

FDOT'S CRITERIA FOR WIND ON PARTIALLY CONSTRUCTED BRIDGES



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BIOGRAPHY

Mr. Golabek has recently joined Kisinger Campo & Associates and is the Chief Structures Engineer. He was previously with FDOT as an Assistant State Structures Design Engineer and was the section leader for In-House Design. While at FDOT, his responsibilities included AASHTO SCOBS technical committees, evaluating and implementing PBES, and managing research projects to improve current bridge design policies. Prior to the FDOT, Mr. Golabek had twenty-one years of experience in bridge design with particular emphasis on steel plate girders with skewed and/or curved geometry. Mr. Golabek earned both his Bachelor of Science in Civil Engineering and Masters in Engineering degrees at the University of South Florida.

Christina Freeman is a Structures Research Engineer with the FDOT M.H. Ansley Structures Research Center, where she studies, manages and implements innovative concepts for bridges throughout Florida. Christina holds B.S. and M.S. degrees in Civil Engineering from Florida State University.

SUMMARY

Florida's Bridge designers have been required to design for wind loads on partially constructed bridges since 2000. Design requirements were updated in 2009 and 2015 based on research and wind tunnel testing.

The Florida Department of Transportation (FDOT) bridge design requirements differ from the AASHTO LRFD Bridge Design Specifications in that wind loads are calculated based on a 3 second gust wind speed similar to the method included in ASCE 7.

FDOT's current design practice is to vary wind pressures applied during construction depending on the time period for which the bridge is vulnerable.

Further described in the paper, prescribed pressure coefficients, based on wind tunnel testing, produce forces in the windward girder and girder system similar to forces measured in wind tunnel tests but do not duplicate the exact shielding behavior.

Wind application to steel girder bridges differs from AASHTO Specifications based on analytical research.

FDOT'S CRITERIA FOR WIND LOADING ON PARTIALLY CONSTRUCTED BRIDGES

Introduction

Florida's bridge designers have been required to design for wind loads on partially constructed bridges since the year 2000, using either a reduced wind pressure or lower load factor for the wind loading defined in the AASHTO Standard Specifications (1) or AASHTO LRFD Bridge Design Specifications (BDS) (2). In 2009, the Florida Department of Transportation (FDOT) reworked the design requirements so that wind loads were calculated based on a 3-second-gust wind speed similar to the method included in ASCE 7-05 (3). The current AASHTO LRFD BDS (2) is based on the fastest mile wind speed. However, this wind speed measurement has been discontinued by the National Weather Service and replaced with the 3-second-gust wind speed. In 2015, the FDOT's design requirements for wind load design were again updated with revised pressure coefficients based on several years of research and wind tunnel testing.

Wind Pressure

The FDOT criteria for calculating wind pressure are based on ASCE 7-05 (3) for both completed and partially completed bridges. Variables used in the equation for the design wind pressure include the velocity pressure exposure coefficient (K_z), basic wind speed (V), gust effect factor (G) and pressure coefficient (C_p). The wind pressure equation provided in the FDOT Structures Manual is presented below:

$$P_z = 2.56 \times 10^{-6} K_z V^2 G C_p$$

Equation 1: Design Wind Pressure

The equation for the velocity pressure exposure coefficient (K_z) presented in FDOT Structures Design Guidelines, later referred to as SDG, (4) is taken directly from ASCE 7-05 with a Wind Exposure Category C assumed. The coefficient depends on structure height, and the same equation is used for complete and partially complete bridges. ASCE 7-05 (3) provided an exception to Wind Exposure Category D for hurricane-prone regions. Therefore, all areas of Florida would be

classified as Wind Exposure Category B or C. Applying Wind Exposure Category C for all sites is a conservative assumption. However, ASCE 7-10 (8) removed the exception to Wind Exposure Category D for hurricane-prone regions, so the equation for the velocity pressure exposure coefficient (K_z) may be revised in future versions of the FDOT SDG (4).

The gust effect factor is taken as 0.85 for bridges with spans less than 250 feet and heights less than 75 feet. For other bridges, the gust effect factor is to be evaluated per ASCE 7-05 (3). For partially completed superstructures, the basic wind speed (V) is replaced by a construction wind speed. Values for the pressure coefficient (C_p) are given in the FDOT SDG. Both variables will be discussed below.

Wind Speed

The FDOT's current design practice specifies the wind speed for each county in Florida, which is based on the ASCE 7-05 wind speed map. Unlike the AASHTO LRFD BDS, which is based on the fastest mile wind, the ASCE 7-05 is based on a 3-second-gust wind speed. Furthermore, FDOT classifies construction wind speeds into two categories: Construction Inactive and Construction Active. Both wind speeds are lower than the wind speed used for final design. Reduced wind speeds are appropriate during construction because construction activities occur over a limited time period, therefore reducing the probability of an extreme wind event occurring. The specified wind speed used for designing partially constructed bridges is based on the limited time period during which the bridge is susceptible to wind.

The Construction Inactive category is defined in the 2015 FDOT SDG (4) as "periods during which construction activities associated with the superstructure do not take place." Typically, this occurs when a bridge superstructure is partially constructed but is not actively being worked on. For a typical girder bridge, this includes non-work hours during which the girder bracing is present. The Construction Inactive wind speed is equal to the final design wind speed times a reduction

factor. The exposure period reduction factor (R_E) varies depending on the time period for which the bridge superstructure is vulnerable, defined as the exposure period. The reduction factor is 0.6 for exposure periods less than one year. For all other exposure periods, a factor of 1.0 is used. The exposure period for a girder bridge is defined as the time period from when the girder is set on the pedestals until the girder is made composite with the bridge deck, and the exposure period for a segmental bridge is defined as the time period from when segments are placed until spans are made continuous.

The ASCE Standard 37-02, *Design Loads for Structures During Construction* (5), and the FHWA *Engineering for Structural Stability in Bridge Construction Reference Manual* (6) provide four different reduction factors ranging from 0.65 to 0.9 for construction periods ranging from less than six weeks to five years. Multiple reduction factors result in a consistent probability of the design wind event occurring. Although those sources allow for varying reduction factors, a single reduction factor is presented in the 2015 FDOT SDG (4) for simplicity. For all bridges, designers are required to list the assumed Construction Inactive wind speed, including the reduction factor which indicates to the Contractor the assumed exposure period.

The Construction Active category is defined as periods during which construction activities take place. A 20-mph construction wind speed is typical. The 20-mph Construction Active wind speed is considered to be the maximum design wind speed during which construction operations, such as girder lifting, bracing installation, and deck placement, would reasonably occur.

Pressure Coefficient and Shielding

The pressure coefficient is used in the formula for wind pressure on a structure. It is a direct factor on wind pressure. The wind pressure is multiplied by the projected area of a structure to calculate the total wind load. The wind load is used for girder design, cross-frame design, and to determine substructure (and falsework) reactions. Since bridge girders are designed to be efficient for longitudinal moment, stiffness in the lateral direction is low, particularly for long spans. Once

the superstructure is complete, the deck provides sufficient transverse stiffness. However, for partially constructed bridges consisting of bridge girders without a hardened deck, the controlling load case is frequently transverse wind load. As a direct factor on wind load, the pressure coefficient has a significant impact on that temporary load case.

During the 2009 revision to the FDOT's wind load requirements, it became apparent that information on the pressure coefficients for bridge girder shapes was lacking. Therefore, research by the University of Florida was started (7). The scope of the research included tests on 1-, 2-, 5- and 10-girder systems with steel I-girders, Florida I-beams and trapezoidal box girders. Various spacing, cross-slopes and horizontal wind angles were tested. The research found that some amount of shielding occurred for all girder types, and negative pressures were common for the second and third girders of a cross-section. The results of the research were codified into simple pressure coefficients and application instructions. The prescribed pressure coefficients produce forces in the windward girder and girder system similar to forces measured in the wind tunnel tests but do not duplicate the exact shielding behavior.

The objective of the research project was to supplement the limited information available for girder-type bridge superstructure shape drag coefficients. The drag coefficient, also called a pressure coefficient, is one of several shape factors which indicate the contribution of the geometry of an object to the aerodynamic force. Various structural shapes divert wind flow, and therefore the applied load, differently. At the time of the research, most of the existing research investigations for I-shapes were for basic truss and building members with width-to-depth ratios less than approximately $\frac{1}{2}$. However, most steel I-shapes used in long-span bridges have lower width-to-depth ratios. (7) Concrete I-girder shapes have similar width-to-depth ratios as girders in previous investigations, but drag coefficients for thick wall shapes and, specifically, the concrete I-shape girders used in Florida have not been investigated. Information was also limited for box girders, specifically, open-top trapezoidal box girders, which are present before a deck has been cast.

At the time of the 2009 revision, pressure coefficients specified in the FDOT SDG (4) were

based on rectangular shapes with width-to-depth

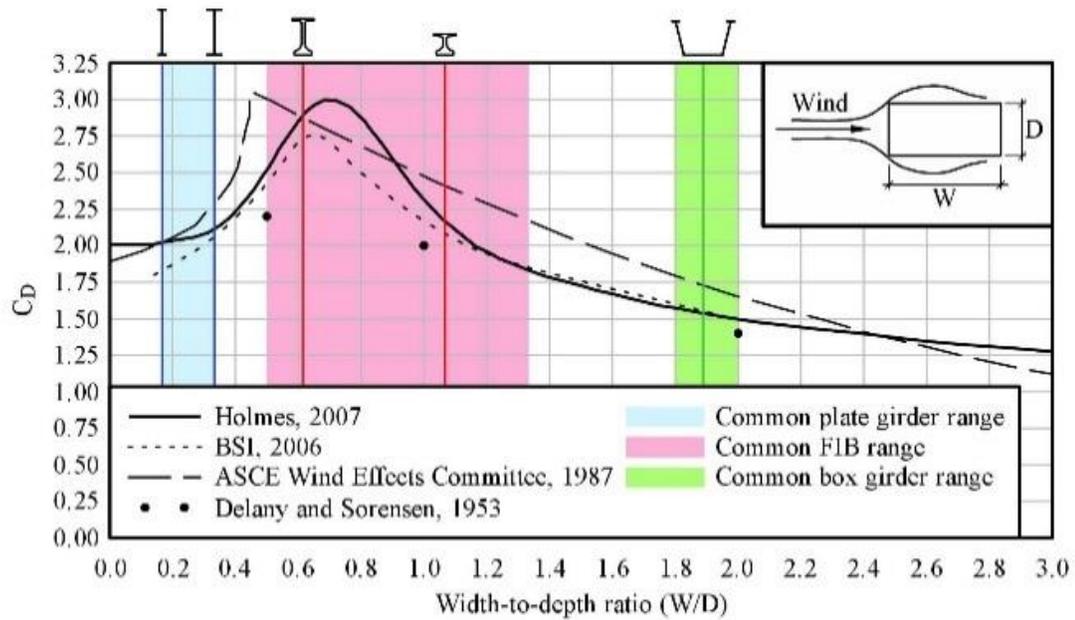


Figure 1: Pressure Coefficient vs. Width-to-Depth Ratio (7)

ratios matching those for common bridge girder shapes. The pressure coefficients measured for rectangular shapes vary dramatically over the width-to-depth range for Florida I-beam girders, as can be seen in Figure 1. Additionally, the profile of typical girders is much different than a rectangle, therefore, research for more appropriate pressure coefficients was warranted.

Five different bridge girder shapes were investigated in the wind tunnel testing. The shapes included two different steel girders, two different Florida I-beam girders, and a box girder. The different steel and Florida I-beam girders selected had different aspect ratios in an effort to capture the upper and lower bound of pressure coefficients for those shapes. For the steel girders, the aspect ratios selected included a typical 8-foot-tall girder with the widest commonly used flange and the narrowest flange commonly used. For the Florida I-beams, 45-inch- and 78-inch-tall beams were selected for testing. At the time of the research, the 78-inch-tall beam was the tallest standard beam and, therefore, the most susceptible to instability. The 78-inch-tall beam is also close to the peak of the pressure coefficient versus aspect ratio chart for rectangular shapes which was used to estimate pressure coefficients for girder shapes in the 2009

FDOT SDG (4). The test setup consisted of one or multiple girders in a cross-section in order to quantify the effects of shielding. The steel girders and Florida I-beams were tested in cross-sections consisting of one, two, five or ten girders. The box girder cross-sections tested consisted of one or two girders. The box girder is a representative 6-foot-deep cross-section.

The steel girders were tested at girder spacing ranging from 10 feet to 14 feet, while the Florida I-beams were tested at a spacing range from 10 feet to 13 feet, and the box girders were tested at girder spacings from 20 feet and 22 feet. To account for natural variation in wind angle, the bridge cross-sections were tested at wind angles ranging from negative to positive five degrees, in 2.5 degree increments, as recommended by the commercial wind tunnel test facility (7). The steel girders were tested at two percent and eight percent cross-slopes, representing the lower and upper bounds for common bridge construction. The Florida I-beams were tested at the most common two percent cross-slope. Box girders constructed for Florida bridges are typically oriented along the cross-slope; therefore, the box girders were tested in a flat configuration with an amplified wind angle to account for cross-slope.

Wind tunnel testing was performed at the Boundary Layer Wind Tunnel Laboratory at the University of Western Ontario (London, ON). Figure 2 shows a photograph of the wind tunnel test facility. Girders were constructed at a reduced scale and tested in a smooth flow. During each wind tunnel test, the load effects on only one girder was measured. Other scale girders are positioned around the test girder within an adjustable frame to achieve the desired girder position. The entire bridge cross-section was rotated in order to achieve the desired wind angle.

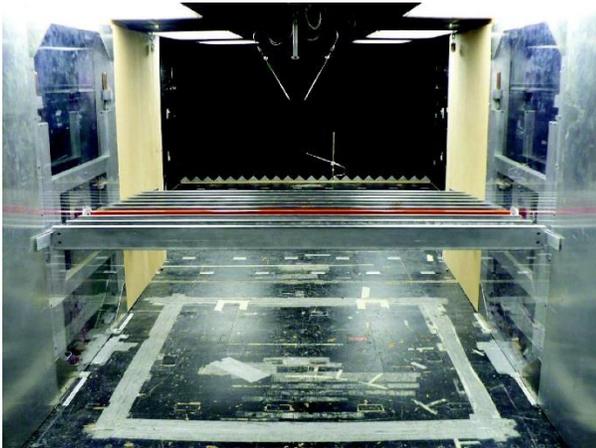


Figure 2: Wind Tunnel Test Facility (7)

The wind tunnel testing produced results in terms of drag, lift, and torque applied to the centroid of the girder. The drag and torque coefficients were converted into one equivalent drag coefficient applied at the center of wind pressure. The equivalent drag coefficient was used to produce

codified pressure coefficients for the FDOT SDG (4).

Several notable behaviors were observed in the test data. Shielding was present in all of the wind tunnel tests. For multiple girder cross-sections, the windward girder provided some amount of shielding for all down-wind girders. The highest amount of pressure occurred on the windward girder. Negative pressure was commonly observed for the second and third girders in a cross-section. In fact, there was no significant positive drag recorded for the second girder of any cross-section in all of the I-shaped girder tests performed. Positive drag was recorded for the second girder during the two-box-girder test, but the magnitude of drag on the first girder was three times the magnitude on the second girder. The occurrence of a drag coefficient plateau value was evident in the ten-girder tests, but not in the five-girder tests. Figure 3 shows all test results for Florida I-Beam cross-sections. Some multi-girder tests indicated a negative or zero drag on the third girder of the cross-section while other tests recorded positive drag on the third girder. The girder spacing to girder depth ratio for the cross-section was found to be a good indicator for the occurrence of positive or negative drag for the third girder in a cross-section. See Figure 4. When the girder spacing to girder depth ratio was greater than 3, positive drag was recorded for the third girder of the cross-section. Otherwise, the drag coefficient for the third girder was approximately zero or negative.

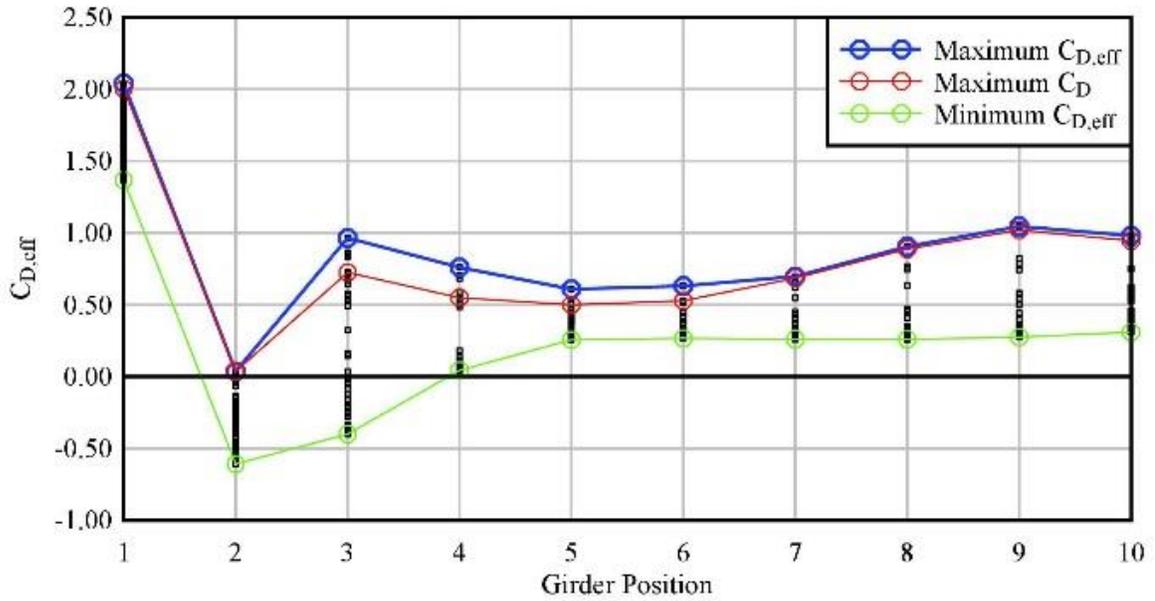


Figure 3: Florida I-Beam Test Results (7)

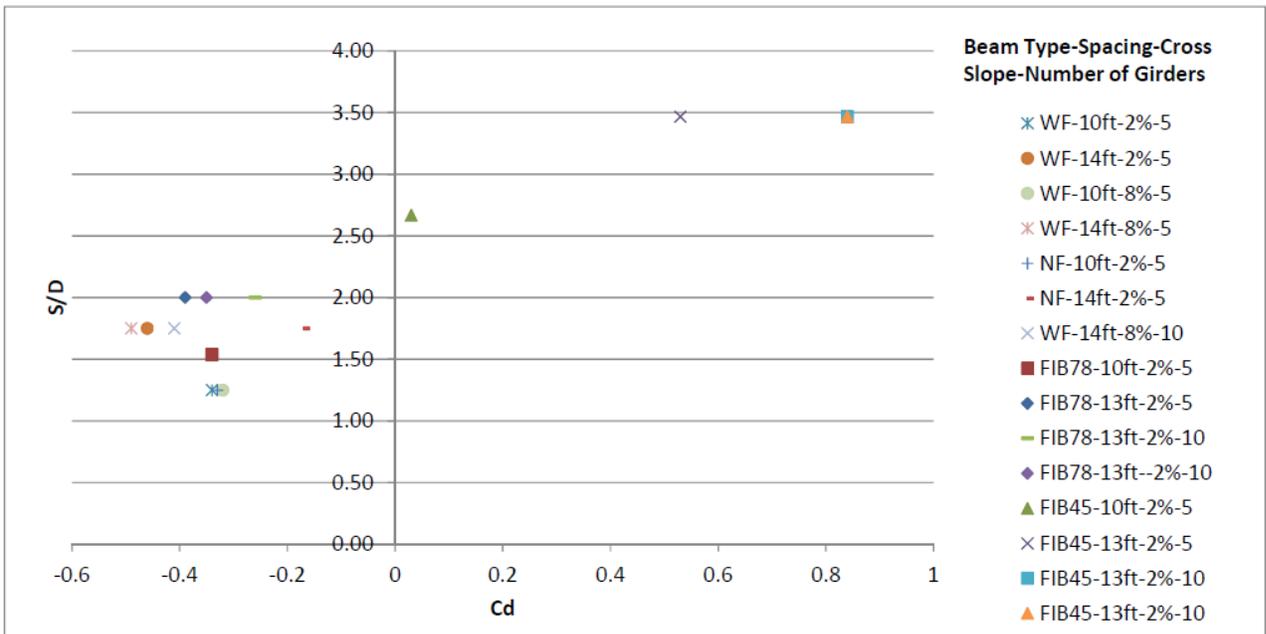


Figure 4: Girder Spacing-to-Depth Ratio (S/D) vs. Third Girder Pressure Coefficient

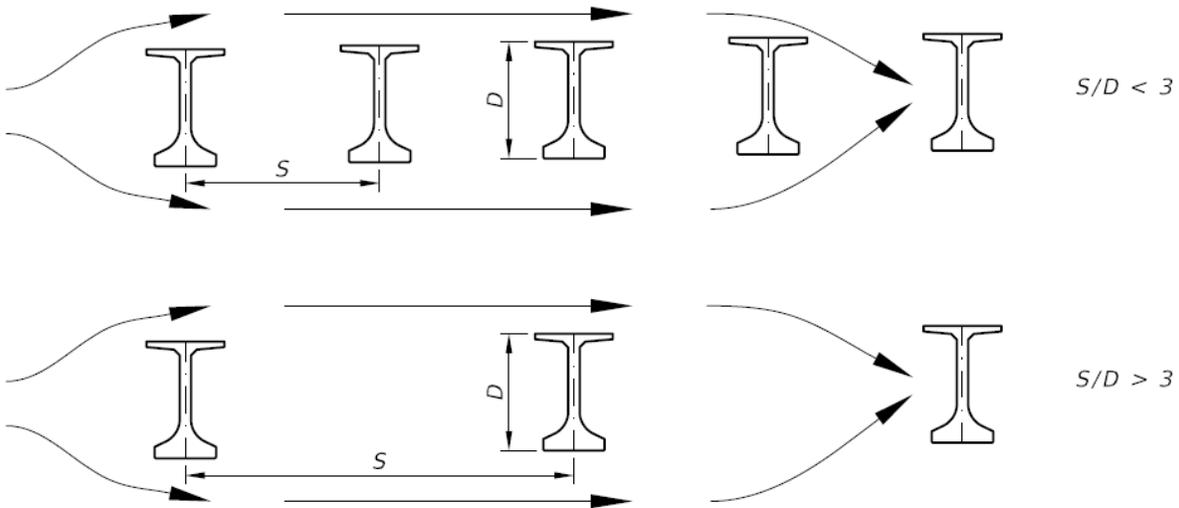


Figure 5: Shielding Behavior

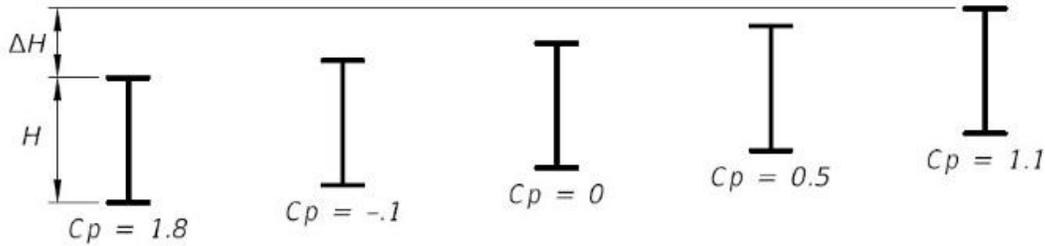


Figure 6: Global Pressure Coefficient

Component Type	Pressure Coefficient (C_p)					
	$S/D \leq 3$		$S/D > 3$			
	Girders 1-5	Girder 6+	Girder 1	Girder 2	Girder 3+	
Superstructure	I-Shaped Steel Girder	2.2	1.1	2.5	0	1.1
	I-Shaped Concrete Girder	2.0	1.0	2.0	0	1.0
	U-Shaped Girder	2.2				
	Flat Slab or Segmental Box Girder	1.5				
Substructure	1.6					

Figure 7: Pressure Coefficient during Construction (4)

The relationship between shielding and the S/D ratio is represented pictorially in Figure 5. The first girder in a cross-section provides a disruption of the wind flow. For the typical girder spacing tested, the disruption resulted in negative or zero pressure on the second girder. Depending on the spacing of girders, the wind flow disruption may prevent positive pressure from being applied to several subsequent girders as well.

The wind tunnel test results presented drag coefficients for application to the individual height of each girder within a bridge cross-section. However, the typical design practice is to apply the wind pressure to the projected height of a partially constructed bridge superstructure. The wind tunnel test drag coefficient results were converted to a drag coefficient which can be applied to the projected height of the cross-section. This was done by summing the total drag on a

tested cross-section and dividing by the projected height for that specific test. An example is shown in Figure 6.

$$Global\ C_p = \frac{\sum C_p \times H}{(H + \Delta H)}$$

Equation 2: Global Pressure Coefficient

The global pressure coefficient (C_p) was calculated for 1-, 2-, 5- and 10-girder cross-sections. The result was a constant value for cross-sections with five or less girders and a girder spacing-to-depth ratio less than three. For other configurations, the global pressure coefficient varied more dramatically, and the wind pressure must be applied to the full height of each individual girder for cross-sections with a girder spacing-to-depth ratio greater than three or for girders beyond the fifth girder in cross-sections with a girder spacing-to-depth ratio less than or equal to three.

Design Criteria

The results of wind tunnel testing produced pressure coefficients which can be summarized by Figure 7. The shaded regions indicate the configurations for which wind load is applied to the projected height. Otherwise, wind load is applied to the individual height of each girder. The prescribed pressure coefficients and use of the The wind tunnel tests indicated negative pressure for the second girder in a series for most tests. When positive pressure is applied to the first girder and negative pressure for the second girder, the result is compression for the cross-bracing between girders. The effective compression is the sum of the positive wind pressure on the first girder and the negative pressure on the second girder, so the effective pressure coefficient is higher than the pressure coefficient for the first girder. The wind tunnel test results were analyzed to determine the maximum sum of first and second girder pressure coefficients. For I-shaped steel girders, the maximum value was determined to be 2.9, for I-shaped concrete girders, the maximum value was 2.6 and for U-shaped girders, the maximum value was 3.3. Those values are presented in the FDOT SDG (4) to be used for single brace or cross-frame design.

projected area method is intended to predict forces on the windward girder and girder system similar to forces measured in the wind tunnel tests. The prescribed pressure coefficients do not indicate the exact shielding behavior.

Pictorially, the wind application for cross-sections with a girder spacing-to-depth ratio less than 3 is shown below. Wind is applied to the projected area for the first five girders or all girders if the cross-section consists of five or less girders.

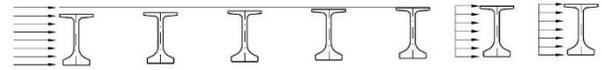


Figure 8: Wind Application for Spacing-to-Depth Ratio Less than 3

The wind application for cross-sections with a girder spacing-to-depth ratio greater than 3 is shown in Figure 9. Wind is applied to the individual height of each girder using the pressure coefficient shown in Figure 7. Note that the pressure coefficient is lower for girders three and beyond. The pressure coefficient for the second girder is zero.



Figure 9: Wind Application for Spacing-to-Depth Ratio Less than 3

Steel Girders

For steel girder bridges, the conventional practice for determining wind loads is using the projected area method. The projected area is defined as the summation of all component areas as seen at an elevation at 90 degrees to the longitudinal axis of the structure. During construction, the projected area is usually the sum of the girder height and the additional height caused by the cross-slope of the superstructure (see Figure 10), multiplied by the girder length. Shielding of the interior girders (as explained previously) was implied.

The AASHTO LRFD BDS (2) Article 4.6.2.7 (AASHTO method) gives an approximate method to determine the lateral moment in the steel girder flanges. For a conventional framing system meeting the third load path criteria as defined in AASHTO LRFD BDS (2) Article 4.6.2.7.1, the flange lateral moment can be determined by equation 3.

$$M_w = \frac{WL_b^2}{10} + \frac{WL^2}{8N_b}$$

Equation 3: Flange lateral moment

where M_w is the total lateral moment in the flange due to the factored wind loading (kip-ft), W is the factored wind force per unit length applied to the flange (kip/ft), L_b is the spacing of cross-frames or diaphragms (ft), L is the span length (ft), and N_b is the number of longitudinal members. The factored wind force is customarily divided equally between the top and bottom flange. The flange lateral bending stress can be determined by:

$$f_\ell = M_w / S_f$$

Equation 4: Flange Lateral Bending Stress

where f_ℓ is the flange lateral bending stress, M_w is defined above, and S_f is the individual flange section modulus taken about the weak axis of the girder.

In order to analytically study the behavior of steel I-girder bridges subjected to wind loads, data from the wind tunnel tests was used to develop wind pressures to be used in finite element method (FEM) models. Results from these models were used to evaluate distribution of the wind loads within the “girder system”, flange lateral bending, and comparison to the AASHTO method. Three FEM models were developed based on actual bridges which are described in Table 1. The term girder system is defined as a group of two or more girders that are connected by cross-frames or diaphragms at supports and intermediate locations.

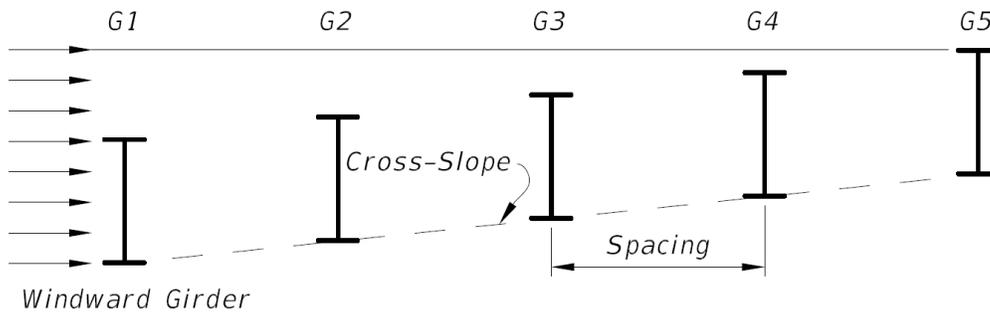


Figure 10: Bridge Cross-Section

Table 1: Bridge Study Parameters

Bridge	E1	E2	E3
Number of Spans	3	2	2
Span Lengths (ft)	184 – 154 – 184	130 – 160	136 – 147
Number of Girders	4	5	5
Girder Spacing (ft)	11.25	11	10
Web Height (ft)	8	4.5	5.67
Girder Spacing-to-Depth Ratio (ft)	1.4	2.4	1.8
Cross-Slope (%)	2	2	2
Cross-Frame Spacing, L_b (ft)			
Span 1	23	17.33	16.5
Span 2	22	17.75	18.0
Span 3	23	N/A	N/A

Table 2: Wind Load

Bridge	E1		E2		E3	
	Span 1 & 3	Span 2	Span 1	Span 2	Span 1	Span 2

Wind Load (kip/ft)	0.15		0.09		0.11	
Flange Lateral Moment (kip-ft)	164	117	42	61	54	63

As previously discussed, FDOT uses wind speed to calculate a wind pressure. A basic wind pressure was calculated assuming a structure height of 70', wind speed of 130 mph, and an exposure period reduction factor, $R_E = 0.60$ (FDOT criteria). Therefore, the applied pressure becomes:

$$P_z = 0.0155 \times C_p \text{ (ksf)}$$

Equation 5: Applied Pressure

The wind pressure equation allows a designer to apply wind loading as a pressure to a 3D FEM model, as a uniform line load to a 2D Grillage model, or line girder model; or a designer can simply use the equation shown in Article C4.6.2.7.1 of AASHTO (2).

For the three example bridges, the flange lateral moments using the AASHTO method are shown in Table 2. The projected area method is used since the girder spacing to web height (S/D) ratio is less than 3. W is calculating using the equation for P_z (shown above), a C_p value of 2.2 (see Figure 7) and multiplied by the projected height.

The FEM models consisted of plate elements for the webs and beam elements for the flanges and cross-frames as shown in Figure 11. The models were supported in the vertical and transverse directions. Longitudinal restraints were placed such that the structure was free to move in the longitudinal direction and still maintain structural stability. A first order linear analysis was used.

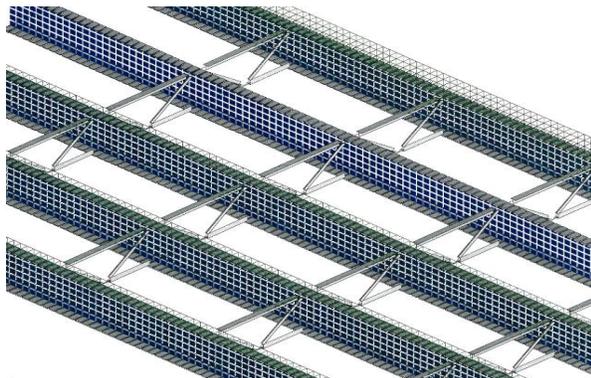


Figure 11: Finite Element Model

Table 3: Pressure Coefficient per Testing

The wind tunnel tests for the steel I-girders included the following variables: number of girders, girder spacing, x-slope, flange width and horizontal wind angle. The results of these tests provided effective pressure coefficients (C_p) for each girder in the cross-section; pressure coefficients were used to calculate the wind pressure. A sample of the tests, defined as Tests A, B, C, and D, are shown in Table 3. Please note that during the wind tunnel testing, the girders were not connected at any intermediate points (i.e., no intermediate cross-frames were used).

For the three example bridges, the wind pressure for each girder was calculated using the pressure coefficient, C_p , shown in Table 3. For example, using Test B results and Bridge E2, the wind pressures applied to the girders (Figure 11), beginning with the windward girder, were 30.2, -2.6, -3.4, 4.2, and 9.9 lb/ft², respectively, in which the first value was applied to the windward girder (Girder 1) and then subsequently to the other girders as shown in Figure 11. For the sake of brevity, results for only Bridge E2 will be presented here.

The top and bottom flange lateral moments for the five-girder system of Bridge E2 using the pressure coefficients from Test B are shown in Figure 12. The sign convention for all the figures is a negative value for a moment which produces a compression stress. Several observations can be made by examining Figure 12. First, the flange lateral moment for the windward girder has a scalloped shape similar to a moment diagram for a continuous girder with the cusps at the cross-frame locations, which is consistent with the AASHTO method. The moment diagrams for the remaining girders are similar but with minimal scalloping due to lower directly applied wind load. Secondly, even though each girder was loaded with a different wind pressure, all the girders share equally in resisting the wind load, again consistent with the AASHTO method. Thirdly, the top and bottom lateral moments are different and dependent on the relative stiffness (moment of inertia) of the top and bottom flanges.

C_p per Girder

Test	Girder Spacing (ft)	Cross-Slope	Wind Angle	1	2	3	4	5	ΣC_p
A	10	2%	2.5°	2.12	-0.08	-0.34	-0.21	0.31	1.80
B	14	2%	-5°	1.95	-0.17	-0.22	0.27	0.64	2.47
C	10	8%	2.5°	2.11	-0.08	-0.32	-0.24	0.29	1.76
D	14	8%	-5°	1.83	-0.14	-0.05	0.48	1.12	3.24

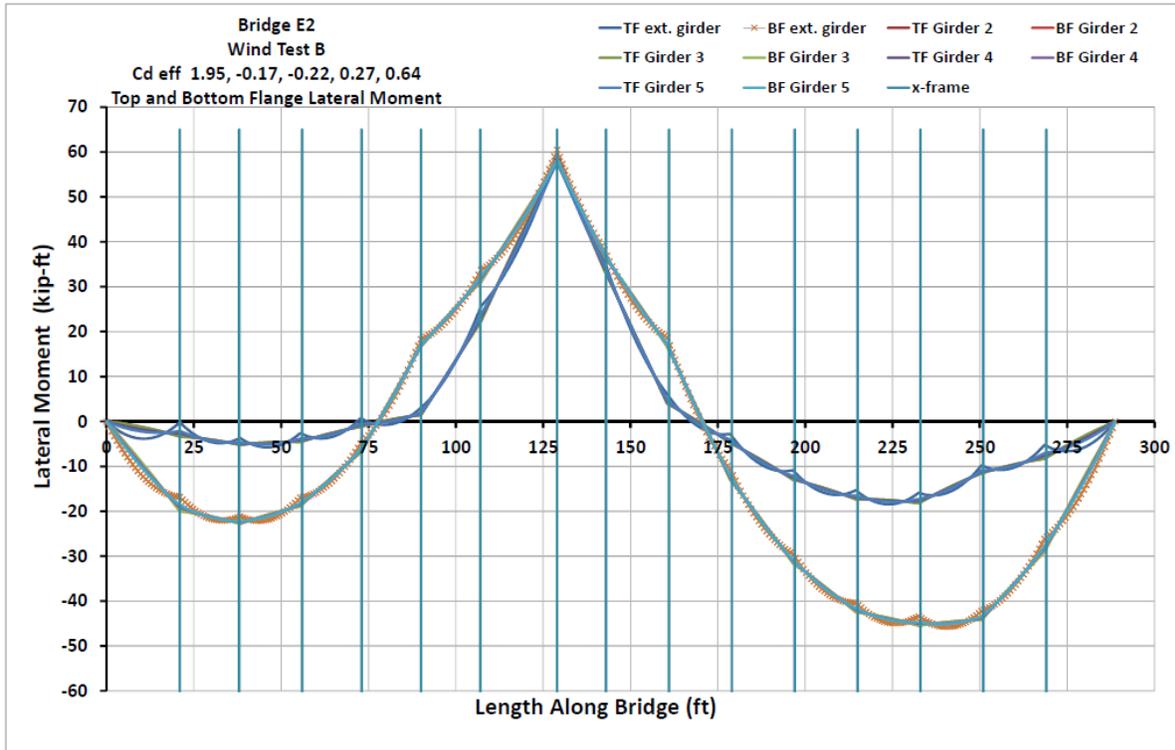


Figure 12: Lateral Moment Due to Wind Load Applied to Individual Girder

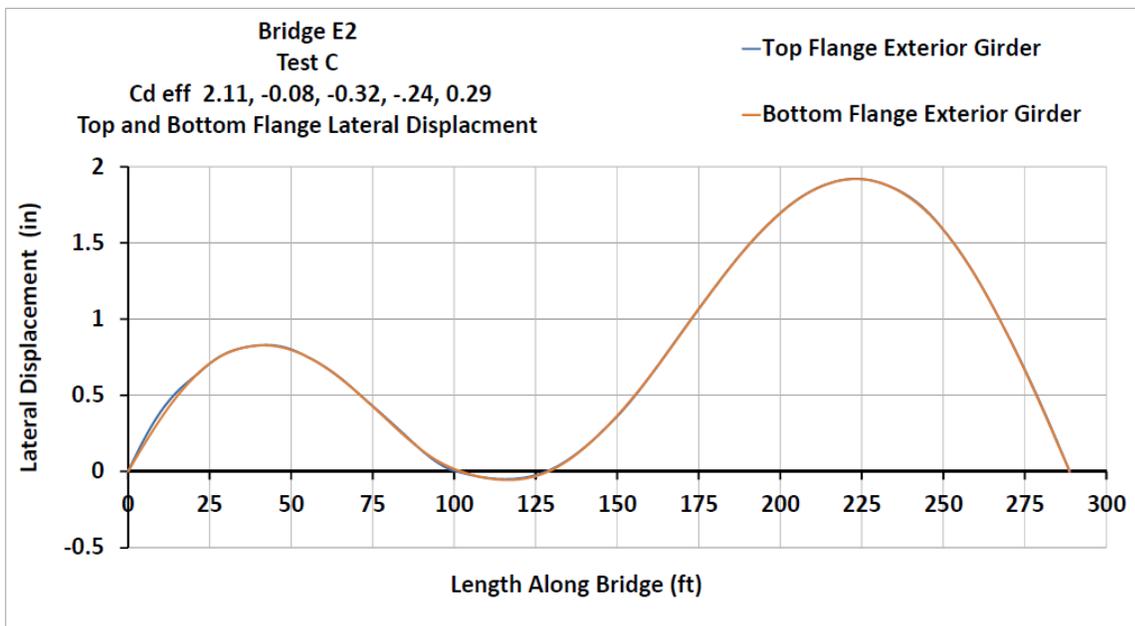


Figure 13: Lateral Displacement Due to Wind Load

As shown in Figure 12, the flange lateral moments are equal in the negative moment region due to the flanges being the same size but are different in the positive moment regions. The difference is proportional to the ratio of the flange moment of inertia, I_f , to the girder weak axis moment of inertia, I_g (I_f/I_g). This behavior is evident when the flange lateral displacements are plotted as shown in Figure 13. Both the girder's top and bottom flanges have the same displacement. The cross-frames resist any rotation of the individual girder, resulting in equal flange lateral displacement, which causes the stiffer element (the larger girder flange) to attract more of the total lateral moment.

Figure 14 includes the girder bottom flange lateral moments but also includes Test A and the wind pressure of the "global" C_p of Test B. The global pressure coefficient is the summation of the individual girder pressure coefficient applied only to the windward girder. For a system of steel I-girders connected by cross-frames or diaphragms, one can see that there is no practical difference in applying the wind pressure to only the windward girder or individual girders in regards to flange lateral moments. This can also be observed by comparing Test A, which has a higher windward

girder C_p (2.12) but has a lower global C_p (1.80) than Test B.

A further evaluation was performed comparing results of the 3D FEM to a line girder analysis. The lateral flange bending stress from the line girder analysis were determined by: (1) dividing the lateral flange moment by the girder weak axis section modulus; and (2) adding moments due to flange bending between cross-frames using the first term of the AASHTO method equation. For simplicity, this additional moment was only added at the flange section transitions and the maximum negative moment location. The comparison of lateral flange bending stresses is shown for Bridge E2 in Figure 15. The line girder analysis gives reasonable results when compared to the 3D FEM analysis. However, the AASHTO method estimates top flange bending stresses of -12 ksi and -8 ksi in the "positive moment sections" of Spans 1 and 2, respectively. These values are considerably higher than those shown in Figure 15 because of the distribution of moments between the top and bottom flange. Since the AASHTO LRFD BDS (2) limits the flange lateral bending stress for steel girders to $0.6 F_y$, it is important for the designer to reasonably estimate these stresses in the partially constructed stages.

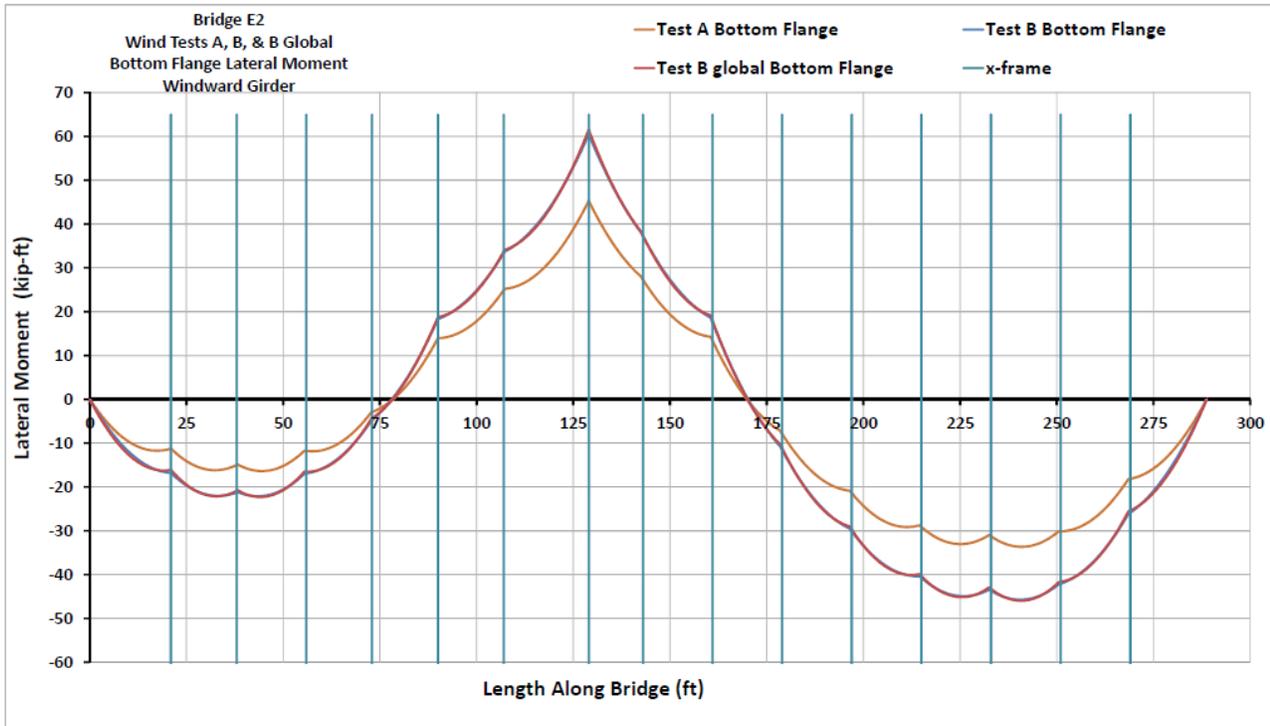


Figure 14: Lateral Moment Due to Wind Load Applied to Individual Girder or Windward Girder

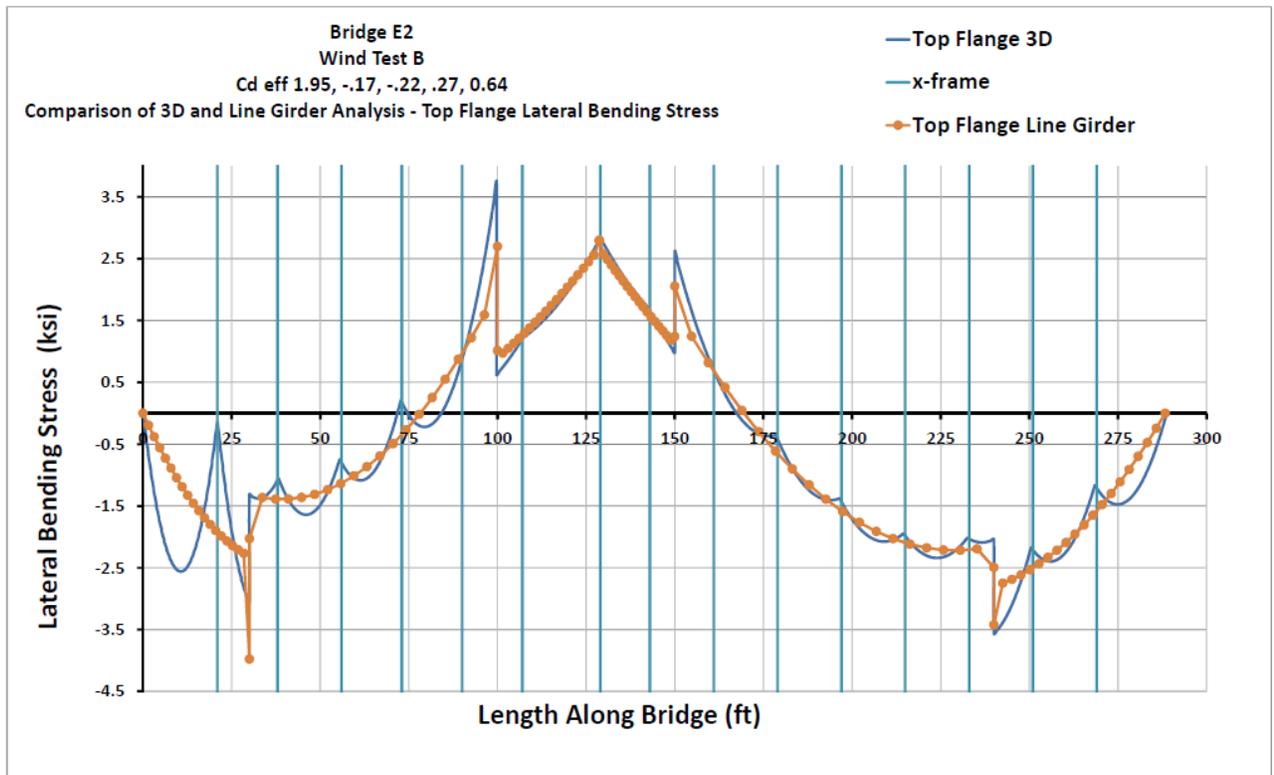


Figure 15: Lateral Bending Stress Due to Wind Load

Conclusions

In summary, the FDOT SDG (4) provides criteria for wind pressure on partially completed bridges and requires the designer to evaluate girder stresses and reactions during construction. FDOT criteria specify two categories for wind speed, Construction Inactive and Construction Active. The Construction Inactive wind speed is based on ASCE 7-05 3-second-gust speed wind maps modified by an exposure period reduction factor. For the Construction Active category, the wind speed is specified as 20 mph (or higher if site condition warrants).

Usually, critical stages for construction of the superstructure are erection of the first girder, a pair of girders, or at a partially completed phase. The contractor may elect to erect more girders by the end of a work shift in order to meet the stress demands imposed by the Construction Inactive wind speed. In general, the wind loading per girder goes down for each girder added. However, wind loads on the substructure or falsework go up.

Wind testing has indicated that the conventional practice of using the projected area method with a corresponding pressure coefficient is sufficient for bridges with five or fewer girders and a girder spacing-to-depth ratio less than three. For other bridges, wind pressure must be applied to the full height of individual girders.

Several 3D FEM models were analyzed for wind load effects on steel I-girders connected by cross-frames or diaphragms. It was determined that the flange lateral bending stress due to global bending can be calculated by using the girder section modulus about its weak axis when intermediate cross-frames are provided to sufficiently prevent rotation of any individual girder. The intermediate cross-frames also provide sufficient connection so that wind load is shared equally among the girders regardless of whether the wind load is applied only to the windward girder or if the load is divided among the girders in the cross-section.

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