FINAL REPORT
Development of Software for Analysis of Traffic Signal Support Structures

by
Michael D. Symans, Ph.D.
Noel E. Gorab, M.S.
Christos D. Varsamis, M.S.
Christopher M. Zaverdas, Ph.D.

Project No. C-14-03

Submitted to NYSDOT by:
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16. Abstract
In this project, a computer program was developed to perform structural analysis and capacity assessment of existing and proposed span wire and mast arm traffic signal support structures. The structures that can be analyzed include single span wire, box span wire, hanging box span wire, X span wire, single mast arm and dual mast arm. The types of attachments that can be considered include traffic signal heads, with or without backplates, overhead signs and other miscellaneous attachments such as traffic cameras and vehicle detectors. The program performs load analysis and capacity assessment in accordance with the strength and serviceability requirements of the Sixth Edition (2013) of the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*.

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EXECUTIVE SUMMARY

In this project, a computer program (T3SAP) was developed to perform structural analysis and capacity assessment of existing and proposed span wire and mast arm traffic signal support structures. The structures that can be analyzed include single span wire, box span wire, hanging box span wire, X span wire, single mast arm and dual mast arm. The types of attachments that can be considered include traffic signal heads, with or without backplates, overhead signs and other miscellaneous attachments such as traffic cameras and vehicle detectors. The program performs load analysis and capacity assessment in accordance with the strength and serviceability requirements of the Sixth Edition (2013) of the *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*, including the errata dated September 2013 and the 2015 Interim Revisions. and the *NYSDOT Standard Specifications* (Sept. 2015). Loads that are considered include dead, ice, wind and wind-induced fatigue. Strength-related capacity assessment is evaluated via allowable stress design. For proposed traffic signal support structures where the full details on the structure may not be available, a simplified analysis can be performed in which the forces transmitted to the poles are determined and the maximum moment at the base of the poles is estimated. For existing traffic signal support structures, where the full details on the structure are more likely to be available, a detailed analysis can be performed in which internal forces throughout the structure are determined and used as part of a capacity assessment.

1. INTRODUCTION

Traffic signal support structures (span wire structures and mast arm structures) are designed to carry various loads while simultaneously meeting serviceability requirements. These structures come in a variety of configurations such as single span wire, X span wire (two single span wire structures that together form an X-shape in plan), box span wire, hanging box span wire, C- and L-shaped span wire, single mast arm and dual mast arm (some common configurations are shown in Fig. 1.1). The current software used by the New York State Department of Transportation (NYSDOT Span Wire Analysis Program; Version 2.0.0; July 2013) (NYSDOT 2013) is limited to single span wire configurations (although it can be adapted for analysis of multi-span span wire structures) and provides limited information regarding the forces acting on the poles. Consequently, NYSDOT identified the need for a new software program that is capable of analyzing multiple structural configurations and that can perform both structural analysis (to determine loads and resulting internal forces) and capacity assessment (to evaluate whether the structure has sufficient capacity to carry the loads). The software developed as part of this project fulfills this need.

In addition to the NYSDOT Span Wire Analysis Program, there are a variety of other software programs available for structural analysis of traffic signal support structures. These include programs associated with the Florida DOT (ATLAS - Analysis of Traffic Lights and Signal Poles) (BSI n.d.), Ohio DOT (SWISS – Span Wire Signal Support) (ODOT 2010), and Pennsylvania DOT (MASTARM, Span Wire, and Strain Pole) (PennDOT 2003). Such software tends to be primarily relevant to the state in which it was developed (for example, they are hard-coded for environmental loads (ice and wind) specific to the state). The software developed as part of this project (T3SAP – Traffic Signal Support Structural Analysis Program) is based on the nationally-recognized AASHTO Standard Specifications (AASHTO 2013) as well as the NYSDOT Standard Specifications (NYSDOT 2015a).

In this report, the AASHTO Standard Specifications (AASHTO 2013) and the NYSDOT Standard Specifications (NYSDOT 2015a) are referred to extensively and thus will often be cited in the following abbreviated forms: AASHTO Specifications or AASHTO Spec. and NYSDOT Specifications or NYSDOT Spec.

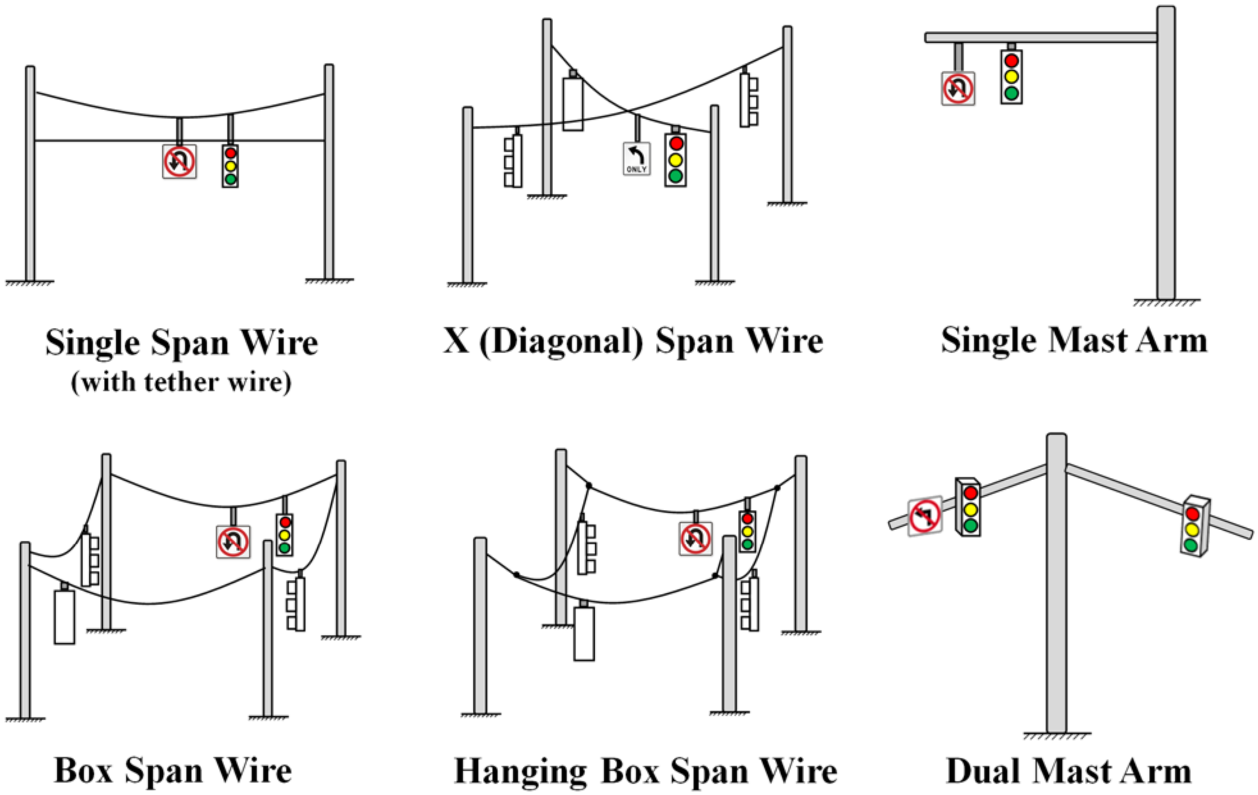

Fig. 1.1 Common traffic signal support structure configurations

2. DESCRIPTION OF T3SAP SOFTWARE

The T3SAP (Traffic Signal Support Structural Analysis Program) software performs structural analysis and capacity assessment of span wire and mast arm structures of various configurations. Structural analysis includes computation of loads and the resulting internal forces and deflections. Capacity assessment includes evaluation of strength criteria (demand-capacity stress ratios) for major components of the structure and serviceability criteria (demand-capacity deflection ratios). The specific criteria used in the demand-capacity computations are those defined in the sixth edition of the AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals" (AASHTO 2013), including the errata dated September 2013 and the 2015 Interim Revisions, and the September 1, 2015 version of the NYSDOT "Standard Specifications" (NYSDOT 2015a). Note that the NYSDOT Standard Specifications references the AASHTO Standard Specifications for defining loads and associated stresses. The loads and strength-related capacities defined in the AASHTO Specification are based on the Allowable Stress Design (ASD) method.

There are two main types of analysis that can be performed with T3SAP. The first type, defined in the software as *Type I Analysis*, is an analysis that does not include capacity assessment. Such an analysis can be useful in the design phase of a new structure where limited information is available. For example, for a span wire structure, a common situation is that the poles have not been designed (this may be the case when the design of the traffic signal poles has been delegated to the pole manufacturer) but the user knows the basic geometry of the structure (height of poles, length of spans) and what signals, signs and other attachments need to be supported by the span wires and the location of those attachments. In such cases, the software performs a limited analysis to determine forces in the span wire cables, forces applied to the poles by the cables, and an estimate of the overturning moment at the base of the poles (overturning moment is needed for foundation design). As another example, for a mast arm structure, a common situation is that poles and mast arms have not been designed (this may be the case when the design of the traffic signal poles and mast arms has been delegated to the pole/mast arm manufacturer) but the user knows the basic geometry of the structure (height of pole, length of arms) and what signals, signs and other attachments need to be supported by the mast arms and the location of those attachments. In such cases, the software performs a limited analysis to estimate forces applied to the pole by the mast arms and to estimate the overturning moment at the base of the pole (overturning moment is needed for foundation design).

The second type of analysis, defined as *Type II Analysis* in the software, is an analysis that includes capacity assessment (evaluation of both strength and serviceability limit states). Such an analysis can be useful for design of new structures or for evaluating whether an existing structure has sufficient capacity to support additional attachments. This type of analysis would be applicable, for example, to existing structures in which detailed information is available on the poles and any mast arms and their connections (e.g., geometry of poles/mast arms, hand hole location and geometry). In such cases, the software performs a more comprehensive analysis to determine internal forces and associated stresses, uses the stresses to check capacity for strength-related limit states, and uses computed deflections to check if prescribed limits are exceeded for serviceability limit states.

3. RESEARCH METHOD

This project began with a review of existing software related to analysis and capacity assessment of traffic signal support structures, including the software that the NYSDOT had developed for analysis of single span wire structures (NYSDOT 2013). Next, relevant specifications and design guidelines were examined to determine what methods would be used in the software for determining loads, performing analysis and evaluating capacity. The primary documents that were reviewed include the following AASHTO and NYSDOT publications:

AASHTO (2013). “Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals,” Sixth Edition (2013) with 2015 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, DC.

NYSDOT (2015a). “New York State Department of Transportation – Standard Specifications,” September 1, 2015, New York State Department of Transportation, Engineering Division, Albany, NY.

NYSDOT (2015b). “New York State Department of Transportation – Standard Sheets,” September 1, 2015, New York State Department of Transportation, Engineering Division, Albany, NY.

NYSDOT (2013). “Span Wire Analysis Program,” User’s Manual (Version 2.0.0), New York State Department of Transportation, Office of Design, Albany, NY.

NYSDOT (1983). “Method for Calculating the Loads Applied to Span Wire Traffic Signal Poles (EI 83-38),” Facilities Design Division, New York State Department of Transportation, September 1983.

NYSDOT (1989). “Method for Calculating the Loads Applied to Span Wire Traffic Signal Poles: Non-Tethered (EI89-003),” Structures Division, New York State Department of Transportation, February 1989.

The software was developed in accordance with the requirements defined in the AASHTO Standard Specifications (2013) and the NYSDOT Standard Specifications (2015a). Specifically, the relevant sections of these documents are as follows:

<u>AASHTO</u>
Section 1 – Introduction
Section 2 – General Features of Design
Section 3 – Loads
Section 4 – Analysis and Design Considerations
Section 5 – Steel Design
Section 10 – Serviceability Considerations

Section 11 – Fatigue Design
Appendix A – Analysis of Span-Wire Structures
Appendix B – Design Aids

<u>NYSDOT</u>
Section 724-02 - Span Wire
Section 724-03 - Traffic Signal Poles
Section 724-04 - Traffic Signal Heads

One challenge with implementing these specifications in software is the application of wind load to the structure. Simplified methods for wind load analysis of span wire structures are defined in Section 3 (Loads) of the AASHTO Specifications but the method does not clearly distinguish between analysis for tethered and non-tethered spans. Furthermore, an accurate wind load analysis for a specific span wire or mast arm structural configuration and geometry requires that the critical wind load direction be determined. In most cases, the critical wind direction is not readily identifiable. Thus, numerical studies were performed to evaluate critical wind load directions for typical span wire configurations and to develop a simplified method for applying the wind load. For mast arm structures, the relative simplicity of the structures allows multiple wind directions to be explicitly considered and thus the critical wind load direction can be determined (no need for developing a simplified method for applying the wind load).

Having defined the loads on the structures, load combinations are employed and utilized to determine the internal forces in the structure based on static equilibrium. Next, the stress demand and stress capacity are determined at critical locations within the structure. Finally, deflections are determined. The load combinations, stress capacities and deflection limits are defined in accordance with the AASHTO Specifications and NYSDOT Specifications and thus relevant sections of those documents were reviewed and implemented in the software.

To verify the accuracy of the T3SAP software, verification studies were performed (see Appendix I and II for two of the studies). For the verification studies shown in the appendices, a single-span span wire structure (Appendix I) and a single mast arm structure (Appendix II) structure was analyzed via manual hand computations and via commercial structural analysis software (complex structure configurations cannot be readily analyzed via hand computations). The results from those analyses were then compared to the output from T3SAP.

Note that, prior to performing the T3SAP verification studies, the output from the NYSDOT Span Wire Analysis Program (NYSDOT 2013) was checked by performing manual computations using various existing structures in the field (denoted as benchmark structures). These computations were performed using the NYSDOT Engineering Instructions (NYSDOT 1983 and 1989) for analysis of tethered (1983) and non-tethered (1989) span wire structures (later, it was decided, in consultation with NYSDOT, that T3SAP should perform span wire analysis in accordance with the method described in Appendix A of the AASHTO Specifications). The comparisons revealed that there are errors in the Span Wire Analysis

Program (one related to computation of the traffic signal conduit dead load (conservative error) and another to the computation of sign conduit dead load (unconservative error)). Furthermore, there were some minor discrepancies between the Span Wire Analysis Program and the associated user manual. With these errors and discrepancies, and the subsequent decision to perform span wire analysis in accordance with the AASHTO Specification rather than the NYSDOT Engineering Instructions, it was recognized that the Span Wire Analysis Program output could not be used to verify the accuracy of the output from T3SAP. Thus, for span wire structures, verification of the output from T3SAP was performed using manual hand computations for relatively simple structures where hand computations are feasible (as noted above, see Appendix I for an example of a verification study for a span wire structure).

For mast arm structures, a few structures in the field were identified as benchmark structures for verifying the accuracy of the output of T3SAP. However, there was insufficient information available to fully define those structures (e.g., missing information about the specific types of attachments). Furthermore, in some cases, the structures were of a complex configuration such that they did not lend themselves well to evaluation via manual hand computations. Consequently, manual computations were not performed for these particular structures. Rather, for purposes of verifying the accuracy of the output from T3SAP, hand computations were performed for relatively simple mast arm structures (as noted above, see Appendix II for an example of a verification study for a mast arm structure).

4. LOADS AND LOAD COMBINATIONS

4.1 INTRODUCTION

According to the AASHTO Specification (Section 3), the types of loads that must be considered for analysis of all traffic signal support structures are dead load (DL), ice load (Ice or I), and wind load (W). In addition, for cantilevered structures (mast arm structures), fatigue loading must be considered. Furthermore, these loads must be applied in certain combinations defined by Group Load Combinations (see Table 4.1). Each part of the structure must be designed for the combination that produces the maximum load effect, using allowable stresses that are increased as indicated in Table 4.1.

Table 4.1 Group load combinations

Group Load	Load Combination	Percentage of Allowable Stress
I	DL	100
II	DL + W	133
III	DL + Ice + $^{1}/_{2}$W	133
IV	Fatigue	See AASHTO Spec. Sect. 11

In order for the T3SAP software to perform computation of loads, certain assumptions need to be made. Because weights and areas of attachments vary from manufacturer to manufacturer and it is not necessarily known in the design phase which manufacturer will supply an attachment, it is impractical to attempt to use exact weight and area values in the analysis. Therefore, the software uses values of weight and area that are intended to represent upper bound values for a given attachment. The upper bound values were provided by NYSDOT and included an evaluation of attachments from manufacturers with which the NYSDOT is approved to issue contracts.

4.2 DEAD LOAD

Dead load consists of the weight of all structural components (poles, arms, etc.) and attachments (traffic signals, signs, etc.). The dead load values for structural components are computed by T3SAP using the geometry of the components and the unit weight of steel. The dead load values for attachments were obtained from various sources, including Appendix A of the User Manual for the NYSDOT Span Wire Program (NYSDOT 2013) and direct communication with NYSDOT. As noted above, the dead load values for attachments are intended to represent upper bound values. A summary of the dead load values used in T3SAP is provided in the T3SAP User Manual.

4.3 ICE LOAD

Ice load is defined in the AASHTO Specification as 3.0 psf (pounds force per square foot) applied around the exposed surfaces of all elements of the structure except, for sign panels, it is only applied on one face. This value is based on an 0.6 in. thickness of ice at a unit weight of 60 pcf (pounds force per cubic foot).

4.4 WIND LOAD

Wind load is the pressure of the wind acting horizontally on all elements of the structure. The design wind pressure specified in Section 3.8.1 of the AASHTO Specifications is given by

$$P_z = 0.00256 K_z G V^2 I_r C_d \quad \text{(psf)} \tag{4.1}$$

This equation is rooted in fundamental fluid flow theory. The basic wind speed, V, is the nominal design 3-second gust wind speed at a height of 33 ft above the ground and for a 50-year mean recurrence interval. Values of basic wind speed for counties in New York State are provided in Table 4.2 (values were specified by NYSDOT and appear to be consistent with the basic wind speed map in the AASHTO Spec.). The wind importance factor, I_r, relates to the specified design life of the structure. Since the recommended design life of traffic signal support structures is 50 years, this factor is typically taken as 1.0. The height and exposure factor, K_z, at a height of 33 ft is 1.0. Since the height of 33 ft represents a typical upper bound for traffic signal support structures and K_z increases with height, the value of this factor is conservatively taken as 1.0 throughout the entire height of the traffic signal support structure. The gust effect factor, G, adjusts the effective pressure for the dynamic interaction of the structure with the gustiness of the wind. The minimum value for this factor is 1.14 and the commentary of the AASHTO Specification recommends that the value 1.14 be used for design of traffic signal support structures. The drag coefficient, C_d, adjusts the effective pressure to account for the geometry of the element to which the pressure is applied. Values of the drag coefficient are quite variable, and therefore the values specified in the AASHTO Specifications represent conservative values based on experimental tests. It is particularly difficult to predict the drag coefficient on traffic signals due to the combination of aerodynamic and non-aerodynamic shapes and open spaces, especially when the signals are free to rotate in an untethered span wire structure. The drag coefficient recommended for signals may be quite conservative for free swinging traffic signals, and therefore its value is permitted to be modified for such signals in accordance with experimental data. The value of the drag coefficient that is recommended in the AASHTO Specifications for traffic signals is 1.2.

The wind loads acting horizontally on a structure are determined using the effective projected area (EPA) of each element of the structure which is equal to the actual area multiplied by the drag coefficient. However, the actual area subjected to wind pressure may be difficult to determine for complex shapes. For example, the shape of a single traffic signal head is complex and is even more so when one bracket supports multiple signal heads. Thus, it would be difficult and impractical to use the actual shape of the signal head to determine the EPA. Therefore, in the T3SAP software, signal head assemblies are approximated as rectangular prisms as shown in Fig. 4.1. Computation of the EPA for these rectangular prisms depends on wind load direction which in turn depends on the type of structure being analyzed. This issue is explained in detail in Section 5 of this report.

Table 4.2 Basic wind speed values for counties in New York State

County in New York State	Basic Wind Speed, V (mph)
Suffolk, Nassau	120
New York (Manhattan), Kings (Brooklyn), Bronx, Richmond (Staten Island), and Queens	110
All other counties (including those in Special Wind Region)	90

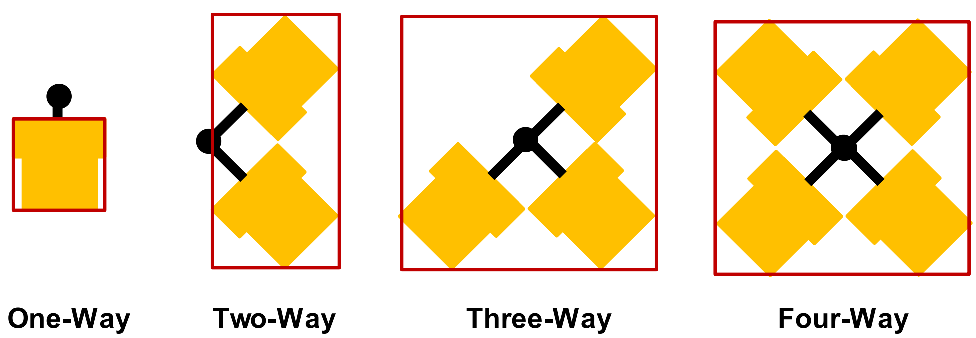

Fig. 4.1 Plan view of traffic signal assemblies illustrating approximate method to determine effective projected area using rectangular prisms.

4.4.1 Wind Load Direction for Maximum Effect

According to AASHTO Specification Section 3.9.3, wind load for single span wire structures (single span wire attached to each pole) should be applied orthogonal to the span for maximum effect. For mast arm structures, the wind load should be applied orthogonal to the arm for design of the arm (Section 3.9.2) and in the direction that maximizes the wind load effect for design of the pole (Section 3.9.3). For other types of span wire structures (e.g., box span structures in which more than one span wire is attached to each pole), the wind load direction that maximizes the effects on the structure must be determined (Section 3.9.3). In an effort to develop a simple procedure for analysis of box span structures, a comprehensive analysis of twenty theoretical box span structures with various geometries (skew angles of 0, 10, 20 and 30 degrees) and various attachments and attachment locations was performed by Gorab (2016) in which wind was applied along multiple directions (direction defined by a 5-degree increment over a 270 degrees for a total of 55 wind load directions) to obtain peak forces in the structure (see Figure 4.2 for an example of the analysis results). Next, simplified methods of applying wind loads were conceived of in an attempt to capture the maximum effects of the wind (straight horizontal continuous and dashed lines in Fig. 4.2 where "full" means full wind pressure orthogonal to a span, "half" means half of the full wind pressure applied orthogonal to a span, "half + full" means half and full wind pressures on adjacent spans and orthogonal to the spans, "full plus full" means full pressure on adjacent spans and orthogonal to the spans, and side area included and excluded refers to whether or not the wind pressure on the effective projected side area of an attachment was considered). An illustration of the "full plus full" load case for evaluation of load applied to Pole B is shown in Fig. 4.3. In this figure, the position of the poles (A, B, C, D) are changed to represent different degrees of intersection skew (angle *a* is defined as the skew angle, angle *b* is maintained at 90 degrees, and all spans have equal length). The dotted lines in Fig. 4.2 represent an "exact" analysis where the resultant wind load is determined for various wind directions over a range of 180 degrees. As can be seen, the "full plus full" wind load application with side area excluded produces a maximum resultant wind force that is nearly identical to the "exact" analysis. Furthermore, the "full plus full" wind load application with side area included exceeds the exact analysis for the full 180-degree range. A detailed discussion of the analysis method and results is provided in Gorab (2016) and led to the "full plus full" wind load application being recommended to the NYSDOT and subsequently adopted in the T3SAP software (details provided below for each type of traffic signal support structure).

The degree of conservativeness in the simplified wind load application is illustrated in Fig. 4.4 where it can be seen that, as expected, the simplified wind load application (full load on adjacent spans) with side area excluded is less conservative (lower span wire tension) than the case with side area included (higher span wire tension). Note that the unconservative results shown in Fig. 4.4 are based on comparison with a conservative approximation (linear analysis of span wire structure) rather than an exact rigorous solution (nonlinear analysis of span wire structure) and thus many, or all, of the unconservative results are likely to be conservative relative to the exact solution.

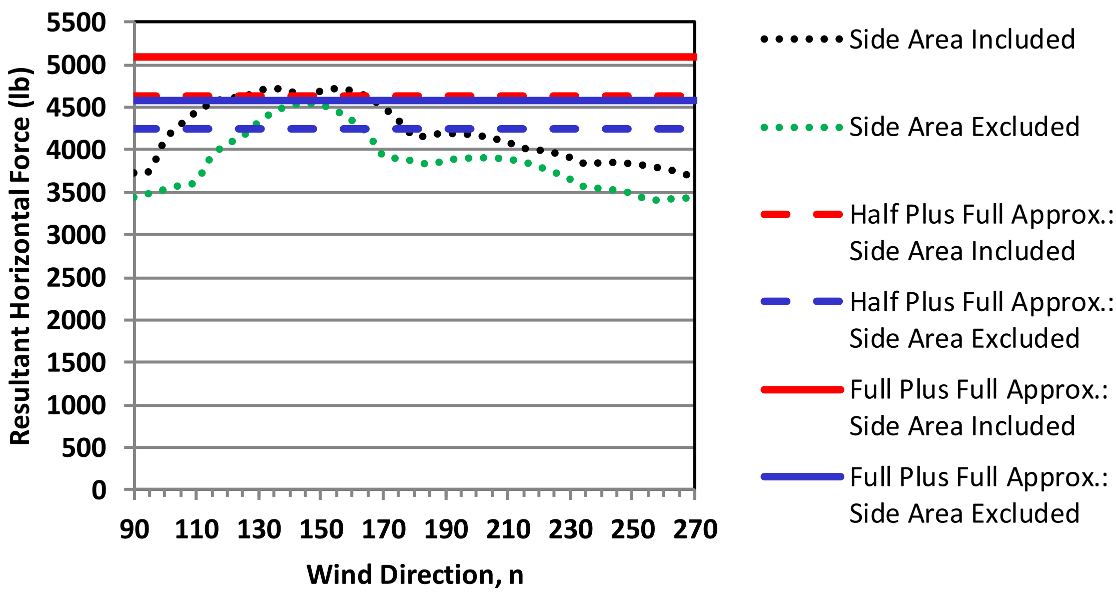

Fig. 4.2 Example of results from wind load study of box span structures

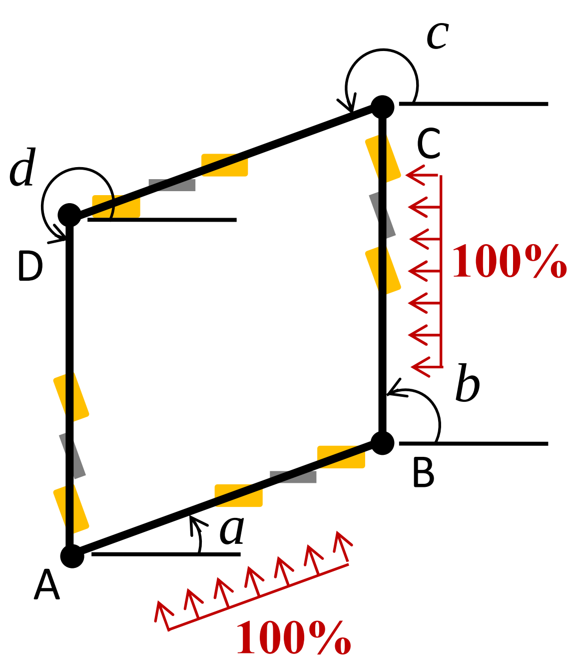

Fig. 4.3 – Illustration of simplified "full plus full" application of wind load to a box span structure (wind loads shown are applied over full length of the span)

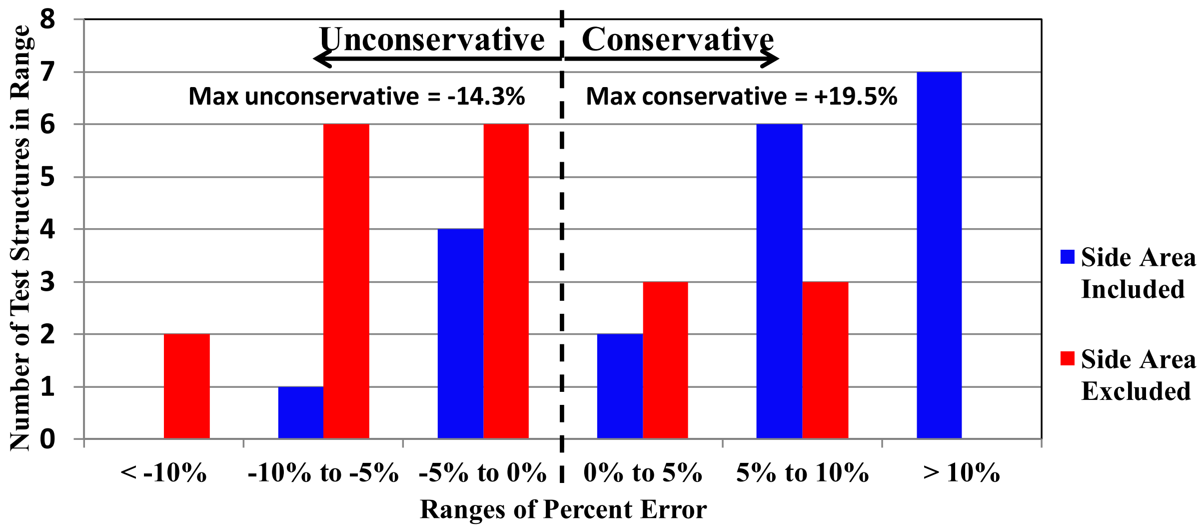

Fig. 4.4 – Error in cable tension for twenty theoretical box span structures analyzed using simplified wind load application (full plus full) relative to wind applied along critical direction.

Based on the work by Gorab (2016), the final recommendations for application of wind loads are as follows (these recommendations were approved by NYSDOT and subsequently adopted in the T3SAP software):

- Effective projected area (EPA) of signal heads. Signal heads approximated as rectangular prisms with side area defined as:
  - 90% of signal front area for signals without backplates
  - 35% of signal front area for signals with backplates (since backplate blocks a portion of the side area)

- Tethered Span Wire Structures (single span)
  - Full wind load applied normal to span.
  - Span analysis performed per AASHTO Specification Appendix A.
  - All wind load assumed to be transferred to main wire (even if tether wire breaks, no wind load reduction due to rotation).
  - Front and side area of attachments included in EPA computations. Attachments assumed to be oriented 45 degrees relative to span wire (square intersection).

- Tethered Span Wire Structures (multi-span box shape)
  - Full wind load applied normal to two adjacent spans (not concurrently in the sense that wind load normal to one span is assumed to act independently of the wind load normal to the adjacent span).
  - Each span analyzed as a single span per AASHTO Specification Appendix A.
  - All wind load assumed to be transferred to main wire

(even if tether wire breaks, no wind load reduction due to rotation).
  - Pole load is resultant of forces exerted by each wire.
  - Front and side area of attachments included in EPA computations.

- Non-Tethered Span Wire Structures (single span)
  - Wind loads reduced to account for attachment rotation:
    a) Estimate angle of rotation based on wind speed.
    b) Reduce EPA of attachments according to angle of rotation
    c) For traffic signals, modify drag coefficient based on wind speed
  - Angle of rotation and drag coefficient based on experimental test results.
  - Full wind load applied normal to span.
  - Span analysis performed per AASHTO Specification Appendix A.
  - Front and side area of attachments included in EPA computations. Attachments assumed to be oriented 45 degrees relative to span wire (square intersection).

- Non-Tethered Span Wire Structures (multi-span box shape)
  - Wind loads reduced to account for attachment rotation:
    a) Estimate angle of rotation based on wind speed.
    b) Reduce EPA of attachments according to angle of rotation
    c) For traffic signals, modify drag coefficient based on wind speed
  - Angle of rotation and drag coefficient based on experimental test results.
  - Full wind load applied normal to two adjacent spans (not concurrently).
  - Each span analyzed as a single span per AASHTO Specification Appendix A.
  - Pole load is resultant of forces exerted by each wire.
  - Only front area of attachments included in EPA computations.

- Single Mast Arm Structures
  - Mast arm forces: Full wind load applied normal to arm.
  - Pole forces: Full wind load applied in multiple directions (over 180 degree range at 5 degree intervals). Wind direction producing largest total wind load used to determine maximum forces (shear, bending moment, and twisting moment) acting on pole.

- Dual Mast Arm Structures
  - Mast arm forces: Full wind load applied normal to each mast arm.
  - Pole forces: Full wind load applied in multiple directions (over 180 degree range at 5 degree intervals). Critical wind directions determined for shear, bending moment, and twisting moment applied to pole.

4.5 FATIGUE LOADING

Fatigue loads are wind-induced loads that cause vibrations in cantilevered and non-cantilevered support structures which may lead to fatigue cracks. Fatigue loads are applied to these support structures as equivalent static loads. The three types of fatigue loading defined in the AASHTO Specifications are galloping, natural wind gust, and truck-induced gusts. Each of these fatigue loads is applied separately. Furthermore, the AASHTO Specification states that fatigue loading is not applicable to span wire structures. Thus, if the type of structure is a mast arm structure, the user is given the option to select any of the three types of fatigue loads. If a fatigue load is selected, the user must also select the fatigue importance factor. To simplify the analysis for truck-induced gust loading, only attachments with a significant projected area on a horizontal plane (i.e., Variable Message Signs (VMS)) are considered to contribute to truck-induced gust loading.

4.5.1 Galloping

Galloping results in large-amplitude, resonant oscillations in a plane normal to the direction of wind flow. The equivalent static shear pressure range due to galloping is

$$P_G = 21 I_f \text{ (psf)} \tag{4.2}$$

where I_f is the fatigue importance factor. This static shear pressure range is applied vertically to the surface area, as viewed in elevation, of all sign panels and traffic signal heads rigidly mounted to the cantilevered horizontal support. A pole with two 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. Galloping may be neglected if the structure is equipped with effective vibration mitigation devices.

4.5.2 Natural Wind Gust

Because of the inherent variability in the velocity and direction of wind, natural wind gusts are the most basic wind phenomena that may induce vibrations in wind loaded cantilevered structures. The equivalent static natural wind pressure range is

$$P_{NW} = 5.2 C_d I_f \text{ (psf)} \tag{4.3}$$

where C_d is the applicable drag coefficient and I_f is the fatigue importance factor. This natural wind gust pressure range is applied in the horizontal direction to the EPA of all elements of the structure and must be considered for any direction.

4.5.3 Truck-Induced Gust

The passage of trucks beneath cantilevered support structures may induce gust loads on the attachments mounted to the horizontal support of cantilevered support structures. The equivalent static truck gust pressure range is

$$P_{TG} = 18.8 C_d I_f \text{ (psf)} \tag{4.4}$$

where C_d is the applicable drag coefficient and I_f is the fatigue importance factor. Although truck-induced gust loads are applied in both horizontal and vertical directions, horizontal support vibrations caused by forces in the vertical direction are most critical. Therefore, the equivalent static gust pressure range is applied in the vertical direction to the area of the horizontal support, signs, and other attachments, projected on a horizontal plane. Consideration of truck-induced gust loading has been included in the AASHTO Specifications due to recent vibration problems on sign structures with large projected areas in the horizontal plane (i.e., variable message sign (VMS) structures). A pole with two horizontal cantilever arms may be designed for truck gust loads applied separately to each individual arm and need not consider truck gust loads applied simultaneously to multiple arms. For typical cantilevered traffic signal and sign support structures (structures without VMS signs), the areas projected on a horizontal plane are insignificant and thus truck-induced gust loads are expected to be minor. Furthermore, the AASHTO Specifications state that "truck-induced gust loading shall be excluded unless required by the owner for the fatigue design of overhead traffic signal structures."

4.5.4 Fatigue Importance Factor

The fatigue importance factor accounts for the risk of hazard to traffic and damage to property. The AASHTO Specification provides importance factor values (ranging from 0.3 to 1.0) for each type of fatigue loading based on a fatigue importance category. There are three importance categories. Structures classified as Category I present a high hazard in the event of failure. It is recommended that all structures without effective vibration mitigation devices on roadways with a speed limit larger than 35 mph and average daily traffic exceeding 10,000 or average daily truck traffic exceeding 1,000 be classified as Category I structures. Structures may also be classified as Category I if the cantilevered span is longer than 50 ft., if the cantilevered span is supporting a VMS, or if the structure is located in an area that is known to have wind conditions that are conducive to vibration. It is recommended that structures be classified as Category III (low hazard) if they are located on roads with speed limits less than 35 mph. It is recommended that structures not meeting the criteria for Category I or Category III be classified as Category II.

5. STRUCTURAL ANALYSIS

5.1 INTRODUCTION

For structural analysis purposes, an idealized model of the structure is developed in T3SAP based on the user-specified elevation and plan geometry of the structures. The ideal model assumes all members are perfectly straight and vertical members are perfectly plumb. Using this model and the applied loads, T3SAP computes internal forces (axial force, shear force, bending moment, twisting moment) at key locations within the structure. For poles and mast arms, the internal forces are obtained using static equilibrium applied to the undeformed geometry of the structure (i.e., first-order elastic analysis). This approach to structural analysis is consistent with Section 4.5 of the AASHTO Specification which states that "The theory of elastic structural analysis shall be used for determining the maximum load effects (axial, shear, bending and torsion) …". In addition, per Section 4.8 of the AASHTO Specification, second-order moments in poles (P-delta effects) can be considered in an approximate manner via a coefficient that modifies the allowable bending stress. For span wire analysis, the cable force at each pole attachment is determined via the *Simplified Method* (explained below) in Appendix A of the AASHTO Specification (except that, instead of defining dead, ice and wind loads associated with the span wire as concentrated loads at equally spaced intervals along the span, these loads were defined as concentrated loads at the same locations as the span wire attachments).

5.2 LOCAL AND GLOBAL COORDINATE SYSTEMS

To keep the analysis organized in the T3SAP software, each individual element has a local coordinate system and the complete structure has a global coordinate system, with both being right-handed coordinate systems. Examples of local coordinate systems are shown in Fig. 5.1 and 5.2 for span wire structures and in Fig. 5.3 and 5.4 for mast arm structures (local coordinate system axes are denoted by lowercase letters x, y and z). Note that the local coordinate system shown in Fig. 5.2 is the same as that defined in Appendix A of the AASHTO Specification. Examples of global coordinate systems are shown in Fig. 5.5 and 5.6 for a span wire and mast arm structure, respectively (global coordinate system axes are denoted by uppercase letters X, Y and Z).

5.3 DESCRIPTION OF ANALYSIS PROCEDURE FOR SPAN WIRE STRUCTURES

5.3.1 Analysis of Single-Span Span Wire Structures

Among span wire structures, single-span configurations are very common. At an intersection, a single-span span wire structure, positioned diagonally across the intersection, may be sufficient to control all traffic. Sometimes a second single-span span wire structure is added to the intersection along the other diagonal (the two span wires may be connected near the center of the intersection). A distinguishing feature of these single-span span wire structures is that there is only one span wire attached to each pole.

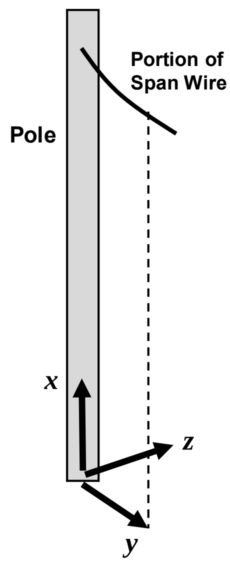

Fig. 5.1 Local coordinate system for pole of span wire structure

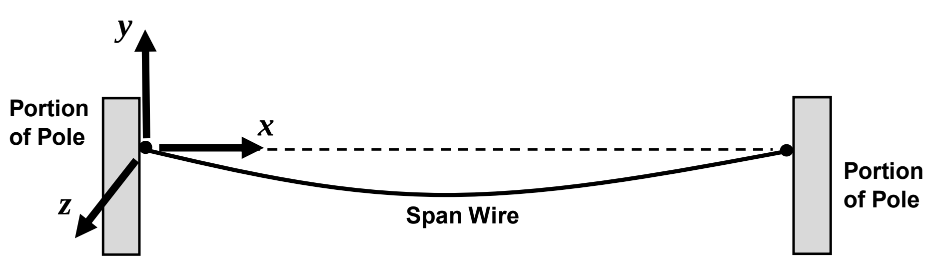

Fig. 5.2 Local coordinate system for span wire of span wire structure

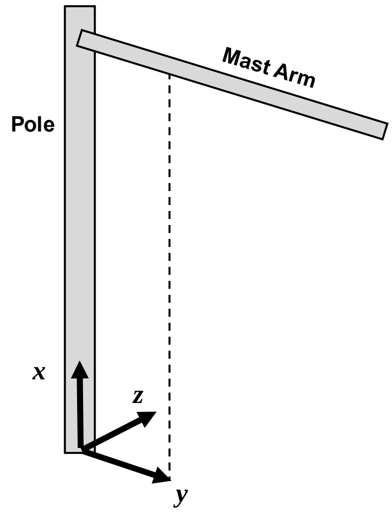

Fig. 5.3 Local coordinate system for pole of mast arm structure

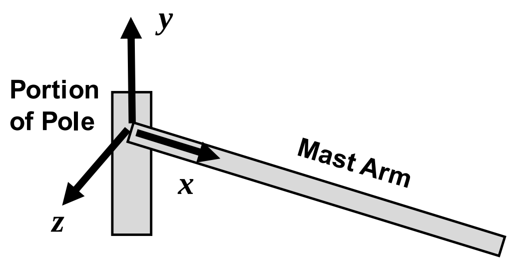

Fig. 5.4 Local coordinate system for arm of mast arm structure

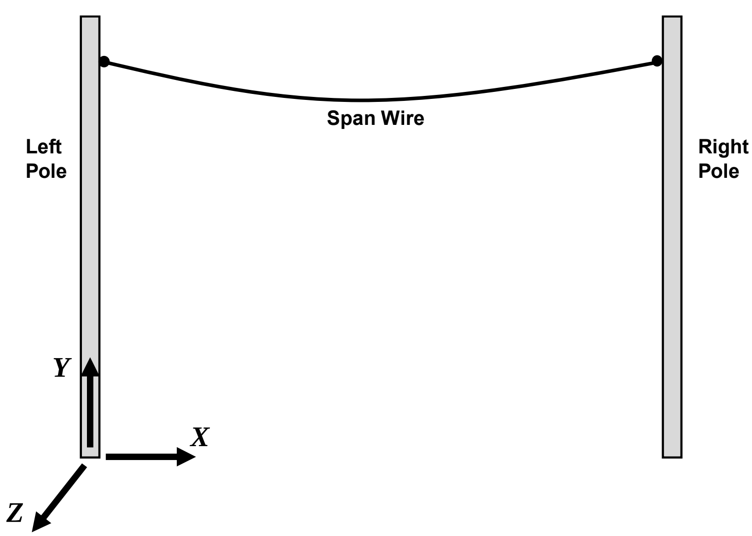

Fig. 5.5 Global coordinate system for span wire structure

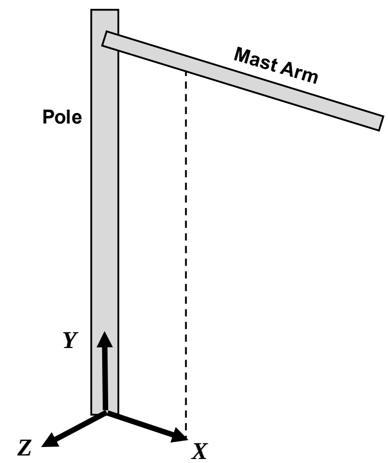

Fig. 5.6 Global coordinate system for mast arm structure

All of the analyses for span wire structures make use of the user-defined sag value which is the largest vertical distance, defined as a percentage of the span length, from a horizontal line between the connections at the poles to a point on the wire (sag is defined for the dead load condition).

5.3.1.1 Tethered Span Wire Structures

For tethered span wire structures, the *Simplified Method* of analysis described in Appendix A.7 of the AASHTO Spec. is used in T3SAP to determine the maximum force applied to each pole from the wire. In this method, the analysis is simplified by assuming that the poles are rigid, which is a conservative assumption since any flexibility of the poles will reduce the tension in the span wire and thus reduce the internal forces in the poles. A two-dimensional static analysis is first performed for the Group I load combination (Dead only). Next, a three-dimensional static analysis, which involves iteration, is performed for the Group II (Dead + Wind) and III (Dead + Ice + ½ Wind) load combinations. Note that, per Commentary C4.7 of the AASHTO Spec., due to the "nonlinear relationship between geometry and forces in span wires, superposition principles should not be applied to combine the effects of different loads" and thus the "analysis should be performed considering a single load case with all loads acting simultaneously." Thus, for each of the load combinations that include wind loading (i.e., Group II and III), a single load case is considered for analysis of the span wire. For load combinations that include wind, the wind is applied normal to the span (per Section 3.9.3 of the AASHTO Spec.). Furthermore, the analysis method used by NYSDOT (NYSDOT 1983) assumes that the tether wire (upper or lower) releases under design wind speeds and thus the wind load on attachments is transferred directly to the main wire but with no reduction for rotation (i.e., attachment rotation due to wind load) since the exact wind speed at which the tether wire will release is unknown (it may not release and thus there would be no rotation). This simplified approach to wind load analysis is recommended herein over explicit consideration of the presence of the tether wire since explicit consideration of the tether wire produced overly conservative results (Gorab 2016) and the actual behavior when the tether wire is present is not well understood. In addition, for application of wind load to the attachments, all attachments are assumed to be oriented 45 degrees in plan relative to the span wire (since the span wire is usually oriented diagonally across an intersection). These recommendations for tethered span wire structures have been adopted in the T3SAP software.

5.3.1.2 Non-Tethered Span Wire Structures

For non-tethered span wire structures, the *Simplified Method* of analysis described in Appendix A.7 of the AASHTO Spec. is used in T3SAP to determine the maximum tension applied to each pole from the wire (as explained above, poles are assumed to be rigid). The same comments as above regarding two- and three-dimensional static analysis and the need to apply loads simultaneously due to the nonlinear nature of the span wire behavior, apply for non-tethered span wire structures. For load combinations that include wind, the wind is applied normal to the span (per Section 3.9.3 of the AASHTO Spec.), all attachments are assumed to be oriented 45 degrees in plan relative to the span wire (since the span wire is usually oriented diagonally across an intersection), and the wind loads on attachments are reduced to account for attachment rotation. (see Fig. 5.7 for an illustration of the rotation of a non-tethered traffic signal under gravity and wind loads).

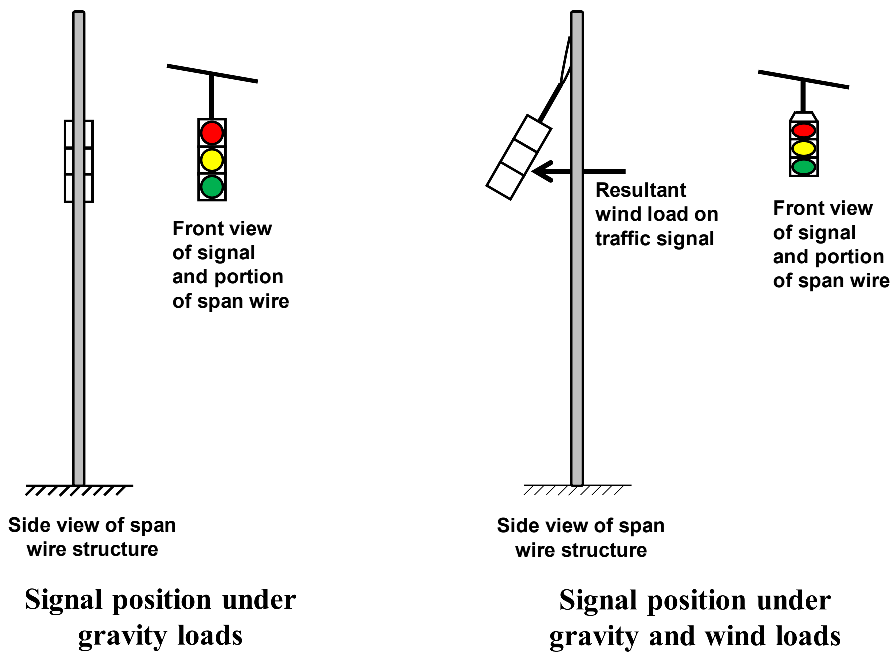

Fig. 5.7 Illustration of non-tethered traffic signal rotating under combined gravity and wind loads

5.3.1.3 Reduction in Wind Load for Non-Tethered Span Wire Structures

The degree to which rotation of attachments affects the resultant wind load on those attachments was estimated using results from an experimental study in which tests were performed on a full-scale, non-tethered span wire structure subjected to controlled wind loading (Cook 2007). The test structure had a fifty-foot long span and was supporting a single traffic signal at mid-span (traffic signal had a doghouse configuration that consists of five heads). For each test, the wind speed was increased from 20 to 115 mph and the signal rotation and cable tension were measured continuously. To evaluate the effect of signal orientation on signal rotation, the trend lines of the measured signal rotation data for the forward facing signal, backward facing signal, and diagonal signal were obtained (see Fig. 5.8). As can be seen in Fig. 5.8, for a given wind speed, the diagonal signal rotates less than the other two orientations. This is expected since the diagonal signal has a more aerodynamic shape. However, the difference in the rotation of the diagonal signal relative to that of the frontward and backward facing signals is small. The experimental test program also considered the effect of the weight of the traffic signal (by adding weight to the doghouse traffic signal) where, as expected, heavier traffic signals generally rotate less than lighter signals (see Fig. 5.9). Finally, it is recognized that, as a signal rotates, its shape changes from the perspective of the oncoming wind and consequently, the drag coefficient changes. In the experimental study, drag coefficients were computed based on measured cable tension and signal rotation. The trend lines for the computed drag coefficients are shown in Fig. 5.10 for three different signal orientations and in Fig. 5.11 for three different signal weights.

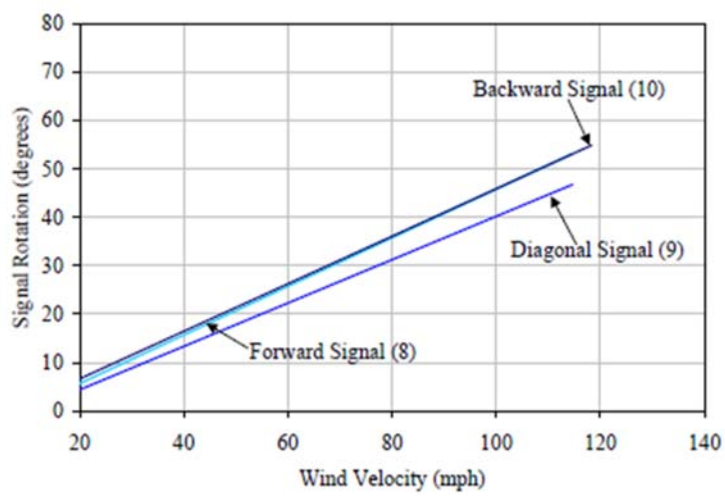

Fig. 5.8 Non-tethered traffic signal rotation for three signal orientations (from Cook 2007)

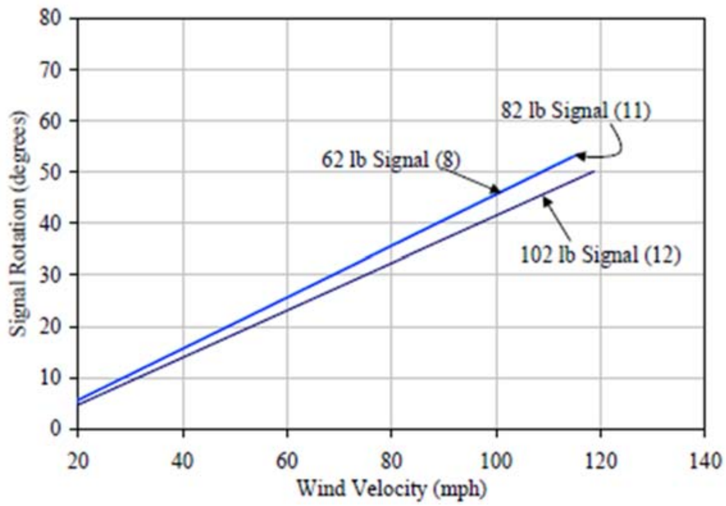

Fig. 5.9 Non-tethered traffic signal rotation for three signal weights (from Cook 2007)

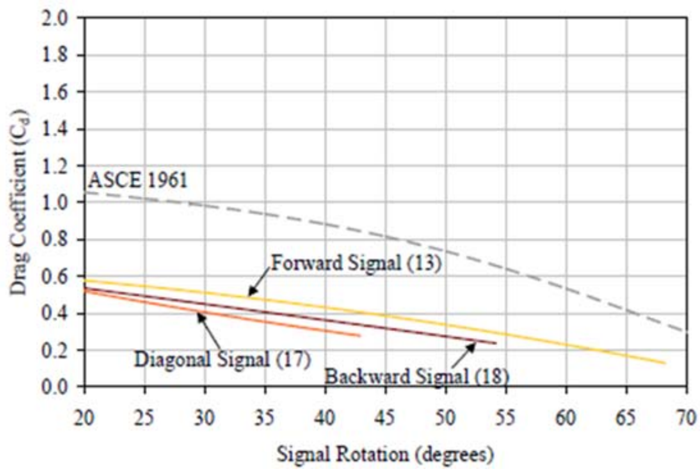

Fig. 5.10 Non-tethered traffic signal drag coeff. for three signal orientations (from Cook 2007)

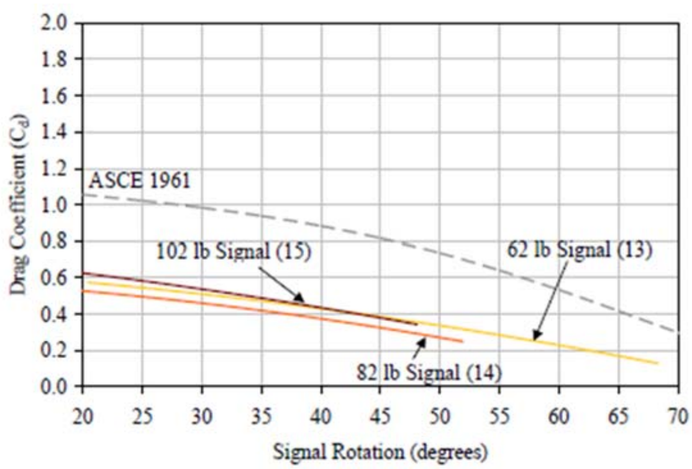

Fig. 5.11 Non-tethered traffic signal drag coefficient for three signal weights (from Cook 2007)

For the Group II load combination in which 100% of wind load is applied, it is recommended that, for non-tethered attachments, the EPA of both the traffic signals and signs be modified to account for rotation and the drag coefficient for traffic signals be modified to account for rotation. Specifically, experimental test results presented by Cook (2007) have been reviewed and analyzed by Gorab (2016), leading to the recommended signal head rotations (used in computation of EPA) and drag coefficients shown in Tables 5.1 and 5.2 for traffic signals that control the flow of traffic in one, two, three or four directions (see Fig. 4.1). The values in these tables are deemed by Gorab (2016) to be conservative and, indeed, a comparison of the values in Tables 5.1 and 5.2 with the trend lines in Figures 5.8-5.11 indicate that the values in the tables are conservative. This method of defining wind load on non-tethered traffic signals is recommended herein since it has a stronger experimental basis and broader applicability than the method currently used by NYSDOT (reduced wind loads are prescribed for specific signal head configurations; NYSDOT 1989). For the Group III load combination in which dead and ice load are applied along with 50% of the wind load, it is recommended that, for non-tethered attachments, the wind load be computed as described in the AASHTO Specifications (i.e., no reduction for rotation since the force that induces attachment rotation (wind load) is halved and the force resisting signal rotation is increased (ice load added to dead load)). These recommendations for defining wind loading on attachments of non-tethered span wire structures, including the recommended values shown in Tables 5.1 and 5.2, have been adopted in the T3SAP software.

Table 5.1 Recommended rotation values for non-tethered traffic signal under wind loading

Wind Speed (mph)	Traffic Signal Rotation (degrees)			
	One-Way Control	Two-Way Control	Three-Way Control	Four-Way Control
90-105	35	20	30	20
106-120	45	30	40	30

Table 5.2 Recommended drag coefficient values for non-tethered traffic signals

Wind Speed (mph)	Traffic Signal Drag Coefficient			
	One-Way Control	Two-Way Control	Three-Way Control	Four-Way Control
90-105	0.70	0.85	0.75	0.85
106-120	0.60	0.75	0.65	0.75

5.3.1.4 Group I Load Combination

For the Group I load combination, dead load is due to the self-weight of the span wire and its attachments and the self-weight of the pole and its attachments. Under these loads, the force that the span wire applies to the top of the pole (not exactly the top but rather where the span wire connects to the pole) has components along two orthogonal axes: vertical (Y) and horizontal along-span (X) (see Fig. 5.12 where the coordinate system is shown with origin at the base of the pole and where *L* is defined as the length of the pole from the base to the span wire attachment). The vertical force component acts downward and at an eccentricity (along the X-axis) with respect to the pole longitudinal centroidal axis. However, the eccentricity is very small relative to the pole height (on the order of 1% of the pole height) and thus the bending moment along the length of the pole due to the vertical force component is small compared to the bending moments induced by the horizontal component, especially at the base of the pole where the bending moment is largest. Thus, the vertical force component is considered to only induce a compressive axial force along the Y-axis. The horizontal force component along the span induces a shear force directed along the X-axis and a bending moment about the Z-axis. Furthermore, the weight of the pole itself and any pole attachments contributes to the axial force in the pole. Thus, the pole is subject to compressive axial force, transverse shear and uniaxial bending and therefore behaves as a beam-column. As shown in Fig. 5.12, the forces applied at the top of the pole (at the height of the span wire connection) are a vertical axial force, N_Y^T , and a transverse shear force, V_X^T . The internal forces at a distance *y* from the span wire connection, as obtained from static equilibrium, are an axial force, $N_Y\left(y\right)$, shear force, $V_X\left(y\right)$, and bending moment, $M_Z\left(y\right)$.

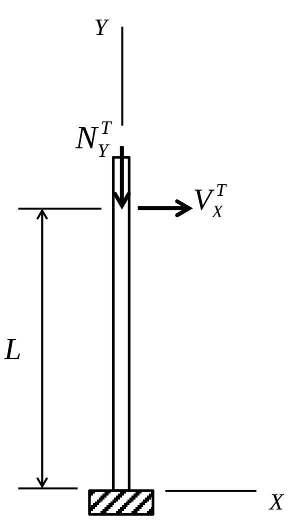

Fig. 5.12 Forces transferred to top of pole for single-span span wire structure for Group I load combination.

5.3.1.5 Group II and III Load Combinations

For the Group II and III load combinations, in addition to dead load (a gravity load), ice load (also a gravity load) and wind load (a lateral load) are considered (see Table 4.1). For these loading conditions, the force exerted by the span wire on each pole has components along three orthogonal axes: vertical (Y), horizontal along-span (X), and horizontal perpendicular to span (Z) (see Fig. 5.13 where the coordinate system is shown with origin at the base of the pole and where L is defined as the length of the pole from the base to the span wire attachment). The vertical force component acts downward and at an eccentricity (along the X-axis) with respect to the pole centroidal axis. Thus, the vertical force component results in a uniform compressive axial force along the Y-axis and a uniform bending moment about the Z-axis. However, as explained above, the eccentricity is very small relative to the pole height and thus the uniform bending moment along the length of the pole due to the vertical force component is small compared to the bending moments induced by the horizontal component, especially at the base of the pole where the bending moment is largest. Thus, the vertical force component is considered to only induce a compressive axial force along the Y-axis. The horizontal force component along the span induces a shear force directed along the X-axis and a bending moment about the Z-axis. The horizontal force component perpendicular to the span acts at an eccentricity (along the X-axis) with respect to the pole centroidal axis. Thus, this force component results in a shear force along the Z-axis, a bending moment about the X-axis, and a twisting moment about the Y-axis. However, the lever arm associated with the twisting moment is small and thus the torsional shear stresses may be small compared to those induced by the direct shear forces. In addition to the forces applied at the span wire connection, the self-weight of the pole and any pole attachments, plus wind and ice load on the pole and any pole attachments, contribute to the axial force, shear force and bending moments within the pole. Thus, the pole is subject to compression, biaxial shearing, biaxial bending, and twisting and therefore behaves as a beam-column with twisting. As shown in Fig. 5.13, the forces applied to the top of the pole are a vertical axial force, N_Y^T, two transverse shear forces, V_X^T and V_Z^T, and a twisting moment, M_Y^T. The internal forces at a distance y from the span wire connection, as obtained from static equilibrium, are an axial force, $N_Y(y)$, two transverse shear forces, $V_X(y)$ and $V_Z(y)$, two bending moments, $M_X(y)$ and $M_Z(y)$, and a twisting moment, $M_Y(y)$.

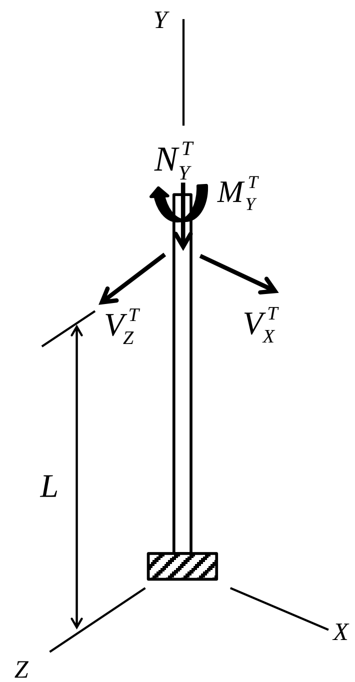

Fig. 5.13 Forces transferred to top of pole for single-span span wire structure for Group II and III load combinations.

5.3.2 Analysis of Multi-Span Span Wire Structures

The T3SAP software is capable of performing analysis for span wire structure configurations in which more than one span wire is attached to a pole. The specific configurations that are available in the software are the box span wire (4 poles, 4 span wires) and the hanging box span wire (4 poles, 4 span wires, 4 hanging wires) (see Fig. 1.1). For these two configurations, the angle between adjacent spans may be 90 degrees (square or rectangular intersection) or an angle larger or smaller than 90 degrees (skewed intersection). Other shapes exist, such as an L-shape (3 poles, 2 span wires) and C-shape (4 poles, 3 span wires), but these are not in common use by NYSDOT and thus are not available in the current version of the software.

To simplify the analysis of multi-span span wire structures, the poles are assumed to be rigid. Under this assumption, each span wire can be analyzed independently of the others. The analysis is performed as outlined above for tethered and non-tethered single-span span wire structures. For poles that have two span wires attached to them, the tensile forces from the two span wires are combined to obtain the resultant forces acting on the pole along three orthogonal axes. For load combinations that include wind, and in accordance with Section 3.9.3 of the AASHTO Spec., wind load from any direction must be considered. However, to simplify the analysis, the wind load is applied normal to the two adjacent spans (this approach to wind load application was justified in Section 4.4.1 of this report where it was explained that the wind load on the two adjacent spans is applied non-concurrently in the sense that wind load normal to one span is assumed to act independently of the wind load normal to the adjacent span). The resultant forces on the pole produce axial compression, biaxial shearing, biaxial bending and twisting and therefore the pole behaves as a beam-column with twisting.

5.4 DESCRIPTION OF ANALYSIS PROCEDURE FOR MAST ARM STRUCTURES

5.4.1 Analysis of Single-Arm Mast Arm Structures

Among mast arm structures, those with a single mast are very common. At an intersection, a single mast arm structure, positioned diagonally across the intersection, may be sufficient to control all traffic. Alternatively, an intersection may be controlled via multiple mast arm structures, each with a single mast arm. A distinguishing feature of these mast arm structures is that there is only one mast arm attached to each pole.

In the T3SAP software, the analysis of single mast arm structures begins with a two-dimensional static analysis that is performed for the Group I load combination (Dead only). Next, a three-dimensional static analysis is performed for the Group II (Dead + Wind) and III (Dead + Ice + ½ Wind) load combinations. Per the NYSDOT Standard Sheets 680-08 and 680-13 (NYSDOT 2015b), mast arm attachments (signals, signs, etc.) are anchored to the arm with U bolts such that they are not free to swing and thus no reductions in wind loads are used for the attachments. For each load combination, the internal forces throughout the mast arm structure are determined from static equilibrium.

5.4.1.1 Group I Load Combination

For the Group I load combination, dead load is due to the self-weight of the mast arm and its attachments and the self-weight of the pole and its attachments. The resulting internal forces along the mast arm consist of vertical shear forces along the Y-axis and bending moments about the Z-axis. The resulting shear force and bending moment at the connection to the pole are shown in Fig. 5.14 (in Fig. 5.14, the coordinate system is shown with origin at the mast arm connection and *L* is defined as the full length of the mast arm). Thus, the mast arm is subject to transverse shear and uniaxial bending and therefore behaves as a pure beam. As shown in Fig. 5.14, the forces at the mast arm connection are a vertical shear force, V_Y^C, and a bending moment, M_Z^C. Similarly, the internal forces in the mast arm at position *x* are a transverse shear force, $V_Y(x)$, and a bending moment, $M_Z(x)$.

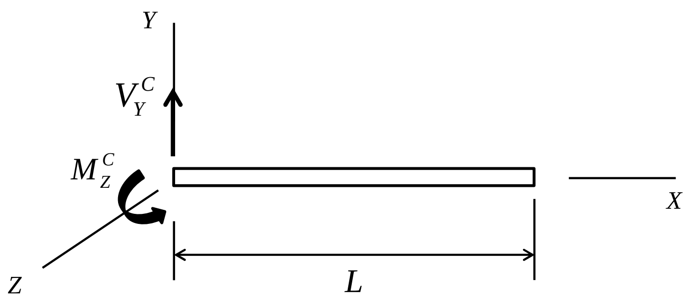

Fig. 5.14 Forces acting at mast arm connection for Group I load combination

At the mast arm to pole connection, the mast arm shear force, V_Y^C , which is eccentric relative to the pole, is transformed to a pole axial force (compression) and a bending moment about the Z-axis. The mast arm bending moment is transferred directly to the pole. The two bending moments are additive. The resulting forces applied to the top of the pole (not exactly the top but rather where the mast arm connects to the pole) are shown in Fig. 5.15 where the coordinate system origin is at the base of the pole and *L* is defined as the length of the pole from the base to the mast arm connection. In addition to these forces, the pole is subject to its own self-weight and the self-weight of pole attachments. The resulting internal forces in the pole are compressive axial forces along the Y-axis and bending moments about the Z-axis and thus the pole behaves as a beam-column with uniaxial bending. As shown in Fig. 5.15, the forces applied to the top of the pole are a vertical axial force, N_Y^T , and a bending moment, M_Z^T . The internal forces at a distance *y* from the mast arm connection, as obtained from static equilibrium, are an axial force, $N_Y(y)$ and a bending moment, $M_Z(y)$.

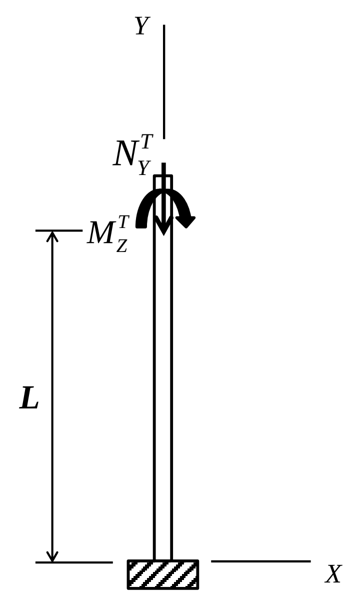

Fig. 5.15 Forces transferred to top of pole for single-arm mast arm structure for Group I load combination.

5.4.1.2 Group II and III Load Combinations

For the Group II and III load combinations, in addition to dead load (a gravity load), ice load (also a gravity load) and wind load (a lateral load) are considered (see Table 4.1). For the dead and ice load, the internal forces along the mast arm consist of vertical shear forces along the Y-axis and bending moments about the Z-axis (see Fig. 5.16 where the coordinate system is shown with origin at the mast arm connection and *L* is defined as the full length of the mast arm). In accordance with Section 3.9.2 of the AASHTO Spec., the wind load is applied perpendicular to the mast arm (along Z direction) for computing internal forces in the mast arm. Thus, the internal forces in the mast arm due to wind consist of transverse shear forces along the Z-axis and bending moments about the Y-axis. Thus, the mast arm is subject to transverse shear along two axes and biaxial bending and therefore behaves as a pure beam with bending about two axes (see Fig. 5.16). As shown in Fig. 5.16, the forces at the mast arm connection consist of two transverse shear forces, V_Y^C and V_Z^C, and two bending moments, M_Z^C and M_Y^C. Similarly, the internal forces along the mast arm (at position *x*) consist of two transverse shear forces, $V_Y(x)$ and $V_Z(x)$, a two bending moments, $M_Z(x)$ and $M_Y(x)$.

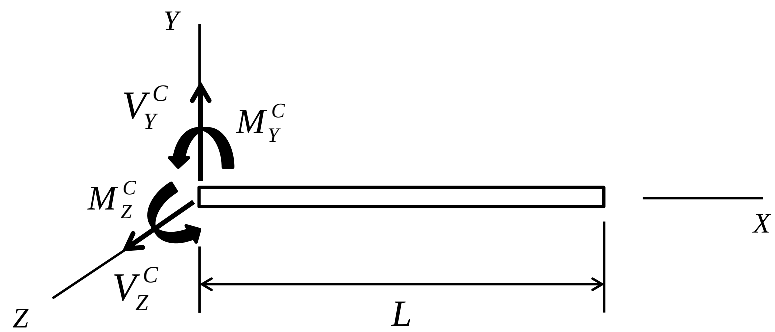

Fig. 5.16 Forces acting at mast arm connection for Group II and III load combinations.

For load combinations that include wind, Section 3.9.3 of the AASHTO Spec. requires that pole internal forces be determined with consideration of wind applied from any direction. In the T3SAP software, the wind load is applied in multiple directions (over a 180 degree range at 5 degree intervals) and the wind direction that produces the largest total wind load is used to determine forces (shear force, bending moment, and twisting moment) applied to the pole. At the mast arm to pole connection, the mast arm shear forces are eccentric relative to the longitudinal centroidal axis of the pole. The mast arm shear force in the Y-direction (from dead and ice load) is transformed to a pole axial force (compression) along the Y-axis and a bending moment about the Z-axis (see Fig. 5.17). The mast arm shear force in the Z-direction (from wind load) is transformed to a pole shear force along the Z-axis, a bending moment about the X-axis and a twisting moment about the Y-axis. The mast arm bending moment about the Z-axis is transferred directly to the pole as a bending moment about the Z-axis. The mast arm bending moment about the Y-axis is transferred to the pole as a twisting moment about the Y-axis. The two Z-axis bending moments are additive and the two Y-axis twisting moments are additive. The resulting forces acting at the top of the pole are shown in Fig. 5.17 and consist of a vertical axial force, N_Y^T, a transverse shear force, V_Z^T, two bending moments, M_X^T and M_Z^T, and a twisting moment, M_Y^T (in Fig. 5.17, the coordinate system is shown with origin at the base of the pole and L is defined as the length of the pole from the base to the mast arm connection). The resulting internal forces in the pole are a compressive axial force along the Y-axis, a shear force along the Z-axis, bending moments about the X-axis and Z-axis and a twisting moment about the Y-axis (neglecting any pole attachments). In addition to the forces applied at the mast arm connection, the self-weight of the pole and any pole attachments, plus wind and ice load on the pole and any pole attachments, contribute to the axial force, shear force and bending moments within the pole. Thus, the pole is subject to axial compression, biaxial shearing, biaxial bending, and twisting and therefore behaves as a beam-column with twisting. The internal forces at a distance y from the mast arm connection, as obtained from static equilibrium, are an axial force, $N_Y(y)$, two transverse shear forces, $V_X(y)$ and $V_Z(y)$, two bending moments, $M_X(y)$ and $M_Z(y)$, and a twisting moment, $M_Y(y)$.

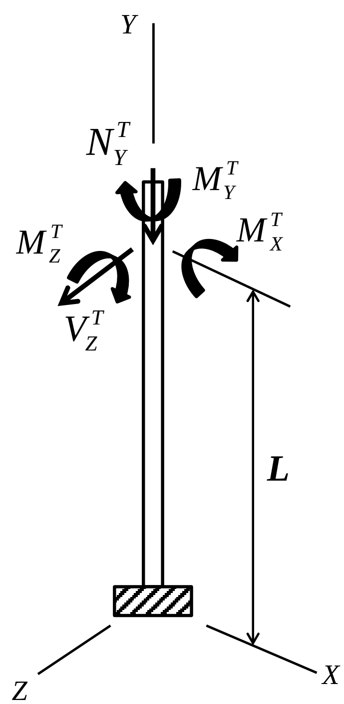

Fig. 5.17 Forces transferred to top of pole for single-arm mast arm structure for Group II and III load combinations.

5.4.2 Analysis of Dual-Arm Mast Arm Structures

The T3SAP software is capable of performing analysis for mast arm structure configurations in which two mast arms are attached to a pole (i.e., a dual mast arm structure). The two mast arms may be perpendicular to each other (L-shape) or may form an obtuse or acute angle. In any of these cases, the internal forces in each mast arm are obtained from static equilibrium.

For load combinations that include wind, in accordance with Section 3.9.2 of the AASHTO Spec., the wind load is applied normal to each mast arm when computing internal forces in each mast arm. For computation of pole internal forces, Section 3.9.3 of the AASHTO Spec. requires that wind from any direction be considered. In the T3SAP software, the wind load is applied in multiple directions (over a 180 degree range at 5 degree intervals) and the wind direction that produces the largest total wind load is used to determine forces applied by each mast arm to the pole. The force applied by each mast arm are transferred to the longitudinal centroidal axis of the pole and are combined to obtain resultant forces acting on the pole along three orthogonal axes. In addition to the forces applied by the mast arms, the self-weight of the pole and any pole attachments, plus wind and ice load on the pole and any pole attachments, contribute to the axial force, shear force and bending moments within the pole. Thus, the pole is subject to axial compression, biaxial shearing, biaxial bending, and twisting and therefore behaves as a beam-column with twisting.

5.5 SECOND-ORDER EFFECTS

For design of traffic signal poles (vertical cantilever supports), Section 4.6.1 of the AASHTO Spec. requires that second-order effects be considered. One approach is to perform a complex nonlinear analysis that requires iteration to arrive at a final solution. Alternatively, Section 4.8 of the AASHTO Spec. permits a simplified analysis in which second-order (P-delta) effects are accounted for in slender poles via application of a Coefficient of Amplification, C_A, to the allowable bending stress. Since the Coefficient of Amplification is less than or equal to unity, its use generally results in a reduction in the allowable bending stress (reduction in bending stress capacity). Note that the results from the first-order analyses described above for the poles of both span wire (Section 5.3) and mast arm (Section 5.4) structures is used to determine the bending stress demand.

5.6 DEFLECTIONS

Per Section 10 of the AASHTO Spec., traffic signal support structures must have adequate stiffness to provide acceptable serviceability performance.

5.6.1 Lateral Deflection of Vertical Supports: Span Wire Structure Pole

Per Section 10.4.2.1 of the AASHTO Spec., under dead loading (Group I load combination), the pole top lateral deflection is limited to 2.5% of the pole height when the pole is subjected to a lateral force. This occurs for span wire structures where the dead loads induce tension in the span wire which in turn applies a lateral force to the pole.

The T3SAP software computes the lateral deflection at the top of each pole based on the assumption of linear elastic behavior of the pole. The lateral deflection is dependent on the pole cross-section geometry and whether the pole is uniform, tapered, or stepped along its length. The cross-section geometry of the steel tubular pole may be round, hexdecagonal (16-sided), dodecagonal (12-sided) or octagonal (8-sided).

For a tapered tubular pole subject to a lateral load at the top of the pole, an expression for the lateral deflection at the top of the pole is defined in Appendix B (Table B.3-1) of the AASHTO Specification:

$$y_{\max} = \frac{WL^3}{2ECt\left(R_B - R_A\right)^3}\left[2\ln\left(\frac{R_B}{R_A}\right) - \left(\frac{R_B - R_A}{R_B}\right)\left(3 - \frac{R_A}{R_B}\right)\right] \tag{5.1}$$

where W is the lateral load, L is the pole length (length from base of pole to point of application of lateral load), E is the modulus of elasticity, C is a cross-sectional constant (values available in AASHTO Spec. Table B.1-1), t is the tube wall thickness, R_B is the radius measured to the mid-thickness of the wall at the bottom of the pole and R_A is the radius measured to the mid-thickness of the wall at the top of the pole. In T3SAP, the lateral load for single-span wire structures is the horizontal component of the span wire force at the connection to the pole. For multi-span span wire structures, the lateral force is taken as the resultant of the horizontal components of each span wire force at the connection to the pole. The pole top lateral deflection is computed as the sum of two deflections: 1) deflection at the height of the span wire connection using Eq. (5.1) and 2) additional lateral deflection due to rotation of pole segment above the span wire connection.

For a uniform pole (no variation in cross-section over its length), $R_A = R_B$ and the above equation reduces to 0/0 which is undefined. For this special case, the lateral deflection at the span wire connection is given by:

$$y_{\max} = \frac{WL^3}{3EI} = \frac{WL^3}{3E\left(CR^3t\right)} \tag{5.2}$$

where R is the radius measured to the mid-thickness of the wall. As an example, for a uniform pole with circular cross-section, $C = \pi \cong 3.14$ and the lateral deflection is given by:

$$y_{\max} \cong \frac{WL^3}{9.42ER^3t} \tag{5.3}$$

For the case of stepped poles having uniform cross-section between steps, the lateral deflection at the span wire connection is computed as the sum of the contributions from each pole segment between steps.

5.6.2 Slope of Vertical Supports: Mast Arm Structure Pole

Per Section 10.4.2.1 of the AASHTO Spec., under dead loading (Group I load combination), the slope at the top of a pole is limited to 30 mm/m (0.03 radians = 1.72 degrees) when the pole is subjected to a moment. This occurs for mast arm structures where the dead loads induce shear and moment in the mast arm and thus a moment is applied at the mast arm connection.

The T3SAP software computes the slope at the top of the pole based on the assumption of linear elastic behavior of the pole. The slope is dependent on the pole cross-section geometry and whether the pole is uniform, tapered, or stepped along its length. The cross-section geometry of the steel tubular pole may be round, hexdecagonal (16-sided), dodecagonal (12-sided) or octagonal (8-sided).

For a tapered tubular pole subject to a moment at the top of the pole, an expression for the slope at the top of the pole is defined in Appendix B (Table B.3-1) of the AASHTO Specification:

$$\theta_{\max} = \frac{ML\left(R_A + R_B\right)}{2ECtR_A^2 R_B^2} \tag{5.4}$$

where M is the applied moment, L is the pole length (length from base of pole to point of application of lateral load), E is the modulus of elasticity, C is a cross-sectional constant (values available in AASHTO Spec. Table B.1-1), t is the tube wall thickness, R_B is the radius measured to the mid-thickness of the wall at the bottom of the pole and R_A is the radius measured to the mid-thickness of the wall at the top of the pole. In T3SAP, the applied moment for single mast arm structures is the bending moment at the mast arm connection to the pole. For dual mast arm structures, the applied moment is taken as the resultant of the bending moments at each mast arm connection to the pole. The pole top slope is computed as the slope at the height of the mast arm connection (the slope at the top of the pole is equal to the slope at the height of the mast arm connection).

For a uniform pole (no variation in cross-section over its length), $R_A = R_B$ and the above equation reduces to:

$$\theta_{\max} = \frac{ML}{ECtR^3} \tag{5.5}$$

where R is the radius measured to the mid-thickness of the wall. As an example, for a uniform pole with circular cross-section, $C = \pi \cong 3.14$ and the slope is given by:

$$\theta_{\max} \cong \frac{ML}{3.14EtR^3} \qquad (5.5)$$

For the case of stepped poles having uniform cross-section between steps, the slope at the height of the mast arm connection is computed as the sum of the contributions from each pole segment between steps.

5.6.3 Deflection of Horizontal Supports: Mast Arms of Mast Arm Structure

Per Section 10.4.2.2 of the AASHTO Spec., adequate stiffness shall be provided for the horizontal supports of cantilevered traffic signal support structures that will result in acceptable serviceability performance. However, no specific dead load deflection limit is specified. Per the associated commentary (C10.4.2.2), the stiffness requirements are left to the designer. Thus, the T3SAP software does not perform any assessment in regard to deflection of mast arms.

6. CAPACITY ASSESSMENT

6.1 STRENGTH-RELATED CAPACITY ASSESSMENT OF MAJOR COMPONENTS

6.1.1 Span Wire Cable

According to Section 5.13 of the AASHTO Spec., "… the maximum tension force encountered [in the cable] times a minimum safety factor of three shall be less than the breaking strength of the cable or connection." This safety criterion may be expressed as:

$$T_{max} N < T_{break} \tag{6.1}$$

where N is the safety factor (equal to 3), T_{max} is the maximum tensile force in the cable and T_{break} is the breaking strength of the cable. Rearranging Eq. (6.1) gives:

$$T_{max} < \frac{T_{break}}{3} \cong 0.33 T_{break} \tag{6.2}$$

For cases with wind load (Group Load Combination II and III), the safety factor is reduced to 2.25 to account for a 1/3 allowable stress increase:

$$T_{max} < \frac{4}{3} \frac{T_{break}}{N} = \frac{4}{3} \frac{T_{break}}{3} = \frac{T_{break}}{2.25} \cong 0.44 T_{break} \tag{6.3}$$

As discussed in Section 5.3 of this report, the T3SAP software determines the maximum tension in the cable, T_{max}, using the *Simplified Method* of analysis described in Appendix A of the AASHTO Specification.

According the NYSDOT Standard Sheet 680-07 (NYSDOT 2015b), the smallest diameter permitted for span wires is 7/16 in. Thus, in the T3SAP software, it is conservatively assumed that 7/16-in. diameter span wire is used. According to Standard Sheet 680-07, for a 7/16 in. diameter span wire, the breaking strength is 16,900 lb and the pole design load (maximum allowable tension force in span wire) is 11,000 lb. Thus, the implied factor of safety is:

$$N = \frac{T_{break}}{T_{max}} = \frac{16900}{11000} \cong 1.54 \tag{6.4}$$

The allowable tension force in the span wire may then be written as:

$$T_{allow} = T_{max} = \frac{T_{break}}{N} \cong \frac{T_{break}}{1.54} \cong 0.65 T_{break} = 0.65(16900) = 11{,}000 \text{ lb} \tag{6.5}$$

Since the T3SAP software performs capacity assessment in accordance with the requirements of both the AASHTO Spec. and the NYSDOT Spec., the more conservative factors of safety defined by the AASHTO Spec. are used in the software for defining the allowable tension force in a span wire cable. Thus, the software computes the span wire demand-capacity ratio as follows:

Group Load Combination I:

$$\frac{Demand}{Capacity} = \frac{T_{max}}{T_{allow}} = \frac{T_{max}}{T_{break}/3} = \frac{T_{max}}{16900/3} = \frac{T_{max}}{5633} \tag{6.6}$$

Group Load Combination II and III:

$$\frac{Demand}{Capacity} = \frac{T_{max}}{T_{allow}} = \frac{T_{max}}{(4/3)(T_{break}/3)} = \frac{T_{max}}{(4/3)(16900/3)} = \frac{T_{max}}{7511} \tag{6.7}$$

In T3SAP, the demand-capacity ratios defined by Eqs. (6.6) and (6.7) are reported in the output file and a warning is issued if either demand-capacity ratio exceeds 0.95. The value of 0.95 was selected in consultation with the NYSDOT.

6.1.2 Poles

Both span wire and mast arm structures utilize vertically-oriented shafts (i.e., poles). For span wire structures, the poles may support one or more span wires. Similarly, for mast arm structures the shafts (a.k.a., masts or posts) may support one or more arms (a.k.a., mast arms).

6.1.2.1 Material

Per Section 724-03 of the NYSDOT Spec., poles shall be galvanized steel and there are numerous steel types and grades that may be used. The T3SAP software utilizes a value of 29,000,000 psi for the modulus of elasticity which is applicable to carbon steel and low alloy steel. In addition, two values of yield stress (36 and 50 ksi) and ultimate stress (58 and 65 ksi) are available which correspond to commonly available A36 and A992 structural steel. In addition, the user can specify other values for the yield stress and ultimate stress.

6.1.2.2 Shape

Per Section 724-03 of the NYSDOT Standard Spec., poles may have round or multisided shapes. In addition, they may be non-tapered (uniform cross-section), continuously tapered or stepped. The cross-section geometry can be round, hexdecagonal (16-sided), dodecagonal (12-sided) or octagonal (8-sided). Per Section 5.14.2 of the AASHTO Spec., fatigue cracking becomes a concern for multi-sided poles that have a small number of sides (fatigue cracks develop at pole-to-transverse plate connection).

6.1.2.3 Local Buckling

Per Section 5.5.1 of the AASHTO Spec., steel sections are classified as compact, noncompact or slender depending on the geometry (width-to-thickness ratios) of the elements that make up the section. Expressions for width-to-thickness ratios (element local slenderness, λ) that define the cutoffs for various cross-sectional shapes are provided in Table 5.5.2-1 of the AASHTO Spec. where:

Compact section: $\lambda \le \lambda_p$

Noncompact section: $\lambda_p < \lambda \le \lambda_r$

Slender section: $\lambda_r < \lambda < \lambda_{max}$

where λ_p is the compact limit, λ_r is the noncompact limit and λ_{max} is the maximum limit.

For round shapes:

$$\lambda = D/t \tag{6.8}$$

where D is the outside diameter and t is the thickness. For multi-sided sections:

$$\lambda = b/t \tag{6.9}$$

where b is the effective width as given by:

$$b = \left[D - 2t - \min\left(2r_b, 8t\right)\right]\tan\left(180^\circ/n\right) \tag{6.10}$$

where D is the outside distance from flat side to flat side, r_b is the inside bend radius and n is the number of sides. If the inside bend radius is not known, the following simplified expression can be used:

$$b = \left(D - 2t - 3t\right)\tan\left(180^\circ/n\right) = \left(D - 5t\right)\tan\left(180^\circ/n\right) \tag{6.11}$$

Since the inside bend radius may not be known by the software user, the T3SAP software computes the effective width using the simplified expression given by Eq. (6.11).

6.1.2.4 Allowable Bending Stress

Expressions that define the allowable bending stress, F_b, for poles are provided in Table 5.6-1 of the AASHTO Specification. For compact sections, the allowable bending stress is not a function of the specific width-to-thickness ratio (it only depends on the yield stress). For noncompact and slender sections, the allowable bending stress is a function of the width-to-thickness ratio. For example, for a round pole with yield stress of 50 ksi, the allowable bending stress is shown in

Fig. 6.1 as a function of the element slenderness (D/t). The vertical dashed lines represent the cutoffs between compact, noncompact and slender sections. Note that the allowable bending stress of multisided poles is not permitted to exceed that of round poles with diameter equal to the outside distance from flat side to flat side (this check is included in the T3SAP software).

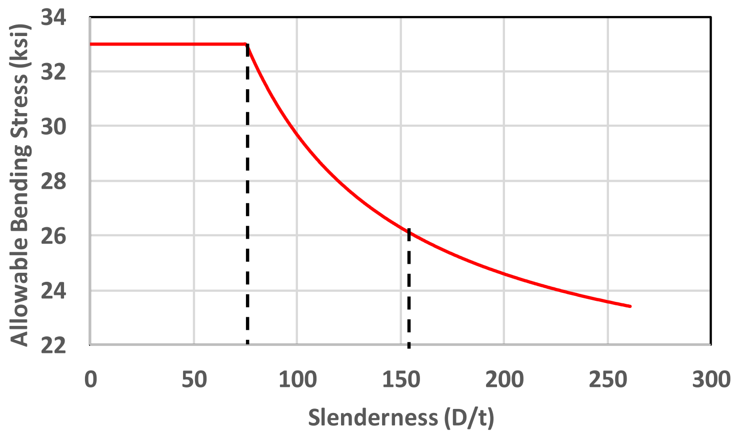

Fig. 6.1 Allowable bending stress for round pole with yield stress of 50 ksi.

6.1.2.5 Allowable Compression Stress

Expressions that define the allowable compression stress are provided in Section 5.10 of the AASHTO Specification. The allowable compression stress is a function of the pole global slenderness $\lambda = kL/r$ where *k* is the effective length factor, *L* is the pole length and *r* is the minimum radius of gyration. Regarding pole length, per Section 724-02 of the NYSDOT Spec., the span wire or mast arm attachment is located 1.5 ft below the top of the pole and thus the region of the pole that is under significant compression is the full pole length minus 1.5 ft. Nonetheless, for conservative capacity assessment, the full pole length is used in T3SAP to compute the global slenderness. Regarding radius of gyration, the "minimum" value refers to the neutral axis about which the radius of gyration is minimum. For tapered or stepped poles, the radius of gyration at the mid-height of the pole is used (mid-height is conservatively taken as half the full pole length). If the pole has low slenderness ($\lambda < C_c$), its behavior is inelastic when it reaches its capacity and thus the allowable compression (axial) stress, F_a, is a function of the yield stress, F_y:

$$F_a = \frac{\left(1-\lambda^2/2C_c^2\right)F_y}{5/3+3\lambda/8C_c-\lambda^3/8C_c^3} \tag{6.12}$$

where

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} \tag{6.13}$$

If the pole has high slenderness ($\lambda \ge C_c$), its behavior is elastic (elastic buckling) when it reaches its capacity and thus the allowable compression stress is independent of the yield stress:

$$F_a = \frac{12\pi^2 E}{23\lambda^2} \tag{6.14}$$

In accordance with Section C5.10 in the commentary of the AASHTO Spec., T3SAP uses an effective length factor, *k*, of 2.1 (design value for cantilevered columns).

Note that, as the slenderness approaches zero, the allowable compression stress defined by Eq. (6.12) approaches $(3/5)F_y = 0.6F_y$ which corresponds to a factor of safety of 5/3 = 1.67. For higher slenderness ratios (elastic buckling limit state), the allowable compression stress defined by Eq. (6.14) implies a factor of safety of 23/12 = 1.92.

6.1.2.6 Allowable Shear Stress

Expressions that define the allowable shear stress for poles are provided in Section 5.11 of the AASHTO Specification. For round poles, the allowable shear stress is a function of the local slenderness ratio, *D*/*t*. If the pole has small local slenderness ($D/t \le 1.16\left(E/F_y\right)^{2/3}$), the allowable shear stress, F_v , is a function of the yield stress:

$$F_v = 0.33F_y \tag{6.15}$$

and if the pole has large local slenderness ($D/t > 1.16\left(E/F_y\right)^{2/3}$), the allowable shear stress is independent of the yield stress:

$$F_v = \frac{0.41E}{\left(D/t\right)^{3/2}} \tag{6.16}$$

For multisided poles, the allowable shear stress is a function of the local slenderness ratio, *b*/*t*. If the pole has local slenderness below a certain value ($b/t \le 2.23\sqrt{E/F_y}$), the allowable shear stress is a function of the yield stress:

$$F_v = 0.33F_y \tag{6.17}$$

and if the pole has local slenderness that exceeds that value ($b/t > 2.23\sqrt{E/F_y}$), the allowable shear stress is independent of the yield stress:

$$F_v = \frac{1.64E}{\left(b/t\right)^2} \tag{6.18}$$

However, per Table 5.5.2-1 in the AASHTO Spec., the local slenderness ratio of multisided tubular sections is not permitted to exceed the following value:

$$\lambda_{max} = \left(b/t\right)_{max} = 2.14\sqrt{E/F_y} \tag{6.19}$$

Consequently, the allowable shear stress for multisided poles is defined in the T3SAP software as $F_v = 0.33F_y$.

For cases with wind load (Group Load Combinations II and III), the maximum allowable shear stress for both round and multisided poles is increased by 1/3 of their nominal value. Considering the limit defined by Eq. (6.19), for these load combinations and multisided poles, the allowable shear stress is defined in the T3SAP software as $F_v = \left(4/3\right)\left(0.33F_y\right) = 0.44F_y$.

6.1.2.7 Combined Stresses

An expression that defines the general demand-capacity ratio, with consideration of multiple actions applied at a given cross-section, is provided in Section C.5.12 of the commentary in the AASHTO Specification:

$$\frac{Demand}{Capacity} = \frac{f_a}{F_a} + \frac{f_b}{F_b} + \left(\frac{f_v}{F_v}\right)^2 \tag{6.20}$$

For single vertical cantilever pole-type supports, the allowable axial stress, F_a, is set equal $0.6F_y$ and the allowable bending stress, F_b, is amplified by an amplification factor, C_A, to account for second-order effects due to the presence of axial load (i.e., P-delta effects). The resulting demand-capacity criteria for checking the adequacy of a particular cross-section of the pole is given by:

$$\frac{Demand}{Capacity} = \frac{f_a}{0.6F_y} + \frac{f_b}{C_A F_b} + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0 \tag{6.21}$$

where f_a, f_b and f_v are the maximum axial, bending and shear stress demands on the cross-section with the shear stress coming from both direct shear and torsional shear. For cases with wind load (Group Load Combinations II and III), the allowable axial, bending and shear stresses in Eq. (6.21) are increased by 1/3 of their nominal value.

In the T3SAP software, the demand-capacity criterion defined by Eq. (6.21) is checked at certain key locations along the pole height (base of pole, elevation at center of hand hole, elevation at steps in a stepped pole, elevation of mast arm or span wire connection and elevation of any pole attachments) and is reported to the user in the output file along with a warning if the demand-capacity ratio exceeds 0.95.

Per Section 4.8.1 of the AASHTO Spec., the amplification factor is only applicable for poles that are globally slender as defined by:

$$\lambda = \frac{kL}{r} \geq C_c = \sqrt{\frac{2\pi^2 E}{F_y}} \tag{6.22}$$

In such cases, the amplification factor is given by:

$$C_A = 1 - \frac{\sqrt[3]{\frac{I_B}{I_T}} P_T + 0.38 D_P}{\frac{2.46 E I_B}{L^2}} \tag{6.23}$$

where I_B and I_T are the second moments of area of the pole at the base (bottom) and top, respectively, P_T is the vertical concentrated load at the top of the pole, D_P is the dead load (weight) of the pole, and L is the length of the pole. In the T3SAP software, for computation of the amplification factor defined in Eq. (6.23), L is conservatively taken as the full length (height) of the pole, I_T is conservatively computed at the top (full height) of the pole, and P_T is taken as the concentrated load at the elevation of the span wire or mast arm connection.

6.1.3 Mast Arms

6.1.3.1 Material

Per Section 724-03 of the NYSDOT Standard Spec., the same criteria as given above (Section 6.1.2) for pole materials are relevant for mast arms.

6.1.3.2 Shape

Per Section 724-03 of the NYSDOT Standard Spec., the same criteria as given above (Section 6.1.2) for pole shapes are relevant for mast arms. Furthermore, mast arms must have the same shape as the pole and they may be of "two-piece construction with a telescoping joint secured by a thru-bolt and locknut."

6.1.3.3 Local Buckling

The categorization of sections as compact, noncompact and slender for mast arms is determined in the same manner as for poles (see Section 6.1.2).

6.1.3.4 Allowable Bending Stress

The allowable bending stress for mast arms is determined in the same manner as for poles (see Section 6.1.2).

6.1.3.5 Allowable Compression Stress

Although a mast arm may be subject to direct compressive or tensile stress due to wind loading acting on attachments, the induced axial stress is expected to be small relative to that due to bending and thus is not considered in the T3SAP software.

6.1.3.6 Allowable Shear Stress

The allowable shear stress for mast arms is determined in the same manner as for poles (see Section 6.1.2).

6.1.3.7 Combined Stresses

An expression that defines the general demand-capacity ratio with consideration of multiple actions applied at a given cross-section is provided in Section C.5.12 of the commentary in the AASHTO Spec. and was previously presented as Eq. (6.20), which is repeated below:

$$\frac{Demand}{Capacity} = \frac{f_a}{F_a} + \frac{f_b}{F_b} + \left(\frac{f_v}{F_v}\right)^2 \qquad \text{(6.20) Repeated}$$

Per Section 5.12.12.1 of the AASHTO Spec., for mast arms subject to low compressive axial force ($f_a \leq 0.15F_a$), the demand-capacity criterion is given by:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0 \qquad (6.24)$$

and per Section 5.12.2.2 of the AASHTO Spec., for mast arms subject to tensile axial force, the demand-capacity criterion is given by:

$$\frac{f_a}{F_t} + \frac{f_b}{F_b} + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0 \qquad (6.25)$$

where F_t is the allowable tensile stress.

The axial stress in the mast arm is expected to be small enough relative to the bending stress such that it is reasonable to neglect the contribution from axial stress. Thus, the T3SAP software uses the following demand-capacity criterion for mast arms:

$$\frac{f_b}{F_b}+\left(\frac{f_v}{F_v}\right)^2 \leq 1.0 \tag{6.26}$$

For cases with wind load (Group Load Combinations II and III), the allowable bending and shear stresses in Eq. (6.26) are increased by 1/3 of their nominal values.

In the T3SAP software, the demand-capacity criterion defined by Eq. (6.26) is checked at certain key locations along the mast arm length (at connection to pole and at location of arm attachments) and is reported to the user in the output file along with a warning if the demand-capacity ratio exceeds 0.95.

6.2 STRENGTH-RELATED CAPACITY ASSESSMENT OF CONNECTIONS

6.2.1 Span Wire to Pole Connection

Per Section 724-03 of the NYSDOT Spec., span wires (cables) attached to multisided poles can be attached using either a thimble eyebolt or a circumferential pole clamp. If attached to a round pole, only a thimble eyebolt is permitted. Per Sheet 680-07 of the NYSDOT Standard Sheets (NYSDOT 2015b), "fittings used with span wires shall develop the full breaking strength of the wire." Thus, if designed properly, the cable is the weak link (cable tends to fray gradually which is a more desirable failure mode than sudden rupture of an eyebolt or pole clamp) and thus fittings, including eyebolts and pole clamps, should not reach their capacity. Therefore, the T3SAP software does not perform a capacity assessment for the span wire to pole connection.

6.2.2 Mast Arm to Pole Connection

Per Sheet 680-06 (sheet 2 of 2) of the NYSDOT Standard Sheets (NYSDOT 2015b), a typical mast arm to pole connection consists of a gusseted box connection with details varying depending upon the manufacturer. This type of connection is illustrated in detail in Figure C5.14.7-1 of the AASHTO Specification. The T3SAP software performs a limited capacity assessment of the mast arm to pole connection. Specifically, the capacity of the bolts in combined shear and tension and the base plate in bearing (bearing by the bolts) are checked.

For mast arm connections, a typical connection contains four bolts where the orientation of bolts is well defined in that the orientation cannot be changed when the structure is constructed in the field (bolts are at the corners of a square or rectangular shape; see Figure 6.2). Thus, based on recommendations from the NYSDOT, two bolts can be relied upon to develop tension when the applied bending moment acts about either the horizontal or vertical axis through the centroid of the bolt group in Figure 6.2 (bending moment about horizontal axis is due to gravity loads; bending about vertical axis is due to wind loading). Note that these moments are resisted by a combination of tension in two of the bolts and bearing (compression) on the flange plate. Also note that, due to symmetry of the connection, the aforementioned axes are principal axes. For Group Load Combinations II and III, the maximum tension in the bolts is taken as the sum of the contribution from bending due to gravity load and bending due to wind load. In addition, for a conservative and simplified analysis, the two remaining bolts are considered to resist the resultant shear force (from biaxial shear forces). The maximum tensile force and shear force acting on a single bolt is determined and used to obtain tensile and shear stress demands.

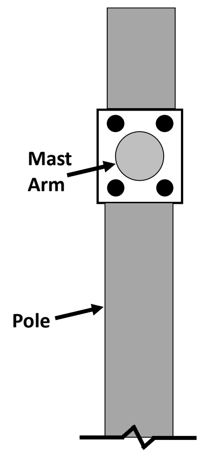

Fig. 6.2 Orientation of bolts at mast arm to pole connection (elevation view)

According to the NYSDOT, it is expected that the mast arm connection in new structures will eventually employ 6 bolts (two vertical lines of 3 bolts). For the case of applied bending moment about the horizontal axis through the bolt group centroid (bending moment due to gravity loads), the middle bolts along each line will provide redundancy in terms of resisting shear. For the case of applied bending moment about the vertical axis through the bolt group centroid (bending moment due to wind load), the middle bolts along each line will provide redundancy in terms of resisting the bending moment and shear force. However, in both cases (bending due to gravity load and bending due to wind load), NYSDOT plans to continue designing conservatively by assuming that two bolts resist the bending moment via tension and two bolts resist the resultant shear force. Thus, the approach to capacity assessment of mast arm-to-pole connections, as described in the previous paragraph and used in the T3SAP software, will remain applicable.

6.2.2.1 Mast Arm Bolts in Tension

For bending moment, M, about one principal axis of the connection, the tensile force, T, in a single bolt is obtained from moment equilibrium:

$$T = \frac{M}{2d} \tag{6.27}$$

where d is the distance between bolts in a direction orthogonal to the axis under consideration. The maximum tension, $T_{\max}$, in a single bolt is then taken as the sum of the contribution from bending about each principal axis. The tensile stress demand, f_t, on a single bolt is then given by:

$$f_t = \frac{T_{max}}{A} \tag{6.28}$$

where A is the tensile stress area of a bolt as given by (from Section 5.17.4.2 of the AASHTO Spec.):

$$A = \frac{\pi}{4}\left(D - \frac{0.9743}{n}\right)^2 \text{ (in.}^2\text{)} \tag{6.29}$$

where D is the nominal diameter of the bolt and n is the number of threads per inch. In the T3SAP software, a value of n = 6 is used (corresponds to a bolt diameter of 1-1/2 inch (larger bolt diameters have smaller values of n and vice-versa). The value of n = 6 is deemed to be a good estimate of the lower bound value of n and thus produces a reasonable estimate for the lower bound on the tensile stress area.

Per Section 5.17.4.2 of the AASHTO Spec., the allowable tensile stress, F_t, on the tensile stress area of the bolts is given by:

$$F_t = 0.50 F_y \tag{6.30}$$

6.2.2.2 Mast Arm Bolts in Shear

For each of the two bolts in shear, the shear stress demand, f_v, is given by:

$$f_v = \frac{V}{2A} \tag{6.31}$$

where V is the shear force demand at the connection (resultant shear force as described above) and A is the tensile stress area of a bolt (conservatively assuming threads are in the shear plane). Per Section 5.17.4.2 of the AASHTO Spec., the allowable shear stress, F_v, on the tensile stress area of the bolts is given by:

$$F_v = 0.30 F_y \tag{6.32}$$

6.2.2.3 Mast Arm Bolts in Combined Tension and Shear

Per Section 5.17. 4.2, the demand-capacity ratio and safety criterion for bolts in combined tension and shear is:

$$\frac{demand}{capacity} = \left(\frac{f_v}{F_v}\right)^2 + \left(\frac{f_t}{F_t}\right)^2 \leq 1.0 \tag{6.33}$$

For cases with wind load (Group Load Combinations II and III), the allowable tensile and shear stresses in Eq. (6.33) are increased by 1/3 of their nominal values.

The T3SAP software computes the demand-capacity ratio defined by Eq. (6.33), reports it in the output file, and issues a warning if the ratio exceeds 0.95.

6.2.2.4 Mast Arm Bolts Bearing on Mast Arm Flange Plate

Per Section 5.17.4.4 of the AASHTO Spec., the demand-capacity ratio and safety criterion for bolts in bearing on the mast arm flange plate is:

$$\frac{demand}{capacity} = \frac{f_v}{F_v} = \frac{f_v}{4tF_u/\pi D} \leq 1.0 \tag{6.34}$$

where *t* is the flange plate thickness, F_u is the flange plate design tensile strength, and *D* is the nominal diameter of the bolt. For cases with wind load (Group Load Combinations II and III), the allowable shear stress in Eq. (6.34) is increased by 1/3 of its nominal value.

The T3SAP software computes the demand-capacity ratio defined by Eq. (6.34), reports it in the output file, and issues a warning if the ratio exceeds 0.95.

6.2.3 Pole Base Plate to Foundation Connection

Per Section 724-03 of the NYSDOT Spec., anchor bolts are to be anchored using double nuts (one nut above the base plate and one nut below). The lower nut is used to level the base plate.

Per Section 5.17.2 of the AASHTO Spec., multiple types of anchor bolts can be used and thus the software requires the user to input the minimum yield strength of the anchor bolts. Also, per Section C5.17.3 of the AASHTO Spec., for a design life of 50 years, a minimum of six anchor bolts should be considered.

Although there are a number of relevant limit states for the pole base plate to foundation connection (see Section 5.17.3 of the AASHTO Spec.), the T3SAP software only performs capacity assessment related to the bolts and bearing of the bolts on the base plate since details of the connection, such as concrete foundation properties and geometry, are not generally readily available. Thus, it is left to the user of the software to perform other capacity assessments as needed. In particular, it should be recognized that the software does not consider prying effects (as noted in Section C5.17.3 of the AASHTO Spec., research has shown that prying effects may be neglected if the base plate thickness is at least equal to the bolt diameter).

For pole base plate to foundation connections, the orientation of the anchor bolts is not as well defined as for the bolts in mast arm to pole connections since the orientation of the bolts depends on how where the bolts are positioned when the concrete is cast (see Figure 6.3 for a span wire structure). Thus, it is possible that only one bolt will develop significant tension under application of the resultant bending moment. As part of the project to develop the T3SAP software, NYSDOT specified that, for capacity assessment of base plate anchor bolts, only one bolt should be considered to develop tension under the applied bending moment and two bolts should be considered to resist the applied shear force. Thus, the T3SAP software performs a capacity assessment of base plate anchor bolts wherein one bolt in tension is assumed to resist the resultant bending moment and two bolts in shear resist the resultant shear force. For conservative and simplified analysis, it is assumed that the bolt that carries the tensile force from the resultant bending moment also carries half the resultant shear force.

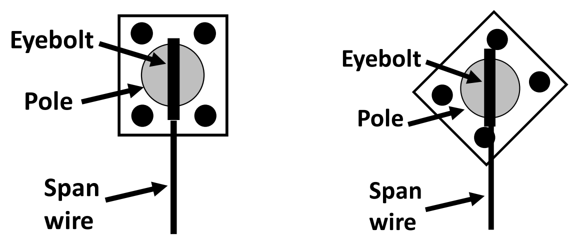

Fig. 6.3 Two possible orientations of base plate anchor bolts for span wire structure (plan views are shown)

6.2.3.1 Tensile, Compressive and Shear Demand on Anchor Bolts

Per Section 5.17.4.1 of the AASHTO Spec., for double-nut anchor bolt connections the maximum axial stress (tension or compression) in a single bolt is obtained as a combination of stress from axial compressive load, N, and the applied bending moment, M:

$$\text{Tensile stress demand: } f_t = f_a - f_b = \frac{N}{A} - \frac{Mc}{I} \tag{6.35}$$

$$\text{Compressive stress demand: } f_c = f_a + f_b = \frac{N}{A} + \frac{Mc}{I} \tag{6.36}$$

where A is the area of the bolt group, c is the distance from the centroid of the bolt group to the centroid of the outermost bolt and I is the second moment of area of the bolt group about the axis of bending. The maximum shear stress acting on a single bolt is obtained as a combination of stress from direct shear force, V, and from torsional moment, T:

$$f_v = \frac{V}{A} + \frac{Tr}{J} \tag{6.37}$$

where A is the area of the bolt group, r is the radial distance from the centroid of the bolt group to the centroid of the outermost bolt and J is the polar moment of area of the bolt group about the bolt group centroid.

6.2.3.2 Tensile, Compressive and Shear Stress Capacity of Bolts

Per Section 5.17.4.2 of the AASHTO Spec., the allowable tensile stress on the tensile stress area of the bolts is given by:

$$F_t = 0.50 F_y \tag{6.38}$$

and the allowable compressive stress on the tensile stress area of the bolts is given by:

$$F_c = 0.60 F_y \tag{6.39}$$

and the allowable shear stress on the tensile stress area of the bolts is given by:

$$F_v = 0.30 F_y \tag{6.40}$$

6.2.3.3 Combined Axial/Shear Demand-Capacity Relation for Anchor Bolts

Per Section 5.17.4.2 of the AASHTO Spec., for anchor bolts subjected to combined tension and shear, the demand-capacity ratio and safety criterion is:

$$\frac{demand}{capacity} = \left(\frac{f_v}{F_v}\right)^2 + \left(\frac{f_t}{F_t}\right)^2 \le 1.0 \tag{6.41}$$

and for anchor bolts subjected to combined compression and shear, the demand-capacity ratio and safety criterion is:

$$\frac{demand}{capacity} = \left(\frac{f_v}{F_v}\right)^2 + \left(\frac{f_c}{F_c}\right)^2 \le 1.0 \tag{6.42}$$

For cases with wind load (Group Load Combinations II and III), the allowable axial stress (tension or compression) and allowable shear stress in Eqs. (6.41) and (6.42) are increased by 1/3 of their nominal values.

The T3SAP software computes the demand-capacity ratios defined by Eqs. (6.41) and (6.42), reports them in the output file, and issues a warning if either ratio exceeds 0.95.

6.2.3.4 Buckling of Anchor Bolts

Per Section 5.17.4.2 of the AASHTO Spec., if the clear distance between the bottom of the lower nut and the concrete surface is larger than four bolt diameters, buckling of the anchor bolt must be considered using the same capacity assessment criteria defined above in Section 6.1.2.5 for poles (i.e., instead of defining the allowable compression stress via Eq. (6.39), the allowable compression (axial) stress is defined by either Eq. (6.12) or (6.14)). However, per Sheet 680-01 of the NYSDOT Standard Sheets (NYSDOT 2015b), the maximum exposed anchor bolt length between the concrete and lower nut is one bolt diameter. As part of the project to develop the T3SAP software, NYSDOT specified that, since four bolt diameters is much larger than one bolt diameter, the anchor bolt buckling limit state does not need to be checked and therefore this limit state is not considered in the T3SAP software.

6.2.3.5 Effect of Anchor Bolt Bending on Combined Axial/Shear Demand-Capacity Relation

Per Section 5.17.4.3 of the AASHTO Spec., if the clear distance between the bottom of the lower nut and the concrete surface (exposed length of anchor bolts) is larger than one bolt diameter, bending stresses in the anchor bolts must be considered. As noted above, per Sheet 680-01 of the NYSDOT Standard Sheets (NYSDOT 2015b), the maximum exposed anchor bolt length between the concrete and lower nut is one bolt diameter. However, as part of the project to develop the T3SAP software, NYSDOT specified that existing structures may have exposed anchor bolt lengths larger than one bolt diameter and thus bolt bending should be considered. Thus, the T3SAP software requires the user to enter a value for the exposed length of the anchor bolts.

Per Section C5.17.4.3 of the AASHTO Spec, to consider bolt bending, the anchor bolt is modeled as a fixed-fixed beam (fixed at concrete surface and fixed at bottom of lower nut). The aforementioned demand-capacity safety criteria for anchor bolts (Eqs. (6.41) and (6.42)) are used except with modified tensile or compressive stress demand. For anchor bolts subjected to combined tension, shear and bending, the demand-capacity ratio and safety criterion is:

$$\frac{demand}{capacity} = \left(\frac{f_v}{F_v}\right)^2 + \left(\frac{f_t + f_b^t}{F_t}\right)^2 \leq 1.0 \qquad (6.43)$$

where the axial tensile stress applied to the bolt is combined with the maximum tensile stress from bending of the bolt, f_b^t. For anchor bolts subjected to combined compression, shear and bending, the demand-capacity ratio and safety criterion is:

$$\frac{demand}{capacity} = \left(\frac{f_v}{F_v}\right)^2 + \left(\frac{f_c + f_b^t}{F_c}\right)^2 \leq 1.0 \qquad (6.44)$$

where the axial compressive stress applied to the bolt is combined with the maximum tensile stress from bending of the bolt (note that the AASHTO Spec. refers to maximum tensile bending stress here but, due to symmetry of the cross-section, its value is equal to the maximum compressive bending stress).

For cases with wind load (Group Load Combinations II and III), the allowable axial stress (tension or compression) and allowable shear stress in Eqs. (6.43) and (6.44) are increased by 1/3 of their nominal values.

The T3SAP software computes the demand-capacity ratios defined by Eqs. (6.43) and (6.44), reports them in the output file, and issues a warning if either ratio exceeds 0.95.

6.2.3.6 Pole Base Plate Bearing Strength at Bolt Holes

Per Section 5.17.4.4 of the AASHTO Spec., if the anchor bolts are required to resist shear or torsion, the anchor bolt holes must be shear holes (see Table 5.17.4.4-1 of the AASHTO Spec. for maximum hole dimensions) and the maximum shear stress acting on a single bolt is obtained as a combination of stress from direct shear force, *V*, and from torsional moment, *T*:

$$f_v = \frac{V}{A} + \frac{Tr}{J} \tag{6.45}$$

and the bolt shear stress is limited to the bearing strength of the base plate bolt holes. The design bearing strength (shear force capacity) of the base plate bolt holes is:

$$R_n = F_u D t \tag{6.46}$$

where t is the thickness of the base plate and F_u is the design tensile strength (ultimate strength) of the base plate. The allowable shear stress for the bolt is then (safety factor of 1.0):

$$F_v = \frac{R_n/A}{\Omega} = \frac{R_n/A}{1.0} = \frac{F_u D t}{\pi D^2/4} = \frac{4tF_u}{\pi D} \tag{6.47}$$

and the demand-capacity criterion for shear stress on the anchor bolt (which relates to bearing failure of the base plate bolt holes) is:

$$\frac{demand}{capacity} = \frac{f_v}{F_v} \leq 1.0 \tag{6.48}$$

For cases with wind load (Group Load Combinations II and III), the allowable shear stress in Eq. (6.48) is increased by 1/3 of its nominal value.

The T3SAP software computes the demand-capacity ratio defined in Eq. (6.48), reports it in the output file and issues a warning if the ratio exceeds 0.95.

6.3 FATIGUE-RELATED CAPACITY ASSESSMENT

6.3.1 Introduction

Per AASHTO Spec. Section C11.1, fatigue can be defined as the damage that may result in fracture after a sufficient number of stress fluctuations.

Per AASHTO Spec. Section 11.4, design for fatigue is applicable to mast arm structures but not to span wire structures.

Per AASHTO Spec. Section 11.5, the design criteria for fatigue is given by the following general demand-capacity ratio and safety criterion:

$$\frac{demand}{capacity} = \frac{\Delta f}{\Delta F} \leq 1.0 \tag{6.49}$$

where Δf is the wind-load-induced stress range and ΔF is the fatigue resistance. Per AASHTO Spec. Section C11.5, design of new structures for finite fatigue life is unreliable and thus infinite fatigue life design is recommended. In this case, the fatigue resistance is set equal to the constant amplitude fatigue threshold (CAFT). The remaining fatigue life of existing structures can be assessed based on a finite life.

There are different approaches to fatigue design including nominal stress-based design, local stress-based design and experiment-based design. The nominal stress-based design is described in Section 11 of the AASHTO Spec. while the other two methods are described in Appendix D of the AASHTO Specification. Since nominal stress-based design is usually sufficient (per AASHTO Spec. Section C11.5) and is described in detail in Section 11 of the AASHTO Spec., it is utilized in the T3SAP software. For such an approach, the demand-capacity ratio and safety criterion becomes:

$$\frac{demand}{capacity} = \frac{(\Delta f)_n}{(\Delta F)_n} \leq 1.0 \tag{6.50}$$

where the subscript *n* indicates nominal stress.

Per AASHTO Spec. Section 11.6, fatigue importance factors, I_F, are used to adjust the level of structural reliability. As illustrated subsequently, these factors are utilized in defining the demand. As discussed in Section 4.5.4 of this report, values of the importance factors are provided in AASHTO Spec. Table 11.6-1 for each type of wind load effect (galloping, natural wind gusts and truck-induced gusts) and for three different structural categories (I, II and III).

The importance factors range from 0.30 to 1.0 for cantilevered mast arm structures and 0.55 to 1.0 for non-cantilevered structures. Category I structures present a high hazard in the event of failure. All structures without effective vibration mitigation devices and with roadway speed limits in excess of 35 mph and average one-way daily traffic (ADT) exceeding 10,000 or average one-way daily truck traffic (ADTT) exceeding 1,000 are classified as Category I. Structures with roadway speed limits less than or equal to 35 mph are classified as Category III. All other structures are classified as Category II. The T3SAP software requires that the user determine the appropriate value for the fatigue importance factor and allows the user to enter that value as an input for the analysis.

6.3.2 Fatigue Design Loads

Per AASHTO Spec. Section 11.7, to avoid large-amplitude vibration and preclude development of fatigue cracks, mast arm structures are designed to resist applicable limit state equivalent static wind loads (pressure ranges) acting separately (i.e., galloping, natural wind gust and truck-induced gust acting separately). These loads are used to determine nominal stress ranges at fatigue-sensitive connection details and at other critical locations.

6.3.2.1 Galloping

Per AASHTO Spec. Section 11.7.1.1, mast arm structures shall be design for galloping-induced cyclic loads by applying an equivalent static shear pressure vertically to the surface area (as viewed in normal elevation) of all signs and traffic signal heads and backplates rigidly mounted to the mast arm. The vertical shear pressure range for galloping, P_G ,is given by:

$$P_G = 21 I_F \text{ (psf)} \tag{6.51}$$

This vertical pressure is multiplied by the surface area of each mast arm attachment to obtain a vertical load at each attachment. Structural analysis then gives the corresponding load at each fatigue sensitive region of the structure. Note that, per AASHTO Spec. Section C11.7.1.1, mast arm structures with more than one arm may be designed for galloping loads applied separately on each arm (galloping is not considered to occur simultaneously on multiple arms).

6.3.2.2 Natural Wind Gust

Per AASHTO Spec. Section 11.7.1.2, mast arm structures shall be designed for natural wind gust by applying an equivalent static horizontal pressure to the exposed area of all support structure members, signs, traffic signals and miscellaneous attachments. The horizontal pressure range for natural wind, P_{NW} , which is based on a yearly mean wind velocity of 11.2 mph, is given by:

$$P_{NW} = 5.2 C_d I_F \text{ (psf)} \tag{6.52}$$

where C_d is the applicable drag coefficient based on the yearly mean wind velocity of 11.2 mph.

The natural wind gust loading should be considered to be applied along any direction of wind. For each wind load direction considered, structural analysis gives the corresponding load at each fatigue sensitive region of the structure.

Note that, per Section C11.7.1.2 of the AASHTO Spec., for a single mast arm structure, the critical wind direction for natural wind gust loading is perpendicular to the arm. For mast arm structures with multiple arms, the critical wind direction is usually not perpendicular to either arm. Thus, the natural wind gust loading should be applied to the exposed surface areas seen in an elevation view oriented perpendicular to the assumed wind gust direction (i.e., wind gust loading applied to projected surface areas) and with a range of wind gust directions considered. In the T3SAP software, a more extensive analysis is performed in which the critical wind load direction is determined for both single and dual mast arm structures by applying the wind loading to the mast arms and associated attachments over an angle range of 180 degrees, using 5 degree increments, and determining the angle that maximizes the resultant wind load.

6.3.2.3 Truck-Induced Gust

Per AASHTO Spec. Section 11.7.1.3, mast arm structures shall be designed for truck-induced gust by applying an equivalent static vertical pressure to the arm and the area of all signs and attachments projected on a horizontal plane (according to NYSDOT, the most significant effect of truck-induced gust is when it is applied to a variable message sign (VMS) and thus the T3SAP software only considers truck-induced gust for structures that support VMS signs). The vertical pressure range for truck-induced gust, P_{TG}, is given by:

$$P_{TG} = 18.8 C_d I_F \text{ (psf)} \tag{6.53}$$

where C_d is the drag coefficient for variable message signs based on a truck speed of 65 mph (taken as 1.7 from AASHTO Spec. Table 3.8.6-1). This vertical pressure range is applied along any 12-ft length, excluding any portion of the structure not located directly above a traffic lane, to create the maximum stress range at fatigue sensitive regions of the structure (note that the 12-ft length is associated with the width of a single truck driving under the mast arm). Note that, per AASHTO Spec. Section C11.7.1.3, mast arm structures with more than one arm may be designed for truck-induced gust loads by applying the vertical pressure separately on each arm (truck-induced gust loads are not considered to occur simultaneously on multiple arms). Also, the magnitude of the applied pressure may be varied depending on the height of the mast arm and the attachments above the traffic lane (full pressure applied for height less than or equal to 20 ft and then linear reduction to a value of zero at 33 ft).

6.3.3 Fatigue Resistance

Per AASHTO Spec. Section 11.9.1, fatigue sensitive details shall be designed in accordance with their respective detail classifications. Classification of structural components, mechanical fasteners and welded details in typical support structures are provided in AASHTO Table

11.9.3.1-1. Any detail that is not contained in Table 11.9.3.1-1 can be classified using alternate methodologies provided in Appendix D of the AASHTO Specification.

Per AASHTO Spec. Section C11.9.1, tube-to-transverse plate connections, hand holes and anchor rods are the most fatigue critical details (based on fatigue cracking observed in the field and in laboratory testing).

Per AASHTO Spec. Section 11.9.2, when Table 11.9.3.1-1 is used to classify a fatigue sensitive detail, the nominal stress range shall be used in fatigue design where nominal stress is determined at the site of potential fatigue cracking (see Table 6.1 below for relevant details as described in AASHTO Spec. Section 11.9.2; other relevant details, such as for anchor bolts, are discussed below). Nominal stress at the fatigue-sensitive site is computed using structural analysis for the applied fatigue design loads (galloping, natural wind gust, truck-induced gust) and using nominal section properties with due account of gross changes in geometry of the cross-section (e.g., at hand holes).

Table 6.1 Relevant details for fatigue design and location where nominal stress is computed (as described in AASHTO Spec. Section 11.9.2)

Detail	**Location of Nominal Stress**
Reinforced hand hole	For design against fatigue cracking at toe of reinforcement-to-tube weld, consider net section property of tube and reinforcement.
	For design against fatigue cracking from the root of reinforcement-to-tube weld, nominal stress computed for cracking at toe is magnified by stress concentration factor of 4.0 (width of hand hole opening is limited to 40% of pole diameter).
Fillet-welded tube-to-transverse-plate connections (socket connections)	Gross section of tube at the fillet-weld toe on the tube.

Per AASHTO Spec. Section 11.9.3, for infinite service life the nominal fatigue resistance is set equal to the constant amplitude fatigue threshold (CAFT):

$$\left(\Delta F\right)_n = \left(\Delta F\right)_{TH} \tag{6.54}$$

Thus, the demand-capacity ratio and safety criterion for infinite service life becomes:

$$\frac{demand}{capacity} = \frac{(\Delta f)_n}{(\Delta F)_n} = \frac{(\Delta f)_n}{(\Delta F)_{TH}} \leq 1.0 \tag{6.55}$$

The T3SAP software computes the demand-capacity ratio defined by Eq. (6.55), reports it in the output file and issues a warning if the ratio exceeds 0.95.

For existing mast arm structures, the remaining fatigue life can be assessed based on a finite life. For finite life, the nominal fatigue resistance is given by:

$$(\Delta F)_n = \sqrt[3]{\frac{A}{N}} \tag{6.56}$$

where A is the finite life constant (available in AASHTO Spec. Table 11.9.3.1-1) and N is the number of wind load induced stress cycles expected during the life time of the structure (N can be estimated from analysis based on historical wind records or directly by field measurements on similar structures). Thus, the demand-capacity ratio and safety criterion for finite service life becomes:

$$\frac{demand}{capacity} = \frac{(\Delta f)_n}{(\Delta F)_n} = \frac{(\Delta f)_n}{\sqrt[3]{A/N}} \leq 1.0 \tag{6.57}$$

The fatigue life, in terms of number of wind load induced stress cycles, can then be obtained from:

$$N \leq \frac{A}{(\Delta f)_n^3} \tag{6.58}$$

The T3SAP software computes the upper limit on the number of wind load induced stress cycles using Eq. (6.58) and reports it in the output file. The remaining fatigue life can be estimated by the user by reducing the value of N by the number of wind load induced stress cycles that the structure has already incurred. The user can then convert the fatigue life to a time value by defining the number of wind-induced stress cycles that occur in a given period of time.

The fatigue resistance (capacity) of typical fatigue-sensitive details and for infinite or finite life designs is determined from AASHTO Spec. Table 11.9.3.1-1 (note that effects of residual stresses due to fabrication and anchor bolt pretension are already accounted for in the table).

Per AASHTO Spec. Section 11.9.3.1, fatigue stress concentration factors for finite life design of connections (tube-to-transverse plate), K_F, are defined in AASHTO Spec. Table 11.9.3.1-3. For infinite life design, the fatigue stress concentration factor is then given by:

$$K_I = \left[1.76 + 1.83 t_T - 4.76 (0.22)^{K_F}\right] K_F \tag{6.59}$$

where t_T is the thickness of the tube in units of inches. The stress concentration factors are a function of the connection geometry and are used in AASHTO Spec. Table 11.9.3.1-1 to determine fatigue resistance. Note that, for some of the fatigue-sensitive details shown in the table, the finite fatigue life stress concentration factor must fall within a specified range or be below some value. If these requirements are not met, the T3SAP software does not perform a fatigue-related capacity assessment and issues a warning that informs the user to check the geometry of the detail.

6.3.4 Fatigue Related Assessment of Mast-Arm-to-Transverse-Plate Connection and Pole-to-Base-Plate Connection

As part of the project to develop the T3SAP software, the NYSDOT specified that the applicable fatigue detail for mast-arm-to-transverse-plate connections and pole-to-base-plate connections is Detail 5.4 in AASHTO Spec. Table 11.9.3.1-1 (fillet-welded tube-to-transverse-plate connection; a so-called "socket connection"). These connections consist of a tube that fits inside the base plate with a fillet weld around the circumference of the tube between the tube wall and top of the base plate and a second weld at the bottom of the tube and the inside of the base plate (although the weld at the bottom of the tube is not clearly visible in Detail 5.4 of the AASHTO Spec., its existence has been verified by NYSDOT). For infinite service life, the demand-capacity ratio and safety criterion is:

$$\frac{demand}{capacity} = \frac{(\Delta f)_n}{(\Delta F)_n} = \frac{(\Delta f)_n}{(\Delta F)_{TH}} \leq 1.0 \tag{6.60}$$

where the fatigue threshold value depends on the stress concentration factor:

$$(\Delta F)_{TH} = \begin{cases} 7.0 \text{ ksi} & \text{for } K_I \leq 4 \\ 4.5 \text{ ksi} & \text{for } 4.0 < K_I \leq 6.5 \\ 2.6 \text{ ksi} & \text{for } 6.5 < K_I \leq 7.7 \end{cases} \tag{6.61}$$

and where $(\Delta f)_n$ is the nominal stress range in the tube wall along the fillet weld toe (the location of potential fatigue-induced cracking is illustrated in Detail 5.4 of the AASHTO Specification).

The T3SAP software computes the demand-capacity ratio defined by Eq. (6.60), reports it in the output file and issues a warning if the ratio exceeds 0.95.

For finite service life, the demand-capacity ratio and safety criterion is:

$$\frac{demand}{capacity}=\frac{(\Delta f)_n}{(\Delta F)_n}=\frac{(\Delta f)_n}{\sqrt[3]{A/N}}\leq 1.0 \tag{6.62}$$

The fatigue life, in terms of number of wind load induced stress cycles, can be obtained as follows:

$$N\leq\frac{A}{(\Delta f)_n^3}=\frac{3.9(10)^8\ \text{ksi}^3}{(\Delta f)_n^3} \tag{6.63}$$

The T3SAP software computes the upper limit on the number of wind load induced stress cycles using Eq. (6.63) and reports it in the output file. The remaining fatigue life can be estimated by the user by reducing the value of *N* by the number of wind load induced stress cycles that the structure has already incurred. The user can then convert the fatigue life to a time value by defining the number of wind-induced stress cycles that occur in a given period of time.

6.3.5 Fatigue Related Assessment of Bolts Connecting Mast Arm to Pole and Connecting Pole to Foundation

The applicable fatigue detail for anchor bolts and mast arm-to-column bolts is Detail 2.3 in AASHTO Spec. Table 11.9.3.1-1 ("Anchor bolts or other fasteners in tension"). For infinite service life, the demand-capacity ratio and safety criterion is:

$$\frac{demand}{capacity}=\frac{(\Delta f)_n}{(\Delta F)_n}=\frac{(\Delta f)_n}{(\Delta F)_{TH}}=\frac{(\Delta f)_n}{7\text{ ksi}}\leq 1.0 \tag{6.64}$$

where the fatigue threshold, $(\Delta F)_{TH}$, is 7 ksi and $(\Delta f)_n$ is the nominal stress range at the root of the threads extending into the tensile stress area. The bolt tensile stress area was previously presented as Eq. (6.29) and is repeated below:

$$A=\frac{\pi}{4}\left(D-\frac{0.9743}{n}\right)^2\ (\text{in.}^2) \qquad \text{(6.29) Repeated}$$

Recall that, for mast arm-to-pole connections, two bolts are considered to carry the tensile load (see Section 6.2.2 of this report) while, for pole-to-foundation connections, a single bolt is considered to carry the tensile load (see Section 6.2.3 of this report).

The T3SAP software computes the demand-capacity ratio defined by Eq. (6.64), reports it in the output file and issues a warning if the ratio exceeds 0.95.

For finite service life, the demand-capacity ratio and safety criterion is:

$$\frac{demand}{capacity} = \frac{(\Delta f)_n}{(\Delta F)_n} = \frac{(\Delta f)_n}{\sqrt[3]{A/N}} \leq 1.0 \tag{6.65}$$

The fatigue life, in terms of number of wind load induced stress cycles, can be obtained as follows:

$$N \leq \frac{A}{(\Delta f)_n^3} = \frac{22.0(10)^8 \text{ ksi}^3}{(\Delta f)_n^3} \tag{6.66}$$

The T3SAP software computes the upper limit on the number of wind load induced stress cycles using Eq. (6.66) and reports it in the output file. The remaining fatigue life can be estimated by the user by reducing the value of *N* by the number of wind load induced stress cycles that the structure has already incurred. The user can then convert the fatigue life to a time value by defining the number of wind-induced stress cycles that occur in a given period of time.

6.3.6 Fatigue Related Assessment of Reinforced Hand Holes

The applicable fatigue detail for reinforced hand holes is Detail 3.2 in AASHTO Spec. Table 11.9.3.1-1 ("Reinforced holes and cutouts"). For infinite service life, the demand-capacity ratio and safety criterion is:

$$\frac{demand}{capacity} = \frac{(\Delta f)_n}{(\Delta F)_n} = \frac{(\Delta f)_n}{(\Delta F)_{TH}} \leq 1.0 \tag{6.67}$$

where the fatigue threshold value depends on the location of potential fatigue cracks:

$$(\Delta F)_{TH} = \begin{cases} 16.0 \text{ ksi} & \text{at root of reinforcement-to-tube weld} \\ 7.0 \text{ ksi} & \text{at toe of reinforcement-to-tube weld} \end{cases} \tag{6.68}$$

and where $(\Delta f)_n$ is the nominal stress range at the associated location.

The T3SAP software computes the demand-capacity ratio defined by Eq. (6.67), reports it in the output file and issues a warning if the ratio exceeds 0.95.

For finite service life, the demand-capacity ratio and safety criterion is:

$$\frac{demand}{capacity} = \frac{(\Delta f)_n}{(\Delta F)_n} = \frac{(\Delta f)_n}{\sqrt[3]{A/N}} \leq 1.0 \tag{6.69}$$

The fatigue life, in terms of number of wind load induced stress cycles, can be obtained as follows:

$$N \le \frac{A}{\left(\Delta f\right)_n^3} \tag{6.70}$$

where the finite life constant depends on the location of potential fatigue cracks:

$$A = \begin{cases} 120.0 \text{ ksi}^3 & \text{at root of reinforcement-to-tube weld} \\ 22.0 \text{ ksi}^3 & \text{at toe of reinforcement-to-tube weld} \end{cases} \tag{6.71}$$

The T3SAP software computes the upper limit on the number of wind load induced stress cycles using Eq. (6.70) and reports it in the output file. The remaining fatigue life can be estimated by the user by reducing the value of *N* by the number of wind load induced stress cycles that the structure has already incurred. The user can then convert the fatigue life to a time value by defining the number of wind-induced stress cycles that occur in a given period of time.

6.4 SERVICEABILITY-RELATED CAPACITY ASSESSMENT

6.4.1 Pole Deflection

Per Section 10 of the AASHTO Spec., traffic signal support structures must have adequate stiffness to provide acceptable serviceability performance.

6.4.1.1 Lateral Deflection of Vertical Supports: Span Wire Structure Pole

Per Section 10.4.2.1 of the AASHTO Spec., under dead loading (Group I load combination), the pole top lateral deflection is limited to 2.5% of the pole height when the pole is subjected to a lateral force. This occurs for span wire structures where the dead loads induce tension in the span wire which in turn applies a lateral force to the pole.

6.4.1.2 Single-Span Span Wire Structure

The T3SAP software computes the lateral deflection demand at the top of each pole based on the assumption of linear elastic behavior of the pole. The lateral deflection is dependent on the pole cross-section geometry and whether the pole is uniform, tapered, or stepped along its length. The cross-section geometry of the steel tubular pole may be round, hexdecagonal (16-sided), dodecagonal (12-sided) or octagonal (8-sided). Expressions for the lateral deflection at the span wire connection are given in Section 5.6.1 of this report and were taken from AASHTO Spec. Appendix B.3. This deflection is combined with the deflection of the pole segment above the span wire connection to obtain the maximum lateral deflection, y_{max}, at the top of the pole.

The demand-capacity ratio and serviceability criterion is:

$$\frac{demand}{capacity} = \frac{y_{max}}{y_{allow}} = \frac{y_{max}/h}{y_{allow}/h} = \frac{y_{max}/h}{0.025} \le 1.0 \tag{6.72}$$

where h is the height to the top of the pole (i.e., full length of pole).

The T3SAP software computes the demand-capacity ratio defined by Eq. (6.72), reports the computed value in the output file and issues a warning if the demand-capacity ratio exceeds 0.95.

6.4.1.3 Multi-Span Span Wire Structure

If two span wires are attached to a pole, the resultant lateral force applied by the span wires is used to compute the maximum lateral deflection, y_{max}, at the top of the pole and then the software computes the demand-capacity ratio defined by Eq. (6.72), reports the computed value in the output file and issues a warning if the demand-capacity ratio exceeds 0.95.

6.4.1.4 Slope of Vertical Supports: Mast Arm Structure Pole

Per Section 10.4.2.1 of the AASHTO Spec., under dead loading (Group I load combination), the pole top slope is limited to 30 mm/m (0.03 radians = 1.72 degrees) when the pole is subjected to a moment. This occurs for mast arm structures where the dead loads induce shear and moment in the mast arm and thus a moment is applied at the mast arm connection.

6.4.1.5 Single-Mast Mast Arm Structure

The T3SAP software computes the slope at the top of the pole based on the assumption of linear elastic behavior of the pole. The slope is dependent on the pole cross-section geometry and whether the pole is uniform, tapered, or stepped along its length. The cross-section geometry of the steel tubular pole may be round, hexdecagonal (16-sided), dodecagonal (12-sided) or octagonal (8-sided). Expressions for the maximum slope, $\theta_{\max}$, which occurs at the top of the pole, are given in Section 5.6.2 of this report and were taken from AASHTO Spec. Appendix B.3.

The demand-capacity ratio and serviceability criterion is:

$$\frac{demand}{capacity} = \frac{\theta_{max}}{\theta_{allow}} = \frac{\theta_{max}}{0.03} \leq 1.0 \tag{6.73}$$

The T3SAP software computes the demand-capacity ratio defined by Eq. (6.73), reports the computed value in the output file and issues a warning if the demand-capacity ratio exceeds 0.95.

6.4.1.6 Multi-Arm Mast Arm Structure

If two arms are attached to a pole, the resultant moment applied to the pole by the mast arms is used to compute the maximum slope, $\theta_{\max}$, at the top of the pole and then the T3SAP software computes the demand-capacity ratio defined by Eq. (6.73), reports the computed value in the output file and issues a warning if the demand-capacity ratio exceeds 0.95.

6.4.2 Mast Arm Deflection

Per Section 10.4.2.2 of the AASHTO Spec., adequate stiffness shall be provided for the horizontal supports of cantilevered traffic signal support structures that will result in acceptable serviceability performance. However, no specific dead load deflection limit is specified. Per the Commentary (C10.4.2.2) of the AASHTO Spec., the stiffness requirements are left to the designer. Thus, the T3SAP software does not perform any assessment regarding the deflection of mast arms.

6.4.3 Fatigue-Related Serviceability

Per AASHTO Spec. Section 11.8, it is recommended that the mast arm tip vertical deflection, δ_{max}, be limited to 8 inches when the equivalent static design wind effect from galloping and truck-induced gusts are applied to the structure. Per AASHTO Spec. Section 11.7, the fatigue-related wind effects are to be considered separately and thus the software checks this vertical deflection limit for galloping loads and then separately checks this limit for truck-induced gust.

The demand-capacity ratio and serviceability criterion is:

$$\frac{demand}{capacity} = \frac{\delta_{max}}{\delta_{allow}} = \frac{\delta_{max}}{8} \leq 1.0 \tag{6.74}$$

where δ_{max} is the maximum vertical deflection (in inches) of the mast arm (occurs at mast arm tip) under equivalent static wind load from galloping and truck-induced gust.

The T3SAP software computes the demand-capacity ratio defined by Eq. (6.74), reports the computed value in the output file and issues a warning if the demand-capacity ratio exceeds 0.95.

6.5 CAPACITY ASSESSMENT OF DAMAGED POLES

As part of the project to develop the T3SAP software, it was decided, in consultation with NYSDOT, that the T3SAP software would not be extended to accommodate damaged poles that are part of a traffic signal support structure. Rather, it is recommended that existing software (DPOLES, BEST 2001), which was specifically developed to assess the condition of a damaged/dented pole, be used instead. Both span wire and mast arm poles can be evaluated by DPOLES. The DPOLES software is regarded as an assessment tool that is complementary to the T3SAP software in that T3SAP is applicable to undamaged structures while DPOLES is applicable to damaged structures.

DPOLES was developed by the BEST Center (Bridge Engineering Software & Technology Center) of the University of Maryland and is available online at no cost. Although a user manual is available, since the software is freeware, there is no technical support. According to the user manual (BEST 2001), "DPOLES can be used to evaluate the structural integrity of structures with damaged poles. An immediate decision can be made as to the integrity of the structure and the possible need to close the affected traffic lanes until the structure is removed." Furthermore, DPOLES is based on AASHTO design requirements and thus, from that perspective, is consistent with the T3SAP software. For example, the final output of DPOLES includes a combined stress ratio (capacity-demand ratio) for each load combination and a recommendation regarding the safety of the pole.

7. CONCLUSIONS

This report describes the development of software (T3SAP – Traffic Signal Support Structural Analysis Program) for structural analysis and capacity assessment of traffic signal support structures (span wire structures and mast arm structures). Structural analysis includes computation of loads and the resulting internal forces and deflections while capacity assessment includes evaluation of strength criteria (demand-capacity stress ratios) for major components of the structure and serviceability criteria (demand-capacity deflection ratios). The specific criteria used in the demand-capacity computations are based on allowable stress design and are defined in the AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals" (AASHTO 2013). In addition, design criteria defined in the NYSDOT "Standard Specifications" (NYSDOT 2015a) and NYSDOT Standard Sheets (NYSDOT 2015b) was utilized.

A particularly challenging aspect of developing the software was determining an appropriate method for wind load analysis of both single-span and multi-span span wire structures. Various approaches to defining the application of wind loading and performing analysis under combined gravity (dead and ice) and lateral (wind) loading were considered, eventually leading to simplified methods for applying the wind loads while generating reasonably accurate predictions of internal forces, which could then be used for capacity assessment. The final recommendations, which have been adopted in the T3SAP software, for how to apply wind loads and perform analysis of both tethered and non-tethered span wire structures are provided below.

Tethered Span Wire Structures

The AASHTO simplified method for analysis of span wire structures under dead, ice, and wind loads, and with the proposed assumption that the tether wire (upper or lower) releases under design wind speeds, is recommended for analysis of tethered span wire structures. In this method 100% of the wind load on attachments is applied to the main wire with no reduction for rotation since the exact wind speed at which the tether wire will release is unknown (it may not release and thus there would be no rotation). This simplified approach to wind load analysis is recommended herein over explicit consideration of the presence of the tether wire since explicit consideration of the tether wire produced overly conservative results (Gorab 2016) and the actual behavior when the tether wire is present is not well understood. These recommendations for tethered span wire structures have been adopted in the T3SAP software.

Non-Tethered Span Wire Structures

The AASHTO simplified method for analysis of span wire structures, with modification of wind loads on attachments to account for attachment rotation, is an appropriate method of analysis for non-tethered span wire structures. In this method, for the Group II load combination in which 100% of wind load is applied, it is recommended that the effective projected area (EPA) of traffic signals and signs be modified to account for rotation. In addition, it is recommended that the drag coefficient for traffic signals be modified to account for rotation. This method of defining wind load on non-tethered traffic signals is recommended herein since it has a stronger experimental basis and broader applicability than the method currently used by NYSDOT (which prescribes reduced values of wind loads for specific signal head configurations). For the Group III load combination in which dead and ice load are applied along with 50% of the wind load, it is recommended that, for non-tethered attachments, the wind load be computed as described in the AASHTO Specifications (i.e., no reduction for rotation since the force that induces attachment rotation (wind load) is halved and the force resisting signal rotation is increased (ice load added to dead load)). These recommendations for defining wind loading on attachments of non-tethered span wire structures, including the recommended rotation and drag coefficient values presented herein, have been adopted in the T3SAP software.

Tethered and Non-Tethered Box Span Wire Structures

The critical wind direction for box span wire structures is generally not evident by inspection and thus a series of wind load analyses are needed to determine the critical direction. As an alternative to performing these analyses, an approximate method for wind load application has been proposed herein in which the pole force for the critical wind direction is determined as follows: the full wind load (100%) is applied normal to each span that the pole supports, each span is analyzed per Appendix A of the AASHTO Spec. (AAHSTO 2013) as if it were a single span structure, and the resultant horizontal force on the pole is taken as the vector sum of the horizontal forces acting on the pole from each span (referred to herein as the “full plus full” method).

For tethered box span wire structures, all wind load is assumed to be transferred to the main wire (even if the tether wire breaks, no wind load reduction is taken even if the attachment would rotate) and both the front and side areas of attachments are included in EPA computations.

For non-tethered box span wire structures, wind loads are reduced to account for attachment rotation. The angle of rotation is estimated based on wind speed and the EPA of attachments is reduced according to the angle of rotation. Furthermore, for traffic signals, the drag coefficient is modified based on wind speed. In addition, only the front area of attachments is included (side area is excluded) in EPA computations.

The above recommendations for box span wire structures have been adopted in the T3SAP software.

Recommendations for Future Work

With the movement towards load and resistance factor design (LRFD) methodology in the field of structural engineering, AASHTO began to develop its specifications using the LRFD methodology. The 2013 edition of the AASHTO Specifications retained the allowable stress design approach due to the lack of information necessary to establish a rational load and resistance factor design approach. This information was not available in existing LRFD codes and specifications and a significant effort was required to perform the necessary probability-based studies for the various types of support structures. These studies were performed as part of NCHRP Project 10-80 (TRB 2015) which provided the basis for the AASHTO “LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals” published in 2015 (AASHTO 2015). In this design methodology, load factors are applied to force effects to account for the variability of loads, the lack of accuracy in structural analysis, and the probability of simultaneous occurrence of different loads. For a given limit state, resistance factors are applied to nominal resistance to account for variability of material properties, structural dimensions and workmanship, and uncertainty in the prediction of resistance. Initial examination of the AASHTO LRFD Specifications (AASHTO 2015) by industry professionals indicated that the specifications are likely more conservative than the AASHTO ASD Specifications (AASHTO 2013).

Since the AASHTO LRFD Specifications may be more conservative than the AASHTO ASD Specifications, the owners of structures may be reluctant to use the LRFD Specifications when evaluating the capacity of an existing structure to which load modifications have been proposed. Since evaluation of the capacity of existing structures was one of the primary motivations for the development of the T3SAP software, it was decided that the initial version of the software should perform capacity assessment in accordance with the ASD Specifications. However, it is expected that new traffic signal support structures will eventually be required to be designed in accordance with the LRFD Specifications and thus it is recommended that the software be expanded to include the capability of performing analysis and capacity assessment in accordance with the LRFD Specifications.

There are some key differences between the LRFD and ASD Specifications that can be highlighted. In the LRFD Specifications, load combinations are specified to reflect general limit states intended to provide for a buildable, serviceable structure capable of safely carrying design loads for a specified time. It has been shown (TRB 2014) that "ice and wind on ice does not practically control the critical load effect" and therefore "these load combinations have been eliminated" (AASHTO 2015). The general limit states defined in the LRFD Specifications are service, fatigue, strength, and extreme event, and the corresponding load combinations are different from those specified in the ASD Specifications. The service limit state is avoided by imposing restrictions on stress and deformation under service load conditions. The fatigue limit state is avoided by ensuring that the expected fatigue load effects remain below the constant amplitude fatigue limit resistance. The strength limit state is avoided by ensuring that strength and stability, both local and global, are provided to resist the statistically significant load combinations that a structure is expected to experience. The extreme event limit state is avoided by ensuring that a structure does not collapse during a major wind event.

In the LRFD Specifications, the mean recurrence interval to be used for selection of basic wind speed has been modified to account for the consequences of failure. Accordingly, the design wind speeds are generally different than those in the ASD Specifications. The wind importance factor, which is included in the ASD Specifications to allow for adjustment of the wind pressures to reflect different mean recurrence intervals, is omitted in the LRFD Specifications. An addition to the wind load computation in the LRFD Specifications is a directionality factor that accounts for the fact that the maximum wind can come from any direction and the probability that the maximum drag coefficient is associated with the maximum wind.

8. STATEMENT ON IMPLEMENTATION

This project resulted in a software program for structural analysis and capacity assessment of traffic signal support structures in accordance with the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals (AASHTO 2013) and the New York State Department of Transportation Standard Specifications (NYSDOT 2015a). The software is called T3SAP (Traffic Signal Support Structural Analysis Program) and was developed using Visual Basic .NET (VB.NET). A training session for users of the software was conducted in September 2018 at the Main Office of NYSDOT in Albany, NY. In addition, prior to submission of this final report, prerelease versions of the software were provided to the NYSDOT for preliminary evaluation.

The first major release of the software (T3SAP, version 1.0.0) has been provided to the NYSDOT along with a user manual. T3SAP (version 1.0.0) is a fully functioning version of the software that has been extensively tested, including detailed verification studies that are presented herein in Appendix I and II.

9. REFERENCES

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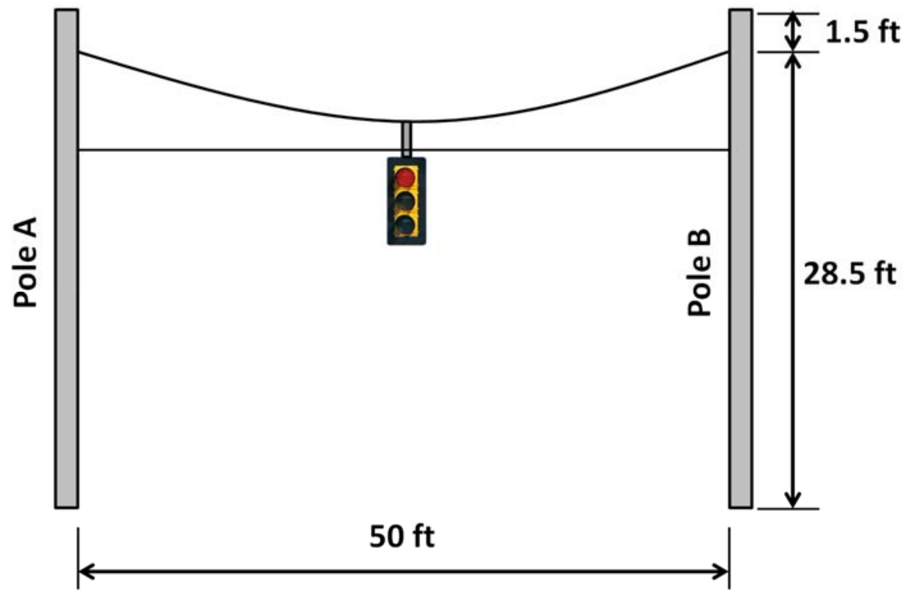
APPENDIX I: VERIFICATION STUDY FOR SINGLE-SPAN SPAN WIRE STRUCTURE

Verification Example 1:

Single-Span Span Wire Structure



Description of Structure

Location: Rensselaer County

Span wire structure with a single span wire and a single attachment.

Attachment: Tethered 3-section aluminum traffic signal with 12 in. diameter LED lenses and aluminum backplate with 5 in. border

Attachment location: Mid-span

Hanger: 1-way hanger with 1.5 ft conduit length (no disconnect)

Horizontal projected length of span wire (span length): 50 ft

Span wire sag: 5%

Height of poles: 30 ft

Cross-section of poles: Round (no taper)

Diameter of poles: 12 in.

Wall thickness of poles: ¼ in.

Pole anchor bolts: Four 1-in. diameter bolts

Pole and anchor bolt yield stress: 36 ksi

Anchor bolt circle diameter: 16 in.

Anchor bolt lower nut to foundation distance: 1 in.

Pole base plate thickness: ¾ in.

Hand hole: 2 ft from base, 4 in. wide, 1/4 in. thick frame, 1.5 in. frame depth

T3SAP Output File

```
6/26/2020 10:39:15 PM
----------------------------------------------------------------------------------------------------
Output from T3SAP 1.0.0 by NYSDOT
Analysis procedures based on AASHTO Spec 6th edition (2013)
----------------------------------------------------------------------------------------------------
PIN Number:         112233
Project Name:       Verification Study #1
County Name:        Rensselaer
Intersection Name:  Hypothetical
Signal Number:      123
Wind Speed:         90 MPH (3-sec Gust Speed)
Units:              U.S. Customary

Designed By:        NG-CZ-CV-MS

Analysis Type:      Analysis Type II
Structure Type:     Single Spanwire

Load Combinations:  Group I   (D)
                    Group II  (D + W)
                    Group III (D + I + 0.5W)
----------------------------------------------------------------------------------------------------
Cable Type:         Tethered
Pole A Height (ft): 30
Pole B Height (ft): 30
Span Length (ft):   50
Sag (%):            5
----------------------------------------------------------------------------------------------------
                    Span AB Point Loads (lb)
                                                                                  Signal
                             Sign                Other
Point Load  Dead Load  Ice Load  Wind Load  Att Type  |Head Mat  Lens Mat  Bckp Mat  Bckp Size  Dsc Hngr  8" Sec
12" Sec  # Ways|  |Type       Hanger      Area (ft²)|  |HPA|  LtL Distance (ft)
    1        75.92       85.00      248.95    Signal    Aluminum    LED     Aluminum    5-inch      No      0
3       1      --          --          --           --          25
----------------------------------------------------------------------------------------------------
                    Span AB Pole Reactions At Span Wire Attachment (lb)

                         Pole A                         Pole B
   Load        Horizontal       Vertical       Horizontal      Vertical
Combination     Reaction        Reaction        Reaction       Reaction
  Group I        379.58          37.96           379.58         37.96
  Group II       1307.22         37.96           1307.22        37.96
  Group III      1019.01         80.46           1019.01        80.46
----------------------------------------------------------------------------------------------------
                    Estimate of Base OT Moment (kip-ft)

   Load          Pole A          Pole B
Combination
  Group I        11.39           11.39
  Group II       39.22           39.22
  Group III      30.57           30.57
----------------------------------------------------------------------------------------------------

                    Cable D/C Ratios

  Cable      Group I    Group II     Group III
    AB        0.068      0.174         0.136
```

Pole A D/C Stress Ratios: Group I

Location Measured From Pole Base (ft)	Axial	Shear	Bending	Combined (CSR Allowable = 1.0)
0	0.024	0.007	0.202	0.208
2	0.023	0.007	0.222	0.228
28.5	0.002	0.007	0	0.000

Pole A D/C Stress Ratios: Group II

Location Measured From Pole Base (ft)	Axial	Shear	Bending	Combined (CSR Allowable = 1.0)
0	0.018	0.019	0.534	0.541
2	0.017	0.019	0.587	0.594
28.5	0.002	0.018	0.000	0.001

Pole A D/C Stress Ratios: Group III

Location Measured From Pole Base (ft)	Axial	Shear	Bending	Combined (CSR Allowable = 1.0)
0	0.024	0.014	0.411	0.418
2	0.023	0.015	0.452	0.459
28.5	0.003	0.014	0.000	0.001

Pole A Base Reactions

Load Combination	Axial (lb)	Shear (lb)	Overturning Moment (kip-ft)	Twisting Moment (kip-ft)
Group I	979.55	379.58	10.82	0
Group II	979.55	1385.44	38.12	0
Group III	1304.79	1044.53	29.32	0

Anchor Bolt & Connection D/C Ratios Pole A

Load Combination	Combined Shear & Compression (Bolt)	Combined Shear & Tension (Bolt)	Bearing (Base Plate)
Group I	0.494	0.630	0.006
Group II	3.325	4.624	0.023
Group III	1.992	2.701	0.017

Pole A Top Deflection D/C Ratio

Group I : 0.131
Group II : NA
Group III: NA

End of T3SAP Output File

NOTE: In Notepad, word wrap was turned on for displaying the output file in this report. When viewing the output file on a computer, turning word wrap off should improve readability of the file.

Comparison of Results

	T3SAP	Hand Computations	SAP2000
Dead Load at Signal Attachment (lb) (includes tributary dead load of cable)	75.92	75.92	75.92
Ice Load at Signal Attachment (lb) (includes tributary ice load of cable)	85.00	85.00	85.00
Wind Load at Signal Attachment (lb) (includes tributary wind load of cable)	248.95	248.95	248.95
Group I: **Span Wire Horizontal Reaction (lb)**	379.58	379.58	379.12
Group I: **Span Wire Vertical Reaction (lb)**	37.96	37.96	37.96
Group II: **Span Wire Horizontal Reaction (lb)**	1307.22	1307.24	1305.65
Group II: **Span Wire Vertical Reaction (lb)**	37.96	37.96	37.96
Group III: **Span Wire Horizontal Reaction (lb)**	1019.01	1019.10	1017.84
Group III: **Span Wire Vertical Reaction (lb)**	80.46	80.46	80.46
Group I: **Estimate of Base OT Moment (kip-ft)**	11.39	11.39	---
Group II: **Estimate of Base OT Moment (kip-ft)**	39.22	39.22	---
Group III: **Estimate of Base OT Moment (kip-ft)**	30.57	30.57	---
Group I: **Cable D/C Ratio**	0.068	0.068	---
Group II:	0.174	0.174	---

Cable D/C Ratio			
Group III: **Cable D/C Ratio**	0.136	0.136	---
Group I: **Pole Axial Stress D/C Ratio at Pole Base**	0.0235	0.0236	---
Group I: **Pole Shear Stress D/C Ratio at Pole Base**	0.0069	0.0069	---
Group I: **Pole Bending Stress D/C Ratio at Pole Base**	0.2017	0.2017	---
Group I: **Pole Combined Stress D/C Ratio at Pole Base**	0.2075	0.2075	---
Group I: **Pole Axial Stress D/C Ratio at Hand Hole**	0.0227	0.0227	---
Group I: **Pole Shear Stress D/C Ratio at Hand Hole**	0.0071	0.0071	---
Group I: **Pole Bending Stress D/C Ratio at Hand Hole**	0.2220	0.2239	---
Group I: **Pole Combined Stress D/C Ratio at Hand Hole**	0.2278	0.2286	---
Group I: **Pole Axial Stress D/C Ratio at Span Wire Connection**	0.0020	0.0020	---
Group I: **Pole Shear Stress D/C Ratio at Span Wire Connection**	0.0069	0.0069	---
Group I: **Pole Bending Stress D/C Ratio at Span Wire Connection**	0	0	---
Group I: **Pole Combined Stress D/C Ratio at Span Wire Connection**	0.0005	0.0005	---
Group II: **Pole Axial Stress D/C Ratio at Pole Base**	0.0177	0.0177	---

Group II: **Pole Shear Stress D/C Ratio at Pole Base**	0.0190	0.0190	---
Group II: **Pole Bending Stress D/C Ratio at Pole Base**	0.5342	0.5329	---
Group II: **Pole Combined Stress D/C Ratio at Pole Base**	0.5410	0.5394	---
Group II: **Pole Axial Stress D/C Ratio at Hand Hole**	0.0171	0.0170	---
Group II: **Pole Shear Stress D/C Ratio at Hand Hole**	0.0195	0.0194	---
Group II: **Pole Bending Stress D/C Ratio at Hand Hole**	0.5869	0.6194	---
Group II: **Pole Combined Stress D/C Ratio at Hand Hole**	0.5935	0.6262	---
Group II: **Pole Axial Stress D/C Ratio at Span Wire Connection**	0.0015	0.0015	---
Group II: **Pole Shear Stress D/C Ratio at Span Wire Connection**	0.0179	0.0179	---
Group II: **Pole Bending Stress D/C Ratio at Span Wire Connection**	0.0002	0.0002	---
Group II: **Pole Combined Stress D/C Ratio at Span Wire Connection**	0.0008	0.0008	---
Group III: **Pole Axial Stress D/C Ratio at Pole Base**	0.0235	0.0235	---
Group III: **Pole Shear Stress D/C Ratio at Pole Base**	0.0143	0.0143	---
Group III: **Pole Bending Stress D/C Ratio at Pole Base**	0.4109	0.4099	---
Group III: **Pole Combined Stress D/C Ratio at Pole Base**	0.4181	0.4171	---

Group III: **Pole Axial Stress D/C Ratio at Hand Hole**	0.0228	0.0227	---
Group III: **Pole Shear Stress D/C Ratio at Hand Hole**	0.0147	0.0147	---
Group III: **Pole Bending Stress D/C Ratio at Hand Hole**	0.4520	0.4527	---
Group III: **Pole Combined Stress D/C Ratio at Hand Hole**	0.4592	0.4599	---
Group III: **Pole Axial Stress D/C Ratio at Span Wire Connection**	0.0025	0.0026	---
Group III: **Pole Shear Stress D/C Ratio at Span Wire Connection**	0.0140	0.0140	---
Group III: **Pole Bending Stress D/C Ratio at Span Wire Connection**	0.0001	0.0001	---
Group III: **Pole Combined Stress D/C Ratio at Span Wire Connection**	0.0008	0.0008	---
Group I: **Pole Base Axial Force Reaction (lb)**	979.55	979.55	978.63
Group I: **Pole Base Shear Force Reaction (lb)**	379.58	379.58	379.58
Group I: **Pole Base OTM Reaction (kip-ft)**	10.82	10.82	10.82
Group II: **Pole Base Axial Force Reaction (lb)**	979.55	979.55	978.55
Group II: **Pole Base Shear Force Reaction (lb)**	1385.44	1385.46	1385.18
Group II: **Pole Base OTM Reaction (kip-ft)**	38.12	38.12	38.11

Group III: **Pole Base Axial Force Reaction (lb)**	1304.79	1304.79	1303.80
Group III: **Pole Base Shear Force Reaction (lb)**	1044.53	1044.62	1039.55
Group III: **Pole Base OTM Reaction (kip-ft)**	29.32	29.32	29.18
Group I: **Anchor Bolt Combined Shear & Compression D/C Ratio**	0.494	0.494	---
Group I: **Anchor Bolt Combined Shear & Tension D/C Ratio**	0.630	0.630	---
Group I: **Anchor bolt bearing D/C ratio**	0.006	0.006	---
Group II: **Anchor Bolt Combined Shear & Compression D/C Ratio**	3.325	3.326	---
Group II: **Anchor Bolt Combined Shear & Tension D/C Ratio**	4.624	4.624	---
Group II: **Anchor bolt bearing D/C ratio**	0.023	0.023	---
Group III: **Anchor Bolt Combined Shear & Compression D/C Ratio**	1.992	1.992	---
Group III: **Anchor Bolt Combined Shear & Tension D/C Ratio**	2.701	2.702	---
Group III: **Anchor bolt bearing D/C ratio**	0.017	0.017	---

Group I: **Pole Top Lateral Deflection D/C Ratio**	0.131	0.131	0.131

NOTE: --- indicates output value is not available in SAP2000

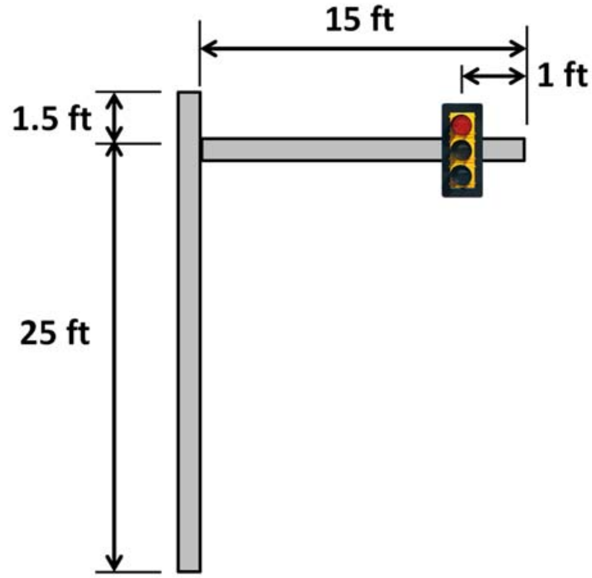
Conclusion:

Hand computations and SAP2000 results are virtually identical to output from T3SAP. Thus, T3SAP is verified for this single span wire case.

APPENDIX II: VERIFICATION STUDY FOR SINGLE MAST ARM STRUCTURE

Verification Example 2:

Single Mast Arm Structure



Description of Structure
Location: Rensselaer County
Mast arm structure with a single mast arm and a single attachment.
Attachment: Tethered 3-section aluminum traffic signal with 12 in. diameter LED lenses and aluminum backplate with 5 in. border
Attachment location: 14 ft from mast arm connection and attached with bracket
Horizontal projected length of mast arm: 15 ft
Cross-section of mast arm: Round with taper from 10 in. to 5 in.
Wall thickness of mast arm: ¼ in.
Mast arm connection bolts: Four 1-in. diameter bolts
Mast arm and connection bolts yield stress: 36 ksi
Mast arm connection height: 14 in.
Mast arm connection width: 12 in.
Mast arm flange plate thickness: ¾ in.
Height of pole: 26.5 ft
Pole height to mast arm connection: 25 ft
Cross-section of pole: Round (no taper)
Diameter of pole: 12 in.
Wall thickness of pole: ¼ in.
Pole anchor bolts: Four 1-in. diameter bolts
Pole and anchor bolt yield stress: 36 ksi
Anchor bolt circle diameter: 14 in.
Anchor bolt nut to foundation distance: 1.5 in.
Pole base plate thickness: ¾ in.
Fatigue loading: Natural wind gust with fatigue importance category I.

T3SAP Output File

```
5/30/2020 4:10:26 PM
--------------------------------------------------------------------------------------------------------------------------------
-------------------------------------------------------------------
Output from T3SAP 1.0.0 by NYSDOT
Analysis procedures based on AASHTO Spec 6th edition (2013)
--------------------------------------------------------------------------------------------------------------------------------
-------------------------------------------------------------------
PIN Number:
Project Name:       Verification Example 2 - Single Mast Arm
County Name:        Rensselaer
Intersection Name:
Signal Number:
Wind Speed:         90 MPH (3-sec Gust Speed)
Units:              U.S. Customary

Designed By:        NG-CZ-CV-MS

Analysis Type:      Analysis Type II
Structure Type:     Single Mast Arm

Load Combinations:  Group I   (D)
                    Group II  (D + W)
                    Group III (D + I + 0.5W)
--------------------------------------------------------------------------------------------------------------------------------
-------------------------------------------------------------------

Connection Height (ft): 25
Arm Length (ft):        15
--------------------------------------------------------------------------------------------------------------------------------
-------------------------------------------------------------------

                    Arm Point Loads (lb)
                                                                                       Signal
                    Sign                 Other
Point Load  Dead Load  Ice Load  Wind Load  Att Type  |Head Mat  Lens Mat  Bckp Mat  Bckp Size  Dsc Hngr  8" Sec  12" Sec  #
Ways|   |Type       Hanger      Area (ft²)|  |HPA|  LtL Distance (ft)
    1        47.32      70.00      220.35    Signal     Aluminum     LED     Aluminum    5-inch      No        0        3
1      --         --         --           --          14
--------------------------------------------------------------------------------------------------------------------------------
-------------------------------------------------------------------


                    Estimate of Base OT Moment (kip-ft)

   Load
Combination
  Group I        3.47
  Group II       8.98
  Group III      7.21
--------------------------------------------------------------------------------------------------------------------------------
-------------------------------------------------------------------

                    Pole Reactions At Mast Arm 1 Connection

   Load         Transverse        Bending
Combination     Shear (lb)     Moment(kip-ft)
  Group I        337.81           2.84
  Group II       513.87           5.18
  Group III      532.60           4.98
--------------------------------------------------------------------------------------------------------------------------------
-------------------------------------------------------------------

                    Mast Arm to Pole Connection D/C Ratio

   Load               Combined Tension            Bearing
Combination            & Shear (Bolt)           (Base Plate)
  Group I                 0.016                    0.006
  Group II                0.067                    0.008
  Group III               0.053                    0.009
--------------------------------------------------------------------------------------------------------------------------------
-------------------------------------------------------------------
```

Arm D/C Stress Ratios: Group I

Location Measured From Arm Connection (ft)	Axial	Shear	Bending	Combined (CSR Allowable = 1.0)
0	0	0.007	0.077	0.077
14	0	0.003	0.001	0.001

Arm D/C Stress Ratios: Group II

Location Measured From Arm Connection (ft)	Axial	Shear	Bending	Combined (CSR Allowable = 1.0)
0	0	0.008	0.106	0.106
14	0	0.008	0.001	0.001

Arm D/C Stress Ratios: Group III

Location Measured From Arm Connection (ft)	Axial	Shear	Bending	Combined (CSR Allowable = 1.0)
0	0	0.009	0.101	0.101
14	0	0.006	0.001	0.001

Fatigue Life Cannot Be Determined for Arm Connection

- Check connection geometry, Plate Thickness < 2 in.
- Check connection geometry, KF > 3.2
- Check connection geometry, KI > 7.7

Pole A D/C Stress Ratios: Group I

Location Measured From Pole Base (ft)	Axial	Shear	Bending	Combined (CSR Allowable = 1.0)
0	0.022	0	0.053	0.059
25	0.007	0	0.053	0.055

Pole A D/C Stress Ratios: Group II

Location Measured From Pole Base (ft)	Axial	Shear	Bending	Combined (CSR Allowable = 1.0)
0	0.016	0.072	0.224	0.235
25	0.005	0.068	0.040	0.046

Pole A D/C Stress Ratios: Group III

Location Measured From Pole Base (ft)	Axial	Shear	Bending	Combined (CSR Allowable = 1.0)
0	0.022	0.036	0.141	0.149
25	0.008	0.034	0.063	0.067

Pole A Base Reactions

Load Combination	Axial (lb)	Shear (lb)	Overturning Moment (kip-ft)	Twisting Moment (kip-ft)
Group I	1169.55	0	2.84	0
Group II	1169.55	729.55	16.00	4.43
Group III	1577.66	364.77	10.05	2.22

Anchor Bolt & Connection D/C Ratios Pole A

Load Combination	Combined Shear & Compression (Bolt)	Combined Shear & Tension (Bolt)	Bearing (Base Plate)
Group I	0.053	0.047	0
Group II	7.943	11.023	0.136
Group III	2.352	3.144	0.068

```
                    Fatigue Life Cannot Be Determined for Pole Connection

- Check connection geometry, Plate Thickness < 2 in.
- Check connection geometry, CBC < 1.25
- Check connection geometry, KF > 3.2
- Check connection geometry, KI > 7.7

----------------------------------------------------------------------------------------------------
-----------------------------------------------------------------

                    Pole Top Deflection D/C Ratio

  Group I  : 0.076
  Group II : NA
  Group III: NA
----------------------------------------------------------------------------------------------------
-----------------------------------------------------------------
----------------------------------------------------------------------------------------------------
-----------------------------------------------------------------

                    4 D/C Ratio Limits Exceeded (> 0.95)

Fatigue Life Cannot Be Determined for Arm Connection
- Check connection geometry, Plate Thickness < 2 in.
- Check connection geometry, KF > 3.2
- Check connection geometry, KI > 7.7
------------------------------------------------------------------------------------
Pole A (Connection @ Base)
- Group II: Combined shear & compression (Bolt) D/C ratio limit exceeded
- Group II: Combined shear & tension (Bolt) D/C ratio limit exceeded
- Group III: Combined shear & compression (Bolt) D/C ratio limit exceeded
- Group III: Combined shear & tension (Bolt) D/C ratio limit exceeded
------------------------------------------------------------------------------------
Fatigue Life Cannot Be Determined for Pole Connection
- Check connection geometry, Plate Thickness < 2 in.
- Check connection geometry, CBC < 1.25
- Check connection geometry, KF > 3.2
- Check connection geometry, KI > 7.7
------------------------------------------------------------------------------------
```

Note that the above output file is for pole base plate and mast arm flange plate thickness of 0.75 inches. This thickness values results in a message saying that fatigue life cannot be determined for either the arm connection or the pole connection and that connection geometry should be checked. Increasing the plate thicknesses to 2.0 inches results in the following additional output which indicates that that the mast arm connection is now acceptable but that the pole connection requires additional modification.

Additional output related to Group IV loading (fatigue):.

```
                    Infinite Fatigue Life D/C Ratios Arm

Mast Arm Transverse Plate to Mast Arm Connection For Natural Wind Gust: 0.146
Mast Arm Connection Bolts For Natural Wind Gust: 0.132

                    Finite Fatigue Life Approximate Number of Lifetime Cycles Allowed for Arm

Mast Arm Transverse Plate to Mast Arm Connection For Natural Wind Gust: 1.387E+009
Mast Arm Connection Bolts For Natural Wind Gust: 2.782E+009
----------------------------------------------------------------------------------------------------
```

```
                    Fatigue Life Cannot Be Determined for Pole Connection

- Check connection geometry, CBC < 1.25

-------------------------------------------------------------------------------------------
```

End of T3SAP Output File

NOTE: In Notepad, word wrap was turned on for displaying the output file in this report. When viewing the output file on a computer, turning word wrap off should improve readability of the file.

Comparison of Results

	T3SAP	Hand Computations	SAP2000
Dead Load at Signal Attachment (lb)	47.32	47.32	47.32
Ice Load at Signal Attachment (lb)	70.00	70.00	70.00
Wind Load Signal at Attachment (lb)	220.35	220.35	220.35
Group I: **Estimate of Base OT Moment (kip-ft)** *Conservative estimate based on sum, rather than resultant, of orthogonal bending moments*	3.47	3.47	3.47
Group II: **Estimate of Base OT Moment (kip-ft)** *Conservative estimate based on sum, rather than resultant, of orthogonal bending moments*	8.98	8.98	8.98
Group II: **Estimate of Base OT Moment (kip-ft)** *Conservative estimate based on sum, rather than resultant, of orthogonal bending moments*	7.21	7.21	7.21
Group I: **Reactions at Mast Arm Connection:** **Transverse Shear (lb)**	337.81	337.81	337.81
Group I: **Reactions at Mast Arm Connection:** **Bending Moment (kip-ft)**	2.84	2.84	2.84
Group II: **Reactions at Mast Arm Connection:** **Transverse Shear (lb)**	513.87	513.87	513.88
Group II: **Reactions at Mast Arm Connection:** **Bending Moment (kip-ft)**	5.18	5.18	5.18

Group III: Reactions at Mast Arm Connection: Transverse Shear (lb)	532.60	532.60	532.61
Group III: Reactions at Mast Arm Connection: Bending Moment (kip-ft)	4.98	4.98	4.98
Group I: Mast Arm Axial Stress D/C Ratio at Connection	0	0	---
Group I: Mast Arm Shear Stress D/C Ratio at Connection	0.0074	0.0074	---
Group I: Mast Arm Bending Stress D/C Ratio at Connection	0.0769	0.0769	---
Group I: Mast Arm Combined Stress D/C Ratio at Connection	0.0770	0.0770	---
Group I: Mast Arm Axial Stress D/C Ratio at Signal Attachment	0	0	---
Group I: Mast Arm Shear Stress D/C Ratio at Signal Attachment	0.0026	0.0026	---
Group I: Mast Arm Bending Stress D/C Ratio at Signal Attachment	0.0007	0.0007	---
Group I: Mast Arm Combined Stress D/C Ratio at Signal Attachment	0.0007	0.0007	---
Group II: Mast Arm Axial Stress D/C Ratio at Connection	0	0	---
Group II: Mast Arm Shear Stress D/C Ratio at Connection	0.0085	0.0085	---

Group II: **Mast Arm Bending Stress D/C Ratio at Connection**	0.1055	0.1055	---
Group II: **Mast Arm Combined Stress D/C Ratio at Connection**	0.1056	0.1056	---
Group II: **Mast Arm Axial Stress D/C Ratio at Signal Attachment**	0	0	---
Group II: **Mast Arm Shear Stress D/C Ratio at Signal Attachment**	0.0076	0.0076	---
Group II: **Mast Arm Bending Stress D/C Ratio at Signal Attachment**	0.0007	0.0007	---
Group II: **Mast Arm Combined Stress D/C Ratio at Signal Attachment**	0.0007	0.0007	---
Group III: **Mast Arm Axial Stress D/C Ratio at Connection**	0	0	---
Group III: **Mast Arm Shear Stress D/C Ratio at Connection**	0.0088	0.0088	---
Group III: **Mast Arm Bending Stress D/C Ratio at Connection**	0.1014	0.1014	---
Group III: **Mast Arm Combined Stress D/C Ratio at Connection**	0.1015	0.1015	---
Group III: **Mast Arm Axial Stress D/C Ratio at Signal Attachment**	0	0	---
Group III: **Mast Arm Shear Stress D/C Ratio at Signal Attachment**	0.0056	0.0056	---

Group III: **Mast Arm Bending Stress D/C Ratio at Signal Attachment**	0.0007	0.0007	**---**
Group III: **Mast Arm Combined Stress D/C Ratio at Signal Attachment**	0.0007	0.0007	**---**
Group I: **Mast Arm to Pole Connection Bolt Combined Tension and Shear D/C Ratio**	0.0159	0.0159	---
Group I: **Mast Arm to Pole Connection Flange Plate Bearing D/C Ratio**	0.0055	0.0055	---
Group II: **Mast Arm to Pole Connection Bolt Combined Tension and Shear D/C Ratio**	0.0669	0.0669	---
Group II: **Mast Arm to Pole Connection Flange Plate Bearing D/C Ratio**	0.0084	0.0084	---
Group III: **Mast Arm to Pole Connection Bolt Combined Tension and Shear D/C Ratio**	0.0531	0.0531	---
Group III: **Mast Arm to Pole Connection Flange Plate Bearing D/C Ratio**	0.0087	0.0087	---
Group I: **Pole Base Axial Force Reaction (lb)**	1169.55	1169.55	1169.55
Group I: **Pole Base Shear Force Reaction (lb)**	0	0	0
Group I: **Pole Base OT Moment Reaction (kip-ft)**	2.84	2.84	2.84
Group I: **Pole Base Twisting Moment Reaction (kip-ft)**	0	0	0

Group II: **Pole Base Axial Force Reaction (lb)**	1169.55	1169.55	1169.55
Group II: **Pole Base Shear Force Reaction (lb)**	729.55	729.55	684.13
Group II: **Pole Base OT Moment Reaction (kip-ft)**	16.00	16.00	12.93
Group II: **Pole Base Twisting Moment Reaction (kip-ft)**	4.43	4.43	3.98
Group III: **Pole Base Axial Force Reaction (lb)**	1577.66	1577.66	1577.67
Group III: **Pole Base Shear Force Reaction (lb)**	364.77	364.77	342.06
Group III: **Pole Base OT Moment Reaction (kip-ft)**	10.05	10.05	6.67
Group III: **Pole Base Twisting Moment Reaction (kip-ft)**	2.22	2.22	1.99
Group I: **Pole Axial Stress D/C Ratio at Connection**	0.0072	0.0072	---
Group I: **Pole Shear Stress D/C Ratio at Connection**	0	0	---
Group I: **Pole Bending Stress D/C Ratio at Connection**	0.0530	0.0530	---
Group I: **Pole Combined Stress D/C Ratio at Connection**	0.0552	0.0552	---
Group I: **Pole Axial Stress D/C Ratio at Base**	0.0219	0.0219	---
Group I: **Pole Shear Stress D/C Ratio at Base**	0	0	---
Group I: **Pole Bending Stress D/C Ratio at Base**	0.0530	0.0530	---

Group I: **Pole Combined Stress D/C Ratio at Base**	0.0591	0.0591	---
Group II: **Pole Axial Stress D/C Ratio at Connection**	0.0054	0.0054	---
Group II: **Pole Shear Stress D/C Ratio at Connection**	0.0681	0.0681	---
Group II: **Pole Bending Stress D/C Ratio at Connection**	0.0399	0.0399	---
Group II: **Pole Combined Stress D/C Ratio at Connection**	0.0462	0.0462	---
Group II: **Pole Axial Stress D/C Ratio at Base**	0.0165	0.0165	---
Group II: **Pole Shear Stress D/C Ratio at Base**	0.0721	0.0721	---
Group II: **Pole Bending Stress D/C Ratio at Base**	0.2243	0.2243	---
Group II: **Pole Combined Stress D/C Ratio at Base**	0.2352	0.2352	---
Group III: **Pole Axial Stress D/C Ratio at Connection**	0.0079	0.0079	---
Group III: **Pole Shear Stress D/C Ratio at Connection**	0.0341	0.0340	---
Group III: **Pole Bending Stress D/C Ratio at Connection**	0.0629	0.0629	---
Group III: **Pole Combined Stress D/C Ratio at Connection**	0.0666	0.0666	---
Group III: **Pole Axial Stress D/C Ratio at Base**	0.0223	0.0223	---
Group III: **Pole Shear Stress D/C Ratio at Base**	0.0361	0.0361	---
Group III: **Pole Bending Stress D/C Ratio at Base**	0.1409	0.1409	---

Group III: **Pole Combined Stress D/C Ratio at Base**	0.1491	0.1491	**---**
Group I: **Anchor Bolt Combined Shear & Compression D/C Ratio**	0.0525	0.0525	**---**
Group I: **Anchor Bolt Combined Shear & Tension D/C Ratio**	0.0467	0.0467	**---**
Group I: **Anchor Bolt Bearing D/C Ratio**	0	0	**---**
Group II: **Anchor Bolt Combined Shear & Compression D/C Ratio**	7.943	7.943	**---**
Group II: **Anchor Bolt Combined Shear & Tension D/C Ratio**	11.023	11.023	**---**
Group II: **Anchor Bolt Bearing D/C Ratio**	0.1364	0.1364	**---**
Group III: **Anchor Bolt Combined Shear & Compression D/C Ratio**	2.352	2.352	**---**
Group III: **Anchor Bolt Combined Shear & Tension D/C Ratio**	3.144	3.143	**---**
Group III: **Anchor Bolt Bearing D/C Ratio**	0.0682	0.0682	**---**
Group IV (Natural Gust Only): **Mast Arm Flange Plate to Tube Connection Infinite Fatigue Life D/C Ratio**	0.146	0.146	---
Group IV (Natural Gust Only): **Mast Arm Connection Bolts Infinite Fatigue Life D/C Ratio**	0.132	0.132	---

Group IV (Natural Gust Only): Mast Arm Flange Plate to Tube Connection Finite Fatigue Life Cycles	1.387×10^9	1.385×10^9	---
Group IV (Natural Gust Only): Mast Arm Connection Bolts Finite Fatigue Life Cycles	2.782×10^9	2.782×10^9	---
Group I: Pole Top Slope D/C Ratio	0.076	0.076	0.076

NOTE: *The analysis results in this table are for pole base plate and mast arm connection plate thicknesses of 0.75 inches EXCEPT for the Group IV loading case (fatigue) results which were obtained in a separate analysis using a thickness of 2.0 inches. As explained above, with 0.75-inch thickness, the connections are not acceptable and thus were modified to have a thickness of 2.0 inches in order to run the fatigue analysis.*

Conclusion:

Hand computations and SAP2000 results are virtually identical to output from T3SAP. Thus, T3SAP is verified for this single mast arm case.