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**MARYLAND DEPARTMENT OF TRANSPORTATION
STATE HIGHWAY ADMINISTRATION**

RESEARCH REPORT

**The Calibration of the AASHTO ASD and LRFD for Maryland
Sign and High Mast Lighting Structure Design**

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FINAL REPORT

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16. Abstract The Maryland Department of Transportation State Highway Administration (MDOT SHA) is working to update the standard design for sign and mast arm lighting structures. The following task were performed as a part of this research to assist with the update process: <ol style="list-style-type: none"> 1. Complete design parameters of MD-specified sign 2. Defining wind pressure applicable by both ASD & LRFD design codes for the sign structure design for different regions of Maryland 3. Determining the additional impact in transition to the LRFD design, including Maryland sign structure foundation designs 4. Developing and calibrating fatigue load resisting connections 5. Reviewing the conformance of SABRE design models with alternate commercial licensed finite element analysis software 					
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Executive Summary

Since the original edition in 1994, the American Association of State Highway and Transportation Officials' (AASHTO) Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, have changed several times (from LTS-4 to LTS-6, then to LRFD LTS-1). In November 2018, the Federal Highway Administration (FHWA) published the Final Rule to Title 23 Code of Federal Regulations Part 625 (NPRM -23 CFR part 625) requiring the use of updated standards for all NHS projects authorized to proceed with design activities on or after December 3, 2018. States DOTs must now choose one of these alternative specifications, LTS-6 (ASD) or LRFD LTS-1, to perform the design, fabrication, and erection of Structural Supports for Highway Signs, Luminaires, and Traffic Signals. In preparation to adopt AASHTO STD LTS-6 and the future LRFD LTS-1 for sign and signal structure, the Maryland Department of Transportation State Highway Administration (MDOT SHA) worked with the Bridge Engineering Software and Technology (BEST) Center, University of Maryland, to develop new signal standards. This research was divided into six sections.

1. Complete design parameters of MD-specified sign and light structures

A previous study completed by the research team, Fatigue Resistant Design Criteria for MDOT SHA Cantilevered Mast Arm Signal Structures, published in 2017, included extensive studies to determine the design parameters for Maryland-specified signal structures. A similar approach was used in this project to identify the Maryland-specified Fatigue Category of Importance Factor (I, II or III) for sign and high mast light structure designs.

In the 2017 study, the fatigue design of mast arm structures was thoroughly studied. The study gathered and discussed the current state-of-the-practice methods. Complete model analysis of traffic signal structure, including structure foundation, was conducted using ANSYS and SABRE programs and self-developed Excel calculation sheets. Recommendations for Maryland structure designs were proposed.

2. Define Maryland wind pressure in LRFD for sign structure design

Both wind pressure formulas for allowable stress design (ASD) and Load and Resistance Factor Design (LRFD) were studied and the changes and their influences are discussed in this report. The change of Maryland wind speed has been specifically analyzed, since AASHTO ASD LTS-6 and LRFD LTS-1 have significantly different approaches in this section. The current wind speed of 100mph for all of Maryland is no longer reasonable. This research recommends that Maryland be divided into three regions: the Eastern Shore, the Appalachia Mountain, and the Baltimore-Washington corridor. The Average Daily Traffic (ADT) in the Eastern Shore and Appalachia Mountain regions can be considered to be within $1000 < ADT \leq 10,000$ and structures can be designed as "typical" supports (MRI = 700 years); while the Baltimore-Washington corridor should be considered to be $ADT > 10,000$ where travel-ways are assigned a high-risk category so that the consequence of failure (MRI=1700 years) is factored in.

The wind speed adopted for STD LTS-6 is 100mph, while for LRFD LTS-1 it is 120mph. If LRFD is adopted for Maryland designs, wind speed of 120mph should be adopted. A wind load pressure analysis was completed using different AASHTO specifications. The contribution of multi-arm influence was also included and discussed.

3. Calibration of LRFD Designs by Adopting Fatigue Design

Maryland signal poles that need to be modified in order to increase the fatigue resistance have been identified with recommended modifications. The recommended modifications of the current design on the signal poles include: a) Groove welds for arm connections, b) Groove welds for pole connections, c) Adopting AASHTO build-up box type for arm connections, and d) 6-bolt patterns for both arm and pole connections. Other structure changes and recommendations were discussed in this study, such as tube-to-tube connections between main chords and bracings for sign structures.

A comparison of the AASHTO STD LTS-6 & LRFD LTS-1 Specs was completed. Their impacts of modifying details on sign structural supports due to fatigue were also studied in this task.

4. LRFD Foundation Design

In the AASHTO LRFD LTS-1, Extreme I, Service I, and II Limit States have Wind Load (W) involved and they should be considered in foundation designs. In this study, the typical Maryland standard foundation types were analyzed. These types include shaft foundation for signal poles, shaft foundation with wing walls for cantilever sign structures, and mat found with pedestal for overhead structures.

For shaft foundation and shaft foundation with wing walls under an assumed soil condition, shaft embedment lengths are longer than the required length in both cohesive and cohesionless soils based on Broms' methods. The reinforcement design of the shaft is sufficient to resist ground moment from the base. Torsion resistances of the shaft are higher than the maximal base torque provided by SABRE software either in the cohesive or cohesionless soil.

For the mat found with pedestal for overhead structures, three individual cases were examined. Both sliding and eccentricity limits were investigated in two horizontal directions. The flexural capacity was checked by calculating the moment capacity of the footing given the number of rebars present. Overall, the spread footings design passes the check under the critical conditions with adequate capacity.

5. Comparison between the AASHTO STD LTS-6 and LRFD LTS-1 Specifications

A detailed comparison between AASHTO LTS-6 and LRFD LTS-1 was completed in this study. The expressions for LTS-6 allowable stress and LRFD LTS-1 nominal strength are dramatically different. In this section, the tube sections are used to demonstrate such differences in expressions and their effects. The interaction equations are also significantly different since one is based on combined stress ratio while the other is combined force. Samples from Maryland Standards were used for checking Strength I Limit State.

For the fatigue study, four case studies with examples from National Cooperative Highway Research Program (NCHRP) Report 718 – *Fatigue Loading and Design Methodology for High-Mast Lighting Tower*; NCHRP Report 796 – *Development and Calibration of AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*; and a New Jersey Department of Transportation (NJDOT) report - *Fatigue Study on Structural Supports for Luminaries, Traffic Signals, Highway Signs (2015)*, are discussed. Their results

were verified using SABRE and STAAD analysis. The verification process indicates that SABRE is accurate for the fatigue analysis. Also, several calculation mistakes used in the NCHRP and NJDOT reports were identified and the correct figures are provided in Chapter 6.

6. SABRE/LRFD System review

A complete review and conformance test on the SABRE program were performed. SABRE design models of highway sign, high post, and traffic signal post were verified by an alternate commercial licensed finite element analysis software, STADD Pro. The comparisons are in good agreement.

Chapter 1 Introduction

1.1 Research Problem Statement

Since the original edition in 1994, the AASHTO Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals have changed several times. They are listed below with a brief description of the changes:

- 1994 (STD LTS-3 and earlier) edition – original; load combinations 1-3
- 2001 (STD LTS-4) edition – revamped; new wind criteria
- 2009 (STD LTS-5) edition – new wind definition; foundation/anchor bolts; fatigue introduced as load combination 4; retrofit/rehab; miscellaneous
- 2013 (STD LTS-6) edition – wind; fatigue (major revision); foundation
- 2015 (LRFD LTS-1) edition – new wind; new load combination; load and resistance factors; same STD LTS-6 fatigue design

The National Cooperative Highway Research Program (NCHRP) sponsored research in the problem areas and AASHTO released the specifications. It is now the states' responsibility to accommodate their designs based on different versions of the codes. Specifications list three types of roadside structures: highway sign structures, high mast light poles, and traffic signal posts. The current (2017) status of Maryland structures is as follows:

- MD structural supports for highway signs: STD LTS-4
- MD structural supports for high mast light poles: STD LTS-5 (based on MDTA Standard)
- MD structural supports for traffic signals: STD LTS-3 (recently to STD LTS-6)

1.2 Research Objectives

The FHWA published the Final Rule to Title 23 Code of Federal Regulations Part 625 (NPRM - 23 CFR part 625) and required for all NHS projects authorized to proceed with design activities on or after December 3, 2018. States DOTs must choose one of these alternative specifications, LTS-6 (ASD) or LRFD LTS-1, to perform the design, fabrication, and erection of Structural Supports for Highway Signs, Luminaires, and Traffic Signals. MDOT SHA worked with the Bridge Engineering Software and Technology (BEST) Center, University of Maryland, to develop new sign standards. The update for overhead sign structures standard design were in progress at MDOT SHA's Office of Traffic and Safety. However, MDOT SHA had the following challenges that required further research:

1. Complete design parameters of MD-specified sign structures by identifying the Maryland-specified Fatigue Category of Importance Factor (I, II or III) for sign and light structure designs.
2. Defining wind pressure applicable by both ASD & LRFD design codes for sign structure design for different regions of Maryland (Eastern Shore, Appalachia Mountain and the Baltimore-Washington corridor)

3. Determining the additional impact in transition to the LRFD design in the future, i.e. wind pressure determination.
4. Developing fatigue load resisting connections, i.e. choosing between pipe-to-pipe vs gusset-plate weld connections for box truss sign structures, e.g., groove welded vs socket welded, etc...
5. Reviewing the conformance of SABRE design models with alternate commercial licensed finite element analysis software, e.g., STADD Pro.

1.3 Benefits

The benefits of this study are as follows:

1. Complete design parameters for the MD-specified sign structures by identifying the Maryland-specified Category of Importance Factor (I, II or III) and wind categories, and other variables for sign/lighting designs
2. Updating MDOT SHA book of standards for Highway, Incidental Structures, and Traffic Control Applications to meet the latest AASHTO design criteria, even to the future mandate LRFD requirement
3. Providing complete structural standard details, including foundation design, for Maryland highway overhead, cantilever, pedestal dynamic message signs (DMS) support sign structures, and high mast light poles.

1.4 Research Plan

Tasks associated with the calibration of the old and new designs:

Task 1: Complete design parameters of MD-specified sign and light structures

Task 2: Determination of wind pressure based on the rationale of wind loads and their mean recurrence interval (MRI) defined in the AASHTO LRFD LTS-1 Specs.

Task 3: Calibration of designs by adopting the fatigue design defined in the AASHTO LRFD LTS-1 Specs. and developing fatigue resisting connections

Task 4: Foundation design by the AASHTO LRFD LTS-1 Specs.

Task 5: Comparison between the AASHTO STD LTS-6 and LRFD LTS-1 Specs.

Task 6: SABRE System review

Task 7: Summary and Report

Chapter 2 Complete design parameters of MD-specified sign and light structures

In a previous project completed by this research team, Fatigue Resistant Design Criteria for MDOT SHA Cantilevered Mast Arm Signal Structures, published in 2017 extensive studies were conducted to determine the complete design parameters for MD-specified signal structures. A similar approach was used in this study to identify the Maryland-specified Fatigue Category of Importance Factor (I, II or III) for sign and high mast light structure designs.

The determination of the design parameters is based on (1) literature review of the previous and current studies conducted by the research team; (2) a previous survey conducted by Maryland for signal structures and Arizona for sign structures; (3) members of the AASHTO Committee on Bridges and Structures, T-12 Structural Supports, and TRB Committee on General Structures (AFF10); and (4) Maryland requirements as determined by MDOT SHA and the research team.

In the 2017 study, the fatigue design of mast arm structures was thoroughly studied. The study gathered and discussed the current state-of-the-practice methods. A complete model analysis of traffic signal structure, including structure foundation, was conducted using ANSYS and SABRE programs and Excel calculation sheets. Recommendations for Maryland structure designs were proposed. A cost analysis of a 60'-arm length pole was studied for reference. The following conclusions can be drawn:

1. The latest fatigue designs in AASHTO LRFD Specification for signal structural supports and different types of mitigation devices were surveyed and studied. Twenty-seven (27) out of 50 state DOTs replied to the 21-question signal structure questionnaire. Information about other states' practices and current mitigation devices were also gathered for reference. The following are the new criteria adopted by the State of Maryland which are summarized, studied and decided based on the survey:
 - Using Importance Category II with mitigation device in the fatigue design for galloping and Importance Category I without; using Category I for other fatigue loading
 - Using 50-year design life
 - Applying groove welds for both arm and pole connections
 - Adopting the AASHTO built-up box type
 - Using signal head back plate
 - Using non-stiffened pole base (as currently practiced)
 - Using 6-bolt pattern for both arm and based connection plates
 - Adopting the PennDOT 24"x24" wing plate as the mitigation device
 - Using 100-mph wind speed for STD LTS-6 design (as currently practiced)

2. A detailed comparison was made between AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals and the existing MDOT SHA Book of Standards. It was concluded that the majority of Maryland signal structures located along state routes are classified as Category I due

to the speed limit and ADT. However, by having effective mitigation devices, the fatigue load category for galloping could be lowered from Category I to II. Among mechanical damping devices, the spring-mass impact damper is an effective mechanical mitigation device, but the wing plate is considered a feasible aerodynamic mitigation device.

3. Several models with different arm lengths (50', 60', 70' and 75') based on MDOT SHA Standards for Highway and Incidental Structures were studied using the SABRE program authored by the research team. Suggested design improvements based on the latest AASHTO fatigue criteria are proposed. To satisfy new fatigue design requirements, new groove weld tube-to-transverse plate connection and new pole/arm size should be adopted into the current design standards.
4. Based on NCHRP Report 469 – *Fatigue-Resistant Design of Cantilevered Signal, Sign, and Light Supports*, preliminary fluid-solid-interaction numerical analyses were conducted with horizontal damping plate to provide the basis of reducing the galloping from Category I to II. For the selected 75'-arm signal pole model with damping plate, dynamic moment, after a few cycles, it can be reduced to 65% of the first cycle. This reduction happens to match the importance factor 1.0 of Category I to 0.65 of Category II. In order to study the effectiveness of the damping plate, a future parametric study based on proper parameters should be completed.
5. The current Maryland signal pole foundation design based on assumed soil properties was also checked under the new proposed signal structure design. Both foundation embedment length and longitudinal reinforcement are within AASHTO requirements. Torsional resistance was also checked and found adequate using the Illinois DOT method.

Chapter 3 Determination of Maryland Wind Pressure in LRFD

3.1 Wind Load defined in the AASHTO STD LTS-6 & LRFD LTS-1 Specifications.

Wind loads based on 2013 ASD (AASHTO STD LTS-6) and 2015 LRFD (AASHTO LRFD-LTS1) are shown below where the highlighted parts are the differences between the two specifications:

a) 2013 wind pressure

$$P_z = 0.00256 K_z * G * V^2 * I_r * C_d$$

b) 2015 LRFD wind pressure

$$P_z = 0.00256 K_z * K_d * G * V^2 * C_d$$

Both formulas have variable V, where V, the wind speed, is now grouped as four MRI categories: 10yr, 300yr, 700yr and 1700yr. AASHTO LRFD LTS-1 Specification. Table 3.1 determines which MRI value should be applied (by ADT and risk). This change is likely to make up for the removal of I_r . The current 100mph for all cases is no longer considered reasonable. On the other hand, the current AASHTO STD LTS-6 (2013) V map is equivalent to the 2015 LRFD 300yr wind map.

3.2 Maryland Wind Speed Study

AASHTO LRFD LTS-1 Specifications (2015) specified the Mean Recurrence Intervals (MRI) as determined in Table 3.1 shown below. The selection of the MRI accounts for the consequence of failure. As defined in the commentary of the AASHTO LRFD Specifications, a “typical” support could cross the travel way during a failure, thereby creating a hazard for travelers (MRI = 700 years). All supports that could cross lifeline travel ways are assigned a high-risk category to consider the consequence of failure (MRI=1700 years).

Table 3-1 Mean recurrence intervals (MRI) specified by AASHTO LRFD

Traffic Volume	Risk Category		
	Typical	High	Low
ADT ≤ 100	300	1700	300
100 < ADT ≤ 1000	700	1700	300
1000 < ADT ≤ 10000	700	1700	300
ADT > 10000	1700	1700	300
Typical: Failure could cross travelway			
High: Support failure could stop a lifeline travelway			
Low: Support failure could not cross travelway			
Roadside sign supports: use 10-yr MRI, see Figure 3.8-4.			

As far as the roadside structure design is concerned, Maryland can be divided into three regions: the Eastern Shore, the Appalachia Mountain, and the Baltimore-Washington corridor. In general, Average Daily Traffic (ADT) in the Eastern Shore and Appalachia Mountain regions can be considered $1000 < ADT \leq 10,000$ and structures can be designed as “typical” supports (MRI = 700 years), while the Baltimore-Washington corridor should be considered $ADT > 10,000$ where travel ways are assigned a high-risk category so that the consequence of failure (MRI=1700 years) is considered.

The research team plotted 14 counties in the eastern part of Maryland (the Baltimore-Washington corridor and Eastern Shore) and, based on the MDOT SHA 2017 Traffic Volume Maps, drew lines that would define the ADT of their routes. In Figure 3.1, the red line indicates that the route had an AADT (Annual Average Daily Traffic) larger than 10,000 (based on 2017 data). The primary results are close to what the research team originally assumed. For the Eastern Shore area, the most routes with an AADT larger than 10,000 were always the primary routes (inter-state, part of the US or MD route). For the counties that have large cities (i.e. Baltimore county and Anne Arundel county), the AADTs of primary routes were always higher than 10,000 with the maximum reaching 227,000.

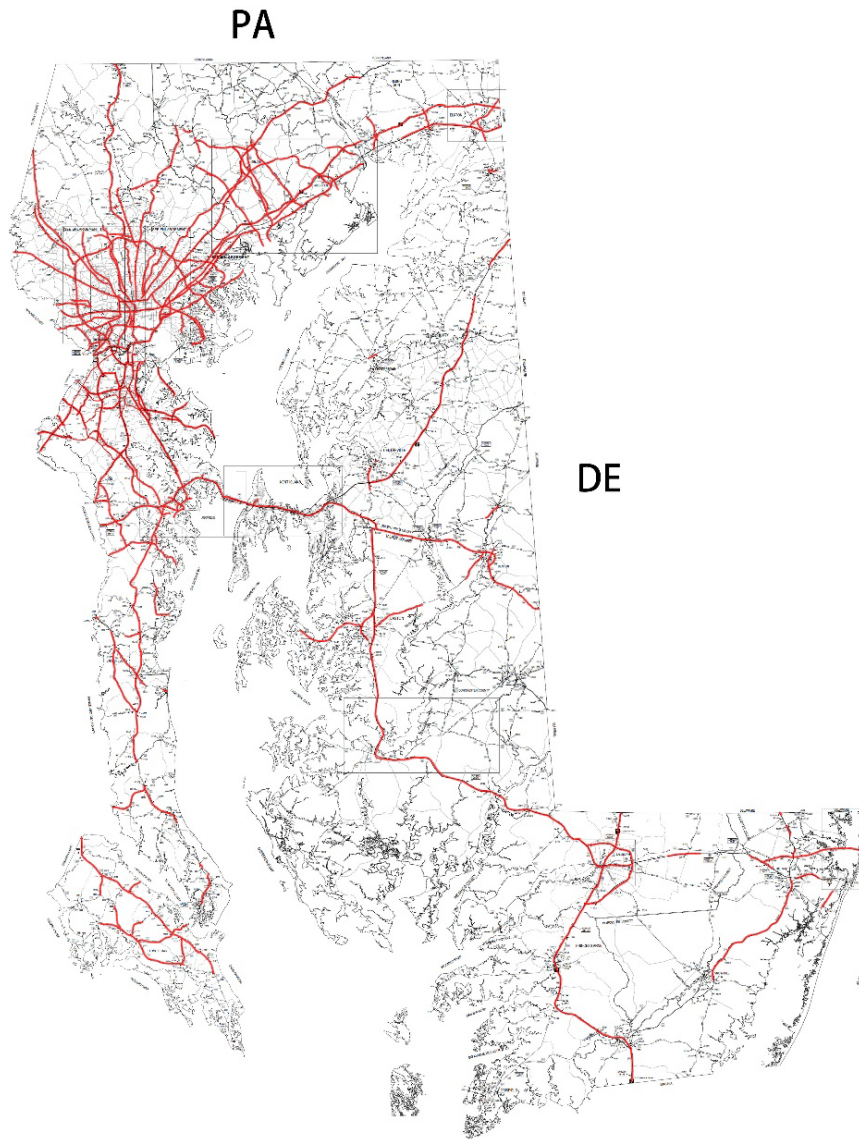


Figure 3-1 Maryland Routes with AADT Counts

3.3 Wind Load Pressure for Wind Load Analysis

This section is to find out Maryland's adoption of wind speeds and historical wind loads.

1. Wind speed adoption for wind load analysis:
 The wind speed adopted for STD LTS-6 is 100mph, while it is 120mph for LRFD LTS-1.
 Their wind load pressure can be calculated based on:
 - a) 2013 STD LTS-6 Specifications wind pressure

$$P_z = 0.00256 K_z * G * V^2 * I_r * C_d$$

b) 2015 LRFD LTS-1 Specifications wind pressure

$$P_z = 0.00256 K_z * K_d * G * V^2 * C_d$$

When given the following parameters:

$K_z = 0.87$ (for 2013) and 0.84 (for the 2015)

$K_d = 0.85$ (signal and sign support structures) for the 2015 LRFD Specifications only

$G = 1.14$ for both LRFD Specifications

$C_d = 1.20$ for both LRFD Specifications

$I_r = 1.00$ for the 2013 LRFD Specifications only

The wind pressure is tabulated here:

Specification 2015 LRFD	2013 ASD	2015 LRFD
Wind speed (V)	100mph	120mph
Pz	30.47psf	36.01psf

2. Wind load application on structures

STD LTS-6: Only the basic load (BL) on one arm plane is considered (Page 3-21 of the AASHTO STD LTS-6)

Load Case	Normal component	Transverse component
1	1.0 BL	0.2 BL
2	0.6 BL	0.3 BL

LRFD LTS-1: Two basic load are considered: (Page 3-24 of the AASHTO LRFD LTS-1) (BLn) on one arm plane / (BLt) on the arm plane spaced at 90 degree

Load Case	Normal component	Transverse component
1	1.0 BLn	0
2	0	1.0 BLt
3	0.75 BLn	0.75BLt

Chapter 4 Calibration of LRFD Designs by Adopting Fatigue Design

4.1 Fatigue Design Defined in the AASHTO STD LTS-6 & LRFD LTS-1 Specifications

Fatigue design was first covered in the AASHTO STD LTS-5 as load combination four (LC4). Fatigue resistance was then modified to the current form in both the AASHTO STD LTS-6 and LRFD LTS-1 Specifications. Section 11 in STD LTS-6 is conceptually identical to the LRFD LTS-1 specifications. Maryland is on schedule to adopt the fatigue design for all structural supports of highway signs, luminaires, and traffic signals. The main item is the adoption of fatigue design in all phases. Fatigue design is associated with fatigue details as shown in the AASHTO LRFD LTS-1 Table 11.9.3.1-1. Details of Maryland signal poles which need to be modified in order to increase the fatigue resistance have been identified and recommended by the research team. The recommended modifications of the current design on the signal poles include:

- a) Groove welds for arm connections
- b) Groove welds for pole connections
- c) Adopting AASHTO build-up box type for arm connections
- d) 6-bolt patterns for both arm and pole connections

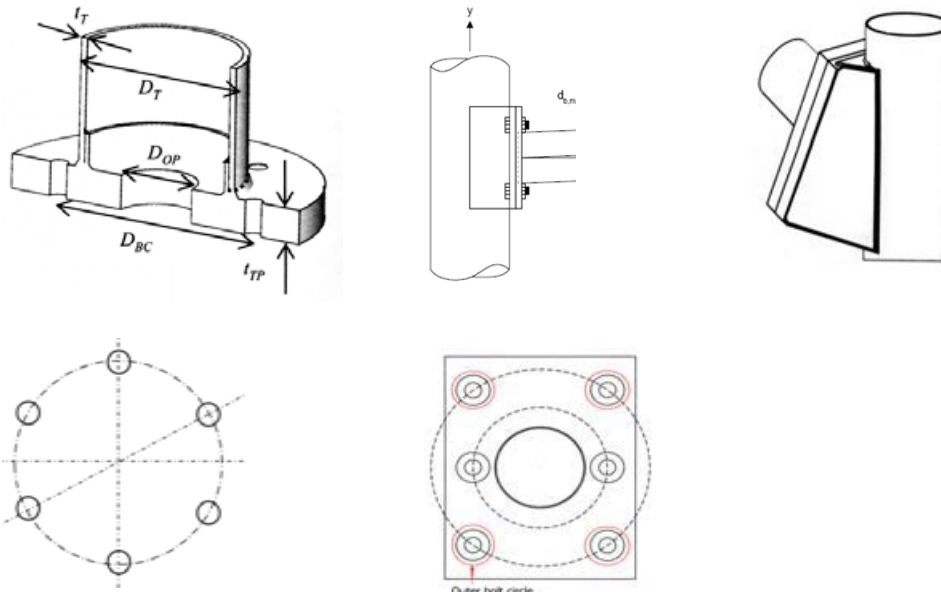


Figure 4.1 – Fatigue Design Details

Similar recommendations and their impacts of modifying details on sign structural supports were also studied in this task. One example is the study of tube-to-tube connections between main chords and bracings for sign structures, i.e. choosing between pipe-to-pipe vs gusset-plate weld connections for box truss sign structures.

4.2 Comparison of the AASHTO STD LTS-6 & LRFD LTS-1 Specifications.

Difference between the two codes for multi-arm piles

STD LTS-6 only calculates the load according to the sign on the most critical arm.

LRFD LTS-1 needs to take the sign configuration of both arms into consideration.

For 90-degree and 180-degree-spaced multiple arms

Bending and shear force should take the component into consideration.

Torque is only from the most critical arm because the two codes write: “When vertical supports have more than two arms, and the arms are mounted opposite or at diverging angles from one balanced or only to the arm that results in the largest torsional load effects when unbalanced.”

Fatigue load for both codes

a) Galloping

Fatigue load due to galloping will only be applied to one arm

Both Codes give the following description:

“A pole with multiple horizontal cantilevered arms may be designed for galloping loads applied separately to each individual arm and need not consider galloping simultaneously occurring on multiple arms.”

For sign structures with box trusses, the galloping can be ignored

“The Owner may choose to exclude galloping loads for the fatigue design of overhead cantilevered sign support structures with quadric-chord (i.e., four-chord) horizontal trusses”

b) Natural wind gust

45-degree components need to be considered for a 90-degree spaced multiple arm

STD LTS-6:

“Design for natural wind gusts shall consider the application of wind gusts for any direction.”

LRFD LTS-1:

“For a pole with multiple arms, such as two perpendicular arms, the critical direction for the natural wind gust is usually not normal to either arm. The design natural wind gust pressure range should be applied to the exposed surface areas seen in an elevation view orientated perpendicular to the assumed wind gust direction.”

c) Truck induced gust

Wind pressure range is applied to the most critical 12' arm length

Both codes:

“This pressure range shall be applied along any 12-ft length to create the maximum stress range, excluding any portion of the structures not located directly above a traffic lane.”

4.3 Fatigue Detail Checks

For this project, the sign structures were checked based on the wind load for the strength design and fatigue loads for fatigue detailed locations.

4.3.1 Cantilever sign structures:

(1) Fillet tube-to-tube connections for the chords

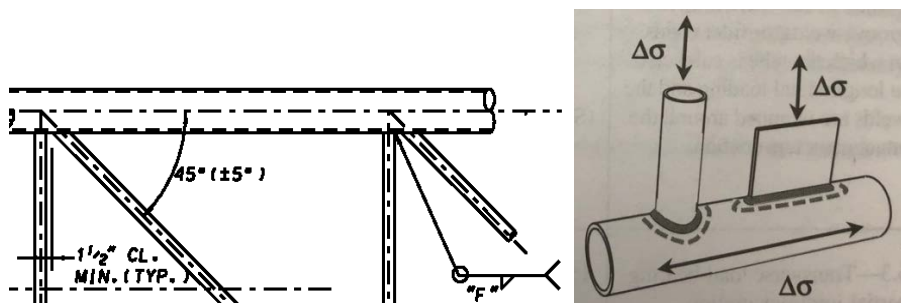


Figure 4.2 – AASHTO Fatigue Detail 5.5

This fatigue design follows Detail 5.5

In a branching member with respect to the stress in the branching member:

$$(\Delta F)_{TH} = 1.2 \text{ksi}; \text{ when } \frac{r}{t} \leq 24 \text{ for the chord member}$$

$$(\Delta F)_{TH} = 1.2 \times \left(\frac{24}{\frac{r}{t}} \right)^{0.7} \text{ksi}; \text{ when } \frac{r}{t} \geq 24 \text{ for the chord member}$$

In a chord member with respect to the stress in the chord member:

$$(\Delta F)_{TH} = 4.5 \text{ksi}$$

(2) Pole to truss connection (Two types)

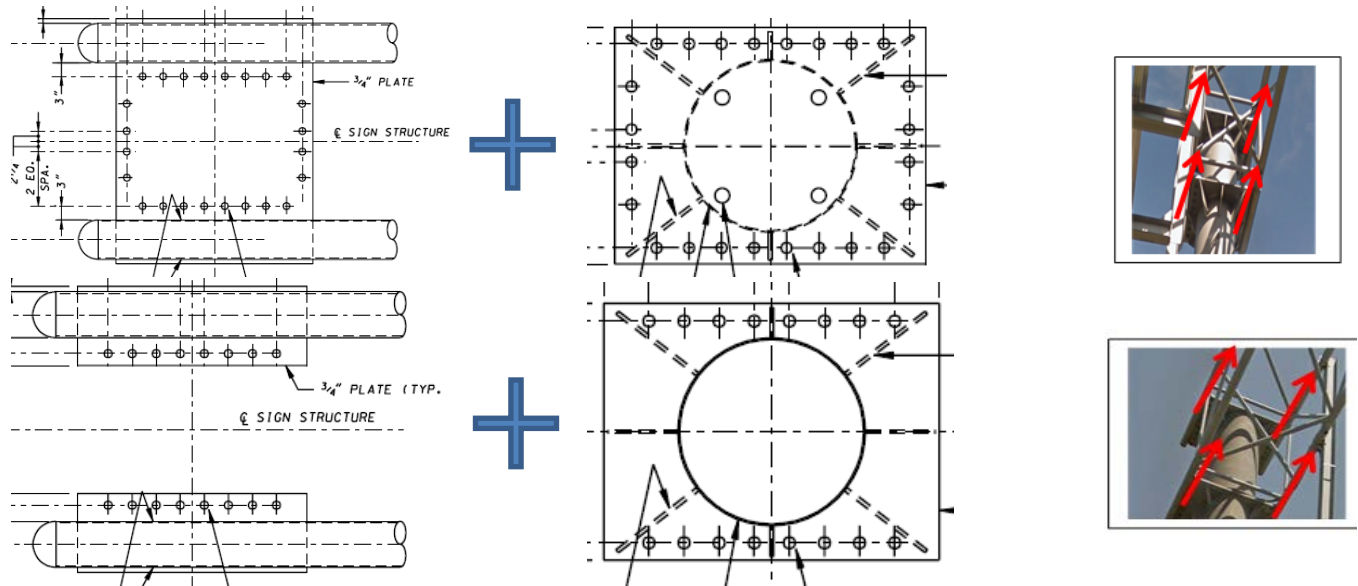


Figure 4.3 – Cantilever Pole to Truss Connection

By investigation, for the cantilever pole to truss connection, it is found that the main forces are along the horizontal chords of the box truss, so it is in action on the top and bottom stiffened plates. It is similar to Detail 5.7 of the AASHTO Specifications. As per AASHTO Article 5.6.7, the connected parts are in infinite life.

(a) for the chord to plate connection:

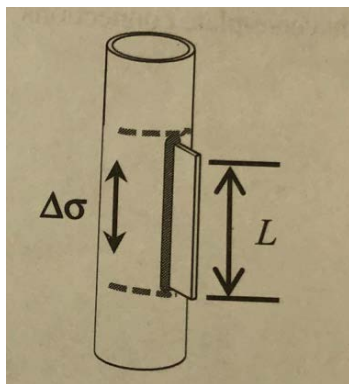


Figure 4.4 – AASHTO Fatigue Detail 6.1

This fatigue design follows Detail 6.1 where the main member is subjected to the longitudinal loading:

$$\begin{aligned}
 L < 2in, (\Delta F)_{TH} &= 10ksi \\
 2in \leq L \leq 12t \text{ and } 4in, (\Delta F)_{TH} &= 7ksi \\
 L > 12t \text{ or } 4in \text{ when } t \leq 1in, (\Delta F)_{TH} &= 4.5ksi
 \end{aligned}$$

(b) for the pole to plate connection:

This design follows the Details 6.2 and 6.3.

Detail 6.2 (stress in tube):

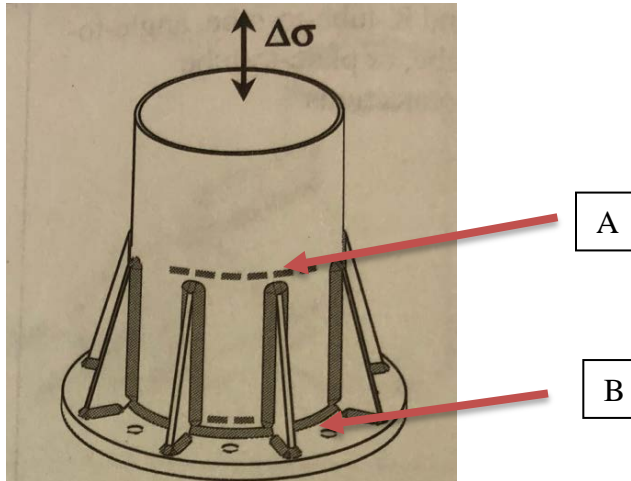


Figure 4.5 – AASHTO Fatigue Detail 6.2

There are two locations to be checked:

A: $K_I \leq 5.5: (\Delta F)_{TH} = 7.0ksi$

B: Following detail 5.4

$$\begin{aligned} K_I \leq 4.0: (\Delta F)_{TH} &= 7.0ksi \\ 4.0 < K_I \leq 6.5: (\Delta F)_{TH} &= 4.5ksi \\ 6.5 < K_I \leq 7.7: (\Delta F)_{TH} &= 2.6ksi \end{aligned}$$

Detail 6.3 (stress in stiffeners):

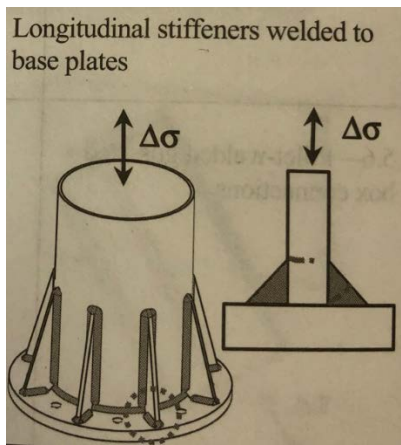


Figure 4.6 – AASHTO Fatigue Detail 6.3

$$(\Delta F)_{TH} = 10.0ksi$$

(3) Pole to base connection:

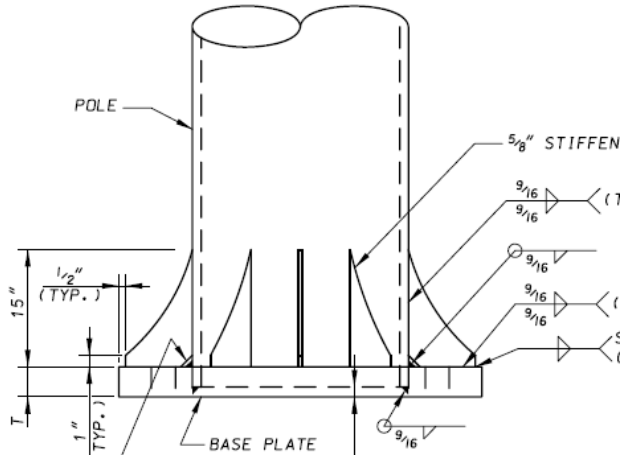


Figure 4.7 – Cantilever Pole to Base Connection

This design follows the Details 6.2 and 6.3

Detail 6.2 (stress in tube):

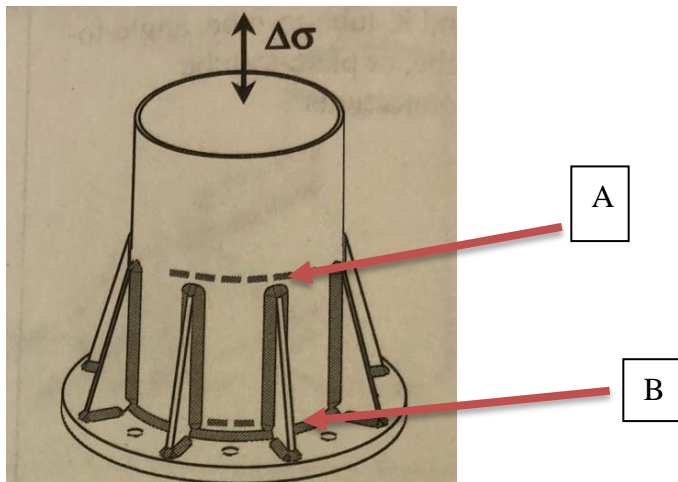


Figure 4.8 – AASHTO Fatigue Detail 6.2

There are two locations to be checked:

A: $K_I \leq 5.5: (\Delta F)_{TH} = 7.0ksi$

B: Following detail 5.4

$$K_I \leq 4.0: (\Delta F)_{TH} = 7.0ksi$$

$$4.0 < K_I \leq 6.5: (\Delta F)_{TH} = 4.5ksi$$

$$6.5 < K_I \leq 7.7: (\Delta F)_{TH} = 2.6ksi$$

Detail 6.3 (stress in stiffeners):

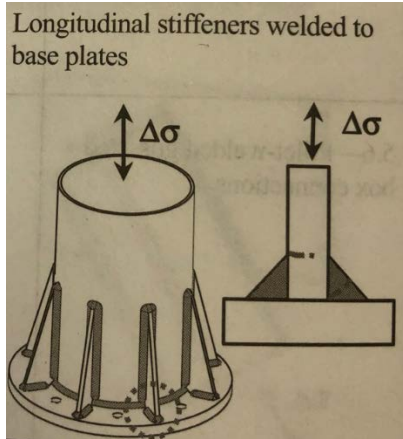


Figure 4.9 – AASHTO Fatigue Detail 6.3

$$(\Delta F)_{TH} = 10.0ksi$$

4.3.2 Overhead sign structures:

1) Fillet weld and tube-to-tube connections for the chords

The check here is the same as the check for cantilevered sign structures in Section 4.3.1 (1)

2) Flange connection for the chords

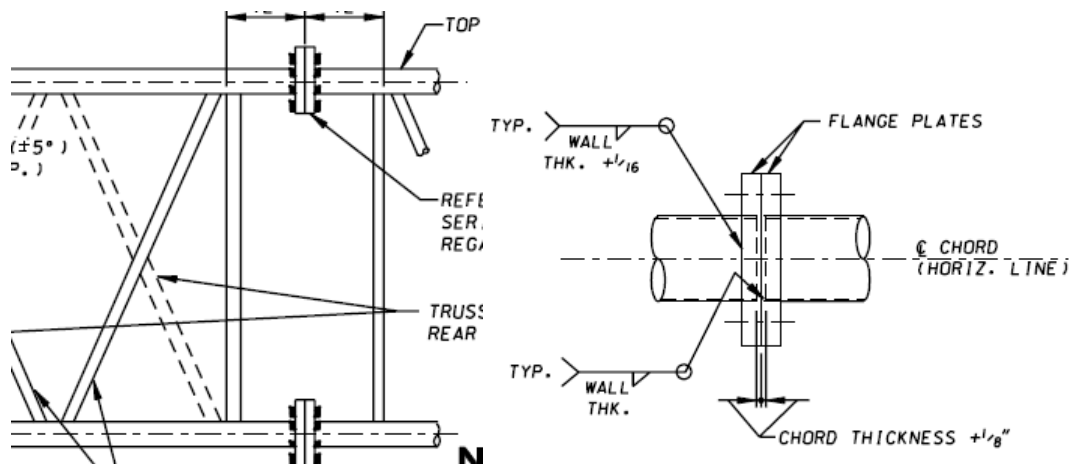


Figure 4.10 – Overhead Tube-to-tube Connection

Detail 5.4

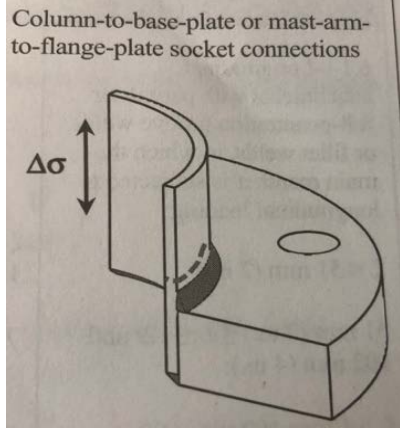


Figure 4.11 – AASHTO Fatigue Detail 5.4

$$K_I \leq 4.0: (\Delta F)_{TH} = 7.0ksi$$

$$4.0 < K_I \leq 6.5: (\Delta F)_{TH} = 4.5ksi$$

$$6.5 < K_I \leq 7.7: (\Delta F)_{TH} = 2.6ksi$$

3) Pole-chord connection

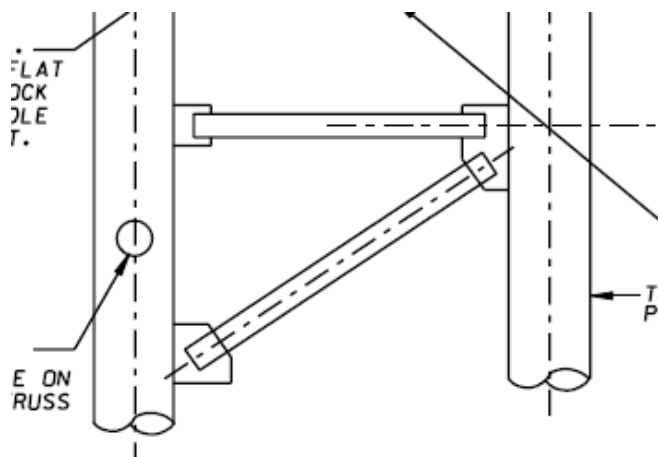


Figure 4.12 – AASHTO Pole-Chord Connection

Detail 5.3 and Detail 5.5:

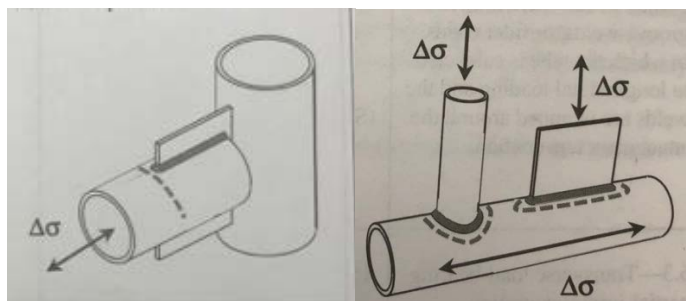


Figure 4.13 – AASHTO Fatigue Details 5.3 and 5.5

Detail 5.3:

$$(\Delta F)_{TH} = 2.6 \text{ksi}$$

Detail 5.5:

In a branching member with respect to the stress in the branching member:

$$(\Delta F)_{TH} = 1.2 \text{ksi}$$

In a main member with respect to the stress in the main member (Column):

$$(\Delta F)_{TH} = 1.0 \text{ksi}; \text{ when } \frac{r}{t} \leq 24 \text{ for the chord member}$$

$$(\Delta F)_{TH} = 1.0 \times \left(\frac{24}{\frac{r}{t}} \right)^{0.7} \text{ksi}; \text{ when } \frac{r}{t} \geq 24 \text{ for the chord member}$$

4) Pole-base connection

Detail 5.4

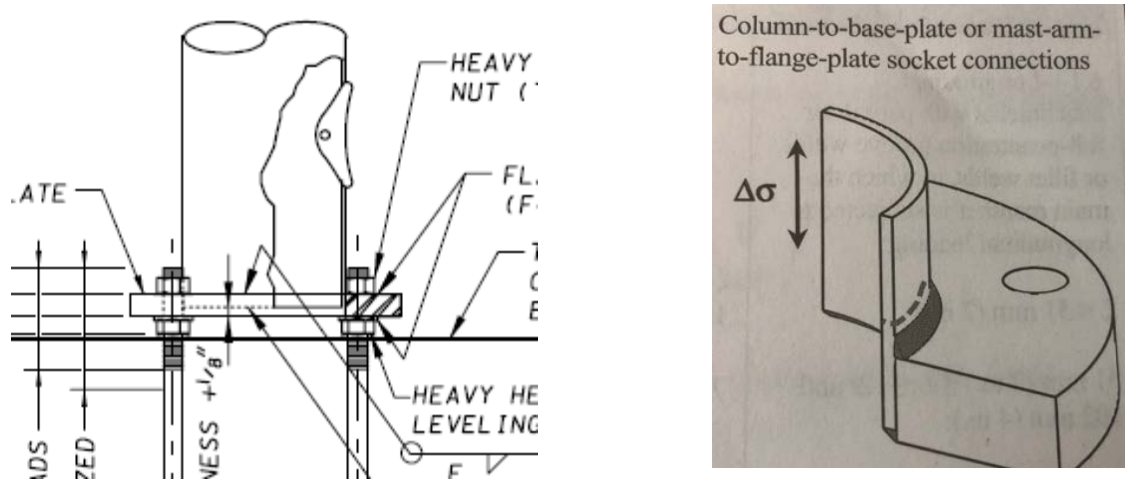


Figure 4.14 – Pole Base Connection and AASHTO Fatigue Detail 5.4

$$K_I \leq 4.0: (\Delta F)_{TH} = 7.0 \text{ksi}$$

$$4.0 < K_I \leq 6.5: (\Delta F)_{TH} = 4.5 \text{ksi}$$

$$6.5 < K_I \leq 7.7: (\Delta F)_{TH} = 2.6 \text{ksi}$$

Chapter 5 LRFD Foundation Design

Fatigue design would not affect the support foundation, but wind load would. In the AASHTO LRFD LTS-1 Extreme I, Service I and II Limit States have Wind Load (W) involved and should be considered in foundation designs. In the Maryland Book of Standard for Highways, Incidental Structures and Traffic Control Applications, three types of foundations are covered for their own respective types of structures:

- a) Shaft foundation for signal poles
- b) Shaft foundation with wing walls for cantilever sign structures
- c) Mat found with pedestal for overhead

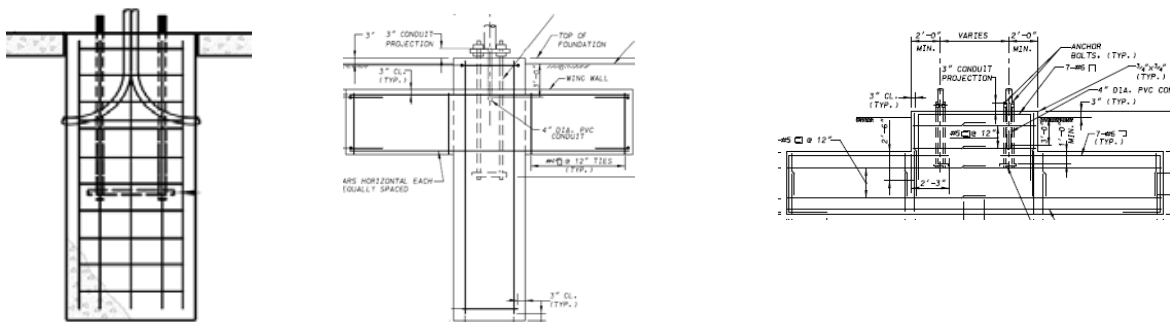


Figure 5-1 Maryland Standard Types for Signal Poles and Sign Structures

5.1 Shaft foundation for signal poles

Different winds and load combinations covered by the AASHTO STD LTS-6 and LRFD LTS-1 may greatly affect the foundation designs. A preliminary study was completed by the research team on shaft foundation designs on cohesive and cohesionless soils by the AASHTO LRFD LTS-1. Slight modifications may be needed in order to cover the LRFD design on a Maryland standard 50' - to 75' -arm sign structures with setup of one road sign, two left-turn signs, and five signal heads. All the standard foundation designs covered in MD801.1 for signal structures, MD801.07 and MD801.19-23 for cantilever and overhead sign structures, respectively and may need modification. Representative samples of these three types are studied in this task.

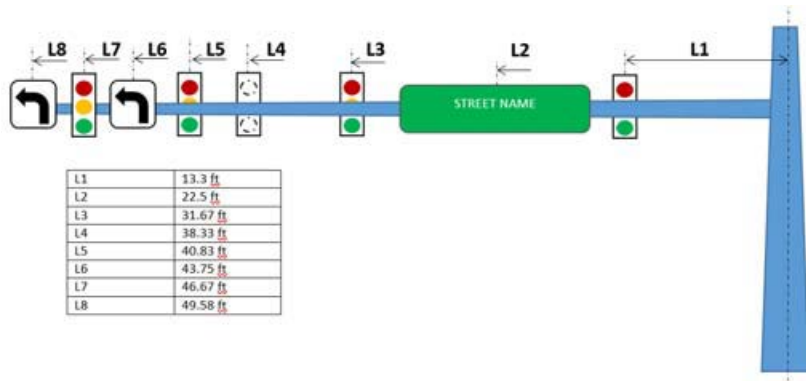


Figure 5-2 Maryland Standard Signal Poles

A detailed discussion of shaft foundation design based on Broms method is covered in Chapter 7— an investigation of Maryland Signal Pole Foundation, in the 2017 report - *Fatigue Resistant Design Criteria for MD SHA Cantilevered Mast Arm Signal Structures*. This chapter furthered the research and the following examples demonstrate a case study of Maryland signal pole foundation, and extended designs for all cases.

Step 1- Length and diameter check

In the foundation check, the 75’ mast arm signal structure is selected as the check model since the base reaction force and moment are the largest among Maryland mast arm signal structures. In the State of Maryland soil conditions are assumed as general properties for the shaft current design check including diameter, embedment length, and longitudinal reinforcement. The properties of cohesive and cohesionless soils are listed in the table below.

Table 5-1 Properties of cohesive and cohesionless soils

Soil Type	Soil Category	Shear Strength	Unit Weight	Internal Friction Angle
Cohesive	Stiff clay	2.16 ksf	N/A	N/A
Cohesionless	Clean gravel-sand	N/A	0.12 kcf	30

The external load is the base reaction of the 75’ mast arm signal structure obtained from SABRE software. Factored ground moment is 174.7 kip-ft and Factored ground shear is 5.35 kips.

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

$$D = 4 \text{ ft}; H = \frac{M_F}{V_F}; q = \frac{V_F}{9cD}$$

where:

Cohesive soil: required embedment length (L) = 9.3 ft < 10 ft (current design length)

Cohesion soil: required embedment length (L) = 8.9 ft < 10 ft (current design length)

Step 2- Longitudinal reinforcement check

For cohesive soil, maximal (M_u) = $V_F(H + 1.5D + 0.5q)$ is located at $(1.5D + q)$ below the groundline.

$$M_u = 207 \text{ kip-ft}$$

For cohesionless soil, maximal (M_u) = $V_F(H + 0.54 \sqrt{\frac{V_F}{\gamma DK_p}})$ is located at $0.82 \sqrt{\frac{V_F}{\gamma DK_p}}$ below the groundline.

$$M_u = 183.1 \text{ kip-ft}$$

Current longitudinal reinforcement design (75'):

$$f_c' = 4 \text{ ksi};$$

$$f_y = 60 \text{ ksi}$$

$$\text{Number of rebars} = 16;$$

$$\text{Size of rebar} = \#10$$

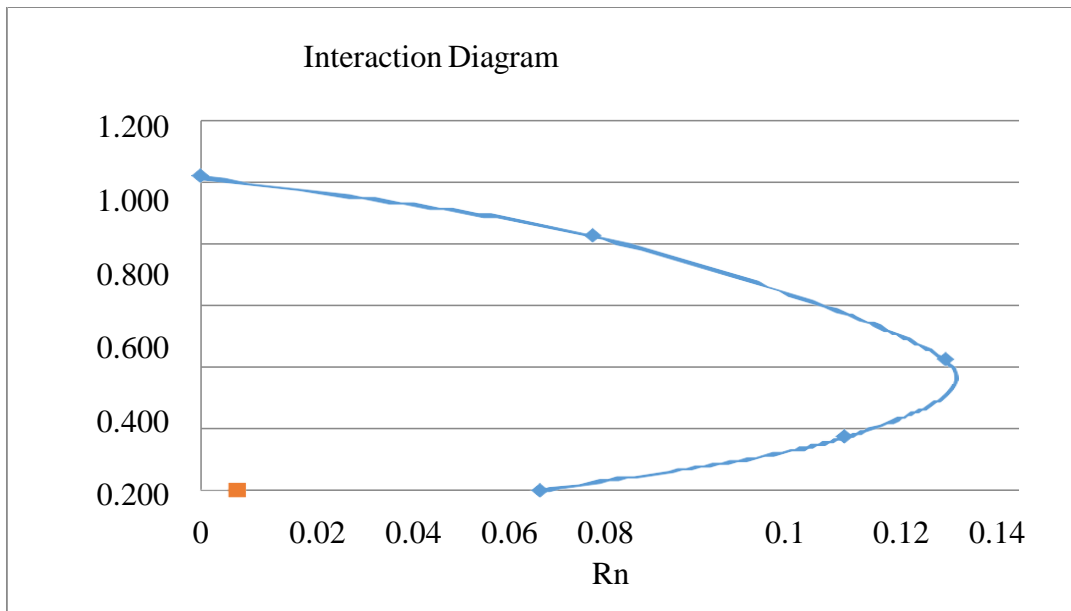


Figure 5-3 Shaft Rebar P-M interaction diagram

$$K_n = \frac{P_n}{f_c' A_g}$$

$$R_n = \frac{M_n}{f_c' A_g h}$$

Step 3- Torsion check

For cohesive soil, the torsional resistance is provided by the adhesion of the soil. According to the Illinois DOT design method, the torsional resistance (T_r) is mainly from the soil from the lateral side and the resistance at the bottom of the shaft is neglected. Base torsion is 131.8 (kip-ft) obtained from SABRE software.

$$T_r = 0.5 \cdot D \cdot A \cdot \alpha \cdot S_u$$

$$T_r = 298.57 \text{ kip-ft} > 131.8 \text{ kip-ft}$$

Where:

D= Diameter of shaft (ft)

L= Length of shaft (ft)

A= Lateral surface area of shaft (ft²)

S_u= Average soil undrained shear strength (ksf)

α= Adhesion factor = 0.55

For cohesionless soil, the torsional resistance (T_r) is provided by friction between the soil and shaft. Based on the Beta method adopted by the Illinois DOT, torsional resistance is associated with the unit weight of soil, and depth of shaft. Resistance at the bottom of the shaft is neglected in the Illinois DOT design method. Base torsion is 131.8 (kip-ft) obtained from SABRE software.

$$T_r = 0.5 \cdot D \cdot A \cdot \beta \cdot \sigma_{vz}$$
$$\beta = 1.5 - 0.135\sqrt{h}, 1.2 \geq \beta \geq 0.25$$

$$T_r = 161.8 \text{ (kip-ft)} > 131.8 \text{ (kip-ft)}$$

Where:

D= Diameter of shaft (ft)

L= Length of shaft (ft)

A= lateral surface area of shaft (ft)

h= depth from ground surface to the mid-depth of soil layer (ft)

σ_{vz}= effective vertical soil pressure to the mid depth of soil layer (ksf)

Step 4 – Summary of the case study

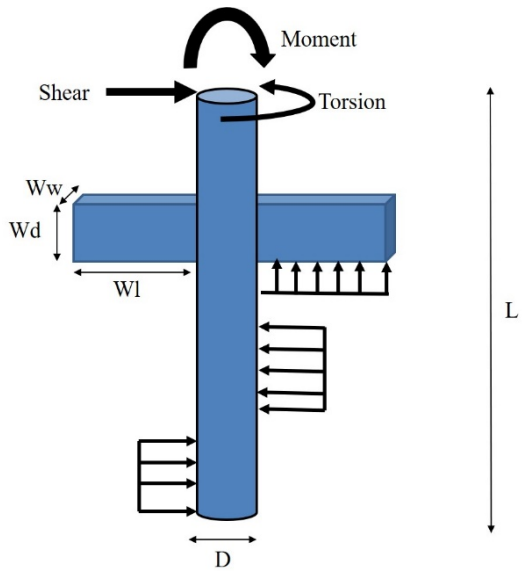
For the current design check based on the assumed soil condition, shaft embedment lengths in both of the cohesive and cohesionless soils are longer than the required length calculated by Brom's method. For the longitudinal reinforcement, the maximal moment occurring on the shaft is under the moment interaction curve which means the reinforcement design is sufficient to resist external ground moment. Torsion resistances of the shaft based on the Illinois DOT design method are higher than the maximal base torsion obtained by SABRE software either in the cohesive or cohesionless soil.

Table 5-2 Case Study comparison

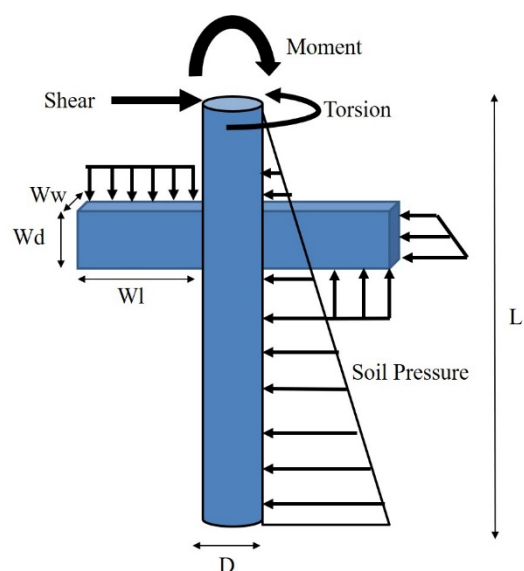
Arm Length (ft)	Soil Type	Design length (ft)	Required length (ft)	Torsion (kip-ft)	Torsion resistance (kip-ft)	Reinforcement check
50	Cohesive	10	7.76	83	298.57	OK
	Cohesionless	10	8.92	83	161.82	OK
60	Cohesive	10	8.98	95.6	298.57	OK
	Cohesionless	10	8.24	95.6	161.82	OK
70	Cohesive	10	8.78	118.2	298.57	OK
	Cohesionless	10	9.35	118.2	161.82	OK
75	Cohesive	10	9.33	131.8	298.57	OK
	Cohesionless	10	8.89	131.8	161.82	OK

5.2 Shaft foundation with wing walls for cantilever sign structures

For cantilever sign structures, the foundation is designed as a short pile with wing walls. The main function of a wing wall is to enlarge the contact area with soil to increase foundation capacity. Like the regular shaft foundation mentioned in Section 5.1, required length, reinforcement and torsion resistance are three check steps to evaluate overall foundation capacity in either cohesive or cohesionless soil. In the AASHTO LRFD, the shaft foundation verification by the Broms' method is required. However, a shaft with wing wall is not a typical shaft foundation case. In order to adopt Broms' method for the required length check, resistance created by wing walls is deducted from the applied load, including ground shear force and moment. The soil resistance of a shaft with wing walls in cohesive and cohesionless soil is demonstrated in Figure 5.4. The required length of shaft could be verified by the remaining ground load through the regular Broms' method. In the cohesive soil case, one bottom resistance is considered and the resistance at the other top side is neglected since soil cohesion from ground to 1.5D below is not considered in the Broms' method. In the cohesionless soil case, top, bottom, and lateral resistances caused by soil pressure are deducted from the ground load. The procedure of the longitudinal reinforcement check is similar with the shaft foundation for signal poles and the maximal moment from soil reaction is yielded by Broms' solution. For torsional capacity, the wing wall increases the lateral contact area against horizontal torque from the shaft top. The integral torsional capacity of the foundation is the combination of the shaft and wing walls. The torsional capacity could be estimated by unit resistance and contact area. The parameters for unit resistance to the estimation are similar with the regular shaft mentioned in Section 5.1. As long as the integral torsional capacity is larger than applied torque, the torsional capacity check is assumed to have passed. The sign structure CF45 (23X16) and CF45 (28X16) are selected as critical cases for foundation analysis and their base reactions are listed in Table 5.3. Their corresponding foundations are CF21 and CF23 and the results in either cohesive or cohesionless soil are listed in Table 5.4.



Shaft foundation w/wing wall in cohesive soil



Shaft foundation w/wing wall in cohesionless soil

Figure 5.4 Soil resistance on shaft foundation w/wing wall

Table 5-3 Base reaction of sign structures

Type	Base Reaction (Kips, Kip-ft)					
	Fx	Fy	Fz	Mx	My	Mz
C-45 (23X16)	0.079	15.21	-19.31	-620.12	592.3	245.34
C-45 (28X16)	0.017	16.86	-23.94	-770.73	695.22	271.20

Table 5-4 Case Study comparison

Type	Soil	Embedded Length (ft)	Rebar	Torsional Capacity (kip-ft)
CF-21	Cohesive	11 / 17 OK	OK	592.2 / 663.2 OK
	Cohesionless	13.3 / 17 OK	OK	592.2 / 727.4 OK
CF-23	Cohesive	11.7 / 18 OK	OK	695.23 / 765.2 OK
	Cohesionless	14.4 / 18 OK	OK	695.23 / 734.6 OK

5.3 Mat found with pedestal for overhead

Foundations of the overhead span sign structures were examined for adequacy against bearing capacity failure, sliding, overturning, and flexural capacity. The calculations were done in MS Excel and the spreadsheet is included in the package. Some of the spreadsheets are also presented in Appendix B.

To ensure adequate capacities of the overhead sign structure foundations against corresponding limit states, base reactions were recorded in SABRE for each individual case with different span lengths and sign areas. Six base reactions were obtained from SABRE to act as applied forces on the spread footing. Namely, the axial forces (F_x , F_y and F_z) and moments (M_x , M_y and M_z) in the x, y, and z-direction. All the applied forces provided by SABRE are factored loads and each case is divided into two situations of whether the soil condition is cohesive or cohesionless. Although there are two signposts on each footing, it is assumed that the resultant forces go through the center of the foundation. Figure 5.5 shows the applied forces on the spread footing. Note that M_y is treated as a torsional force, hence, its resultant is translated into shear stress that is factored in during calculation against sliding of the spread footing.

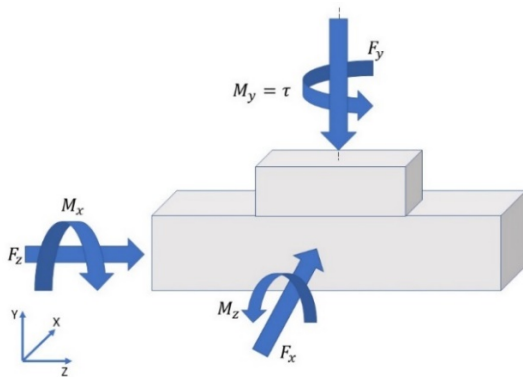


Figure 5-4 Applied Forces on the Spread Footing

Most of the calculations were carried out based on the 2012 AASHTO LRFD Bridge Design Specifications, except for reinforcing steels associated with flexural capacity, where ACI-318-14 was used. Table 5.5 summarizes the sections from the specifications that were used for the calculations. There is no strength reduction factor for the calculation about overturning as AASHTO adopted LRFD design specifications. Under LRFD, the overturning stability used in the ASD method was replaced by the eccentricity limit. According to AASHTO LRFD design specifications, the eccentricity must not exceed one-third of the corresponding footing dimension, B or L, for footings on soils (*AASHTO LRFD Bridge Design Specifications, Customary U.S. Units*. Washington, DC: American Association of State Highway and Transportation Officials, 2012.).

Results are tabulated through Tables 5.5.1-5.5.3. Three individual cases with span length equal 79, 102, and 140 feet were examined. Both sliding and eccentricity limits were investigated in the x- and z-directions, and the flexural capacity was checked by calculating the moment capacity of the footing given the number of rebar present. Overall, all spread footings pass the

check with adequate capacity. It is worth noting that the calculations are carried out conservatively. The major assumptions are:

1. Choose the smallest footing dimensions in each category.
2. The passive earth pressure was neglected during the sliding calculation.
3. The weight of the soil was ignored in calculations on the eccentricity (overturning) limit.
4. Only the top and bottom rebars were considered for flexural capacity.

All the above-mentioned assumptions may reduce the capacity of the footing in each corresponding strength limit state. Therefore, the results are conservative. Nevertheless, these assumptions can be easily modified in the Excel spreadsheet if necessary.

Based on the LRFD specifications, the capacity of the structure shall exceed the factored loads. Hence, the ratio between the capacity and the factored loads must be greater than or equal to one. According to calculation results summarized in Tables 5.5.1-5.5.3, the footings pass the test for each strength limit state, in which the lowest ratio occurs on eccentricity (overturning) in the x-direction. The results are reasonable since the applied moment in the x-direction always presents the greatest demand on the foundation structure.

Table 5-5 AASHTO Code Check to Mat Foundation

Limit States	Code Section	Strength Reduction Factor, ϕ
Bearing Capacity	2012 AASHTO 10.6.3.1.2, 2012 AASHTO 10.5.5.2.2	$\phi_b = 0.45$
Sliding	2012 AASHTO 10.6.3.4	$\phi_\tau = 0.85$ ($c' > 0$), $\phi_\tau = 0.90$ ($c' = 0$), $\phi_{ep} = 0.50$
Overturning	2012 AASHTO 10.6.3.3, 2012 AASHTO Figure 11.6.3.2-1	N/A
Flexural Capacity	ACI 318-14 22.3	$\phi = 0.9$

Table 5-6 Code Check for Example MD Overhead Sign Structure Mat with Span Length = 79'

Footing Mark	F_x (kips)	F_y (kips)	F_z (kips)	M_x (kip·ft)	M_y (kip·ft)	M_z (kip·ft)	
OF-620	-1.1	8.6	-20.0	-599.2	-40.6	11.3	
Soil Type	Q_{ult} (kips)	Sliding (X) (kips)	Sliding (Z) (kips)	Overturning (X) (Eccentricity) (ft)	Overturning (Z) (Eccentricity) (ft)	Flexural (Reinforcement) (kip·ft)	
$C' > 0$	Allowable	2671.5	58.0	58.0	6.7	2.0	1252.413116
	Actual	127.686	3.9	22.8	3.4	0.2	397.7
	Ratio	20.9	15.0	2.5	2.0	11.6	3.1
	Status	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.
$C' = 0$	Allowable	801.9	61.4	61.4	6.7	2.0	1252.4
	Actual	127.686	3.9	22.8	3.4	0.2	397.7
	Ratio	6.3	15.9	2.7	2.0	11.6	3.1
	Status	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.

Table 5-7 Code Check for Example MD Overhead Sign Structure Mat with Span Length = 102'

Footing Mark		F _x (kips)	F _y (kips)	F _z (kips)	M _x (kip·ft)	M _y (kip·ft)	M _z (kip·ft)
OF-620		-2.2	9.4	-15.1	-397.7	-37.3	20.4
Soil Type		Q _{ult} (kips)	Sliding (X) (kips)	Sliding (Z) (kips)	Overturning (X) (Eccentricity) (ft)	Overturning (Z) (Eccentricity) (ft)	Flexural (Reinforcement) (kip·ft)
C' > 0	Allowable	2671.5	58.0	58.0	6.7	2.0	1252.413116
	Actual	127.686	4.8	17.6	3.4	0.2	397.7
	Ratio	20.9	12.1	3.3	2.0	11.6	3.1
	Status	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.
C' = 0	Allowable	801.9	61.4	61.4	6.7	2.0	1252.4
	Actual	127.686	4.8	17.6	3.4	0.2	397.7
	Ratio	6.3	12.8	3.5	2.0	11.6	3.1
	Status	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.

Table 5-8 Code Check for Example MD Overhead Sign Structure Mat with Span Length = 140'

Footing Mark		F _x (kips)	F _y (kips)	F _z (kips)	M _x (kip·ft)	M _y (kip·ft)	M _z (kip·ft)
OF-622		-11.7	25.0	-25.9	-775.9	-56.6	114.3
Soil Type		Q _{ult} (kips)	Sliding (X) (kips)	Sliding (Z) (kips)	Overturning (X) (Eccentricity) (ft)	Overturning (Z) (Eccentricity) (ft)	Flexural (Reinforcement) (kip·ft)
C' > 0	Allowable	2902.8	62.4	62.4	7.3	2.0	1252.413116
	Actual	152.151	14.9	29.1	6.1	0.9	775.9
	Ratio	19.1	4.2	2.1	1.2	2.2	1.6
	Status	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.
C' = 0	Allowable	875.2	66.1	66.1	7.3	2.0	1252.4
	Actual	152.151	14.9	29.1	6.1	0.9	775.9
	Ratio	5.8	4.4	2.3	1.2	2.2	1.6
	Status	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.

Chapter 6 Comparison between the AASHTO STD LTS-6 and LRFD LTS-1 Specifications

6.1 Allowable stresses vs nominal strengths

In the AASHTO STD LTS-6 Specifications, allowable stresses are defined in (1) bending, (2) tension/compression, and (3) shear stresses. On the other hand, in the AASHTO LRFD LTS-1 Specifications, nominal strengths are defined in (1) flexure, (2) tension/compression, and (3) shear/torsion. Their expressions are drastically different. To demonstrate the differences, expressions for tube sections are listed below and their effects are studied.

- a) AASHTO STD LTS-6 Specifications
 - a1) Local buckling
 - a2) Allowable bending stress
 - a3) Tension/compression
 - a4) Shear
- b) AASHTO LRFD LTS-6 Specifications (for tube section only)
 - b1) Local buckling
 - b2) Nominal bending strength
 - b3) Tension/Compression Strength
 - b4) Shear/Torsion

6.2 Interaction equations

In the AASHTO STD LTS-6 Specifications, the check is based on the combined stress ratio, while in the AASHTO LRFD LTS-1 Specifications, the check is based on the combined force interaction equation. Their checking based on Strength I Limit State is calibrated for samples of Maryland Standards.

- a) AASHTO STD LTS-6 Combined Stress Ratio equation
 - a1) Vertical cantilever pole-type support
 - a2) Other members
- b) AASHTO LRFD LTS-1 Combined Force interaction equation

6.3 Fatigue Case Study

A complete list of the verification of the SABRE fatigue analysis using NCHRP Report 718 for HMLT (Examples 1 & 5), NCHRP Report 796 (Examples 2 & 9), and NJDOT (Examples 1 & 2) and separate STAAD analyses are in Appendix C of this report. This section lists out the summary of these four cases.

6.3.1 Fatigue case study 1:

This is a fatigue comparison of NCHRP Report 796 Example 2 for a traffic signal pole study. The sketch of the structure is shown in Figure 6.1.

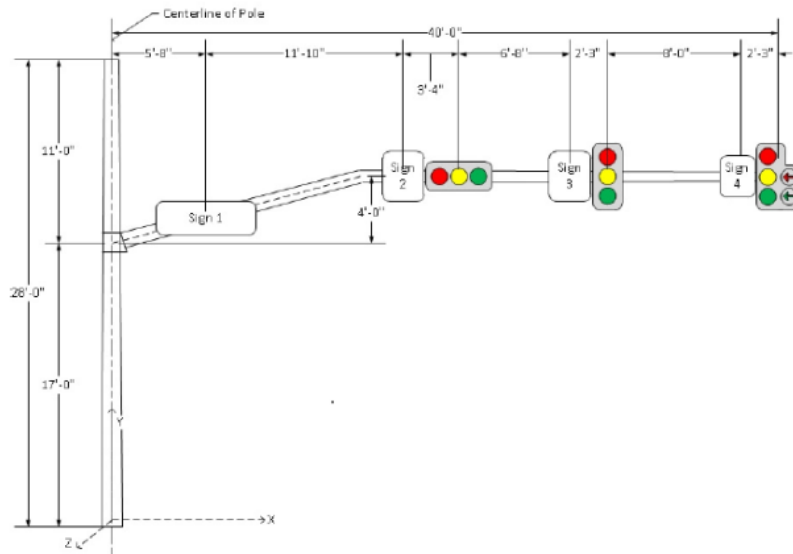


Figure 6-1 NCHRP Report 796 Example 2 for traffic signal pole

Table 6.1 shows wind strength and fatigue comparison results among NCHRP Report 796, STAAD Pro, and SABRE programs.

Table 6-1 Fatigue Comparison among NCHRP Report 796, STAAD Pro, and SABRE programs

NCHRP 796	EX-2	SABRE	NCHRP 796	Staad. Pro	Diff %
Wind strength	Joint Load (kips)	3.365	3.28	3.287	2.59%
	Bending Moment (kips-ft)	66.02	63.8	64.082	3.48%
	Torsion (kips-ft)	71.1	69.8	66.539	1.86%
Galloping	Joint Load (kips)	0.852	0.852	0.852	0.00%
	Bending Moment (kips-ft)	21.24	22.1	21.217	-3.89%
Natural Wind Gusts	Joint Load (kips)	0.782	0.78	0.758	0.26%
	Bending Moment (kips-ft)	14.74	15.054	14.428	-2.09%
	Torsion (kips-ft)	12.85	12.44	13.438	3.30%
Truck Induced Gusts	Joint Load (kips)	0.107	0.1	0.105	7.00%
	Bending Moment (kips-ft)	3.56	3.53	3.56	0.85%

6.3.2 Fatigue case study 2:

This is a fatigue comparison of NCHRP Report 796 Example 9 for cantilevered monotube support for a dynamic message signpost study. The sketch of the structure is shown in Figure 6.2.

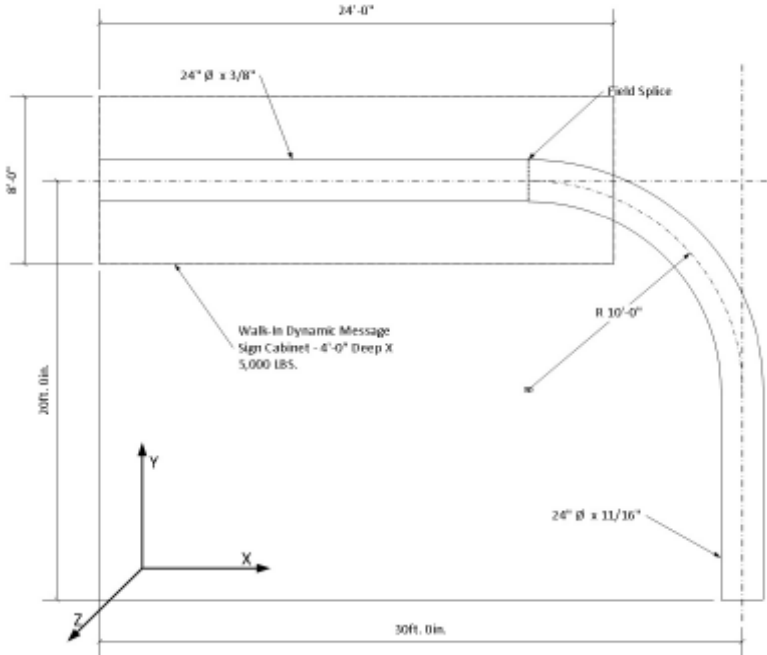


Figure 6-2 NCHRP Report 796 Example 9 for cantilevered monotube support for a dynamic message sign post

Table 6.2 shows wind strength and fatigue comparison results among NCHRP Report 796, STAAD Pro, and SABRE programs.

Table 6-2 Fatigue Comparison among NCHRP Report 796, STAAD Pro and SABRE programs

NCHRP 796	EX-9	SABRE	NCHRP 796	Staad. Pro	Diff %
Wind strength	Joint Load (kips)	8.157	8.833	8.849	-7.65%
	Bending Moment (k	157.93	148.21	158.918	6.56%
	Torsion (kips-ft)	135.72	138.63	135.753	-2.10%
Natrual Wind Gusts	Joint Load (kips)	1.72	1.491	1.497	15.36%
	Bending Moment (k	31.57	29.8	27.504	5.94%
Truck Induced Gusts	Joint Load (kips)	1.286	1.288	1.58	-0.16%
	Bending Moment (k	30.77	30.912	30.896	-0.46%

6.3.3 Fatigue case study 3:

This is a fatigue comparison of NJDOT Report Example 1 for overhead cantilever sign structure-flat panel sign board study. The sketch of the structure is shown in Figure 6.3

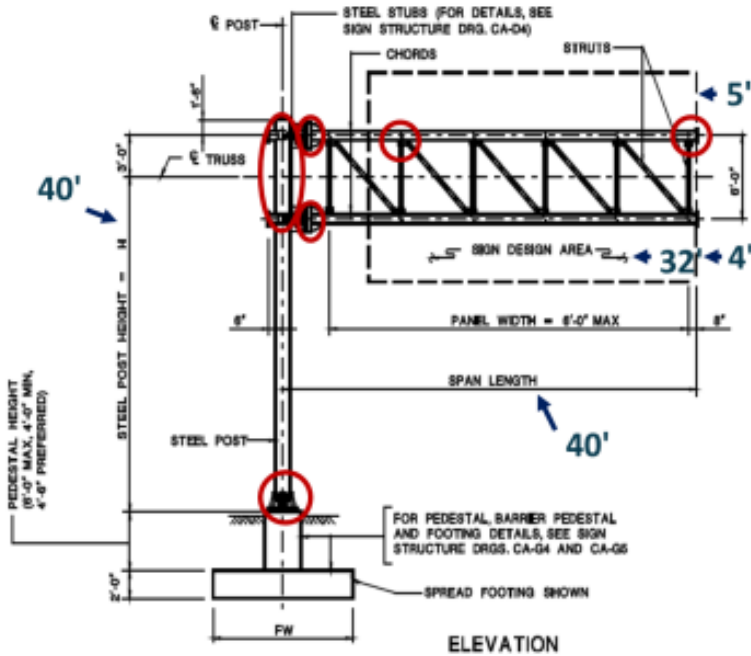


Figure 6-3 NJDOT Example 1 for overhead cantilever sign structure-flat panel sign board

Table 6.3 shows fatigue comparison results among NJDOT Report, STAAD Pro, and SABRE programs.

Table 6-3 Fatigue Comparison among NJDOT Report, STAAD Pro and SABRE programs

Example 1						
Fatigue II		Sabre	NJDOT Report	Corrected	Staad.Pro	
Joint load (kips)	Gallop	7.056	7.056		7.056	
	Natural.W	3.779	4.146	3.2551	3.239	
	NW sign	2.534	3.6048	2.545		
	NW chord	0.703		0.17		
	NW column	0.543	0.5401	0.5401		
	Truck.W	0.408	1.438	0.469	0.408	
	TW sign	0.134	0.357	0.134	0.134	
	TW chord	0.274	1.0811	0.335	0.274	% Diff
Moment (K-ft)	Gallop	167.8	141.6	169.34	169.3	0.89%
	Truck.W	13.7	49.9	14.51	13.87	1.23%
Torsion (k-ft)	Natural.W	63.6	81.9	57.65	61.92	-2.71%

Based on our finding, the summary with the hand-check of example 8-1 with the following updates:

1. Chord Cd value: The report uses $C_d=1.1$. $C_d=0.45$ assumed in SABRE is more reasonable.
2. Sign moment distance: The SABRE evenly distributes the joint loads into three VAMs, while the hand-check considers the distance starts from the center to the last 12 ft. The assumption in SABRE makes the results smaller than it actually is.
3. Single/double chord: AASHTO does not state which is correct for a panel truss case. In this report, both examples consider only a single chord for truck gust.

6.3.4 Fatigue case study 4:

This is a fatigue comparison of NJDOT Report Example 2 for cantilever sign structure category I – VMS/DMS sign board study. The sketch of the structure is shown in Figure 6.4.

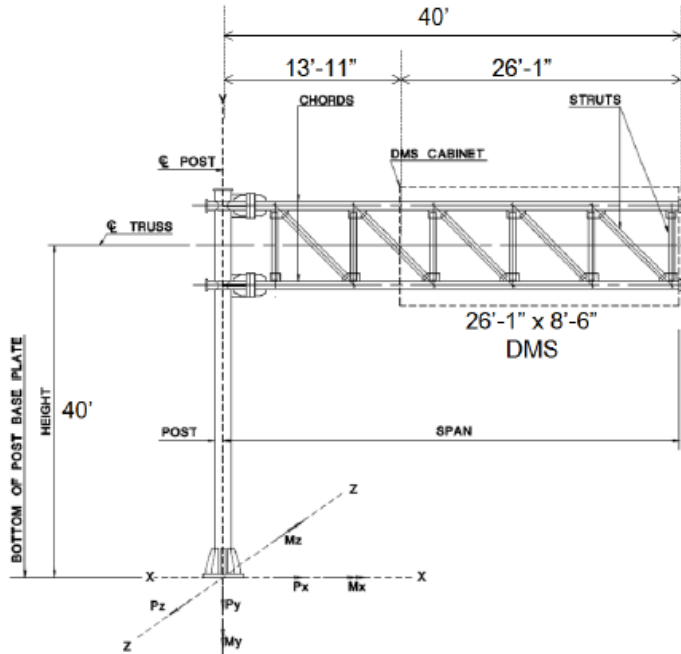


Figure 6-4 NJDOT Example 2 for cantilever sign structure category I – VMS/DMS sign board

Table 6.4 shows fatigue comparison results among NJDOT Report, STAAD Pro, and SABRE programs.

Table 6-4 Fatigue Comparison among NJDOT Report, STAAD Pro, and SABRE programs

Example 2						
Fatigue I		Sabre	Report	Corrected	Staad.Pro	
Joint load (kips)	Gallop	4.655	4.655	4.655	4.641	
	Natural.W	3.498		2.888	2.843	
	NW sign	1.967	1.959	1.959		
	NW chord	0.929		0.293		
	NW column	0.639		0.636		
	Truck.W	1.839	3.373	1.838	1.839	
	TW sign	1.534	3.0682	1.5341	1.534	
	TW chord	0.305	0.3048	0.1524	0.305	
		Sabre	Report SAP2000		% Diff	
Moment (K-ft)	Gallop	125.9	147.08		125	-0.75%
	Natural.W	59.48	131.33		54.95	-8.24%
	Truck.W	62.7	105.33		62.52	-0.29%

6.4 LRFD Case Study

Comparison with NCHRP Report 718 for HMLT (Examples 1 & 5), NCHRP Report 796 (Examples 2 & 9), and NJDOT (Examples 1 & 2): Summarize the latest comparison results and list out the differences and the causes.

Table 6-5 Fatigue Comparison of NCHRP Report 796 Example 9 and SABRE programs

NCHRP 796	EX-9	SABRE		NCHRP 796	
Natural Wind Gusts	Joint Load (kips)	Sign	Pole	Sign	Pole
		1.192	0.115	1.188	0.303
		Total	1.307	Total	1.491
	Bending Moment (kips-ft)	Sign	Pole	Sign	Pole
		23.84	0.65	23.8	6.05
		Total	24.49	Total	29.8
Truck induced Gusts	Joint Load (kips)	Sign	pole	Sign	pole
		1.534	0	2.17	1.24
		Total	1.534	Total	3.41
	Bending Moment (kips-ft)	Sign	Pole	Sign	Pole
		27.53	0	39	18.61
		Total	27.53	Total	57.6

- 1) The forces on the circular curve pole were not included.
- 2) The report uses the whole length of the sign panel for calculation for the truck induced gust.

Table 6-5a Different applying joint for the horizontal force on the pole

HMLT EX1	L= 80ft EX1	SABRE		NCHRP 718	
Joint Load (kips)		Sign	Pole	Sign	Pole
		0.092	0.534	0.093	0.529
		Total	0.637	Total	0.622
Bending Moment (kips-ft)		Base	27.53	Base	25.4

Table 6-5b Different applying joint for the horizontal force on the pole

HMLT EX5	L= 160ft EX5	SABRE		NCHRP 718	
Joint Load (kips)		Sign	Pole	Sign	Pole
		0.109	1.926	0.1152	1.9829
		Total	2.035	Total	2.0981
Bending Moment (kips-ft)		Base	160.4	Base	145.536

Complete design, input and output for NJDOT Examples 1, NCHRP Report 796 Examples 2 & 9, and NCHRP Report 718 Examples 1 are shown as Examples 1-4 in Appendix C.

Chapter 7 SABRE/LRFD System review

SABRE was developed by the research team in the early 1990s and they have maintained it ever since. In order to establish confidence in the program, a complete review and conformance test on was performed. SABRE design models of highway sign, high post, and traffic signal post were verified by an alternate commercial licensed finite element analysis software called STADD Pro.

The following is a list of items which were modified in the SABRE program to accommodate HMLT and others (See Appendix D):

- 1) A new configuration in data type (21000)
- 2) Program can run without the arm (31000, 33000, 34000)
- 3) Sign can be attached to the pole member (07012)
- 4) More segments for the pole (31000) (now the limit is 23 segments)
- 5) Parameters needed and only equivalent horizontal fatigue load needed (very similar to the natural wind pressure, but with the different pressure range, and no galloping and truck gust):

Equivalent static pressure range:

$$P_{CW} = P_{FLS} C_d$$

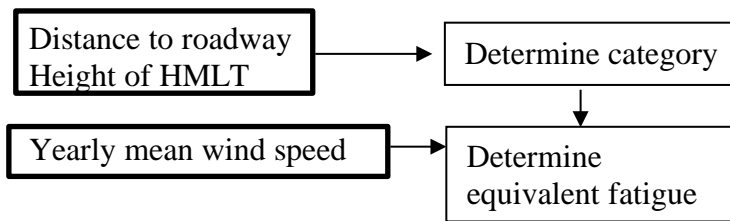


Table 7-1 HMLT Hazard Level defined by AASHTO

Hazard Level	Importance Category
High (distance to roadway \leq height of HMLT)	I
Low (distance to roadway $>$ height of HMLT)	II

Table 7-2 HMTL Pressure Range defined by AASHTO

Fatigue-Limit-State Pressure Range for HMLT Design, P_{FLS}		
Fatigue Design Case	Importance Category	
	I	II
$V_{mean} \leq 9$ mph	6.5 psf	5.8 psf
$9 \text{ mph} < V_{mean} \leq 11$ mph	6.5 psf	6.5 psf
$V_{mean} > 11$ mph	7.2 psf	7.2 psf

Chapter 8 Summary and Report

A summary of all six tasks listed above and their respective conclusions are included in this chapter. The report also summarizes the calibration of the AASHTO ASD and LRFD for Maryland sign and high mast light pole structure design.

1. Complete design parameters of MD-specified sign and light structures

A previous study completed by the research team, Fatigue Resistant Design Criteria for MDOT SHA Cantilevered Mast Arm Signal Structures, published in 2017, included extensive studies to determine the design parameters for Maryland-specified signal structures. A similar approach was used in this project to identify the Maryland-specified Fatigue Category of Importance Factor (I, II or III) for sign and high mast light structure designs.

In the 2017 study, the fatigue design of mast arm structures was thoroughly studied. The study gathered and discussed the current state-of-the-practice methods. Complete model analysis of traffic signal structure, including structure foundation, was conducted using ANSYS and SABRE programs and self-developed Excel calculation sheets. Recommendations for Maryland structure designs were proposed.

2. Define Maryland wind pressure in LRFD for sign structure design

Both wind pressure formulas for allowable stress design (ASD) and LRFD are studied. The changes and their influences are discussed. The change of Maryland wind speed has been specifically analyzed, since AASHTO ASD LTS-6 and LRFD LTS-1 have significantly different approaches in this section. The current wind speed of 100mph for all of Maryland is no longer reasonable. This research recommends that Maryland be divided into three regions: the Eastern Shore, the Appalachia Mountain, and the Baltimore-Washington corridor. The Average Daily Traffic (ADT) in the Eastern Shore and Appalachia Mountain regions can be considered to be within $1000 < ADT \leq 10,000$ and structures can be designed as “typical” supports (MRI = 700 years); while the Baltimore-Washington corridor should be considered to be $ADT > 10,000$ where travel-ways are assigned a high-risk category so that the consequence of failure (MRI=1700 years) is factored in.

The wind speed adopted for STD LTS-6 is 100mph, while for LRFD LTS-1 it is 120mph. If LRFD is adopted for Maryland designs, wind speed of 120mph should be adopted. A wind load pressure analysis was completed using different AASHTO specifications. The contribution of multi-arm influence was also included and discussed.

3. Calibration of LRFD Designs by Adopting Fatigue Design

Maryland signal poles that need to be modified in order to increase the fatigue resistance have been identified with recommended modifications. The recommended modifications of the current design on the signal poles include: a) Groove welds for arm connections, b) Groove welds for pole connections, c) Adopting AASHTO build-up box type for arm connections, and d) 6-bolt patterns for both arm and pole connections. Other structure changes and recommendations were discussed in this study, such as tube-to-tube connections between main chords and bracings for sign structures.

A comparison of the AASHTO STD LTS-6 & LRFD LTS-1 Specs was completed. Their impacts of modifying details on sign structural supports due to fatigue were also studied in this task.

4. LRFD Foundation Design

In the AASHTO LRFD LTS-1, Extreme I, Service I, and II Limit States have Wind Load (W) involved and they should be considered in foundation designs. In this study, the typical Maryland standard foundation types were analyzed. These types include shaft foundation for signal poles, shaft foundation with wing walls for cantilever sign structures, and mat found with pedestal for overhead structures.

For shaft foundation and shaft foundation with wing walls under an assumed soil condition, shaft embedment lengths are longer than the required length in both cohesive and cohesionless soils based on Broms' methods. The reinforcement design of the shaft is sufficient to resist ground moment from the base. Torsion resistances of the shaft are higher than the maximal base torque provided by SABRE software either in the cohesive or cohesionless soil.

For the mat found with pedestal for overhead structures, three individual cases were examined. Both sliding and eccentricity limits were investigated in two horizontal directions. The flexural capacity was checked by calculating the moment capacity of the footing given the number of rebars present. Overall, the spread footings design passes the check under the critical conditions with adequate capacity.

5. Comparison between the AASHTO STD LTS-6 and LRFD LTS-1 Specifications

A detailed comparison between AASHTO LTS-6 and LRFD LTS-1 was completed in this study. The expressions for LTS-6 allowable stress and LRFD LTS-1 nominal strength are dramatically different. In this section, the tube sections are used to demonstrate such differences in expressions and their effects. The interaction equations are also significantly different since one is based on combined stress ratio while the other is combined force. Samples from Maryland Standards were used for checking Strength I Limit State.

For the fatigue study, four case studies with examples from National Cooperative Highway Research Program (NCHRP) Report 718 – *Fatigue Loading and Design Methodology for High-Mast Lighting Tower*; NCHRP Report 796 – *Development and Calibration of AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*; and a New Jersey Department of Transportation (NJDOT) report - *Fatigue Study on Structural Supports for Luminaries, Traffic Signals, Highway Signs (2015)*, are discussed. Their results were verified using SABRE and STAAD analysis. The verification process indicates that SABRE is accurate for the fatigue analysis. Also, several calculation mistakes used in the NCHRP and NJDOT reports were identified and the correct figures are provided in Chapter 6.

6. SABRE/LRFD System review

A complete review and conformance test on the SABRE program were performed. SABRE design models of highway sign, high post, and traffic signal post were verified by an alternate commercial licensed finite element analysis software, STADD Pro. The comparisons are in good agreement.

Appendix A Reference

- AASHTO (2013) "Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals," AASHTO LTS 6th Edition, Washington, DC
- AASHTO (2015), "LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals," AASHTO LRFD LTS-1, Washington, DC
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http://www.roads.maryland.gov/OPR_Research/MD-17-SHA-UM-4-20_Signal-Poles-Fatigue_Report.pdf
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- Roda, A., Najm, H., Choi, H., (2015), "Fatigue Study on Structural Supports for Luminaries, Traffic Signals, Highway Signs," CAIT/Rutgers
- Report 469 (2002), "Fatigue-Resistant Design of Cantilevered Signal, Sign, and Light Supports," Transportation Research Board, Washington, DC
- NCHRP Report 718 (2012), "Fatigue Loading and Design Methodology for High-Mast Lighting Towers," Transportation Research Board, Washington, DC
- NCHRP Report 796 (2014), "Development and Calibration of AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals," Transportation Research Board, Washington, DC

Appendix B Spreadsheets for Overhead Sign Structure

An example among the Excel spreadsheets used for calculations about the overhead sign structures is presented in this section. This example demonstrates the calculation for an overhead sign structure spread footing with a span length of 79 ft. Each spread footing is checked with the same method, and the format is consistent throughout the entire calculation. The four limit states to be checked for each overhead sign structure foundation are listed below:

- Bearing Capacity
- Overturning (eccentricity)
- Sliding
- Flexural Capacity

User inputs are required for the calculation due to various dimensions and soil conditions. Footing dimensions can be directly entered, and the spreadsheets are also divided into two categories to account for cohesive or cohesionless soil conditions. Once the dimensions and base reactions (demands) are entered, the footing will be checked against the four limit states mentioned above. The conclusions are marked in yellow for each limit state.

RECTANGULAR FOOTING (Span Length = 79 feet)								
Design Parameters (User Inputs)					Demands (User Inputs)			
cohesion	C	=	2160	psf	M_x	=	-599.214	k*ft
friction angle	ϕ	=	30	degrees	M_y	=	-40.644	k*ft
unit weight of soil	γ	=	120	pcf	M_z	=	11.294	k*ft
unit weight of concrete	γ_c	=	150	pcf	P_x	=	-1.069	kip
depth of footing	D_f	=	10	ft	P_y	=	8.620	kip
water table	D_w	=	20	ft	P_z	=	-19.966	kip
adhesion	c'_a	=	2160	psf	Footing Mark		OF-620	
width	B	=	6	ft				
base length	L	=	20	ft	B	=	X	
Section 1 height	H_1	=	3	ft	L	=	Z	
Base Height	H_2	=	5	ft				
Concrete Yield Strength	f'_c	=	5	ksi				
Steel Yield Strength	f_y	=	60	ksi				
# 6 rebar	d_1	=	0.75	in.				
Rebar Clear Cover	cc	=	3	in.				
Cubic Yard Concrete	V_{conc}	=	29.2	yd ³				

Figure Appendix 1- User inputs and base reactions (demands)

Check for Bearing Capacity (AASHTO 10.6.3.1)			
N_c	=	30.1	
S_c	=	1.18339	
i_c	=	1	
N_q	=	18.4	
s_q	=	1.17321	
d_q	=	1	
i_q	=	1	
N_γ	=	22.4	
S_γ	=	0.88	
i_γ	=	1	
c_{wq}	=	1	
$c_{w\gamma}$	=	1	
N_{cm}	=	35.62	
N_{qm}	=	21.587	
$N_{\gamma m}$	=	19.712	
q_n	=	109940	psf
$q_{allowable}$	=	49472.9	psf
ϕ_{ult}	=	0.45	
$Q_{allowable}$	=	2671.54	kips
Q_{demand}	=	127.686	kips
$FS_{bearing\ capacity}$	=	20.9227	
FS_{min}	=	3	
Status		O.K.	
$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$			

Figure Appendix 2- Check for bearing capacity

Check for Overturning AASHTO 10.6.3.3					
Overturning - Eccentricity (Z direction)					
V_{conc} (yd ³)	V_{conc} (ft ³)	W_{conc} (kips)	e	B/3	Status
29.2	788.4	118.26	0.172146119	2	O.K.
Overturning - Eccentricity (X direction)					
V_{conc} (yd ³)	V_{conc} (ft ³)	W_{conc} (kips)	e	L/3	Status
29.2	788.4	118.26	3.362506342	6.7	O.K.

Figure Appendix 3- Check for overturning (eccentricity)

AASHTO 10.6.3.4					
Check for Sliding (Z direction)					
T (lb·ft)	J (ft ⁴)	r (ft)	τ (psf)	Area (ft ²)	Shear (kips)
40644	4360	3	27.9661	100	2.796605505
R _t (kips)	φ _t	R _{ep} (kips)	φ _{eq}	V (kips)	tanφ
68.2774	0.85	310.8737847	0.5	118.26	0.577350269
Resisting force		R _R	=	58.0358	kips
R _R = φR = φ _t R _t + φ _{ep} R _{ep}		F _d	=	22.7626	kips
Sliding Safety Factor		FS _{sliding}	=	2.54961	
FS _{sliding} = $\frac{\sum F_R}{\sum F_d}$		FS _{min}	=	1.5	
		Status		O.K.	
Check for Sliding (X direction)					
T (lb·ft)	J (ft ⁴)	r (ft)	τ (psf)	Area (ft ²)	Shear (kips)
40644	4360	10	93.2202	30	2.796605505
R _t (kips)	φ _t	R _{ep} (kips)	φ _{eq}	V (kips)	tanφ
68.2774	0.85	1036.245949	0.5	118.26	0.577350269
Resisting force		R _R	=	58.0358	kips
R _R = φR = φ _t R _t + φ _{ep} R _{ep}		F _d	=	3.866	kips
Sliding Safety Factor		FS _{sliding}	=	15.0134	
FS _{sliding} = $\frac{\sum F_R}{\sum F_d}$		FS _{min}	=	1.5	
		Status		O.K.	

Figure Appendix 4- Check for sliding

Check for Reinforcement						
b (in)	d (in)	d' (in)	no. CS	no. TS	A's (in ²)	As (in ²)
6	92.625	3.375	7	7	3.092505268	3.09251
Compression steels do not yield, tension steels yield						
ε _{cu}	β ₁	A	B	C	c (in)	a (in)
0.003	0.85	21.675	83.49764225	-908.04	4.826889936	4.10286
Moment Capacity						
ε _s '	ε _s	ε _y	Mn (kip·ft)	φ	φMn (kip·ft)	FS _{moment}
0.0009	0.05457	0.00207	1391.570129	0.9	1252.413116	61.5195
					Status	O.K.

Figure Appendix 5- Check for flexural capacity

Appendix C Verification of SABRE Fatigue Analysis using Reports and STAAD-PRO

Example C1 – Planal cantilever sign structure

Source: NJDOT report “Fatigue Study on Structural Supports for Luminaries, Traffic Signals, Highway Signs”, Chapter 8 example 1

C1.1 Example overview

Example I was demonstrated for a cantilevered sign with flat panel sign board for Category II based on the AASHTO LRFD design specifications (2015), used here for fatigue verification.

Structural parameters:

Span length = 40 ft

Sign design length = 32 ft

Sign design height = 15 ft

Sign design area = 480 ft²

Steel post height = 40 ft (30 x 0.625 in)

Steel truss chord = 18 in x 0.5 in

Steel truss struts = 5.563 in x 0.375 in

Steel truss stubs = 18 in x 0.5 in

$x_1=40$ ft, $x_2=24$ ft and $x_3=2.5$ ft

$y_1=48$ ft, $y_2=37$ ft and $y_3=24$ ft

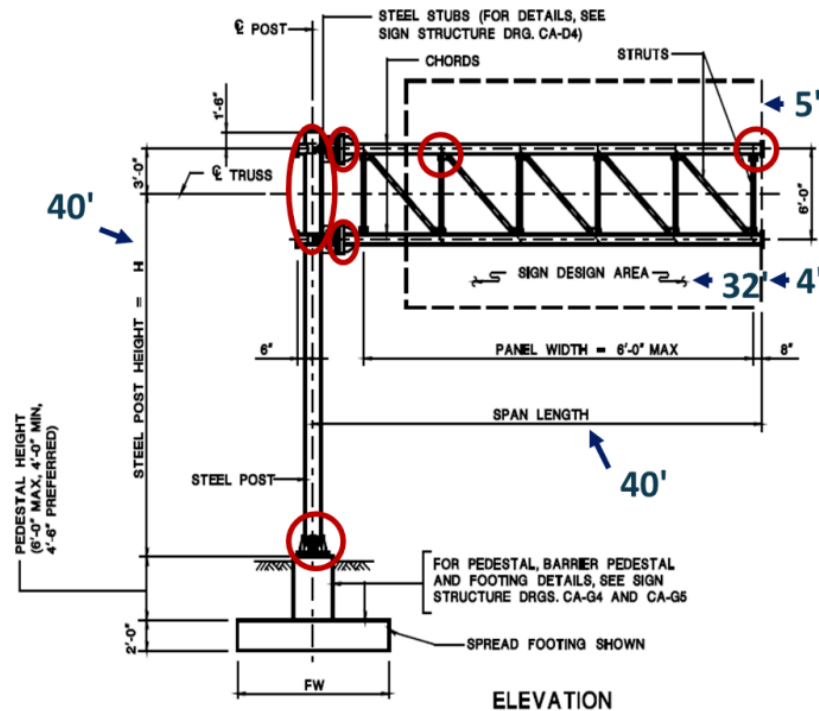


Figure Appendix 6- Elevation view of the sign structure

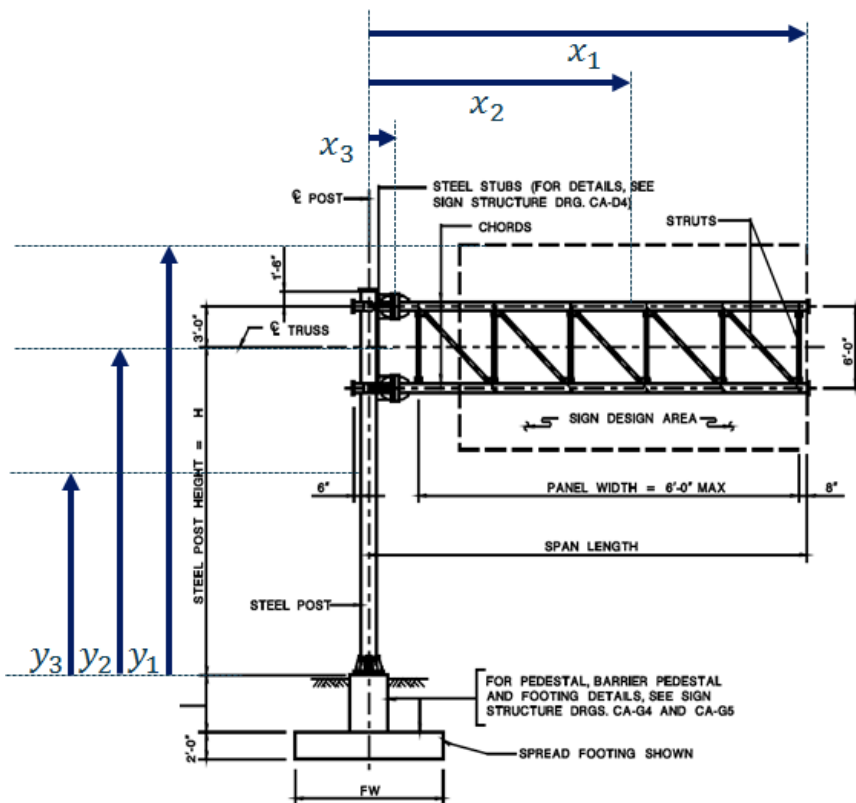


Figure Appendix 7- Elevation view of the sign

C1.2 SABRE program inputs

Signs/Signals	Yield Stresses (Steel only)	Definition of Sections	VAMS
Bracing	Element Definition	Dimensions	Cross Sections
Configuration			
DATA TYPE: 21000			
<input checked="" type="radio"/> CANTILEVER	<input type="radio"/> SPAN	<input type="radio"/> BUTTERFLY	
<input type="radio"/> SPAN WITH 1 CANTILEVER	<input type="radio"/> DOUBLE SPAN		

Figure Appendix 8- Configuration selection

DATA TYPE: 34000 Cantilever

BEAM LENGTH :

LOW CHORD BEAM ELEV.:

BEAM DEPTH :

BEAM WIDTH :

: CONNECTION LENGTH

TOWER WIDTH :

LAST BRACE :

FIRST BRACE:

TOP ELEV. :

BOT. ELEV. :

Figure Appendix 9- Structure dimensions

DATA TYPE: 38000

FRONT PATTERN :

TOP PATTERN :

REAR PATTERN :

BOTTOM PATTERN :

PERP. SECTION :

DIAG. SECTION :

PATTERN :

PERP. SECTION :

DIAG. SECTION :

Note: If there is no perpendicular member, leave Perp. Section blank.

Figure Appendix 10- Bracing pattern

DATA TYPE: 04012

Section Lookup

Sec No	Standard Section			Tubular Shapes				General Sections						
	ID	DTH inch (mm)	WT/FT (Lb./ft)	ID	OUT D inch (mm)	Tw inch (mm)	Kt	ID	WT/FT Kip/ft (KN/m)	S1 inch (mm)	S2 inch (mm)	S3 inch (mm)	S4 inch (mm)	S5 inch (mm)
1				2	30.	0.625	1.							
2				2	18.	0.5	1.							
3				999										
4				2	5.563	0.375	1.							
5	W	6	12					9	.012	4.000	6.030	0.280		0.230
6														
7														
8														

Figure Appendix 11- Definition of sections

DATA TYPE: 07012

VAM ID	Top VAM Coords			Length feet (m)	Supporting Members *			Attached Sign Units			Walkway Units			
	No	Sect.	X feet (m)		Y feet (m)	Z feet (m)	No.	No.	No.	No.	No.	No.	No.	
1	5		14.	43.	-0.5	6.	19	25		1				
2	5		24.	43.	-0.5	6.	20	26		1				
3	5		34.	43.	-0.5	6.	22	28		1				
4														
5														

Figure Appendix 12- Definition of VAM members

Signs/Signals

DATA TYPE: 09012

Sign No.	Dimensions			Slope in/ft (mm/m)	Density k/cf (Kg/m^3)	Lower Left Coord			Drag Optional Cd
	Width feet (m)	Height feet (m)	Thick. inch (mm)			X feet (m)	Y feet (m)	Z feet (m)	
1	32.	15.	0.55		0.175	8.	32.5	-0.25	
2									

Figure Appendix 13- Sign dimensions

Parameters	Additional Joint Loads	Walkways
DATA TYPE: 01042		
WIND VELOCITY	<input type="text" value="80.0"/> (mph or m/s)	
MEAN REGULAR INTERVAL	<input type="text" value="50"/> (years)	
GUST FACTOR	<input type="text" value="1.14"/> Regular Method - 1.14 (4th or 5th ed.)	
ICE LOADS	<input type="text" value="3.0"/> (psf or KPa)	ICE LOAD OPTIONS <input type="text" value="0"/> 0- One Side
TRUSS BRACING OPTIONS	<input type="text" value="0"/> 0- Moment Bracing (default)	
D. L. DETAIL FACTOR	<input type="text" value="1.0"/>	
WIND IMPORTANCE FACTOR	<input type="text" value="1.0"/> (default = 1.0; see AASHTO Table 3-2)	
For Fatigue Design Only		
YEARLY MEAN VELOCITY FOR NATURAL WIND GUST	<input type="text" value="11.2"/> (default = 11.2 mph or 5 m/s)	
VEHICLE SPEED FOR TRUCK-INDUCED GUST	<input type="text" value="65."/> (default = 65 mph or 30 m/s)	
FATIGUE IMPORTANCE FACTOR OPTIONS		
GALLOPING (default = 1.0)	NATURAL WIND (default = 1.0)	TRUCK - INDUCED GUST (default = 1.0)
<input type="text" value=".7"/>	<input type="text" value=".85"/>	<input type="text" value=".9"/>

Figure Appendix 14- Detailed parameters



N3dot chapter-8 ex-1

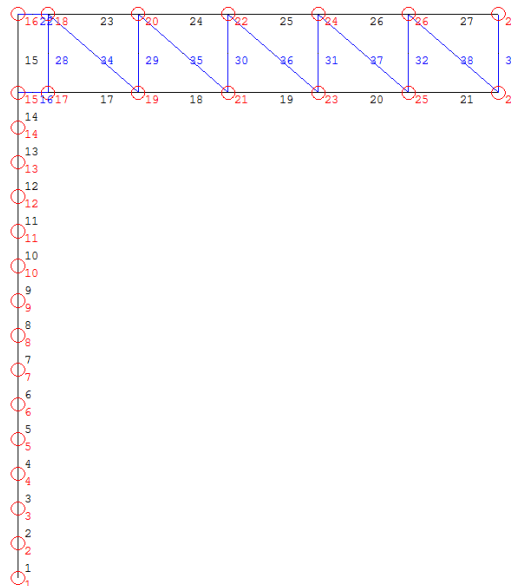


Figure Appendix 15- SABRE graphic output

C1.3 SABRE calculation, NJDOT report, and STAAD fatigue calculation comparison

Table Appendix 1- Comparison table

Example I	(Fatigue II)	SABRE	Report	STAAD
Joint load (kips)	Gallop	7.056	7.056	7.056
	Natural wind total	3.761	3.2551*	3.237*
	sign	2.534	2.545	2.544
	chord	0.703	0.17	0.17
	column	0.525	0.5401	0.522
	Truck. W	0.408	0.271	0.271
	TW sign	0.134	0.134	0.134
	TW chord	0.274	0.137**	0.137**
Moment (K-ft)	Gallop	169.9	169	169.3
	Natural. W	63.6	57.65	61.1
	Truck. W	13.92	8.87	9.21

*The NJDOT report misused the sign C_d as 1.7, which should be $C_d=1.2$. The corrected result is listed in the table.

**The NJDOT report misused the chord C_d as 1.1 and applied it to the full arm. The correct calculation should pick C_d as 0.45 and applied to the last 12 ft of the arm. The corrected result is listed in the table.

Example C2 - Signal pole structure

Source: NCHRP Report 796, Appendix C, Example 2

C2.1 Example overview

A traffic signal pole is located in Chicago, IL. The pole has three signals and adjoining signs. The geometry, signal and sign locations, and weights are provided in the figure below. The pole location is windy and fatigue limit state must be checked. The structure is located on a busy roadway where the typical traffic speed limit is 45 mph and the ADT are 5000. The roadway is not a lifeline roadway. Material properties, section data, and geometry are defined below.

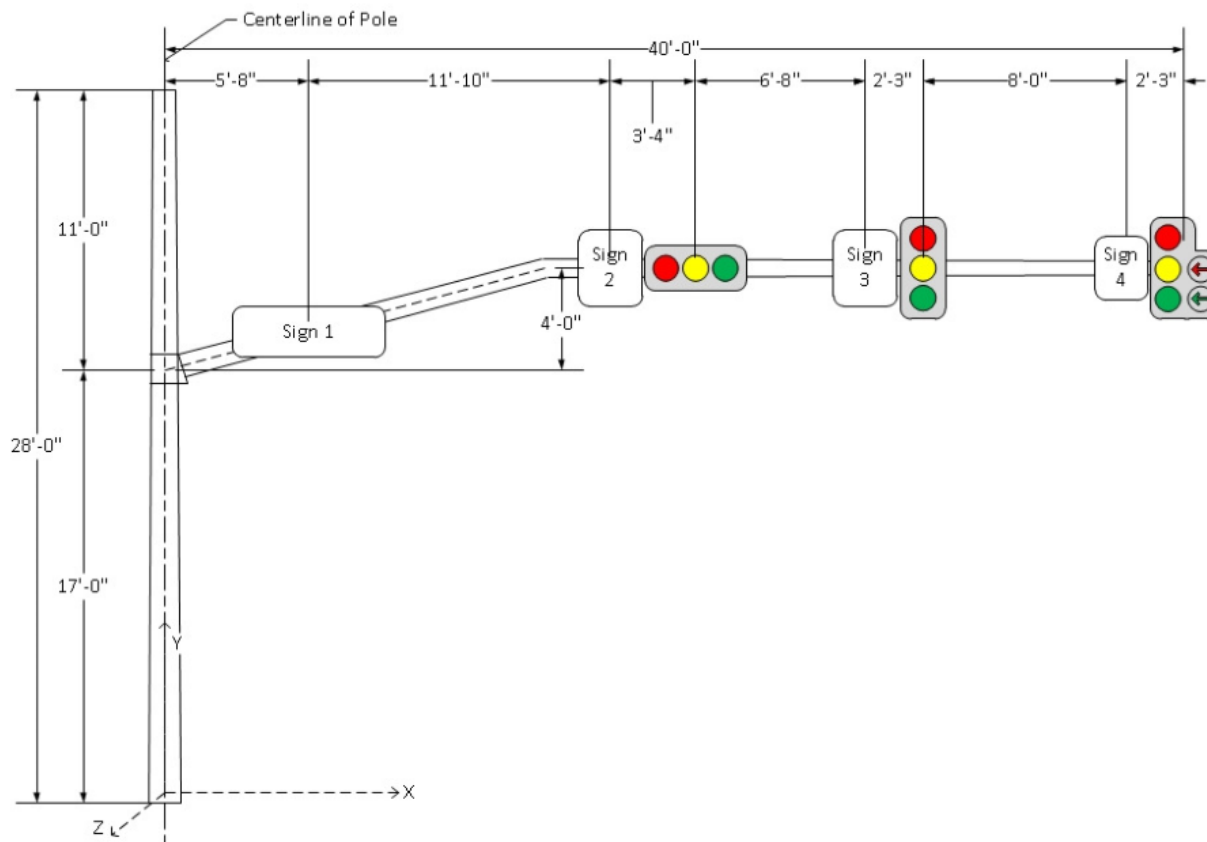


Figure Appendix 16– Elevation view of the signal pole

Structural parameters:

$$E_s = 29000 \text{ ksi}, F_y = 48 \text{ ksi}, \gamma_{\text{steel}} = 490 \text{ lbf/ft}^3$$

$$\text{Arm_taper} = 0.14 \text{ in/ft}, t_{\text{arm}} = 0.3125 \text{ in}, t_{\text{pole}} = 0.3125 \text{ in}$$

$$\text{Arm_base_diameter} = 12 \text{ in}$$

$$\text{Arm_end_mast_CL_length} = 39.05 \text{ ft}, \text{Arm_angle} = 15 \text{ deg}, \text{Arm_rise} = 4 \text{ ft}$$

$$T_{\text{connection}} = 1 \text{ ft}, h_{\text{arm}} = 17 \text{ ft}$$

$$L_{\text{luminaire_arm}} = 20, D_{\text{luminaire_arm}} = 3.8 \text{ in}$$

$$A_{\text{luminaire}} = 3.3 \text{ ft}^2$$

$$D_{\text{pole_base}} = 15 \text{ in}, \text{Pole_taper} = 0.14 \text{ in/ft}, h_{\text{pole}} = 28 \text{ ft}$$

$$H_{\text{luminaire}} = 31.75 \text{ ft}, d_{y_PoleBaseToArmBase} = 17 \text{ ft}$$

C2.2 SABRE program input

Signs/Signals	Yield Stresses (Steel only)	Definition of Sections	VAMS
Bracing	Element Definition	Dimensions	Cross Sections
Configuration			

DATA TYPE: 21000

CANTILEVER
 SPAN
 BUTTERFLY

SPAN WITH 1 CANTILEVER
 DOUBLE SPAN

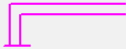
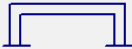









Figure C2.2 - Configuration selection

DATA TYPE: 34000

BEAM LENGTH : 40.

LOW CHORD BEAM ELEV.: 17.

BEAM DEPTH :

BEAM WIDTH :

1.0 : CONNECTION LENGTH

TOWER WIDTH :

LAST BRACE :

FIRST BRACE:

TOP ELEV. : 28.

BOT. ELEV. : 0

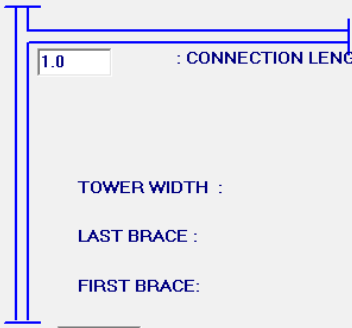


Figure Appendix 17- Structure dimensions

DATA TYPE: 04012				Section Lookup										
Sec No	Standard Section			Tubular Shapes			General Sections							
	ID	DTH inch (mm)	WT/FT (Lb./ft)	ID	OUT D inch (mm)	Tw inch (mm)	Kt	ID	WT/FT Kip/ft (KN/m)	S1 inch (mm)	S2 inch (mm)	S3 inch (mm)	S4 inch (mm)	S5 inch (mm)
1				2	15.	0.313	1.							
2				2	11.08	0.313	1.							
3				2	12.	0.313	1.							
4				999	0	0	1.							
5				2	6.4	0.313	1.							
6	W	4	13					9	0.001	4.06	4.16	0.345		0.280
7														
8														

Figure Appendix 18- Definition of sections

DATA TYPE: 07012														
VAM ID		Top VAM Coords			Length feet (m)	Supporting Members *			Attached Sign Units			Walkway Units		
No	Sect.	X feet (m)	Y feet (m)	Z feet (m)		No.	No.	No.	No.	No.	No.	No.	No.	No.
1	6	5.67	19.52	0.25	2.	13			1					
2	6	17.5	22.5	0.25	3.	18			2					
3	6	20.83	22.085	0.25	2.175	19			3					
4	6	27.5	22.5	0.25	3.	22			4					
5	6	29.75	23.175	0.25	4.35	23			5					
6	6	37.75	22.25	0.25	2.5	26			6					
7	6	39.9	23.6	0.25	5.2	26			7					
8														

Figure Appendix 19- Definition of VAM members

DATA TYPE: 09012										
Sign No.	Dimensions			Slope in/ft (mm/m)	Density k/cf (Kg/m ³)	Lower Left Coord			Drag Optional Cd	
	Width feet (m)	Height feet (m)	Thick. inch (mm)			X feet (m)	Y feet (m)	Z feet (m)		
1	6.	2.	0.6	0	0.034	2.67	17.52	0.6		
2	2.5	3.	0.6	0	0.040	16.25	19.5	0.6		
3	4.0	2.175	12	0	0.006	18.83	19.91	0.6		
4	2.5	3.	0.6	0	0.040	26.25	19.5	0.6		
5	2.	4.35	12.	0	0.006	28.75	18.825	0.6		
6	2.	2.5	0.6	0	0.040	36.75	19.75	0.6		
7	2.5	5.2	12.	0	0.006	38.75	18.4	0.6		
8										

Figure Appendix 20- Sign dimensions

*An equilibrium section is selected for the irregular signal section.

DATA TYPE: 01042

WIND VELOCITY	<input type="text" value="115"/>	(mph or m/s)
MEAN REGULAR INTERVAL	<input type="text" value="50"/>	(years)
GUST FACTOR	<input type="text" value="1.14"/>	<input type="text" value="Regular Method - 1.14 (4th or 5th ed.)"/>
ICE LOADS	<input type="text" value="3.0"/>	(psf or KPa) ICE LOAD OPTIONS <input type="text" value="0"/> <input type="text" value="0- One Side"/>
TRUSS BRACING OPTIONS	<input type="checkbox"/>	<input type="text" value="0- Moment Bracing (default)"/>
D. L. DETAIL FACTOR	<input type="text" value="1.0"/>	
WIND IMPORTANCE FACTOR	<input type="text" value="1.0"/>	(default = 1.0: see AASHTO Table 3-2)
For Fatigue Design Only		
YEARLY MEAN VELOCITY FOR NATURAL WIND GUST	<input type="text" value="12.9"/>	(default = 11.2 mph or 5 m/s)
VEHICLE SPEED FOR TRUCK-INDUCED GUST	<input type="text" value="45"/>	(default = 65 mph or 30 m/s)
FATIGUE IMPORTANCE FACTOR OPTIONS	<input type="text"/>	
GALLOPING (default = 1.0)	NATURAL WIND (default = 1.0)	TRUCK - INDUCED GUST (default = 1.0)
<input type="text" value=".65"/>	<input type="text" value=".80"/>	<input type="text" value=".85"/>

Figure Appendix 21- Detailed parameters

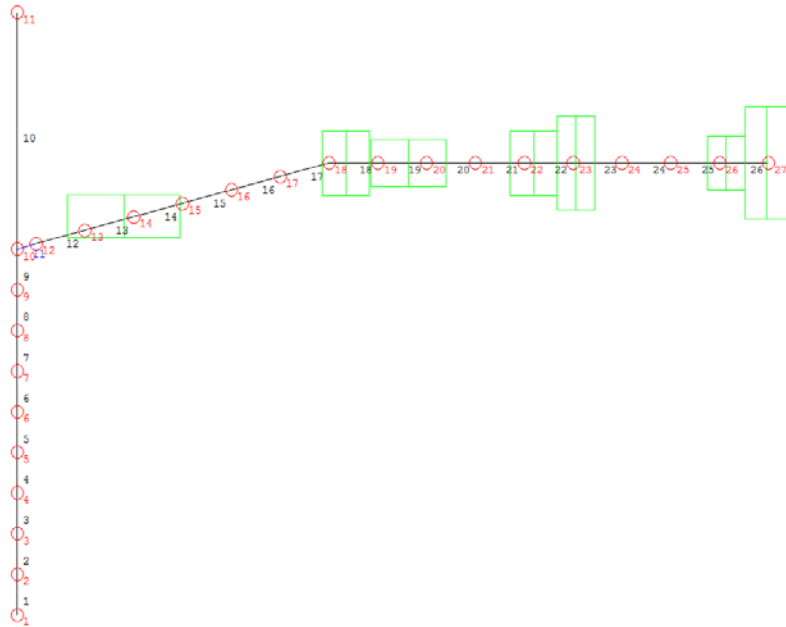
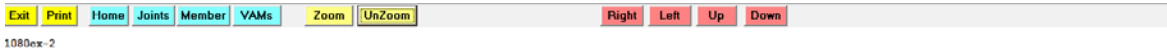


Figure Appendix 22- SABRE graphic output

C2.3 SABRE calculation, NCHRP Report and STAAD fatigue calculation comparison

Table Appendix 2- Comparison table of joint loads

EX-2	(Fatigue II)	SABRE	Report	STAAD
Joint Load (kips)	Wind Load Strength Total	3.378	3.28	3.287
	Sign	-	2.311	2.326
	Pole and Arm	-	0.969	0.961
	Gallop Total	0.89	0.852	0.852
	Sign	0.89	0.852	0.852
	Pole and Arm	-	-	-
	Natural wind Total	0.781	0.78	0.758
	Sign	0.393	0.404	0.389
	Pole and Arm	0.388	0.376	0.369
	Truck induced Total	0.106	0.10	0.105
	Sign	0.042	0.0385	0.038
	Pole and Arm	0.064	0.0615	0.067

Table Appendix 3- Comparison table of moments

EX-2	(Fatigue II)	SABRE	Report	STAAD
Moment (K-ft)	Wind Load Strength Total (bending)	66.29	63.8	64.082
	Sign	-	47.43	45.967
	Pole and Arm	-	16.412	18.114
	Wind Load Strength Total (torsion)	71.52	69.8	66.539
	Sign	-	59.91	55.936
	Pole and Arm	-	9.89	10.602
	Gallop Total	22.81	22.1	21.217
	Sign	22.81	22.1	21.217
	Pole and Arm	-	-	-
	Natural wind Total (Bending)	14.68	15.054	14.428
	Sign	8.057	8.284	8.282
	Pole and Arm	6.623	6.77	6.146
	Natural wind Total (Torsion)	12.79	12.44	13.438
	Sign	10.6	10.47	10.055
	Pole and Arm	2.83	1.97	3.383
	Truck induced Total	3.56	3.53	3.56
	Sign	-	1.377	1.338
	Pole and Arm	-	2.152	2.222

Example C3 – Cantilever monotube sign structure

Source: NCHRP Report 796, Appendix C, Example 9

C3.1 Example overview

Design a cantilever monotube structure in Ft. Collins, CO. It supports a dynamic message sign weighing 5,000 pounds. Assume a 24" diameter circular tube fabricated from A36 steel. Bolts are ASTM A 325 bolts. The structure would cross a lifeline travel way on failure.

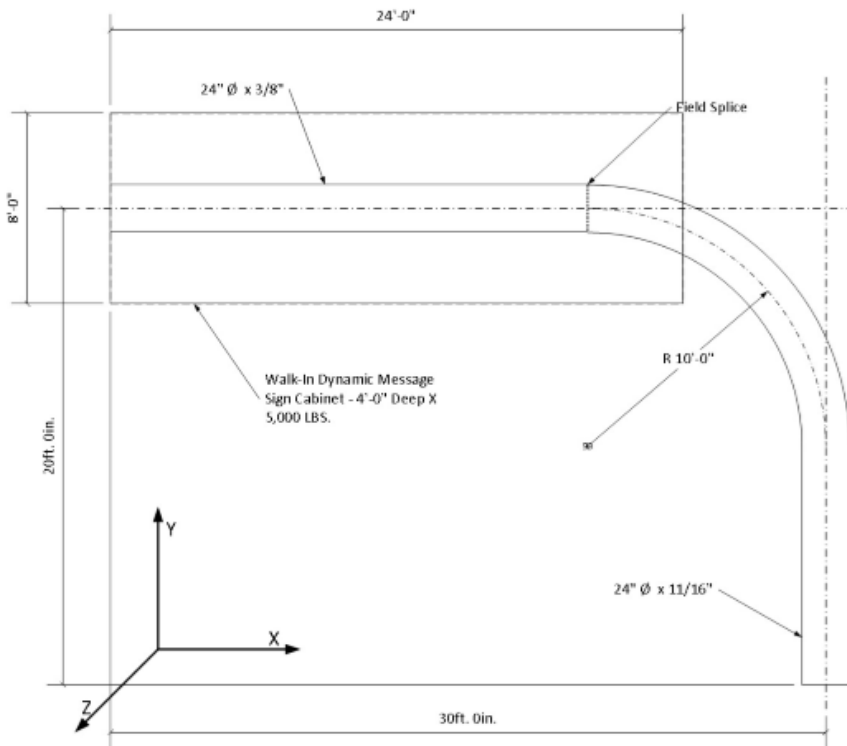


Figure Appendix 23- Elevation view of the sign structure

Structural parameters:

$F_y = 36 \text{ ksi}$, $E_s = 29000 \text{ ksi}$

$L_{\text{total}} = 30 \text{ ft}$, $h_{\text{total}} = 20 \text{ ft}$

Sign:

$b_{\text{sign}} = 24 \text{ ft}$, $h_{\text{sign}} = 8 \text{ ft}$

$d_{\text{sign}} = 4 \text{ ft}$, $P_{\text{sign}} = 5 \text{ kip}$

$EPA_{\text{SIGN}_X} = 32 \text{ ft}^2$

BendRadius = 10ft

Wind: $V_{E1} = 120 \text{ mph}$

Tubes:

$D_{\text{bot}} = 24 \text{ in}$, $t_{\text{bot}} = 11/16 \text{ in}$, $r_{\text{bend}} = 10 \text{ ft}$

$D_{\text{top}} = 24 \text{ in}$, $t_{\text{top}} = 3/8 \text{ in}$, $L_{\text{top}} = 14 \text{ ft}$

C3.2 SABRE program input

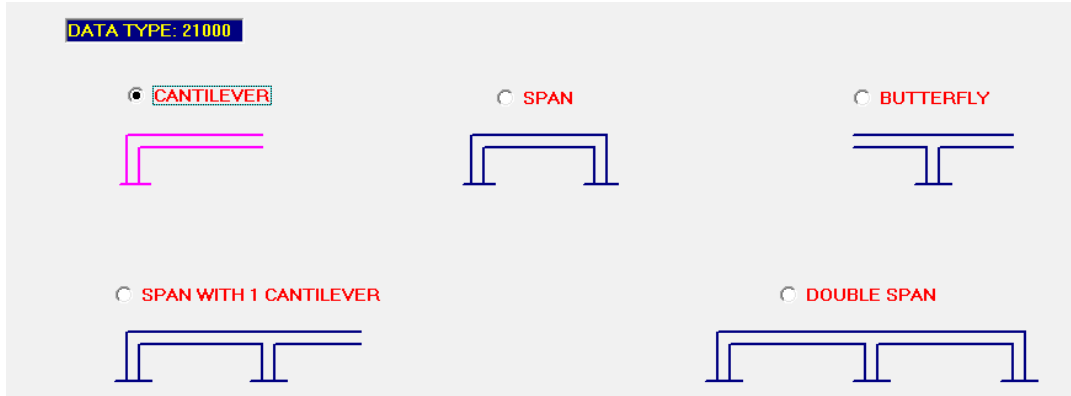


Figure Appendix 24- Configuration selection

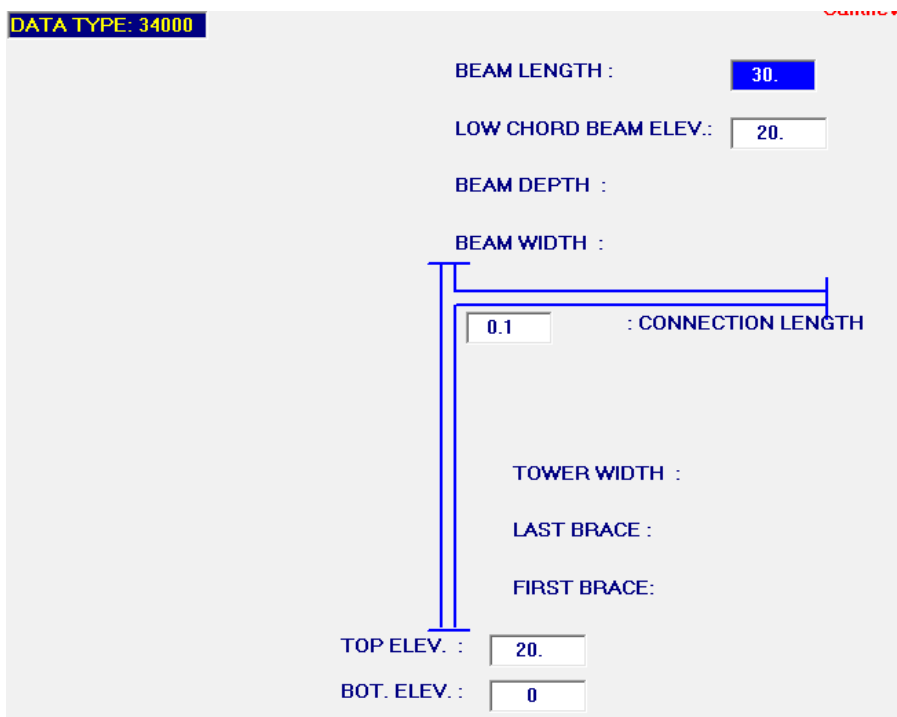


Figure Appendix 25- Structure dimensions

DATA TYPE: 04012

Section Lookup

Sec No	Standard Section			Tubular Shapes			General Sections							
	ID	DTH inch (mm)	WT/FT (Lb./ft)	ID	OUT D inch (mm)	Tw inch (mm)	Kt	ID	WT/FT Kip/ft (KN/m)	S1 inch (mm)	S2 inch (mm)	S3 inch (mm)	S4 inch (mm)	S5 inch (mm)
1				2	24.00	0.688	1.00							
2				2	24.00	0.375	1.00							
3				999										
4				2	24.00	0.375	1.00							
5	W	6	12					9	0	4.000	6.030	0.280		0.230
6				2	24.00	0.375	1.00							
7														
8														

Figure Appendix 26- Definition of sections

DATA TYPE: 07012

VAM ID	Top VAM Coords					Length feet (m)	Supporting Members *			Attached Sign Units			Walkway Units		
	No	Sept.	X feet (m)	Y feet (m)	Z feet (m)		No.	No.	No.	No.	No.	No.	No.	No.	No.
1	5		12.	24.	.7	8.	14			1					
2	5		18.	24.	.7	8.	18			1					
3	5		24.	24.	.7	8.	22			1					
4															

Figure Appendix 27- Definition of VAM members

DATA TYPE: 09012

Sign No.	Dimensions			Slope in/ft (mm/m)	Density k/cf (Kg/m^3)	Lower Left Coord			Drag
	Width feet (m)	Height feet (m)	Thick. inch (mm)			X feet (m)	Y feet (m)	Z feet (m)	
1	24.	8.	48.		0.0065	6.	16.	1.13	
2									
3									

Figure Appendix 28- Sign dimensions

DATA TYPE: 01042

WIND VELOCITY (mph or m/s)

MEAN REGULAR INTERVAL (years)

GUST FACTOR Regular Method - 1.14 (4th or 5th ed.)

ICE LOADS (psf or KPa) ICE LOAD OPTIONS 0- One Side

TRUSS BRACING OPTIONS 0- Moment Bracing (default)

D. L. DETAIL FACTOR

WIND IMPORTANCE FACTOR (default = 1.0; see AASHTO Table 3-2)

For Fatigue Design Only

YEARLY MEAN VELOCITY FOR NATURAL WIND GUST (default = 11.2 mph or 5 m/s)

VEHICLE SPEED FOR TRUCK-INDUCED GUST (default = 65 mph or 30 m/s)

FATIGUE IMPORTANCE FACTOR OPTIONS

GALLOPING (default = 1.0) NATURAL WIND (default = 1.0) TRUCK - INDUCED GUST (default = 1.0)

Figure Appendix 29- Detailed parameters

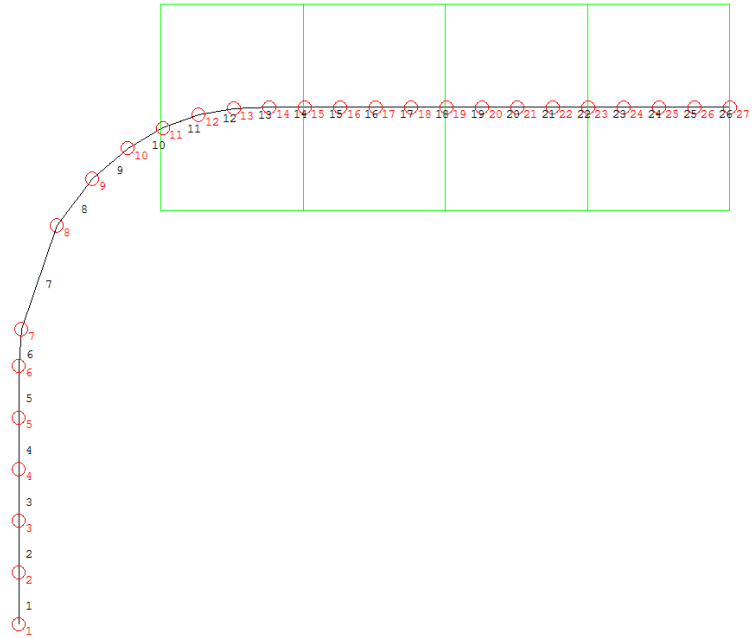


Figure Appendix 30- SABRE graphic output

C3.3 SABRE calculation, NCHRP Report and STAAD calculation comparison

Table Appendix 4- Comparison table of joint loads

EX-9	(Fatigue I)	SABRE	Report	STAAD
Joint Load (kips)	Wind Load Strength Total	8.157	8.833	8.849
	Sign	-	7.32	7.397
	Pole and Arm	-	1.513	1.452
	Natural wind Total	1.72	1.491	1.497
	Sign	1.192	1.188	1.188
	Pole and Arm	0.528	0.303	0.309
	Truck induced Total	1.286	1.288	1.58
	Sign	1.083	1.085	1.085
	Pole and Arm	0.203	0.203	0.202

Table Appendix 5- Comparison table of moments

EX-9	(Fatigue I)	SABRE	Report	STAAD
Moment (K-ft)	Wind Load Strength Total (bending)	157.93	148.21	158.918
	Sign	-	146	147.947
	Pole and Arm	-	2.21	10.971
	Wind Load Strength Total (torsion)	135.72	138.63	135.753
	Sign	-	132	133.152
	Pole and Arm	-	6.63	2.601
	Natural wind Total	31.57	29.8	27.504
	Sign	23.84	23.8	23.8
	Pole and Arm	7.73	6.05	6.05
	Truck induced Total	30.806	30.912	30.896
	Sign	-	26.04	26.035
	Pole and Arm	-	4.872	4.861

Example C4 – HMLT structure

Source: NCHRP Report 718, Example 1

C4.1 Example overview

The support structure for the 80-ft HMLT shown consists of a tapered prismatic tube with access handhole and circular baseplate and is to be designed for fatigue according to Section 11 of the AASHTO Specification. The shape of the tube is a regular polygon with 16 sides. The column-to-baseplate connection is groove welded with the backing ring attached to the base plate, and the handhole detail is of the “doubler” plate type. The location of the HMLT is in a region where the NOAA weather data suggests the annual mean wind speed will not exceed 9 mph, and the lighting plan for the interchange does not require the HMLT to be any closer than 100 feet from the roadway. Furthermore, it is assumed the structure will not be built with a vibration mitigating device.

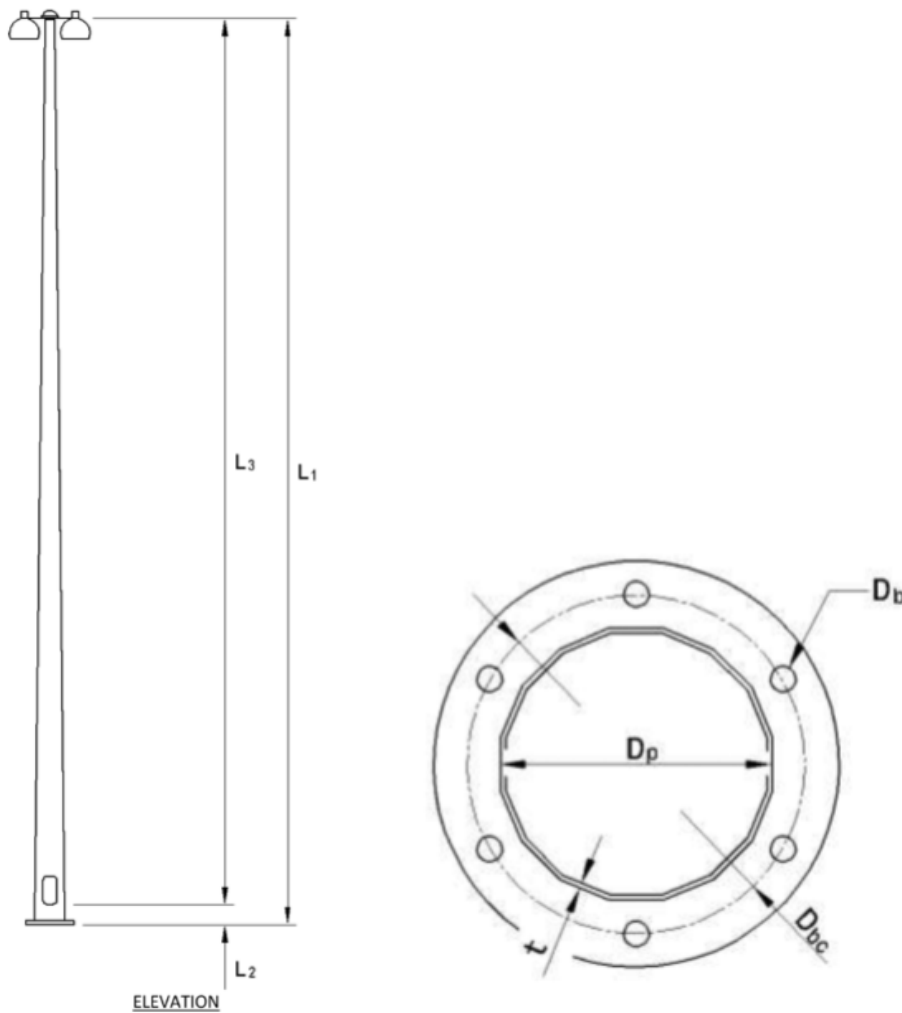


Figure Appendix 31- Elevation view of the HMLT

Structural parameters:

Pole:

Pole height 80 ft

Length from pole baseplate to termination of handhole 2 ft

Length from termination of handhole to luminaire 78 ft

Pole diameter at base (across flats) 18 in

Pole taper rate 0.14 in/ft, thickness 0.25 in

Anchor Bolts:

Number of bolts 6

Bolt group diameter 23 in

Nominal bolt diameter 1.5 in

Luminaire:

Effective projected area on vertical plane, EPA=16 ft²

C4.2 SABRE program input

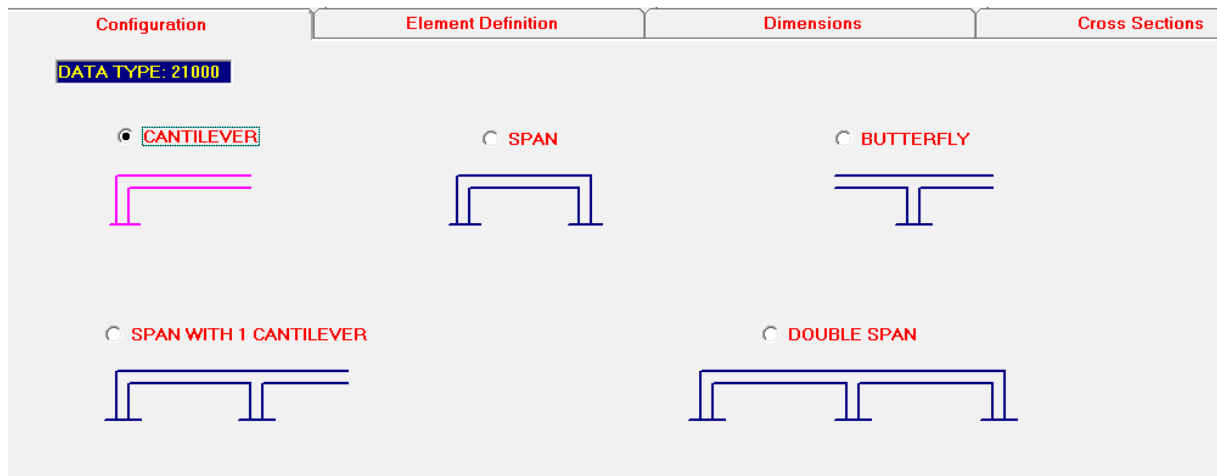


Figure Appendix 32- Configuration selection

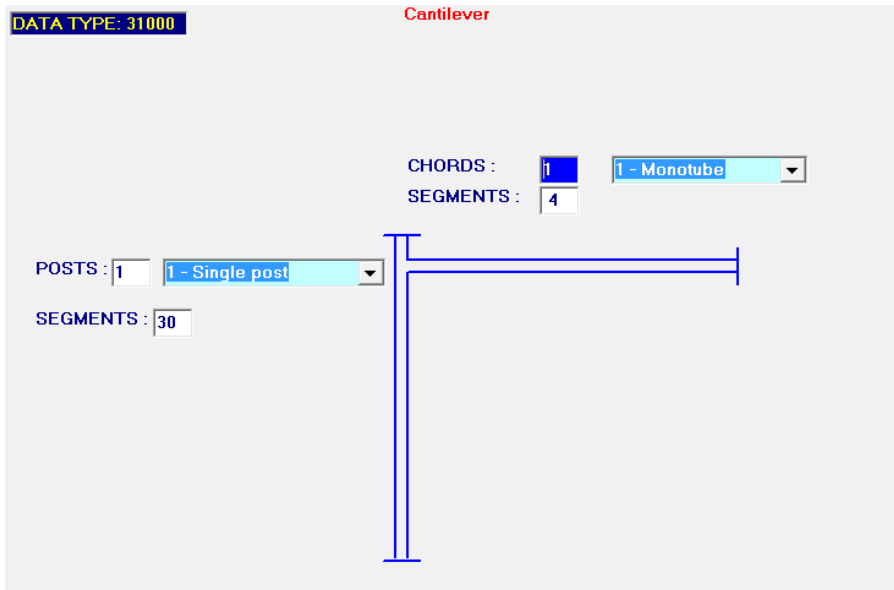


Figure Appendix 33- Element Definition

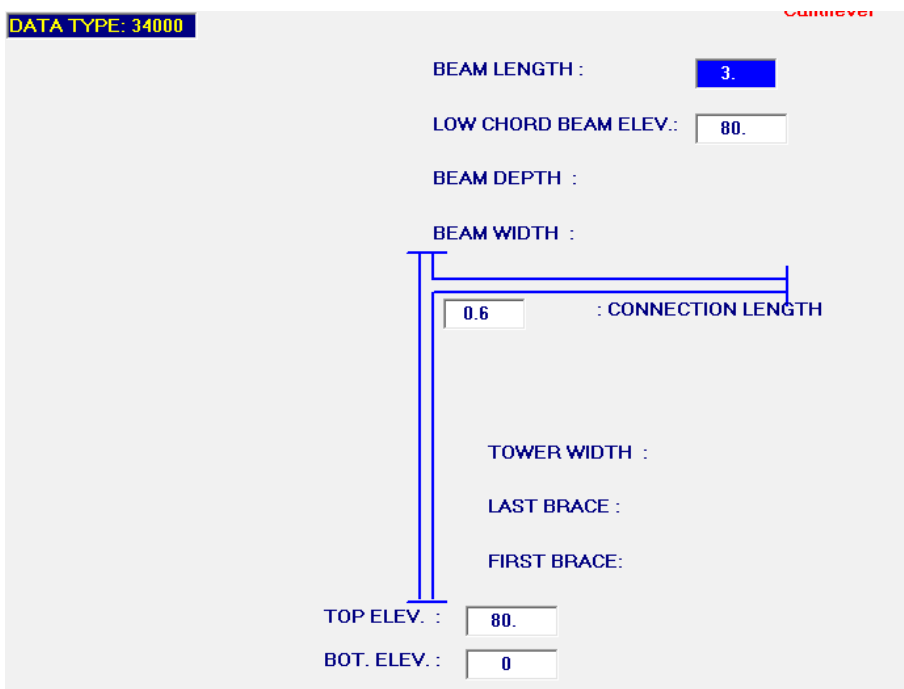


Figure Appendix 34- Structure dimensions

DATA TYPE: 04012				Section Lookup										
Sec No	Standard Section			Tubular Shapes				General Sections						
	ID	DTH inch (mm)	WT/FT (Lb./ft)	ID	OUT D inch (mm)	Tw inch (mm)	Kt	ID	WT/FT Kip/ft (KN/m)	S1 inch (mm)	S2 inch (mm)	S3 inch (mm)	S4 inch (mm)	S5 inch (mm)
1				11	18.	0.25	1.0							
2				11	6.8	0.25	1.0							
3				2	28.32	0.25	1.0							
4				999	0	0	1.0							
5				2	15.	0.25	1.0							
6	W	4	3					9	0	4.060	4.160	0.345		0.280
7				2	11.48	0.188	1.0							
8				2	7.76	0.188	1.0							
9				2	8.14	0.188	1.0							
10				2	5.83	0.188	1.0							
11														

Figure Appendix 35- Definition of sections

DATA TYPE: 07012															
VAM ID	Top VAM Coords					Length feet (m)	Supporting Members *			Attached Sign Units			Walkway Units		
	No	Sect.	X feet (m)	Y feet (m)	Z feet (m)		No.	No.	No.	No.	No.	No.	No.	No.	No.
1	6		2.75	80.	0.1	4.	35			1					
2															
3															
4															

Figure Appendix 36- Definition of VAM members

DATA TYPE: 09012										
Sign No.	Dimensions			Slope in/ft (mm/m)	Density k/cf (Kg/m^3)	Lower Left Coord			Drag Optional Cd	
	Width feet (m)	Height feet (m)	Thick. inch (mm)			X feet (m)	Y feet (m)	Z feet (m)		
1	4.	4.	4.09	0	0.019	0.75	76.	0		
2										
3										

Figure Appendix 37- Sign dimensions

DATA TYPE: 01042

WIND VELOCITY	<input type="text" value="120."/>	(mph or m/s)
MEAN REGULAR INTERVAL	<input type="text" value="50"/>	(years)
GUST FACTOR	<input type="text" value="1.14"/>	<input type="text" value="Regular Method - 1.14 (4th or 5th ed.)"/>
ICE LOADS	<input type="text" value="3.0"/>	(psf or KPa) ICE LOAD OPTIONS <input type="text" value="0"/> <input type="text" value="0- One Side"/>
TRUSS BRACING OPTIONS	<input type="text" value="0"/>	<input type="text" value="0- Moment Bracing (default)"/>
D. L. DETAIL FACTOR	<input type="text" value="1.0"/>	
WIND IMPORTANCE FACTOR	<input type="text" value="0.95"/>	(default = 1.0; see AASHTO Table 3-2)
For Fatigue Design Only		
YEARLY MEAN VELOCITY FOR NATURAL WIND GUST	<input type="text" value="9."/>	(default = 11.2 mph or 5 m/s)
VEHICLE SPEED FOR TRUCK-INDUCED GUST	<input type="text" value="65."/>	(default = 65 mph or 30 m/s)
FATIGUE IMPORTANCE FACTOR OPTIONS	<input type="text"/>	
GALLOPING (default = 1.0)	NATURAL WIND (default = 1.0)	TRUCK - INDUCED GUST (default = 1.0)
<input type="text"/>	<input type="text" value="1.5"/>	<input type="text"/>

Figure Appendix 38- Detailed parameters

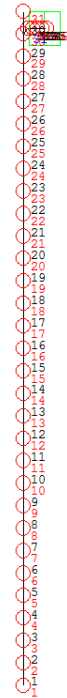


Figure Appendix 39- SABRE graphic output

C4.3 SABRE calculation, NCHRP Report, and STAAD fatigue calculation comparison

Table Appendix 6- Comparison table

HMLT	(Fatigue II)	SABRE	Report	STAAD
Joint Load (kips)	Total	0.628	0.622	0.632
	Sign	0.091	0.093	0.093
	Pole	0.537	0.529	0.539
Bending moment (K-ft)	Base	27.59	25.4	25.83

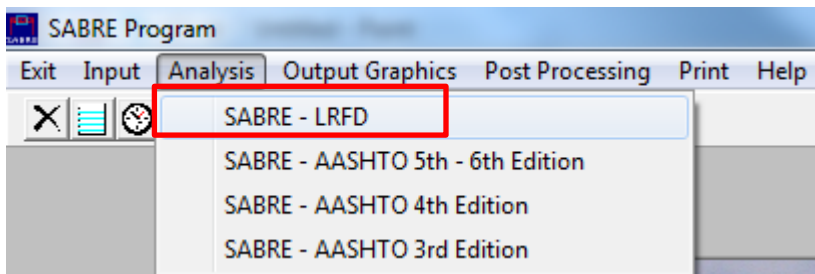
Appendix D SABRE Program with AASHTO LRFD Specifications

D.1 SABRE Input Module for LRFD

Use the same entry Wind Importance Factor for ASD and Directionality Factor for LRFD

Parameters	Additional Joint Loads	Walkways
DATA TYPE: 01042		
WIND VELOCITY	<input type="text" value="120"/> (mph or m/s)	<input type="text"/> (mph or m/s) (SERVICE I, LRFD only)
MEAN REGULAR INTERVAL	<input type="text" value="50"/> (years)	
GUST FACTOR	<input type="text" value="1.14"/> Regular Method - 1.14 (4th or 5th ed.)	
ICE LOADS	<input type="text" value="3.0"/> (psf or KPa)	ICE LOAD OPTIONS <input type="text" value="0"/> 0- One Side
TRUSS BRACING OPTIONS	<input type="checkbox"/> 0- Moment Bracing (default)	
D. L. DETAIL FACTOR	<input type="text" value="1.0"/>	
<div style="border: 2px solid red; padding: 5px;"> ASD: WIND IMPORTANCE FACTOR or <input type="text"/> (default = 1.0; see AASHTO Table 3-2) LRFD: DIRECTIONALITY FACTOR, Kd <input type="text"/> <input type="text"/> (post) <input type="text"/> (others) (post default = 0.95, others = 0.85, see AASHTO LRFD Table 3.8.5-1) </div>		
<u>For Fatigue Design Only</u>		
YEARLY MEAN VELOCITY FOR NATURAL WIND GUST	<input type="text"/>	(default = 11.2 mph or 5 m/s)
VEHICLE SPEED FOR TRUCK-INDUCED GUST	<input type="text"/>	(default = 65 mph or 30 m/s)
FATIGUE IMPORTANCE FACTOR OPTIONS <input type="text"/>		
GALLOPING (default = 1.0)	NATURAL WIND (default = 1.0)	TRUCK - INDUCED GUST (default = 1.0)
<input type="text"/>	<input type="text"/>	<input type="text"/>
<u>Overall Effective Length K (Table 7.4.1 of the User's Manual)</u>		
CASE 1: VERTICAL (POLE TYPE)	<input type="text"/>	(default = 2.0)
CASE 2: VERTICAL (TRUSS TYPE)	<input type="text"/>	(default = 1.2)
CASE 3: HORIZONTAL (POLE AND TRUSS)	<input type="text"/>	(default = 0.65)

D.2 SABRE Analysis Module for LRFD



D.3 SABRE Load Combinations for LRFD

Current Load Combinations

A. Strength Limit State

1. Group 1 – $1.25DL + 1.6LL$ (T8.1, check for T10.1 & T13.1)

B. Extreme Limit State

2. Group 2 – $1.1DC+W$ (Comb 1 +Z) (T8.2, check for T10.2, T13.2)
3. Group 2 – $1.1DC+W$ (Comb 2 +Z) (T8.3, check for T10.3, T13.3)
4. Group 2 – $1.1DC+W$ (Comb 3 +Z) (T8.4, check for T10.4, T13.4)
5. Group 2 – $1.1DC+W$ (Comb 1 -Z) (T8.5, check for T10.5, T13.5)
6. Group 2 – $1.1DC+W$ (Comb 2 -Z) (T8.6, check for T10.6, T13.6)
7. Group 2 – $1.1DC+W$ (Comb 3 -Z) (T8.7, check for T10.7, T13.7)
8. Group 3 – $0.9DC+W$ (Comb 1 +Z) (T8.8, check for T10.8, T13.8)
9. Group 3 – $0.9DC+W$ (Comb 2 +Z) (T8.9, check for T10.9, T13.9)
10. Group 3 – $0.9DC+W$ (Comb 3 +Z) (T8.10, check for T10.10, T13.10)
11. Group 3 – $0.9DC+W$ (Comb 1 -Z) (T8.11, check for T10.11, T13.11)
12. Group 3 – $0.9DC+W$ (Comb 2 -Z) (T8.12, check for T10.12, T13.12)
13. Group 3 – $0.9DC+W$ (Comb 3 -Z) (T8.13, check for T10.13, T13.13)

C. LRFD Strength/Extreme Limit State Combination Checks

- a. Tower and Truss Member Capacities (T11.2 & T12.2)
- b. Tower and Truss Member Maximum Combined Force Ratios (CSR) (T11.4 & T12.4)

D. Service I Limit State

14. Group 4 – $1.0DC+ W_{SE}$ (Comb 1 +Z) (T8.14, check for T9.14)
15. Group 4 – $1.0DC+ W_{SE}$ (Comb 2 +Z) (T8.15, check for T9.15)
16. Group 4 – $1.0DC+ W_{SE}$ (Comb 3 +Z) (T8.16, check for T9.16)
17. Group 4 – $1.0DC+ W_{SE}$ (Comb 1 -Z) (T8.17, check for T9.17)
18. Group 4 – $1.0DC+ W_{SE}$ (Comb 2 -Z) (T8.18, check for T9.18)
19. Group 4 – $1.0DC+ W_{SE}$ (Comb 3 -Z) (T8.19, check for T9.19)

E. LRFD Service Limit State Deflection Checks

- c. Maximum Joint Deflection tables (T9.10)

F. LRFD Fatigue Limit State Checks

20. Group 5 – Galloping (T19.1, T9.11, T10.11, check for T20.1)
21. Group 5 – Natural Wind Gust (T19.2, T9.12, T10.12, check for T20.2)
22. Group 5 – Truck Gust (T19.3, T9.13, T10.13, check for T20.3)