NCHRP

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

REPORT 494

Structural Supports for Highway Signs, Luminaires, and Traffic Signals

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

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Structural Supports for Highway Signs, Luminaires, and Traffic Signals

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SUBJECT AREAS
Bridges, Other Structures, and Hydraulics and Hydrology

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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NCHRP PROJECT 17-10(2) PANEL
Field of Traffic—Area of Safety

AUTHOR ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 17-10(2) by the Department of Civil and Environmental Engineering, University of Alabama at Birmingham (UAB).

Dr. Fouad H. Fouad, Professor of Civil Engineering, UAB, was the principal investigator. Co-investigators on this project are Dr. James Davidson, Assistant Professor, UAB; Dr. Norbert Delatte, Assistant Professor, UAB; and Ms. Elizabeth A. Calvert, structural design consultant. Additionally, Mr. Edgar Nunez and Mr. Ramy Abdalla, both Research Engineers at UAB, and Dr. Shen-En Chen, Assistant Professor, UAB, have provided significant contributions.

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FOREWORD

By David B. Beal,
Staff Officer
Transportation Research Board

This report contains the findings of a study to develop recommended revisions to the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*. The report describes the research effort and provides recommendations for updating and refining the specifications. A strategic plan for further enhancement of the specifications and for conversion of the specifications to load and resistance factor format is also included. The material in this report will be of immediate interest to specification writers, structural engineers, and designers.

The AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals,* fourth edition, includes many of the recommended provisions developed in NCHRP Project 17-10. Nevertheless, when those specifications were published, information needed to provide comprehensive specifications was either unavailable or incomplete for several topics. In particular, information was needed on the consequences of changes in the wind speed map, fatigue in noncantilevered structures, foundation selection criteria, drag coefficients for multisided tapered poles, connection plate flatness criteria, strength of rectangular poles bent about the diagonal, and performance specifications for fiber-reinforced composite structural elements. A strategic plan for future enhancement of the specifications, including conversion to load and resistance factor design format, also was needed.

Under NCHRP Project 17-10(2), researchers at the University of Alabama at Birmingham addressed these topics with the assistance of Elizabeth Calvert. This report summarizes the findings and recommendations for each of the topics listed above, which can be used to develop comprehensive support specifications. The enhancements outlined in the strategic plan may be undertaken as time and resources permit. A series of unpublished appendixes that provide full details of the research findings and recommendations for each topic are available from NCHRP in electronic form.

CONTENTS

STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES, AND TRAFFIC SIGNALS

SUMMARY

NCHRP Project 17-10 was conducted for the main purpose of revamping the 1994 edition of the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* (hereafter referred to as the *Supports Specifications*). The project addressed specific technical topics; provided major updates based on current codes and standards; presented new wind maps and wind loading criteria; and introduced new sections on fiber-reinforced composites, wood structures, and fatigue design. Project 17-10 resulted in a totally revised *Supports Specifications* that was submitted to the AASHTO Highway Subcommittee on Bridges and Structures (HSCOBS), adopted in 1999, and published in 2001.

In spite of the extensive research efforts performed under NCHRP Project 17-10, a number of technical issues still needed further investigation or refinement. The project panel identified and prioritized a number of technical topics for study in NCHRP Project 17-10(2). The main objective of NCHRP Project 17-10(2) was to enhance the *Supports Specifications* developed under Project 17-10 and to provide a strategic plan for future development of the specifications. Specific technical topics addressed by Project 17-10(2) included the following:

- **Wind load analysis.** A report for consideration by the AASHTO HSCOBS was prepared to address the basis for the differences in the wind speed maps, the differences in design loads resulting from wind speed maps, and the treatment of gusts in the 1994 and 2001 *Supports Specifications*. Approximate gust effect factors for wind-sensitive structures were also developed.
- **Fatigue and vibration in noncantilevered support structures.** Fatigue and vibration in noncantilevered support structures were studied, and a set of fatigue loads was recommended. Additionally, connection details to minimize fatigue effects, effectiveness of gussets in reducing fatigue problems, and vibration-mitigation measures were evaluated and documented. The material is presented in a format that is suitable for consideration for inclusion in the specifications.
- **Foundations and anchor bolts.** Selection criteria and design guidance for support structure foundations were provided, and a simplified design method for anchor

bolt design was proposed based on state-of-the-art information on anchorage to concrete.

- **Drag coefficient transitions for multisided to round shapes.** A drag coefficient transition equation was developed for tapered poles that transition from multisided shapes to round shapes.
- **Connection plate and base plate flatness tolerances.** Current practice and state DOT specifications were reviewed to identify the need for structure-specific connection plate and base plate flatness tolerances for erection. Tolerances based on the findings were recommended.
- **Bending about the diagonal for rectangular steel sections.** Strength and failure criteria for bending about the diagonal axis of rectangular steel sections were established. Design guidelines were developed and proposed for bending about the diagonal for rectangular steel sections.
- **Fiber-reinforced composites.** A performance specification, including acceptance testing procedures, was developed for fiber-reinforced composite support structures.
- **Design examples.** Sixteen design examples that represent good design practice for various types of support structures while illustrating key features of the specifications were developed.
- **Retrofit and rehabilitation of fatigue-damaged support structures.** A report that addresses for retrofitting and rehabilitating fatigue-damaged support structures was prepared. Guidance on repair and replacement decisions was also provided as part of this report.

Furthermore, a strategic plan for future enhancement of the *Supports Specifications* was developed, including a plan for conversion of the specifications to load and resistance factor design (LRFD) philosophy and format. The importance of converting to an LRFD approach was emphasized, and the necessary analytical work to achieve this conversion was reviewed.

CHAPTER **1**

INTRODUCTION AND RESEARCH APPROACH

1.1 BACKGROUND

NCHRP Project 17-10, whose results are summarized in *NCHRP Report 411* (*1*), was conducted for the main purpose of revamping the 1994 edition of AASHTO's *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* (*2*) (hereafter referred to as the *Supports Specifications*). The project addressed specific technical topics; provided major updates based on current codes and standards; presented new wind maps and wind loading criteria; and introduced new sections on fiber-reinforced composites, wood structures, and fatigue design. *NCHRP Report 411* adopted an allowable stress design (ASD) philosophy and was presented in a user-friendly specification/commentary side-by-side format. Project 17-10 resulted in a totally revised *Supports Specifications* that was submitted to the AASHTO Highway Subcommittee on Bridges and Structures (HSCOBS), adopted in 1999 (*3*), and published in 2001 (*4*).

1.2 RESEARCH PROBLEM STATEMENT

In spite of the extensive research efforts performed under NCHRP Project 17-10, a number of technical issues still needed further investigation or refinement. The project panel identified and prioritized a number of technical topics for study in NCHRP Project 17-10(2). Addressing these topics would result in a more comprehensive and enhanced state-of-the-art *Supports Specifications.* The 2001 *Supports Specifications* also had to be upgraded to the more rational load and resistance factor design (LRFD) philosophy. LRFD was and is the basis for most design specifications, since it provides a consistent approach with respect to strength evaluation and structural safety. Furthermore, since the *Supports Specifications* was and is a comprehensive document incorporating various materials, design criteria, and structure types, a strategic plan for future enhancement and maintenance of the specifications was needed.

1.3 RESEARCH OBJECTIVES AND APPROACH

1.3.1 Objectives

The main objective of NCHRP Project 17-10(2) was to address a number of additional technical topics to further enhance the *Supports Specifications* developed under Project 17-10 and to provide a strategic plan for future development of the specifications. Topics addressed by Project 17-10(2) included the following:

- Impact of the newly adopted wind loading criteria and wind map;
- Treatment of wind-sensitive structures;
- Fatigue of noncantilevered support structures;
- Anchor bolt design details, including embedment length and pretensioning;
- Foundation selection criteria;
- Drag coefficient transition for multisided tapered poles;
- Connection plate flatness criteria;
- Flexural strength of square or rectangular steel tube sections bent about the diagonal;
- Performance specifications and acceptance testing for fiber-reinforced composite members;
- A set of complete and detailed design examples illustrating the use of the 2001 *Supports Specifications*;
- Retrofitting and rehabilitating of fatigue-damaged structures; and
- Development of a strategic plan for converting the specifications to an LRFD philosophy and a plan depicting future enhancements to the specifications.

1.3.2 Approach

The research consisted of an extensive review and analysis of theoretical and experimental research investigations related to the project topics. Unpublished works and currently ongoing research studies were also identified and obtained. The primary focus was to obtain the most up-to-date technical information that has materialized since the completion of NCHRP Project 17-10. The assembled information was evaluated and presented in the form of design equations, allowable stresses, improved methods of analysis, design details, construction tolerances, procedures for repair of structures, and recommendations for improved design and fabrication.

1.4 SCOPE OF THE INVESTIGATION

The planned work for this project consisted of the following thirteen tasks:

- **Task 1—Literature review, industry contacts, and survey.** Review relevant practice, performance data, research findings, and other information related to sign, signal, and luminaire structures. This information will be assembled from foreign and domestic technical literature and from unpublished experiences of engineers, owners, material suppliers, fabricators, and others. Information on actual field performance is of particular interest.
- **Task 2—Wind loads report.** Develop a report for consideration by HSCOBS that addresses the basis for the differences in the wind speed maps, the differences in design loads resulting from wind speed maps, and the treatment of gusts in the current specifications and the specifications proposed in NCHRP Project 17-10. Use the information in ANSI/ASCE 7-95 or successor document to develop gust effect factors using the procedures presented in its commentary for wind-sensitive structures.
- **Task 3—Interim report.** Submit an interim report, within 6 months, that documents the results of Tasks 1 and 2 and expands on the work plan for Tasks 4 through 13. Following project panel review of the interim report, meet with the panel to discuss the interim report and remaining tasks.
- **Task 4—Fatigue and vibration in noncantilevered support structures.** Develop a report for consideration by the AASHTO HSCOBS that addresses fatigue and vibration in noncantilevered support structures. The report shall include (a) proposed connection design detailing that minimizes fatigue effects for support structures other than cantilevered, (b) an evaluation of the effectiveness of gussets in reducing fatigue problems, (c) recommendations for appropriate design and fabrication guidelines, and (d) proposed vibration mitigation measures. All material shall be prepared in a format suitable for consideration for inclusion in the specifications.
- **Task 5—Foundations and anchor bolts.** Provide selection criteria and design guidance for support-structure foundations, including reinforced and unreinforced cast-in-place drilled piles, steel screw-in foundations, and spread footings. Include design details for anchor bolts. At a minimum, address anchorage type (hooked or straight), embedment length, and pretensioning.
- **Task 6—Drag coefficient transitions for multisided to round shapes.** Use existing research findings to develop drag coefficients for tapered poles that transition from multisided shapes to round shapes.
- **Task 7—Connection plate and base plate flatness tolerances.** Review current practice to identify the need for structure-specific connection plate and base plate flatness tolerances for erection. Recommend tolerances based on the findings.
- **Task 8—Bending about the diagonal for rectangular steel sections.** Establish strength and failure criteria for bending about the diagonal axis of rectangular sections. Recommend design guidelines.
- **Task 9—Fiber-reinforced composites.** Develop performance specifications and acceptance testing procedures for fiber-reinforced composite support structures.
- **Task 10—Design examples.** Prepare complete design examples that show good design practice for support structures while illustrating key features of the specifications. Examples shall cover a realistic number of structure types covered by the specification. At least one example shall be prepared for each material covered in the specification.
- **Task 11—Retrofit and rehabilitation of fatigue-damaged support structures.** Prepare a manual for retrofitting and rehabilitating fatigue-damaged support structures. It is anticipated that this manual can be developed using the experience of the states and fabricators. Guidance on repair/replacement decisions is also provided.
- **Task 12—Strategic plan for future enhancements.** Develop a strategic plan for future enhancements to the support structure specifications. In particular, a plan for conversion of the specifications to LRFD philosophy and format shall be developed. The strategic plan includes a discussion of data requirements and necessary experimental and analytical work needed for LRFD specifications.
- **Task 13—Final report.** Submit a final report that documents the entire research effort. The reports developed in Tasks 2, 4, and 11 and the design examples prepared in Task 10 shall be included as separate appendices. Where applicable, suggested changes to the specifications are recommended.

1.5 ORGANIZATION OF THE REPORT

Chapter 2 summarizes the work performed and findings that fulfill the 12 objectives of the project. Chapter 3 discusses the proposed technical revisions to the specifications and the recommended changes. Chapter 4 provides the study conclusions and suggested topics for future research. The appendixes provide detailed reports on selected topics.

CHAPTER 2

RESEARCH FINDINGS

The project initially focused on 12 topics that were identified as needing revisions and updates. The following is a list of the topics that were studied in this project:

- Impact of the newly adopted wind loading criteria and wind map;
- Treatment of wind-sensitive structures;
- Fatigue of noncantilevered support structures;
- Anchor bolt design details, including embedment length and pretensioning;
- Foundation selection criteria;
- Drag coefficient transition for multisided tapered poles where the cross-section is nearly round;
- Connection plate flatness criteria;
- Flexural strength of square or rectangular steel tube sections bent about the diagonal;
- Performance specifications and acceptance testing for fiber-reinforced composite members;
- Retrofitting and rehabilitating of fatigue-damaged structures;
- A set of complete and detailed design examples illustrating the use of the 2001 *Supports Specifications*; and
- Development of a strategic plan for converting the specifications to an LRFD philosophy and a plan depicting future enhancements to the specifications.

This chapter summarizes the work performed and the research findings. Where applicable, suggested changes to the specifications are described.

2.1 LITERATURE REVIEW, INDUSTRY CONTACTS, AND SURVEY

The work for the project included reviewing relevant practice; performance data; research findings; and other information related to sign, signal, and luminaire support structures. This information was assembled from foreign and domestic technical literature and from unpublished experiences of engineers, owners, material suppliers, fabricators, and others. Information on actual field performance was of particular interest.

It is recognized that a significant portion of NCHRP Project 17-10(2) work was of a practical nature, depending on field experiences, and was not readily available in the published literature. Hence, much of the information developed in the project was based on the experiences of state departments of transportation (DOTs), manufacturers of support structures, material suppliers, and engineers. Close contacts with these parties were maintained throughout the duration of the project and provided an essential ingredient to the success of the project. Information gained from these sources was studied and evaluated with special care, since no official peer-reviewed publication of the material is available. Based on the surveys of Task 1, the state DOTs and manufacturers with the most pertinent information related to Project 17-10(2) were identified. Typical state DOTs that were contacted included Wyoming, Florida, New York, California, Michigan, Texas, and Virginia. Typical manufacturers that were contacted included Hapco, Newmark, P&K Pole Products, Shakespeare, Strongwell, Union Metal, Valmont, Walpar, and Whatley. Visits, as necessary, were made to gather test data and discuss important details and design information.

2.1.1 State DOT Survey and Results

The survey sent by the research team to state DOTs, as well as the results of that survey, are given in Appendix A. Forty-seven agencies responded. Many state DOTs provided materials such as repair manuals, sample calculations, and sample designs that were referenced in later stages of this research. Some states also identified additional manufacturers that were contacted.

2.1.2 Manufacturers' Survey and Results

The survey sent by the research team to support structure manufacturers, as well as the results of that survey, are provided in Appendix A. In addition to survey responses, some of the support structure manufacturers provided sample drawings, test data, calculations, and product catalogs, which were reviewed as part of the project work. Additional manufacturers identified by state DOTs were contacted to expand on this information.

2.2 WIND LOADS REPORT

The 2001 *Supports Specifications* has been updated to reflect currently accepted wind engineering practice. Wind loading provisions in the specifications are based on ASCE 7-95 (*5*), with specific modifications for structural supports for highway signs, luminaires, and traffic signals. The new wind load provisions in the *Supports Specifications* are based on a 3-second gust wind speed, rather than a fastest-mile wind speed, and a new map has been adopted. An increase or reduction in calculated wind pressures result from the use of the updated wind map.

The main goal of this task is to provide an in-depth explanation of the new wind map (3-second gust) and its impact on the calculated design wind pressures on structural supports. In addition, the flexibility of the structural support as it affects the gust effect factors is considered, and methods for incorporating the flexibility of the structure in the wind loading computation are evaluated.

A complete document entitled "Wind Loads Report" is provided in Appendix B. It provides a detailed study of the new wind load provisions in the 2001 *Supports Specifications*. More specifically, the report addresses the following:

- The basis for wind pressure calculations in the 1994 and 2001 specifications,
- Wind load comparisons between the 1994 and the 2001 specifications,
- Recommendations for gust effect factors for flexible structures, and
- Suggested research needs pertaining to wind loads for structural supports.

Analytical studies performed showed that the new wind provisions will generally result in wind provisions that will be significantly different from those calculated using the 1994 *Supports Specifications*. On average, wind loads on support structures, except for an increase for roadside signs, computed in accordance with the new wind provisions are comparable to the loads computed using the 1994 *Supports Specifications*. In comparing the 1994 and 2001 *Supports Specifications* wind load provisions, it is apparent that changes in wind pressure, either decreasing or increasing, are highly site specific. The wind pressures will double for a short street-lighting pole in Mobile, Alabama, or Orlando, Florida. The wind pressures will be reduced by half for a roadside sign in Hawaii.

The effect of the flexibility of support structures on the gust effect factor was studied. Equations provided by ASCE 7-98 for wind-sensitive structures were reviewed to determine the applicability to support structures. Simplified equations were developed for high mast lighting and traffic signal structures. These equations should be used with caution and may result in an increase in wind pressure on the structures. Until further verification of these equations using wind tunnel testing, it is recommended that they be used for general guidance.

2.3 FATIGUE AND VIBRATION IN NONCANTILEVERED SUPPORT STRUCTURES

The 1994 edition of the *Supports Specifications* provides little guidance on the design for fatigue and vibration. NCHRP Project 17-10 introduced a new section entitled "Fatigue Design," which is based primarily on the work of NCHRP Project 10-38 (*6*). The "Fatigue Design" section of the *Supports Specifications* addresses fatigue design of cantilevered steel and aluminum structures. Noncantilevered structures were not included, since such structures were not within the scope of NCHRP Project 10-38. Therefore, the overall objective of the task represented by this section of the report was to address fatigue and vibration in noncantilevered support structures.

The results of the survey sent to state DOTs indicated concerns for fatigue of noncantilevered support structures. Out of the 48 replies received, 8 identified problems with noncantilevered support structures. State DOTs that indicated having experienced problems were contacted. Also, the research team met with researchers of NCHRP Project 10-38(2) and visited a local sign support manufacturer. The effort included a comprehensive literature review of up-to-date publications regarding the fatigue of support structures. Related issues such as vibration mitigation methods and the effects of gussets were also investigated. Finally, an in-depth analytical investigation, using finite element analysis, was conducted to determine equivalent static fatigue loads for noncantilevered structures. This section summarizes the major research findings of this work. A detailed report documenting the analytical procedures and results is presented in Appendix C. The DOT survey results regarding fatigue and vibration in noncantilevered support structures is provided in the appendix, as well as an extensive reference list.

2.3.1 Susceptibility of Noncantilevered Support Structures to Wind-Induced Vibration

Fatigue and vibration problems that have occurred in noncantilevered support structures were investigated. However, the question of susceptibility to wind vibration phenomena such as galloping and vortex shedding is unresolved. A presentation of the different wind loading phenomena and why they occur, along with a discussion of the susceptibility issue, is provided in Appendix C, where it is pointed out that laboratory testing and further field evaluation are needed to determine susceptibility. Such testing was not included in the scope of this project. However, the research continued with the task of establishing equivalent static fatigue loads by assuming that noncantilevered structures were susceptible to the same vibration phenomena as cantilevered support structures and by using the finite element methods as an analysis tool. As a result of communication with researchers, manufacturers, and state DOT engineers regarding susceptibility issues, galloping loads were excluded for truss support structures while the loads for other types of noncantilevered support structures were adjusted using importance factors that are similar to the factors used for cantilevered structures.

2.3.2 Establishing Equivalent Static Fatigue Loads for Noncantilevered Support Structures

Finite element analyses were conducted using six support structure models to determine the applicability of the fatigue loads for cantilevered support structures recommended by NCHRP Project 10-38 to noncantilevered support structures. The analytical study, which is presented in Appendix C, resulted in the following load recommendations for noncantilevered support structures:

- **Galloping.** Apply 21-psf shear pressure vertically to the projected area of the signs mounted to monotube support structures as viewed in the normal elevation (same as cantilevered support structures). Galloping will only apply to horizontal monotubes; noncantilever structures and truss-type supports are excluded.
- **Vortex shedding.** Apply the static load model for vortex shedding of a cantilevered nontapered element in the support specifications suggested by NCHRP Project 17-10 to the nontapered element in the noncantilevered support structure. This model involves the transverse application of the equivalent static pressure range to a nontapered horizontal member (vertical direction) of the monotube structures. This design requirement may be disregarded as long as the signs or sign blanks are used during construction.
- **Natural wind loads.** Use the value of 5.2 psf multiplied by the drag coefficient, which is the same value as that used for cantilevered structures. The natural wind gust pressure range shall be applied in the horizontal direction to the exposed area of all support structure members, signs, and attachments.
- **Truck-induced loads.** Use 7.5 psf for the horizontal pressure applied to the area of the sign and the area of the support structures and 10.2 psf for vertical pressure applied to the area of the support structure and the projected area of the sign. These pressures should be applied along the entire span of the structure or along 24 feet of the span, whichever is smaller.
- **Deflection.** Use the limits as provided in the current specifications of L/150 for deflection due to dead load and ice load and not excessive for vertical deflection due to galloping and vertical truck-induced fatigue loads.

It should be pointed out that the recommendations were based on the analysis of selected support structures. Experimental work is needed to verify the susceptibility of these structures to the vibration phenomenon. Because of the lack of such experimental work and fatigue case documentation, the research team used fatigue design loads developed from the analytical study as a basis to correlate the loads for noncantilevered support structures with the loads used for cantilevered structures and applied importance factors to adjust these load values. Importance factors for noncantilevered support structures were based on discussions with other researchers and NCHRP Project 17-10(2) panel comments. These factors are similar to the factors for cantilevered sign support structures. Galloping was excluded as a loading case for truss support structures because other researchers indicated that noncantilevered truss support structures are not susceptible to galloping. Section 2.3.6 introduces importance factors for vibration and fatigue design for noncantilevered sign support structures.

2.3.3 Fatigue Categorization of Connection Details for Noncantilevered Support Structures

The research team reviewed sign support structure drawings and specifications provided by state DOTs. The research team also studied NCHRP reports dealing with fatigue in bridge connections that provided the basis for categorization in the *AASHTO LRFD Bridge Design Specifications* (*7*). Details for noncantilevered support structures were compared with the cantilevered structure details and were categorized. The various types of connections, the factors that affect the fatigue resistance of welded connections, and the fatigue resistance of bolted connections were synthesized. Categorization of details was suggested. Ten examples of these connection details are presented in Appendix C. The connections included the typical beam column connections in monotube support structures, splice joints in monotube support structures, and U-bolts used in the chord to the upright connection in truss structures.

2.3.4 Effectiveness of Gussets in Sign Support Structures

Results of the effectiveness of gussets in sign support structures were based on a study by Gilani and Whittaker (*8*, *9*). An example that demonstrates the effect of gusset plates was provided. Providing gusset plates was recommended to increase the moment capacity of the connection. The moment capacity can be further increased by appropriate design of the gusset.

2.3.5 Vibration Mitigation Measures in Sign Support Structures

Various vibration mitigation measures were studied. These include those considered by Kaczinski et al. (*6*), Cook et al. in Florida (*9a*), and Hamilton et al. in Wyoming (*9b*). The effectiveness of these measures in increasing the damping in support structures was discussed. A brief analytical study was conducted to demonstrate the effect of increased damping. Some of the devices were shown to increase the damping; however, the only way to verify the effectiveness of vibration mitigation is through testing and/or monitoring of structures in service.

2.3.6 Suggested Additions to Section 11 of the 2001 Specifications

<u>Add to Specifications, Section 11.4:</u>

d) Noncantilevered overhead sign and traffic signal support structures

<u>Add to Specifications, Table 11-1:</u>

Importance Factors for Vibration and Fatigue Design for Noncantilevered Sign Support Structures

Category	Structure Type	Importance Factors			
		Galloping	Vortex shedding	Natural Wind	Truck Gust
I	Truss	X	X	1.0	1.0
	Monotube	1.0	X*	1.0	1.0
II	Truss	X	X	0.85	0.9
	Monotube	0.72	X*	0.85	0.9

* Use the value of 1.0 for construction stage if signs or sign blanks are not used.

<u>Change to Specifications, Section 11.7.1:</u>

From "Overhead cantilever . . ." to "Overhead cantilevered and noncantilevered . . ."

<u>Add to Specifications, Section 11.7.1, Commentary:</u>

Galloping loads are applied to monotube noncantilevered support structures. Overhead noncantilevered truss-type support structures do not appear to be susceptible to galloping because of their inherently high degree of three-dimensional rigidity.

<u>Add to Specifications, Section 11.7.2:</u>

The equivalent static pressure P_{VS} shall be applied transversely (vertical direction) to nontapered horizontal monotubes of noncantilevered support structures if signs or sign blanks are not used during construction.

<u>Change to Specifications, Section 11.7.3:</u>

From "Overhead cantilever . . ." to "Overhead cantilevered and noncantilevered . . ."

<u>Add to Specifications, Section 11.7.4:</u>

Noncantilevered overhead sign and traffic signal support structures shall be designed to resist the following static, truck-induced, gust pressure range of

$$P_{TG} = 360 C_d I_F \text{ (Pa)} \qquad \text{Eq. 11-7}$$

$$P_{TG} = 7.5\ C_d I_F \text{ (psf)}$$

$$P_{TG} = 490 C_d I_F \text{ (Pa)} \qquad \text{Eq. 11-8}$$

$$P_{TG} = 10.2 C_d I_F \text{ (psf)}$$

The pressure range given by Eq. 11-7 shall be applied horizontally to the area of signs and horizontal members, while the values given by Eq. 11-8 shall be applied in the vertical direction to the area of the structure elements and the projected area of the sign. These pressures should be applied along the entire span of the structure or 24 feet (8 meters) of the span, whichever is smaller.

<u>Add to Commentary, Section 11.7.4:</u>

Regarding the noncantilevered sign support structures, the pressure values were based on analytical work of project NCHRP Project 17-10(2) considering truck-induced pressure values measured by Cook (1996). The values measured were applied to four noncantilever sign support structures as a dynamic load, and the equivalent static loads were obtained. The average value of the static loads was then multiplied by the factor of 1.3 to account for increase in the relative truck speed due to head wind.

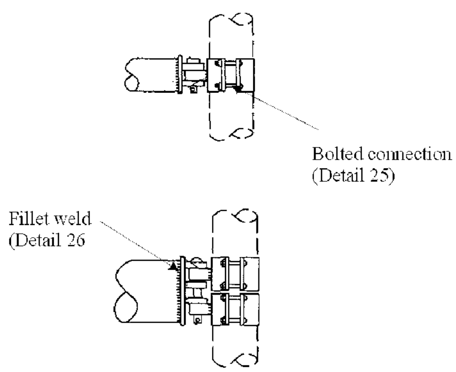
<u>Change to Specifications, Section 11.8 (Deflection):</u>

From "cantilevered single-arm . . ." to "cantilevered single-arm and noncantilevered monotube . . ."

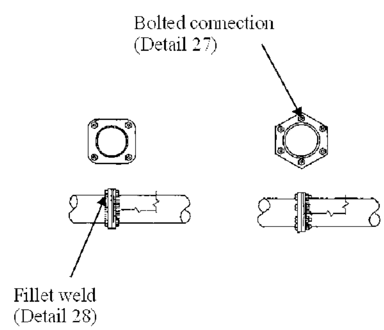
Add to Specifications, Table 11-2:

Construction	Detail	Stress Category	Application	Example
Mechanically Fastened Connections	25. Bolts in tension	D	Beam column connection for monotube structures	16
Fillet-Welded Connections	26. Fillet weld with one side normal to the applied stress	E'	Beam column connection for monotube structures	16
Mechanically Fastened Connections	27. High-strength bolts in tension	D	Monotube or truss-chord splice	17
Fillet-Welded Connections	28. Fillet weld with one side normal to the applied stress	E'	Monotube or truss-chord splice	17
Mechanically Fastened Connections	29. U-bolts tied to the transverse truss column to keep the chords in place	D	Horizontal truss connection with the vertical truss	18
Mechanically Fastened Connections	30. Net section of full-tightened, high-tension bolts in shear	B	Truss-bolted joint	18

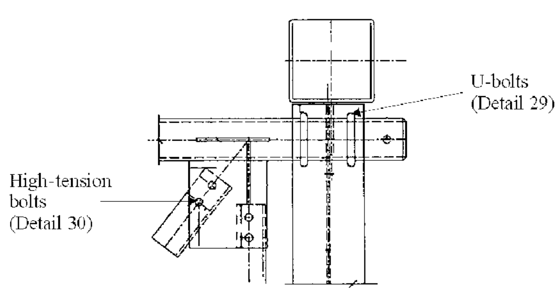
Add to the Specifications, Figure 11-1:



Beam-Column Connection for monotube structure
Example 16



Monotube beam splice
Example 17



Beam-Column Connection for Truss Structure
Example 18

2.4 ANCHORAGE TO CONCRETE

The 2001 *Supports Specifications* document does not address the design of anchorage to concrete. NCHRP Project 17-10 proposed a revision to the 1994 AASHTO *Supports Specifications* and addressed the issue of anchor bolt design, among other topics (*10*, *11*). On the issue of anchor bolt design, the topics addressed were anchor bolt materials, allowable stresses, the use of hooked bolts, minimum embedment length of headed cast-in-place anchor bolts, the effect of edge distance, and the effect of spacing between anchor bolts.

The latest American Concrete Institute (ACI) anchorage procedure was evaluated. This procedure has been published by a number of sources, including the ASCE (*12*), the International Conference of Building Officials (*13*), and Cook (*14*). The procedure was published as Appendix D in the 2002 edition of the ACI 318 *Building Code Requirements for Structural Concrete and Commentary*.

The research team obtained an updated copy of this procedure, which includes the use of anchor bolts over 25 inches in length outlined in document CB-30, dated June of 2000 (*16*). This is the current state of the art in anchorage to concrete. The procedure is presented in a LRFD format. Using a wind load factor of 1.3 in accordance with ACI 318 (*15*), and the appropriate strength reduction factors (ϕ) (*16*), the equations have been converted for use in the *Supports Specifications*.

2.4.1 *NCHRP Report 411* Recommendations

The 1994 edition of the *Supports Specifications* (*2*) contains little information on the design of embedment length, effect of edge distance, spacing of anchors, or fatigue of anchors in tension (*1*). NCHRP Project 17-10 reviewed 17 documents and proposed a revision to the anchor bolt section of the 1994 *Supports Specifications*. The project addressed the following issues:

- Anchor bolt material,
- Allowable stresses, and
- The use of hooked bolts.

Appendix C was added to the report and addressed these topics:

- Minimum embedment length of headed cast-in-place anchor bolts,
- The effect of edge distance, and
- The effect of spacing between anchor bolts.

However, Appendix C was not included in the 2001 AASHTO *Supports Specifications* (*4*).

2.4.1.1 Anchor Bolt Materials

The 1994 *Supports Specifications* allowed many different types of anchoring materials, including anchor bolts, bolts made from reinforcing steel, and other threaded materials (*1*). *NCHRP Report 411* proposed that anchor materials be limited to grades 36-, 55-, and 105-ksi steel because of information from AASHTO M314 and other ASTM standards.

Tension in the anchor bolt is resisted by bearing of the embedded anchor end on the concrete. The bearing area may be provided by one of the following:

- Headed bolt,
- Headed bolt with washer,
- Nut,
- Nut with washer, and
- Hook of bent bolt.

2.4.1.2 Allowable Stress

NCHRP Report 411 compared the allowable tensile and shear stresses in the current *Supports Specifications* to those in the American Institute of Steel Construction (AISC) manual. The failure mode of the 1994 *Supports Specifications* is based on the yield stress and tensile stress area. It allowed for a tensile stress of 50% of the yield stress and an allowable shear stress of 30% of the yield stress. In the AISC ASD procedure, the allowable tensile stress is limited to 33% of the ultimate stress, and the allowable shear stress is limited to 17% of the ultimate stress (*17*). This limit differs from steel design in the AISC LRFD manual, which bases its failure on ultimate strength and nominal area (*18*). *NCHRP Report 411* recommended that the current *Supports Specifications* allowable stresses remain at 50% of yield stress (F_y) for tensile loads because it was the more conservative value (*1*). This cautious approach seemed appropriate because of the lack of information at the time that the report was written.

2.4.1.3 Use of Hooked Bolts

NCHRP Report 411 recommended the use of headed anchor bolts over hooked bolts, but allowed hooked bolts up to 55-ksi steel, since research showed that hooked anchor bolts made from higher-strength steel did not develop their full tensile strength. Headed bolts give much more efficient pullout strengths than hooked bolts because hooked bolts can straighten and pull out under high tensile loads, and crushing in the concrete occurs at the area of the hook. It also noted that AISC LRFD (*18*) states that high-strength steels are not recommended for use in hooked bolts because bending with heat might materially alter the steel's strength. Also, ACI 355.1R-91 states that hooked bolts have been known to straighten out in pullout tests, and headed anchors of the same size and length have significantly higher capacities (*19*).

Nevertheless, hooked anchor bolts should be allowed because a larger-diameter hooked bolt may be cheaper than a smaller-diameter headed bolt of equivalent capacity.

2.4.2 Review of Current State Transportation Agency Practice

State transportation agencies were asked about typical bolt materials, diameters, and lengths. The results are reported in Tables 15 to 21 of Appendix A. The survey results indicated that a wide variety of anchor bolt strengths are in use, with ASTM A36, A307, and A325 being the most common. Bolt diameters ranged from a minimum of 0.5 inch to a maximum of 2.5 to 3.25 inches, depending on the type of structure. Average bolt diameters were between 1 and 2 inches for all structure types investigated.

Anchor bolt lengths varied considerably. For street light poles, bolt lengths ranged from 12 to 90 inches, with an average of approximately 40 inches. For overhead cantilever structures, bolt lengths ranged from 30 to 180 inches, with an average of approximately 58 inches. Reported anchor bolt lengths were between these values for other structure types. The longest values reported by state transportation agencies exceeded the shortest by a factor of 4 to 7.5, depending on type of structure, indicating that there is a lack of standardization in design procedures.

Both hooked and straight anchor bolts are commonly used. Hooked bolts are used most commonly for lightly loaded structures such as traffic signal supports and street light supports (60% to 70% of structures). Straight anchor bolts are often used for heavily loaded structure types such as overhead bridge, overhead cantilever, and high-level lighting poles (42% to 63% of structures). Pretensioned bolts are rarely used, with only 5% to 10.5% usage reported, depending on structure type.

2.4.3 Review of the ACI Anchor Bolt Design Procedure

As of the late 1990s, the main source of information on cast-in-place anchors to concrete was ACI 349 Appendix B (*20*) and the *PCI Design Handbook* (*21*). Recently, the proposed Appendix D of ACI 318-02—published in Appendixes A and B of Portland Cement Association publication *Strength Design of Anchorage to Concrete* (*14*), in ASCE Standard 7-98 (*12*), and in the 2000 *International Building Code* (*13*)—has become the most current and complete information on anchorage to concrete.

This procedure has been in development since the mid 1970s, when ACI 349 Appendix B (*20*) and *PCI Design Handbook* (*21*) adopted the 45-degree cone method. The 45-degree cone method was developed from tests conducted at the University of Tennessee for the Tennessee Valley Authority. In the late 1980s, tests at the University of Stuttgart led to the development of the Kappa method. This development was based on tests conducted on different types of anchor materials, with a variety of embedment lengths and edge distances.

In the mid-1990s, the Kappa method was refined and made easier to use by the original developers of the Kappa method, which resulted in the Concrete Capacity Design (CCD) method. Meanwhile, an international database was compiled from anchor tests and field performance. ACI committees 318, 349, and 355 evaluated the CCD method and 45-degree cone method using this database. The result was to recommend the CCD method proposed as ACI 318 Appendix D (*15*). In this method, the concrete breakout strength calculations are based on the same model suggested by the Kappa method, which assumes a prism angle of 35 degrees.

The design procedure is based on ultimate strength design. The procedure applies to headed studs, headed bolts, hooked bolts, and cast-in-place anchors, but not post-installed anchors. The procedure is for static loads and does not apply to fatigue or impact loading. The procedure is not for loads from blasts and shock waves.

For bolts more than 25 inches long, additional reinforcement in the direction of the load must be added (*16*). Appendix D of the 2002 edition of the ACI 318 is outlined in the June 2000 revision of draft *CB-30* (*16*). The ACI procedure refers to Chapter 12 of ACI 318 for attaching rebar to the anchor (*15*) in cases where anchors are spliced to reinforcing steel. The procedure addresses both the strength of the steel anchors and the strength related to the embedment in the concrete, with the smallest capacity controlling the design. The strength of the steel anchors is based on the steel properties and size of the anchor. The strength of the embedment is based on embedment depth, spacing, concrete strength, edge distance, and size of the head or hook. Concrete strengths should be limited to 10,000 psi with this procedure.

2.4.4 Basic Principles

The ACI procedure provides for tension and shear failure modes and for tension-shear interaction. Tension failure modes include steel strength of anchor in tension, concrete breakout strength of anchor in tension, pullout strength of anchor in tension, and concrete side-face blowout strength of anchor in tension. If concrete breakout and concrete side-face blowout are prevented by using confining reinforcement in the foundation, then only the steel strength of the anchor and the pullout strength of the anchor need to be considered for determining tension capacity.

Shear failure modes include steel strength of anchor in shear and concrete breakout strength of anchor in shear. If confining reinforcement is provided, concrete breakout in shear can be prevented. Failure may also occur through interaction between tension and shear.

2.4.5 Proposed Simplified Procedure

A simplified design procedure for anchor bolt design has been developed. It is primarily based on the new provisions on anchoring to concrete that are provided as Appendix D in the 2002 edition of ACI 318 *Building Code Requirements for Structural Concrete and Commentary*. The procedure is limited to the following conditions:

- Anchor bolts are hooked or headed;
- Foundations have vertical reinforcing steel and confining reinforcement, with anchor bolts placed inside of the reinforcement;
- Foundation-reinforcing steel is uncoated; and
- If hooked bolts are used, the length of the hook is at least 4.5 times the anchor bolt diameter.

For cases where these assumptions do not apply, such as when lightly loaded bolts are embedded in an unreinforced foundation, designers should refer to the complete procedure as documented in several sources (*12–15*). It is assumed that shear will not govern capacity, since the high aspect ratio of support structures ensures that tension bolt forces due to overturning moment are higher than bolt shear forces by a factor of 14 or more. Moreover, the use of confining reinforcement will prevent shear breakout.

Briefly, the procedure requires determination of (a) the required anchor bolt diameter and bearing area for headed bolts or (b) the required anchor bolt diameter for hooked bolts. Next, the bolt length is determined so that the failure plane will intersect the foundation-reinforcing steel below the point where the capacity of the reinforcing steel is fully developed. These equations are developed using a wind load factor of 1.3 and the appropriate ACI strength reduction factors (ϕ) of 0.80 for steel strength and 0.75 for concrete bearing, solving for required diameters.

Previous design procedures did not account for anchor bolt head or hook bearing against concrete. High bearing stresses can lead to localized concrete crushing, loose anchor bolts, and, thus, poor structure performance.

2.4.5.1 Headed Bolt Diameter

The category of headed bolts includes (a) bolts with head-bearing area increased by using nuts or washers and (b) threaded rods with nuts, with or without washers. The required bolt diameter, based on steel strength, is

$$d_{\text{anchor}} = 1.44\sqrt{\frac{N}{F_{ut}}} \qquad \text{(Eq. 1)}$$

Where

N = tensile force on anchor bolt and
F_{ut} = ultimate tensile strength of bolt, but not more than 1.9 times the bolt yield strength (1.9 F_y), or 125,000 psi.

2.4.5.2 Threaded Rod Adjustment

For threaded bolts, the bolt diameter, d_{anchor}, must be adjusted to account for the threads. The adjusted diameter, d'_{anchor}, is increased as follows, using an adaptation of a formula from the CB-30 procedure commentary (*16*):

$$d'_{\text{anchor}} = d_{\text{anchor}}\left(1 + \frac{0.9743}{n_t}\right) \qquad \text{(Eq. 2)}$$

Where

n_t = number of threads per inch.

2.4.5.3 Bolt Head Diameter

The required bearing area of the anchor bolt head, A_b, is

$$A_b = 0.191\frac{N}{f'_c} \qquad \text{(Eq. 3)}$$

Where

f'_c = specified compressive strength of the concrete.

Bearing areas for various bolt sizes are provided in Table 2 of Cook (*14*). Therefore, for a round washer, the required bolt head diameter, d_{head}, is

$$d_{\text{head}} = \sqrt{\frac{4}{\pi}A_b + d'^{2}_{\text{anchor}}} \qquad \text{(Eq. 4)}$$

For this equation, the actual anchor bolt diameter, not the minimum, must be used. This requirement is important because the required bolt diameter will be rounded up to the next available size. If the actual diameter is not used in the equation above, the equation will not necessarily provide sufficient bearing area. The required area may be provided by a bolt head, by a nut, or by a washer on top of a bolt head or nut.

2.4.5.4 Hooked Bolt Diameter

The required bolt diameter to prevent crushing failure of the concrete against the hook is

$$d_{\text{anchor}} = 0.614\sqrt{\frac{N}{f'_c}} \qquad \text{(Eq. 5)}$$

This formula assumes that the distance from the inner surface of the shaft of a J-bolt or L-bolt to the outer tip of the J-bolt or L-bolt (e_h) is a minimum of 4.5 times the anchor diameter.

Anchor bolt steel strength should also be checked. However, combining the first equation with the hooked bolt equation indicates that the concrete bearing strength will govern unless concrete strength f_c' is at least 18% of F_{ut}. Since A36 steel would require concrete with a strength of over 10,400 psi, it may safely be assumed that steel strength will not govern capacity for hooked bolts.

2.4.5.5 Anchor Bolt Length

The length of anchor embedded in the concrete, l_{anchor}, must be sufficient to develop the yield strength of the reinforcing steel, as described earlier.

$$\ell_{\text{anchor}} = \ell_d + c_{\text{top}} + s_{\text{anchor}} \quad \text{(Eq. 6)}$$

Where

ℓ_d = required reinforcing bar development length,
c_{top} = clearance between top of vertical reinforcing steel and top of foundation, and
s_{anchor} = horizontal center-to-center spacing between anchor bolt and vertical reinforcing steel.

For Gr. 60 Number 6 and smaller bars, the required bar development length is

$$\ell_d = \frac{2,400}{\sqrt{f_c'}} d_b \quad \text{(Eq. 7)}$$

Where

d_b = diameter of the bar.

For Gr. 60 Number 7 and larger bars, the required bar development length is

$$\ell_d = \frac{3,000}{\sqrt{f_c'}} d_b \quad \text{(Eq. 8)}$$

For epoxy-coated bars, the bar development length must be increased by 20% or 50% according to the provisions of ACI 318 (*15*). These equations only provide for the anchor bolt length within the concrete—the portion of the bolt protruding above the concrete must be added to determine the total length of the bolt.

2.4.6 Sample Calculations and Discussion

In order to determine how the use of this procedure would affect the design of anchorage for support structure foundations, comparisons were made for anchor bolts with varying applied forces, three grades of steel, and bolt diameters of up to 2 inches. Shear loads were assumed to be negligible. Concrete breakout and concrete side-face blowout were assumed to be controlled by adequate vertical reinforcement and shear reinforcement (confining steel). For all support structure foundations, 3,000-psi concrete was used. Results are shown in Table 1.

Bolt sizes, as determined by the steel in tension, were compared for the 2001 *Supports Specifications* and the proposed method using Equations 1 and 2. The bolt sizes are shown in Columns 4 and 5 in Table 1. Bolt sizes range from 0 to 0.5 inches smaller for the proposed procedure.

It is also important to consider the bearing area, since localized crushing failure of the concrete against the head of the bolt could lead to bolt looseness and vibration of the support structure. (Equation 3 is used to determine the minimum bearing area of a headed bolt. (Equation 5 is used to determine the minimum bearing area of a hooked anchor bolt. When using the proposed procedure for headed bolts without washers, the bearing area of the bolt head or nut, rather than the steel strength, governs the bolt size. This is shown by comparing Column 5 and 7 in Table 1. When using the proposed procedure for headed bolts with washers, the bearing area of a hardened washer is usually adequate for bolt sizes determined by steel strength, except for the 105-ksi yield strength bolts. This is shown by comparing Column 5 and 6 in Table 1. When using the proposed procedure for hooked bolts, the bearing area of the hook, rather than the steel strength, governs the bolt size. This is shown by comparing Column 8 to Columns 5, 6, and 7 in Table 1. The required diameter for hooked anchor bolt is significantly higher than it is for headed bolts.

For headed bolts without washers, tensile capacity is governed by the minimum required bearing area, rather than by the steel strength. As a result, there is no reduction in bolt diameter for using higher strength steel, such as the Grade 55 or 105-ksi steel, as shown in Column 7 in Table 1. The same conclusion applies for hooked anchor bolts, as shown in Column 8.

Required embedment length of a headed anchor bolt was calculated for the pullout capacity of the concrete, using the 35-degree failure cone assumed in the ACI anchorage design procedure (*14, 16*). Typical required embedment depths were 16 to 24 inches for loads of approximately 50 to 90 kips. However, this calculation does not account for the reduction in capacity due to overlapping failure cones or edge distance. In order to achieve this capacity, bolt spacing and/or edge distance must be 48 to 72 inches—clearly impractical.

2.4.7 Transfer of Anchor Bolt Load to Foundation Reinforcement

For most support structures, it is not practical to resist bolt tension loads through concrete breakout strength. A different mechanism for resisting these loads is necessary. Therefore, the anchor bolt tension forces should be transferred to the foundation-reinforcing steel through the mechanism illustrated in Figure 1. In order to use this mechanism, the anchor bolts must be inside the foundation reinforcement cage.

The previous design conditions, addressed above in Section 2.4.6, ensure that the bolt diameter is large enough to

TABLE 1 Calculated bolt sizes

Tension in Bolt (kips)	F_y (ksi)	F_{ut} (ksi)	2001 Spec.	Proposed Method			
			Min. Bolt Dia. Based on Steel Strength (in)	Min. Bolt Dia. Based on Steel Strength (in)	Min. Bolt Dia. Based on Bearing Area with Hardened Washer (in)	Min. Bolt Dia. Based on Bearing of Hex Head or Nut (in)	Min. Bolt Dia. Based on Bearing of Hooked Bolt (in)
3	36	58	1/2"	1/2"	1/2"	1/2"	5/8"
10	36	58	7/8"	3/4"	3/4"	3/4"	1-1/8"
20	36	58	1-1/4"	1"	1"	1-1/8"	1-5/8"
30	36	58	1-1/2"	1-1/4"	1-1/4"	1-3/8"	2"
40	36	58	1-3/4"	1-3/8"	1-3/8"	1-1/2"	-
50	36	58	2"	1-1/2"	1-1/2"	1-3/4" Heavy Hex	-
60	36	58	2"	1-3/4"	1-3/4"	1-3/4" Heavy Hex	-
5	55	75	1/2"	3/8"	3/8"	1/2"	7/8"
10	55	75	3/4"	5/8"	5/8"	3/4"	1-1/8"
20	55	75	1"	7/8"	7/8"	1-1/8"	1-5/8"
30	55	75	1-1/4"	1-1/8"	1-1/8"	1-3/8"	2"
40	55	75	1-3/8"	1-1/4"	1-1/4"	1-1/2"	-
50	55	75	1-1/2"	1-3/8"	1-3/8"	1-3/4" Heavy Hex	-
60	55	75	1-3/4"	1-1/2"	1-1/2"	1-3/4" Heavy Hex	-
70	55	75	1-3/4"	1-3/4"	1-3/4"	2" Heavy Hex	-
80	55	75	2"	1-3/4"	1-3/4"	-	-
90	55	75	2"	2"	2"	-	-
10	105	125	1/2"	1/2"	1/2"	3/4"	1-1/8"
20	105	125	3/4"	3/4"	3/4"	1-1/8"	1-5/8"
30	105	125	7/8"	7/8"	1"	1-3/8"	2"
40	105	125	1"	1"	1-1/8"	1-1/2"	-
50	105	125	1-1/8"	1-1/8"	1-1/4"	1-3/4" Heavy Hex	-
60	105	125	1-1/4"	1-1/4"	1-3/8"	1-3/4" Heavy Hex	-
70	105	125	1-3/8"	1-1/4"	1-3/8"	2" Heavy Hex	-
80	105	125	1-3/8"	1-3/8"	1-1/2"	-	-
90	105	125	1-1/2"	1-1/2"	1-3/4"	-	-
100	105	125	1-3/4"	1-1/2"	1-3/4"	-	-
110	105	125	1-3/4"	1-3/4"	2"	-	-
120	105	125	1-3/4"	1-3/4"	2"	-	-
130	105	125	1-3/4"	1-3/4"	-	-	-
140	105	125	2"	1-3/4"	-	-	-
150	105	125	2"	2"	-	-	-
160	105	125	2"	2"	-	-	-
170	105	125	2"	2"	-	-	-

prevent failure and that the hook or head area is large enough to prevent concrete crushing. Therefore, it is assumed that, as a worst case, a crack develops just below the head or hook of the bolt and propagates through the foundation at a 35-degree angle, which is consistent with the assumed ACI failure cone. This failure plane intersects the foundation-reinforcing steel, which provides the rest of the anchorage load path.

In order to develop this mechanism, the required anchor bolt length is equal to the development length of the reinforcing bar, plus the clearance to the top of the foundation (c_{top}), plus the spacing between the bar and the anchor (s_{anchor}). This last requirement conservatively assumes a 45-degree failure plane.

For example, assume a foundation is reinforced with Number 8 reinforcing bars, and 3,000-psi concrete is used. The required development length for these reinforcing bars is 55 inches. If the top clearance is 2 inches and the spacing between the anchors and longitudinal reinforcement is 9 inches, then anchor bolts 66 inches long should be used. This is somewhat longer than the average value for anchor bolts reported by state transportation agencies, but much less than the maximum value. The development length was calculated using conservative, default values for all parameters. A more rigorous calculation using the new provisions of ACI 318 Code (*14, 16*) would probably result in a shorter required bolt length.

2.4.8 Recommendations for Anchor Bolt and Foundation Design

For headed anchor bolts, the benefit of using higher-strength steels is limited. Required bolt diameter is reduced, but the head diameter cannot be reduced because it is governed by concrete bearing stress. Furthermore, lower-yield steels are more ductile. Because of the upper limit of 125,000 psi on F_{ut} in Equation 1, using steel with yield strength

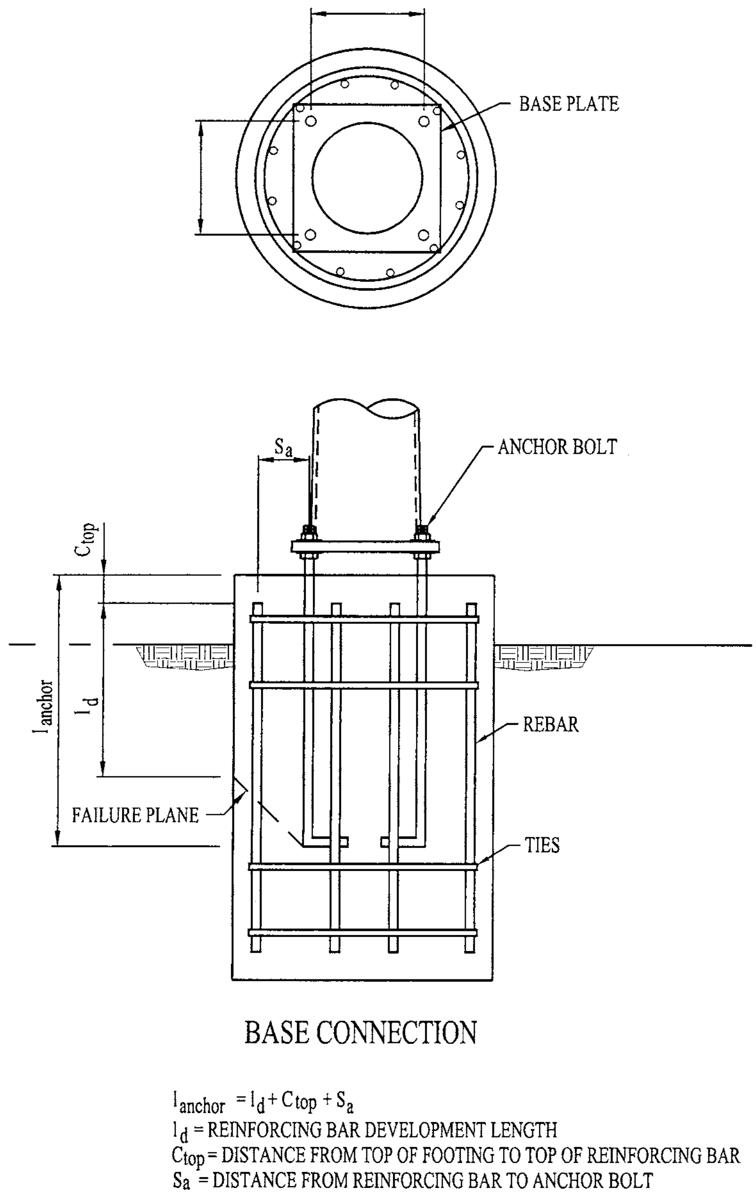


Figure 1. Development length of anchor bolt in a drilled shaft.

greater than 105,000 psi will not reduce the required bolt diameter.

There is no benefit in using higher-strength steels for hooked anchor bolts, since capacity is governed by the bearing stress of the hook on the concrete. Using higher-strength concrete for the foundation may reduce the required head diameter for headed bolts and the required bolt diameter for hooked bolts.

Typically, required diameters for hooked bolts will be much larger than the required diameters for headed bolts for 3,000-psi concrete. For lightly loaded structures, this difference may have little effect, since practical considerations may dictate the use of anchor bolts at least 0.5 inch in diameter. For heavily loaded structures, however, there may be a significant advantage to using headed bolts.

Because the anchor bolt length is governed by the development length of the foundation-reinforcing steel, the required anchor bolt length may be reduced by using smaller-diameter reinforcing bars or higher-strength concrete in the foundation. The calculations in Table 1 have been based on the use of 3,000-psi concrete. Using higher-strength concrete may reduce required bolt diameters and lengths and thus result in a lower overall cost of the support structure.

As an example, the required anchor bolt length for the example cited above can be reduced from 66 inches to 37 inches if 5,000-psi concrete and Number 6 reinforcing bars are used for the foundation. For a 1-inch and 1 3/8-inch headed bolt with and without a washer, respectively, bolt sizes that are designed for a 30-kip load and 5,000-psi concrete may be reduced to 7/8 inch with or without a washer. The diameter of the hooked bolt may be reduced from 2 inches to 1.5 inches if 5,000-psi concrete is used. Using higher-strength concrete reduces the required diameter for hooked bolts.

2.4.9 Pretensioned Anchor Bolts

As noted in Section 2.4.2, pretensioned anchor bolts are not widely used by state transportation agencies. The literature review did not uncover any references on the design or performance of pretensioned anchor bolts for support structure foundations. The ultimate strength of the anchor bolt in tension is not affected by pretensioning.

However, for designs where fatigue is an important consideration, pretensioning may offer a benefit by reducing the vibration of the structure and reducing the stress range in the anchor bolt. For these designs, use of lower-yield point ductile steel combined with pretensioning may provide better fatigue performance. However, this better fatigue performance needs to be verified through experimental research.

2.4.10 Summary and Recommendation

The simplified procedure developed above is applicable to most support structures. This procedure requires that the anchor bolts transfer the load to the foundation-reinforcing steel. For headed bolts with hardened washers, the simplified procedure may result in smaller bolt diameters than current practices, while headed bolts without washers will result in slightly larger bolt diameters. For hooked bolts, required diameters will be larger. The simplified procedure requires only seven equations, with at most five calculations required. Anchor bolt diameters and lengths may be reduced if higher-strength foundation concrete or smaller-diameter, foundation-reinforcing bars are used.

2.5 FOUNDATIONS

2.5.1 Introduction

The 2001 AASHTO *Supports Specifications* provided new information on foundation design, including

- Rational procedure for calculating embedment depth of laterally loaded drilled shafts and direct embedded poles (Brom's method),
- Eccentrically loaded spread footings, and
- Pile foundations.

This project was to provide additional information on foundation design, including

- Survey of state DOT practices regarding foundation design,
- Design guidance for foundation design, and
- Qualitative criteria for foundations.

Major documents reviewed included the *AASHTO LRFD Bridge Design Specifications* (*7*) and a number of other publications pertaining to foundations of structures (*1, 2, 4, 22–28*). A survey of all state DOTs was conducted. Information was requested on the types of foundations commonly used. Table 2 shows the frequency of use for different foundations by structure type.

From the survey results, reinforced drilled shafts are the most common type of foundation, particularly for overhead cantilevers, high-level lighting poles, and traffic light supports. Next in frequency of use are spread footings, used mostly for overhead cantilever and overhead bridge structures. The other three types of foundation are rarely used (less than 33% of the time) in support structure applications.

TABLE 2 Support structure foundation frequency of use

Structure Type	Reinforced cast-in-place drilled shafts	Unreinforced cast-in-place drilled shafts	Steel screw-in foundations	Spread footings	Directly embedded poles
Overhead Cantilever	Common	None	Rare	Intermediate	None
Overhead Bridge	Intermediate	None	Rare	Intermediate	None
Roadside Sign	Intermediate	Rare	Rare	Rare	Rare
Street Light Poles	Intermediate	Rare	Rare	Rare	Rare
High-Level Lighting Poles	Common	None	None	Rare	None
Traffic Signal Supports	Common	None	None	Rare	Rare
Span Wire Supports	Intermediate	None	None	Rare	Rare

Notation
Common = 67–100% of the states reporting use.
Intermediate = 34–66% of the states reporting use.
Rare = 1–33% of the states reporting use.
None = 0% of the states reporting use.

The selection of an appropriate foundation depends on the magnitude of the applied loads (wind/lateral and gravity), as well as the soil site characteristics. For lightly loaded structures, such as roadside signs, direct embedment provides an economical alternative. Steel screw-in foundations may be mechanically installed, speeding construction for a large number of similar structures. For larger structures, selection between drilled shafts and spread footings depends upon the magnitude of loads applied and soil properties. In resisting overturning moments, drilled shafts rely on depth of embedment, and spread footings rely on footing dimensions.

Several factors need to be considered for selecting the appropriate type of foundation for structural supports. Important considerations include the support structure type, structural stiffness, transmitted loads, soil properties, soil-structure interactions, groundwater conditions, depth to bedrocks, and the fixity of the ends of the pile. While many of these factors may require in-depth geotechnical investigations, the most fundamental parameters involved directly in the design of a foundation include

- Type of support structure,
- Soil strength parameters, and
- Magnitude and direction of applied loads.

Accurate assessment of soil properties (i.e., unit weight, cohesion, and soil friction angle) is critical to the stability of the structure. Heavily involved subsurface explorations are typically required for accurate assessment of soil strength parameters. *AASHTO LRFD Bridge Design Specifications* lists different recommended in situ soil-testing techniques for different soil types (*7*). The use of field-testing techniques such as the Standard Penetration Test (SPT) or the Cone Penetration Test (CPT) is addressed in ASTM testing standards. In some cases, in-depth determination of soil properties may not be possible for support structures. The 2001 *Supports Specifications* allows waiving of subsurface exploration if the failure of the structure will not pose significant hazard. Nonetheless, the design engineer should practice good judgment when soil parameters are not available for design.

2.5.2 Drilled Shaft Foundation

A drilled shaft is a pile that is constructed by making an excavation in the soil with an auger or a drilling bucket. Drilled shafts may also be referred to as caisson, pier, drilled pile, or cast-in-place drilled pile. Laterally loaded drilled shafts can be used when the soil provides adequate lateral resistance.

Drilled shafts for sign structures can be loaded axially (limited cases), laterally, or a combination of both. Design of axially loaded drilled shafts is covered extensively in the bridge specifications. Contrasting to the design of bridges, support structures may involve single- or multiple-pile structures (*26*). In either case, the embedment depth is the most critical design factor. In the *Supports Specifications,* Brom's method can be used to determine the embedment depth for laterally loaded drilled shafts in both cohesive and cohesionless soil. Caution should be taken when assumptions are made for more complicated soils such as mixed cohesive/cohesionless soils.

For the state of Alaska, all footings for lighting standards are specified to be cast-in-place and should have a minimum depth of 6 feet for a 3-foot diameter shaft and a minimum depth of 9 feet for 2-foot diameter shaft (*29*). For traffic signal and accessories foundations, a minimum 3-foot depth is required. For traffic signal pole foundations, a minimum depth of 9 feet for all signal poles with 1 signal mast arm, and a depth of 12 feet for all signal poles with 2 signal mast arms, is required. For overhead tubular sign structures, the minimum drilled shaft depth is 19 feet, plus an adjustment for slope.

Other states also have required minimum depths for drilled shafts. Arizona uses a minimum depth of 16 feet, based on a uniform soil condition, with unit weight of 110 pcf, friction angle of 29 degrees, and modulus of subgrade *k* of 50 psf/ft (*30*). Some states, such as Indiana (*31*), specify required depths by the foundation types: Type I, 18 feet; Type II, 15 feet; Type III, 12 feet. And for specific structures, they specify minimum embedment depth of 15 feet for a double-arm sign structure, 8 feet for light foundations, and 20 feet for high-mast-tower foundations. For caisson or drill shaft footings, a minimum depth of 12 feet is required in Pennsylvania (*32*). Washington specifies 3 types of foundations for sign bridges and monotube signs, as shown in Table 3 (*33*).

Table 3 is reproduced herein to illustrate an example of state DOT foundation requirements. It shows correlations between span length and the shaft depth, but does not explicitly indicate soil effects. Soil effect is considered when selecting the foundation types. Foundation Types 1 and 2 are designed for a lateral bearing pressure of 2,500 psf. Type 3 is designed for lateral soil bearings between 1,500 psf and 2,500 psf. Type 2 is an alternative to Type 1 foundation.

Breakaway supports are typically used on light roadside sign structures. For breakaway sign support foundations, West Virginia specifies the minimum depth based on post sizes, as shown in Table 4 (*34*). For breakaway signposts, Iowa DOT limits the depth based on the steel post sections, as shown in Table 5 (*35*).

2.5.3 Screw-In Helix Foundations

Screw-in helix foundations consist of a galvanized round pipe or tube with a formed helix plate at the embedded end (Figure 2) and a connection base plate at the top end. The foundation derives its lateral load capacity from its length and diameter and the properties of the soil. Since most screw-in foundations are power installed, the base plate should have adequate thickness to resist wear and provide connection for street lighting poles or roadside signs. The base plate can be

TABLE 3 Foundation dimension for Washington DOT (*33*)

Type 1 foundation (drilled shaft)			
Structure types	Span length	Shaft diameter	Depth
Cantilever	< 20’	4’6”	12’6”
	20’ to 30’	4’6”	15’
Sign bridge	< 60’	4’6”	15’6”
	60’ to 90’	4’6”	19’
	90’ to 120’	5’	21’9”
	120’ to 150’	5’	23’6”
Type 2 foundation (drilled shaft)			
Structure types	Span length	Depth	
Cantilever	< 20’	7’3”	
	20’ to 30’	9’6”	
Sign bridge	< 60’	9’	
	60’ to 90’	11’	
	90’ to 120’	12’9”	
	120’ to 150’	13’6”	
Type 3 foundation (drill shaft)			
Structure types	Span length	Depth	
Cantilever	< 20’	11’	
	20’ to 30’	13’9”	
Sign bridge	< 60’	12’9”	
	60’ to 90’	15’6”	
	90’ to 120’	17’9”	
	120’ to 150’	19’6”	

TABLE 4 Roadside sign support foundations for West Virginia (*34*)

Post Beam Size	Width	Depth	Concrete (yd^3)	Reinforcement
S4X7.7	1’6”	4’	0.3	6-#4
W6X12	2’6”	4’	0.7	6-#4
W8X18	2’6”	5’6”	1	6-#6
W10X22	2’6”	6’6”	1.2	6-#8

TABLE 5 Foundations for steel posts for Iowa (*35*)

Post size	Stub length	Footing diameter	Footing depth
W6x12	2½’	2’	6’
W6x15	2½’	2’	6½’
W8x18	2½’	2’	7’
W8x21	3’	2’8”	7½’
W10x22	3’	2’8”	8’
W10x26	3’	2’8”	8½’
W12x26	3’	2’8”	9’

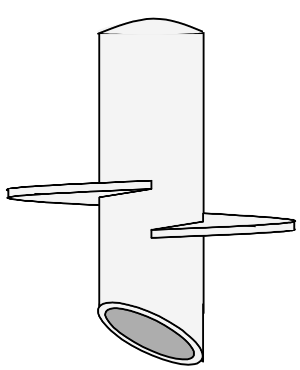


Figure 2. Typical lead section of steel screw-in foundation.

round or square, with a fixed or variable bolt circle. Figure 3 shows a typical fixed-bolt circle base plate. The screw-in helix foundation is typically made of a high-strength steel pipe shaft for resistance to bending moments and installation torque. This foundation can be used for street lighting poles and roadside signs. The product has standardized dimensions. Typical foundation lengths are 5, 8, and 10 feet. Typical diameters are 6 to 9 inches. Electrical wiring is fed through a hand hole in the side of the shaft. The foundation can be installed easily using conventional rotary equipment. Standards and specifications from Missouri mentioned screw anchor foundation for aluminum lighting poles; however, no minimum embedment depth requirements were provided (*36*).

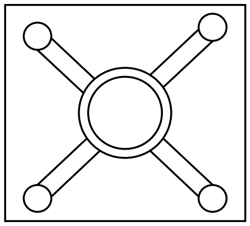


Figure 3. Typical fixed-bolt circle base plate.

Steel screw-in foundations may be designed using the load and allowable stress provisions of Sections 3 and 5 in the 2001 *Supports Specifications*. The foundation depth may be determined using an acceptable procedure for a laterally loaded drilled shaft, such as the Brom's method.

After determination of the embedment depth, a hole is augered. The pole or post is placed in the hole, and a suitable backfill material is placed around the structure. The backfill material, such as crushed limestone or gravel, should be placed in lifts and each lift adequately tamped. Concrete backfill has also been commonly used.

2.5.4 Direct Embedded Poles

Small poles or posts for lighting and roadside signs may be embedded directly in the earth. Embedment length can be determined by the same methods for drilled shafts or by using the approximate chart given in Section 13 of the 2001 *Supports Specifications*.

A review of available state design specifications and manuals indicates that direct embedment is commonly used for signal post structures, standards, mileposts, and so forth. Direct embedment poles may be timber, steel, concrete, or fiberglass. Typical embedment length may range from 17% to 25% of total pole length. Both California and Connecticut DOTs specify that wood poles placed should be placed in ground at least 6 feet deep (*37, 38*). South Dakota, on the other hand, requires wood posts to be 4 feet (1.2 meters) deep (*39*). For direct embedment of U channel or square posts, Ohio requires a 42-inch (1.05-meter) depth; for strain pole embedment, it requires a 6-foot depth (*40*).

2.5.5 Pile Foundations

Piles can be displacement, metal shell, steel pipe, steel H, or concrete piles. They may be designed for friction conditions or embedment till bedrock. The sizes of piles are governed by the design load and the load transfer mechanisms. The axial capacity, lateral capacity, and settlement of piles in various types of soils may be estimated according to methods prescribed in the *AASHTO LRFD Bridge Design Specifications*.

Since signs and luminaires provide visual aids for drivers and pedestrians, the allowable settlement should be limited by the design height of the sign or luminaire structure. Excessive settlements of signs or luminaires can cause inconvenience to drivers and pedestrians. For calculation of settlement, the designer may refer to *AASHTO LRFD Bridge Design Specifications*.

Piles are commonly used for heavy, vertical, lateral loadings or large, overturning moments. In many cases, piles are basically the same as drilled shafts, even though the states may call them differently. In the state of Alabama (*41*), pile footings are recommended if, in the opinion of the engineer, footings cannot be founded at a reasonable depth on rock or other satisfactory foundation material, or if satisfactory foundation is more then 10 feet (3 meters) below the bottom of the footing. The design shall be such as to produce a minimum factor of safety against overturn of 1.5 (same as for drilled shafts).

Some states specify the minimum pile lengths in their standards and specifications. For example, for steel pipe piles, Alaska specifies that if subsurface consists of shallow bedrocks, minimum pile lengths should be 6 feet (*29*). Generally, the same minimum depth requirements for drilled shafts can be applied also to piles.

2.5.6 Spread Footing

Spread footings may be used to support structures on firm shallow soil strata. Typically, cast-in-place, spread footings should be designed to support the design loads with adequate bearing and structural capacity, as well as limited settlements. The required bearing capacity is related to the equivalent bearing pressure developed along the contact surface between the soil and the supporting foundation. Since the dominant load on a sign structure may be lateral, spread footings should be designed with adequate strength to resist eccentric loads.

Alaska specifies that spread footings should have a minimum 4-foot backfill from the top of the base, with base dimensions of 7 1/4 feet by 7 1/4 feet. For controller and signal bases, the spread footing should be 4 feet deep, whereas push button base posts require a 2-foot depth. Traffic signal pole foundations should be spread footings with a 2-foot-thick base and a 3-foot-deep shaft (*29*).

For overhead sign structures, Pennsylvania suggests dimensions of spread footings to be at least 5 feet deep with a width in the range of 5 to 15 feet and a length of 8 to 24 feet (*32*).

2.5.7 Selection of Foundation Types

As mentioned earlier, the decision in selecting a foundation type for a particular application depends on the soil strength, soil types, and loadings. Since the geological formation and the loading patterns differ from state to state, the geotechnical engineer should determine the selection of foundation types.

General selection criteria are provided for guidance in Table 6. The recommended minimum depth will be related to the structure types. However, these values are for reference only. Table 7 shows summary selection criteria for the different types of foundations for signs, luminaires, traffic signals, and overhead sign support structures. The selection criteria are based on load condition, soil stiffness conditions, and site accessibility. Five types of foundation are considered in the table; however, composite or combined foundations are also possible.

2.5.8 Summary

The objective of this section was to provide design guidance and selection criteria for the design of foundations for signs, luminaires, traffic signals, and overhead sign support structures. This objective was accomplished by reviewing foundation design guidelines in the literature. The design guidance in this section also addressed screw-in helix foundations and direct embedded foundations. Based on the soil stiffness profile and the type of loading, a general table for foundation selection was also provided.

2.6 DRAG COEFFICIENT TRANSITIONS FOR MULTISIDED TO ROUND CROSS SECTIONS

2.6.1 General

Table 3-6 of the 2001 *Supports Specifications* provides drag coefficients for round and multisided shapes, which are consistent with the 1994 *Supports Specifications*, except for some minor changes related to the use of International System of Units (SI) units and the 3-second gust wind velocity. The drag coefficients do not appropriately address the condition of a tapered pole with a multisided cross section that

TABLE 6 General foundation selection criteria

Foundation Type	Applicable Soil Conditions	Recommended Minimum Depth	Load Levels
Shallow Foundations ($D/B \le 2$)			
Spread footings	Any conditions where bearing capacity is adequate for applied load. May use on a single stratum. Groundwater is below footing.		Medium–heavy
Deep Foundations ($D/B \ge 2$)			
Bearing piles (primarily axially loaded)	Surface and near-surface soils have low bearing capacity and competent soil bedrock is at greater depth. Sufficient skin friction can be developed by soil-to-pile perimeter to carry anticipated loads. Point-bearing where insufficient skin friction can be developed.	For solid rock, minimum 6 ft. For borrows, minimum 16 ft.	Heavy
Drilled shafts (laterally loaded)	Depends on depth of competent stratum.	For lighting structures, cantilever structures, 10- to 12-ft minimum. For sign bridges, 12–16 ft. For overhead sign structures, multiple shafts 8–12 ft.	Light–heavy
Direct embedment (laterally loaded)	For soil with adequate bearings for light structures. For stiff soil near surface. No significant erosion problem.	Timber poles: minimum 20% of pole length. U channel or square posts: minimum 3 ft. Same as for drilled shafts.	Light–medium
Screw-in helix foundations (laterally loaded)	Where the resistances against torsion, bending and shear, and combination loads are required (*28*).	Same as drilled shafts or direct embedment.	Light–medium
B: width or diameter D: depth			

TABLE 7 Qualitative selection criteria for foundations based on load and soil

Types of Foundation		Load Level	Soil Stiffness Conditions
Axially Loaded	Driven Piles	Heavy	Weak strata lacking adequate lateral resistance or when bedrock is relatively deep
Laterally Loaded	Spread Footing	Medium–Heavy	Soft soil near surface, groundwater level is below footing
	Screw-In Helix Foundation	Light–Medium	Stiff soil near surface with adequate lateral resistance
	Direct Embedment		
	Drilled Shaft		

has large bend radii or is small enough that the cross section approaches a round section.

In general, multisided shapes have higher drag coefficients than round shapes do. Figure 4 provides a comparison of drag coefficient values for various round and multisided shapes. The drag coefficients are based on the equations in Table 3-6 of the *Supports Specifications*, which are reproduced in Table 8. For example, for $C_vVd > 78$, the drag coefficient is 0.45 for a round shape and between 0.55 and 0.83 for a 16-sided shape, where C_v is the square root of the importance factor, V is the basic wind velocity, and d is the depth of the member. This value represents an increase of 22% to 84% in drag coefficient for the multisided shape. Considering a tapered pole with a multisided cross section, a drag

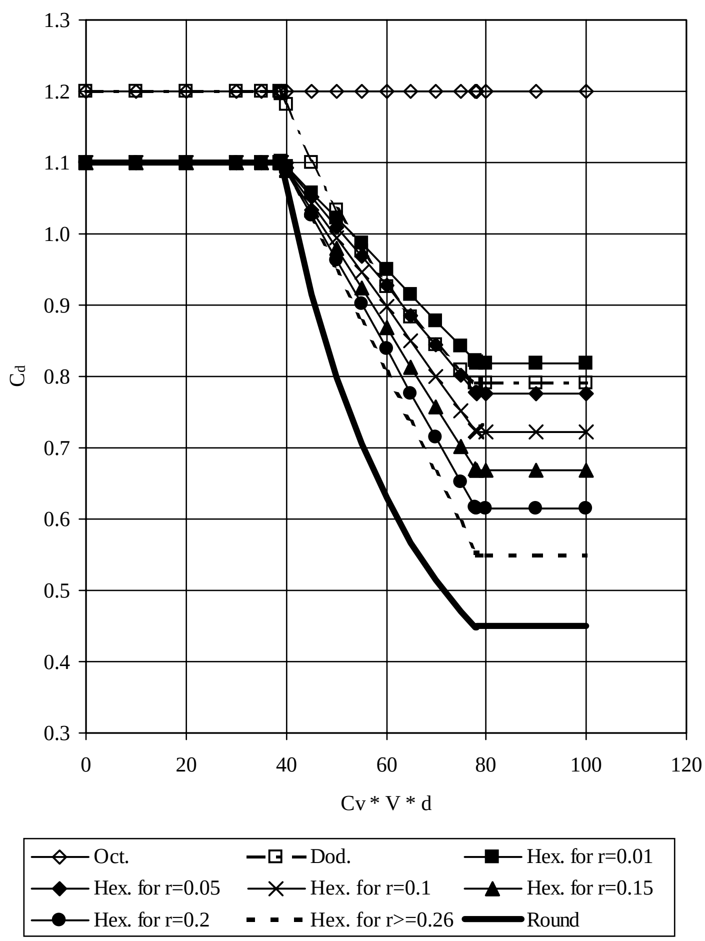


Figure 4. Shape factors for round and multisided sections.

TABLE 8 Drag coefficient values for round and multisided shapes (from Table 3-6, wind drag coefficients, C_d, in the *Supports Specifications* [2])

	$C_vVd \le 5.33$ (*39*)	5.33 (*39*) $< C_vVd < 10.66$ (*78*)	$C_vVd \ge 10.66$ (*78*)
Hexdecagonal: $0 \le r < 0.26$	1.10	$1.37 + 1.08r - \frac{C_vVd}{19.8} - \frac{C_vVdr}{4.94}$ (SI) $1.37 + 1.08r \ \frac{C_vVd}{145} - \frac{C_vVdr}{36}$ (U.S. Cust.)	$0.83 - 1.08r$
Hexdecagonal: $r \ge 0.26$	1.10	$0.55 + \frac{(10.66 - C_vVd)}{9.67}$ (SI) $0.55 + \frac{(78.2 - C_vVd)}{71}$ (U.S. Cust.)	0.55
Dodecagonal (See Note 1)	1.20	$\frac{3.28}{(C_vVd)^{0.6}}$ (SI) $\frac{10.8}{(C_vVd)^{0.6}}$ (U.S. Cust.)	0.79
Octagonal	1.20	1.20	1.20
Cylindrical	1.10	$\frac{9.69}{(C_vVd)^{1.3}}$ (SI) $\frac{129}{(C_vVd)^{1.3}}$ (U.S. Cust.)	0.45

Notes:
1. Valid for members having a ratio of corner radius to distance between parallel faces equal to or greater than 0.125.
2. Units in top row are SI, with English Customary in parentheses.
3. Notation:
   C_v: velocity conversion factor
   V: basic wind velocity (m/s, mph)
   d: depth (diameter) of member (m, ft)

coefficient of 0.55 to 0.83 would normally be used according to the *Supports Specifications*. At higher elevations on the tapered pole, where the pole dimensions become smaller, the shape of the cross section could approach a round shape. It may be justifiable to use a lower drag coefficient at the higher elevations.

A procedure has been developed and presented in Appendix D that will provide a drag coefficient for the transition from multisided to round shape.

2.6.2 Proposed Equation

A drag coefficient transition equation has been developed for multisided tapered poles with geometry approaching round sections. The proposed transition method for drag coefficients may be expressed as follows:

If $r \le r_m$, then $C_d = C_{dm}$ (Eq. 9)

If $r_m < r < r_r$,

$$\text{then } C_d = C_{dr} + (C_{dm} - C_{dr})\left(\frac{r - r_r}{r_m - r_r}\right) \quad \text{(Eq. 10)}$$

If $r \ge r_r$, then $C_d = C_{dr}$ (Eq. 11)

Where

C_d = drag coefficient to be used in the design;
C_{dm} = drag coefficient for the multisided section;
C_{dr} = drag coefficient for the round section;
r = ratio of the corner radius to the radius of the inscribed circle for the multisided cross section;
r_m = ratio of the corner radius to the radius of the inscribed circle where the cross section is considered multisided, as shown in Table 9; and
r_r = ratio of the corner radius to the radius of the inscribed circle where the cross section is considered round, as shown in Table 9.

Figure 5 illustrates the proposed equations for the multisided shapes. Table 9 presents four methods that were considered in the study. The ratios for multisided to round sections (r_m and r_r) vary for each method, as shown in the table.

2.6.3 Comparison of Proposed Design Method to the Existing Equations

Comparisons of four proposed transition equations (Methods 1, 2, 3, and 4) were made for 16-, 12-, and 8-sided and round cross sections and are presented in Appendix D. The four methods vary with r_m and r_r, as provided in Table 2. Method 4 is recommended for use on the basis of the analytical work and visual observations that were made. Selected tables and graphs for Method 4 are presented below. Figure 6

TABLE 9 Ratio *r* for drag coefficient transition equations

Method	Type of Cross Section	r_m	r_r
Method No. 1	16-sided	0.2752	0.4844
	12-sided	0.5975	0.7136
	8-sided	0.8277	0.8774
Method No. 2	16-sided	0.26	0.75
	12-sided		
	8-sided		
Method No. 3	16-sided	0.50	1.00
	12-sided		
	8-sided		
Method No. 4	16-sided	0.26	0.625
	12-sided	0.50	0.75
	8-sided	0.75	1.00

Note: See Section 2.6.3 for description of methods.

shows the transitional limits provided in Table 9 for Method 4. The proposed transition equations are plotted in Figures 7 through 18 with bend radii of 2 and 4 inches, mean recurrence intervals of 50 and 25 years, and wind speed of 90 mph. Tables 10 and 11 provide drag coefficients for various diameters for Method 4. Percentage difference between the proposed method and the multisided shape is provided also. When comparing the 2- and 4-inch bend radii, the change in the drag coefficient is larger for the 4-inch bend radius and also affects a large range of diameters.

2.6.4 Summary

A method was developed to determine the drag coefficient for the transition from multisided to round shape. The proposed method uses a linear transition equation with respect to the variable *r* to interpolate between the drag coefficient for round poles, C_{dr}, and the drag coefficient for multisided poles, C_{dm}. Four sets of constants (representing r_m and r_r) were used in evaluating the transition equation. The limiting constants of Method 4 are recommended for use. The accuracy of the proposed design method, however, should be verified through experimental results. Wind tunnel tests using a wide range of multisided pole sections with different values for *r* would be needed.

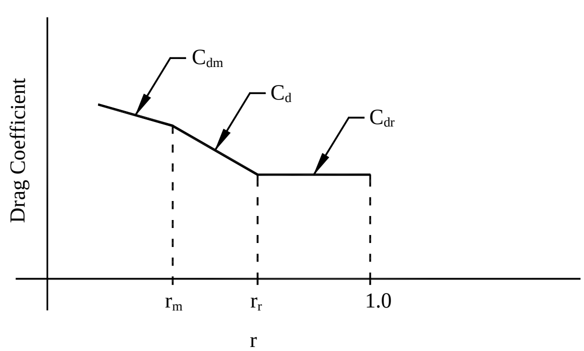


Figure 5. Proposed drag coefficient for multisided sections.

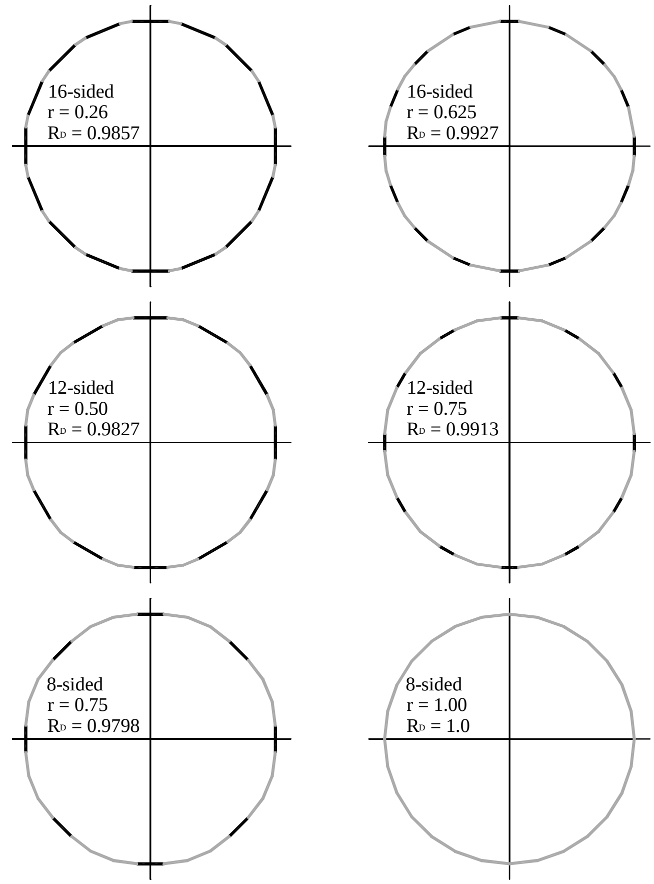


Figure 6. Transitional limits for Method 4.

2.7 CONNECTION PLATE AND BASE PLATE FLATNESS TOLERANCES

Current practices were reviewed to identify the need for structure-specific connection plate and base plate flatness tolerances for erection. A survey of state departments of transportation, manufacturers, and consulting engineers was made to determine the current practice. The survey was studied to determine a consensus of tolerances for specific structure types and connection details. Different tolerances for lighting, traffic signal, and sign support structures were investigated. Recommended tolerances based on the research findings are provided.

2.7.1 Survey Results

Eight state DOTs indicated that they specified tolerances for flatness of base plates and connection plates for overhead

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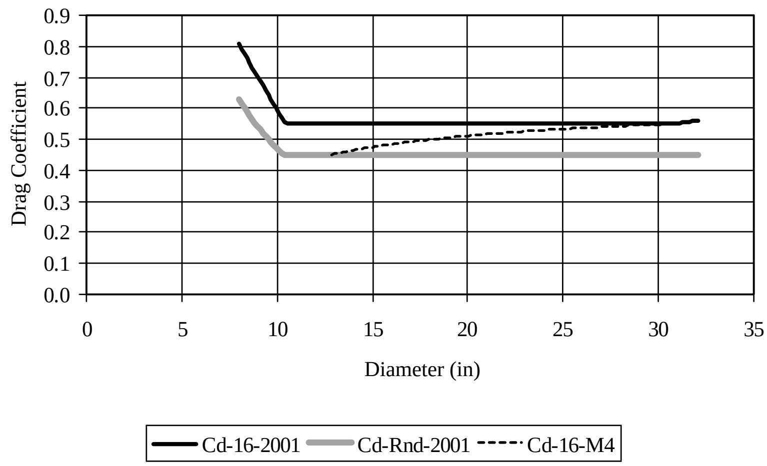

Figure 7. Drag coefficient comparison: Method 4, 50-year mean resistance interval, 90 mph, 4-inch bend radius, 16-sided.

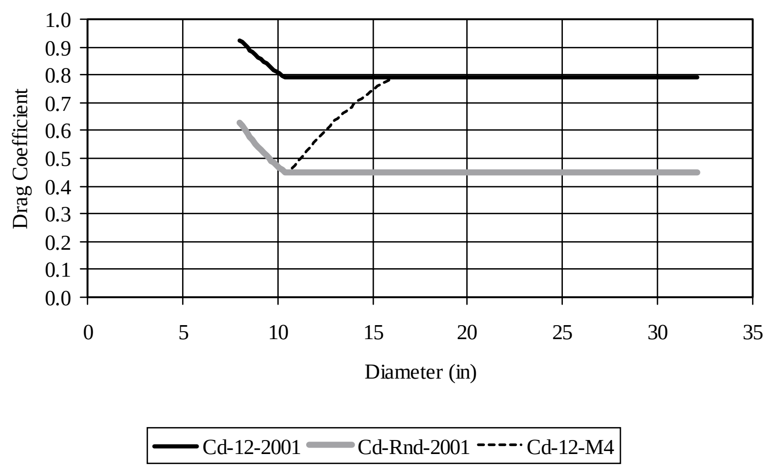

Figure 8. Drag coefficient comparison: Method 4, 50-year mean resistance interval, 90 mph, 4-inch bend radius, 12-sided.

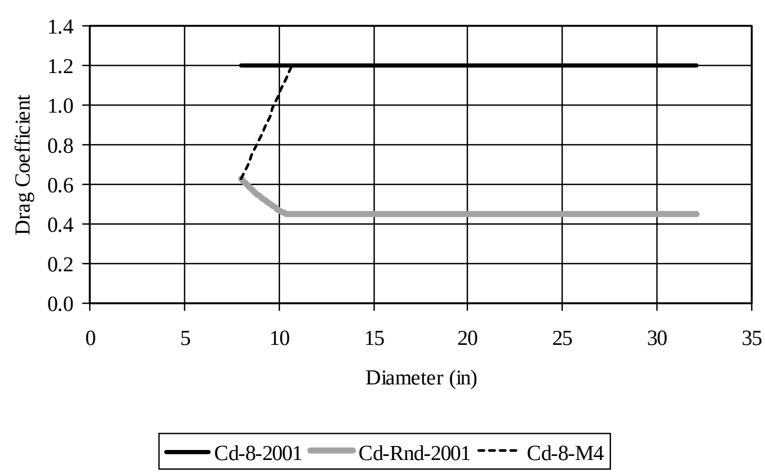

Figure 9. Drag coefficient comparison: Method 4, 50-year mean resistance interval, 90 mph, 4-inch bend radius, 8-sided.

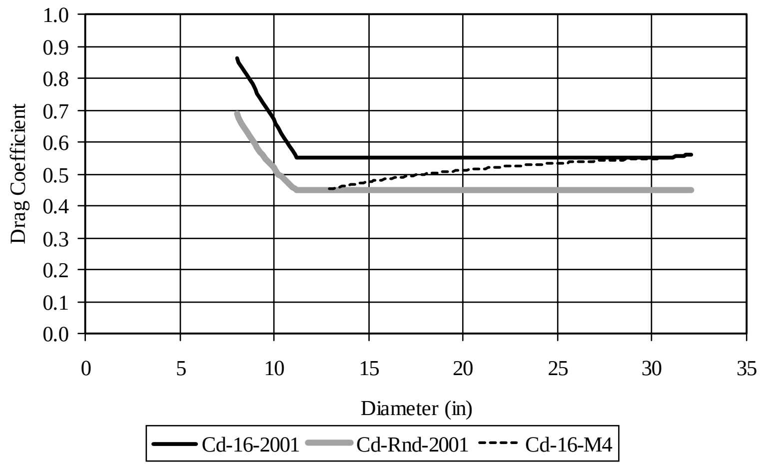

Figure 10. Drag coefficient comparison: Method 4, 25-year mean resistance interval, 90 mph, 4-inch bend radius, 16-sided.

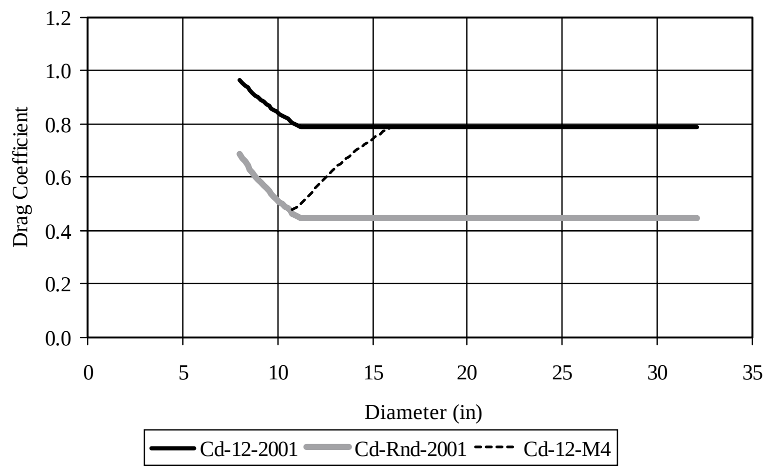

Figure 11. Drag coefficient comparison: Method 4, 25-year mean resistance interval, 90 mph, 4-inch bend radius, 12-sided.

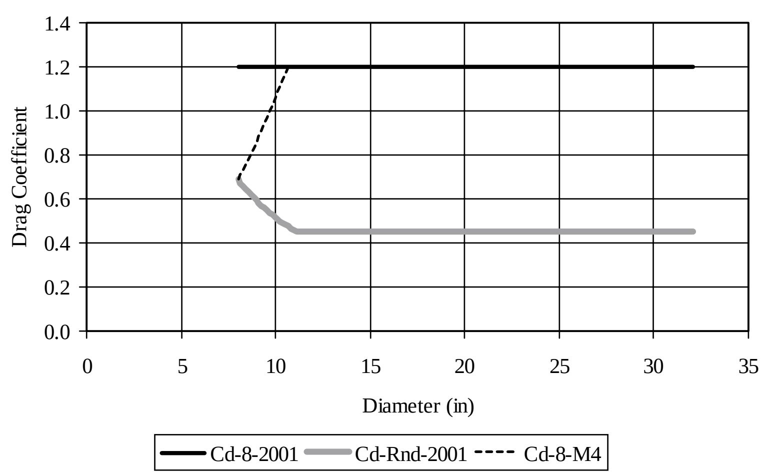

Figure 12. Drag coefficient comparison: Method 4, 25-year mean resistance interval, 90 mph, 4-inch bend radius, 8-sided.

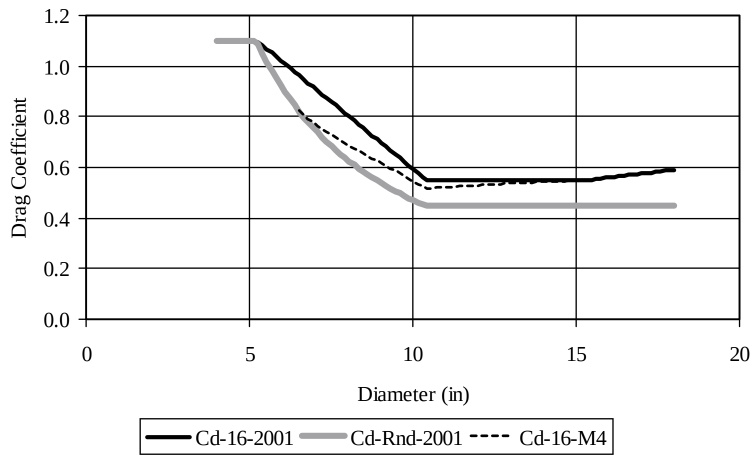

Figure 13. Drag coefficient comparison: Method 4, 50-year mean resistance interval, 90 mph, 2-inch bend radius, 16-sided.

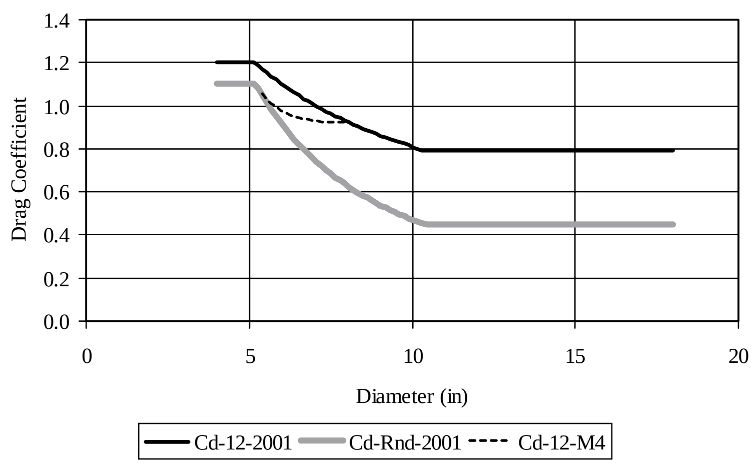

Figure 14. Drag coefficient comparison: Method 4, 50-year mean resistance interval, 90 mph, 2-inch bend radius, 12-sided.

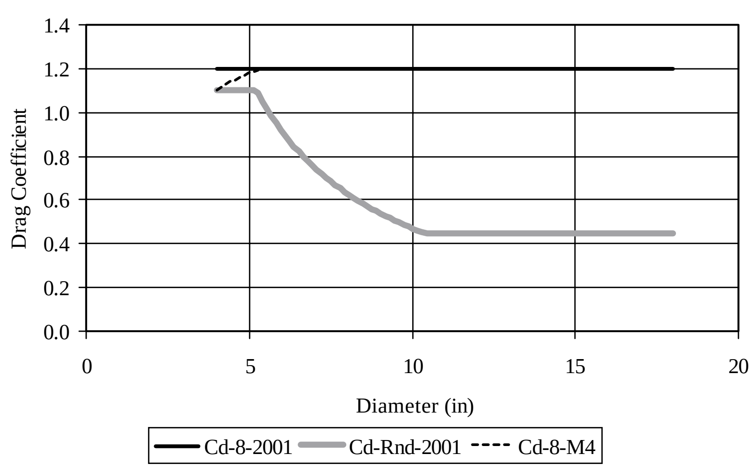

Figure 15. Drag coefficient comparison: Method 4, 50-year mean resistance interval, 90 mph, 2-inch bend radius, 8-sided.

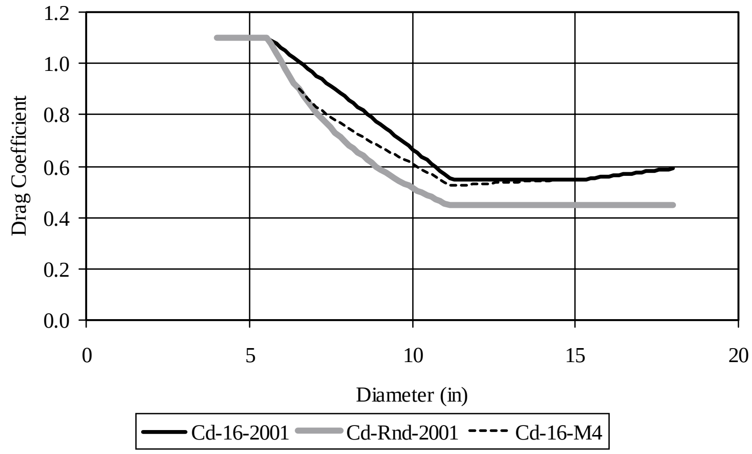

Figure 16. Drag coefficient comparison: Method 4, 25-year mean resistance interval, 90 mph, 2-inch bend radius, 16-sided.

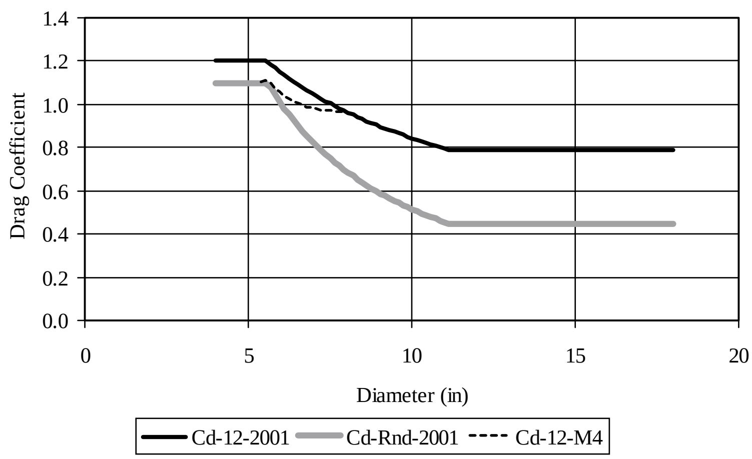

Figure 17. Drag coefficient comparison: Method 4, 25-year mean resistance interval, 90 mph, 2-inch bend radius, 12-sided.

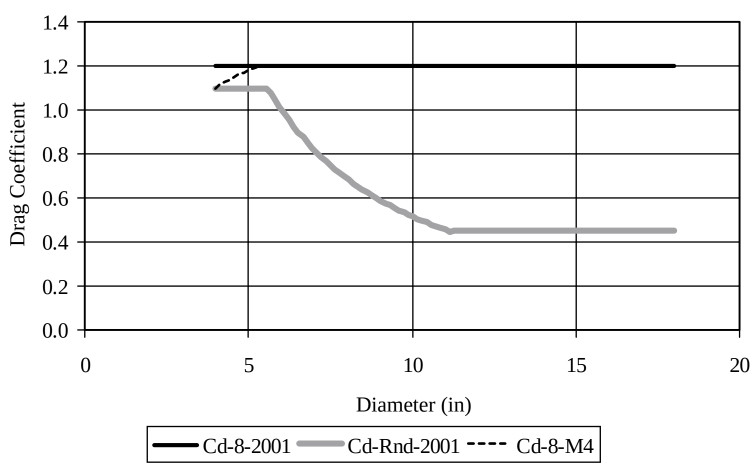

Figure 18. Drag coefficient comparison: Method 4, 25-year mean resistance, 90 mph, 2-inch bend radius, 8-sided.

TABLE 10 Drag coefficient: Method 4 (4-inch bend radius)

			16-sided			12-sided			8-sided		
			r_m	0.2600		r_m	0.5000		r_m	0.7500	
Wind	Imp.	Bend	r_r	0.6250		r_r	0.7500		r_r	1.0000	
Speed (mph)	Fact.	Rad. (in)	Dia. (in)	C_d Cd-16-M4	C_{dm} % diff.	Dia. (in)	C_d Cd-12-M4	C_{dm} % diff.	Dia. (in)	C_d Cd-8-M4	C_{dm} % diff.
50-year mean resistance interval											
90	1.00	4	8.0	0.6295	-22%	8.0	0.6295	-32%	8.0	0.6295	-48%
90	1.00	4	8.8	0.5529	-23%	10.4	0.4487	-43%	9.0	0.8291	-31%
90	1.00	4	17.4	0.4951	-10%	13.0	0.6356	-20%	9.5	0.9522	-21%
90	1.00	4	22.1	0.5222	-5%	14.3	0.7092	-10%	10.1	1.0763	-10%
90	1.00	4	30.7	0.5498	0%	15.1	0.7514	-5%	10.4	1.1378	-5%
90	1.00	4				16.0	0.7891	0%	10.7	1.1986	0%
150	1.00	4	8.0	0.4500	-18%	8.0	0.4500	-43%	8.0	0.4500	-63%
150	1.00	4	14.3	0.4680	-15%	10.9	0.4755	-40%	8.4	0.5996	-50%
150	1.00	4	17.4	0.4951	-10%	11.9	0.5572	-29%	8.8	0.7351	-39%
150	1.00	4	22.1	0.5222	-5%	13.0	0.6356	-20%	9.3	0.8582	-28%
150	1.00	4	30.7	0.5498	0%	14.3	0.7092	-10%	9.7	0.9707	-19%
150	1.00	4				15.1	0.7514	-5%	10.1	1.0738	-11%
150	1.00	4				16.0	0.7891	0%	10.4	1.1379	-5%
150	1.00	4							10.7	1.1986	0%
25-year mean resistance interval											
90	0.87	4	8.0	0.6891	-20%	8.0	0.6891	-29%	8.0	0.6891	-43%
90	0.87	4	9.5	0.5482	-23%	10.7	0.4745	-42%	8.8	0.8313	-31%
90	0.87	4	17.4	0.4951	-10%	11.9	0.5572	-29%	9.5	0.9691	-19%
90	0.87	4	22.1	0.5222	-5%	13.0	0.6356	-20%	10.1	1.0837	-10%
90	0.87	4	30.7	0.5498	0%	14.3	0.7092	-10%	10.4	1.1413	-5%
90	0.87	4				15.1	0.7514	-5%	10.7	1.1986	0%
90	0.87	4				16.0	0.7891	0%			
150	0.80	4	8.0	0.4500	-18%	8.0	0.4500	-43%	8.0	0.4500	-63%
150	0.80	4	17.2	0.4941	-10%	10.9	0.4755	-40%	8.4	0.5996	-50%
150	0.80	4	22.1	0.5222	-5%	11.9	0.5572	-29%	8.8	0.7351	-39%
150	0.80	4	30.7	0.5498	0%	13.0	0.6356	-20%	9.3	0.8582	-28%
150	0.80	4				14.3	0.7092	-10%	9.7	0.9707	-19%
150	0.80	4				15.1	0.7514	-5%	10.1	1.0738	-11%
150	0.80	4				16.0	0.7891	0%	10.4	1.1379	-5%
150	0.80	4							10.7	1.1986	0%

bridge structures. Six states specified limits for overhead cantilevers. Four states specified limits on other types of structures. All states that specified limits on tolerances were contacted by phone.

The basic findings are as follows:

- Only four states (California, Iowa, New Mexico, and Texas) have developed tolerance specifications.
- Only one state (California) differentiates between base plates (slip base and normal) and connection plates.
- No state addresses different types of structures (lighting, traffic signal, and sign support).
- No state addresses any effects on design, aesthetics, or fatigue, nor were any references addressing these issues found.
- Most states specify manufacturing tolerances on flatness of steel plate prior to welding and galvanizing.

2.7.2 Evaluation of the Survey Responses

Several states indicated that they referred to ASTM A6 (*42*), which specifies manufacturing tolerances for flatness of steel plates. This ASTM standard, however, does not address tolerances on flatness of plates after welding or galvanizing. After the base and connection plates are welded to structural members, some warping can be expected. Therefore, a specification for steel before fabrication is not appropriate.

The most extensive work in this area seems to have been done by Caltrans (*37, 43*). Caltrans has developed *Standard Specifications and these Special Provisions* for support structures. The support structures are addressed in two separate sections—"Section 56: Sign Structures" and "Section 86: Signal & Lighting Structures." These special provisions cover connection plate and base plate flatness tolerances. Furthermore, Caltrans assembled a committee of manufacturers to review the documents and comment on feasibility of implementation. Caltrans was contacted by telephone for further discussions. One example cited by Caltrans is $^{1}/_{8}$ inch in 1 foot, or approximately $^{1}/_{100}$ out-of-flatness limit.

In the survey, California, Iowa, Michigan, Missouri, New Jersey, New Mexico, New York, Oklahoma, Texas, and Utah reported having information pertaining to this work. On being contacted to obtain this information, most said that they did not, in fact, have tolerances. They were, in fact, referring to flatness of plate, not flatness of base and connec-

TABLE 11 Drag coefficient: Method 4 (2-inch bend radius)

Wind Speed (mph)	Imp. Fact.	Bend Rad. (in)	16-sided			12-sided			8-sided		
			r_m 0.2600			r_m 0.5000			r_m 0.7500		
			r_r 0.6250			r_r 0.7500			r_r 1.0000		
			Dia. (in)	C_d Cd-16-M4	C_{dm} % diff.	Dia. (in)	C_d Cd-12-M4	C_{dm} % diff.	Dia. (in)	C_d Cd-8-M4	C_{dm} % diff.
50-year mean resistance interval											
90	1.00	2	4.0	1.1000	0%	4.0	1.1000	-8%	4.0	1.1000	-8%
90	1.00	2	5.1	1.1000	0%	5.8	0.9942	-11%	4.4	1.1380	-5%
90	1.00	2	5.5	1.0150	-5%	7.4	0.9261	-5%	5.3	1.1952	0%
90	1.00	2	6.0	0.9230	-10%	7.9	0.9255	-1%			
90	1.00	2	7.2	0.7523	-15%						
90	1.00	2	9.5	0.5870	-10%						
90	1.00	2	11.1	0.5229	-5%						
90	1.00	2	15.3	0.5498	0%						
150	1.00	2	4.0	0.7979	-16%	4.0	0.7979	-23%	4.0	0.7979	-34%
150	1.00	2	5.3	0.5589	-23%	5.3	0.5589	-36%	4.7	0.9764	-19%
150	1.00	2	8.6	0.4941	-10%	6.5	0.6356	-20%	5.0	1.0723	-11%
150	1.00	2	11.1	0.5229	-5%	7.2	0.7165	-9%	5.1	1.1224	-6%
150	1.00	2	15.3	0.5498	0%	7.5	0.7447	-6%	5.3	1.1732	-2%
150	1.00	2				7.9	0.7831	-1%			
25-year mean resistance interval											
90	0.87	2	4.0	1.1000	0%	4.0	1.1000	-8%	4.0	1.1000	-8%
90	0.87	2	5.5	1.1000	0%	6.1	1.0390	-9%	4.4	1.1380	-5%
90	0.87	2	6.0	1.0104	-5%	7.2	0.9749	-5%	5.3	1.1958	0%
90	0.87	2	6.5	0.9025	-11%	7.9	0.9657	-1%			
90	0.87	2	7.6	0.7781	-13%						
90	0.87	2	9.6	0.6361	-10%						
90	0.87	2	11.1	0.5251	-5%						
90	0.87	2	15.3	0.5498	0%						
150	0.80	2	4.0	0.9224	-10%	4.0	0.9224	-16%	4.0	0.9224	-23%
150	0.80	2	6.0	0.5493	-23%	5.4	0.6355	-31%	5.0	1.0922	-9%
150	0.80	2	8.6	0.4941	-10%	6.4	0.6660	-20%	5.1	1.1336	-6%
150	0.80	2	11.1	0.5229	-5%	7.2	0.7165	-9%	5.3	1.1768	-2%
150	0.80	2	15.3	0.5498	0%	7.5	0.7447	-6%			
150	0.80	2				7.9	0.7831	-1%			

tion plates after they are welded to structures. As an example, Missouri noted that its steel plate had to meet Table 13 of ASTM A6 (*36*).

Texas has supplied a copy of "Item 613: High Mast Illumination Poles" and "Item 686: Traffic Signal Pole Assemblies (Steel)," each of which contains fabrication tolerances, from its 1993 state *Standard Specifications* (*44*). Texas also supplied detail drawings for these structures illustrating tolerances.

The California (*43*), Iowa (*45*), New Mexico (*46*), and Texas (*44*) provisions are summarized in Table 12. The California and Texas provisions are very close, but the Iowa and New Mexico provisions are much stricter. The New Mexico provision is generic to steel structures and not specific to support structures—the provision refers to "sole masonry and shoe plates." Likewise, the Iowa provision does not refer specifically to support structures.

2.7.3 Recommendations

Caltrans has made the most extensive effort to coordinate its specification with the industry. The provision from the

TABLE 12 Comparison of state DOT flatness tolerances

State	Connection Type	U.S.	Metric	Decimal
California	Base plate	0.1181" in 12" 0.1969" overall	3 mm in 305 mm 5 mm overall	0.0098 in/in
	Connection plate	0.0787" in 12" 0.1181" overall	2 mm in 305 mm 3 mm overall	0.0066 in/in
Iowa	Base plate and connection plate	1/32" in 12" (0.0313" in 12")	1 mm in 400 mm	0.0025 in/in
New Mexico	Base plate and connection plate	1/16" overall (0.0625" overall)	1 mm overall	
Texas	Base plate and connection plate	3/16" in 24" (0.1875" in 24")	4.76 mm in 610 mm	0.0078 in/in

March 29, 2000, revision of the specification (*37*) in subparagraph 10 of paragraph 4 of Section 86-2.04, "Standards, Steel Pedestals, and Posts," reads as follows:

> Surfaces of base plates which are to come in contact with concrete, grout, or washers and leveling nuts shall be flat to within 3 mm tolerance in 305 mm, and to within 5 mm tolerance overall. Faying surfaces of 1) plates in high-strength bolted connections, 2) plates in joints where cap screws are used to secure luminaire and signal arms, and 3) plates used for breakaway slip base assemblies shall be flat to within 2 mm tolerance in 305 mm, and within 3 mm tolerance overall.

However, support manufacturers pointed to the limiting nature of the Caltrans specification, especially since the specification exceeds the flatness tolerances specified in ASTM A6 for plates prior to welding and galvanizing. Practical tolerances were discussed with support manufacturers. As a result, it is recommended that the *Supports Specifications* adopt a provision similar to the Caltrans limit on flatness of base plates and connection plates with changes to reflect practical manufacturing limitations. A new proposed section on tolerances for flatness of base plates and connections to be added to the specifications column of the *Supports Specifications* should read as follows:

> 5.14.4 Tolerances for Flatness of Base Plates and Connection Plates
>
> All base plates and plates used in high-strength bolted connection shall comply with the dimensional plate flatness requirements of ASTM A6, after fabrication and galvanization.
>
> Surfaces of base plates that come in contact with concrete, grout, or washers and leveling nuts shall be flat to within 4 mm tolerance in 305 mm, and to within 6 mm tolerance overall. Faying surfaces of 1) plates in high-strength bolted connections, 2) plates in joints where cap screws are used to secure luminaire and signal arms, and 3) plates used for breakaway slip base assemblies shall be flat to within 3 mm tolerance in 305 mm and within 4 mm tolerance overall.

2.8 SQUARE AND RECTANGULAR STEEL TUBES BENT ABOUT A DIAGONAL AXIS

Preliminary test results have shown that the strength of the square cross section is greater when bent about the diagonal than what is predicted by current calculations. One reason contributing to higher strengths is the fact that, for compact sections, the shape factor is higher for tubular sections bent about the diagonal than the shape factor for square sections bent about a principal axis. For noncompact and slender sections, it appears that the corner of the cross section provides added stiffness that results in the section resisting higher loads before locally buckling. It is not appropriate to apply the same allowable stresses of square tubular shapes bent about a principal axis to square tubular shapes bent about the diagonal. Tubular shapes bent about the diagonal would have a linearly varying stress distribution, while tubular shapes bent about a principal axis would have a uniformly compressed flange on one side that would contribute to a lower buckling stress. It appears that the strain distribution in the tube wall would have an effect on the local buckling strength of the section.

A design method was developed to determine the allowable bending capacity of square and rectangular tubes bent about the diagonal axis. Compared with the allowable capacity in the 2001 *Supports Specifications*, the proposed design method was shown to greatly increase the allowable moment for compact tubes. No increase is provided for slender tubes.

A literature review (*47*, *48*) revealed a nonlinear interaction equation for the biaxial bending strength of square and rectangular tubes that can be bent into the inelastic range without buckling. The following equation is proposed for compact sections:

$$\left(\frac{M_x}{M_{xall}}\right)^{1.6} + \left(\frac{M_y}{M_{yall}}\right)^{1.6} \le 1.0 \qquad \text{(Eq. 12)}$$

Where

M_x = applied moment about the x-axis,
M_y = applied moment about the y-axis,
M_{xall} = allowable moment about the x-axis, and
M_{yall} = allowable moment about the y-axis.

Local buckling of the tube walls is a failure mode that must also be taken into account (*48, 49*). The wall slenderness limits for tubes bent about the diagonal axis can be larger than for tubes that are bent about one geometric axis because the strain distribution in the wall is triangular. The following equation is proposed for slender sections:

$$\left(\frac{M_x}{M_{xall}}\right)^{1.0} + \left(\frac{M_y}{M_{yall}}\right)^{1.0} \le 1.0 \qquad \text{(Eq. 13)}$$

The proposed design procedure applies to square and rectangular tubes that are bent about a skewed axis. The tube must also be designed separately for the maximum moments about each geometric axis. The following interaction equation is proposed:

$$\left(\frac{M_x}{M_{xall}}\right)^{\alpha} + \left(\frac{M_y}{M_{yall}}\right)^{\alpha} \le 1.0 \qquad \text{(Eq. 14)}$$

For tubes with $\lambda \le \lambda_r$,

$$\alpha = 1.6 \qquad \text{(Eq. 15)}$$

$$M_{xall} = 0.6 S_x F_y \qquad \text{(Eq. 16)}$$

$$M_{yall} = 0.6 S_y F_y \qquad \text{(Eq. 17)}$$

Where

λ = width-to-thickness ratio of the wall in question,
λ_r = width-to-thickness ratio at the compact limit,
α = a constant,
F_y = yield stress in ksi,
S_x = section modulus about the x-axis, and
S_y = section modulus about the y-axis.

For tubes with $\lambda_r < \lambda \leq \lambda_{max}$,

$$\alpha = 1.0 \qquad \text{(Eq. 18)}$$

$$M_{xall} = S_x F_b \qquad \text{(Eq. 19)}$$

$$M_{yall} = S_y F_b \qquad \text{(Eq. 20)}$$

Where

$$F_b = 0.74 F_y \left(1 - \frac{0.194}{\sqrt{\dfrac{E}{F_y}}} \frac{b}{t} \right) \qquad \text{(Eq. 21)}$$

$$\lambda_r = \frac{260}{\sqrt{F_y}} \qquad \text{(Eq. 22)}$$

$$\lambda_{max} = \frac{365}{\sqrt{F_y}} \qquad \text{(Eq. 23)}$$

λ_{max} = maximum width-to-thickness ratio allowed,
E = modulus of elasticity in ksi,
b = flat width of the tube wall in question, and
t = tube wall thickness.

The x-axis and y-axis are defined as shown in Figure 19.

The interaction equation can also be presented in the following terms of flexural stress:

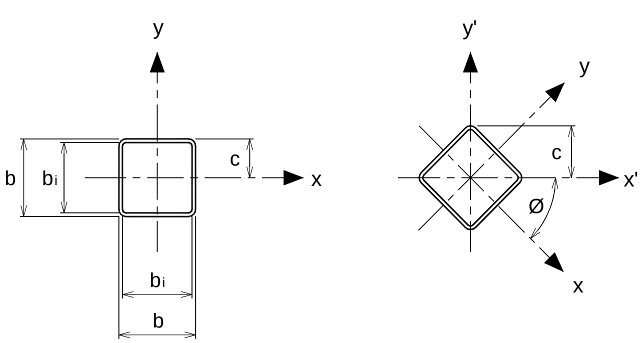

Figure 19. Notation for bending about the diagonal of a square steel tube.

$$\left[\frac{f_x}{F_b} \right]^{\alpha} + \left[\frac{f_y}{F_b} \right]^{\alpha} \leq 1.0 \qquad \text{(Eq. 24)}$$

Where

f_x = applied bending stress about the x-axis,
f_y = applied bending stress about the y-axis, and
F_b = allowable bending stress.

For tubes with $\lambda \leq \lambda_r$, where $\lambda_r = \dfrac{260}{\sqrt{F_y}}$,

$$\alpha = 1.6 \qquad \text{(Eq. 25)}$$

$$F_b = 0.6 F_y \qquad \text{(Eq. 26)}$$

For tubes with $\lambda_r < \lambda \leq \lambda_{max}$, where $\lambda_{max} = \dfrac{365}{\sqrt{F_y}}$,

$$\alpha = 1.0 \qquad \text{(Eq. 27)}$$

$$F_b = 0.74 F_y \left(1 - \frac{0.194}{\sqrt{\dfrac{E}{F_y}}} \frac{b}{t} \right) \qquad \text{(Eq. 28)}$$

Using data from an unpublished source (*50*), the ratios of experimental moment to calculated moment based on the proposed design procedure provided safety factors that ranged from 1.95 to 2.85, which are consistent with the safety factors in the *Supports Specifications*. The basis for the development of the proposed design procedure and two design examples illustrating the use of the procedure are provided in Appendix E.

2.9 PERFORMANCE SPECIFICATION FOR FIBER-REINFORCED COMPOSITES

The 2001 *Supports Specifications* included a new section, “Fiber-Reinforced Composites Design.” Although this section is intended to address a number of composites in the future, currently it focuses only on fiberglass-reinforced plastic (FRP) composites. Information on manufacturing, design, and testing of FRP is given in the section. Since fiber-reinforced composite materials have properties that highly depend on the manufacturing method and materials used, Project 17-10(2) was charged with developing performance specifications and acceptance testing procedures for FRP, as well as quality control procedures for FRP members. Such a specification is needed to evaluate the structural safety and behavior of structures in lieu of computational methods, which may not be available for certain types of FRP structures. The main sections of the proposed performance specification include the following:

- Determination of bending strength for FRP poles;
- Determination of torsional strength for FRP poles;
- Determination of fatigue strength for FRP poles;
- Properties of raw materials, such as polyester resin and glass fiber reinforcement;
- Specification of protective materials, such as coatings or veils, which are required to protect against weathering, solar ultraviolet rays, and wind abrasion; and
- Deflection criteria.

2.9.1 DOT Survey and Documents Reviewed

A survey of all state DOTs was conducted to gather information on the use of fiber-reinforced composite (FRC) and performance specifications for FRC structures. According to the survey, 19% of the state DOTs use or have used FRC structures, and 81% of the state DOTs do not use or have not used FRC structures. When the state DOTs were asked whether they have specifications or acceptance testing procedures for FRC structures, 2% responded yes, 2% responded no, and 96% responded not applicable. Considering the state DOTs that use or have used FRC, the major applications are street lighting (67%), roadside signs (11%), and other structures (22%). In summary, the results of the survey indicate that FRP is mainly used for lighting poles and roadside supports. The survey also revealed that very few states have specifications for FRP structures.

The most updated literature on FRC testing and design was studied and evaluated. Special emphasis was put on test methods and performance evaluation of FRP members, as well as existing specifications for FRP lighting poles. The major references reviewed included the following:

- ASTM D4923-01, "Standard Specification for Reinforced Thermosetting Plastic Poles" (*51*);
- *NCHRP Report 411: Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (*1*);
- *Structural Design of Polymer Composites—Eurocomp Design Code and Handbook* (*52*);
- British Standard BD26/94, "Design of Lighting Columns" (*53*);
- ANSI C136.20, *Fiber-Reinforced Plastic (FRP) Lighting Poles* (*54*);
- Caltrans Standard Special Provision 86.08.5 and Caltrans Test 683 (*55*);
- *City of Seattle Standard Specification 8-32: Poles, Pedestals and Foundations* (*56*);
- Carolina Power and Light Company, *Group Specification GS-32 for Fiberglass Post* (*57*);
- Illinois Power Company, *Material Specifications for Fiberglass Street Lighting Poles* (*58*);
- Newmark International, Inc., *Specification for Fiberglass Reinforced Composite Poles* (*59*);
- Shakespeare Company, *Shakespeare Composite Utility Pole Specifications* (*60*);
- Southern California Edison Company, Specification MS32-1995, *Fiberglass Reinforced Plastic Street Lighting Electroliers* (*61*);
- Southern Company, *Fiberglass Lighting Pole Specifications* (*62*);
- Hawaii DOT, *Special Provisions, Specifications, Proposal and Contract for Furnishing and Delivering Street Light and Traffic Signal Poles, Arms, Bases, Luminaires and Cable* (*63*);
- Pacific Gas and Electric Company, *Specification for Non-Wood Distribution Poles* (*64*);
- Public Service Company of New Mexico, *PNM Specification SL2 Revision 3: Aluminum/Fiberglass Lighting Poles* (*65*);
- ASTM G154-00a, "Standard Practice for Operating Fluorescent Light Apparatus for UV Exposure of Non-Metallic Materials" (*66*); and
- ASTM D635-98, "Standard Test Method for Rate of Burning and/or Extent and Time of Burning of Plastics in a Horizontal Position" (*67*).

FRP manufacturers were contacted regarding quality control procedures and performance specifications. Manufacturers that were contacted included Newmark International, Inc.; the Shakespeare Company; Strongwell Corporation; and Valmont Industries, Inc. Utility companies that have developed performance specifications for FRP lighting poles were also contacted, and their specifications were obtained and reviewed.

The proposed performance specification for FRP lighting poles was developed using the information gathered from state DOTs, FRP pole manufacturers, and utility companies' specifications. Special effort was made to produce a specification that is general in nature with provisions that are independent of manufacturer and manufacturing method.

2.9.2 Performance Specifications and Acceptance Testing Procedures

The 2001 *Supports Specifications* includes a method for the evaluation of bending performance of FRP poles based on full-scale testing. The method presented is based on ASTM D4923-01, with modifications for safety factors according to ANSI C136.20. The current *Supports Specifications* does not address torsional strength and fatigue performance, but it references ASTM D4923-01 for provisions on those topics. Also, the *Supports Specifications* does not include provisions for the evaluation of weathering performance for FRP members.

The proposed performance specification for FRP lighting poles includes provisions for performance verification by testing. Tests include bending, torsion, fatigue, and weathering exposure. It also includes construction requirements, quality control, and acceptance criteria for FRP poles. The proposed performance specification is presented in a side-by-side specification/commentary format in Appendix F. Pertinent sections of the proposed performance specification are briefly discussed herein.

2.9.2.1 Design Loads

The proposed performance specification for FRP lighting poles references the 2001 *Supports Specifications* for the evaluation of the gravitational and environmental loads applied to the poles to ensure that the structural capacity of FRP poles match or exceed the load requirements prescribed by the 2001 *Supports Specifications*.

2.9.2.2 Deflection Limits

The lateral deflections limits proposed by the performance specification for FRP lighting poles are as follows:

- 15% of the pole height for lateral deflection under design dead-plus-wind loads (maximum deflection),
- 30-mm/m (0.35-in/ft) maximum slope at top of the pole under dead load from arm and attachments (maximum slope), and
- 5% of the pole height for lateral deflection under an 890-N (200-lb) lateral top load (stiffness requirement).

The lateral deflection is measured from the deflected tip end of the pole along a line perpendicular to the longitudinal axis of the undeflected pole. The maximum deflection limit constitutes a safeguard against highly flexible structures, whereas the maximum slope limit is a serviceability requirement for aesthetic considerations. These two limits are equivalent to the deflection limit requirements of Section 10.4.2.1 of the 2001 *Supports Specifications*. The stiffness requirement is intended to ensure that the pole does not deflect excessively when maintenance workers use ladders to access attachments located at the top of the pole.

2.9.2.3 Bending Test

The provisions for the performance evaluation of FRP poles in bending are based on ASTM D4923, with the following modifications:

- The safety factor against failure in bending is 2.0, which accounts for the variability of materials and performance often found in FRP products from different manufacturers and different production methods.
- When testing FRP poles with hand holes, the hand hole should be placed on the compression side of the pole, and the hand hole cover should be removed during testing.
- Deflections should be recorded with each 223-N (50-lb) load increment.
- The maximum acceptable permanent deflection after testing should not exceed 2% of the maximum deflection of the pole during the test. This deflection should be recorded 5 minutes after unloading.

The safety factor of 2.0 against failure in bending is greater than the safety factor of 1.5 specified by ASTM D4923. This additional safety accounts for the variability in mechanical properties of FRP. It also reflects the consensus of the safety factors among different FRP specifications reviewed and matches the bending safety factor requirements of ANSI C136.20.

2.9.2.4 Torsion Test

The provisions for the performance evaluation of FRP poles in torsion are based on ASTM D4923, with the following modifications:

- The safety factor against failure is 2.0.
- Specification of the load increments during testing is 223 N (50 lb), with deflections recorded at each load increment.

A safety factor of 2.0 against failure in torsion is used in order to match the safety factor used against bending failure.

2.9.2.5 Fatigue Test

The provisions for the fatigue performance evaluation of FRP poles are based on ASTM D4923, with the following modifications:

- Bending or torsional fatigue loads applied to the pole correspond to an equivalent wind pressure of 110 Pa (2.3 psf). (Note: The use of a wind pressure eliminates the dependence on wind speed for pressure computations for fatigue, since the formulations for wind pressure in the 2001 *Supports Specifications* have changed from a fastest-mile wind speed to a 3-second gust approach).
- Fatigue loads are applied for 106 cycles, with no more than 200 cycles per minute.
- Destructive testing of the pole for bending and/or torsion strength should be performed after application of fatigue loads.

2.9.2.6 Weathering Performance

The provisions for the weathering performance evaluation of FRP poles require that the surface of the pole be exposed to a minimum of 2,500 hours of accelerated weathering according to ASTM G154. The proposed specification requires the use of UV-B lamps with 313-nm wavelength and a testing cycle of 4 hours of UV exposure at 60 degrees Celsius and 4 hours of condensation at 50 degrees Celsius. UV-B 313 lamps produce the shortest wavelengths found in sunlight in the surface of the earth. This particular type of sunlight is known to be responsible for considerable polymer damage capable of affecting the mechanical performance of FRP poles.

After testing, the finished surface shall not exhibit fading, fiber exposure, chalking, cracking, or crazing. The weathering performance requirements are based on the weathering provisions of Caltrans 86.08.5 and Hawaii DOT HWY-C-06-98. ASTM standard G53 (*68*), originally referenced by the 2001 *Supports Specifications* for the evaluation of weathering resistance of FRP poles, was replaced by ASTM standard G154 in 2000.

2.9.2.7 Flame Resistance

FRP poles should be flame resistant in order to avoid propagation of fires induced by short circuits or fuel spills. The proposed performance specification for FRP poles requires that specimens tested for flame resistance according to ASTM D635 be manufactured with the same materials and identical manufacturing process as the actual poles. The specimens are considered flame resistant if they cease to burn before the gauge mark of 3.94 inches (100 mm) is reached. This requirement is adopted from the Hawaii DOT HWY-C-06-98 and also reflects the general consensus of flame resistance requirements among other specifications reviewed.

2.9.2.8 Fabrication Details

The proposed specification for FRP lighting poles contains provisions for construction of FRP poles. Those provisions include surface protection, finish, wiring access, hand holes, and pole caps. The construction provisions include suggested values of burial depth for direct-burial poles and suggested values of bolt circle for base-mounted poles according to ANSI C136.20. For direct-burial poles, the proposed specification for FRP lighting poles requires an antirotation device to prevent rotation of the pole and enhance lateral stability for wind loads.

2.9.2.9 Quality Control and Acceptance Criteria

The proposed performance specification contains a set of provisions on quality control and acceptance criteria for FRP materials. Those provisions specify that the manufacturer must submit detailed technical information about the proposed product for approval prior to production. The submitted technical information should include samples of the manufactured proposed product, stating clearly the kind and quality of the fibers and resin used in the composite. This is required because the structural properties of FRP members are highly dependent on the type and quality of glass fibers and resins used, as well as the manufacturing process. The new provisions on quality control also include requirements for storage and shipping of finished FRP products to prevent damage prior to installation.

2.9.3 Summary

A proposed performance specification was developed for FRP poles based on input from DOTs, FRP pole manufacturers, and the review of electric utility companies' specifications. The proposed performance specification covers acceptance testing procedures for FRP, including bending, torsion, fatigue, weathering testing, and limits on deflections. Fabrication requirements, flame resistance, and quality control of materials and finished product are also covered. The proposed performance specification for FRP lighting poles is presented in a side-by-side specification/commentary format in Appendix F.

2.10 INSPECTION, RETROFIT, REPAIR, AND REHABILITATION OF FATIGUE-DAMAGED SUPPORT STRUCTURES

The objective of this work was to address inspection, repair, retrofitting, and rehabilitation of fatigue-damaged support structures. A separate report prepared to address these issues is provided in Appendix G. A brief summary of the findings of that report is given herein.

A survey was sent to all state DOTs to determine which states repair fatigue-damaged support structures. The survey (presented in Appendix A) revealed that 26% of states that responded have repaired fatigue-damaged structures. However, only Michigan has developed a maintenance plan, procedures, or manual for retrofit and rehabilitation of sign support and luminaire structures. Contacts were made with state DOTs that indicated knowledge of repair to fatigue-damaged structures. The research team met with the investigators of NCHRP Project 10-38 and visited a local sign support manufacturer. The research effort included a comprehensive review of available relevant literature.

2.10.1 Fatigue Cases

State DOTs were contacted to collect information on fatigue problems that have been encountered. Documentation for only a few fatigue cases was found in the form of memorandums and internal reports. These cases were reviewed and are presented in the report. The cases found include fatigue cracking of support structures in Michigan, Wisconsin, and Wyoming. Cracks due to galvanizing that were observed in a visit to a local sign support manufacturer are also discussed.

2.10.2 Inspection

Relevant literature was reviewed. State DOT offices and FHWA engineers were contacted to gather additional information about the methods used for inspection of support structures. The report presents inspection procedures for high-mast luminaires and sign support structures and summarizes methods of weld inspections.

2.10.3 Repair and Rehabilitation Procedures

Relevant literature was reviewed. *NCHRP Report 206: Detection and Repair of Fatigue Damage in Welded Highway Bridges* (*69*) was studied in detail. The various techniques used to extend the fatigue life are summarized, and descriptions of repair methods for specific fatigue cases are illustrated.

2.10.4 Vibration Mitigation Methods

Vibration mitigation devices can be used to reduce the loads that cause fatigue and extend the life of the structure. The most effective type of mitigation depends upon the type and direction of wind loading phenomena causing vibration. The latest information that could be obtained from NCHRP Project 10-38 and 10-38(2) regarding vibration and mitigation devices was reviewed and included in the appendix. These devices include commercially available dampers such as the Hapco and Alcoa dampers, along with new configurations suggested by NCHRP Project 10-38(2) such as the strut and liquid-tuned dampers. A complete evaluation of the methods available to mitigate fatigue-induced effects is included in the final results of NCHRP 10-38(2).

2.11 STRATEGIC PLAN FOR FUTURE *SUPPORTS SPECIFICATIONS* ENHANCEMENTS

The 2001 AASHTO *Supports Specifications* was a direct result of the extensive research work performed under NCHRP Project 17-10. Although the work on this project was initiated in 1995 and completed in 1997, the 2001 *Supports Specifications* was published in summer 2001. NCHRP Project 17-10(2) was initiated in 1999 to address further issues for the refinement of the *Supports Specifications*. Results of NCHRP Project 17-10(2), as well as future enhancements, must be implemented in the *Supports Specifications* in order to keep the document the most current and up to date. This section provides an outline of a strategic plan for enhancing the *Supports Specifications* and maintaining its currency in the future.

2.11.1 Introduction

The *Supports Specifications* is the main document for the design of structural supports in the United States. It governs the design, construction, and safety of the various types of structural supports and is currently adopted by all state DOTs and by other agencies. Although the 2001 edition has just been published, several revisions have already been identified and are ready for inclusion in the specifications. Also, revisions will be necessary to integrate the results of NCHRP Projects 17-10(2) and 10-38(2), updates in material or load specifications referenced by the *Supports Specifications*, and other technical information that will be developed in the future. Therefore, a mechanism for making changes and upgrades on a regular basis is needed in order to efficiently manage the document in the future.

The information in the 2001 *Supports Specifications* is comprehensive in nature. It incorporates several new engineering concepts, design criteria, structure types, and materials. Additionally, as new information becomes available through research and practice, consideration should be given for incorporating this new information in the specifications. As such, it will be difficult to maintain the specifications and their state-of-the-art features using solely committee volunteer effort. A plan is needed that will outline a mechanism by which the *Supports Specifications* will be regularly updated, maintained, and published in a timely manner.

Furthermore, it was recommended by NCHRP Project 17-10 in 1995 to revise the *Supports Specifications* using an allowable stress design (ASD) approach. This decision was primarily based on the lack of vital information necessary to establish a rational LRFD approach and on the fact that the existing ASD specifications at that time had not been revised for a number of years and contained several deficiencies that needed upgrading. It was therefore recommended to bring the specifications to an up-to-date ASD format that would be easily upgradeable to an LRFD approach in the future.

2.11.2 Conversion to an LRFD Philosophy

2.11.2.1 Problem Statement

The 2001 *Supports Specifications* adopts an ASD approach and format. However, the recent trend for most design specifications—including the AASHTO *LRFD Bridge Design Specifications*, AISC *Manual of Steel Construction*, and the *Aluminum Design Manual*—is to adopt an LRFD method. The move toward adopting LRFD is due to the more consistent approach that the method provides with respect to strength evaluation and structural safety. An LRFD *Supports Specifications* would also be easier to incorporate future engineering developments and test information into, since the specifications derives much of their information from national specifications that are based on LRFD. LRFD will greatly facilitate updates to the *Supports Specifications* as material specifications evolve or new materials emerge. Engineers would also benefit from a unified design approach that is consistent with other commonly used design specifications and simpler to apply. Hence, converting the *Supports Specifications* to an LRFD philosophy and format is highly recommended for the next major update of the document.

2.11.2.2 LRFD Philosophy

The LRFD methodology is primarily based on the following equation:

$$\Sigma \gamma_i Q_i \leq \phi R_n \tag{Eq. 29}$$

Where

γ_i = load factor, a statistically based multiplier applied to force effects;
ϕ = resistance factor, a statistically based multiplier applied to nominal strength;
Q_i = force effect; and
R_n = nominal resistance.

The factors that are applied to loads and resistance are the intended safety factors that account for uncertainties associated with the loads, materials, and various design assumptions. The main characteristic of the LRFD philosophy is that it provides a consistent approach to strength evaluation and structural safety. The approach takes into account the variability in the behavior of structural elements and accounts for safety in a more rational way. The LRFD method assigns different overload factors γ_i to loads depending on the type of load and on the factored load combination that must be considered. The resistance factor ϕ is another safety-related provision, which accounts for the type of member, material, and limit state considered. Those two factors could be adjusted to ensure more uniform reliability in the design.

Although literature on LRFD is available for different materials, the application of the method to support structures cannot be achieved by simply adopting the load and resistance factors implemented in other specifications. Probability-based studies and calibrations with ASD are needed to establish the load and resistance factors pertaining to support structures. Adding to the complexity of the needed work for establishing an LRFD specification is the wide range of materials, including fiber-reinforced composites, that are covered by the specifications.

The development of the LRFD method is based on the following main considerations:

- Probability-based model,
- Calibration with the allowable stress specifications,
- Judgment and past experience, and
- Comparative design studies of representative structures.

The LRFD approach requires that a procedure be available to determine values for the resistance factors and the load factors. For a structural member or element designed according to current specification, it is possible to compute the relative reliability of this design from data defining probability distribution and statistics of the resistance, loads, and load effects. The relative reliability is expressed as the reliability index. By repeatedly determining the reliability index for many structural designs, the relative reliability of different structural members built from different structural materials can be compared. For a selected value of the reliability index, using reliability analysis methods, it is possible to compute the resistance and load factors. The process is elaborate and requires a considerable amount of work, especially if new structural materials are investigated.

Hence, to adopt an LRFD approach for support structures, an extensive study is needed to develop the load and resistance factors pertaining to this category of structures. Probability-based studies, considering the variables involved in the design of different categories of support structures, may result in design parameters that differ for each category of structures. Experimental studies may also be needed to establish reliable strength equations for emerging materials or for special shapes used extensively for support structures.

2.11.2.3 Reliability-Based Design Approach

The LRFD method is a probability-based design method, involving predicting the probability that loads imposed on a structure or member would cause the structure or member to cease its intended function. Load factors are developed to account for uncertainties in estimating dead and live loads. Resistance factors are developed to account for uncertainties in material strengths, dimensions, and quality of construction. In order to develop load and resistance factors for support structures, a statistical study is needed to simulate variations in materials and loadings that are unique to support structures. Variations in resistance of typical materials and shapes would need to be studied, as well as variations in loadings.

The variations can be simulated using the Monte Carlo method, which generates random numbers to evaluate a function containing several variables expressing the scatter of material properties, physical dimension variations, and extreme loadings. The values are mapped to a resulting frequency distribution that allows for the probability of exceedance to be determined.

The variation in resistance can be determined by using Monte Carlo simulations. Statistics must be generated for limit states for support structures. The term limit state describes a circumstance where a structure or one of its components does not perform its intended function in terms of strength or serviceability. Statistical information on variations in material properties and physical dimensions of structural shapes is obtained. Individual variations in material properties and physical dimensions are combined to provide a probable resistance (e.g., bending moment, shear strength, tensile strength, compressive strength) for a particular structural shape.

The variation in loading can also be determined using Monte Carlo simulations. Statistical data on loadings will need to be reviewed in the context of support structures, especially statistics on extreme wind load effects on support structures. These extreme wind-load-effect statistics may be generated using Monte Carlo simulations. In addition, the most significant load ratios for support structures will need to be determined. These load ratios will be much different for support structures than for building or bridge structures.

Reliability is the percentage of times that the strength of a structure will equal or exceed the maximum loading applied during the structure's estimated life. Reliability indexes indicating the reliability associated with the *Supports Specifications* will be determined using the statistical information generated on resistance and loadings. These indexes will be determined using the advanced first-order, second-moment (AFOSM) method. Then the AFOSM method will be used to determine load and resistance factors for support structures that optimally achieve the reliability overload ratios associated with these structures. The calibration procedure that is required to establish load and resistance factors involves the following steps:

1. Select typical structures.
2. Establish a statistical database for load and resistance parameters.
3. Develop load and resistance models.
4. Develop a reliability analysis procedure.
5. Select a target reliability index.
6. Calculate load and resistance factors.

Proper implementation of an LRFD method, which provides appropriate load and resistance factors that are specific to support structures, will lead to a more consistent approach to strength evaluation and structural safety than any other design approach will. Support structures will be designed for the same reliability regardless of the material used. The specifications will be consistent with other nationally recognized specifications; this consistency will facilitate future updates of the document. Engineers will also benefit from a unified design approach that is rational and compatible with other design specifications.

2.11.2.4 Step-by-Step Procedure for the Determination of Load and Resistance Factors

The literature has abundant information on methods and procedures used to convert design specifications to LRFD approach. Much of this information has been previously reported by NCHRP Project 17-10 (*1*). Extensive work has also been performed in converting the bridge specifications into an LRFD philosophy (*7*, *70*, *71*). The work involved the development of load and resistance models, selection of the reliability analysis method, and calculation of the reliability indexes. This work can serve as a model in converting the *Supports Specifications* to an LRFD philosophy. However, it should be noted that structural supports substantially differ from bridge structures in many aspects, and these differences must be considered in the conversion process. The low redundancy, high structure flexibility, predominance of wind loading, and susceptibility to wind vibrations are only a few of the characteristics of structural supports that must be considered in the LRFD model to ensure correct prediction of structural performance.

A general procedure for the calculation of the load and resistance factors for the *Supports Specifications* may be outlined as follows:

1. **Selection of representative structures.** A large number of structural supports will be selected for analysis from various geographical regions of the United States. The structures will represent the different support types, materials, and configurations. The selection will focus on the typical structures that are commonly used in different parts of the United States.
2. **Establishing a statistical database for the load parameter.** Data on the different type of loads pertinent to support structures will be gathered. Updating the *Supports Specifications* for the most recent wind loads and map will also be part of this work. The statistical model of load, *Q*, will include the cumulative distribution function (CDF) and/or the mean, bias, and coefficients of variation of the load derived using the background information for wind and other loads in the *Supports Specifications*.
3. **Establishing statistical data for the resistance parameter.** Statistical data related to the materials used for support structures will be gathered. These data include material tests, component tests, and field measurements.
4. **Formulation of strength limit states.** Strength limit states for the various components of support structures will be formulated, and the basic parameters of resistance needed for those limit states will be identified. This step will involve considering the various straining actions (axial force, bending, shear, etc.) and the types of structural member that are commonly applied to structural supports (e.g., multisided shape, round tube, and I-shaped). This step is particularly important for new materials where strength limit states have not been formulated.
5. **Development of the resistance model.** Resistance models will be developed for the different structural materials, components, and limit states identified in Step 4. The most current LRFD specifications for the different materials will be used as the basis for the derivation of the resistance models. For new materials, such as fiber-reinforced composites, the most recent literature information will be used as the basis for the proposed strength

equations. Other factors such as ductility and durability will also be considered in the resistance model.

The statistical model of resistance, R_n, will include the CDF and/or the mean, bias, and coefficients of variation of the resistance derived using the available statistical database of Step 2.

6. **Selection of the target reliability index.** The reliability index, β, will be calculated for a wide spectrum of real support structures designed according to the 2001 *Supports Specifications*, as well as structures designed according to earlier versions of the *Supports Specifications*. This calculation is particularly necessary, since the 2001 specifications have just been published and documented experience using this specification is not available. A target reliability index, β_T, will be selected to provide a consistent and uniform safety margin for all materials. The target reliability index may vary for different types of support structures depending on the importance of the structure and the intended level of safety. Care should be taken in selecting the target reliability for the different support structures. For example, the *AASHTO LRFD Bridge Design Specifications* (*7*) adopts a value of 3.5; however, arbitrarily adopting this value could result in very conservative designs and uneconomical support structures.
7. **Reliability analysis.** Reliability is measured in terms of the reliability index, β, which is a measure of structural performance. Once a target reliability index, β_T, has been selected, the calibration of load factors and resistance factors to achieve approximately the value of the target reliability may proceed using an iterative procedure as described by Nowak (*70, 71*). A larger number of hypothetical support structures with variations in design parameters will be used in the calibration process.
8. **Selection of the load and resistance factors.** Load factors, γ_i, will be calculated so that the factored loads have a predetermined probability of being exceeded. Resistance factors, ϕ, are calculated so that the structural reliability is close to the target value, β_T.

Experimental work may be needed if only limited data can be found for certain materials. Fiber-reinforced composites are an example where structural testing may be required to develop strength equations.

A less rigorous alternative to conducting the full calibration described above is using a confidence limit approach. In this case, the resistance factor, ϕ, would be selected such that the 95th percentile resistance is achieved. This approach could provide quick results and interim data until full calibration could be performed.

2.11.2.5 Time and Cost Estimates

It is estimated that this project will cost approximately \$500,000 and be performed over a 30-month period.

2.11.3 Future Enhancements

Technical topics that need to be investigated for further improvement of the *Supports Specifications* are given herein with a brief description of each topic.

2.11.3.1 LRFD Conversion

Details related to the work needed to convert the specifications to an LRFD philosophy and format are given in Section 2.11.2. It is highly recommended that subsequent editions of the 2001 *Supports Specifications* adopt an LRFD approach. Since the 2001 specifications offer the most current ASD information, it is prudent to begin the conversion process immediately.

2.11.3.2 New Wind Map and Wind Provisions

The proposed *Supports Specifications* is based on information in ASCE 7-95. ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures*, has just been published. The ASCE 7-98 standard should be reviewed for updates that could be applicable to the *Supports Specifications* wind loading criteria. Important considerations in the newly published ASCE 7-98 document include the load factor for wind, a wind directionality factor, and changes in the hurricane contours on the wind map.

2.11.3.3 Structure Design Differences Between the 1994 and 2001 Supports Specifications

Perform a study to compare design differences that occur on an overall structure level between the 1994 and 2001 specifications. This study would encompass major design evaluations of typical support structures across the country. The design examples developed as part of NCHRP Project 17-10(2) would be used as a starting point for the design comparisons. The study will also investigate the effect of different load types (wind, fatigue, etc.) in controlling the design of the structure. This would considerably simplify the design calculations and provide the designer with better understanding of the intent of the specifications.

2.11.3.4 Fatigue and Vibration

Although a survey was distributed to all state DOTs and many persons were contacted for additional information, the question of whether noncantilevered sign support structures are susceptible to fatigue from galloping and vortex shedding has not been assertively answered. There is need for additional investigation in this regard by (a) additional contacts and review of failure cases and (b) laboratory and field testing/

monitoring. Research is needed to evaluate the dynamic behavior of existing support structures that exhibit fatigue and/or vibration problems. This research should include monitoring displacement amplitudes, calculating stresses in the critical connections, and comparing the response with the static loads recommended in this research.

Several states have attributed problems to fatigue from excessive vibration, but these cases really were not analyzed to determine which type of wind loading is causing the problem. A uniform inspection procedure used by various states to help them accurately determine the cause of the fatigue problem would be useful. A database or collection of data could then be analyzed. Such information as the vibration amplitude and direction, wind speeds, and uniformity of the winds would be needed.

Regarding deflection limits, a range of noncantilevered support structures was checked against the current $L/150$ requirement (where L equals the span of the structure), and all of the structures analyzed met the requirement. However, the $L/150$ requirement was really developed for checking stiffness of monotube structures. Further study is needed to determine if the $L/150$ is really appropriate for noncantilevered truss structures.

Based on the assumption that noncantilevered sign support structures are susceptible to the same vibration phenomena as cantilevered support structures, the finite element method was used to determine whether the fatigue loads recommended for cantilevered sign support structures are appropriate for noncantilevered structures. Recommendations were made. However, the impact of the recommendations on design was not evaluated. Before the recommendations are adopted, the effect of the recommendations should be evaluated in terms of increasing weight, geometry, cost, etc.

2.11.3.5 Categorization of Fatigue Connection Details

The research team developed recommendations regarding categorization of connection details for noncantilevered support structures by reviewing (a) the categorization already adopted for cantilevered sign support structures and (b) connection categorization used for bridge design. Actually, the categorization adopted for cantilevered sign support structures was developed by reviewing bridge connection categories, plus very limited testing. There is need to conduct additional fatigue testing of typical sign support structure details to categorize connection details accurately.

2.11.3.6 Vibration Mitigation Methods

Available information from NCHRP Project 10-38(2) regarding effectiveness of vibration mitigation methods was synthesized. It appears that the effectiveness of such vibration mitigation methods is being measured in terms of reduction in displacement amplitude and reduction in vibration duration after the load vanishes (i.e., truck wind gusts). The current research appears to be limited to cantilevered support structures. The research team's colleagues have also indicated their idea regarding how much of a fatigue load reduction can be accounted for if mitigation devices are used. Additional research is needed to demonstrate the effectiveness of vibration mitigation devices in terms of stresses and to relate the effectiveness to reduction in fatigue design loads. A range of support structure configurations should be included. Also, the final results of NCHRP Project 10-38(2) need to be included.

2.11.3.7 Variable Message Signs

Variable message signs (VMSs) have been introduced in the past few years. Since their introduction, their use has been increasing rapidly and their design and configuration are constantly changing. VMSs have dimensions, configurations, and weights that differ significantly from those of usual signs and thus could highly impact the design of the supporting structure. Current VMS sizes have reached 36 feet long by 12 feet high by 4½ feet deep, with a weight exceeding 5,000 pounds. The current *Supports Specifications* does not address VMSs or their supporting structures. With the increasing use of VMSs, it is imperative that future versions of the *Supports Specifications* include guidance and provisions for the design of VMS supporting structures and their foundations.

Two failures of VMS supporting structures have been reported. One case was in Virginia where the sign was attached and supported by a cantilever structure. Another case was in California where a Caltrans 18-inch monotube structural support that was designed to carry a VMS failed at a fatigue-generated crack just above the weld at the base plate. Specific design considerations should be developed to guide the designer and ensure safer designs in the future.

VMSs are heavier and much larger than normal signs are. VMSs are attached to the supporting structure in such a manner that torsional loads become significant. Because of the size of VMSs, the effects of the aerodynamic forces are important and must be carefully evaluated. Truck-induced wind gusts and vortex shedding cause vibrations and fatigue. The presence of the torsional loads compounds the vibration and deflections induced by the wind and aerodynamic forces. The connection or method of attaching the VMS to the support structure is also important and must be addressed by the engineer.

Although the current AASHTO *Supports Specifications* does not address VMSs or their supporting structures, engineers have erroneously used the current *Supports Specifications* in the design of such structures. The VMS was treated as a normal thin and flat sign. This treatment could result in faulty designs, since several of the provisions of the current *Supports Specifications* may not directly apply to structures supporting VMSs. For example, the section on wind loads in the current *Supports Specifications* contains information that was developed mainly for wind pressures acting on thin flat-faced signs, not VMS. The wind pressure, drag coefficients, and uplift forces must be revised to account for the VMS structures that are much larger in size (structures that could have a roof of 4 to 5 feet and a bottom of 3 to 4 feet).

Additional design information is needed in order for the engineer to design safe and economical VMS structures. Both theoretical and field studies are needed to document the behavior of the VMS structures and to develop guidelines for their proper design and construction. Information on the effects of high torsional loads in combination with dead and live loads must be evaluated. Complex aerodynamic forces from wind and moving traffic as a result of the large physical dimensions of the sign warrant careful evaluation. The induced movements and vibrations cause fatigue and could be detrimental to the sensitive electronic equipment of the VMS. Wind tunnel tests are needed to develop the necessary information on wind pressure and drag coefficients. Ice loads should be investigated, and water ponding on the flat surfaces should also be taken into consideration. The method of connecting the VMS to the supporting structure is an important design consideration. The center of gravity of the VMS is usually placed off the centerline of the horizontal support. This placement can produce significant torsional loads and deflections. The VMS could be integrated in the supporting truss structure to eliminate such torsional loads. Foundation design requirements must also be reviewed in light of the larger loads that are transmitted in this case. Finally, it should be useful to develop a mechanism for interaction where the analysis and design efforts of the engineer/fabricator of the VMS and the engineer/fabricator of the support structure are coordinated in order to achieve a safe and economical design.

2.11.3.8 Bridge-Mounted Signs

There has been a tendency to place large signs and signs with maintenance walkways on bridges in lieu of adding an additional structure, either as an effort to save money or because of utilities or drainage that conflict with foundations for structures. However, these structures are difficult to fabricate and construct, as well as significantly higher in cost. Potential problems and safety hazards may arise if such structures are used without adequate attention to special design considerations. The current specifications provide no information on the design of sign structures connected to bridge structures.

Research is needed to investigate the important issues regarding the design and construction of bridge-mounted signs, and the information gained should be incorporated into the *Supports Specifications*. At a minimum, the following issues should be investigated:

- The effect of sign loads on existing bridge girders;
- The effect of vibration and deflection of bridge girders on the members that support the signs and maintenance walkways;
- Forces at connections to the bridge girders and bridge deck (and guidelines on acceptable methods of connecting to steel and prestressed concrete girders, concrete slab, and parapet walls);
- The need for DOTs to furnish complete and up-to-date information regarding the bridges to which these structures are connected, along with project documents;
- Requirements for verifying the adequacy of an existing bridge to support bridge-mounted signs;
- Guidelines on design responsibilities (i.e., who is responsible for what part of each design);
- The issue of whether specifications should prohibit mounting VMSs directly to bridge girders; and
- Cost comparisons of bridge-mounted structures versus cantilever or span structures.

2.11.3.9 Strength of Fiber-Reinforced Composite Support Structures

The use of fiber-reinforced composite, especially fiberglass-reinforced plastic (FRP), as support structures is rapidly increasing. The FRP products, however, have varying physical properties that depend on the materials and fabrication method used. No design specifications or codes are currently available that cover the design requirements for FRP structures.

An experimental program is needed to evaluate the strength properties of cross-sectional shapes typically used for support structures and to verify the strength equations given in the 2001 *Supports Specifications*. The study should focus on commonly used materials, namely polyester resin and glass fiber reinforcement. Products from the three major manufacturing methods (i.e., pultrusion, centrifugally casting, and filament winding) should be considered. For each of the manufacturing methods, the testing program should include several parameters, such as cross-sectional shape, fiber orientation, glass-resin ratio, wall thickness, taper, fatigue strength of mast arm connections, and base plates. The tests should consist of full-scale structural testing, as well as material (coupon) testing, in order to develop generalized equations for predicting the strength of the cross section in flexure, shear, axial, and torsion applications. The experimental work should also establish local buckling criteria for the section. Information on the stiffness and deflection of these structures should also be evaluated.

2.11.3.10 Gust Effect Factors for Flexible Structures

The gust effect factors for wind-sensitive and non–wind-sensitive structures provided in Sections 2.6.3, 4.4, 4.5.2, and

4.5.3 of the "Wind Loads Report" in Appendix B should be verified by physical wind tunnel testing on typical support structures. Insufficient research is available on gust effect factors for flexible support structures, and the only published information was developed primarily for buildings. As part of this work, the impact of the proposed gust factor equations for flexible structures on the design of support structures should be assessed.

2.11.3.11 Strength of Round and Polygonal Tubular Sections

The allowable stress formulas for steel polygonal tubes in the current *Supports Specifications* are based on analytical background and only limited experimental work. The allowable stress equations for bending of round tubular sections are mainly based on early work performed by Plantema (*72*) and Schilling (*73*) that was analytical in nature. Experimental work was very limited and addressed only round tubes with yield points of 36 to 52 ksi. However, the allowable stresses for polygonal cross sections are based on testing of sections made of higher-strength steels (ASTM A572 plate steel material). This research program at the Transmission Line Mechanical Research Center, however, was limited to 16 test specimens. More tests are needed on polygonal tubes to verify the available strength equations. Typically, ASTM A595 coil steel is used for the manufacture of round tubes, and A572 Grade 55 or 65 plate steel is used for round or polygonal shapes. Preliminary test data indicate that the type of steel and fabrication process do have a significant effect on the bending strength of the cross section.

An experimental program is needed to determine the strength, and hence allowable stresses, for sections of different steels and fabrication processes. The experimental work should consider different cross-sectional shapes with the two types of steels commonly used. Updated strength equations (or allowable stress equations) should be derived from this work. Another parameter to consider in this study is the upper limit of D/t, where D equals the diameter of the tube and t equals the thickness of the tube. No flexural strength test data are available for D/t greater than $12{,}000/F_y$, where F_y equals the yield stress. Information on the bending strength of tubes with D/t exceeding that limit is needed for the limited cases where such a slenderness may be used.

2.11.3.12 Bending About the Diagonal of Square and Rectangular Steel Tubes

A method for the design of square and rectangular tubes bent about the diagonal axis was developed through NCHRP Project 17-10(2). Generally, the method provides greater allowable moments as compared with calculations in accordance with the 2001 *Supports Specifications*. Experimental verification of the new design method is needed.

Full-scale structural bending tests are proposed to evaluate the performance and strength of cantilevered structures with square or rectangular cross sections. The experimental program should consider various sizes, as well as sections with sharp and rounded corners. The objective should be to determine the effect of the strain distribution in the wall on local buckling. Also, the influence of the corner on the strength of the section when bent about the diagonal should be studied. The data should provide information on the usable strength of the section and, hence, the allowable stresses that could be permitted when bending about the diagonal occurs. Compact, noncompact, and maximum slenderness width-thickness ratio limits for sections bent about the diagonal should be determined. The proposed design equations should be checked against the actual test results, and further adjustments should be made to produce an improved design model.

2.11.3.13 Drag Coefficient Transitions for Multisided Sections

A drag coefficient transition equation was developed as part of this study to provide values for sections that transition from multisided to round shapes. The equation was based on analytical procedures. Experimental verification using wind tunnel testing is required to validate the proposed equation.

2.11.4 Maintenance of the Specifications

The current *Supports Specifications* (2001) is comprehensive and quite detailed as compared with previous editions. It references a number of major national specifications, which are continually being updated to incorporate results of new research and practice. Thus, there is a greater need to keep the specifications current and reflecting the most recent technical information. This upkeep should be achieved through an organized process that incorporates into the specifications the results of new studies and technologies in a systematic and routine manner. Maintaining the specifications and their state-of-the-art features using solely committee volunteer effort as performed in the past is difficult and impractical.

To address this problem, the different approaches adopted by national organizations in updating and maintaining their specifications were reviewed. Such specifications included the AASHTO LRFD Bridge Design Specifications, ASCE 7 Minimum Design Loads for Building and Other Structures, International Building Code, AISC Manual of Steel Construction-LRFD, and ACI 318 Building Code Requirements for Structural Concrete. Important in this evaluation were how the changes are initially proposed, how the changes are addressed, and how they are ultimately incorporated into the revised specification. The cycle of making specification revisions, publishing interims, and the relationship to other major standards were noted. For example, major updates could occur every 3 or 6 years to conform to specific codes such as ASCE 7 or AISC, and interims could be provided every year.

Researchers of NCHRP Project 12-42 were contacted to discuss their approach for updating the AASHTO LRFD Bridge Design Specifications. The process was noted for how these researchers obtained input from AASHTO technical committees, how the chairs of the AASHTO technical committees conducted reviews, and how Modjeski and Masters helped to address the changes. A similar, but less extensive, approach may be suitable for the *Supports Specifications.* The technical committee under AASHTO HSCOBS specifically designated to structural supports, T12, should play an active role in overseeing future enhancements to the specifications.

Elements of the suggested mechanism for regularly updating and maintaining the *Supports Specifications* may be outlined as follows:

1. Specific areas of the specifications that need the most updates will be identified. The University of Alabama at Birmingham (UAB) will organize a national committee comprised of design professionals and manufacturers that will prepare the agenda items on needed updates.
2. T12 will consider the following:
   i. Suggested changes submitted by the national committee.
   ii. Suggested changes submitted by state bridge engineers.
3. T12 will prioritize topics for future study and recommend changes to be made.
4. T12 will involve HSCOBS members or contractors to perform the needed work. If significant funding is needed to conduct the work, NCHRP or state “pooled” funds will be used to provide the financial support.
5. Proposed changes or results of research work will be reviewed and assessed by UAB, in conjunction with the national structural supports committee, to determine the impact on the specifications and to ensure the proper specifications format.
6. T12 will review the results of the assessment and the proposed changes.
7. State DOTs will review the proposed changes and provide comments to T12.
8. HSCOBS will vote to adopt the proposed changes.

2.11.5 Summary

Despite the dedicated efforts over the past few years to improve the *Supports Specifications*, numerous areas of the specifications still need to be revised and updated. This need is primarily due to the constant revamping of national specifications, as well as the rapid development of technical information and the emergence of new materials relevant to structural supports. This section outlined a strategic plan for converting the supports specifications to the desired LRFD philosophy, discussed future enhancements that are necessary to keep the specifications up to date, and provided suggestions for maintaining the currency of the specifications in the future.

CHAPTER 3

INTERPRETATION, APPRAISAL, AND APPLICATION

3.1 WIND LOADS REPORT

A review was made of the new wind loads provisions in the 2001 *Supports Specifications*. As part of this review, an in-depth study of the new 3-second gust wind map and its impact on the calculated design wind pressures on structural supports was performed. Flexibility of the structural support as it affects the gust effect factors was also considered, as well as methods for incorporating the flexibility of the structure in the wind loading computation.

A complete document, entitled "Wind Loads Report," is provided in Appendix B. Topics addressed in the report include the following:

- The basis for wind pressure calculations in the 1994 and 2001 specifications,
- Wind load comparisons between the 1994 and 2001 specifications,
- Recommendations for gust effect factors for flexible structures, and
- Suggested research needs pertaining to wind loads for structural supports.

3.2 FATIGUE AND VIBRATION IN NONCANTILEVERED SUPPORT STRUCTURES

Recommendations were developed for the fatigue and vibration design of noncantilevered support structures. The investigation included a review of relevant literature, contacts with state DOTs and support manufacturers to identify problems that have been encountered, meetings with investigators of related projects, and an in-depth analytical study to formulate design load recommendations. Fatigue loads and deflection criteria for noncantilever structures were proposed for inclusion in the *Supports Specifications*. This task also included investigation of the effectiveness of gussets, categorization of connection details for noncantilevered support structures, and the effectiveness of vibration mitigation devices. A comprehensive report that includes the methodology and all findings is provided in Appendix C.

3.3 ANCHORAGE TO CONCRETE

A simplified anchor bolt design procedure for structural supports based on state-of-the art information on the design of anchorage to concrete has been developed. The simplified procedure produces designs that are consistent with average values reported by state DOTs.

3.4 FOUNDATIONS

Design guidance on selection criteria for foundation types is provided. The design guidance includes additional information on two foundation types that are suitable for structural supports: the screw-in helix and the directly embedded foundations.

3.5 DRAG COEFFICIENT TRANSITIONS FOR MULTISIDED TO ROUND CROSS SECTIONS

A drag coefficient transition equation has been developed based on analytical procedures for multisided tapered poles with geometry approaching round sections. The accuracy of the proposed design method should be verified through experimental results. Detailed analyses using the proposed transition equations are given in Appendix D.

3.6 CONNECTION PLATE AND BASE PLATE FLATNESS TOLERANCES

Current practices were reviewed to identify the need for structure-specific connection plate and base plate flatness tolerances for erection. The majority of information was obtained from state DOTs and contacts with support structure manufacturers. The effect of the flatness tolerance on practicality and manufacturing was considered. A new proposed section on tolerances for flatness of base plates and connections is recommended for addition to the *Supports Specifications*.

3.7 SQUARE AND RECTANGULAR STEEL TUBES BENT ABOUT A DIAGONAL AXIS

A design method was developed to determine the allowable bending capacity of square and rectangular steel tubes bent about the diagonal axis. The proposed design procedure applies to square and rectangular tubes that are bent about a skewed axis. The tube must also be designed separately for the maximum moments about each geometric axis. The recommended procedure is outlined for inclusion in the *Supports Specifications*. Appendix E provides the basis for the proposed design method, as well as design examples illustrating the use of the method.

3.8 PERFORMANCE SPECIFICATION FOR FIBER-REINFORCED COMPOSITES

A proposed performance specification was developed for FRP poles based on input from DOTs, FRP pole manufacturers, and electric utility companies' specifications. The proposed performance specification covers acceptance testing procedures for FRP, including bending, torsion, fatigue, weathering testing, and limits on deflections. Manufacturing requirements, flame resistance, and quality control of materials and finished product are also covered.

3.9 INSPECTION, RETROFIT, REPAIR, AND REHABILITATION OF FATIGUE-DAMAGED SUPPORT STRUCTURES

Recommendations were made for the inspection, repair, retrofitting, and rehabilitation of fatigue-damaged support structures. A survey was sent to all state DOTs to determine which states conduct routine inspections and which states have developed procedures for repairing and rehabilitating fatigue-damaged support structures. Other than the procedures provided by the Michigan DOT, very little useful documentation was gathered from state DOTs. Information from state DOTs and various other sources—such as published reports on weld inspection, meetings with sign support manufacturers, and discussions with other researchers—was synthesized into a comprehensive report presented as Appendix G.

3.10 DESIGN EXAMPLES

Sixteen design examples illustrating the use of the 2001 *Supports Specifications* were developed in this research project. Because proposed changes in the *Supports Specifications* will impact the reliability of these examples, the examples are not published.

3.11 STRATEGIC PLAN FOR FUTURE *SUPPORTS SPECIFICATIONS* ENHANCEMENTS

Despite the dedicated efforts over the past few years to improve the *Supports Specifications*, several areas of the specifications still need to be revised and updated. This need is primarily due to the constant revamping of national specifications, as well as the rapid development of technical information and the emergence of new materials relevant to structural supports. A number of technical topics for improvements of the specifications are outlined. Additionally, it is necessary to begin the process of converting the specifications to a load and resistance factor philosophy. A strategic plan for converting the specifications to the desired LRFD philosophy is presented. The strategic plan delineates future enhancements to the specifications and suggestions for maintaining the currency of the specifications.

CHAPTER **4**

SUMMARY, CONCLUSIONS, AND SUGGESTED RESEARCH

4.1 SUMMARY AND CONCLUSIONS

The objective of this project was to provide additional enhancements to the *Supports Specifications*. Topics covered in this work included wind loading, fatigue design, foundation selection, anchor bolts, flatness tolerances, bending about the diagonal, drag coefficient transitions, FRP performance specifications, design examples, and a strategic plan for conversion to LRFD. General conclusions from this research can be summarized as follows:

- A report entitled “Wind Load Report” was prepared to explain the new wind provisions of the 2001 *Supports Specifications* and to compare them with previous editions of the specifications.
- Recommendations for fatigue and vibration design of noncantilevered structures were made based on finite element analysis and previous experimental work on cantilevered support structures.
- The state-of-the-art procedure adopted by ACI 318-02 on anchorage to concrete was reviewed and formed the basis for the proposed simplified procedure.
- Foundation selection criteria were provided for support structures.
- A transition equation for estimating the drag coefficient of multisided tapered poles when the cross section approaches a round section was developed.
- Tolerances for flatness of base plates and connections plates were recommended.
- Design guidelines for steel tubes bent about the diagonal were formulated, and a design procedure was outlined.
- Performance specification and acceptance testing procedures were developed for fiber-reinforced composite support structures.
- A synthesis report on the inspection, repair, and rehabilitation of fatigue-damaged support structures was prepared.
- Sixteen design examples were prepared to illustrate the use of the 2001 *Supports Specifications*.
- A strategic plan for the future enhancement of the specifications and for converting the specifications to LRFD philosophy was prepared.

4.2 IMPLEMENTATION

The 2001 *Supports Specifications* offers new information and significant updates in the design of highway signs, luminaires, and traffic signals over previous editions of the specifications. NCHRP Project 17-10(2) proposes new updates that will further enhance the specifications. The following suggestions are given in an effort to encourage and enhance the implementation of the *Supports Specifications*:

- Perform a study to compare design differences that occur on an overall structural level between the 1994 and 2001 specifications. This study will involve design evaluations of typical support structures across the country.
- Develop a design guide to provide state-of-the-art design information and design aids in support of the specifications. The design guide will provide an informal and informative discussion of topics in the specifications.
- Conduct a lecture series introducing the proposed *Supports Specifications*. A design handbook and course notes with detailed examples of typical, as well as specific, designs will be prepared for the workshop.
- Develop a web page for the *Supports Specifications* that will be a resource for users of the *Supports Specifications*. General information and guidance on specific issues in the specifications will be covered.

4.3 ADDITIONAL IMPROVEMENTS

An objective of this project was to identify items that need future research and modification beyond what will be accomplished in this project. The following sections provide a brief description of those topics that are recommended for future research. Additional information is given in Section 2.11. The topics recommended for future research may be listed as follows:

- LRFD conversion,
- Review of ASCE 7-98 new wind map and wind provisions,

- Evaluation of design differences between the 1994 and 2001 *Supports Specifications*,
- Fatigue and vibration studies for noncantilevered sign structures,
- Mitigation techniques for vibration and oscillation,
- Variable message signs,
- Bridge-mounted signs,
- Strength of fiber-reinforced composite cross sections,
- Gust effect factors for flexible structures,
- Bending about the diagonal of square and rectangular steel tubes, and
- Drag coefficients for multisided sections that are nearly round.

4.3.1 LRFD Conversion

A study to convert the specifications to an LRFD philosophy and format is needed. The LRFD design approach to strength evaluation and to structural safety is more rational and consistent than the current approach in the *Supports Specifications*. Load and resistance factors for support structures will be developed using probability-based studies. The LRFD philosophy will facilitate the incorporation of future engineering developments and test information, since the specifications derive much of their information from national specifications that are based on LRFD.

4.3.2 Review of ASCE 7-98 New Wind Map and Wind Provisions

Revisions to the specifications are needed to incorporate the revised wind map and changes in ASCE-98. The impact of these changes on the design of support structures should also be evaluated.

4.3.3 Evaluation of Design Differences Between the 1994 and 2001 *Supports Specifications*

The impact of the new provisions in the 2001 *Supports Specifications* on the overall design of support structures needs to be evaluated. Complete structures should be designed to determine design differences.

4.3.4 Fatigue and Vibration Studies for Noncantilevered Sign Structures

The design recommendations made for noncantilevered structures should be verified through experimental work and wind tunnel tests. Research is also needed to evaluate the dynamic behavior and susceptibility of vibrations of noncantilevered structures. The impact of the design recommendations from a cost standpoint should be evaluated.

4.3.5 Mitigation Techniques for Vibration and Oscillation

Vibration mitigation techniques and devices for support structures have been studied by a number of researchers. A synthesis of this work is needed. The effect of mitigation techniques on reducing fatigue loads should be evaluated from the standpoint of economy and safety.

4.3.6 Variable Message Signs

Additional information needs to be introduced in the specifications regarding VMSs. These structures are heavier and much larger than conventional signs, and, thus, special considerations may be required to ensure proper design of the supporting structure.

4.3.7 Bridge-Mounted Signs

Sometimes, large signs are attached to bridge structures in lieu of new structures being constructed. These large signs are difficult to fabricate and construct, as well as significantly higher in cost. Potential problems and safety hazards may arise if such structures are used without adequate attention to (a) the design of the connection and (b) the loads and vibrations transferred to both the bridge and the sign support structure.

4.3.8 Strength of Fiber-Reinforced Composite Cross Sections

No design specifications or codes are currently available that cover the design requirements for FRP structures except the 2001 *Supports Specifications*. However, the equations in the specifications have not been verified experimentally. An experimental program is needed to evaluate the strength properties of cross-sectional shapes typically used for support structures.

4.3.9 Gust Effect Factors for Flexible Structures

Gust effect factors for wind-sensitive and non–wind-sensitive structures were provided as part of this project. These values should be verified by physical wind tunnel testing on typical support structures. The effect of the proposed gust factor equations for flexible structures on the design of support structures should also be assessed.

4.3.10 Bending About the Diagonal of Square and Rectangular Steel Tubes

An analytical method was developed for the design of square and rectangular steel sections when subjected to bending about a skewed axis. Full-scale testing is needed to verify the accuracy of the method and to further refine the analytical model.

4.3.11 Drag Coefficients for Multisided Sections that Are Nearly Round

An experimental program is needed to verify the transition equation that was developed in this project to calculate the drag coefficients for multisided sections as their geometry approaches a round section. The experimental work should involve wind tunnel testing.

REFERENCES

1. Fouad, Fouad H., Elizabeth A. Calvert, and Edgar Nunez, *NCHRP Report 411: Structural Supports for Highway Signs, Luminaires, and Traffic Signals.* Transportation Research Board of the National Academies, Washington, D.C. (1998).
2. AASHTO, *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*. Third Edition, American Association of State Highway and Transportation Officials, Washington, D.C. (1994).
3. AASHTO, *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, Third Draft*. American Association of State Highway and Transportation Officials, Washington, D.C. (March 1999).
4. AASHTO, *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*. Fourth Edition, American Association of State Highway and Transportation Officials, Washington, D.C. (2001).
5. ASCE, *Minimum Design Loads for Buildings and Other Structures*, ANSI/ASCE 7-95, American Society of Civil Engineers, Reston, Virginia (1996).
6. Kaczinski, M. R., R. J. Dexter, and J. P. Van Dien, *NCHRP Report 412: Fatigue Resistant Design of Cantilevered Signal, Sign and Light Supports.* Transportation Research Board of the National Academies, Washington, D.C. (1998).
7. AASHTO, *AASHTO LRFD Bridge Design Specifications*, First Edition, American Association of State Highway and Transportation Officials, Washington, D.C. (1994).
8. Gilani, A., and A. Whittaker, "Fatigue-Life Evaluation of Steel Post Structures I: Background and Analysis." *Journal of Structural Engineering*, American Society of Civil Engineers, New York, New York (2000).
9. Gilani, A., and A. Whittaker, "Fatigue-Life Evaluation of Steel Post Structures I: Experimentation." *Journal of Structural Engineering*, American Society of Civil Engineers, New York, New York (2000).

9a. Cook, R. A., D. Bloomquist, M. A. Kalajian, V. A. Cannon, and D. Arnold, *Mechanical Damping Systems for Traffic Signal Mast Arms*, University of Florida, Gainesville, Florida, October 1998.

9b. Hamilton, H. R., J. A. Puckett, B. Gray, and P. Wang, *Traffic Signal Pole Research*, Interim Report to Wyoming Department of Transportation, University of Wyoming, Laramie, Wyoming, January 2000.

10. Fouad, Fouad H., Elizabeth A. Calvert, and Edgar Nunez, *Structural Supports for Highway Signs, Luminaires, and Traffic Signals: Draft Final Report*. NCHRP Project 17-10, Transportation Research Board of the National Academies (September 1997).
11. Fouad, Fouad H., Elizabeth A. Calvert, and Edgar Nunez, *Structural Supports for Highway Signs, Luminaires, and Traffic Signals: Draft Final Specification*. NCHRP Project 17-10, Transportation Research Board of the National Academies (September 1997).
12. ASCE, *Minimum Design Loads for Buildings and Other Structures*, ANSI/ASCE 7-98, American Society of Civil Engineers, Reston, Virginia (2000).
13. ICBO, *International Building Code,* International Conference of Building Officials, Whittier, California (2000).
14. Cook, R. A., *Strength Design of Anchorage to Concrete*, Portland Cement Association, Stokie, Illinois (1999).
15. ACI, *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-99, American Concrete Institute, Farmington Hills, Michigan (1999).
16. Cook, R. A., *Anchoring to Concrete*. Draft CB-30, American Concrete Institute, Farmington Hills, Michigan (unpublished, 2000).
17. AISC, *Manual of Steel Construction—Allowable Stress Design*, Ninth Edition, American Institute of Steel Construction, Chicago, Illinois (1989).
18. AISC, *Manual of Steel Construction—Load and Resistance Factor Design*, American Institute of Steel Construction, Chicago, Illinois (1998).
19. ACI, *State-of-the-Art Report on Anchorage to Concrete,* ACI 355.1R-91, American Concrete Institute, Farmington Hills, Michigan (1991).
20. ACI, "Code Requirements for Nuclear Safety Related Concrete Structures, Appendix B—Steel Embedments," *ACI Manual of Concrete Practice*, ACI 349R-97, American Concrete Institute, Farmington Hills, Michigan (1998).
21. Precast/Prestressed Concrete Institute, *PCI Design Handbook: Precast and Prestressed Concrete*, Chicago, Illinois (1999).
22. AASHTO, *AASHTO LRFD Bridge Design Specifications—2000 Interim Revisions*, American Association of State Highway and Transportation Officials, Washington, D.C. (2000).
23. AASHTO, *AASHTO LRFD Bridge Design Specifications—Interim Revisions*, American Association of State Highway and Transportation Officials, Washington, D.C. (1999).
24. AASHTO, *AASHTO LRFD Bridge Design Specifications*, Second Edition, American Association of State Highway and Transportation Officials, Washington, D.C. (1998).
25. AASHTO, *Standard Specifications for Bridge Design*, 16th Edition, American Association of State Highway and Transportation Officials, Washington, D.C. (1996).
26. Reese, L. C., *Handbook on Design of Piles and Drilled Shafts Under Lateral Load*, Report FHWA-IP-84-11, Federal Highway Administration, Washington, D.C. (1984).
27. Bowles, J. E., *Foundation Analysis and Design*, Fifth Edition, McGraw-Hill, New York, New York (1996).
28. Hubbell/Chance, *Chance Encyclopedia of Anchors*, Centralia, Missouri (1998).
29. Alaska State Department of Transportation, *Standard Drawings Manual* (2001).
30. Arizona State Department of Transportation, *Bridge Group Structure Detail Drawings—Traffic Structures* (2001).
31. Indiana State Department of Transportation, *Standard Specification Book* (1999).
32. Pennsylvania State Department of Transportation, *Publication 218 M—Bridge Design Manual—M21-01, Section BD643M—Sign Structures* (July 2001).
33. Washington State Department of Transportation, *Bridge and Sign Structure Design Manual—M21-01 English* (July 1998).

34. West Virginia State Department of Transportation, *Standard Specification Roads and Bridges* (2000).
35. Iowa State Department of Transportation, *Design Manuals* (2002).
36. Missouri State Department of Transportation, *Standards and Specifications for Highway Construction—Section 900* (2002).
37. California State Department of Transportation, “Signals, Lighting and Electrical Systems,” *Standard Specifications*, Section 86 (March 29, 2000).
38. Connecticut State Department of Transportation, *Form 815 Standard Specification for Roads, Bridges and Incidental Construction* (2000).
39. South Dakota State Department of Transportation, *Standard Specifications* (2002).
40. Ohio State Department of Transportation, *Standard Traffic Engineering Drawings* (2002).
41. Alabama State Department of Transportation, *Structures Design and Detailing Manual* (2001).
42. ASTM A6-00a, “Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling,” *Annual Book of ASTM Standards*, American Society for Testing and Materials, West Conshohocken, Pennsylvania (2001).
43. California State Department of Transportation, “Signals, Lighting and Electrical Systems,” *Standard Specifications and these Special Provisions*, Section 56 (March 29, 2000).
44. Texas Department of Transportation, “High Mast Illumination Poles” and “Traffic Signal Pole Assemblies (Steel),” *Standard Specifications*, Items 613 and 686 (1993).
45. Iowa Department of Transportation, *Steel Structures Specification* (April 27, 1999).
46. New Mexico State Highway and Transportation Department, *Standard Specifications for Highway and Bridge Construction* (1996).
47. Tebedge, N., and W. F. Chen, “Design Criteria for H-Columns Under Biaxial Loading.” *Journal of the Structural Division*, ASCE, Vol. 100, No. ST3 (March 1974).
48. Galambos, T. V., ed., *Guide to Stability Design Criteria for Metal Structures*, Structural Stability Research Council, Fifth Edition, John Wiley and Sons, New York (1998).
49. Sherman, D. R., “Tubular Members,” *Constructional Steel Design: An International Guide*, Elsevier Science Publishers, Ltd., Essex, England (1992).
50. Unpublished test results, Valmont Industries, Inc.
51. ASTM D4923-01, “Standard Specification for Reinforced Thermosetting Plastic Poles,” *Annual Book of ASTM Standards*, American Society for Testing and Materials, West Conshohocken, Pennsylvania (2001).
52. Clarke, J. L., ed., *Structural Design of Polymer Composites—Eurocomp Design Code and Handbook*, E&F Spon, London (1996).
53. “Design of Lighting Columns,” *Design Manual for Roads and Bridges*, British Standard BD26/94, British Standard Institute, London, United Kingdom (September 1994).
54. ANSI C136.20, *Fiber-Reinforced Plastic (FRP) Lighting Poles*, American National Standards Institute, Washington, D.C. (1996).
55. California Department of Transportation, *Evaluation of Deflection and Bending Characteristics of Fiber-Reinforced Plastic Lighting Standards*, Caltrans Study #F92TL12, Sacramento, California (November 1995).
56. City of Seattle, *City of Seattle Standard Specification 8-32: Poles, Pedestals and Foundations*, Seattle, Washington (2001).
57. Carolina Power and Light Company, *Group Specification GS-32 for Fiberglass Post*, Raleigh, North Carolina (February 1993).
58. Illinois Power Company, *Material Specifications for Fiberglass Street Lighting Poles*, Decatur, Illinois (1988).
59. Newmark International, Inc., *Specification for Fiberglass Reinforced Composite Poles*, Birmingham, Alabama (1998).
60. Shakespeare Company, *Shakespeare Composite Utility Pole Specifications*, Newberry, South Carolina (2001).
61. Southern California Edison Company, *Fiberglass Reinforced Plastic Street Lighting Electroliers*, Specification MS32-1995, Rosemead, California (1995).
62. Southern Company, *Fiberglass Lighting Pole Specifications*, SES-PD-087, Atlanta, Georgia (June 1994).
63. Hawaii Department of Transportation, *Special Provisions, Specifications, Proposal and Contract for Furnishing and Delivering Street Light and Traffic Signal Poles, Arms, Bases, Luminaires and Cable*, Project HWY-C-06-98, Honolulu, Hawaii (1998).
64. Pacific Gas and Electric Company, *Specification for Non-Wood Distribution Poles*, PG&E Specification No. 60, San Francisco, California (January 1995).
65. Public Service Company of New Mexico, *PNM Specification SL2 Revision 3: Aluminum/Fiberglass Lighting Poles*, Albuquerque, New Mexico (August 1997).
66. ASTM G154-00a, “Standard Practice for Operating Fluorescent Light Apparatus for UV Exposure of Non-Metallic Materials,” *Annual Book of ASTM Standards*, American Society for Testing and Materials, West Conshohocken, Pennsylvania (2001).
67. ASTM D635-98, “Standard Test Method for Rate of Burning and/or Extent and Time of Burning of Plastics in a Horizontal Position,” *Annual Book of ASTM Standards*, American Society for Testing and Materials, West Conshohocken, Pennsylvania (2001).
68. ASTM G53-93, “Standard Practice for Operating Light and Water Exposure Apparatus (Fluorescent UV-Condensation Type) for Exposure of Non-Metallic Materials,” *Annual Book of ASTM Standards*, American Society for Testing and Materials, West Conshohocken, Pennsylvania (1997).
69. Fisher, J. W., H. Hausmann, M. D. Sullivan, and A. W. Pense, *NCHRP Report 206: Detection and Repair of Fatigue Damage in Welded Highway Bridges*, Transportation Research Board of the National Academies, Washington, D.C. (1979).
70. Nowak, A. S., “Calibration of LRFD Bridge Code,” *Journal of Structural Engineering*, Vol. 121, No. 8 (August 1995).
71. Nowak, A. S., *NCHRP Report 368: Calibration of LRFD Bridge Design Code*, Transportation Research Board of the National Academies (1999).
72. Plantema, F. J., “Collapsing Stresses of Circular Cylinders and Round Tubes,” *Report S*. 280, Nat. Luchtvaartlabatiorium, Amsterdam, the Netherlands (1946).
73. Schilling, C. G., “Buckling Strength of Circular Tubes,” *Journal of the Structural Division*, Vol. 91, No. ST5, Proc. Paper 4520, American Society of Civil Engineers, Reston, Virginia (October 1965).

APPENDIXES

The appendixes submitted by the research agency are not published herein. However, for a limited time, the following appendixes are available upon request from the NCHRP:

- Appendix A: Project Surveys and Results
- Appendix B: Wind Loads Report
- Appendix C: Fatigue-Resistant Design of Noncantilevered Sign Support Structures
- Appendix D: Sample Calculation for Proposed Drag Coefficient Transition Equation for Tapered Multisided Sections
- Appendix E: Proposed Combined Stress Ratio Equation for Square and Rectangular Tubes Bent About a Diagonal Axis
- Appendix F: Proposed Performance Specification for FRP Poles
- Appendix G: Retrofitting and Rehabilitating Fatigue-Damaged Support Structures

Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation