Design of Sports Lighting Support Structures

WILL YOUR STRUCTURES PERFORM TO EXPECTATION?

ENSURING STRUCTURAL INTEGRITY

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Abstract

For many years, tubular steel poles have been utilized in various industries.

A popular support structure for the sports lighting industry due to their strength, reliability, and ease of installation, the steel pole has served the industry well. The industry has steel pole installations that have been in service upward of fifty years. However, in recent years, there have been numerous failures of sports lighting structures. While manufacturing, installation, and maintenance issues have contributed to these failures, design issues have also contributed. Careful attention to the design requirements for these structures will prolong their lifespan and ensure public safety.

As with any steel support structure, consistent application of a design standard is critical. Historically, the sports lighting industry has not utilized a consistent standard for the design of its support structures. Some pole structures are purchased to meet a recognized structural code with appropriate load and strength safety factors and others are sold as general "commercial" design. Design issues include improper factors of safety, inadequate base plate design, insufficient anchor bolts, improper application of wind and wind coefficients, undersized welds, improper material specifications, and ignoring fatigue issues. Standardizing the design process will improve the safety of these structures and reduce confusion during the procurement process.

This article will present current methods used to design sports lighting structures to the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals, 5th Edition, 2009. The document is specifically for pole structures, specifies factor of safety, addresses fatigue issues, and addresses wind induced vibration issues.

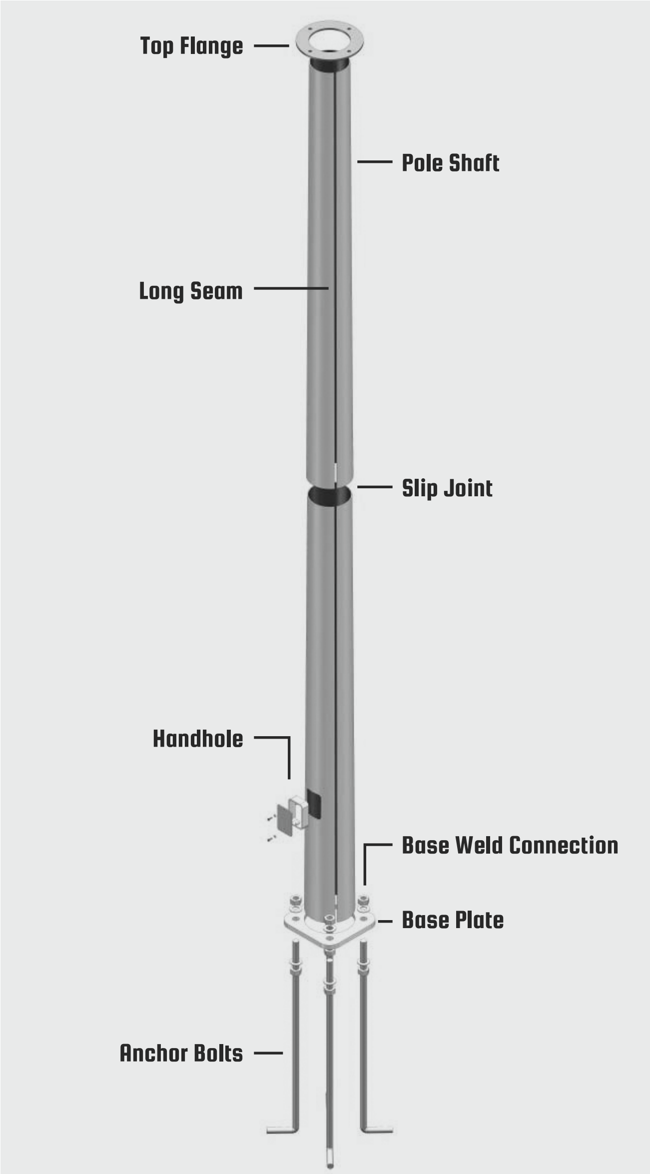
Introduction

Tubular steel poles are a popular support structure in many industries. Poles have been utilized as support structures in the sports lighting, utility, transportation, and communications industries for many decades. Combining a long history of reliable performance, competitive pricing, and ease of use and installation, steel poles are the sports lighting industry's preferred support structure and have performed admirably at some of America's most popular sporting venues. A typical installation of sports lighting poles can be seen in **Figure I** below. However, in recent years, there have been numerous failures of sports lighting structures across the country.

In some cases, the property damage has been significant. As with America's other aging infrastructure, the cost of ignoring this issue can be significant to public safety and welfare. While manufacturing, installation, and maintenance issues have contributed to these failures, design issues have also played a role. Careful attention to the design requirements for sports lighting pole structures will prolong their lifespan and ensure public safety.

Figure I

Typical sports lighting pole installation



Steel Poles

Steel poles in the sports lighting industry can be anchor based, direct burial, or stub based. AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, Fifth Edition, 2009, defines a pole as a vertical support that is long, relatively slender, and generally rounded or multisided (Specifications, 2009). Anchor based poles are supported with anchor bolts embedded into a concrete foundation. Direct burial structures are embedded into the soil. Stub based poles are flanged to a pipe section that is also directly embedded into the soil. Typically galvanized and in some cases weathering steel, steel poles are pressed in polygonal shapes or comprised of round cross sections. Polygonal pole shells are longseamed (vertical weld along pole axis joining pole half-shells) via submerged arc welding (SAW) techniques and round tapered poles are longseamed via SAW or electric resistance welding (ERW) methods. In most cases for structural efficiency, the structures taper over their height to a smaller tip diameter at the top. Steel sports lighting poles are typically fabricated with high strength steel plate and range in height from 55 ft to 150 ft. The structures can be designed to support as little as four lighting fixtures or as many as dozens of fixtures. A standard sport lighting pole and its components can be seen in **Figure 2** below.

Pole collapses in the sports lighting industry, while an infrequent occurrence, have increased in occurrence and have made news in recent years. With their proximity to areas where the public gathers for sporting events, there is a significant potential for loss of life and injury. Collapses of sports lighting poles are shown in **Figure 3**.

Figure 2 Top

Sports lighting pole structure components

Figure 3 Bottom

Sports lighting pole structure components

Lack Of Design Consistency

Historically, the sports lighting industry has not utilized a consistent standard for the design of its support structures. There has been significant latitude on design techniques for sports lighting poles causing confusion with owners, designers, and those procuring light support structures. Typically, the supporting pole structures are packaged with the light fixtures and provided as a component of the lighting system by the supplier. Unlike other industries where the support structures are purchased by a knowledgeable owner directly from the pole manufacturer, owners of sports lighting poles are typically not active in the specifications development or procurement process and are downstream in the supply chain. As a result, owners have had very little input into the specifications and design processes for their sports lighting pole structures.

Some pole structures today are purchased to meet a recognized structural code with appropriate load and strength safety factors and others are sold as general “commercial” design. Design issues include improper factors of safety, inadequate base plate design, insufficient anchor bolts, improper application of wind and wind coefficients, undersized welds, improper material specifications, and ignoring fatigue issues. These issues have resulted in drastic differences in pole structure design and quality depending on the supplier. As with any steel support structure, consistent application of a design standard is critical. Standardizing the design process will improve the safety of these structures, reduce confusion during the procurement process, and ensure the longevity of the structure.

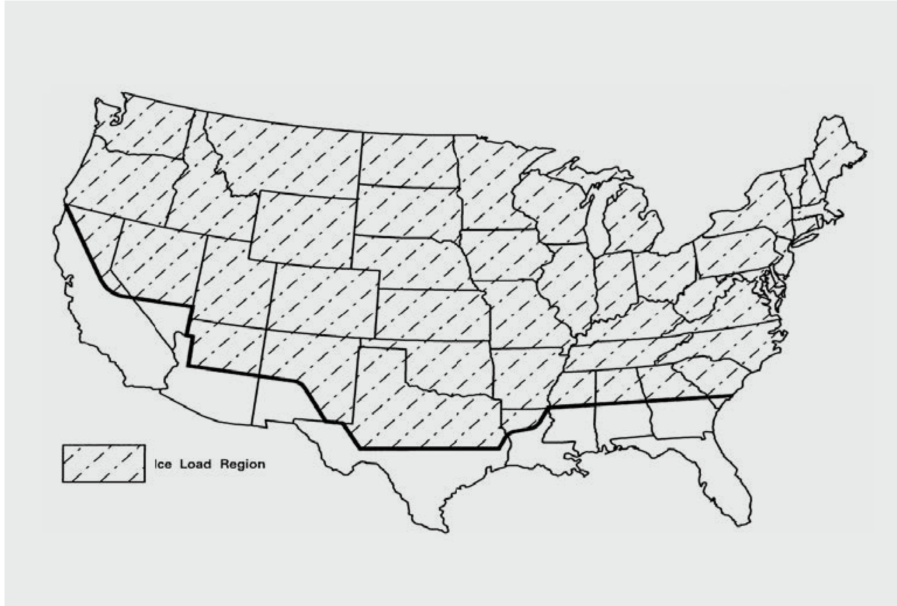
Aashto Standard Specifications

AASHTO’s *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, Fifth Edition*, (Specifications, 2009) are applicable to the structural design of supports for highway signs, luminaires, and traffic signals. The document is intended to serve as a standard and guide for the design, fabrication, and erection of these types of structures. As the only available standard to address the design of luminaire support structures, the Specifications should be utilized for the design of sports lighting poles. The Specifications state in Article 1.4.2 that structural supports for luminaires include typical lighting poles, pole top-mounted luminaire poles, and high-level poles (Specifications, 2009). Commentary C1.4.2 further defines high-level lighting poles as structures normally in heights from 55 ft (17 m) to 150 ft (46 m) or more, usually supporting four (4) to twelve (12) luminaires and used to illuminate large areas (Specifications, 2009). This definition clearly covers the application of sports lighting pole structures.

The Specifications were the result of National Cooperative Highway Research Program Project (NCHRP) 17-10 and the corresponding NCHRP Report 411 (1998) and replace the previous 2001 version of the AASHTO Standard Specifications (2001). Note that the Specifications are only the minimum requirements necessary to provide for public safety. The owner in conjunction with the designer may require the pole design be greater than the minimum requirements as established in the Specifications.

Pole Loading

Section 3 of the Specifications specifies the minimum requirements for loads and forces, the limits of their application, and load combinations that are used for the design of lighting pole structures. Criteria for dead load, live load, ice load, and wind load is addressed.



Dead Load

Consists of the weight of the pole, lights, support baskets or arms, and any other appurtenances. Temporary loads applied during maintenance should also be considered (Article 3.5, Specifications 2009).

Live Load

Consists of a single load of 500 lb (2200 N) distributed over 2 ft (0.6 m) transversely to the member used for design of members for walkways and platforms. This load represents the weight of a person and equipment during servicing of the structure and is only applied to members of walkways and service platforms (Article 3.6, Specifications 2009).

Ice Load

Consists of a load of 3.0 psf (145 Pa) applied around the surfaces of the pole and luminaires. The map in **Figure 4** above from the Specifications shows where ice loading should be considered in the United States (Figure 3-1, Specifications, 2009). The loading is based on a 0.60 in (15 mm) radial thickness of ice at a unit weight of 60 pcf (960 kg/m3) applied uniformly over the exposed surface (Article 3.7 and Commentary C3.7, Specifications, 2009).

Wind Load

The pressure of the wind acting horizontally on the pole, lights, support baskets or arms, and any other appurtenances corresponding to the appropriate 50-yr mean recurrence interval basic wind speed and the appropriate wind importance factor, Ir (Article 3.8, Specifications, 2009). Wind load is defined in terms of 3-s gust wind speeds instead of the fastest-mile wind speed utilized in the previous version of the Specifications (2001). A 3-s gust wind speed is defined as the average wind speed measured over an interval of three (3) seconds. The country map of 3-s wind speeds is included on the next page in **Figure 5** with permission of ASCE.

Figure 4 Top Right

Ice Load Map (Figure 3-1 from Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 2009, by the American Association of State Highway and Transportation Officials, Washington D.C. Used by permission.)

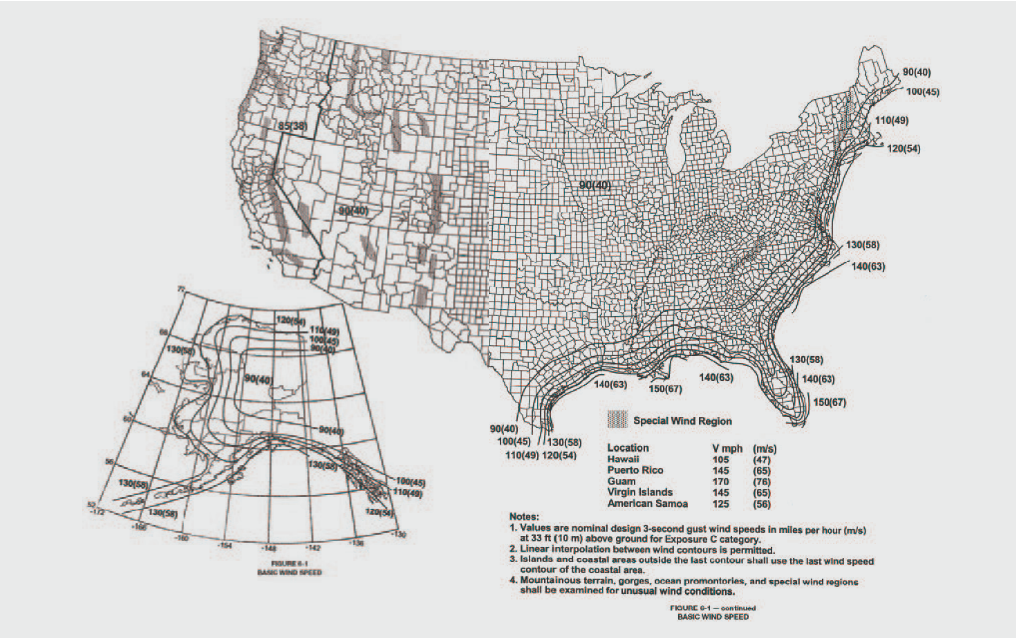


Figure 5

Basic Wind Speed Map in mph (m/s) - Figure 3-2 from ASCE 7-05 (with permission from ASCE)

Wind Pressure

The design wind pressure calculation is based on fundamental fluid-flow theory and formulations presented in ASCE 7-95, *Minimum Design Loads for Buildings and Other Structures* (1995), and is computed as follows:

$P_{NW} = 5.2\ C_d\ I_F\ (psf)$	**Equation II-5, Specifications, 2009**
$P_{NW} = 250\ C_d\ I_F\ (Pa)$	**Equation II-5, Specifications, 2009**

The height and exposure factor, K_z, is related to height and is determined from Table 3-5 in the Specifications or calculated by equation C3-1. The gust effect factor, *G*, is a minimum of 1.14. Previous versions of the Specifications addressed wind sensitivity by incorporating an increased gust coefficient of 1.3. This gust coefficient corresponded to a gust effect factor of 1.69 = (1.3)(1.3) for use with fastest-mile wind speeds. The fastest-mile gust coefficient of 1.3 is converted to a 3-s gust coefficient by multiplying the gust coefficient of 1.3 by the ratio of the fastest-mile wind speed to the 3-s gust wind speed. The corresponding gust effect factor, *G*, is then found by squaring the 3-s gust coefficient (Commentary C3.8.5, Specifications 2009). The basic wind speed, *V*, is determined from the ASCE wind map in **Figure 5** above (Figures 3.2 a and b, ASCE 7-05) associated with a height of 33 ft (10 m) for open terrain and associated with a 50-yr mean recurrence interval (annual probability of two percent that the wind speeds will be met or exceeded). The wind importance factor, I_r is determined from Table 3-2 in the Specifications and is selected based on the specified design life of the structure. For a 50-yr recurrence interval, I_r is 1.00.

Continue to next page.

Wind Pressure (Cont.)

The wind drag coefficient, Cd, is determined from Table 3-6 of the Specifications. For a pole structure, Cd will depend on the number of flats (shape) of the pole, the ratio of corner radius to radius of inscribed circle, and the wind speed. For attachments such as luminaires, the drag coefficient is typically provided by the light fixture supplier in terms of effective projected area (EPA), which is the drag coefficient multiplied by the projected area. If the EPA is provided, the drag coefficient is taken as 1.0.

Group loading combinations are addressed specifically in Article 3.4 of the Specifications. Each individual load is to be combined into group load combinations as shown in Table 1 below (Table 3-1, Specifications, 2009). Each part of the structure shall be designed for the combination producing the maximum load effect using allowable stresses increased as indicated for the group load.

Percentages of allowable stress are applicable for the allowable stress design method. No load reduction factors shall be applied in conjunction with these increased allowable stresses.

A. *W* shall be computed based on the wind pressure. A minimum value of 1200 Pa (25 psf) shall be used for *W* in Group III.

B. See Section 11 for fatigue loads and stress range limits.

C. See Article 3.6 regarding application of live load.

Group Load	Load Combination	% of Allowable Stress [A]
I	DL	100
II	DL+W	133
III	DL+ICE+½(W) [B]	133
IV	Fatigue	[C]

Table I

Group Load Combinations (Table 3-1 from Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 2009, by the American Association of State Highway and Transportation Officials, Washington D.C. Used by permission.)

$$\frac{f_a}{0.6F_y} + \frac{f_b}{C_A F_b} + (f_v / F_v)^2 \leq 1.0C_A F_b$$

Equation 5-16, Specifications, 2009

Section 4 of the Specifications describes methods of analysis for the structural design of poles. Although the engineering community has been trending to Load and Resistance Factor Design, (LRFD, AISC 1994), the Specifications follow an allowable stress design, (ASD, AISC 1989) approach for design. ASD is based on elastic stress calculations where the strength of the member is divided by a factor of safety. The allowable stress value is compared to actual calculated stresses in the member and structure. For pole structures, Article 4.8 of the Specifications require secondorder effects be accounted for in the design. Secondary bending moments caused by the axial load should be accounted for by an approximate simplified method in Article 4.8.1 or a more exact method where the member is analyzed considering the actual deflected shape of the structure in Article 4.8.2 (Specifications, 2009).

Section 5 specifies design provisions for steel poles. Article 5.5 addresses local buckling and the classification of steel sections as compact, non-compact, or slender element sections. For a section to qualify as compact or non-compact, the widththickness ratios of compression elements must not exceed the applicable corresponding values given in Table 5-1 of the Specifications. For design, Table 5-3 provides the allowable bending stress, F_b, for tubular members. Pole structures subjected to axial compression, bending moment, shear, and torsion should satisfy the following requirement from Article 5.12.1.

Equation 5-16 may be increased by 1/3 for load combination Groups II and III involving wind. C_A is calculated in accordance with Article 4.8.1 to estimate secondorder effects or is 1.0 if the more exact method of 4.8.2 is utilized (Specifications, 2009).

Per Article 5.14 of the Specifications, the minimum thickness of material used forb main supporting members shall be 3/16 in (4.76 mm). Telescoping slip joint field splices should be detailed such that the minimum length shall be 1.5 times the inside diameter of the female pole section. All welding design should be per the latest edition of the American Welding Society Structural Welding Code D1.1 (2010). Article 5.15 states that full-penetration groove welds shall be used for pole and arms sections joined by circumferential welds. Longitudinal seam welds for pole and arms sections shall have 60% minimum penetration expect within 6 in (150 mm) of circumferential welds where the weld shall be full-penetration and in slip joint areas where it shall be full-penetration the length of the slip joint plus 6 in (150 mm).

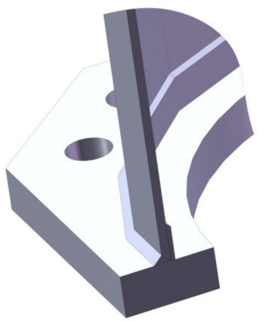


Figure 6

Pole base complete penetration groove weld joint (CJP)

Figure 7

Pole base socket weld joint

Plate Thickness

The Specifications state in Article 5.14.2 that the pole base plate thickness should be considered in the design of the structure and that the thickness of unstiffened base plates should be equal to or greater than the nominal diameter of the connection bolt. In steel poles, the flexibility of this joint can greatly contribute to the reduced fatigue strength of this connection. While the Specifications do not provide detailed guidance on base plate design techniques, it is recommended that careful consideration be given to the design of the pole base plate connection so that premature failure of this joint due to design, fatigue, or manufacturing issues does not occur. The pole shaft to base plate weld connection should be a full-penetration groove weld (CJP) or socket-type joint with two fillet welds per Article 5.15.3 as shown below in cutaway **Figures 6 and 7**. The CJP connection base plate is butted against the pole shaft and consists of a groove weld with 100% complete weld penetration and reinforcing fillet weld. The socket connection base plate sleeves over the pole wall and is welded with double fillet welds. For base plate materials per Article 5.4, all steels greater than 1/2 in (13 mm) in thickness that are main carrying load members shall meet the current Charpy V-Notch impact requirements in AASHTO *Standard Specifications for Highway Bridges*, 17th Edition (2002).

Anchor Bolts

Article 5.17 addresses anchor bolt connections and provides minimum requirements for the design of steel anchor bolts used to transmit loads in the critical connection from the pole to the foundation. The Specifications require castin- place anchor bolts be used conforming to the requirements of ASTM F1554 (2007) or hooked smooth bars with a yield strength not exceeding 55 ksi (380 MPa). Headed anchor bolts are preferred to reduce the possibility of pull-out. To reduce susceptibility to corrosion and fatigue, for a design life of 50 years, a minimum of six (6) anchor bolts should be considered at the base plate connection (C5.17.3, Specifications, 2009). This is not widely practiced by designers and pole manufacturers in the industry today. For a single anchor bolt subjected to combined tension and shear, the following equation shall be satisfied:

$$(f_v / F_v)^2 + (f_t / F_T)^2 \leq 1.0$$ **Equation 5-24, Specifications, 2009**

$$(f_v / F_v)^2 + (f_c / F_C)^2 \leq 1.0$$ **Equation 5-25, Specifications, 2009**

Equations 5-24 and 5-25 may be increased by 1/3 for load combination Groups II and III. If the clear distance between the bottom of the bottom leveling nut and the top of concrete is less than the nominal anchor bolt diameter, bending in the anchor bolt from shear forces or torsion may be ignored. If the clear distance exceeds one bolt diameter, bending of the anchor shall be considered per Article 5.17.4.3 (Specifications, 2009).

Serviceability Requirements

Horizontal deflection limits for poles are defined in Section 10, specifically Article 10.4.2 in the Specifications. According to the Commentary in C10.4, deflection limits serve two purposes: 1) Provide for an aesthetically pleasing structure under dead load; and 2) Provide adequate structural stiffness that will result in acceptable serviceability performance (Specifications, 2009). Limits for Group I load combinations (dead load only) include a deflection limit of 2.5% of the structure height and 0.35 in/ft (30 mm/m) slope. For pole structures under Group II load combination (dead load and wind load), deflection should be limited to 15% of the structure height. The 15% deflection limitation for Group II load combination is not a serviceability requirement, but it constitutes a safeguard against the design of highly flexible structures. While serviceability may be more critical for certain types of traffic structures than sports lighting poles, the owner and designer should understand the ramifications of overly flexible structures and the resulting fatigue consequences.

Fatigue

OVERVIEW

Fatigue is damage resulting in fracture caused by stress fluctuations due to cyclic loading. The fatigue and premature failure of structures has cost lives and industry billions of dollars. Specifically for pole structures, fatigue can be very detrimental to the long term performance of the structure and can risk public safety. Sports lighting poles are exposed to several wind phenomena that can produce cyclic loads. The resulting vibrations can be significant and can shorten the lifespan of the pole. A pole structure is especially susceptible to vortex shedding and natural wind gusts; the amplitude of vibration and resulting stress ranges are increased by the low levels of stiffness and damping possessed by a flexible pole structure (C11.7, Specifications, 2009). As tall, slender, cantilevered structures with no base connection redundancy, this phenomenon should be acknowledged by owners and considered carefully by the sports lighting pole designer.

HIGH-LEVEL HIGH-MAST LIGHTING STRUCTURES

Section 11 of the Specifications requires fatigue design for high-level, high-mast lighting structures. To avoid largeamplitude vibrations and to preclude the development of fatigue cracks at the base connection of a pole structure, sports lighting poles should be designed to resist limit state equivalent static wind loads acting separately per Article 11.7. These loads should be used to calculate nominal stress ranges near the fatigue-sensitive base connection of the pole and deflections for service limits described in Article 11.8. Stresses due to these loads on all components of the pole should be limited to satisfy the requirements of their respective detail categories within the constant-amplitude fatigue limits (CAFL) provided in Article 11.9 (Specifications, 2009). The basis of the pole fatigue design provisions in the Specifications is the National Cooperative Highway Research Program Project Report 412 (1998).

IMPORTANCE FACTOR

Fatigue importance factors are introduced in Article 11.6 of the Specifications to adjust the level of structural reliability of a pole structure. The fatigue importance factor, I_F, accounts for risk of hazard and should be applied to the limit state wind load effects specified in Article 11.7. The Commentary of the Specifications in C11.6 recommends this value be determined by the owner. In the case of sports lighting structures, the owner should be generally aware of these provisions and determine this in conjunction with the advice of the pole designer. The Commentary in C11.6 also states that high-mast structures (without mitigation devices) in excess of 100 ft may be classified as Fatigue Category 1. Typically, sports lighting poles present a high hazard in the event of failure and as a result should be designed to resist wind loading and vibration phenomena. Based on Section 11.0 and Table 11-1 in the Specifications, the fatigue importance factor, I_F, is 1.0 for cantilevered lighting pole structures for both vortex shedding and natural wind gusts (Specifications, 2009). The importance categories and fatigue importance factors in the Specifications are from NCHRP Reports 469 (2002) and 494 (2003).

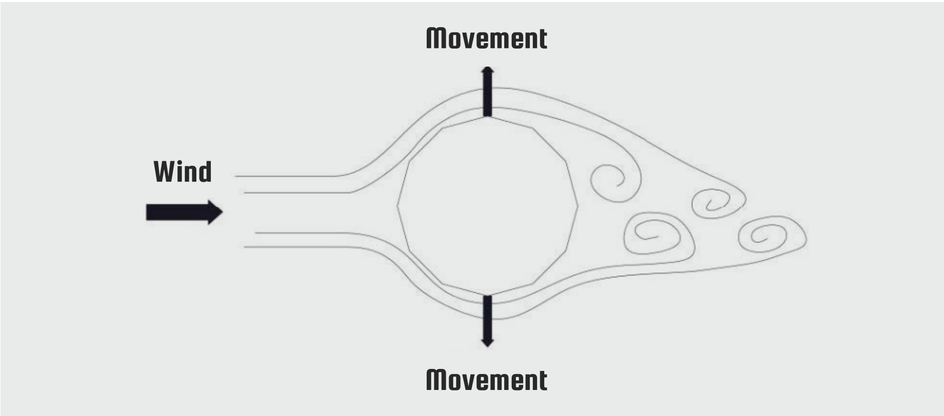


Figure 8

Vortex shedding phenomenon

NCHRP Report 469 (2002) shows that poles with tapers exceeding 0.14 in/ft (0.0117 m/m) can experience vortex shedding. A taper of 0.14 in/ft (0.0117 m/m) is very common for a sports lighting pole. According to the Commentary in C11.7.2 of the Specifications, tapered poles can experience vortex shedding in second or third mode vibrations which can lead to fatigue problems. Per Article 11.7.2, high-level, highmast lighting structures should be designed to resist vortex shedding-induced loads for critical wind velocities less than approximately 45 mph (20 m/s). The critical wind velocity, V_c, in mph at which vortex shedding lock-in can occur may be calculated by the Strouhal relationship as follows for circular sections:

For Single Sided Sections

$$(f_v / F_v)^2 + (f_t / F_T)^2 \leq 1.0$$

Equation 11-2, Specifications, 2009

For Multi-Sided Sections

$$(f_v / F_v)^2 + (f_t / F_T)^2 \leq 1.0$$

Equation 11-3, Specifications, 2009

For multi-sided sections where f_n is the natural frequency of the structure in cycles per second; d and b are the diameter and flat-to-flat width of the pole shaft for circular or multi-sided sections (ft, m), respectively; and S_n is the Strouhal number (0.18 for circular sections or 0.15 for polygonal sections). For tapered poles, d and b are the average diameter and width (Specifications, 2009).

Article 11.7.2 designates the equivalent static pressure range to be used for the design of vortex shedding-induced loads for poles as follows:

Equation 11-4, Specifications, 2009

$$P_{VS} = (0.00256\, V_C^2 C_d I_F) / 2\beta \quad (psf)$$

$$P_{VS} = (0.613\, V_C^2 C_d I_F) / 2\beta \quad (Pa)$$

Where V_c is mph (m/s); C_d is the drag coefficient as specified in Section 3 which is based on the critical wind velocity V_c; and β is the damping ratio, which may be estimated as 0.005. The equivalent static pressure, P_{vs}, is to be applied transversely (horizontal direction) to pole structures (Specifications, 2009).

Natural Wind Gust

Natural wind gusts are basic wind phenomena that are variable in velocity and direction and can induce vibrations in pole structures. This is also a fairly common phenomenon with pole structures. Per Article 11.7.3, steel poles should be designed to resist an equivalent static natural wind gust pressure range of:

Equation 11-5, Specifications, 2009

$P_{NW} = (5.2\ C_d I_F)$ (psf) $\quad$ $P_{NW} = (250\ C_d I_F)$ (Pa)

where C_d is the appropriate drag coefficient based on the yearly mean wind velocity of 11.2 mph (5 m/s) specified in Section 3 of the Specifications. The natural wind gust pressure range should be applied in the horizontal direction to all exposed areas and should consider the application of gusts for any direction of wind. The Specifications allow the owner to modify the natural wind gust pressure if there are more detailed wind records available.

Fatigue Resistance

Constant-amplitude fatigue limits (CAFL) are the nominal stress ranges below which a particular fatigue detail can withstand an infinite number of repetitions without fatigue failure. Typical fatigue sensitive connections in steel poles are the base plate to shaft weld connection and the slip joint previously shown in **Figures 6, 7, and 2**, respectively. Fatigue details and corresponding stress categories are specified in Table 11-2 and illustrated in Figure 11-1 of the Specifications for use by the designer. Allowable CAFL's are specified in the Specifications Article 11.9, Table 11-3 (Specifications, 2009). For steel pole base connections **(Figures 6 and 7)**, the Stress Category and corresponding CAFL is as follows:

CJP groove weld → Stress Category E' → CAFL = 2.6 ksi (18 MPa)

Socket weld → Stress Category E' → CAFL = 2.6 ksi (18 MPa)

Table 11-2 of the Specifications, footnote j, states that fillet welds for socket connections shall be unequal leg welds, with the long leg of the fillet weld along the column. The termination of the longer weld leg should contact the pole shaft's surface at approximately a 30° angle. For pole slip joints **(Figure 2)**, where the telescoping overlap is greater than or equal to 1.5 diameters, the Stress Category and corresponding CAFL is as follows:

Slip Joint → Stress Category B → CAFL = 16 ksi (110 MPa)

Note that the wind loads from Article 11.7 should be utilized to compute the fatigue stress range.

Conclusion

Active involvement of the owner, communication with the pole designer, and professional responsibility is crucial to the accurate structural design of steel poles for sports lighting applications. Standardizing the design process will improve the safety of these structures and reduce confusion during the procurement process. The owner should ensure the following:

1. Hire an experienced professional engineer (P.E.) to develop the technical specification for the procurement process for the pole structures

2. Require the poles be designed to AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals*, 5th Edition, 2009 (Specifications, 2009)

3. If the poles are being procured packaged with the lights and electrical components, know who is fabricating the pole structures

4. Require a P.E. certification of pole and foundation designs provided prior to shipping and installation of thestructures

5. Have a third party review the pole and foundation designs

6. Maintain all project records including specifications, site specific soils information, P.E. documentation, and the pole fabricator's drawings

The AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals* (Specifications, 2009) discussed in this paper is specifically for pole structures and specifies factor of safety, addresses fatigue issues, addresses wind induced vibration issues, and other design requirements. With longevity and proven performance in the traffic industry, the owner who specifies this document as a design standard will have sports lighting pole structures that will perform satisfactorily and safely for many years.

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