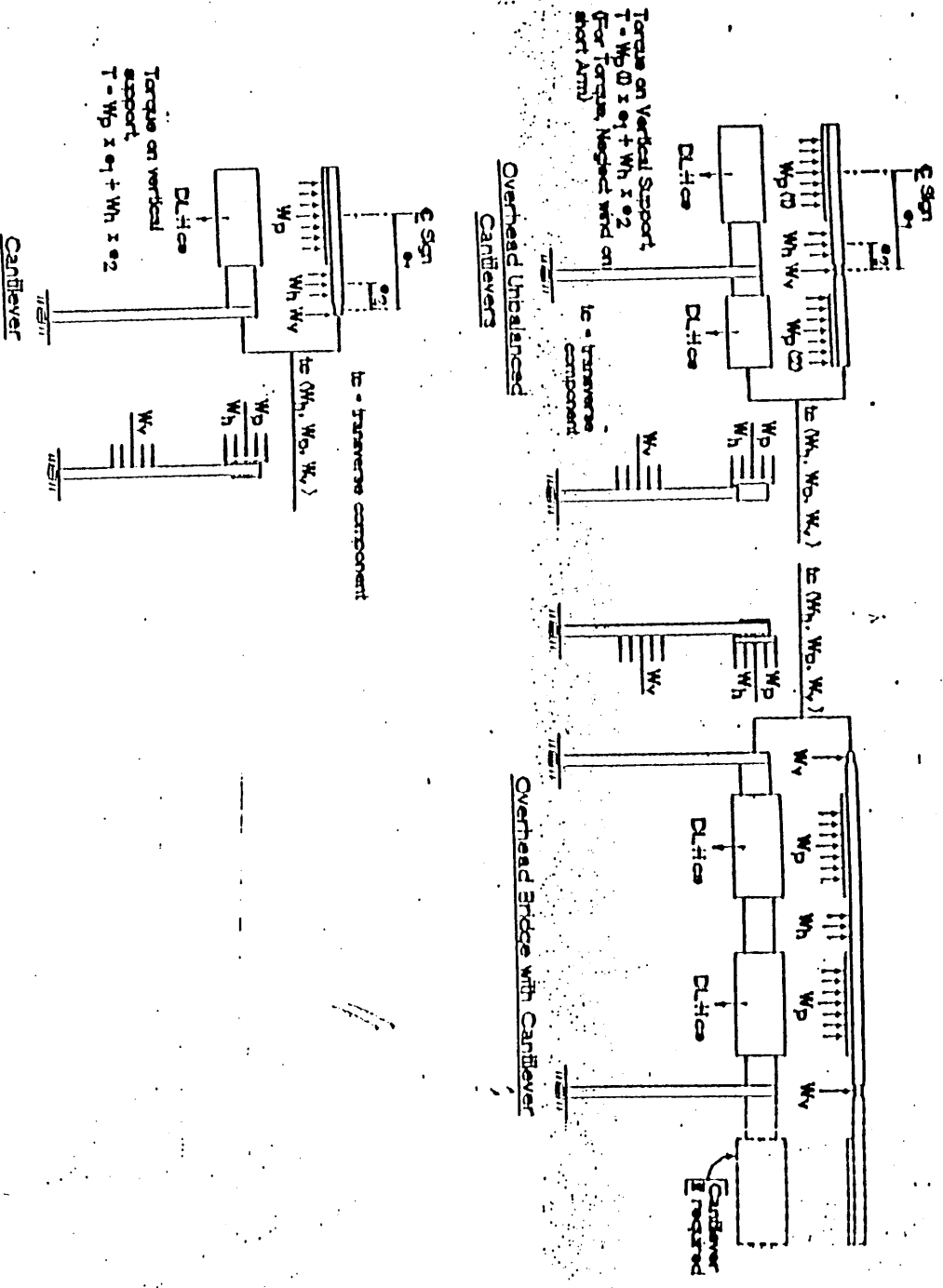


Figure 3-1. Loads on Sign Support Structures

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10.1 SCOPE

This section provides serviceability requirements for support structures.

10.2 DEFINITIONS

Camber -- the condition of the horizontal support being arched.

Quadri-chord truss -- a horizontal member composed of four longitudinal chords connected by bracing.

Rake -- to slant or incline from the vertical.

Tri-chord truss -- a horizontal member composed of three longitudinal chords connected by bracing.

10.3 NOTATION

d_{DL}	=	deflection at free end of horizontal support under dead load (mm, in)
d_p	=	deflection at tip of vertical support under dead load (mm, in)
d_{pDL}	=	total dead load deflection at free end of horizontal support (mm, in)
E	=	modulus of elasticity (MPa, psi)
H	=	height of vertical support (mm, in)
I	=	moment of inertia of vertical support (mm ⁴ , in ⁴)
L	=	distance between supports for an overhead bridge structure; distance from vertical support to free for horizontal cantilevered support (mm, in)
M	=	moment due to dead loads applied to the vertical support at the connection of the horizontal support (N-mm, lb-in)
u	=	pre-fabricated camber (slope) in the horizontal cantilevered arm (mm/mm, in/in)

10.4 DEFLECTION

Highway support structures of all materials should be designed to have adequate structural stiffness that will result in acceptable serviceability performance. Deflections for specific structure types shall be limited as provided in Articles 10.4.1 and 10.4.2. Permanent camber for specific structure types shall be provided per Article 10.5.

The deflection limits that are set by this Specification are to serve two purposes. The first purpose is to provide an aesthetically pleasing structure under dead load conditions. The second purpose is to provide adequate structural stiffness that will result in acceptance serviceability performance under applied loads.

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10.4.1 Overhead Bridge Supports for Signs

For overhead bridge monotube and truss structures supporting signs, the maximum vertical deflection of the horizontal support due to Group 1 load combination with the addition of ice loads (i.e., dead load plus ice) shall be limited to $\frac{150}{L}$ where L is the span length. For those locations where ice loading is not applicable, only deflection due to Group 1 load combination shall be used.

COMMENTARY

Research was sponsored by the Arizona Department of Transportation (Elsani *et al*, 1984; Martin *et al*, 1985) to determine an appropriate deflection limitation for steel monotube bridge support structures. This research included field tests and analytical studies using computer modeling. The studies investigated the static and dynamic behavior of monotube bridge sign support structures. It determined a dead load deflection limit that should be specified for monotube bridge structures. The 1989 Interim Specification was revised to limit deflection to the span divided by 150 for dead and ice load applications based on this research, replacing the previous limit of $d^3/400$ in feet. A later study (Lundgren, 1989) indicated that since the deflection criteria was an aesthetic limitation, it could be increased to the span divided by 100; however, no additional work has been found to justify changing the deflection limit to a more liberal value. Although this study considered only steel members, the deflection limit has been generalized for other materials since aesthetics was the governing consideration.

Other types of overhead sign supports (i.e., 2-chord, tri-chord, and quadri-chord trusses) generally have higher stiffness than the monotube type. It is believed that this dead load deflection limit of the span divided by 150 (i.e., $L/150$) could be adopted as a conservative limit for those types of overhead sign and traffic signal support structures made with 2-chord, tri-chord or quadri-chord trusses.

10.4.2 Cantilevered Supports for Signs

10.4.2.1 Vertical Supports

Unless exception is allowed by the Owner, the horizontal deflection limit for vertical

The dead load deflection and slope limitations were developed based on aesthetic

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supports shall be limited as follows:

Under Group I load combination (dead load only), slope at top of vertical support shall be limited to 30 mm per meter (0.35 inches per 12 inches).

Deflections shall be computed by usual methods or formulas for elastic deflections.

COMMENTARY

considerations and general consensus in the pole industry. The 1.5% deflection limit was generally developed for transverse load applications, such as strain pole applications, where a dead load due to span wire tension could cause unsightly deflection. The horizontal linear displacement at the top of the structure is measured in relation to a tangent to the centerline of the structure's base. The slope limitation of 30 mm per meter (0.35 inches per 12 inches), which is equivalent to an angular rotation of $1^{\circ} 40'$, was developed mainly for street lighting poles with a single mast arm, where the mast arm applied a concentrated dead load moment that could also cause unsightly deflections. It is measured by the angular rotation of the centerline at the top of the structure in relation to the centerline at its base. The slope criterion will generally control when the vertical support is subjected to concentrated moment loads resulting from the effect of eccentric loads of single or unbalanced multiple horizontally mounted arm members and their appurtenances.

10.4.2.2 Horizontal Supports

Adequate stiffness shall be provided for the horizontal supports of cantilevered sign structures that will result in acceptable serviceability performance.

No dead load deflection limit is prescribed for horizontal supports of cantilevered sign and traffic signal structures. Stiffness requirements are determined by the Designer. Structures are typically raked and/or the horizontal supports are cambered such that the deflection at the end of the arm is above a horizontal reference line for the unloaded configuration, which provides the appearance of a structure that is not overloaded. Camber requirements for cantilevered sign and traffic signal structures are provided in Article 10.5.

The vibration deflection limitation for mast arms is a new provision to prevent excessive displacements when a support structure is subjected to galloping and truck-induced cyclic loading. The 1994 edition of this Specification only addressed dead load deflection requirements of the vertical support.

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10.4.3 Vibration

Structural supports that are susceptible to vibration should be equipped with appropriate damping or energy-absorbing devices. All aluminum overhead sign structures should be equipped with appropriate damping or energy-absorbing devices to prevent significant wind-induced vibration in the structure, both before and after installation of sign panels.

Vibrations may be due to wind-induced loads, such as galloping or vortex shedding. Moving traffic may induce gusts on nearby structures, such as a large truck passing under overhead sign structures. Vibrations may also be due to support movement, such as those found on bridges and elevated roadways.

Steel and aluminum overhead bridge sign structures may be subject to damaging vibrations and oscillations when sign panels and/or traffic signals are not in place during erection or maintenance of the structure. To avoid these vibrations and oscillations, considerations should be given for providing temporary damping devices attached to the structure, such as blank sign panels.

10.4.3.1 Requirements for Individual Truss Members

Specification limitations for $\frac{L}{r}$ ratios should be adequate to prevent excessive vibration.

Vibration in truss structures can occur in individual members. Slender tension members and redundant diagonals are particularly susceptible to vibration. Resistance to local vibrations can be provided by increasing member stiffness, thereby reducing flexural deflection and raising vibration response frequencies.

10.5 CAMBER

Permanent camber to $\frac{L}{1000}$, where L is unsupported length of the horizontal support, shall be provided in addition to dead load camber for overhead sign structures.

The camber requirement applies to overhead bridge sign structures and to sign supports with a horizontal cantilevered support. The permanent camber should compensate for deflections and estimated foundation rotation due to permanently applied loads. The permanent camber is in addition to the dead load camber, which compensates for dead load deflection.

Camber is the condition of the horizontal support being arched. Permanent camber is the

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condition of the horizontal support being arched upward after application of the dead loads. The horizontal support should be arched upward such that the vertical distance from the attachment point(s) to location of maximum deflection for the horizontal support is equal to $\frac{L}{1000}$. The permanent camber provides the visual effect of a low-pitched arch, which is more appealing than a horizontal support that is deflected downward.

Permanent camber can be provided by raking the vertical support and/or cambering the horizontal support. Raking the vertical support involves installing the vertical support with an initial deflection. The vertical support is raked during construction by adjusting the leveling nuts at the base of the structure. Cambering the horizontal support involves fabricating the support with an initial slope or curvature.

The following procedure may be used to calculate the camber required to compensate for dead load deflection in a cantilevered sign support structure with a monotube vertical support.

The cantilevered horizontal support should be cambered during fabrication, such that the permanent camber after application of dead load is a minimum of $\frac{L}{1000}$ above the horizontal plane, where L is the span of the horizontal support. The procedure shown is for a non-tapered vertical support with a constant stiffness.

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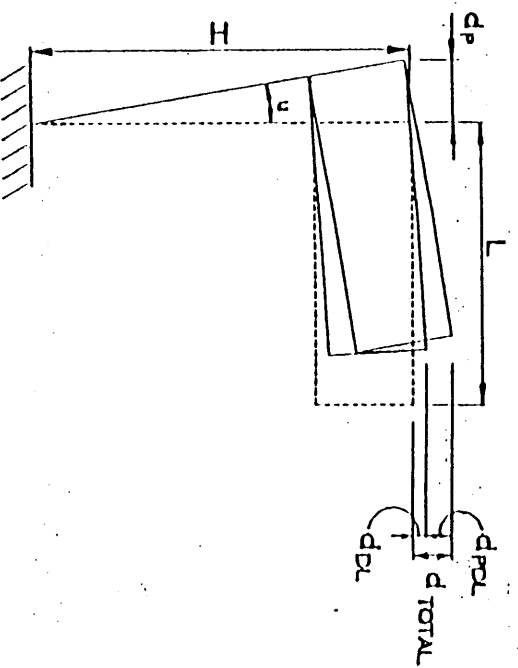


Figure 10-1. Camber of Sign Structure.

- determine horizontal deflection at tip of vertical support due to dead load (deflection of vertical support not shown in figure):

$$d_p = \frac{MH^2}{2EI}$$

- Determine deflection of horizontal support due to slope at tip of vertical support:

$$d_{pDL} = \frac{2d_p L}{H}$$

- Determine deflection of horizontal support due to dead load acting on the horizontal support, d_{DL} .

- Calculate total dead load deflection at the tip of the cantilevered arm:

$$d_{TOTAL} = d_{DL} + d_{pDL}$$

- Determine the slope u of the pre-fabricated camber in the horizontal support, such that:

$$u = \frac{1}{1000} \frac{d_{DL}}{L} + \frac{d_{pDL}}{L}$$

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- f). This slope will result in a final deflection at the end of the horizontal arm equal to $\frac{L}{1000}$ above the horizontal plane.
- g). Provide fabrication details indicating the prefabricated camber (slope) μ in horizontal support.

The above procedure does not consider the raking of the vertical support. The vertical support may be raked to center the top of this support over its base.

A slope for some cantilevered horizontal supports has been provided by tilting the arm by a small angle at its base connection.

When the total dead load deflection is very small, some vertical support have been raked to compensate for the full deflection of the cantilevered horizontal support.

10.6. REFERENCES

Lengel, J.C., and Sharp, M.L., "Vibration and Damping of Aluminum Overhead Sign Structures," *Highway Research Record*, No. 259, 1969.

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Section 11 - Fatigue Design

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11.1 SCOPE

This section contains provisions for the fatigue design of variable message sign structures (VMS).

This section focuses on fatigue, which is defined herein as the damage that may result in fracture after a sufficient number of stress fluctuations. It is based on the recently completed NCHRP Project 10-38 *Fatigue Resistant Design of Cantilevered Signal, Sign and Light Supports* (Kaczinski, *et al*, 1996). The study focused on critical support structures that show susceptibility to fatigue failures. A continuation of the project is underway to further refine the proposed design criteria.

11.2 DEFINITIONS

Constant-amplitude fatigue threshold -- also known as constant-amplitude fatigue limit (CAFL) or endurance limit, a stress range below which a fatigue life appears to be infinite.

Fatigue -- damage resulting in fracture due to stress fluctuations.

In-plane bending -- bending in-plane for the main member (column) caused by galloping or truck-induced gusts.

Limit-state wind load effect -- a specifically defined load criteria.

Load bearing attachment -- attachment to main member where there is a transverse load range in the attachment itself in addition to any primary stress range in the main member.

Non-load bearing attachment - -attachment to main member where the only significant stress range is the primary stress in the main member.

Out-of-plane bending -- bending out-of-plane for the main member (column) caused by natural wind gusts.

Pressure range -- magnitude of force, in terms of pressure, of a limit state wind load effect.

Stress range -- magnitude of stress fluctuations.

Yearly mean wind velocity -- long term average of the wind speed for a given area.

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11.3 NOTATION

C_d	=	worst case drag coefficient from Section 3, "Loads", for given attachment or member
d	=	diameter of a circular section (m,ft)
D	=	inside diameter of exposed end of female section for slip-joint splice (mm,in)
E	=	modulus of elasticity (MPa, ksi)
g	=	acceleration of gravity (9810 mm/s^2 , 386 in/s^2)
I	=	moment of inertia (mm^4 , in^4)
I_{avg}	=	average moment of inertia for a tapered pole (mm^4 , in^4)
I_{top}	=	moment of inertia at top of tapered pole (mm^4 , in^4)
I_{bottom}	=	moment of inertia at bottom of tapered pole (mm^4 , in^4)
L	=	length of the pole (mm, in)
L	=	slip-splice overlap length (mm, in)
P_G	=	galloping pressure range (Pa, psf)
P_{NW}	=	natural wind gust pressure range (Pa, psf)
P_{TC}	=	truck-induced gust pressure range (Pa, psf)
r	=	radius of chord or column (mm, in)
S_r	=	design stress range of the main member or branching member (MPa, ksi)
t	=	thickness (mm, in)
V_{mean}	=	yearly mean wind velocity for a given area (m/s, mph)
w	=	weight of the pole per unit length (N/mm, k/in)
β	=	damping ratio

11.4 APPLICABLE STRUCTURE TYPES

Sign structures carrying variable message signs (VMS).

The fatigue design procedures outlined in this section may be applicable to steel and aluminum structures in general.

In general, overhead structures should be designed for fatigue due to multiple loadings from galloping, natural wind gusts, and truck-induced wind gusts.

NCHRP Project 10-38, *Fatigue Resistant Design of Cantilevered Signal, Sign and Light Supports* (Kaczinski, et al, 1996) is the basis for the fatigue design provisions.

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11.5 DESIGN CRITERIA

Variable message sign support

structures shall be designed for fatigue to resist each of the equivalent static wind load effects specified in Article 11.7. Stresses due to these loads on all components, mechanical fasteners, and weld details shall be designed to satisfy the requirements of their respective detail categories within the constant-amplitude fatigue thresholds provided in Table 11-3. A summary of typical fatigue-sensitive support structure connection details is presented in Table 11-2 and is illustrated in Figure 11-1.

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Accurate load spectra and life prediction techniques for defining fatigue loadings are generally not available. The assessment of stress fluctuations and the corresponding number of cycles for all wind-induced events (lifetime loading histogram) is practically impossible. With this uncertainty, the design of sign, supports for a finite fatigue life becomes impractical. Therefore, an "infinite life" fatigue design approach is recommended and considered sound practice. It is generally based on the constant-amplitude fatigue limit (CAFL). The CAFL values provided in Table 11-3 are approximately the same as those given in Table 10.3.1.A of the *Standard Specifications for Highway Bridges* (AASHTO, 1996).

An "infinite-life" fatigue approach was developed in an experimental study that considered several critical welded details (Fisher *et al*, 1993). The "infinite-life" fatigue approach can be used when the number of wind load cycles expected during the lifetime of the structures is greater than the number of cycles at the constant-amplitude fatigue limit (CAFL). This is particularly the case for structural supports where the wind load cycles in 25 years or greater lifetime are expected to exceed 100 million cycles, whereas typical weld details reach the CAFL at 10 to 20 million cycles.

Fatigue critical details should be designed with nominal stress ranges that are below the appropriate constant-amplitude fatigue limit (CAFL). To assist designers, a categorization of typical support structure details to the existing AASHTO and AWS fatigue design categories is provided in Table 11-2 and Figure 11-1. Based on a review of State Department of Transportation standard drawings and manufacturer literature, the above referenced list of typical support connection details was produced. This list

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should not be considered as a complete set of all possible connection details, but rather it is intended to remove the uncertainty associated with applying the provisions of the *Standard Specifications for Highway Bridges* to the fatigue design of support structures.

This detailed categorization of fatigue sensitive connection details can be used by designers and fabricators to produce more fatigue resistant structures. Proper detailing will improve the fatigue resistance of these structures.

The notes for Table 11-2 specify the use of Stress Category K_2 . This stress category corresponds to the category for cyclic punching shear stress in tubular members specified by the *AWS Structural Welding Code D1.1 - Steel*. Fatigue design for the column's wall under this condition may require sizes of the built-up box connection and/or column wall thicknesses that are excessive for practical use. For this occurrence, an adequate fatigue resistant connection other than the built-up box shown in Figure 11-1 shall be used.

11.7 FATIGUE DESIGN LOADS

To avoid the development of fatigue cracks in various connection details, structures shall be designed to resist limit - state equivalent static wind loads acting on the structure. These loads, listed below, shall be used to calculate nominal stress ranges at fatigue sensitive connection details, as described in Article 11.5. The calculated nominal stress range shall not exceed the CAFL values given in Table 11-3 for a particular connection detail. The loads to design for are:

Support structures that are exposed to several wind phenomena that can produce cyclic loads. Vibrations associated with these cyclic forces can become significant. Recent research (Kaczinski *et al*, 1996) has identified, natural wind gusts, and truck-induced gusts as wind-loading mechanisms that can induce fatigue damage in sign structures.

Design pressures for each of the possible fatigue wind-loading mechanisms are

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SPECIFICATIONS	COMMENTARY
<ol style="list-style-type: none">1. Galloping - Article 11.7.1.2. Natural Wind Gust - Article 11.7.33. Truck Induced Gust - Article 11.7.4 <p>The Galloping load shall be applied to the structure independently from the Gust Loads.</p> <p>The Natural Wind Gust and Truck Induced Gust loads shall be applied to the structure simultaneously.</p>	<p>presented as an equivalent static wind pressure range. These pressure ranges should be applied to the structure as prescribed by this Section in a simple static analysis to determine stress ranges at fatigue sensitive details and maximum mast arm deflections for VMS sign support structures.</p>

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<p>11.7.1 Galloping</p>	
<p>Overhead cantilevered sign support structures shall be designed for galloping-induced cyclic loads by applying an equivalent static shear pressure vertically to the surface area as viewed in normal elevation of all sign panels. The magnitude of this vertical shear pressure range shall be equal to the following:</p>	<p>Galloping, or Den Hartog instability, results in large amplitude, resonant oscillations in a plane normal to the direction of wind flow. It is usually limited to structures with non-symmetrical cross sections, such as sign and traffic signal structures with attachments to the horizontal cantilevered arm. Structures without attachments to the cantilevered horizontal support are not susceptible to galloping induced wind load effects.</p>
$P_g = 1000 \text{ (Pa)}$ $P_g = 21 \text{ (psf)}$	
<p>The load shall be applied to the structure as a vertical force equal to $P_g \times$ (Area of Sign Face) placed at the center of gravity of each sign panel.</p>	<p>The results of wind tunnel (Kaczinski <i>et al</i>, 1998) and water tank (McDonald <i>et al</i>, 1995) testing, as well as the oscillations observed on cantilevered support structures in the field, are consistent with the characteristics of the galloping phenomena. These characteristics include the sudden onset of large-amplitude, across-wind vibrations which increase with increases in wind velocity. It is important to note, however, that galloping is typically not caused by support structure members, but rather by the attachments to the horizontal cantilevered arm, such as signs and traffic signals.</p>
<p>In lieu of designing to resist periodic galloping forces, cantilevered sign structures may be erected with approved vibration mitigation devices. Vibration mitigation devices should be approved by the Owner and should be based on historical or research verification of its vibration damping characteristics.</p>	<p>The geometry and orientation of these attachments, as well as the wind direction, directly influence the susceptibility of cantilevered support structures to galloping. Galloping of sign attachments is independent of aspect ratio and is more prevalent with wind flows from the front of the structure.</p>
	<p>By conducting wind tunnel tests and analytical calibrations to field data and wind tunnel test results, an equivalent static vertical shear of 1000 Pa (21 psf) was determined for the galloping phenomena. This vertical shear range should be applied to the entire frontal area of each of the sign attachments in a static analysis to determine stress ranges at critical connection details. For example, if a 2.5 m by 3.0 m (8 ft. by 10 ft.) sign panel is mounted to a horizontal</p>

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mast arm, a static force of 7500 N (1680 lb) should be applied vertically to the structure at the center of gravity of the sign panel.

11.7.3 Natural Wind Gust

Sign, be designed to resist an equivalent static natural wind gust pressure range of

$$\begin{aligned} P_{NW} &= 250 C_d (Pa) \\ P_{NW} &= 5.2 C_d (psf) \end{aligned}$$

Eq. 11-5

where C_d is the worst case drag coefficient specified in Section 3, "Loads", for the considered element to which the pressure range is to be applied. A C_d of 2.0 shall be used for VMS's. The natural wind gust pressure range shall be applied in the horizontal direction to the exposed area of all support structure members, signs, and/or miscellaneous attachments.

The design natural wind gust pressure range is based on a yearly mean wind speed of 5 m/s (11.2 mph). For locations with more detailed wind records, particularly sites with higher wind speeds, the natural wind gust pressure may be modified at the discretion of the Owner.

Due to the inherent variability in the velocity and direction of air flow, natural wind gusts are the most basic wind phenomena that may induce vibrations in wind-loaded structures. The equivalent static natural wind gust pressure range specified for design was developed with data obtained from an analytical study of the response of cantilevered support structures subject to random gust loads (Kaczinski *et al*, 1996). This parametric study was based on the 0.01 percent exceedance for a yearly mean wind velocity of 5 m/s (11.2 mph), which is a reasonable upper-bound of yearly mean wind velocities for most locations in the country. There are locations, however, where the yearly mean wind velocity is larger than 5 m/s (11.2 mph). For installation sites with more detailed information regarding yearly mean wind speeds (particularly sites with higher wind speeds), the following equivalent static natural wind gust pressure range shall be used for design:

$$\begin{aligned} P_{NW} &= 250 C_d \left(\frac{V_{mean}^2}{25} \right) (Pa) \\ P_{NW} &= 5.2 C_d \left(\frac{V_{mean}^2}{125} \right) (psf) \end{aligned}$$

Eq. C 11-5

11.7.4 Truck-Induced Gust

Overhead sign support structures shall be designed to resist an equivalent static truck gust pressure range of

$$\begin{aligned} P_{TG} &= 525 Pa \\ P_{TG} &= 11 psf \end{aligned}$$

The load for this specification came from the research report titled "*Fatigue Related Wind Loads on Highway Support Structures*" by Kevin Johns and Robert Dexter.

The passage of trucks beneath support structures induce gust loads on the attachments

Section 11 - Fatigue Design

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<p>This pressure range shall be applied in the vertical direction to the bottom of a VMS and any appurtenances that might be attached to the bottom of the sign. This pressure range shall be applied along a 3.7m (12 ft.) long strip of sign, horizontal support, and any attachments located within this length. The load shall be transversely located so as to maximize the load effect for that portion of the structure under investigation (e.g., moment in horizontal support, or shear in connections and anchor bolts.) The equivalent static truck pressure range may be reduced for locations where vehicle speeds are less than 30 m/s (65 mph). The reduction shall be proportional to square of the vehicle speeds.</p>	<p>mounted to the horizontal support of these structures. Although loads are applied in both the horizontal and vertical direction, horizontal support vibrations caused by forces in the vertical direction are most critical. Therefore, truck gust pressures are applied only to the exposed horizontal surface of the attachment and horizontal support.</p> <p>Recent vibration problems on sign structures with large projected areas in the horizontal plane, such as variable message sign enclosures, has focused attention on vertical gust pressures created by the passage of trucks beneath the sign. To improve fuel economy, many trucks are outfitted with deflectors to divert the wind flow upward and minimize the drag created by the trailer. It has been proposed to represent this gust pressure as that imposed by a 30 m/s (65 mph) wind.</p>

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Table 11-2. Fatigue Details of Support Structures

Construction	Detail	Stress Category	Application	Example
Plain Members	1. With rolled or cleaned surfaces. Flame-cut edges with ANSI/AASHTO/AWS D5.1 (Article 3.2.2) smoothness of 1,000 micro-in. or less.	A	-	-
	2. Slip-joint splice where L is greater than 1.5 diameters.	B	High-level towers.	1
	3. Net section of fully-tightened high-strength (ASTM A325, A490) bolted connections.	B	Bolted joints.	2
Mechanically Fastened Connections	4. Net section of other mechanically fastened connections: Steel: Aluminum:	D E	-	3
	5. Anchor bolts or other fasteners in tension; stress range based on the tensile stress area. Misalignments of less than 1:40 with firm contact existing between anchor bolt nuts and base plate.	D	Anchor bolts. Bolted mast-arm-to-column connections.	8
	6. Connection of members or attachment of miscellaneous signs, traffic signals, etc., with clamps or U-bolts.	D	-	-
	7. Net section of holes and cutouts.	D	Wire outlet holes. Drainage holes. Unreinforced handholes.	5
	8. Tubes with continuous full- or partial-penetration groove welds parallel to the direction of the applied stress.	B'	Longitudinal seam welds.	6
Groove Welded Connections	9. Full-penetration groove-welded splices with welds ground to provide a smooth transition between members (with or without backing ring removed).	D	Column or mast arm butt-splices.	4
	10. Full-penetration groove-welded splices with weld reinforcement not removed (with or without backing ring removed).	E	Column or mast arm butt-splices.	4
	11. Full-penetration groove-welded tube-to-transverse plate connections with the backing ring attached to the plate with a full-penetration weld.	E	Column-to-base-plate connections. Mast-arm-to-flange-plate connections.	5
	12. Full-penetration groove-welded tube-to-transverse plate connections (backing ring not removed).	E'	Column-to-base-plate connections. Mast-arm-to-flange-plate connections.	5

SPECIFICATIONS

COMMENTARY

Table 11-2. Fatigue Details of Support Structures (continued)

Construction	Detail	Stress Category	Application	Example
Fillet-Welded Connections	13. Fillet-welded lap splices.	E	Column or mast arm lap splices.	3
	14. Axially loaded members with fillet-welded end connections without notches perpendicular to the applied stress. Welds distributed around the axis of the member so as to balance weld stresses.	E	Angle-to-gusset connections with welds terminated short of plate edge. Slotted tube-to-gusset connectors with coped holes.	2, 6
	15. Axially loaded members with fillet welded end connections with notches perpendicular to the applied stress. Welds distributed around the axis of the member so as to balance weld stresses.	E'	Angle-to-gusset connections. Slotted tube-to-gusset connections without coped holes.	2, 6
	16. Fillet-welded tube-to-transverse plate connections.	E'	Column-to-base-plate or mast-arm-to-flange-plate socket connections.	7, 8
	17. Fillet-welded connections with one-sided welds normal to the direction of the applied stress.	E'	Built-up box mast-arm-to-column connections.	8
	18. Fillet-welded mast-arm-to-column pass-through connections.	E'	Mast-arm-to-column pass-through connections.	9
	19. Fillet welded T-, Y-, and K-tube-to-tube, angle-to-tube, or plate-to-tube connections.	See Notes a and b	Chord-to-vertical or chord-to-diagonal truss connections (see Note a). Mast-arm directly welded to column (see Note b). Built-up box connection (see Note b).	8, 10, 11
	20. Non-load bearing longitudinal attachments with partial- or full-penetration groove welds, or fillet welds, in which the main member is subjected to longitudinal loading.	C D E	Welds terminations at ends of longitudinal stiffeners. Reinforcement at handholes.	12, 13
	21. Non-load bearing longitudinal attachments with $L > 102 \text{ mm}$ (4 in) and full-penetration groove welds. The main member is subjected to longitudinal loading and weld termination embodies a transition radius or taper with the weld termination ground smooth:	C D E	Welds terminations at ends of longitudinal stiffeners.	14
	22. Non-load bearing longitudinal attachments with $L > 102 \text{ mm}$ (4 in) and fillets or partial-penetration groove welds. The main member is subjected to longitudinal loading and the weld termination embodies a transition radius or taper with the weld termination ground smooth:	C D E	Weld terminations at ends of longitudinal stiffeners.	14
Attachments	23. Transverse load-bearing fillet-welded attachments where $t \leq 13 \text{ mm}$ (0.5 in) and the main member is subjected to minimal axial and/or flexural loads. (When $t > 13 \text{ mm}$ (0.5 in), see Note d).	C	Longitudinal stiffeners welded to baseplates.	12, 14

SPECIFICATIONS

COMMENTARY

Table 11-2. Fatigue Details of Support Structures (continued)

Construction	Detail	Stress Category	Application	Example
Attachments (continued)	24. Transverse load-bearing longitudinal attachments with partial- or full-penetration groove welds or fillet welds, in which the non-tubular main member is subjected to longitudinal loading and the weld termination embosses a transition radius which is ground smooth: $R > 51 \text{ mm (2 in)}$ or $15^\circ < \alpha \leq 60^\circ$; $R \leq 51 \text{ mm (2 in)}$ or $\alpha > 60^\circ$;	D E See Note c	Gusset-plate-to-chord attachments	15

Notes:

- a) Stress Category ET with respect to stress in branching member provided that $\frac{r}{t_c} \leq 24$ for the chord member. When $\frac{r}{t_c} > 24$, then the fatigue strength equals:

$$(F)_n = (\Delta F)_n^E \times \left(\frac{24}{\frac{r}{t_c}} \right)^{0.7}, \text{ where } (\Delta F)_n^E \text{ is the CAFL for Category ET.}$$

Stress Category E with respect to stress in chord.

- b) Stress Category ET with respect to stress in branching member.

Stress Category K₂ with respect to stress in main member (column) provided that $\frac{r}{t_c} \leq 24$ for the main member. When $\frac{r}{t_c} > 24$, then the

fatigue strength equals:

$$(F)_n = (\Delta F)_n^{K_2} \times \left(\frac{24}{\frac{r}{t_c}} \right)^{0.7}, \text{ where } (\Delta F)_n^{K_2} \text{ is the CAFL for Category K}_2.$$

The design stress range in the main member equals:

$$(SR)_{\text{main member}} = (SR)_{\text{branching member}} \left(\frac{t_b}{t_c} \right)^c$$

Where t_b is the wall thickness of the branching member, t_c is the wall thickness of the main member (column), and c is the ovalizing parameter for the main member equal to 0.57 for in-plane bending, and equal to 1.5 for out-of-plane bending in the main member. (SR)_{branching} is the stress range in the branching member. (See commentary of Article 11.5)

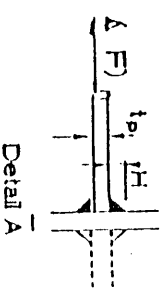
The main member shall also be designed for Stress Category E using the section modulus of the main member and moment just below the connection of the bending member.

- c) First check with respect to the longitudinal stress range in the main member per the requirements for non-load bearing longitudinal attachments. The attachment must then be separately checked with respect to the transverse stress range in the attachment per the requirements for transverse load-bearing longitudinal attachments.

- d) When $D \geq 13 \text{ mm (0.5 in)}$, the fatigue strength shall be the lesser of Category C or the following:

$$(\Delta F)^C = (\Delta F)_n^C \times \left(\frac{0.094 + 1.23 \frac{H}{t_p}}{t_p/t_c} \right) \text{ (MPa)}$$

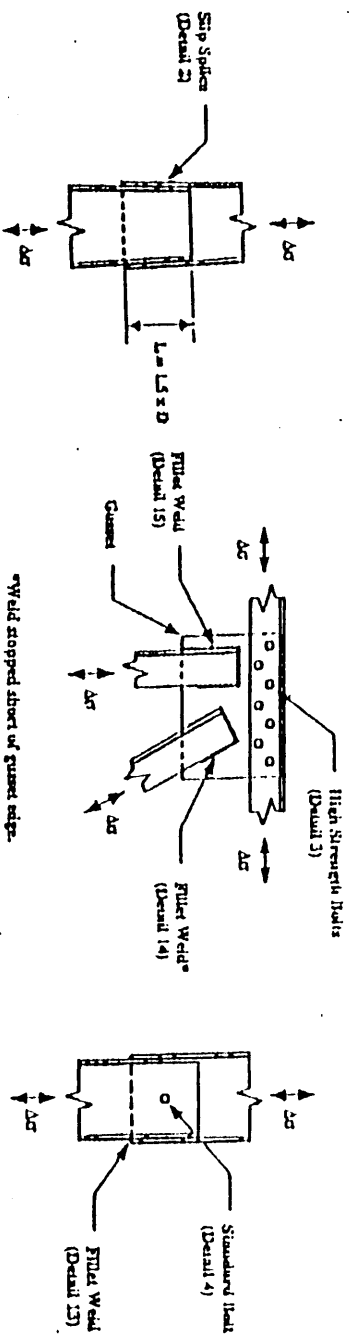
$$(\Delta F)^C = (\Delta F)_n^C \times \left(\frac{0.055 + 0.72 \frac{H}{t_p}}{t_p/t_c} \right) \text{ (ksi)}$$



where $(\Delta F)_n^C$ is the CAFL for Category C, H is the effective weld throat (mm, in), and t_p is the plate thickness (mm, in) as defined in Detail A.

Table 11-3. Constant-Amplitude Fatigue Thresholds

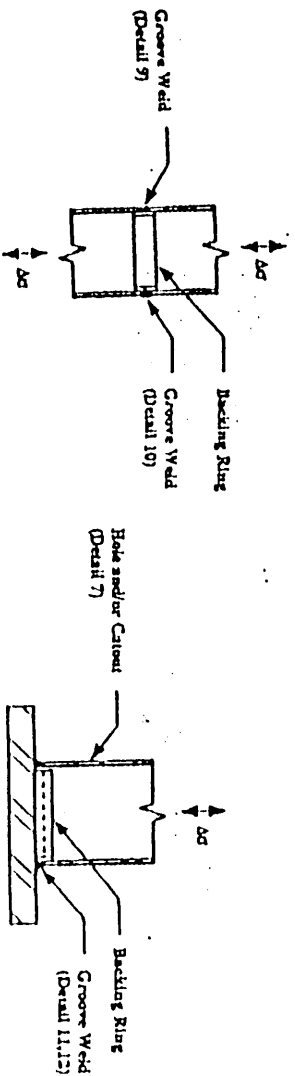
Detail Category	Steel Threshold	Aluminum Threshold
	MPa	ksi
A	165	24
B	110	16
B'	83	12
C	69	10
D	48	7
E	31	4.5
E'	18	2.6
E _T	8	1.2
K ₂	7	1.0



Slip-Joint Splice
Example 1

Double-Angle T-Beam Gasket
Example 2

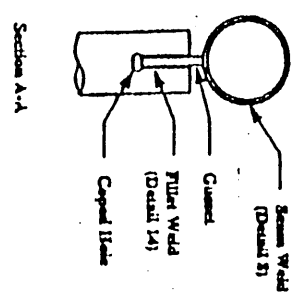
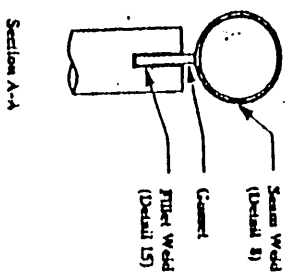
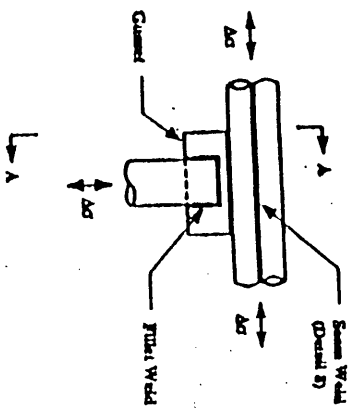
Fillet-Welded Lap-Splice
Example 3



Groove-Welded Butt-Splice
Example 4

Groove-Welded Tube-to-Plate Transverse Plate Connection
Example 5

Figure 11-1. Illustrative Examples



Studded Tube-to-Gusset Connection

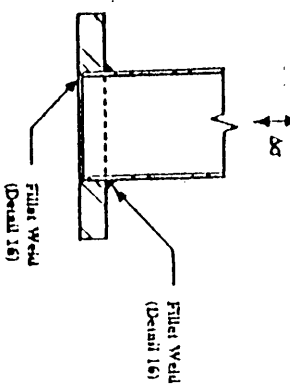
Example 6

Studded Tube-to-Gusset Connection

Example 6

Studded Tube-to-Gusset Connection

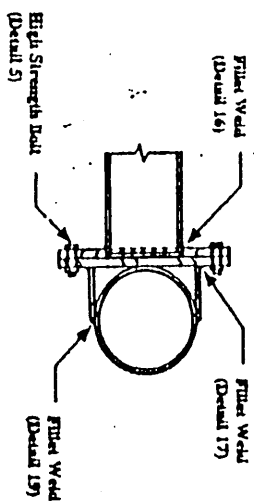
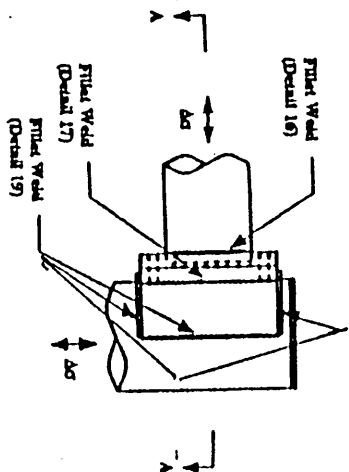
Example 6



Fillet-Welded Socket Connection

Example 7

Figure 11-1. Illustrative Examples
(2 of 5)



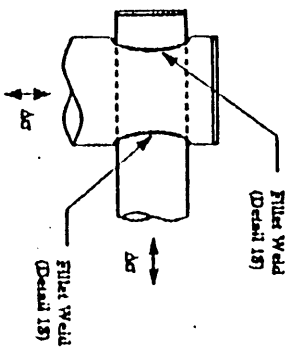
Section A-A

Fillet-Welded Mast-Arnado-Column Connection
(Buitt-Up Box)

Fillet-Welded Mast-Arnado-Column Connection
(Buitt-Up Box)

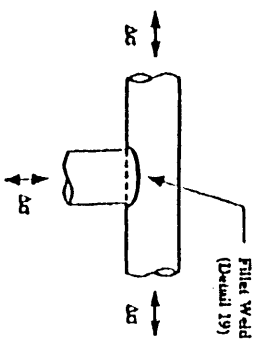
Example 8

Example 8



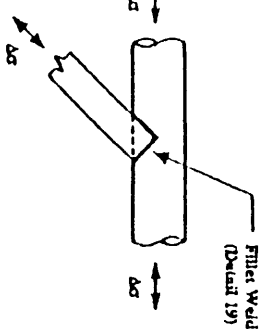
Fillet-Welded Tube-to-Tube Column
Pass-Through Connection

Example 9



Fillet-Welded Tube-to-Tube Connection

Example 10



Fillet-Welded Angle-to-Tube Connection

Example 11

Figure 11-1. Illustrative Examples

(3 of 5)

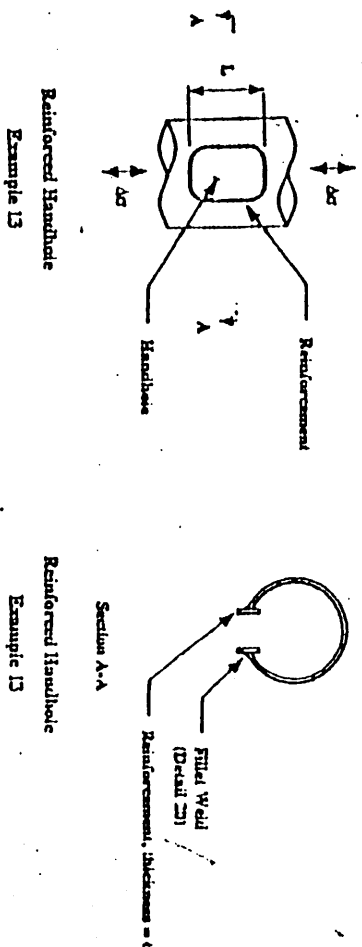
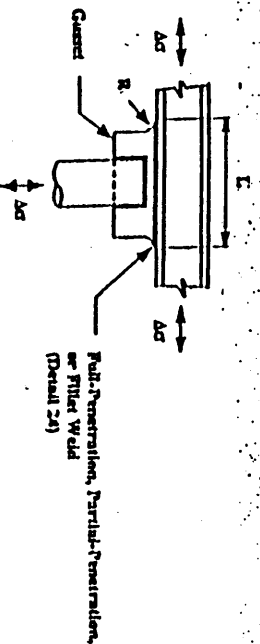


Figure 11-1. Illustrative Examples
(4 of 5)



Transverse Load-Bearing Longitudinal Attachment

Example 15

Figure 11-1. Illustrative Examples

(5 of 5)

11.9 REFERENCES

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