2017 AASHTO Training Manual

December 2017

MSC4014 12/17

**valmont** **V**
STRUCTURES

Table of Contents

valmont V
STRUCTURES

I. 2017 AASHTO General Overview

General Overview

1. **Scope**: *The Specifications have been revised with significant additions made to their content and scope.* Objectives of this revision include:
   - Update technical content
   - Provide a commentary
   - Address fiberglass-reinforced plastic composites
   - Adopt a new, user-friendly format similar to the LRFD Bridge Design Specification
   - Adopt SI units

2. **Applicable Structures**: The 2001 AASHTO Standard Specifications are applicable to the structural design of supports for highway signs, luminaries, and traffic signals. This has not changed since the previous edition.

3. **Design Considerations:** The 1994 *(the previous) Support Specification was based on studies and reports that were published in the late 1960s. Since that time, significant research work has been performed that could highly impact the Supports Specifications.* This includes:
   - New wind loading provisions
        New Wind Pressure Formula
        New Basic Wind Speeds
        New Wind Importance Factor
        New Gust Effect Factor
        New Drag Coefficients
   - Alternate design philosophies for wind loading
   - New material and design parameters
        Fiber-Reinforced Composites Design
        Wood Design
        Serviceability Requirements
        Fatigue Design

4. **Impact:** *The new information should also be evaluated carefully to determine the effects of the revisions on the various types of support structures and the consequences of adopting such new material.* Valmont has found that some of these changes can have some very profound affects on the structure size and/or the connection details.
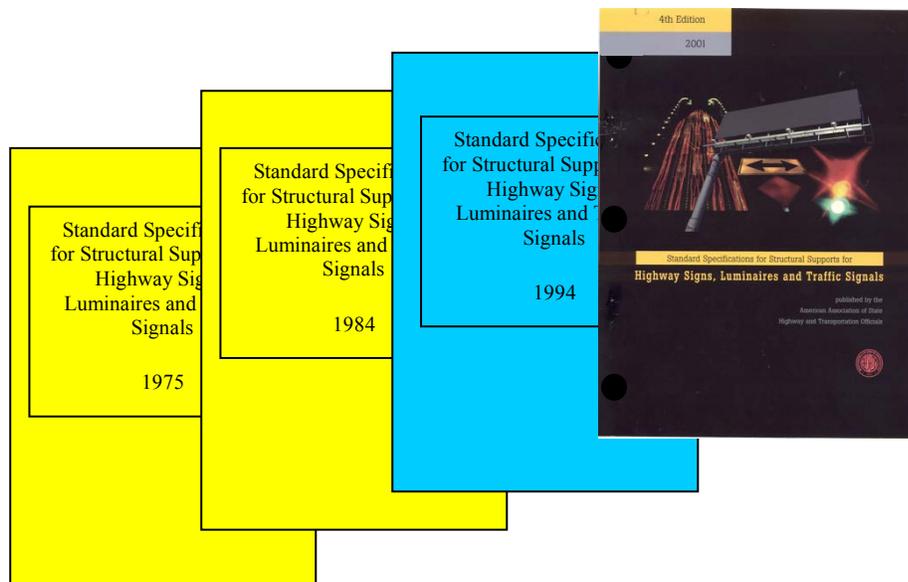

AASHTO Specifications from 1975 to 2001

II. Wind and Allowable Stress Design

Three Second Gust vs. Fastest Mile Wind

1. **What are the new requirements?** One of the major changes in the new AASHTO specification wind pressure equation is the use of a 3-second gust wind speed in place of a fastest-mile wind speed used in the previous specification. This change was made due to the fact that the US National Weather Service no longer collects fastest-mile wind speed data. Instead, the National Weather Service currently records wind speed data based on peak 3-second gusts. The 3-second wind gust map found in AASHTO is based on the wind map found in ASCE 7-95.

2. **How are they different?** The difference between the two methods of measuring wind speed is the following:

   a)   Fastest-mile wind speed is the average speed during the time required for the passage over an anemometer of a volume of air with a horizontal length of one mile.

   b)   A 3-second gust wind speed is the average speed of the wind during a 3-second interval.

The duration of the fastest-mile gust wind is the time it takes the volume of air to travel the one-mile (see figure). Therefore, for a fastest-mile wind, measured at 80 mph, the duration would be approximately 45 seconds. The 3-second gust wind speed, on the other hand, is measured over a much shorter time interval (see figure). This results in the fastest-mile wind speed being more of an average wind speed while the 3-second gust wind is more of a measure of the gusted wind speed.
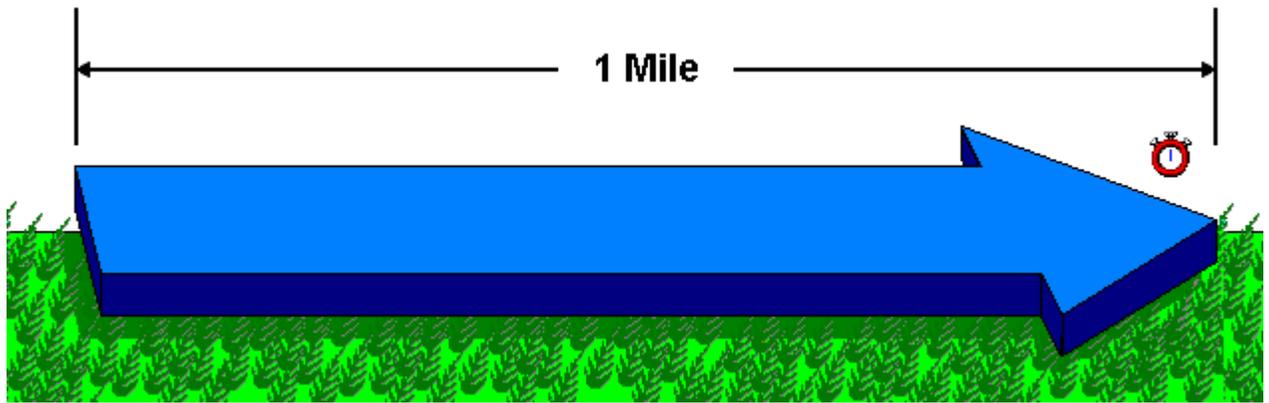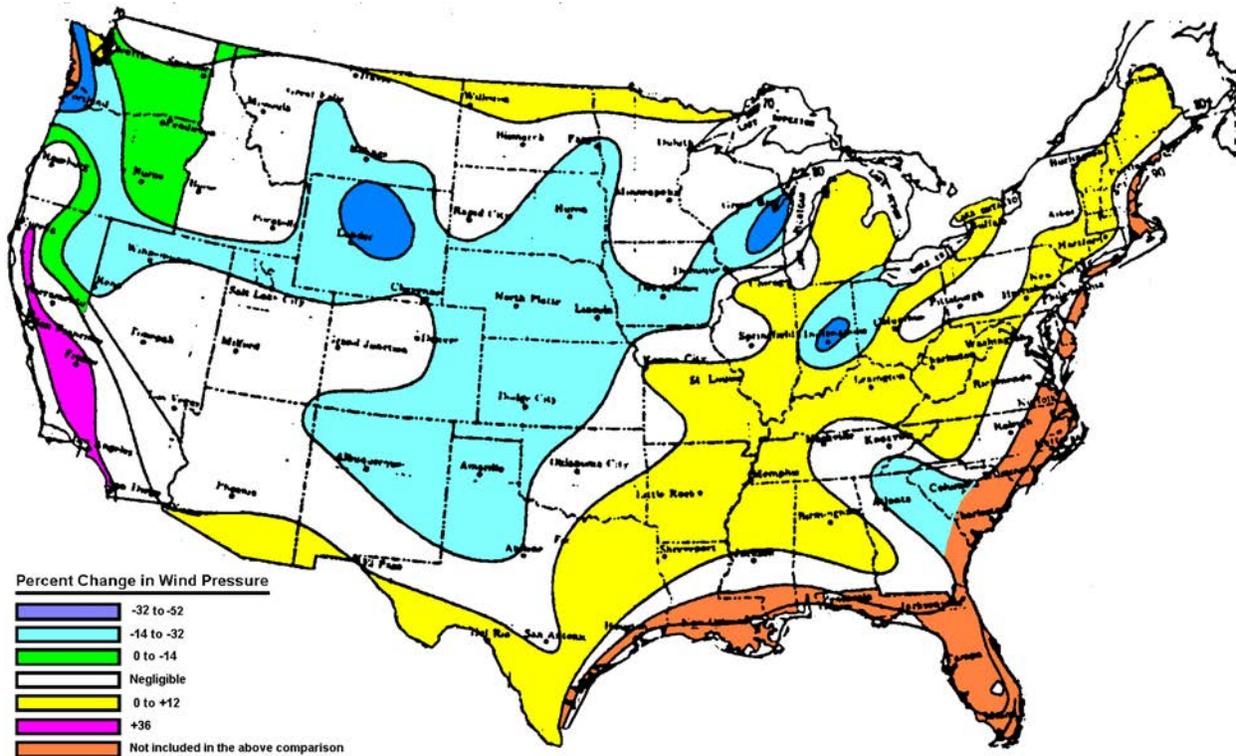

Fastest Mile Gust Wind
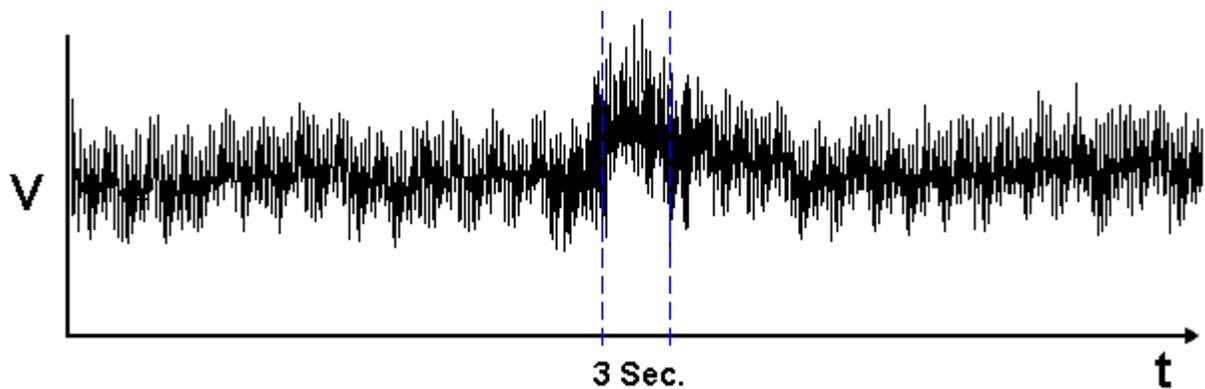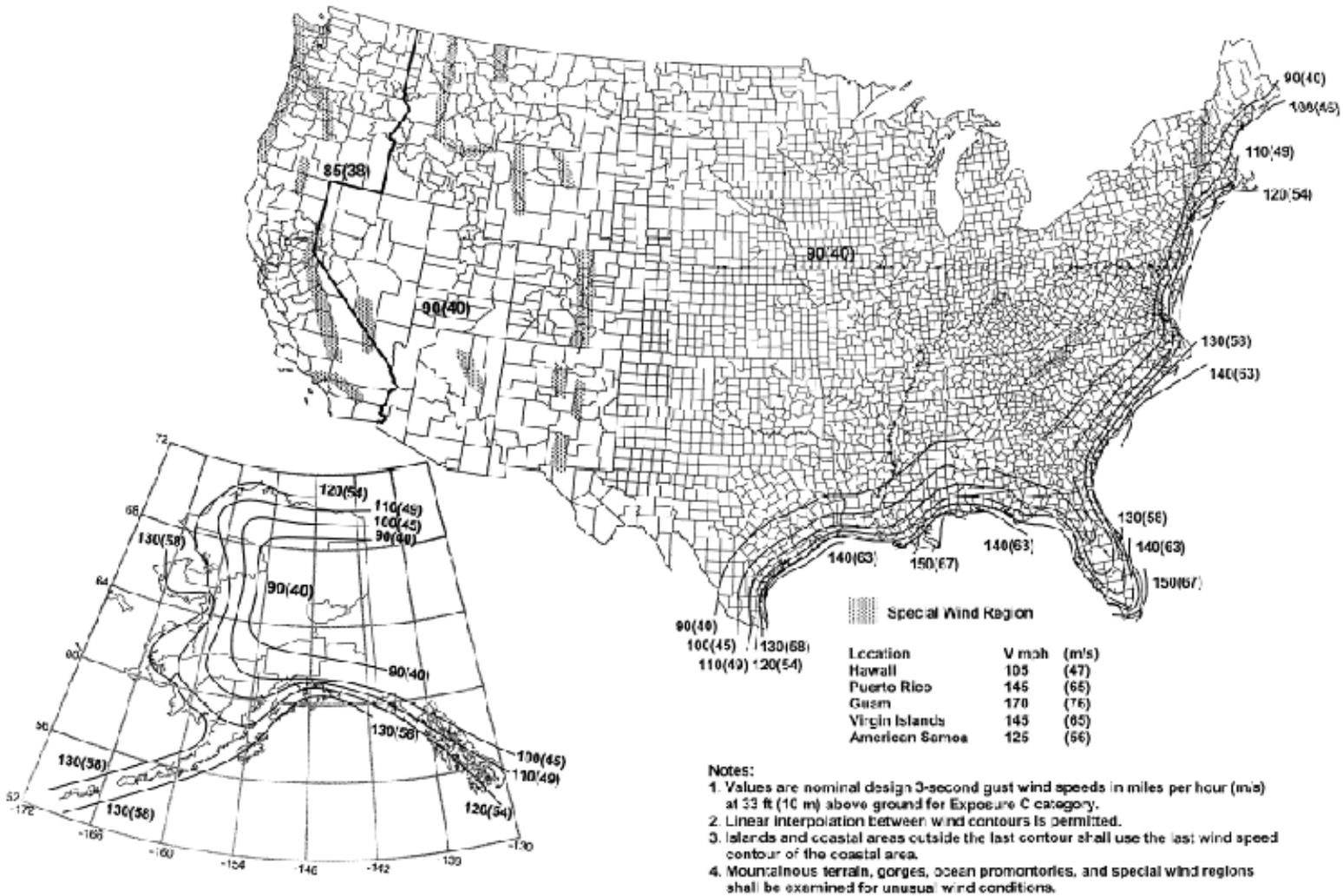

3-Second Gust Wind

New Wind Map

1. **What are the new requirements?** There are three individual wind speed maps in the previous AASHTO specification. The three maps represent the fastest-mile wind speeds in the US for mean recurrence intervals of 10, 25 and 50 years. These maps were considered as authoritative into the early 1980's, but have not been used in ANSI/ASCE wind load standards since 1982. The new AASHTO specification replaces the three fastest-mile wind maps with a single 3-second gust wind speed map with a mean recurrence interval of 50 years. The new 3-second gust wind map was taken from ASCE 7-95 and is based on a statistical analysis of peak gust data collected at 485 weather stations and mathematical predictions of hurricane wind speeds in coastal areas.

2. **How are they different?** The differences in the new wind map compared to the previous are the following.
    a) The new map is based on a 3-second gust wind instead of a fastest-mile wind and some select locations, as the previous wind maps were.
    b) Under the new wind map, the entire US, with the exception of the coasts, has a wind speed of 90 mph.
    c) Instead of using separate maps for different recurrence intervals, importance facts are used with the new map to make adjustments in wind speeds due to differing recurrence intervals.

The major result of the revised wind speed map is that the design wind speed in most of the country has changed. The new 90 mph 3-second gust wind speed which will apply to most of the US is equivalent to a 73.8 mph fastest-mile wind speed (i.e., 90 X 0.82 = 73.8). The effect of this change will be that in large areas of the country the wind pressure will increase by 12% or more and other areas it will decrease by approximately 15% to 40% (see figure below, Note: this figure should not be used for design purposes).

Percent Change in Wind Pressure, 1994 vs. 2001 AASHTO

New Wind Map

Location — V mph (m/s):
Hawaii 105 (47)
Puerto Rico 145 (65)
Guam 170 (76)
Virgin Islands 145 (65)
American Samoa 125 (56)

Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10 m) above ground for Exposure C category.
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

Static Wind Pressure Formula

1. **What are the new requirements?** The Static Wind Pressure Formula from the new AASHTO specification is fundamentally the same as that found in the previous specification (see below).

    $P = 0.00256 \ (1.3V)^2 \ C_d C_h$ (previous AASHTO specification)

    $P = 0.00256 \ K_Z \ G \ V^2 \ I_r \ C_d$ (equation 3-1 of the 2001 AASHTO specification)
   Where: P = Static Wind Pressure (psf)
               V = Design Wind Velocity (mph)
               K_2 = Height and Exposure Factor
               G = Gust Effect Factor
               I_r = Wind Importance Factor
               C_d = Drag Coefficient
               C_h = Height Coefficient

    Although the format of the new equation is slightly different, both equations are based on the same fundamental fluid-flow theory of pressure acting on an immersed body. Both equations also account for the gusting of the wind, dynamic interaction, height of above ground, recurrence interval, etc.

2. **How are they different?**
    a) Introduction of an importance factor to account for different reoccurrence intervals.
    b) The gust response factor has been decreased to account for the change from fastest-mile to 3-second gust. The gust response factor is now applied to the pressure instead of the wind velocity.
    c) The height and exposure factor has been revised. The minimum value has increased and it is now a curve instead of a step function (see figure below).

3. **Impact:** Although the wind load equation is fundamentally unchanged, the overall impact on the design of lighting and traffic structures could be quite great due to changes in criteria that affect the individual variables that make up the equation. One significant example of these changes is the revision of the wind map, which has dramatically changed the design wind speed in much of the country. A second example is the change to the height and exposure factor, which could result in higher pressures for structures under 20' and possibly lower pressures for taller structures. A third example is the change from using a 50-year reoccurrence interval to using a 25-year reoccurrence interval for traffic signal and smaller lighting structures.
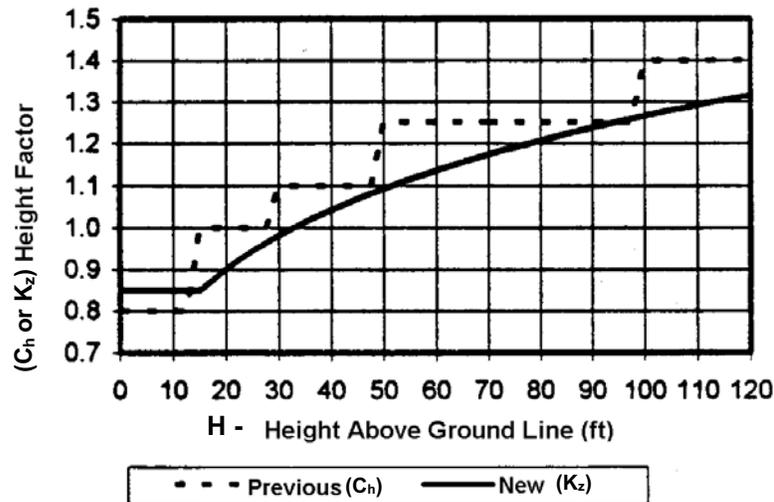
Height Factor Comparison

Importance Factors and Recurrence Intervals

1. **What are the new requirements?** There are three individual fastest-mile wind speed maps in the previous AASHTO specification, each representing a different recurrence interval. The new AASHTO specification replaced the three wind maps with a one 3-second gust wind speed map for a mean recurrence interval of 50-years. In order to account for recurrence intervals other than 50 years, the new specification uses importance factors to make adjustments in wind speeds due to differing recurrence intervals. The importance factors are based on studies that showed ratios of the wind speeds given for a specific recurrence interval and that of a 50-year recurrence interval are relatively independent of the site. Therefore, the following relationship was developed $V_r = V_{50} (I_r)^{1/2}$ thus allowing for the use of a single wind speed map.

<table>
<tr><td colspan="4" align="center">Table 3-2. Wind Importance Factors, I_r</td></tr>
<tr><td>**Recurrence Interval Years**</td><td>**V = 38 - 45 m/s (85-100 mph)**</td><td>**V > 45 m/s (100 mph) (Hurricane)**</td><td>**Alaska**</td></tr>
<tr><td>100</td><td>1.15</td><td>1.23</td><td>1.13</td></tr>
<tr><td>50</td><td>1.00</td><td>1.00</td><td>1.00</td></tr>
<tr><td>25</td><td>0.87</td><td>0.80</td><td>0.89</td></tr>
<tr><td>10</td><td>0.71</td><td>0.54</td><td>0.76</td></tr>
</table>

2. **How are they different?** The importance factors make it easier to change from one recurrence interval to another. No longer will three different wind maps be required. Every kind of structure, no matter what its required recurrence interval, will use the same single wind speed map.

3. **Impact:** The previous AASHTO specification requires lighting or traffic structures (50' or less) be designed to a 50-year recurrence interval, unless otherwise specified by the owner. The new AASHTO specification also requires a lighting or traffic structure (50' or less) be designed to a 50-year recurrence interval, unless otherwise specified by the owner.

<table>
<tr><td colspan="2" align="center">Table 3-3. Recommended Minimum Design Life</td></tr>
<tr><td>**Design Life**</td><td>**Structure Type**</td></tr>
<tr><td>50-year</td><td>• Luminaire support structures
• Overhead sign structures
• Traffic signal structures</td></tr>
<tr><td>10-year</td><td>• Roadside sign structures</td></tr>
</table>

Gust Effect Factor

1. **What are the new requirements?** The gust effect factor is used to adjust the basic effective wind pressure to account for the gust of the wind and for any dynamic amplification. The new wind map in AASHTO is based on a 3-second gust instead of fastest-mile as the previous specification was. Due to the shorter time interval of the 3-second wind, the wind speeds in the new map are for the most part higher than those found in the previous AASHTO wind map. The wind speeds in the new map therefore already account for much of the effects produced by the gusting of the wind (i.e., higher effective wind pressure).

2. **How are they different?:** To account for the higher wind pressure produced by the 3-second gust wind maps, the gust effect factor in the new specification was lowered from $(1.3)^2$ (i.e., 1.69) to 1.14. The following equation was used to convert the fastest-mile gust coefficient of 1.3 to a 3-second gust coefficient:

$$G = [\, 1.3 \, (\, V_{fm} / V_{3\text{-}sec} \,) \,]^2 = [\, 1.3 \, (\, 0.82 \,) \,]^2 = 1.14$$

Where:
G	= Gust Effect Factor
$V_{3\text{-}sec}$	= 3-second gust wind speed
1.3	= Fastest-mile gust coefficient
V_{fm}	= Fastest-mile wind speed
0.82	= Ratio of the fastest-mile to the 3-gust wind speed per AASI/ASCE 7-95

The large reduction in gust factors is based on Fig C6-1 of ASCE 7-95 (see Fig below). This reduction in gust effect factor negates the impact of the higher 3-second gust wind speed, which resulted from moving from a fastest-mile to a 3-second gust wind speed.
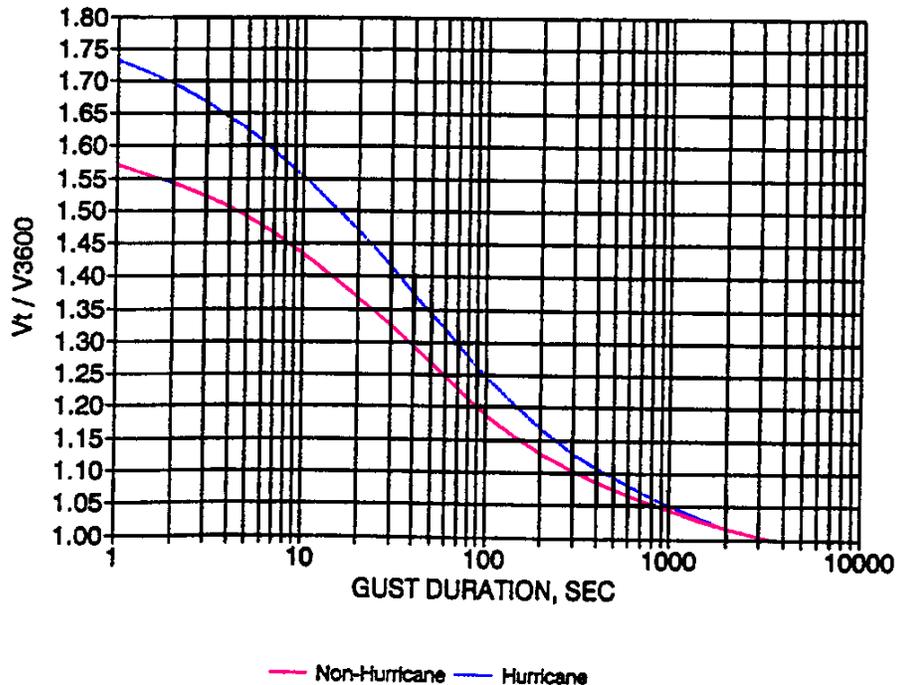
ASCE 7-95, Figure C6-1

Miscellaneous Static Load Changes

1. **Drag Coefficients:** There have been few changes in drag coefficients for members, between the previous specification and the new specification. One noted change is the increased drag coefficients for square members. Other modifications were made to constants in the C_d equations due to criterion change from fastest-mile wind to 3-second gust and also to convert the SI equations from kilometers/hour to meters/second.

2. **Safety Factors:** The allowable stress increase factor for Group II and III loads has been changed from 1.40 to 1.33. The reason for this change was to provide consistency with major codes in the US. This increase in safety factors (see table below) will not necessarily mean additional conservatism in designs due to the new wind load criteria, which will result in lower wind pressures in most areas.

3. **Allowable Axial Compressive Stress:** The allowable stress equation for inelastic buckling of axially loaded members has been updated to comply with the current equations in AISC Manual of Steel Construction- Allowable Stress Design that employs a variable safety factor instead of a constant factor of 1.92. This change results in the allowable compressive stress increasing, in the new specification, for KL/r values less than 80.

4. **Allowable Bending Stress:** The 2001 AASHTO Specification utilizes a new set of bending allowable stress equations for multi-sided tubular sections. New equations and compact limits are based on more recent research conducted by Electric Power Research Institute (EPRI).

5. **Alternate Wind Loading Criteria:** An alternate method for calculating wind pressures is contained in Appendix C of the 2001 AASHTO Specification. This alternate method is the same method contained in the 1994 edition and earlier editions of these Specifications. The alternate method shall be used to determine design pressures only when specified by the owner.

Ultimate Safety Factors for Round Compact Cross Sections			
	Previous and New	**Previous**	**New**
Material	**Safety factor without increase in allowable stress (Group I Load)**	**Safety factor with 1.4 increase in allowable stress (Group II & III Loads)**	**Safety factor with 1.33 increase in allowable stress (Group II & III Loads)**
Steel	1.925	1.375	1.447
Aluminum	1.95 Min.	1.393	1.466

III. Fatigue Design

General Fatigue

1. **Scope:** Section 11, of the 2001 AASHTO Specification, focuses on the design of critical structures for the effects of fatigue damage due to cyclic loads. The intent of Section 11 is to provide a methodology for the design of structures to prevent fatigue damage of the extent that it will affect the operability of the structure.

2. **Applicable Structures:** Section 11 applies only to specific types of traffic and lighting structures which are considered as critical and that are also susceptible to fatigue damage. Structures which fall under the scope of this Section include the following:
    a) Overhead Cantilevered Sign Structures
    b) Overhead Cantilevered Traffic Signal Structures
   c) High-level Lighting Structures

    The provisions of Section 11 are not applicable for the design of span wire poles. Common lighting poles and roadside sign structures were also not included within the scope of the Section, since they are smaller structures and it is AASHTO's opinion that they have not normally exhibited problems with fatigue damage. An exception to this would be small square poles. However, the AASHTO Commentary suggests caution be exercised when evaluating these types of structures.

3. **Design Considerations:** The allowable stress for fatigue is quite different than the allowable stress for the static Group I, II, and III load cases. For fatigue, the allowable stress is not a function of the particular yield strength of the material. Instead, the allowable stress is based on the constant-amplitude fatigue limit (CAFL) of the particular material type involved (i.e., either steel or aluminum) and the type of connection being evaluated. CAFL's for steel can be as high as 24 ksi and, depending on the type of connection, can be lower than 1 ksi.

    Load cases, which must be considered when designing structures for fatigue under Section 11, include galloping, vortex shedding, natural wind gust, and truck-induced wind gust.

    Section 11 also requires that the deflection of mast arms under the fatigue load cases be considered as to avoid serviceability problems due to motorists being unable to clearly see the arm attachments or being concerned about passing under the structure.

4. **Impact:** Section 11 of the new AASHTO Specification could have a tremendous impact on the design and size of traffic structures. With our current industry connections, the size of this type of structure could increase by 200% or more. Below is an example of a Valmont sign/signal traffic structure for the state of Colorado. Under the 1994 AASHTO Specification, the pole base diameter was 15" and the mast arm diameter was 12". However, if redesigned to the new specification, the pole diameter would increase to 24" and the mast arm diameter to 23.5". The thickness of both the pole and mast arm would also increase resulting in an increase in structure weight in excess of 150%.

1994 AASHTO Design   2001 AASHTO Design
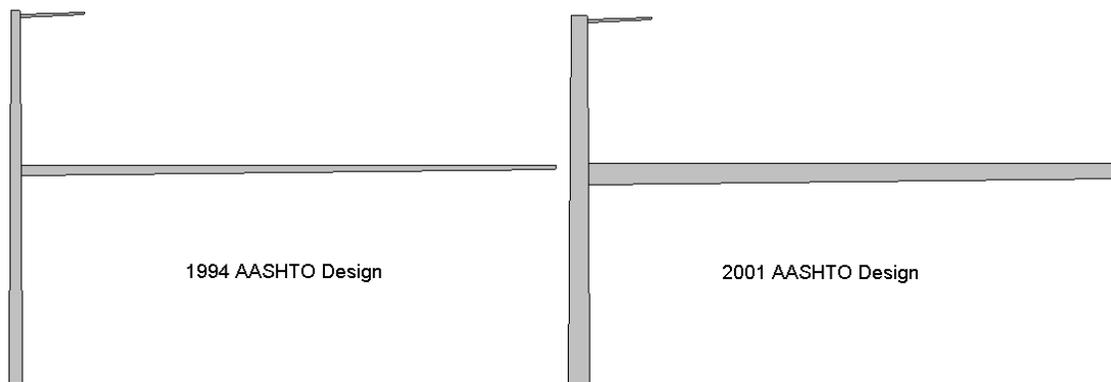
Design Comparison, 1994 vs. 2001 AASHTO Specification

Galloping Load Case

1. **What is it?** Galloping results in large amplitude, resonant oscillations in a plane perpendicular to the direction of the wind. It is generally limited to sign/signal mast arms with attachments to the arm. Galloping is typically not caused by the support members, but rather by the attachments to the horizontal mast arms. Therefore, structures without attachments are not susceptible to galloping induced wind load effects.

2. **Applicable Structures:** Overhead cantilevered sign structures and overhead cantilevered traffic signal structures.

3. **To Calculate Applied Pressure:** Galloping Pressure (P_G) is calculated by the following formula:

    $P_G = 21(I_F)$ (psf)   where I_F is the Fatigue Importance Factor from Table 11-1

4. **To Calculate Applied Force:** Galloping Pressure (P_G) is multiplied by the frontal projected area of each sign panel, traffic signal head including back plate, and all other devices attached to the mast arm. The Galloping Pressure (P_G) is <u>not</u> applied to the projected area of the arm or the pole members.

    $F_G = P_G(A)$ (lbs)   where A is the Frontal Projected Area of the Attached Device

5. **Direction of Applied Force:** The Galloping Force is to be applied in the vertical direction (see figure below).

6. **Load Application:** Loads should be applied separately to each individual arm. The pole should be designed assuming that only one arm is experiencing galloping at any one time.

7. **Owner Specified Options:**
   * As an alternative to designing a structure to withstand galloping fatigue loads, the Owner may choose to install an approved vibration mitigation device.
   * As an alternative to designing a structure to withstand galloping fatigue loads, the Owner may choose to install an approved vibration mitigation device, when a structure displays a galloping problem.
   * The Owner may choose to exclude galloping loads for overhead cantilevered sign support structures with quadri-chord horizontal trusses.

Galloping Load Case
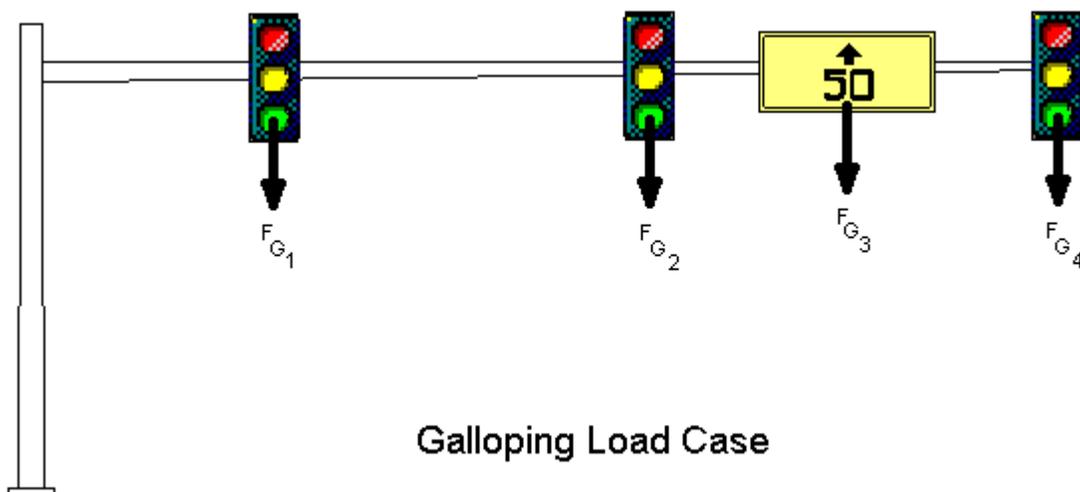
Natural Wind Gust Load Case

1. **What is it?** Natural wind gust stresses result from the inherent variability in the direction and velocity of the wind induced airflow around the structure. Natural wind gusts are the most basic phenomena that may induce cyclic loads in lighting and traffic structures.

2. **Applicable Structures:** Overhead cantilevered sign structures, overhead cantilevered traffic signal structures, and high-level lighting supports.

3. **To Calculate Applied Pressure:** Natural Wind Gust Pressure (P_{NW}) is calculated by the following formula:

   $P_{NW} = 5.2(C_d)(I_F)$ (psf)
   where C_d is the appropriate drag coefficient (using **V = 45 mph** = $(5.2 / 0.00256)^{0.5}$) and I_F is the Fatigue Importance Factor from Table 11-1

4. **To Calculate Applied Force:** Natural Wind Gust Pressure (P_{NW}) is multiplied by the frontal projected area of all structural members (i.e. mast arm and pole), each sign panel, traffic signal head including back plate, and all other devices attached to the mast arm.

   $F_{NW} = P_{NW}(A)$ (lbs)   where A is the Frontal Projected Area of the appropriate object

5. **Direction of Applied Force:** The Natural Wind Gust Force is to be applied in the horizontal direction (see figure below).

6. **Load Application:** Loads shall be applied simultaneously to the entire structure and in the worst possible direction.

7. **Owner Specified Options:**
   - For locations with more detailed wind records, the natural wind gust pressure may be modified at the discretion of the Owner.

Natural Wind Gust Load Case
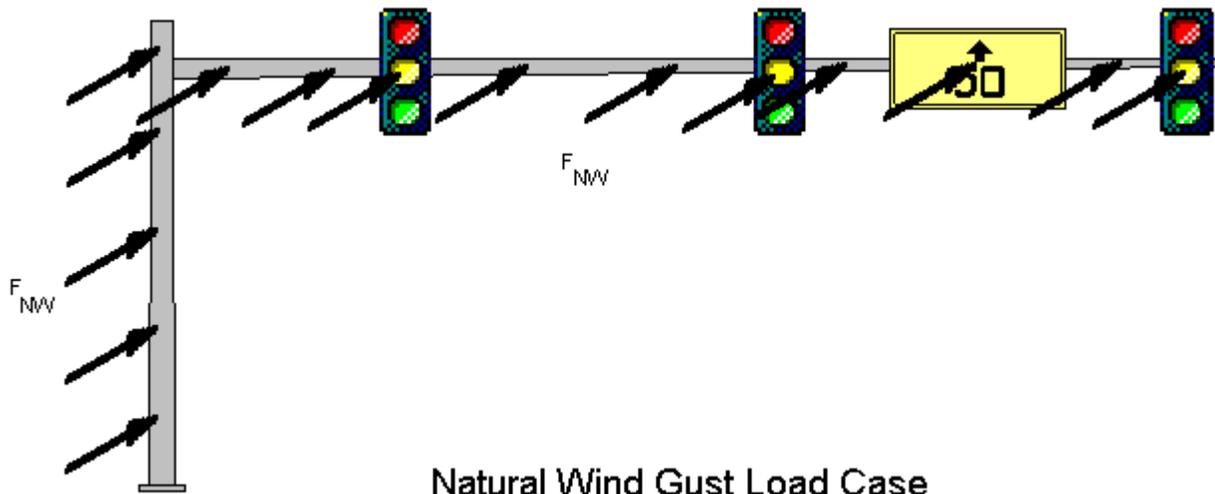
Truck-Induced Gust Load Case

1. **What is it?** Truck-induced gust loads are caused by the passage of trucks under traffic structures. These gusts of wind are caused by moving trucks and create both horizontal and vertical pressure on the structure. The vertical mast arm vibration results in the most critical stresses and therefore only the vertical pressures are evaluated.

2. **Applicable Structures:** Overhead cantilevered sign structures and overhead cantilevered traffic signal structures.

3. **To Calculate Applied Pressure:** Truck Gust Pressure (P_{TG}) is calculated by the following formula:

 P_{TG} = 18.8(C_d)(I_F)   (psf)   (as modified per the approved changes to the AASHTO Specification) where C_d is the appropriate drag coefficient (using **V = 85.7 mph** = $(18.8 / 0.00256)^{0.5}$ for height up to 19.7 ft. and posted speed limits approximately 65 mph) and I_F is the Fatigue Importance Factor from Table 11-1

4. **To Calculate Applied Force:** Truck Gust Pressure (P_{TG}) is applied in the vertical direction, to the horizontal plane (bottom projected area), along any 12' length of the mast arm. The pressure shall also be applied to the bottom area of each traffic signal head, sign panel, and all other devices located within that 12' length of the mast arm, to create the maximum stress range (as clarified by the approved changes to the AASHTO Specification).

    F_{TG} = P_{TG}(A)   (lbs)
    where A is the Horizontal Projected Area (Bottom) of the appropriate object within 12' of the mast arm

5. **Direction of Applied Force:** The Truck Gust Force is to be applied in the vertical direction (see figure below).

6. **Load Application:** Poles with multiple 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. The Gust Pressure need not be applied to any portion of the structure not located directly above the traffic lane. For structures installed at locations where the posted speed limit is much less than 65 mph, the design pressure may be recalculated based on this lower wind velocity using Equation C11-6 (as modified per the approved changes to the AASHTO Specification). For structures with arms and/or devices above 19.7 ft., the applied pressure may be linearly reduced to a value of zero at 32.8 ft. (as modified per the approved changes to the AASHTO Specification).

7. **Owner Specified Options:**
    * Truck-Induced Gust loading may be excluded for the design of overhead cantilevered traffic signal support structures, as allowed by the Owner.
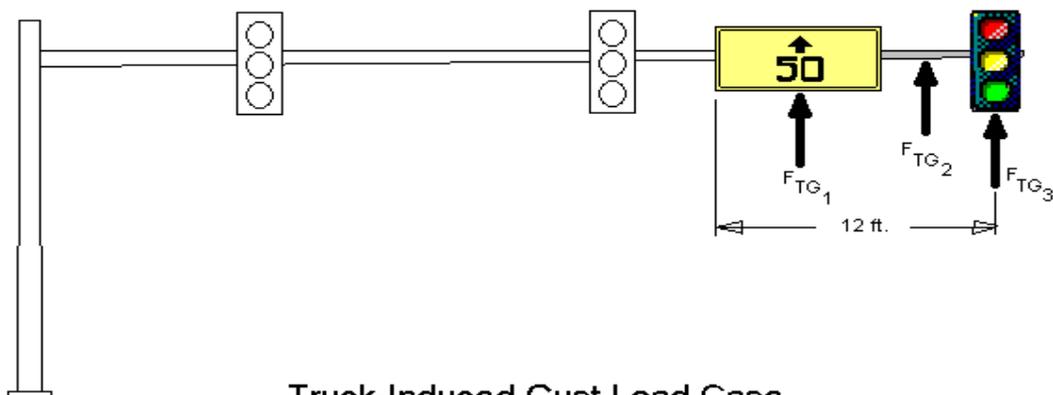
Truck-Induced Gust Load Case

Vortex Shedding Load Case

1. **What is it?** Vortex shedding loads are due to the resonant vibration of a structure caused by the shedding of vortices of air on alternate sides of a member (i.e., pole). When these vortices shed from the pole they create a change in pressure from one side of the pole to the other. If the frequency of the shedding of vortices from alternate sides of the pole matches the natural frequency of the pole, the pole will begin to vibrate in a direction normal to the flow of air past the pole.

   The frequency of the shedding of vortices is a function of the wind speed and the diameter of the pole. Therefore, for a given wind speed, there is a limited range of pole diameters which will result in vortices shedding from the pole at a frequency which matches the fundamental frequency of the structure and therefore could incite significant vibration. Because of this fact, members with significant tapers (i.e., 0.14 in/ft or greater) are not susceptible to vibration caused by vortex shedding. This is due to the fact that only a small portion of the pole would have a diameter that would cause vortices to be shed at the same natural frequency of the structure (see figure below).

2. **Applicable Structures:** Non-tapered high-level lighting supports (Note: the commentary states that the Owner or Designer may require high-level lighting structures to be designed for vortex shedding, when the taper is less than 0.14 in/ft.)

3. **To Calculate Applied Pressure:** Vortex Shedding Pressure (P_{VS}) is calculated by the following formula (see figure below):

   $P_{VS} = (\ 0.00118(V_C^2)(C_d)(I_F)\ ) / 2\beta\ $ (psf)
   where V_C is calculated from equation 11-2 or 11-3, C_d is the appropriate drag coefficient (using $V = V_c$), I_F is the Fatigue Importance Factor from Table 11-1, and β is the dampening ratio (0.005)

4. **To Calculate Applied Force:** The Vortex Shedding Pressure (P_{VS}) is multiplied by the frontal projected area of the pole and mast arm, but not to any attachments.

   $F_{VS} = P_{VS}(A)\quad$ (lbs)$\qquad$ where A is the Frontal Projected Area of the tubular member

5. **Direction of Applied Force:** The Vortex Shedding Forces (F_{VS}) are applied transversely to poles (i.e., horizontal direction) and vertically to horizontal mast arms. The force is always applied perpendicular to the applied pressure (P_{VS}).

6. **Options:** In lieu of designing to resist periodic vortex shedding forces, approved vibration mitigation devices may be used.

**Non-Tapered** ... Dc ... Dc ... **Tapered**

Vortex Shedding Load Case
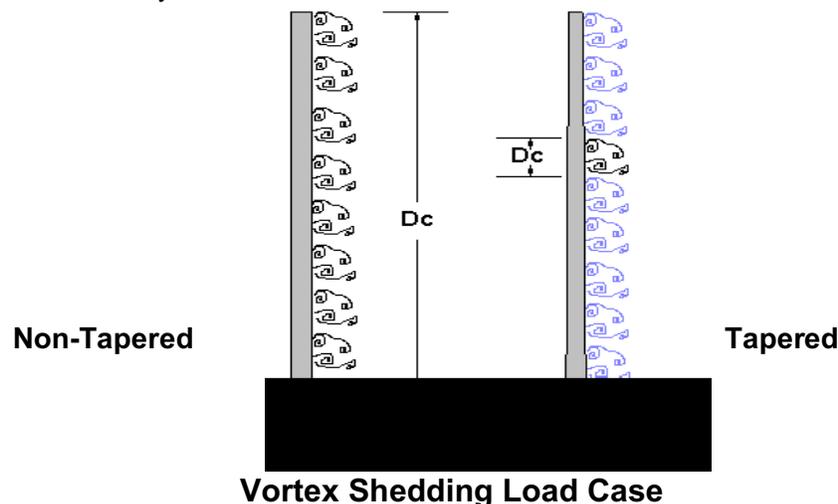
Allowable Fatigue Stress

1. **What is it?** The allowable stress for fatigue is quite different than the allowable stress for the static Group I, II, and III load cases. For fatigue, the allowable stress is not a function of the particular yield strength of the material. Instead, the allowable stress is based on the constant-amplitude fatigue limit (CAFL) of the particular material type involved (i.e., either steel or aluminum) and the type of connection being evaluated. CAFL's for steel can be as high as 24 ksi, and depending on the type of connection, can be less than 1 ksi. The CAFL's for the different fatigue detail categories are found in Table 11-3 of the Specification.

2. **Constant-Amplitude Fatigue Limit (CAFL):** The CAFL is the stress range, which the fatigue life of appears to be infinite. The CAFL is sometimes called the fatigue endurance limit (see figure).
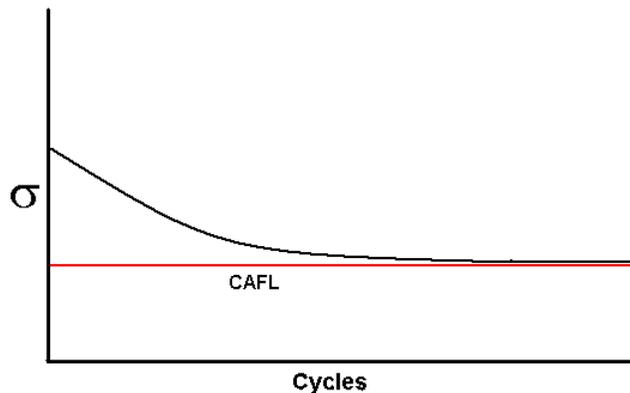
Constant-Amplitude Fatigue Limit

3. **Infinite-Life Fatigue Approach:** The CAFL values in Table 11-3 were determined based on an "infinite-life" fatigue approach. This approach was used due to the premise that there is no practical way to determine how many cycles of any given stress that a structure will undergo during its life. Therefore, the allowable stress range for connections is based on the idea that the connection could be at the CAFL level an infinite number of times during its life. This approach is considered quite conservative due to the fact that while a typical connection might undergo 100 million or more cycles of stress, no more than 10 to 20 million of those stress cycles will be near the CAFL level (see figure below).
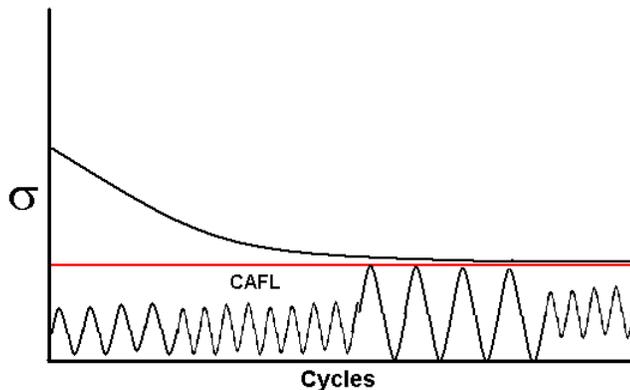
**Infinite-Life Fatigue Approach**

Fatigue Importance Factors

1. **What is it?** Importance factors are used to adjust the level of reliability of structures based on the degree of hazard to traffic and damage to property, which might result due to the failure of the structure. The level of reliability is adjusted by multiplying a given importance factor by the static pressure for a given load case. The higher the importance factor, the higher the static pressure used for design. Therefore, the higher the resulting reliability of the structure.

   The fatigue importance factor should be determined by the Owner.

2. **Fatigue Categories:** The fatigue importance factors are listed for three different fatigue categories. The first being Category I, which is for critical cantilevered support structures installed on major highways. The second is Category II, which is for other cantilevered support structures installed on major highways and all cantilevered support structures installed on secondary highways. The last is Category III, which is all cantilevered supports, installed at all other locations. (see table below)

3. **Load Cases and Fatigue Categories:** Each fatigue category has specific importance factors listed for each of the four fatigue load cases. The difference in importance factors between fatigue categories varies from load case to load case. For example, difference in importance factor between category II and III is only about 7% for the truck-induced gust load cases, where as for the galloping load case, the importance factors differ by as much as 31%. Therefore, the controlling load case could change depending on what is the fatigue category.

4. **Combined Structures:** The importance factors also differ depending if the structure in question is a sign supporting structure, a signal supporting structure, or a lighting structure. If the structure is supporting a combination of signs, signals, and/or lighting, the highest of the importance factors should be used to design the structure.

Table 11-1. Fatigue Importance Factors, I_F					
Fatigue Category		Galloping	Vortex Shedding	Natural Wind Gust	Truck-Induced Gusts
I	**Sign**	1.00	X	1.00	1.00
	Traffic signal	1.00	X	1.00	1.00
	Lighting	X	1.00	1.00	X
II	**Sign**	0.65	X	0.75	0.89
	Traffic signal	0.65	X	0.80	0.84
	Lighting	X	0.65	0.72	X
III	**Sign**	0.31	X	0.49	0.77
	Traffic signal	0.30	X	0.59	0.68
	Lighting	X	0.30	0.44	X

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STRUCTURES

Fatigue Stress Categories

1. **What is it?** AASHTO has provided a table of typical light and traffic support structure details showing the stress category of each connection (see Figure 11-1 and Table 11-2). This table categorizes connections by type and indicates the stress category that each type of connection falls into. The stress category indicates the fatigue sensitivity of the connection and is used to find the applicable CAFL in AASHTO Table 11-3.

2. **Stress Categories of Common Connections and Components:**
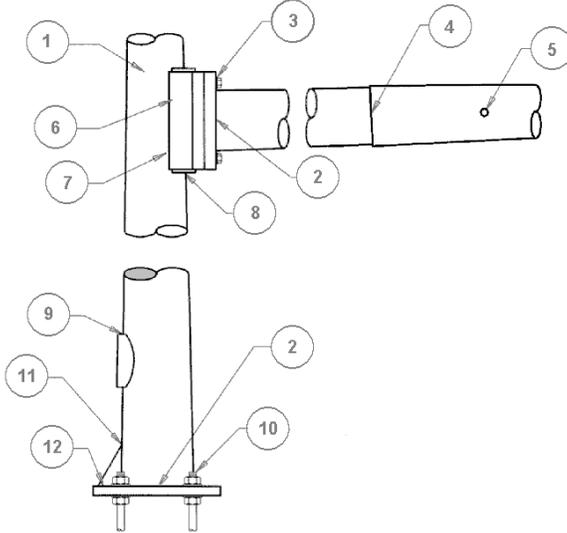
From AASHTO Table 11-3	
Stress Category	Steel CAFL (ksi)
A	24.0
B	16.0
B'	12
C	10.0
D	7.0
E	4.5
E'	2.6
ET	1.2
K_2	<=1.0

	Item	AASHTO Table (11-2) Detail	Stress Category	Stress Based on Tube S_x
1	Pole/mast tube: 1. No longitudinal seam weld (e.g., aluminum shaft) 2. With longitudinal seam weld (e.g., steel shaft)	1 8	A B'	X X
2	Pole to base plate and mast to flange connections: 1. Socket weld 2. Full penetration groove-weld with the backing ring attached to the plate with a continuous full penetration weld, or with a continuous interior fillet weld (as modified per the approved changes to the AASHTO Specification).	16 11	E' E	X X
3	Fasteners in tension	5	D	N/A
4	Slip joint	2	B	X
5	Unreinforced hole	7	D	X
6	Simplex gussets	19	ET	N/A
7	Pole wall at simplex connection	19	K2	Note 1
8	Pole cross section just below simplex connection	19	E	X
9	Most Reinforced hand holes	20	E	X
10	Anchor bolts	5	D	N/A
11	Weld termination at end of longitudinal stiffener (fillet or partial penetration groove weld): Stress Category is dependent on stiffener slope, length, and radius. Nondestructive weld inspection should be used in the vicinity of the weld termination. Grinding of weld terminations to a smooth transition with the tube face is not allowed in areas with fillet welds or partial-penetration welds. Full penetration welds should be used in those areas. Recent fatigue comparison test conducted by Valmont Industries resulted in failures of the pole sections at the gusset terminations.	20, 21, 22	C, D, E	X
12	Longitudinal stiffener to base plate	23	C	Tube + Gusset S_x

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IV. Design Comparisons

Size Impact Due to Fatigue Category

Change in Structure Weight: The new AASHTO Specification could have a huge impact on the size and cost of structures. One factor, which could determine the magnitude of the impact, is the fatigue category the Owner specifies. The chart below shows how the average weights of three Valmont traffic structures are influenced by different fatigue categories.

Influence of Fatigue Category On Structure Weight
(Average Weight of 3 Traffic Structures)
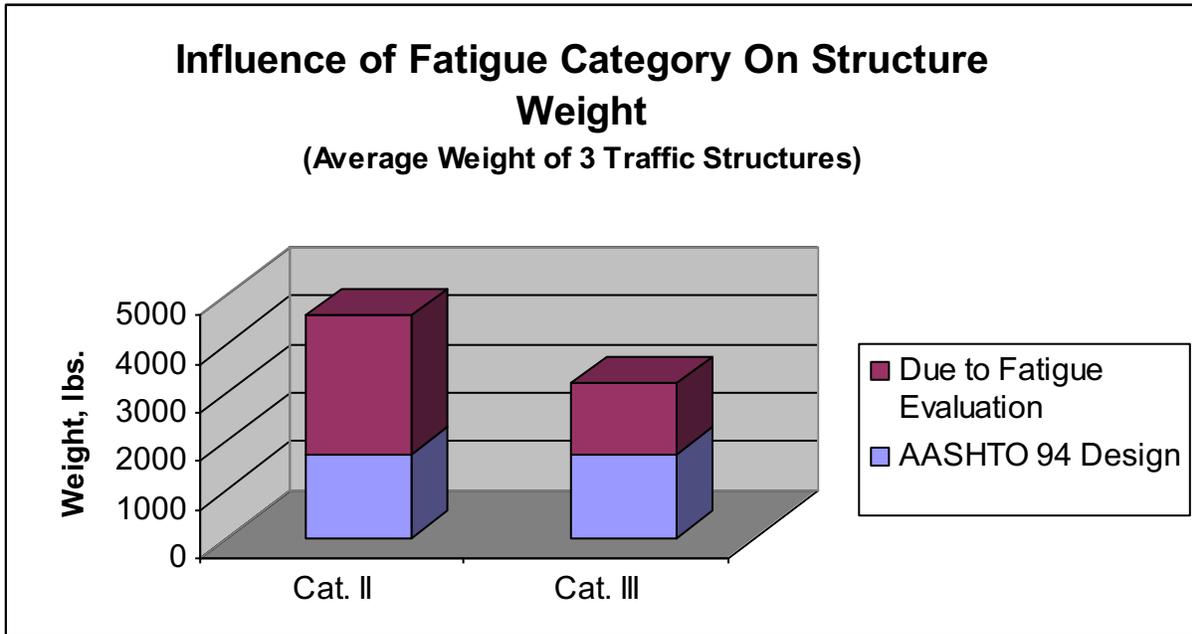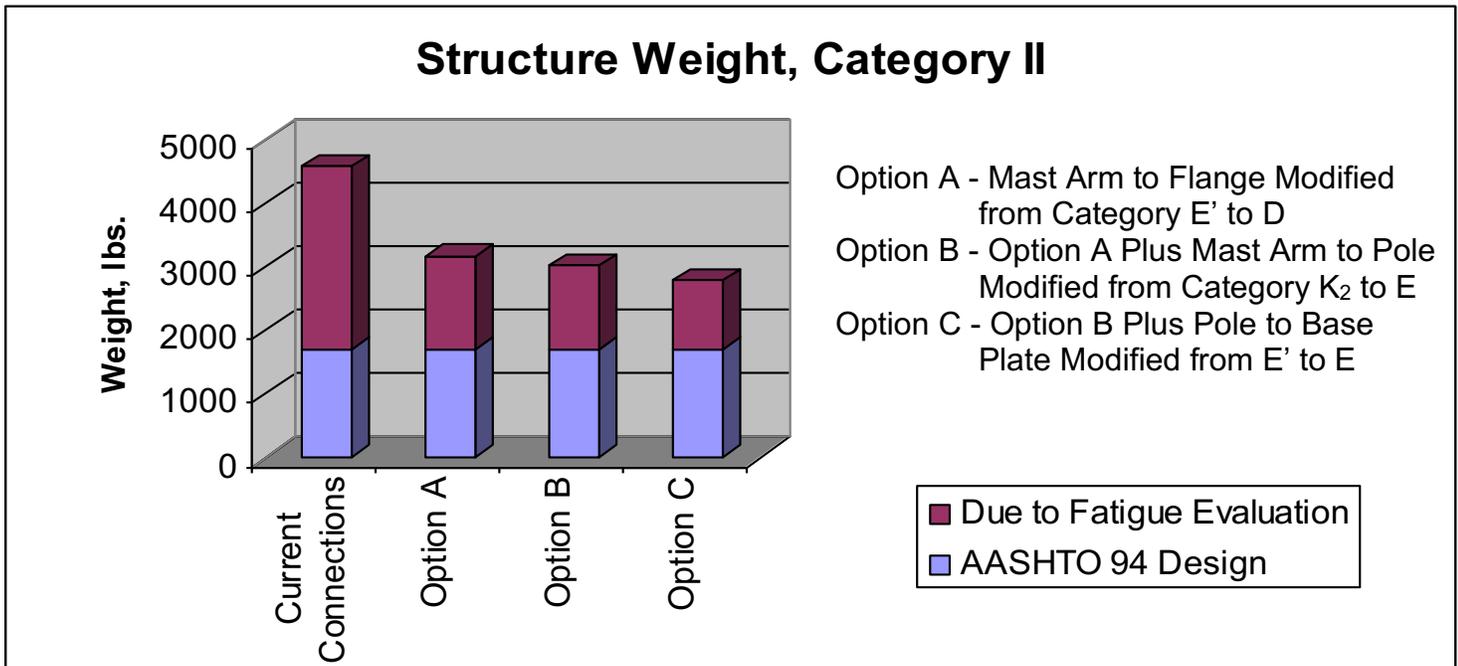
Size Impact Due to Galloping Load Case

Change in Structure Weight: Galloping loads are another factor, which will have a major impact on the size and weight of traffic signal structures. Per the new Specifications, as an alternative to designing a structure to withstand galloping fatigue loads, the Owner may choose to install an approved vibration mitigation device or choose to install an approved vibration mitigation device only when a structure displays a galloping problem. The chart below shows how the average weights of three Valmont traffic structures are influenced by galloping loads.

Influence of Galloping On Structure Weight
(Average Weight of 3 Traffic Structures)
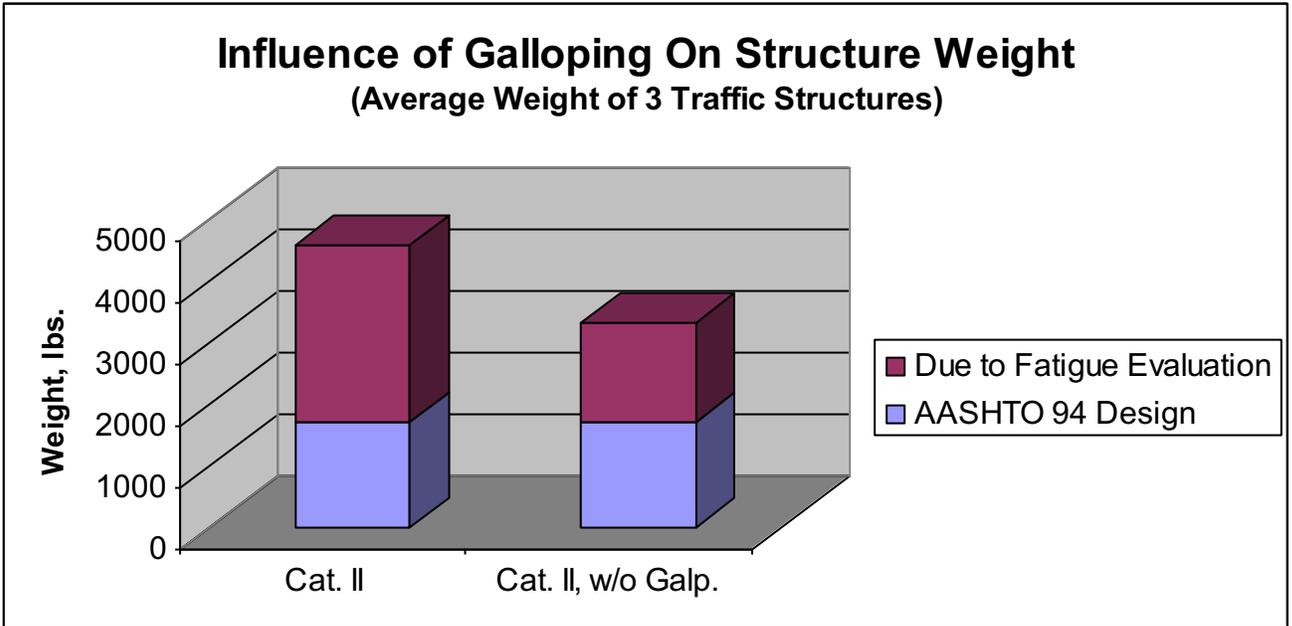
Size Impact Due to Connection Design

Change in Structure Weight: The design of connections could play a large role in determining the size of future light and traffic support structures. Changes to some common connections could reduce the size of these structures by a factor of two or more. This impact can be seen in the chart below which shows the impact of adding stiffeners* to the mast arm connection, a small improvement in the base plate connection, and redesigning the simplex to pole connection to take punching shear stresses out of the pole wall.

The graph compares designs between the 1994 and 2001 AASHTO Specifications for various connection options that were considered feasible. The first bar in the graph shows the increase in weight of a structure using current connection practices. The second bar (i.e., Option A) compares weights if the mast arm to flange plate connection was improved from a stress category E' connection to a stress category D connection. The third bar (i.e., Option B) shows the increase if, in addition, the mast arm to pole connection was improved from a stress category K_2 connection to a stress category E connection. The fourth and last bar (i.e., Option C) shows the weight increase if, in addition, the pole to base plate connection was also improved from a stress category E' to a stress category E connection.

Structure Weight, Category II

Option A - Mast Arm to Flange Modified from Category E' to D
Option B - Option A Plus Mast Arm to Pole Modified from Category K_2 to E
Option C - Option B Plus Pole to Base Plate Modified from E' to E

■ Due to Fatigue Evaluation
□ AASHTO 94 Design

*Caution: Recent fatigue test by Valmont shows that the addition of stiffeners may substantially reduce the fatigue strength of the tube to plate connections.