Report No. K-TRAN: KU-98-6
FINAL REPORT

DEVELOPMENT OF DESIGN SPECIFICATIONS, DETAILS AND DESIGN CRITERIA FOR TRAFFIC LIGHT POLES

Mario M. Medina, Ph.D., P.E.

The University of Kansas
Lawrence, Kansas

SEPTEMBER 2006

K-TRAN

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:
KANSAS DEPARTMENT OF TRANSPORTATION
KANSAS STATE UNIVERSITY
THE UNIVERSITY OF KANSAS

1 Report No. K-TRAN: KU-98-6	2 Government Accession No.	3 Recipient Catalog No.
4 Title and Subtitle DEVELOPMENT OF DESIGN SPECIFICATIONS, DETAILS AND DESIGN CRITERIA FOR TRAFFIC LIGHT POLES		5 Report Date September 2006 6 Performing Organization Code
7 Author(s) Mario A. Medina		8 Performing Organization Report No.
9 Performing Organization Name and Address University of Kansas Civil, Environmental & Architectural Engineering Department 1530 West 15th Street, Room 2134 Lawrence, Kansas 66045-7609		10 Work Unit No. (TRAIS) 11 Contract or Grant No. C1035
12 Sponsoring Agency Name and Address Kansas Department of Transportation Bureau of Materials and Research 700 SW Harrison Street Topeka, Kansas 66603-3754		13 Type of Report and Period Covered Final Report August 1998 – August 2006 14 Sponsoring Agency Code RE-0156-01
15 Supplementary Notes For more information write to address in block 9.		

16 Abstract

Current rules and fabrication methods employed in the design of traffic light poles do not adequately address fatigue and fracture issues associated with the connection of mast arms to the vertical poles and the connection of the poles to the foundations of structures. The purpose of this work was to collect existing data on this issue and to develop new design specifications guidelines based on the findings. The new AASHTO Specification was critically reviewed, new design criteria for traffic light poles design were suggested, typical design drawings and details were prepared, and calculation procedures were outlined. All this is presented in the report, together with sample calculations and recommendations from manufacturers who reviewed this work.

17 Key Words Design, Fatigue, Fracture, Light Poles, Specifications, Standards and Structures		18 Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19 Security Classification (of this report) Unclassified	20 Security Classification (of this page) Unclassified	21 No. of pages 60	22 Price

Form DOT F 1700.7 (8-72)

DEVELOPMENT OF DESIGN SPECIFICATIONS, DETAILS AND DESIGN CRITERIA FOR TRAFFIC LIGHT POLES

Final Report

Prepared by

Mario A. Medina, Ph.D., P.E.

A Report on Research Sponsored By

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

and

UNIVERSITY OF KANSAS CENTER FOR RESEARCH, INC.
LAWRENCE, KANSAS

September 2006

PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

NOTICE

The authors and the state of Kansas do not endorse products or manufacturers. Trade and manufacturers names appear herein solely because they are considered essential to the object of this report.

This information is available in alternative accessible formats. To obtain an alternative format, contact the Office of Transportation Information, Kansas Department of Transportation, 700 SW Harrison, Topeka, Kansas 66603-3754 or phone (785) 296-3585 (Voice) (TDD).

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the views or the policies of the state of Kansas. This report does not constitute a standard, specification or regulation.

ABSTRACT

Current rules and fabrication methods employed in the design of traffic light poles do not adequately address fatigue and fracture issues associated with the connection of mast arms to the vertical poles and the connection of the poles to the foundations of structures. The purpose of this work was to collect existing data on this issue and to develop new design specifications guidelines based on the findings. The new AASHTO Specification was critically reviewed, new design criteria for traffic light poles design were suggested, typical design drawings and details were prepared, and calculation procedures were outlined. All this is presented in the present report, together with sample calculations and recommendations from manufacturers who reviewed this work

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Chapter 1

Introduction

1.1 Problem Statement

In recent years there have been a number of failures of traffic light poles and other signage, indicating that the existing design rules and fabrication methods are not adequate in addressing fatigue and fracture issues associated with the connection of mast arms to the vertical poles and the connection of the poles to the foundations of traffic light poles.

1.2 Purpose of Present Work

The purpose of this work was to collect the existing data from studies conducted at KU, KDOT, and elsewhere as well as to develop new design specifications based on these data.

1.2.1 Specific Tasks

The following tasks were proposed and performed. Their statuses are reported here:

- The new 2001 AASHTO Specification was critically reviewed. A summary of important points from this standard is presented in Section 4. (AASHTO, 2001)
- Suggested design criteria for traffic light poles were developed for KDOT use. These are provided in Section 5.
- A review of the current state of the art on traffic light pole design was conducted and current design and material specifications now used by KDOT. This is presented in Section 6.
- In Section 7, typical design drawings and details that were developed based on the surveyed literature and on calculations are presented

- Calculation procedures were developed to aid in the design of traffic light poles based on the foregoing information. This information is presented in Section 8.
- Sample design calculations are presented. Conclusion and recommendations are provided in Section 10.
- Conversations and meetings were held with manufacturers. The manufacturers reviewed this draft report and their suggestions are included in Section 9.

Chapter 2

Review of the 2001 AASHTO Specifications and Summary of Important Points

2.1 Historical Perspective

Traffic light, sign, and signal support structures are designed using the AASHTO Specifications. The 2001 AASHTO Specifications (4^{th} Ed.) represent the latest version of the specifications; the previous one is the 1994 version. A more recent 2003 *Interim* version, which features revisions to the 2001 version, was published recently.

Because the 1994 AASHTO Specifications were unclear when dealing with issues related to vibration and fatigue failure of support structures, the National Cooperative Highway Research Program (NCHRP) funded Project 10-38, which was conducted at the University of Minnesota (NCHRP 10-38). Project 10-38 was completed in January 2001. The recommendations and commentaries from this Project were incorporated into the revised 2001 Specifications, specifically vibration and fatigue provisions.

Because only a couple of years have passed since the publication of the 2001 AASHTO Specifications only a few agencies have used this document as guideline, and thus it is too early to assess its acceptance by the industry. A survey of the few agencies and manufacturers, which have used the new version of the specification, indicated that the proposed revisions, although complex, are clear and practical. In addition, the survey indicated that in some cases the new provisions “force” designers to increase structural element sizes and add more fatigue-resistant design details, which increases the costs of the structures. Estimates of percent cost increase places the price hikes at no more than 20 percent.

The bulk of revisions to the 1994 version of the AASHTO Specifications were based on two reports: NCHRP Report 411 (NCHRP Project 17-10) and NCHRP Report 412 (NCHRP Project 10-38). NCHRP Report 411, "*Structural Supports for Highway Signs, Luminaires, and Traffic Signals,"* provided detailed information on the development of wind loading criteria by using new isotach maps, revised allowable bending stresses for steel, deflection limitations, the analysis of second order effects, fatigue and vibration provisions, anchor bolt requirements, span wire design philosophies and added new sections on composites and wood support structures. The report identified proposed changes to the specifications and provided an assessment of the impact of the change. The recommended specification reflected state-of-the-art design philosophies and manufacturing processes and included guidance for poles fabricated from steel, aluminum, prestressed concrete, timber, and fiber-reinforced plastic composites. NCHRP Report 412, *"Fatigue-Resistant Design of Cantilevered Signal, Sign and Light Supports,"* recommended specifications and commentary for the fatigue-resistant design of cantilevered signal, sign, and light supports. The objective of this project was to develop rational design procedures and recommended specifications for cantilevered signal, sign, and light support structures that consider wind-induced cyclic stresses.

The 2001 AASHTO *Standard Specifications for Structural Support for Highway Signs, Luminaires, and Traffic Signals,* includes many of the recommended provisions developed in NCHRP Project 17-10 and NCHRP Project 10-38. These are summarized below.

2.2 Wind Loads

An examination of the 1994 version suggested that wind loading criteria were based primarily on information and procedures dating back to the 1960's and 1970's. The 2001 Specifications were updated to include wind-load provisions based on current wind engineering practices and introduced the use of ASCE 7-95 as standard, with modifications for structural supports for highway signs, luminaries, and traffic signals (ASCE/ANSI 7-95, 1996). The new wind-load provisions were based on *a 3-second gust wind speeds* (rather than the previously used *fastest-mile wind speed*). As a result, a new wind map was developed and adopted, which represented the major change in the 2001 Specifications. The use of this map would yield changes in calculated wind pressures. These changes, however, are highly site-specific and are also dependent on wind elevation and structure type.

2.2.1 Design Procedure Using the 2001 Specifications (Wind Loads)

- Determine the standard wind velocity for the appropriate region using the wind speed map in Figure 3-2 "Basic Wind Speed, m/s (mph)"
- Use Table 3-3 "Recommended Minimum Design Life" to find the recommended design life based on the type of structure and its height.
- Use this design life value as a recurrence interval in Table 3-2 "Wind Importance Factors, I_r" to find the importance factor.
- Calculate K_z using either Table 3-5 "Height and Exposure Factors, K_z" or Section 3.8.4 "Height and Exposure Factors, K_z" (Note: If the total height of the structure is less than 15 feet, set $z = 15$).
- Choose the appropriate drag coefficient from Table 3-6 "Wind Drag Coefficient, C_d" based on the type and shape of the structure.
- Choose the appropriate gust effect factor, G. The recommended value for this factor is usually 1.14.
- Calculate the wind pressure using Equation 3.1

$$P_z = 0.00256 K_z G V^2 I_r C_d$$ (Equation 3.1 of the AASHTO Specifications, 2001)

Where:

P_z	=	*Pressure (lb/ft^2)*
K_z	=	*Height and Exposure Factor*
G	=	*Gust Effect Factor*
V	=	*Wind Speed (mph)*
I_r	=	*Importance Factor*
C_d	=	*Drag Coefficient*

- Multiply each pressure by the appropriate surface area of the section.
- Apply each load horizontally to the respective section.
- Sum all horizontal wind forces to find the total wind load on the piece of the structure.

2.3 Foundations

In relation to foundations, the 2001 AASHTO Specifications provide new information on pile foundation, eccentrically loaded spread footings, and a procedure for calculating embedment depth of laterally loaded drilled shafts and direct embedded poles. A survey of various DOTs indicated that reinforced cast-in-place drilled shafts are the most commonly used (>67%) type of foundation for traffic light supports. None of the surveyed DOTs used unreinforced cast-in-place drilled shafts or steel screw-in foundations. Spread footings and directly embedded poles were rarely used (<33%).

Factors, such as support structure type, stiffness, transmitted loads, soil properties and soil-structure interactions, groundwater conditions, and depth of bedrock need to be taken into account when selecting an appropriate type of foundation. If the failure of the structure would not pose significant hazard, the 2001 Specifications is flexible in regards to subsurface exploration.

To determine the embedment depth of laterally loaded drilled shafts, the 2001 Specifications suggest the use of Brom's method in both cohesive and cohesionless soils.

For cohesive soils, the embedment length, *L*, is found from:

$$L = 1.5D + q\left[1 + \sqrt{2 + \frac{(4H + 6D)}{q}}\right] \qquad \text{(Equation 2.2)}$$

Where:

$$H = \frac{M_F}{V_F}$$

and

$$q = \frac{V_F}{9cD}$$

in the above equations

c = shear strength of cohesive soil (k/ft^2)

D = with or diameter of foundation (ft)

q = coefficient (ft)

M_F = applied moment at groundline including an appropriate safety factor (k-ft)

V_F = applied shear load at groundline including an appropriate safety factor (k)

For cohesionless soils, the embedment length, L, is found from (using trial and error)

$$L^3 - \frac{2V_F L}{K_p \gamma D} - \frac{2M_F}{K_p \gamma D} = 0 \qquad \text{(Equation 2.3)}$$

Where

$$K_p = tan^2\left(45 + \frac{\phi}{2}\right)$$

in the above equations

ϕ = *angle of internal friction (deg)*

γ = *effective unit weight of soil (k/ft^3)*

2.4 Wind Drag Coefficients

Only minor changes in this section were included in the 2001 Specifications. The changes are related to the use of SI Units and the use of the 3-second gust wind velocity. Table 3-6 "Wind Drag Coefficients, C_d" provides a comprehensive list of coefficients. For traffic signals, the recommended coefficient is 1.2. A footnote to this coefficient reads: *"Wind loads on free swinging traffic signals may be modified, as agreed by the owner of the structure, based on experimental data."*

2.5 The Use of Fiber-Reinforced Composites

Section 8 "Fiber-Reinforced Composite Design" was included in the 2001 Specifications. At this time, this section focuses only on fiberglass-reinforced plastic (FRP) composites. Section 8 provides information on this composite including mechanical properties, manufacturing, design, and testing.

2.6 Fatigue Design

A Section on *Fatigue Design* (Section 11, "Fatigue Design") was added to the 2001 Specifications. In contrast with the 1994 version, which only considered vortex shedding, the 2001 version states the following: "*In general, overhead cantilevered sign and traffic signal* *structures should be designed for fatigue due to individual loadings from galloping, natural wind gusts, and truck-induced wind gusts…Vortex shedding should be considered for single-member cantilevered members that have tapers less than 0.14 in/ft, such as lighting structures or mast arms without attachments."* A summary of highlights follows:

2.6.1 Simplification of Dynamic Fatigue Loads

In the 2001 Specifications, the dynamic fatigue loads produced by vortex shedding, galloping, natural wind gusts, and truck induced wind gusts were simplified by using equivalent static loads, which create similar stress responses. As such, designers need not conduct complex dynamic analyses, but rather simple static analyses.

2.6.2 Fatigue Importance Factors

A fatigue importance factor, I_F, accounts for the degree of hazard to traffic and damage to property. These factors are used to adjust the magnitude of the fatigue pressures. Fatigue importance factors are given in Table 11-1 "Fatigue Importance Factors, I_F," on the 2001 Specifications. The three importance categories, Category I, Category II, and Category III, refer to (I) *critical cantilevered support structures* installed on *major highways,* (II) *other cantilevered support structures* installed on *secondary highways,* and (III) *cantilevered support structures* installed at all *other locations*, respectively. For category I, the importance factors are always 1.0. That is, the fatigue loads are not reduced. This is so because Category I structures are designed to withstand the least frequently occurring wind-induced fatigue loads and also because in the case of failure, such structures would create a greater hazard. The range of factors in Category II is from 0.65 for galloping in sign and traffic signals supports and vortex shedding in lighting poles to 0.89 for truck-induced gusts of sign supports. Category III factors are consistent with the 1994 Specifications in which fatigue provisions were not included. Factors in

Category III range from 0.30 for galloping in traffic signals and vortex shedding in lighting poles to 0.77 for truck-induced gusts in sign supports. It is important to note that factors in Category II are simply the average values between values in Categories I and III.

2.6.3 Fatigue Design Loads

The following is stated in the 2001 Specifications: "To avoid large-amplitude vibrations and to preclude the development of fatigue cracks in various connection details and at other critical locations, cantilevered support structures shall be designed to resist each of the following applicable limit state equivalent wind loads acting separately." The limit state equivalent wind loads that this statement refers to are: galloping, vortex shedding, natural wind gusts, and truck-induced wind gusts.

2.6.3.1 Galloping:

Galloping is an instability typical of flexible, slender structures having certain prismatic cross-sectional shapes. (Simiu, 1978) Other structures predisposed to galloping include lightly damped, flexible structures with non-symmetrical cross-sections. Traffic signal structures fall under this category and are therefore susceptible to galloping oscillations.

The equivalent static load for galloping is given in terms of vertical shear pressure, which is applied on the vertical plane of mast arm attachments, such as signs, signal heads, and signal head backplates. The magnitude of the pressure developed from galloping is in terms of importance factor, I_F, and is defined as:

$$P_G = 21\, I_F \text{ (in psf)} \qquad \text{(Equation 2.4)}$$

According to the 2001 Specifications, galloping loads may be ignored if an approved mitigation device is used. Installing a sign blank, mounted horizontally and directly above the traffic signal attachment closest to the tip of the mast arm, has been shown to be an effective mitigating device for traffic signal support structures with *horizontally mounted* traffic signal attachments. For *vertically mounted* traffic signal attachments, a sign black horizontally mounted near the tip of the mast arm has been proven to mitigate galloping vibration in traffic signals. (McDonald, 1995) The sign blanks measured 16 in. x 66 in. Stock bridge devices have been proven to work in mitigating galloping (Mututwa, 2004). Smaller damping plates did not effectively mitigate oscillations from galloping. Also, damping plates mounted at locations other than directly above the outermost signal attachment were not effective in mitigating this type of vibration.

2.6.3.2 Vortex Shedding

Vortex Shedding is the instance where alternating low-pressure zones are generated on the downwind side of a structural element. These alternating low-pressure zones cause the structural element to move towards the low-pressure zone, causing movement perpendicular to the direction of the wind. When the critical wind speed of the structural element is reached, these forces can cause the element to resonate where large forces and deflections are experienced. The equivalent static pressure range, P_{VS}, to be applied in the direction perpendicular to the wind and to the area projected on the vertical plane, is calculated by:

$$P_{VS} = \frac{0.00118 V_C^2 C_D I_F}{2\beta} \qquad \text{(in psf)} \qquad \text{(Equation 2.5)}$$

Where the critical wind velocity for a prismatic member, V_C, is given by

$$V_C = \frac{f_n d}{S_n} \qquad \text{(for circular sections)}$$

and

$$V_C = \frac{f_n b}{S_n} \qquad \text{(for multisided section)}$$

where

f_n	=	the first natural frequency of the structure (Hz)
d	=	element diameter (m)
b	=	the flat-to-flat width of the member (m),
S_n	=	the Strouhal number
C_D	=	the drag coefficient for the section of interest
I_F	=	fatigue importance factor
β	=	damping ratio (0.005)

According to the 2001 Specifications, vortex-shedding loads may be ignored if an approved mitigation device is used. In regards to vortex shedding mitigating devices, there is significant uncertainty as to what works and what does not. Further testing is needed in this area. It is important to note that according to the 2001 AASHTO support structures that are composed of tapered members do not appear to be prone to vortex-shedding induced vibrations when tapered at least 0.14 in/ft. However, since there are reports of tapered poles that have exhibited vortex shedding, this issue needs further research (Mututwa, 2004).

2.6.3.3 Natural Wind Gusts:

Natural Wind Gusts occur because there is variability in the velocity and direction of wind currents. These fluctuations in flow velocity induce variable pressures on the various structural components, which then cause vibrations on the structure. Natural wind gusts are applied in the direction parallel to the wind flow to the horizontally projected areas of all members, and sign, signals, and traffic lights. The pressure from natural wind gusts is calculated from:

$$P_{NW} = 5.2C_D I_F \quad \text{(in psf)} \qquad \text{(Equation 2.6)}$$

(for mean speeds below 11.2 mph)

or

$$P_{NW} = 5.2C_D\left(\frac{V_m^2}{125}\right)I_F \quad \text{(in psf)}$$

(for locations with more detailed meteorological data)

Where

V_m = yearly mean wind velocity (mph)

The 2001 Specifications does not make mention of natural wind gusts mitigating devices nor was any found in the literature.

2.6.3.4 Truck-Induced Gusts:

Passing trucks beneath cantilevered support structures induce gust loads on the underside and frontal area of the members and the mounted attachments on the mast arm. Truck-induced gust equivalent static pressures are applied to the areas on the undersides of members, signs, signals, and other attachments. At a minimum, these pressures should be applied to the outer 12 ft of the mast arm. This distance is equivalent to the width of one traffic lane. The truck-gust pressures, P_{TG}, are defined as follows:

$$P_{TG} = 18.8C_D I_F \quad \text{(in psf)} \qquad \text{(Equation 2.7)}$$

(for truck speeds of 65 mph)

or

$$P_{TG} = 18.8C_D\left(\frac{V}{65}\right)^2 I_F \quad \text{(in psf)}$$

(for truck speeds less than 65 mph)

where

V = truck velocity (mph)

The 2001 Specifications permit leaving out this load on traffic signal structures at the discretion of the owner.

2.7 Deflection

No deflection limit is stated in the 2001 Specifications. The Specifications only state that mast arm tip deflections should not be excessive for reason of serviceability of the structure. NCHRP Report 412 recommended that the vertical displacement of the tip of the mast arm of traffic signals be limited to 8 in.

2.8 Fatigue Resistance of Connection Details

The 2001 Specifications contain Table 11-2 "Fatigue Details of Cantilevered Support Structures" and Table 11-3 "Constant-Amplitude Fatigue Thresholds". Table 11-2 lists 24 typical cantilevered support structure details, which were taken from standard plans provided by various DOTs. Table 11-3 is to be used in combination with Table 11-2. It lists the constant amplitude fatigue thresholds (CAFT) of steel and aluminum for nine detail categories.

2.9 Design Procedure Using the 2001 Specifications (Fatigue Loads)

- Use Table 11-1 "Fatigue Importance Factors, I_F" to find the appropriate importance factors for each type of loading. (Some or all types of loading may not apply to a particular structure).
- Using Table 3-6[1], "Wind Drag Coefficients, C_d" determine the appropriate wind drag coefficients for each part of the structure.
- Determine the first natural frequency of the structure.
- Determine the dimensions of the structure.
- Determine the Strouhal number (Suggested values are: 0.18 for circular sections, 0.15 for multisided sections, 0.11 for rectangular sections).
- Use Equation 11-2 or 11-3 to calculate the critical wind velocity.

[1] It is assumed that values listed in Table 3.6 are based on 3-second gusts; however, the wind speeds used for natural wind, truck induced gusts, and vortex shedding are based on yearly mean wind speeds.

- If applicable, calculate galloping-induced pressures using Equation 11-1.
- If applicable, calculate vortex-shedding-induced pressures using Equation 11-4. (The damping ration shall be taken as 0.005 unless otherwise specified).
- If applicable, calculate the natural wind gust-induced pressures using Equation 11-5.
- If applicable, calculate the truck-induced gust pressures using Equation 11-6.
- Calculate moments due to galloping (if applicable), vortex-shedding (if applicable), natural wind gusts (if applicable), and truck-induced gusts (if applicable).
- Calculate stress ranges in the anchor rods, pole to base connection (i.e., baseplate socket connection), and mast arm to pole connections (i.e., flange plate, flange plate socket, and built-up box).
- Compare stress ranges to those provided in Table 11-2 "Fatigue Details of Cantilevered Support Structures" and Table 11-3 "Constant Amplitude Fatigue Threshold" to see if the requirements are met.
- Calculate deflection using the method of superposition based on individual displacements by the signals, signs, and mast arm. Compare results against requirements.

Chapter 3

Suggested Design Criteria for Structural Supports for Traffic Signals

3.1 Suggested Criterion #1:

The plans for the proposed structural supports for traffic signals (supports) shall be in conformity with latest versions of pertinent specifications, standards, manuals, and guidelines and shall be specific to the proposed location. The supports must be designed to promote the safety and welfare of the public.

3.2 Suggested Criterion # 2:

The proposed supports shall be cost-effective, durable, and shall minimize post-construction maintenance and repair costs. Designers shall look to take advantage of local materials, construction techniques and labor.

3.3 Suggested Criterion # 3:

The proposed supports shall not, in their design and appearance, be inconsistent with the appearance of other existing structural supports in the neighborhood.

Chapter 4

Review of Current State of the Art on Traffic Light Pole Design

Practices at several DOTs around the country were examined. These include the DOTs of the states of Alabama, Arizona, California, Colorado, Connecticut, Delaware, Georgia, Illinois, Iowa, Kentucky, Maryland, Michigan, Minnesota, Missouri, Montana, Nebraska, New Jersey, Oklahoma, Oregon, Texas, Wisconsin, and Wyoming. In addition, the KANSAS ELECTRONIC STANDARDS INDEX was consulted and the following Standards were reviewed:

	Standard Name	**Revision Dates**	**Description**
1	TE 120A	04/29/2003	Traffic Signal Specifications [Sheet 1 of 4]
2	TE 120B	04/29/2003	Traffic Signal Specifications [Sheet 2 of 4]
3	TE 120C	04/29/2003	Traffic Signal Specifications [Sheet 3 of 4]
4	TE 120D	04/29/2003	Traffic Signal Specifications [Sheet 4 of 4]
5	TE111A	04/29/2003	Mast Arm Pole and Foundation Details

4.1 Typical Design Drawings and Details

Based on these reviews and from calculations the following, *structural-related* specifications, were compiled.

4.1.2 Principal Requirements

- Traffic signal structures must be designed in accordance with AASHTO Standard Specification for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (2001 Edition)

- Traffic signal structures shall be designed to resist without destruction all applied loads as established by the Bureau of Traffic Engineering, including wind and fatigue loads developed by a wind velocity of at least 90 mph in accordance with AASHTO Standard Specifications. Any deflections caused by standard loads and/or wind shall never result in a clearance between the roadway and the lowest point of the signal assembly of less than 15 ft.

4.1.3 Pole and Mast Arm Assembly Materials

Members and components shall meet the requirements of the latest editions of the standards as follows:

- Poles and mast arms
  - ASTM A595 Grade A (55 ksi yield) or B (60 ksi yield) – for round members
  - ASTM A570 or ASTM A572 Grade 55, 60, or 65 – for multi-sided members
- Steel plates
  - ASTM A36 or ASTM A709 Grade 36 or ASTM A572 Grade 50
- Anchor bolts
  - ASTM F1554 Grade 55
- Nuts for anchor bolts
  - ASTM A563 Grade A Heavy Hex

- Washers for anchor bolts
    - ASTM F436 Type I
- Bolts (other than anchor bolts)
    - ASTM A325 Type I
- Nut covers
    - ASTM B26
- Stainless Steel Screws
    - AISI 316
- Caps
    - ASTM A1011 Grade 55, 60, or 65 ksi, or
    - ASTM B209, or
    - Others, such as zinc, aluminum, and ASTM Steel A36
- Threaded Bars and Studs
    - ASTM A36 or ASTM A307

All steel components shall be galvanized as to meet the requirements of the latest editions of the standards as follows:

- All nuts, bolts, washers, and threaded bars and studs
    - ASTM A153 Class C or D (hot dip galvanized)
- Pole and mast arm and other steel accessories/items not included above
    - ASTM A123
- All welding of steel shall conform to the requirements of ANSI/AWS D1.1.

4.1.4 Other Requirements

- All poles and arms shall be tapered with the diameter changing at the rate of 0.14 in./ft and be made only of one length of structural steel sheet of not less than No. 7 Manufacturing Standard Gauge.

- Mast arms 38-ft in length or greater may have arm extensions of structural steel sheet of not less than No. 11 Manufacturing Standard Gauge, with bolted telescopic field joints so as to develop full strength of the adjacent shaft sections to resist bending.
- Mast arm camber angle shall be about 2-3 degrees with respect to the horizontal.

A typical single mast-arm and pole assembly is shown in Figure 4.1.

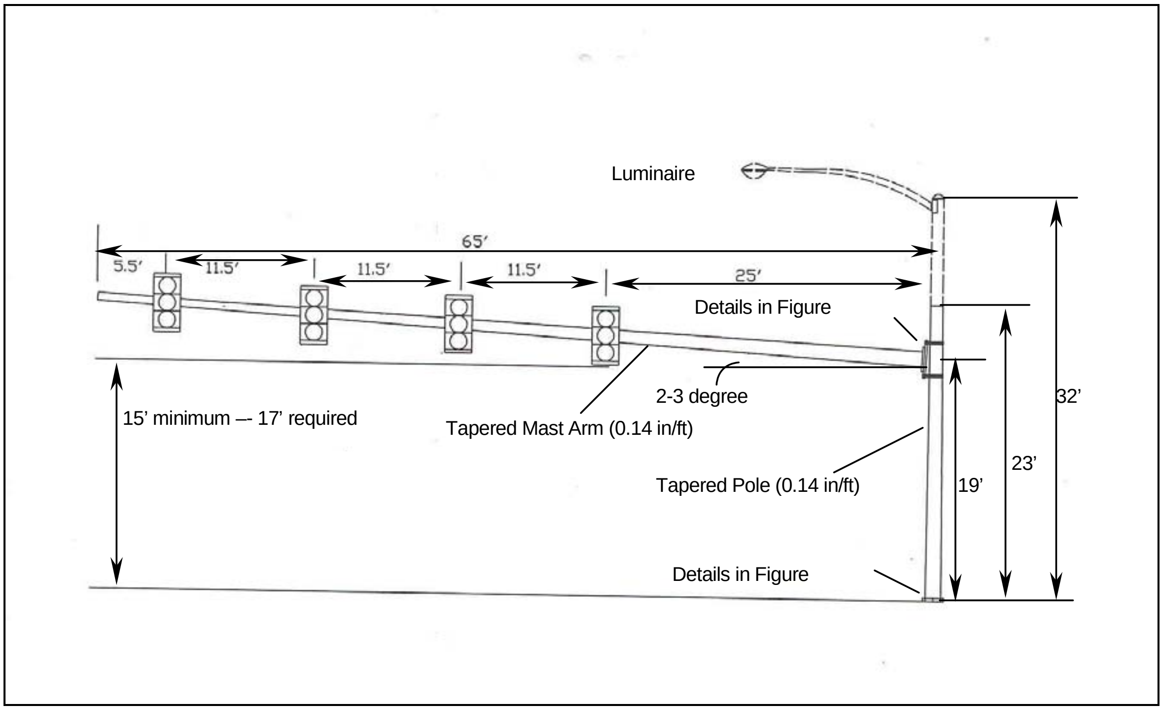

Figure 4.1: Typical Single Mast Arm and Pole Assembly

(Dimensions are typical, which will vary depending on structure size)

The recommended dimensions are shown in Table 4.1 for poles and Table 4.2 for mast arms.

Table 4.1: Recommended Dimensions for Pole Diameters based on Mast Arm Lengths for 90 mph Winds

Mast Arm Length	Poles			
	D_{base}	$D_{at\ MA\ connection}$	Thickness	Base Plate
(ft)	(in)	(in)	(in)	(in)
20	11.0	8.3	0.239	18 x 18
24	11.0	8.3	0.239	18 x 18
28	11.5	8.8	0.239	18 x 18
32	12.5	9.8	0.239	18 x 18
36	13.0	10.3	0.239	18 x 18
40	13.0	10.3	0.239	18 x 18
For mast arm lengths of over 40 ft consult manufacturer				

Table 4.2: Recommended Dimensions for Mast Arm Diameters based on Mast Arm Lengths for 90 mph Winds

Mast Arm Length	Mast Arm		
	D_{base}	D_{tip}	Thickness
(ft)	(in)	(in)	(in)
20	7.5	4.7	0.179
24	8.5	5.1	0.179
28	9.0	5.1	0.179
32	9.0	4.5	0.239
36	9.5	4.5	0.239
40	10.0	4.4	0.239
44	10.5	4.3	0.239
48	11.0	4.3	0.239
For mast arm lengths of over 48 ft consult manufacturer			

4.2 Mast Arm to Pole Connection Details

After reviewing current designs of mast-arm to pole connection details it became evident that this is the critical area where most fatigue related failures occur. The 2001 Specifications suggested several ways of improving this detail. These included full penetration connections together with significant increases in flange plate sizes, bolt diameters, mast-arm diameters, and mast-arm thicknesses. After performing calculations, samples of which are found in Section 9, and based on literature of past projects available at the University of Kansas, one simpler way to meet this connections requirement is by employing the “saddle” type support bracket shown in

Figures 4.2 thru 4.5. A similar support is provided in the 2001 Specifications' 2002 Interim Report in Figure 11-1 and is termed "Ring-Stiffened Built-up Box."

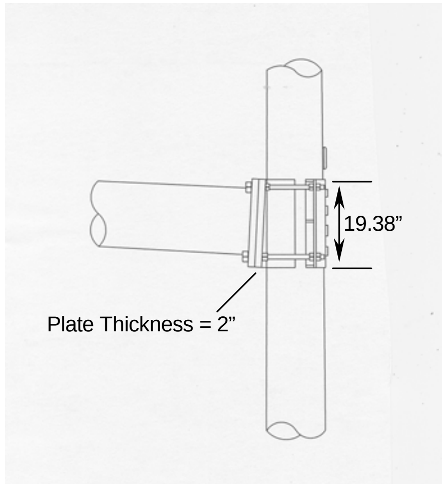

Figure 4.2: "Saddle" Type Support Bracket

(Dimensions shown are typical. These would vary depending on structure size)

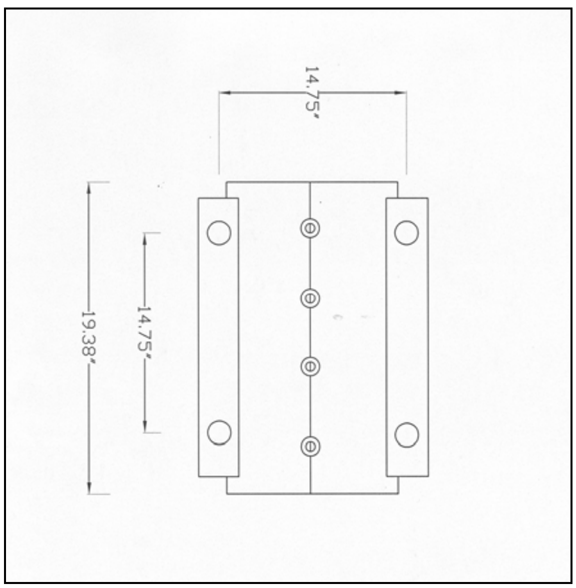

Figure 4.3: Back of Mast Arm Base

(Dimensions shown are typical. These would vary depending on structure size)

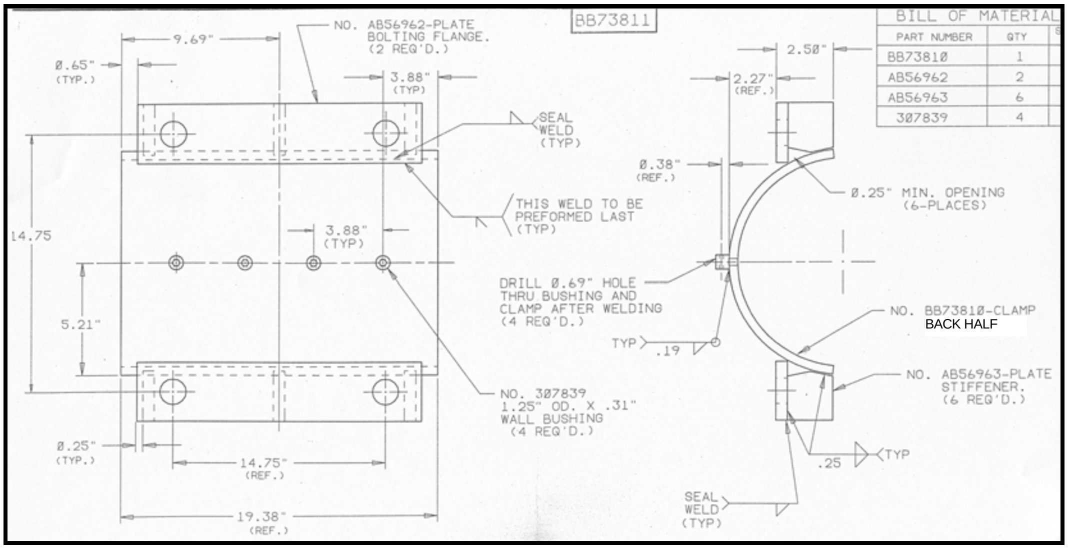

Figure 4.4: Details for "Saddle" Type Support Bracket (Source: Valmont)

(Dimensions shown are typical. These would vary depending on structure size)

Figure 4.5: Pictures of "Saddle" Type Support Bracket

4.3 Anchor Rod and Pole to Baseplate Connection Details

According to the 2001 Specification, the stress category for anchor bolts in tension based on the tensile stress area is D, which for steel has a Constant Amplitude Fatigue Threshold (CAFT) of 7 ksi. Our calculations suggested that current designs would meet this requirement provided that the anchor rod diameter did not fall below 1.5 in. Furthermore, an anchor bolt diameter of 1.75 in. is recommended based on the calculations. No other revisions were made to the existing foundation details, which is shown in Figure 4.6.

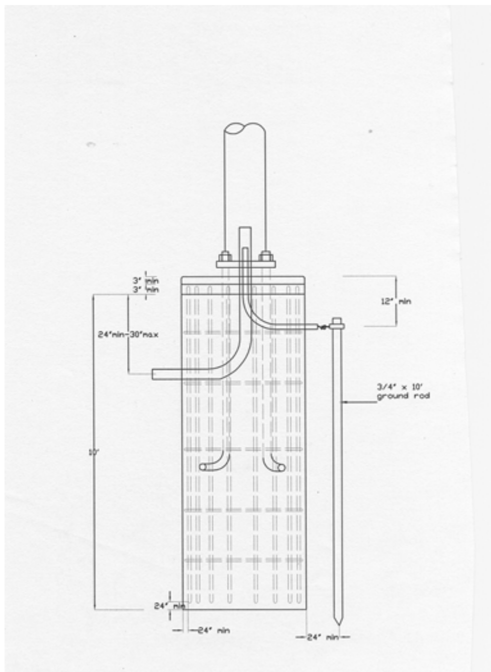

Figure 4.6: KDOT's Foundation Detail

Observation and calculations regarding the pole to baseplate socket connection indicated that under current specifications, the socket connection is classified as a category E', which has a fatigue limit of 2.6 ksi. In some cases the current designs do not meet requirements related to galloping. Based on calculations, the potential redesigns include increasing the pole diameter, improving the detail category to E (4.25 ksi), which is for full penetration connections, and increasing the pole thickness. Because of the practicality and the economics involved, the simplest and recommended redesign provided by the specifications is to improve the stress category to C (10 ksi) by using stiffeners (Figure 4.7). However, in practice this is not a recommended feature because the process of welding a gusset to a pole requires taking material from the weld and grinding, which introduces microcracks and residual stresses.

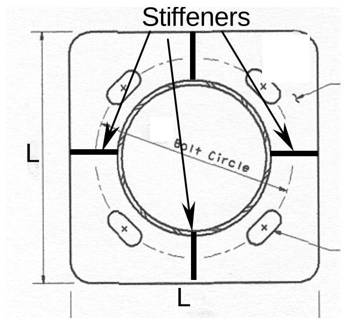

Figure 4.7: Base Plate Depicting the Location of the Proposed Stiffeners

Chapter 5

Calculation Procedures

Simplified procedures for calculating stress ranges in anchor rods, base plate socket connection, mast-arm to pole connection flange plate bolts, mast-arm to pole plate socket connection, mast-arm to pole built-up box, and for deflection are presented in this section. Sample calculations from a traffic signal structure located in Lawrence, Kansas are presented in Chapter 6.

5.1 Anchor Rod Stress Range Calculation Procedure

The stress range acting in an anchor rod is determined by:

$$S_{Range,AR(Z)} = \frac{M_{Z,Gal}(\bar{x})}{I_{AR,Z}} \qquad \text{(Equation 5.1)}$$

$$S_{Range,AR(X)} = \frac{M_{X,Nat-Wind}(\bar{z})}{I_{AR,X}} \qquad \text{(Equation 5.2)}$$

Where

$M_{Z,Gal}$ is the moment due to galloping, about the z-axis, and $M_{X,Nat\text{-}Wind}$ is the moment due to natural winds, about the x-axis. These moments are calculated by:

$$M_{Z,Gal} = \sum F_{G,i} x_i \qquad \text{(Equation 5.3)}$$

$$M_{X,Nat-Wind} = F_{NW,pole}\left(\frac{L_1}{2}\right) + \sum F_{NW,Signal,i}\, y_{Signal,i} + \sum F_{NW,MA-Section,i}\, y_{MA-Section,i} + \sum F_{NW,Sign,i}\, y_{Sign,i}$$

(Equation 5.4)

where $F_{G,i}$ is the magnitude of the equivalent static vertical shear force acting upon each sign panel and traffic signal heads and backplates rigidly mounted to the cantilevered horizontal support. F_{NW} represents the static load ranges from natural winds to be applied to the pole, mast-arm sections, signal and sign attachments. $\bar{x}$ and x_i represent the x-distance from the center of the base plate to the anchor bolts and the horizontal distance from the base of the mast-arm to the horizontal center of each of the signal panels and traffic signal heads, respectively. The y distances are vertical distances from the roadway to the vertical center of each sign, signal, or mast-arm pole. The moment of inertia of the anchor rod group about the z-axis, $I_{AR,Z}$ is calculated by:

$$I_{AR,Z} = \sum A_{T,AR} \bar{x}^2 \qquad \text{(Equation 5.5)}$$

where

$A_{T,AR}$ is the tensile stress area of the anchor rods, and it is calculated by:

$$A_{T,AR} = \frac{\pi}{4}\left[D_{AR} - \frac{0.9743}{n}\right]^2 \qquad \text{(Equation 5.6)}$$

where D_{AR} is the nominal anchor rod diameter and n is the thread series (threads/inch). The moment of inertia of the anchor rod group about the x-axis, $I_{AR,X}$ is calculated by:

$$I_{AR,X} = \sum A_{T,AR} \bar{z}^2 \qquad \text{(Equation 5.7)}$$

where $\bar{z}$ is the distance in the z-direction from the center of the base plate to the furthest anchor rod.

5.2 Pole Stress Range at Pole-to-Baseplate Connection Calculation Procedure

The stress range of the pole at the pole-to-baseplate connection is calculated by:

$$S_{Range,Pole} = \frac{M_{Z,Gal}\left(\frac{D_{base,pole}}{2}\right)}{I_{pole}} \qquad \text{(Equation 5.8)}$$

where the moment of inertia of the pole is calculated by:

$$I_{pole} = \frac{\pi}{64}\left[D_{Base,pole}^{4} - \left(D_{Base,pole} - 2t\right)^{4}\right] \qquad \text{(Equation 5.9)}$$

where t is the thickness of the pole.

5.3 Flange Plate Bolt Stress Range at Pole-to-Mast-Arm Connection Calculation Procedure

The stress range acting in a flange plate bolt is determined by:

$$S_{Range,Bolt(Z)} = \frac{M_{Z,Gal}(\bar{y})}{I_{Bolt,Z}} \qquad \text{(Equation 5.10)}$$

$$S_{Range,Bolt(Y)} = \frac{M_{Y,Nat-Wind}(\bar{z})}{I_{Bolt,Y}} \qquad \text{(Equation 5.11)}$$

Where $I_{Bolt,Z}$ and $I_{Bolt,Y}$ are the moments of inertia of the bolt group. These are calculated as follows:

$$I_{Bolt,Z} = \sum A_{T,Bolt}\bar{y}^{2} \qquad \text{(Equation 5.12)}$$

and

$$I_{Bolt,Y} = \sum A_{T,Bolt}\bar{z}^{2} \qquad \text{(Equation 5.13)}$$

5.4 Mast-Arm-to-Flange-Plate Socket Connection Calculation Procedure

The stress range of the pole at the pole-to-baseplate connection is calculated by:

$$S_{Range,Mast-Arm} = \frac{M_{Z,Gal}\left(\frac{D_{base,mast-arm}}{2}\right)}{I_{mast-arm}} \qquad \text{(Equation 5.14)}$$

Where the moment of inertia of the mast arm is calculated by:

$$I_{mast-arm} = \frac{\pi}{64}\left[D^4_{Base,mast-arm} - \left(D_{Base,mast} - 2t\right)^4\right] \qquad \text{(Equation 5.15)}$$

Where t is the thickness of the mast-arm.

5.5 Built-up Box Stress Range Calculation Procedure

The stress range for the built-box is calculated by:

$$S_{Range,Box,Z} = \frac{M_{Z,Gal}\left(\frac{H}{2}\right)}{I_{box,z}} \qquad \text{(Equation 5.16)}$$

and

$$S_{Range,Box,Y} = \frac{M_{Y,Nat-Wind}\left(\frac{D_{MA-Pole}}{2}\right)}{I_{box,y}} \qquad \text{(Equation 5.17)}$$

where H is the height of the flange plate, $D_{MA-Pole}$ is the diameter of the pole at the height where it connects to the mast-arm. The moments of inertia of the built-up box are calculated as follows:

$$I_{box,z} = \sum \frac{tH^3}{12} + \sum A\left(\frac{H}{2} - \frac{t}{2}\right)^2 \qquad \text{(Equation 5.18)}$$

and

$$I_{box,y} = \sum \frac{tD^3_{MA-Pole}}{12} + \sum A\left(\frac{D_{MA-Pole}}{2} - \frac{t}{2}\right)^2 \qquad \text{(Equation 5.19)}$$

where t is the gusset thickness.

5.6 Deflection Calculation Procedure

The vertical displacement of the mast arm is calculated by:

$$Def_y = Def_{mast-arm} + \sum Def_i \qquad \text{(Equation 5.20)}$$

Where

$$Def_i = \left[\frac{F_{G,attachment,i}\, x_{base-to-attachement,i}}{6EI_{mast-arm,avg}} \right] \left[3L_{mast-arm} - x_{base-to-attachement,i} \right]$$

(Equation 5.21)

And

$$Def_{mast-arm} = \left[\frac{M_{Z,Gal}\, L_{pole-at-MA-connection}}{EI_{pole,avg}} \right] \left[L_{mast-arm} \right] \qquad \text{(Equation 5.22)}$$

Chapter 6

Sample Calculations

The following calculations were performed for a newly installed traffic light/sign structure in Lawrence, Kansas, at the intersection of Iowa and 31st Street. Actual measurements were taken, but some assumptions were also made. The picture and reference for the calculations is shown in Figure 6.1.

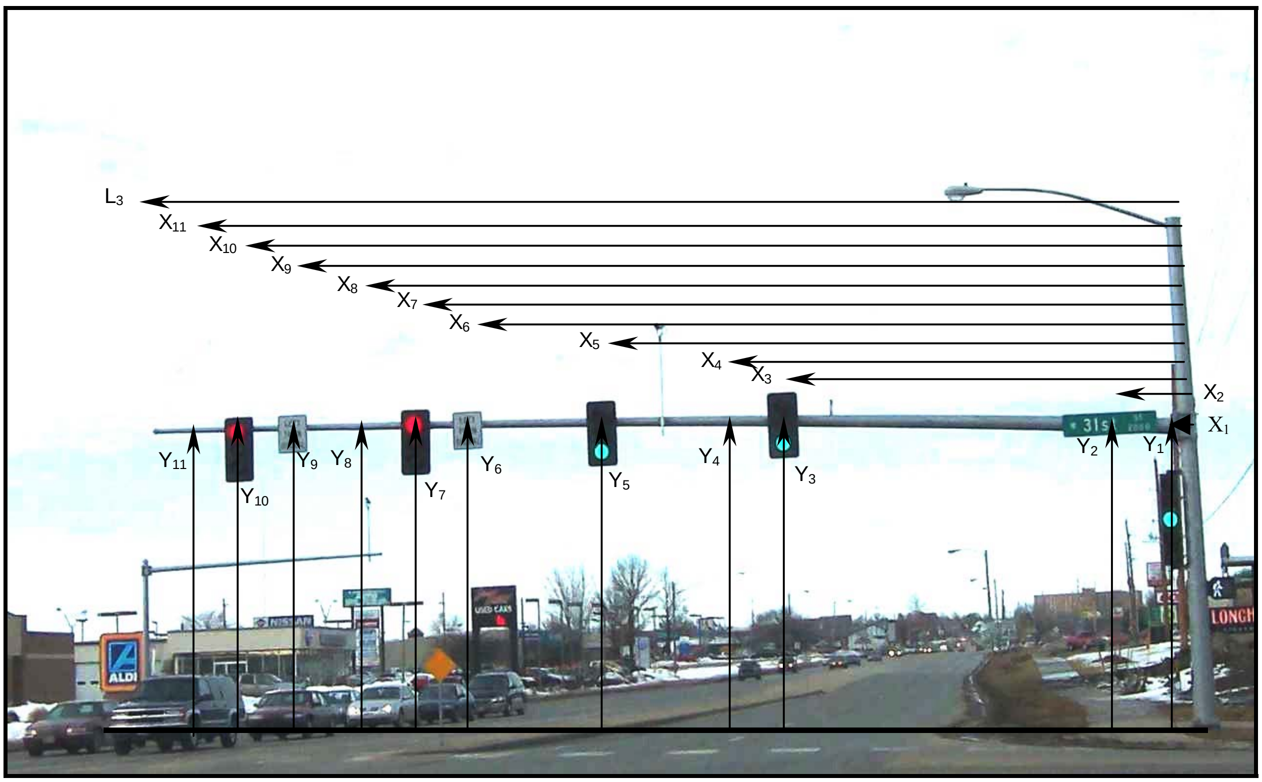

Figure 6.1: Reference Traffic Light-Signal Structure

6.1 Dimensions

6.1.1 Pole

• Pole Length, L1	=	32 ft
• Pole Length to Mast Arm, L2	=	19 ft
• Pole Diameter at Base, DB,pole	=	16 in
• Pole Diameter at MA-Connection, DMA,pole	=	13.34 in
• Pole Diameter at Tip, Dt,pole	=	11.52 in
• Thickness of Pole, t	=	0.313 in
• Taper	=	0.14 in/ft

6.1.2 Mast Arm

• Mast Arm Length, L_3	=	65 ft
• Mast Arm Diameter at Base, $D_{B,MA}$	=	13 in
• Thickness of Mast Arm, t	=	0.313 in
• Taper	=	0.14 in/ft
• Horizontal Distance to Signal 1, x_3	=	25 ft
• Horizontal Distance to Signal 2, x_5	=	36.5 ft
• Horizontal Distance to Signal 3, x_7	=	48 ft
• Horizontal Distance to Signal 4, x_{10}	=	59.5 ft
• Horizontal Distance to Sign 1, x_2	=	3.9063 ft
• Horizontal Distance to Sign 2, x_6	=	44.9219 ft
• Horizontal Distance to Sign 3, x_9	=	56.25 ft
• Street Sign Panel Length	=	5.8594 ft
• Left Turn Signal Panel length	=	1.85 ft

6.1.3 Mast Arm Subdivision Horizontal Distances

- Length of Mast Arm Section 1, M_1
  - $M_1 = x_2 – (5.8594/2) = 0.9766$ ft
- Length of Mast Arm Section 2, M_2
  - $M_2 = x_6 – (1.85/2)-(x_2+(5.8594/2)) = 37.1609$ ft
- Length of Mast Arm Section 3, M_3
  - $M_3 = x_9 – (1.85/2)-(x_6+(1.85/2)) = 9.4781$ ft
- Length of Mast Arm Section 4, M_4
  - $M_4 = L_3 – (x_9+(1.85/2)) = 7.8250$ ft

6.1.4 Horizontal Distances to Centroids of Mast Arm Sections

- Horizontal Distance to Centroid of Section 1, x_1
  - $x_1 = (x_2 – 5.8594/2)/2 = 0.4883$ ft
- Horizontal Distance to Centroid of Section 2, x_4
  - $x_4 = (x_2 + 5.8594/2) + [(x_6 – (1.85/2)) – (x_2 + 5.8594/2))]/2 = 25.4165$ ft
- Horizontal Distance to Centroid of Section 3, x8
  - $x_8 = (x_6 + 1.85/2) + [(x_9 – (1.85/2)) – (x_6 + 1.85/2))]/2 = 50.5860$ ft
- Horizontal Distance to Centroid of Section 4, x_{11}
  - $x_{11} = (x_9 + 1.85/2) + [L_3 – (x_9 + 1.85/2)]/2 = 61.0875$ ft

6.1.5 Heights from Ground to Signals, Signs, and Centroids of Sections

- Height to Signal 1, y_3
  - $y_3 = L_2 + x_3 (\tan 3^o) = 20.3102$ ft
- Height to Signal 2, y_5
  - $y_5 = L_2 + x_5 (\tan 3^o) = 20.9129$ ft
- Height to Signal 3, y_7
  - $y_7 = L_2 + x_7 (\tan 3^o) = 21.5156$ ft
- Height to Signal 4, y_{10}
  - $y_{10} = L_2 + x_{10} (\tan 3^o) = 22.1183$ ft
- Height to Sign 1, y_2
  - $y_2 = L_2 + x_2 (\tan 3^o) = 19.2047$ ft
- Height to Sign 2, y_6
  - $y_6 = L_2 + x_6 (\tan 3^o) = 21.3543$ ft
- Height to Sign 3, y_9
  - $y_9 = L_2 + x_9 (\tan 3^o) = 21.9479$ ft

- Height to Centroid of Section 1, y_1
  - $y_1 = L_2 + x_1 (\tan 3^o) = 19.0256$ ft
- Height to Centroid of Section 2, y_4
  - $y_4 = L_2 + x_4 (\tan 3^o) = 20.3320$ ft
- Height to Centroid of Section 3, y_8
  - $y_8 = L_2 + x_8 (\tan 3^o) = 21.6511$ ft
- Height to Centroid of Section 4, y_{11}
  - $y_{11} = L_2 + x_{11} (\tan 3^o) = 22.2015$ ft

6.1.6 Signal and Sign Projected Areas

- Area projected on a vertical plane by signal, $A_{signal,v} = (1.85)(3.8498) = 7.1221$ ft^2
- Area projected on a horizontal plane by signal, $A_{signal,h} = (1.85)(0.55) = 1.0175$ ft^2
- Area projected on a vertical plane by sign 1, $A_{sign\ 1,v} = (5.8594)(1.25) = 7.3243$ ft^2
- Area projected on a vertical plane by sign 2, $A_{sign\ 2,v} = (1.85)(2.3437) = 4.3359$ ft^2
- Area projected on a vertical plane by sign 3, $A_{sign\ 3,v} = (1.85)(2.3437) = 4.3359$ ft^2

6.1.7 Anchor Rods

- Nominal anchor rod diameter, D_{AR} = 1.5 in
- Thread series = 5 UNC
- Number of anchor rods = 4
- Anchor rod circle diameter, D_{AR} = 22.63 in
- Effective anchor rod area = 1.41 in^2

6.1.8 Flange Plate Bolts

- Nominal bolt diameter, D_B = 1.5 in
- Thread series = 8 UN
- Number of bolts = 4
- Effective bolt area = 1.49 in^2

6.2 Critical Fatigue Details

6.2.1 Anchor Rod

Detail 5, Table 11-2 (2001 Specifications)

6.2.2 Pole to Baseplate Fillet Welded Connection

Detail 16, Table 11-2 (2001 Specifications)

6.2.3 Mast Arm to Pole Connection

Details 5, 16, 17, and 19 (2001 Specifications)

6.3 Calculation of Limit State Fatigue Loads

6.3.1 Galloping

The magnitude of the vertical shear pressure range was calculated using Equation. 11-1 (2001 Specifications)

$$P_G = 21.0\ I_F$$
$$P_G = 21.0\ (1.0) = 21.0\ \text{psf}$$

$$F_{G,\ signal} = P_G\ (A_{signal,v}) = (21.0\ \text{lbf/ft}^2)(7.1221\ \text{ft}^2) = 149.5641\ \text{lbf}$$
$$F_{G,\ sign\ 1} = P_G\ (A_{sign\ 1,v}) = (21.0\ \text{lbf/ft}^2)(7.3243\ \text{ft}^2) = 153.8103\ \text{lbf}$$
$$F_{G,\ sign\ 2} = P_G\ (A_{sign\ 2,v}) = (21.0\ \text{lbf/ft}^2)(4.3359\ \text{ft}^2) = 91.0539\ \text{lbf}$$
$$F_{G,\ sign\ 3} = P_G\ (A_{sign\ 3,v}) = (21.0\ \text{lbf/ft}^2)(4.3359\ \text{ft}^2) = 91.0539\ \text{lbf}$$

6.3.2 Vortex Shedding

Tapered poles with tapers of 0.14 in/ft or higher are not required to resist vortex-shedding-induced loads.

6.3.3 Natural Wind Gusts

Based on Kansas yearly mean wind velocity of 11 mph, the equivalent static natural wind gusts pressure ranges were calculated as follows (Equation 11-5, 2001 Specifications):

$$P_{NW} = 5.2\ C_D\ I_F$$

C_D for traffic signals = 1.2

C_D for sign 1 = 1.2

C_D for signs 2 and 3 = 1.1

C_D for pole and mast arm = 1.1

$F_{NW,\ pole} = P_{NW}\ L_1\ D_{pole,avg}$ = (5.2)(1.1)(1.0)(19.0 ft)(14.67 in)(1/12)ft/in = 132.86 lbf

$F_{NW,\ MA\text{-}1} = P_{NW}\ M_1\ D_{MA\text{-}1,avg}$ = (5.2)(1.1)(1.0)(0.9766 ft)(12.93 in)(1/12)ft/in = 6.02 lbf

$F_{NW,\ MA\text{-}2} = P_{NW}\ M_2\ D_{MA\text{-}2,avg}$ = (5.2)(1.1)(1.0)(37.1609 ft)(9.4417 in)(1/12)ft/in = 167.24 lbf

$F_{NW,\ MA\text{-}3} = P_{NW}\ M_3\ D_{MA\text{-}3,avg}$ = (5.2)(1.1)(1.0)(9.4781 ft)(5.9185 in)(1/12)ft/in = 26.74 lbf

$F_{NW,\ MA\text{-}4} = P_{NW}\ M_4\ D_{MA\text{-}4,avg}$ = (5.2)(1.1)(1.0)(7.8250 ft)(4.4478 in)(1/12)ft/in = 16.59 lbf

$F_{NW,\ signal} = P_{NW}\ A_{signal,v}$ = (5.2)(1.2)(1.0)(7.1221ft^2) = 44.4419 lbf

$F_{NW,\ sign\ 1} = P_{NW}\ A_{sign\ 1,v}$ = (5.2)(1.2)(1.0)(7.3243ft^2) = 45.7036 lbf

$F_{NW,\ sign\ 2} = P_{NW}\ A_{sign\ 2,v}$ = (5.2)(1.1)(1.0)(4.3359ft^2) = 24.8013 lbf

$F_{NW,\ sign\ 3} = P_{NW}\ A_{sign\ 3,v}$ = (5.2)(1.1)(1.0)(4.33593ft^2) = 24.8013 lbf

6.3.4 Truck Gusts

The equivalent static truck gust pressure range was calculated as follows: (Equation 11-6, 2001 Specifications)

$$P_{TG} = 36.6\ C_D\ I_F$$

(which is to be calculated only on the outer 12 ft of the mast arm)

$F_{TG,\ MA} = P_{TG,\ MA}\ L\ D_{MA\text{-}12,avg}$ = (36.6)(1.1)(1.0)(12.0 ft)(4.74 in)(1/12)ft/in = 190.8324 lbf

$F_{TG,\ signal} = P_{TG,\ signal}\ A_{signal,h}$ = (36.6)(1.2)(1.0)(1.0175 ft^2) = 44.6886 lbf

$F_{TG,\ sign\ 3} = P_{TG,\ sign\ 3}\ A_{sign\ 3,h}$ = (36.6)(1.1)(1.0)(0.8356[(2)] ft^2) = 33.6413 lbf

[2] Assumed based on diameter of pole and thickness of panel

6.4 Calculation of Bending Moments

6.4.1 Moments Due to Galloping

Moment at the centerline of the pole (about the z-axis):

$$M_{z,\,Gal} = F_{G,\,signal}\,(x_3 + x_5 + x_7 + x_{10}) + F_{G,\,sign\,1}\,(x_2) + F_{G,\,sign\,2}\,(x_6) + F_{G,\,sign\,3}\,(x_9)$$

$$M_{z,\,Gal} = 149.5641\,(25 + 36.5 + 48 + 59.5) + 153.8103\,(3.9063) + 91.0539\,(44.9219) + 91.0539\,(56.25) = 35{,}089.25 \text{ lbf-ft} = 421.07 \text{ kip.in}$$

6.4.2 Moments Due to Natural Winds

Moment at the base of the pole (about the x-axis):

$$M_{x,\,Nat\text{-}Wind} = F_{NW,\,pole}\,(L_1/2\,) + F_{NW,\,signal}\,(y_3 + y_5 + y_7 + y_{10}) + F_{NW,\,mast\text{-}1}\,(y_1) + F_{NW,\,mast\text{-}2}\,(y_4) + F_{NW,\,mast\text{-}3}\,(y_8) + F_{NW,\,mast\text{-}4}\,(y_{11}) + F_{NW,\,sign\text{-}1}\,(y_2) + F_{NW,\,sign\text{-}2}\,(y_6) + F_{NW,\,sign\text{-}3}\,(y_9)$$

$$M_{x,\,Nat\text{-}Wind} = 132.86(32/2) + 44.4419(20.3102 + 20.9129 + 21.5156 + 22.1183) + 6.02(19.0256) + 167.24(20.3320) + 26.74(21.6511) + 16.59(22.2015) + 45.7036(19.2047) + 24.8013(21.3543) + 24.8013(21.9479) = 12{,}310.76 \text{ lbf-ft} = 147.73 \text{ kip-in}$$

Moments at the base of the mast arm (about the y-axis):

$$M_{y,\,Nat\text{-}Wind} = F_{NW,\,signal}\,(x_3 + x_5 + x_7 + x_{10}) + F_{NW,\,mast\text{-}1}\,(x_1) + F_{NW,\,mast\text{-}2}\,(x_4) + F_{NW,\,mast\text{-}3}\,(x_8) + F_{NW,\,mast\text{-}4}\,(x_{11}) + F_{NW,\,sign\text{-}1}\,(x_2) + F_{NW,\,sign\text{-}2}\,(x_6) + F_{NW,\,sign\text{-}3}\,(x_9)$$

$$M_{y,\,Nat\text{-}Wind} = 44.4419\,(25 + 36.5 + 48 + 59.5) + 6.02(0.4883) + 167.24(25.4165) + 26.74(50.5860) + 16.59(61.0875) + 45.7036(3.9063) + 24.8013(44.9219) + 24.8013(56.25) = 16{,}818.11 \text{ lbf-ft} = 201.82 \text{ kip-in}$$

6.4.3 Moments Due to Truck Gusts

Moment at the centerline of the pole (about the z-axis):

$$M_{z,\text{ Truck-Gust}} = F_{TG,\text{ mast-arm-12}}\,(L_3 - 12/2) + F_{TG,\text{ signal 4}}\,(x_{10}) + F_{TG,\text{ sign}}\,3\,(x_9)$$

$$M_{z,\text{ Truck-Gust}} = 190.83(65\text{-}12/2) + 44.6886(59.5) + 33.6413(56.25)$$
$$= 15{,}810.26 \text{ lbf-ft} = 189.72 \text{ kip-in}$$

6.5 Stress Range Calculations

6.5.1 Anchor Rods

The centroidal distances to the anchor rods are:

$$\bar{z} \quad = \quad (D_{ARC}/2)\sin 45^{o} = (22.63/2)\sin 45^{o} = 8.00 \text{ in}$$
$$\bar{x} \quad = \quad (D_{ARC}/2)\cos 45^{o} = (22.63/2)\cos 45^{o} = 8.00 \text{ in}$$

The tensile strength areas are:

$$A_{T,AR} = (\pi/4)[D_{AR} - (0.9743/n)]^2 = (\pi/4)[1.5 - (0.9743/5)]^2 = 1.34 \text{ in}^2$$

The moments of inertia about the x and z axes are:

$$I_{AR,X} = \sum A_{T,AR}\bar{z}^2 = 4(1.34 \text{ in}^2)\,(8.00 \text{ in})^2 = 343.0 \text{ in}^4$$
$$I_{AR,Z} = \sum A_{T,AR}\bar{x}^2 = 4(1.34 \text{ in}^2)\,(8.00 \text{ in})^2 = 343.0 \text{ in}^4$$

6.5.1.1 Anchor Rod Stress Range

The anchor rods must be checked with respect to galloping and natural wind gusts. The minimum axial stress range found in an anchor rod was:

$$S_{Range,AR(Z)} = \frac{M_{Z,Gal}(\bar{x})}{I_{AR,Z}} = \frac{(421.07kip - in)(8.0in)}{343in^4} = 9.82ksi$$

$$S_{Range,AR(X)} = \frac{M_{X,Nat-Wind}(\bar{z})}{I_{AR,X}} = \frac{(147.73kip-in)(8.0in)}{343in^4} = 3.44ksi$$

According to the 2001 Specification, the stress category for anchor bolts in tension based on the tensile stress area is D, which for steel has a Constant Amplitude Fatigue Threshold (CAFT) of 7 ksi. ***Therefore, the current design (based on the assumptions made) does not meet requirements related to galloping. If the anchor rod diameters were increased to 1.75 in, requirements would be met.***

For example:

$$S_{Range,AR(Z)} = \frac{M_{Z,Gal}(\bar{x})}{I_{AR,Z}} = \frac{(421.07kip-in)(8.0in)}{486.4in^4} = 6.93ksi$$

$$S_{Range,AR(X)} = \frac{M_{X,Nat-Wind}(\bar{z})}{I_{AR,X}} = \frac{(147.73kip-in)(8.0in)}{486.4in^4} = 3.44ksi$$

Where the tensile strength areas are:

$$A_{T,AR} = (\pi/4)[D_{AR} - (0.9743/n)]^2 = (\pi/4)[1.75 - (0.9743/5)]^2 = 1.90\text{ in}^2$$

and the moments of inertia about the x and z axes are:

$$I_{AR,X} = \sum A_{T,AR}\bar{z}^2 = 4(1.90\text{ in}^2)(8.00\text{ in})^2 = 486.4\text{ in}^4$$

$$I_{AR,Z} = \sum A_{T,AR}\bar{x}^2 = 4(1.90\text{ in}^2)(8.00\text{ in})^2 = 486.4\text{ in}^4$$

6.5.2 Pole to Baseplate Socket Connection Stress Range

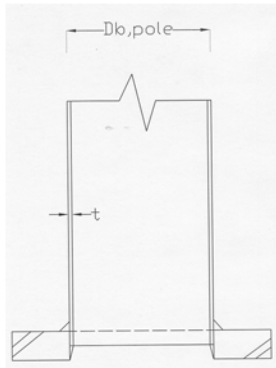

Figure 6.1: Pole to Baseplate Socket Connection Details

Referring to Figure 6.1, the moment of inertia of the pole is

$$I_{pole} = \frac{\pi}{64}\left[D^4_{Base,pole} - \left(D_{Base,pole} - 2t\right)^4\right] = \frac{\pi}{64}\left[16^4 - \left(16 - 0.626\right)^4\right] = 474.68\ in^4$$

and since galloping controls the design of the anchor bolts, galloping controls the design of the pole-to-base connection. The strength range for this detail is found from:

$$S_{Range,Pole} = \frac{M_{Z,Gal}\left(\frac{D_{base,pole}}{2}\right)}{I_{pole}} = \frac{(421.07)\left(\frac{16}{2}\right)}{474.68} = 7.10 \text{ ksi}$$

The socket connection is classified as a category E', which has a fatigue limit of 2.6 ksi. Therefore, the current design (based on the assumptions made) does not meet requirements related to galloping.

Based on calculations, the potential redesigns include increasing the pole diameter to 26 in, or increasing the pole diameter 21 in while improving the detail category to E (4.25 ksi) and increasing the pole thickness to 0.55 in.

6.5.3 Mast Arm to Pole Connection Stress Range

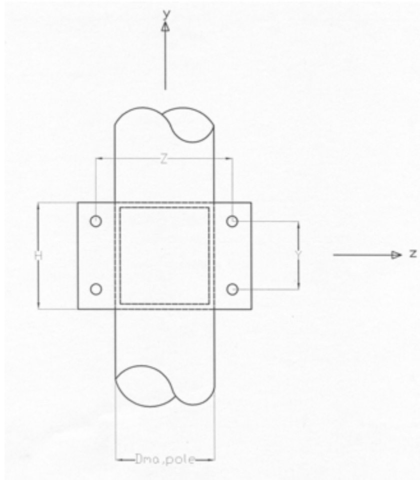

Figure 6.2: Flange Plate Bolt Group Details

Referring to Figure 6.2, the information is as follows for the flange plate bolt group.

Nominal bolt diameter, D_B	= 1.5 in
Thread series	= 8 UN
Effective bolt area, $A_{T,B}$	= 1.49 in^2
Number of bolts	= 4
Z	= 17.85 in
Y	= 16 in

6.5.3.1 Moment of Inertia

Centroidal distances

$\bar{z}$ = Z/2 = 8.93 in

$\bar{y}$ = Y/2 = 8.0 in

the tensile strength areas of the bolts are:

$$A_{T,B} = (\pi/4)[D_B - (0.9743/n)]^2 = (\pi/4)[1.5 - (0.9743/8)]^2 = 1.49\ in^2$$

the moments of inertia about the y and z-axes are:

$$I_{Bolt,Z} = \sum A_{T,Bolt}\bar{y}^2 = 4(1.49\ \text{in}^2)\ (8.00\ \text{in})^2 = 382.0\ \text{in}^4$$

$$I_{Bolt,Y} = \sum A_{T,Bolt}\bar{z}^2 = 4(1.49\ \text{in}^2)\ (8.93\ \text{in})^2 = 475.28\ \text{in}^4$$

6.5.3.2 Flange Plate Bolt Stress Range

The flange plate bolt stress range was calculated by checking the moment due to galloping (about the z-axis) and the moment due to natural wind (about the y-axis). The bolt stress ranges were:

$$S_{Range,Bolt(Z)} = \frac{M_{Z,Gal}(\bar{y})}{I_{Bolt,Z}} = \frac{(421.07kip-in)(8.0in)}{382.0\ in^4} = 8.82\ ksi$$

$$S_{Range,Bolt(Y)} = \frac{M_{Y,Nat-Wind}(\bar{z})}{I_{Bolt,Y}} = \frac{(201.82kip-in)(8.93in)}{475.28in^4} = 3.79\ ksi$$

Since bolts are classified as D category (7 ksi) and since the stress range due to galloping is higher, the current design (based on the assumptions made) is under designed.

Based on calculations, the potential redesigns include increasing the size of the plate to 25 in x 25 in. or increase the bolt diameter to 1.75 in.

6.5.3.3 Mast Arm to Flange Plate Socket Connection Stress Range

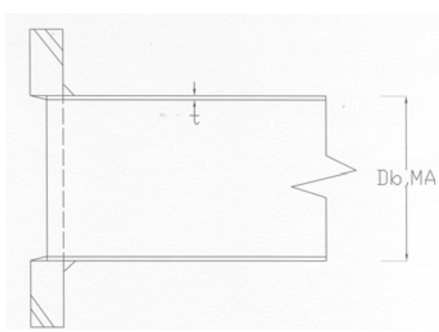

Figure 6.3: Mast Arm to Flange Plate Socket Connection Details

Referring to Figure 6.3, the moment of inertia was calculated as

$$I_{mast-arm} = \frac{\pi}{64}\left[D^4_{Base,mast-arm} - \left(D_{Base,mast} - 2t\right)^4\right] = \frac{\pi}{64}\left[13.0^4 - \left(13.0 - 0.626\right)^4\right] = 251.16 \text{ in}^4$$

Since galloping is the controlling mode of vibration, the stress range due to galloping is found by:

$$S_{Range,Mast-Arm} = \frac{M_{Z,Gal}\left(\frac{D_{base,mast-arm}}{2}\right)}{I_{mast-arm}} = \frac{421.07\left(\frac{13.0}{2}\right)}{251.16} = 10.9 \text{ ksi}$$

Since this detail is classified as E’ category (2.6 ksi) and since the stress range due to galloping is higher, the current design (based on the assumptions made) is under designed.

Based on calculations, increasing the diameter of the mast arm and increasing the category to E (4.25 ksi) and increasing the thickness of the mast arm did not improve the design unless the aforementioned dimensions were <u>significantly</u> altered.

One recommendation would be to use the “saddle” type bracket support recommended by the University of Kansas (Figures 4.2 thru 4.5) or the ring-stiffened built-up box. According to a finite element analysis, the “saddle” support allows the stress at the welds to be reduced by 1/3 (Yan, 2001). Thus, if this support were used, the stress at the socket connections would be approximately 3.63 ksi, and if the detail category were improved to E (4.25 ksi), requirements would be met. It is also recommended to replace the weld with a full penetration weld.

6.5.3.4 Built-up Box Stress Range

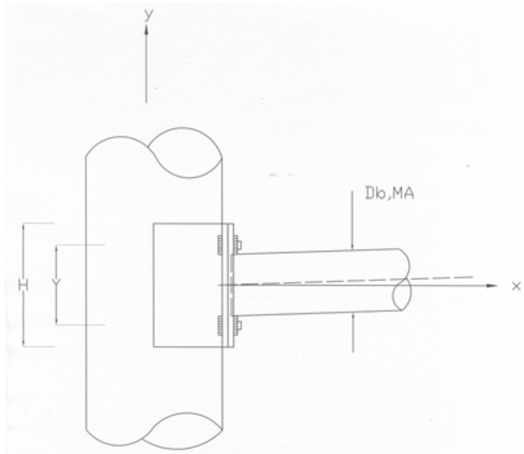

Figure 6.4: Built-up Box Details

Based on Figure 6.4, Moments of inertia for built-up box were

$$I_{box,z} = \sum \frac{tH^3}{12} + \sum A\left(\frac{H}{2} - \frac{t}{2}\right)^2 =$$

$$= \frac{2(0.375in)(19.50in)^3}{12} + 2(0.375)(13.34)\left(\frac{19.5}{2} - \frac{0.375}{2}\right)^2$$

$= 1378.3$ in^4

and

$$I_{box,y} = \sum \frac{tD^3_{MA-Pole}}{12} + \sum A\left(\frac{D_{MA-Pole}}{2} - \frac{t}{2}\right)^2 =$$

$$= \frac{2(0.375in)(13.34in)^3}{12} + 2(0.375)(19.5)\left(\frac{13.34}{2} - \frac{0.375}{2}\right)^2$$

$= 762.95$ in^4

The stress range for the built-box is calculated by:

$$S_{Range,Box,Z} = \frac{M_{Z,Gal}\left(\frac{H}{2}\right)}{I_{box,z}} = \frac{421.07 kip - in\left(\frac{19.50}{2}\right)in}{1378.3\ in^4}$$ = 2.97 ksi > 2.6 ksi (Category E') > 1.2 ksi (Category ET – Detail 19, Note (b) in Table 11-2)

and

$$S_{Range,Box,Y} = \frac{M_{Y,Nat-Wind}\left(\frac{D_{MA-Pole}}{2}\right)}{I_{box,y}} = \frac{201.82 kip - in\left(\frac{13.34}{2}\right)in}{762.95\ in^4}$$ = 1.76 ksi > 1.2 ksi (Category ET – Detail 19, Note (b) in Table 11-2)

The pole must also be checked for category E fatigue detail (4.5 ksi) at the bottom of the welded connection on the branching member (built-up box), which is Detail 19, Note (b) in Table 11-2 of the 2001 Specifications. This was done using

$$S_{Range,Pole-at-MA-Connection} = \frac{M_{Z,Gal}\left(\frac{D_{pole-MA}}{2}\right)}{I_{pole-MA}} = \frac{(421.07)\left(\frac{13.34}{2}\right)}{271.89}$$ = 10.33 ksi > 4.5 ksi

According to these calculations, the built-up box is also under-designed. Possible redesigns include increasing the plate size, improving the categories for fatigue details, increasing pole and mast arm diameters and thicknesses, all of which seem very impractical. A possible recommendation could be the use the “saddle” type bracket support recommended by the University of Kansas or the ring-stiffened built-up box and to consider changing the requirements to Fatigue Category II.

6.6 Deflection Calculations

Vertical plane displacement ranges of the original structure were calculated using the method of superposition based on individual displacements by the signals, signs, and the mast arm. The displacement quantities were based on the average moment of inertia of each of the components.

6.6.1 Average Moment of Inertia of the Pole

$$I_{pole, (base)} = 474.68 \text{ in}^4$$

The moment of inertia of the pole at the connection height was:

$$I_{pole(MA)} = \frac{\pi}{64}\left[D^4_{pole,MA} - \left(D_{pole(MA)} - 2t\right)^4\right] = \frac{\pi}{64}\left[13.34^4 - \left(13.34 - 0.626\right)^4\right] = 271.89 \text{ in}^4$$

The average moment of the pole, $I_{pole, avg}$ was 373.29 in^4

6.6.2 Average Moment of Inertia of the Mast Arm

$$I_{mast\ arm, (base)} = 252.16 \text{ in}^4$$

The moment of inertia of the pole at the connection height was:

$$I_{MA-tip} = \frac{\pi}{64}\left[D^4_{MA-tip} - \left(D_{MA-tip} - 2t\right)^4\right] = \frac{\pi}{64}\left[3.9^4 - \left(3.9 - 0.626\right)^4\right] = 5.72 \text{ in}^4$$

The average moment of the mast arm, Imast arm, avg was 128.44 in4

The displacements were calculated as follows:

$$Def_1 = \left[\frac{F_{G,sign1}x_2^2}{6EI_{mast-arm,avg}}\right]\left[3L_3 - x_2\right] =$$

$$= \left[\frac{(0.1538kip)(3.9063\,ft)^2(1728in^3/ft^3)}{6(29000ksi)(128.44in^4)}\right]\left[3(65\,ft) - 3.9063\,ft\right] = 0.0347 \text{ in}$$

$$Def_2 = \left[\frac{F_{G,signal-1}x_3^2}{6EI_{mast-arm,avg}}\right]\left[3L_3 - x_3\right]$$

$$= \left[\frac{(0.1496kip)(25\,ft)^2(1728in^3/ft^3)}{6(29000ksi)(128.44in^4)}\right]\left[3(65\,ft) - 25\,ft\right] = 1.2287 \text{ in}$$

$$Def_3 = \left[\frac{F_{G,signal-2}x_5^2}{6EI_{mast-arm,avg}}\right]\left[3L_3 - x_5\right]$$

$$=\left[\frac{(0.1496kip)(36.5ft)^2(1728in^3/ft^3)}{6(29000ksi)(128.44in^4)}\right]\left[3(65ft)-36.5ft\right] = 2.442 \text{ in}$$

$$Def_4=\left[\frac{F_{G,sign2}x_6^2}{6EI_{mast-arm,avg}}\right]\left[3L_3-x_6\right]$$

$$=\left[\frac{(0.091kip)(44.9219ft)^2(1728in^3/ft^3)}{6(29000ksi)(128.44in^4)}\right]\left[3(65ft)-44.9219ft\right] = 2.13 \text{ in}$$

$$Def_5=\left[\frac{F_{G,signal-3}x_7^2}{6EI_{mast-arm,avg}}\right]\left[3L_3-x_7\right]$$

$$=\left[\frac{(0.1496kip)(48ft)^2(1728in^3/ft^3)}{6(29000ksi)(128.44in^4)}\right]\left[3(65ft)-48ft\right] = 3.90 \text{ in}$$

$$Def_6=\left[\frac{F_{G,sign3}x_9^2}{6EI_{mast-arm,avg}}\right]\left[3L_3-x_9\right]$$

$$=\left[\frac{(0.091kip)(56.25ft)^2(1728in^3/ft^3)}{6(29000ksi)(128.44in^4)}\right]\left[3(65ft)-56.25ft\right] =3.09 \text{ in}$$

$$Def_7=\left[\frac{F_{G,signal-4}x_{10}^2}{6EI_{mast-arm,avg}}\right]\left[3L_3-x_{10}\right]$$

$$=\left[\frac{(0.1496kip)(59.5ft)^2(1728in^3/ft^3)}{6(29000ksi)(128.44in^4)}\right]\left[3(65ft)-59.5ft\right] = 5.55 \text{ in}$$

The mast arm displacement range was calculated by:

$$Def_{mast-arm} = \left[\frac{M_{Z,Gal} L_{pole-at-MA-connection}}{EI_{pole,avg}} \right] \left[L_{mast-arm} \right] =$$

$$= \left[\frac{(421.07 kip - in)(19 ft)(12 in/ft)}{(29000 ksi)(373.29 in^4)} \right] (65 ft)(12 in/ft) = 6.91 \text{ in}$$

Def_{total} = 0.0347 + 1.2287 + 2.442 + 2.13 + 3.92 + 3.09 + 5.55 + 6.91 = 25.31 in

Based on KDOT's requirements that the tip of the mast arm clearance be 15 ft above the ground at all times and that the bottom side of the signal and sign panels be at least 17 ft above the ground on the original structure, the structure of this example does not meet this requirements by a deflection of 1.31 in.

Based on calculations, increasing the diameter of the mast arm to 13.5 in at the base would be enough to meet requirements. Also, shortening the mast arm to 64 ft from 65 ft would allow this design to meet requirements.

Chapter 7

Conclusions and Recommendations

The 2001 AASHTO Specifications together with the current literature on the design of traffic signal structures were reviewed. In addition, a survey of many State DOTs was conducted. Based on the findings, design criteria, specifications, and design details were compiled. Based also on these findings and calculations, it was concluded that KDOT revisions of its traffic structure design practices are minor. In essence, the recommendations of this study are mainly directed at keeping the anchor rod diameters at 1.5 in. or more, preferably 1.75 in., use of full penetration welds in the pole to baseplate connections, institute the use of the "saddle" type bracket support recommended by the University of Kansas or the ring-stiffened built-up box type support for the connection of the mast arms to the poles, and to consider moving the requirements from Fatigue Category I to Fatigue Category II.

One recommendation for the ongoing research is to perform calculations on a number of structures located throughout the State. The sample population of structures should include those in use for a one-traffic light, for two-, three-, and so forth. The calculations must be performed with actual measured data for each of the structures. Instrumentation (e.g., strain gauges) of a sample of structures should be considered as well as actual videotaping of structures for a period of time.

REFERENCES

2001 AASHTO Specifications: American Association of State Highway and Transportation Officials, Standard Specifications for Structural Support for Highway Signs, Luminaires, and Traffic Signals, Washington, D.C., 2001.

NCHRP Project 10-38: Fatigue-Resistant Design of Cantilevered Signal, Sign, and Light Supports.

ASCE/ANSI 7-95 (1996), Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Reston, VA. This is considered the most authoritative standard on wind loading in the United States.

Simiu, E., and Scalan, R.H. (1978), Wind Effects on Structures: An Introduction to Wind Engineering. 2nd Ed.,, John Wiley and Sons, New York.

McDonald, J.R., et al., (1995), "Wind Load Effects on Signals, Luminaires, and Traffic Signal Structures." Report 1303-1F, Wind Engineering Research Center, Texas Tech University, Lubbock, TX.

Mututwa, Peter, Valmont Structures, Private Communications, 2004.

Yan, Zhijie, 2001, "Fatigue Study of Traffic Signal Structures Under Aerodynamic Loading Using Finite Element Method," CE890 Special Problems Report, Civil, Environmental, and Architectural Engineering, Department, University of Kansas, Lawrence.

K - TRAN

KANSAS TRANSPORTATION RESEARCH
AND
NEW - DEVELOPMENTS PROGRAM

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:

KANSAS DEPARTMENT OF TRANSPORTATION

THE UNIVERSITY OF KANSAS

KANSAS STATE UNIVERSITY