FDOT

Florida Department of Transportation

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STRUCTURES DESIGN BULLETIN 24-01
(FHWA Approved: May 11, 2024)

DATE: June 11, 2024

TO: District Directors of Transportation Operations, District Directors of Transportation Development, District Design Engineers, District Consultant Project Management Engineers, District Structures Design Engineers, District Structures Maintenance Engineers, District Program Management Engineers, FDOT Structures Manual Holders

FROM: Will Potter, P.E., State Structures Design Engineer

DocuSigned by:
Will Potter
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COPIES: Will Watts, Dan Hurtado, Michael Shepard, Tim Lattner, Rudy Powell, Lance Grace, Sue Zheng, Derwood Sheppard, Daniel Strickland, Patrick Overton, Scott Arnold, Felix Padilla, Christina Freeman, Ben Goldsberry, Darren Lucas, Andre Pavlov, Vickie Young, Rafiq Darji (FHWA)

SUBJECT: ***Analysis and Design Criteria for Straight Steel I-Girders***

This bulletin implements changes to the required method of analysis and design for straight steel I-girder bridges. Additionally, there are corresponding modifications to criteria not exclusive to straight steel I-girder bridges. These changes affect the ***Structures Design Guidelines*** (***SDG***) and the ***Structures Detailing Manual*** (***SDM***).

REQUIREMENTS

1. Delete ***Structures Design Guidelines*** Section 1.13 and replace with Attachment ‘A’.
2. Delete ***Structures Design Guidelines*** Section 2.9 and replace with Attachment ‘B’.
3. Add new Section 2.12.3 to the ***Structures Design Guidelines***. See Attachment ‘C’.
4. Replace ***Structures Design Guidelines*** Section 5.1.A with the following:

   A. In addition to ***LRFD*** Section 4, see ***SDG*** 5.13 for straight steel I-girder units and ***SDG*** 5.14 for horizontally curved steel I-girder units.

5. Replace ***Structures Design Guidelines*** Section 5.3.2 with Attachment ‘D’.
6. Add new Section 5.3.3 to the ***Structures Design Guidelines*** as follows:

   5.3.3 Fatigue (LRFD 6.6.1.2)

   A. In addition to ***LRFD*** 6.6.1.2.3, components and details on longitudinal primary members having Detail Categories A, B, B’, C and C’ must meet the Fatigue I limit state.

B. Do not use Detail Category E or E' as defined in ***LRFD*** Table 6.6.1.2.3-1. Category E' welds are allowed for use in cross-frame connections.

C. For Fracture Critical Members (FCM), use fatigue details classified as Detail Category C or better as defined in ***LRFD*** Table 6.6.1.2.3-1.

7. Replace ***Structures Design Guidelines*** Section 5.4 with the following:

5.4 Bolts (LRFD 6.4.3.1 and 6.13.2)

A. Design structural bolted connections as either bearing or slip-critical connections in accordance with ***LRFD*** 6.13.2 and as modified by the following:

1. ***SDG*** 5.13.1 and ***SDG*** 5.14.
2. For composite box girder bridges, design all connections of primary members with slip-critical connections. Secondary members may be designed using bearing connections.
3. Use slip-critical connections for steel integral and/or straddle caps (regardless of their classification of superstructure or substructure).

B. Use standard size holes for all bearing-type connections.

C. Use ASTM F3125 Grade A325, Type 1, high-strength bolts for painted connections, and Type 3 bolts for unpainted weathering steel connections.

D. Do not use ASTM F3125 Grade A490 bolts.

E. Non-high-strength bolts must conform to ASTM A307.

F. Bolt diameters of 3/4, 7/8, 1, or 1 1/8-inch typically should be used. Larger bolts may be used with prior approval by the SSDE. Use one diameter and grade of bolt for any individual connection. Specify bolt grade, diameter, associated hole diameter/ size (round/slotted), layout and spacing in the Plans. Maintain minimum edge distance requirements. See also ***SDM*** Chapter 16.

8. Add new bullet '6' to ***Structures Design Guidelines*** Section 5.6.3.D as follows:

6. On the Plans, designate diaphragms as primary members and the connections as slip-critical.

9. Modify ***Structures Design Guidelines*** Section 5.7.A as follows:

A. Design cross-frames and diaphragms (cross-frames at piers and abutments) with bolted connections at transverse and bearing stiffener locations. Generally, a "K-frame" detailed to eliminate variation from one cross-frame to another is the most economical arrangement and should be used. See ***SDM*** 16.7. For straight bridges with a constant cross section, parallel girders, and a girder-spacing-to-girder-depth ratio less than one, an "X-frame" design is generally the most economical and must be considered.

Modification for Non-Conventional Projects:
Delete ***SDG*** 5.7.A.

10. Add new bullet 'E' to ***Structures Design Guidelines*** Section 5.7 as follows:

E. For Case 1 and 2 steel I-girder units (see ***SDG*** 5.13.1), slotted holes in the cross-frame connections are not allowed except as permitted in cross-frame bays that are located under a closure pour. For other skewed steel I-girder units, slotted holes in the cross-frame connections are not preferred but may be used if the structural analysis considers such effects. See ***SDM*** Figure 16.3-2.

11. Modify ***Structures Design Guidelines*** Sections 5.11, 5.11.1, 5.11.2, and 5.11.3.E as follows:

5.11 Connections and Splices (LRFD 6.13)

C. When the thickness of the plate adjacent to the nut is greater than or equal to ¾-inch, base the strength of the connection on the bolt shear strength with threads excluded from the shear plane. For cross-frames, the strength of the bolted connections can be based on the bolt shear strength with threads excluded from the shear plane when a Plan Note is included requiring that the washer and direct-tension-indicator (DTI), if used, be placed under the nut and the thickness of the connecting parts (e.g., connection plate, gusset plate, and chord member as applicable), meet the following:

1. When using 7/8-inch diameter bolts, all connecting parts must be ½-inch minimum.
2. When using 1-inch diameter bolts, one of the connecting parts must be 5/8-inch minimum and the other part must be 1/2-inch minimum.

Commentary: The requirements for excluding threads for cross-frame connections are based on criteria contained in the upcoming 10th Edition of the AASHTO LRFD Bridge Design Specifications which provides for a significant reduction in the number of bolts when threads can be excluded. The ¾-inch plate thickness criteria will be retained until the Department adopts the 10th edition.

5.11.1 Slip Resistance (LRFD 6.13.2.8)

A. Design slip-critical bolted connections for Class A faying surface condition except as noted below. For weathering steel bridges that are not to be painted, design bolted connections for Class B faying surface condition.

5.11.2 Welded Connections (LRFD 6.13.3)

A. Do not show a specific, pre-qualified, complete-joint penetration weld designation on the Plans unless a certain type of weld, i.e., "V," "J," "U," etc., is required. See ***SDM*** Chapter 16, Structural Steel Girders.

Commentary: The fabricator should be allowed to select the type of complete-joint penetration weld to optimize fabrication costs, and all welds should be shown on the shop drawings.

B. Show on the Plans the weld filler metal strength (F_{exx}) for fillet welds assumed in the design.

5.11.3 Welded Splices (LRFD 6.13.6.2)

E. Keep the flange plates of adjacent girders the same thickness where possible. See ***SDG*** 5.13 for additional requirements.

12. Add new Section 5.13 to the ***Structures Design Guidelines***. See Attachment 'E'.

13. Add new Section 5.14 to the ***Structures Design Guidelines.*** See Attachment 'F'.

14. Renumber ***Structures Design Guidelines*** Section 5.13 to 5.15 as follows:

5.15 Global Displacement Amplification in Narrow I-Girder Bridge Units

15. Renumber ***Structures Design Guidelines*** Equation 5-1 to Equation 5-9.

16. Modify ***Structures Design Guidelines*** Section 7.1.1.B as follows:

B. Design bridge widening or rehabilitation projects in accordance with ***SDG*** 7.3 and load rate in accordance with ***SDG*** 1.7. Do not isolate and evaluate the widened portion of the bridge separately from the rest of the bridge, except as permitted in ***SDG*** 7.6.3. After preparing widening or rehabilitation plans, if any ***LRFR*** design inventory or any FL 120 permit rating factors (***MBE*** Section 6, Part A) are less than 1.0, calculate rating factors using ***LFR*** (***MBE*** Section 6, Part B). If any ***LFR*** inventory rating factors remain less than 1.0, replacement or strengthening is required unless a Design Variation is approved. If any ***LRFR*** or ***LFR*** inventory load rating factors are less than 1.0, a revised load rating may be performed using one of the additional procedures in C.1, C.2 or C.3 to obtain a satisfactory rating.

17. Add new bullet 'C' to ***Structures Design Guidelines*** Section 7.3.8 as follows:

C. For steel I-girder units where the cross-frames are not required to be strengthened per Table 7.6.3-1, the fatigue limit state evaluation for the cross-frames is not required.

18. Add new bullet 'H' to ***Structures Design Guidelines*** Section 7.3.9 as follows:

H. When welding is required during rehabilitation or widening of an existing structure, show the type of existing base metal on the Plans. If the base metal type cannot be determined, or if the type is not an approved base metal included in the most current edition of the AASHTO/AWS D1.5 Bridge Welding Code, consult with the State Materials Office to obtain recommendations on how the welding should be specified. Some destructive sampling of the existing structure may be required in order to provide these recommendations. The welding inspection for the rehabilitation or modification for bridge structures should follow the current AASHTO/AWS D1.5 requirements suitable for the type of weld and service conditions and be specified on the Plans. Inspection criteria may change based on the actual field conditions.

Modification for Non-Conventional Projects:
Delete ***SDG*** 7.3.9.H and see the RFP for requirements.

19. Delete ***Structures Design Guidelines*** Section 7.6 and replace with Attachment 'G'.

20. Replace ***Structures Detailing Manual*** Section 5.2.F with the following:

F. Structural Analysis Program [used for the superstructure design] name, version, and version release date. [state whether the live load distribution factor(s) were determined by ***LRFD*** 4.6.2, RMA, or RMA 3D-FEA]

21. Replace ***Structures Detailing Manual*** Section 5.3.B with the following:

B. Primary Members

1. Primary members are identified within these Plans or as specified in Specifications Section 460.
2. Test primary members subjected to tensile stresses in accordance with Specifications Section 962.

22. Replace ***Structures Detailing Manual*** Section 5.3.E with the following:

E. Bolted Connections

1. Use [X]" diameter high strength bolts in accordance with ASTM F3125 Grade A325, Type [X] for all bolted connections. [state other sizes as applicable]
2. All bolt holes must have a diameter of [X]”. [state other sizes as applicable]
3. Use a Class [X] surface condition for all connections identified as slip-critical.
4. For cross-frame connections identified as bearing, install high strength bolts in accordance with Section 460-5 of the Specifications. Prepare faying surfaces to SSPC SP-10. Threads are excluded from the shear plane. The washer and direct-tension-indicator (DTI), if used, must be placed under the nut. Do not place a washer or DTI under the bolt head. [use as applicable, refer to ***SDG*** 5.4, ***SDG*** 5.11, ***SDM*** 16.7, and ***SDM*** 16.8]
5. Position bolt heads on the exterior/exposed face of the girders.
6. For cross-frames designated as primary members, do not fully punch bolt holes. [use as applicable, refer to ***SDG*** 5.3.2]

23. Replace ***Structures Detailing Manual*** Section 13.10.E with the following:

E. Concrete is the preferred material for integral piers. However, if a steel pier cap is used, follow these guidelines (see Figure 13.10-2):

1. Specify structural steel in accordance with the requirements of ***SDG*** 5.3.1.
2. Show splice plates and bolt spacing layout. Indicate bolt size and type.
3. Indicate weld type and size using AWS standard welding symbols.
4. Identify all Primary and Fracture Critical Members (FCM) in the Plans.
5. Include a plan note requiring full shop fit-up.
6. Specify all bolted connections as slip-critical. Include a note, if applicable, stating threads are excluded from the shear plane.

24. Replace ***Structures Detailing Manual*** Section 16.7.C with the following:

C. Plate thickness for gusset plates.

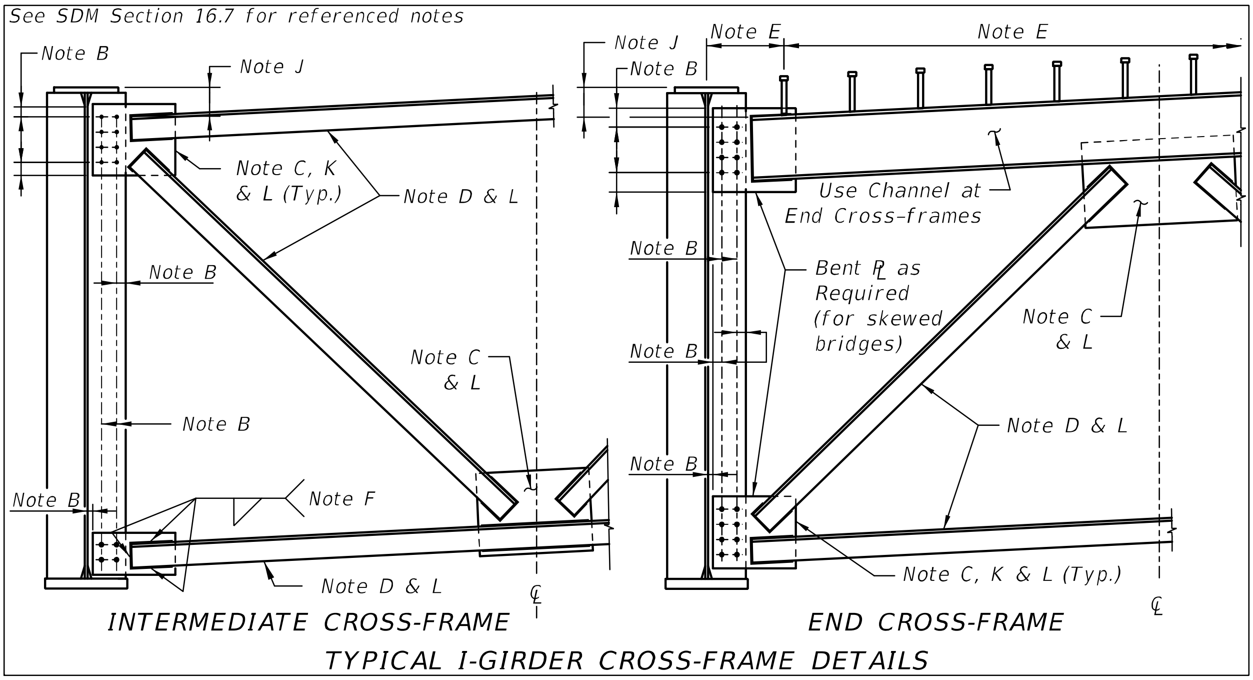
25. Add new bullets 'K' and 'L' to ***Structures Detailing Manual*** Section 16.7 as follows:

K. Connection type. Specify as slip-critical or bearing. Include a note, if applicable, stating threads are excluded from the shear plane and the washer and direct-tension-indicator (DTI), if used, is to be placed under the nut.

L. Member type. Specify as primary or secondary.

26. Replace ***Structures Detailing Manual*** Figure 16.7-1 with the following:

Figure 16.7-1 Typical I-Girder Cross-Frame Details.



27. Add new bullets 'K' and 'L' to ***Structures Detailing Manual*** Section 16.8 as follows:

K. Connection type. Specify as slip-critical or bearing. Include a note, if applicable, stating threads are excluded from the shear plane and the washer and direct-tension-indicator (DTI), if used, is to be placed under the nut.

L. Member type. Specify as primary or secondary.

28. Replace ***Structures Detailing Manual*** Figures 16.8-2 and 16.8-3 with the following:

Figure 16.8-2 Steel Box Girder Pier Diaphragms

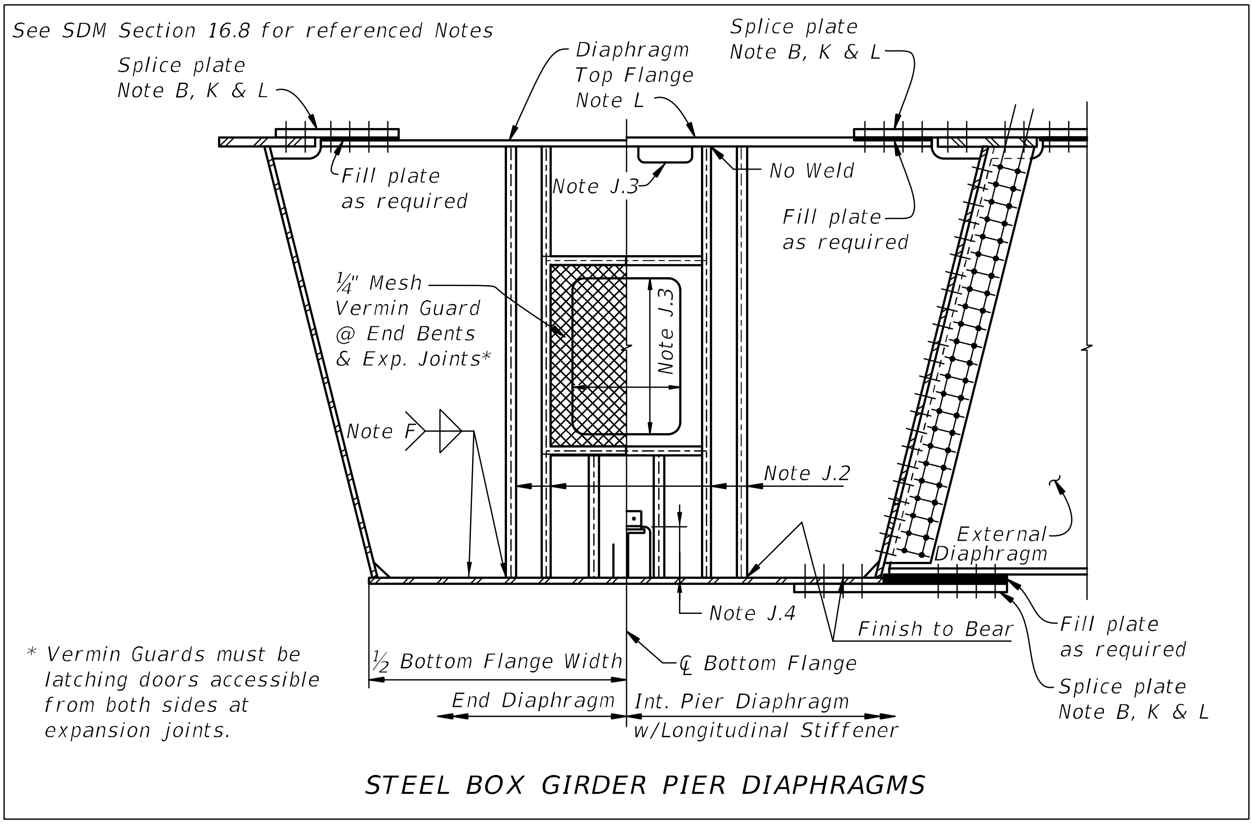
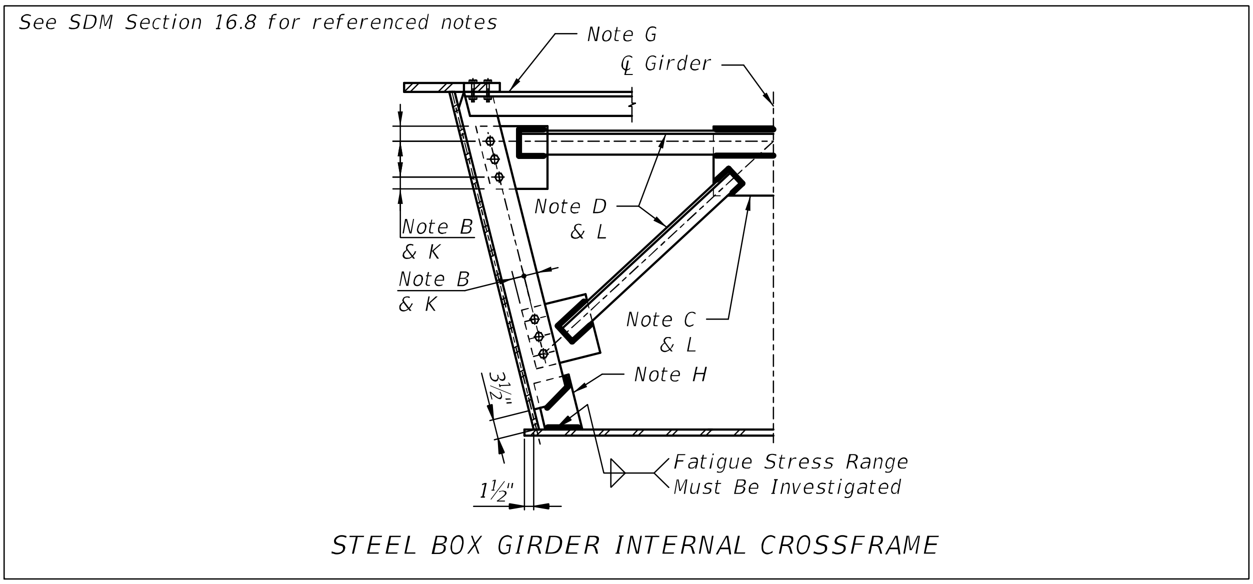


Figure 16.8-3 Steel Box Girder Internal Cross-Frame



29. Add new bullet '6' to ***Structures Detailing Manual*** Section 16.11.C as follows:

6. Specify the field splice as a slip-critical connection. Include a note, if applicable, stating threads are excluded from the shear plane.

COMMENTARY

Straight steel I-girder bridges have traditionally been analyzed and designed using simplified one-dimensional line girder analysis (LGA) models in which the live load forces are determined using live load distribution factors. A refined analysis, such as a 2D grid or 3D finite element analysis, may be required for certain skewed steel I-girder bridges to capture the system behavior accounting for interaction between the girders and cross-frames. These refined analyses require more expensive software and greater time to generate the model, process the output, prepare the calculations, and check the results. The revisions included in this bulletin extend the limits for when a one-dimensional LGA can be used to analyze and design skewed straight steel I-girder bridges, thereby saving significant time and resources on the design effort. Furthermore, experience has shown that the use of LGA models with LLFDs result in girder designs with sizes and proportions that provide an economical and robust design.

BACKGROUND

The AASHTO LRFD Bridge Design Specifications (BDS) permits the use of a line girder analysis (LGA) for skewed straight steel I-girder (SSSI-G) bridges with minimal criteria to evaluate skew effects on girder moments, shears, deflections, flange lateral bending, and cross-frame forces. NCHRP Report 725 [1] provided guidance on the accuracy of differing analysis methods regarding SSSI-G bridges. The FDOT also sponsored research projects that evaluated differing design aspects of SSSI-G bridges [2, 3]. Considering the research results, the FDOT Structures Design Guidelines changed the basis for the required level of analysis for SSSI-G bridges from the skew angle to the skew index. Subsequently, FDOT Research Projects BE535 [4] and BEB13 [5] helped develop new criteria to define straight steel I-girder units by a case number determined by skew angle, skew index, and cross-frame configuration. Case numbers then define the required method of analysis, classification of cross-frame members (primary or secondary), cross-frame connection type (bearing or slip-critical), and cross-frame requirements for bridge widenings (load rating and strengthening). Within the new criteria, Case 1 and 2 requires an LGA with specific analysis criteria including flange lateral bending stresses and cross-frame forces. Case 3 also requires an LGA but is supplemented by a Refined Method of Analysis (RMA) to determine fatigue stress ranges, bearing reactions, flange lateral bending stresses, girder cambers, and cross-frames forces. Cases 4 and 5 are required to use an RMA, with Case 5 needing a full 3D Finite Element Analysis (FEA). An evaluation of the inventory of FDOT bridges constructed between 2000 and 2014 shows that approximately 95% of the bridges have a skew index ≤ 0.45 and skew angle ≤ 50°; therefore, it's anticipated that only 5% to 15% of future SSSI-G steel units will fall under the Case 4 or 5 classifications.

The upcoming AASHTO LRFD BDS 10th Edition (2024 release) contains changes related to cross-frame design for SSSI-G bridges [6, 7]. The revisions address: 1) designation of primary and secondary members, 2) fatigue loading pattern and load modifier for cross-frames, and 3) bearing or slip-critical connections. These changes are implemented in this bulletin.

In addition, commentary language in the upcoming AASHTO LRFD BDS 10th Edition [5] contains guidance on choosing either an LGA or RMA based on fatigue, namely the magnitude of the ADTT(SL). FDOT research also evaluated the magnitude of fatigue loads in cross-frames and its effects on the cross-frame design when using either the Fatigue I or II Limit States. The FDOT Structures Design Office has observed there is a wide range of methodologies that designers use to calculate the ADTT(SL) for determining when the Fatigue I or II Limit States apply. Therefore, a procedure is included regarding the calculation of ADTT(SL) with criteria for using the Fatigue I or II Limit State for cross-frame design. Additionally, criteria is added regarding the fatigue design of the primary longitudinal girders.

Finally, other sections of the SDG and SDM were revised associated with the addition of SDG Section 5.13. Namely, these changes are regarding primary and secondary member designations and when to specify bearing or slip-critical bolted connections for the cross-frames.

References

1. White, D.W., Coletti, D., Chavel, B.W., Sanchez, A., Ozgur, C., Jimenez Chong, J.M., Leon, R.T., Medlock, R.D., Cisneros, R.A., Galambos, T.V., Yadlosky, J.M., Gatti, W.J., and Kowatch, G.T. (2012). *Guidelines for Analytical Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges*, NCHRP Report 725, Transportation Research Board, National Research Council, Washington, D.C.
2. Gull, J.H., Azizinamini, A. (2014). *Steel Plate Girder Diaphragms and Cross-Bracing Loads,* Research Report BDK80 977-20, Florida Department of Transportation, Tallahassee, Florida.
3. Gull, J.H., Azizinamini, A. (2014). *Steel Framing for Highly Skewed Bridges to Reduce/Eliminate Distortion near Skewed Supports,* Research Report BDK80 977-21, Florida Department of Transportation, Tallahassee, Florida.
4. White, D.W., Sherman, R.J., Kamath, A., Heath, J.A., Adams, B.K., Anand, A. (2022). *Applicability of Approximate Methods of Analysis for Skewed Straight Steel I-Girder Bridges,* Research Reports BEB13 and BED03, Florida Department of Transportation, Tallahassee, Florida.
5. White, D.W., Kamath, A., Heath, J.A., Adams, B.K., Anand, A. (2020). *Straight Steel I Girder Bridges with Skew Index Approaching 0.3*, Research Report BE535, Florida Department of Transportation, Tallahassee, Florida.
6. AASHTO Committee on Bridges and Structures, 2022 Agenda Item 25
7. AASHTO Committee on Bridges and Structures, 2021 Agenda Item 37

IMPLEMENTATION

These requirements may be implemented on design-bid-build projects at the discretion of the District.

These requirements are effective immediately on all design-build projects for which the final RFP has not been released. Design-build projects that have had the final RFP released are exempt from these requirements unless otherwise directed by the District.

CONTACT

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WAP/vay

Attachment 'A'

SDG 1.13 – Methods of Structural Analysis

1.13 METHODS OF STRUCTURAL ANALYSIS

A. Use one of the following methods of structural analysis for the superstructure:

1. A Line Girder Analysis (LGA) meeting ***LRFD*** 4.6.2 and in accordance with the following:

    a. ***SDG*** 2.9 for concrete beams.

    b. ***SDG*** 5.13.2 for straight steel I-girder units.

2. A Refined Method of Analysis (RMA) meeting ***LRFD*** 4.6.3 is required for the following situations:

    a. To account for the effects of the substructure and/or foundation stiffness for structure types such as, but not limited to, the following:

    i. Continuous girder superstructure units supported on straddle piers or integral pier caps.

    ii. Continuous girder superstructure units supported on hammerhead piers with cantilevers > 20-feet or C-piers with cantilevers > 15-feet. Cantilevers are measured from the face of column to the centerline of exterior girder.

    iii. Bridge widenings where the existing bridge is supported by substructure elements of different stiffness than the widened section. See also ***SDG*** 7.3.7.

    b. For skewed straight steel I-girder units not meeting the LGA requirements of ***SDG*** 5.13.1.

    c. For horizontally curved steel I-girder units per ***SDG*** 5.14.

    d. For determining live load distribution factors for beam or girder supported superstructures where the beams or girders are not parallel, or approximately parallel, to the direction of traffic on the bridge, e.g. bridges used in conjunction with braided ramps.

    e. For all other superstructures not meeting ***LRFD*** 4.6.2.

B. Delete the third paragraph of ***LRFD*** 4.6.3.1 and add the following:

Show the name, version, and version release date of the software used for the superstructure design on the General Notes and on the FDOT Load Rating Summary Tables. State whether the live load distribution factors were determined by ***LRFD*** 4.6.2, RMA, or RMA 3D-FEA.

Attachment 'B'

SDG 2.9 – Live Load Distribution Factors

2.9 LIVE LOAD DISTRIBUTION FACTORS (LRFD 4.6.2.2)

A. For bridge superstructures with concrete beams meeting the requirements of ***LRFD*** 4.6.2.2, live load distribution factors (LLDF) shall not be less than the values given by the approximate methods. For bridge superstructures with straight steel I-girders, see ***SDG*** 5.13.

B. In ***LRFD*** 4.6.2.2.2, extend the Range of Applicability as follows:

1. ***LRFD*** Table 4.6.2.2.2b-1:

   a. For Florida-U beam bridges (Type "c" cross-section) change the depth parameter range to 18 < d <72, and the span length parameter range to 20 < L < 170.

   b. For prestressed concrete slab beam bridges (Type "f" cross-section) change the width parameter to 29 < b < 60, and the number of beams parameter range to $4 \le N_b \le 20$.

   c. For prestressed concrete I-beam bridges (Type "k" cross-section) change the longitudinal stiffness parameter range to 10,000 < Kg < 8,500,000.

2. ***LRFD*** Table 4.6.2.2.3a-1:

   a. For Florida-U beam bridges (Type "c" cross-section) change the depth parameter range to 18 < d < 72, and the span length parameter range to 20 < L < 170.

   b. For prestressed concrete slab beam bridges (Type "f" cross-section) change the number of beams parameter range to $4 \le N_b \le 20$, the width parameter to 29 < b < 60, and the moment of inertia range to 5,700 < I < 610,000.

3. ***LRFD*** Table 4.6.2.2.3b-1: for prestressed concrete slab beam bridges (Type "f" cross-section) change the width parameter to 29 < b < 60.

4. ***LRFD*** Table 4.6.2.2.3c-1:

   a. For Florida-U beam bridges (Type "c" cross-section) change the depth parameter range to 18 < d < 72, and the span length parameter range to 20 < L < 170.

   b. For prestressed concrete slab beam bridges (Type "f" cross-section) change the depth parameter range to 12 < d < 60, the width parameter to 29 < b < 60, and the number of beams parameter range to $4 \le N_b \le 20$.

*Commentary: The **LRFD** distribution factor equations are largely based on work conducted in NCHRP Project 12-26. When one or more of the parameters are outside the listed range of applicability, the equation could still remain valid, particularly when the value(s) is (are) only slightly outside the range. The extended values given in the **SDG** which are considered slightly outside of the **LRFD** range of applicability allow the use of the FDOT standard prestressed concrete beams. If one or more of the parameters greatly exceed the range of applicability, engineering judgment needs to be exercised.*

C. When widening existing AASHTO and Florida Bulb-T beam bridges with Florida-I Beams, the live load distribution factors may be calculated using the ***LRFD*** 4.6.2.2 approximate method.

*Commentary: The **LRFD** approximate method produces distribution factors that are conservative when compared to refined analyses even though the beam stiffnesses and spacings vary significantly.*

Attachment 'C'

SDG 2.12.3 – Fatigue

2.12.3 Fatigue (LRFD 3.6.1.4.2)

When designing for the Fatigue II limit state, use the following procedure to calculate the $ADTT_{SL}$ (***LRFD*** 3.6.1.4.2):

A. Calculate the AADT Growth Rate, GR, using the opening year and design year traffic data per ***SDM*** 7.2.D.

$GR = (AADT_{20}/AADT_{0})^{1/20}$

Where : $AADT_0$ = opening year

$AADT_{20}$ = design year (+20)

Use AADT if available, if not use ADT.

B. Calculate the estimated AADT at the mid-service life of the bridge, $AADT_{38}$.

$AADT_{38}=AADT_0 \cdot GR^{38} \cdot D$

Where: D = directionality factor. Use D=1 for bridges supporting one lane of traffic in both directions. This accounts for a future condition where the bridge is a one directional two-lane bridge.

The $AADT_{38}$ per lane is limited to 20,000 ($AADT_{38}$ / number of lanes ≤ 20,000).

*Commentary: The design year traffic data is typically 20 years from the opening date and is included in the roadway Typical Section package. The +20-year traffic data uses traffic forecasting procedures as referenced in **FDM** 120. Due to the number of uncertainties in traffic forecasting for a bridge service life of 75 years, the procedure described here uses an approximate growth factor using the opening and +20-year traffic data and extrapolates out to 38 years which is at the middle of the bridge service life. The $AADT_{38}$ is assumed to be an average AADT over the 75-year service life of the bridge and is used for the Fatigue II limit state. If another design year and/or bridge service life is used, adjust the GR and mid-life AADT accordingly.*

C. Calculate the $ADTT_{SL}$.

$ADTT_{SL}=AADT_{38} \cdot T \cdot p$

Where: T = percentage of truck traffic (round to next whole number; from Traffic Data, see ***SDM*** 7.2.D).

p = fraction of truck traffic in a single lane (***LRFD*** Table 3.6.1.4.2-1)

D. For the following roadways with a Limited Access functional classification (Interstate, Expressway, and Freeway), the $ADTT_{SL}$ needs to be equal to or greater than:

- Ramps (1 lane): 1700,
- Ramps (2 or more lanes): 3100,
- Mainlines and Collector-Distributor (C-D): 3100.

Commentary: It is anticipated that many cross-frames connections will be designed for the Fatigue II Limit State.

E. For cross-frames use a value of "n", as defined in ***LRFD*** 6.6.1.2.5, equal to 1.0.

Attachment 'D'

SDG 5.3.2 – Fracture

5.3.2 Fracture (LRFD 6.6.2)

A. Replace rows 10 and 11 in ***LRFD*** Table 6.6.2.1-1 as shown below. Members in row 11 are exempt from Charpy V-notch (CVN) testing. See ***SDM*** 2.14 for the definition of skew angle.

Diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in straight I-girder units that are designated as Case 1, 2, or 3 (see ***SDG*** 5.13.1), and in horizontally curved I-girder bridges satisfying all the conditions specified in Article 4.6.1.2.4b (for neglecting the effects of curvature)	Secondary
Diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in straight I-girder units that are designated as Case 4 or 5 (see ***SDG*** 5.13.1), and in horizontally curved I-girder bridges not satisfying one or more of the conditions specified in Article 4.6.1.2.4b (for neglecting the effects of curvature)	Primary

B. Add the following rows to ***LRFD*** Table 6.6.2.1-1 immediately after row 11:

For composite box-girder bridges: • Intermediate internal cross-frame members and their mechanically fastened or welded gusset plates. • Intermediate internal diaphragms that are not provided for continuity. • Except as specified herein, intermediate external diaphragms or cross-frame members and mechanically fastened or welded intermediate external cross-frame gusset plates. • Internal support diaphragms in straight bridges without skewed supports or in horizontally-curved bridges satisfying all the conditions specified in Article 4.6.1.2.4c for neglecting the effects of curvature.	Secondary
For composite box-girder bridges: • Intermediate internal diaphragms that are provided for continuity and their associated intermediate external diaphragms. • External support diaphragms or cross-frame members and mechanically fastened or welded external support cross-frame gusset plates. • Internal support diaphragms in straight bridges with skewed supports or in horizontally-curved bridges not satisfying one or more of the conditions specified in Article 4.6.1.2.4c for neglecting the effects of curvature. • Intermediate external diaphragms provided in bridges with concrete decks designed using the empirical design method to satisfy the design conditions specified in Article 9.7.2.4. • Intermediate external diaphragms provided in accordance with SDG 5.6.3.D.	Primary

C. Fracture critical members are defined as tension members or tension components of nonredundant members whose failure would result in the collapse of the structure. Examples include:

1. All tension components of two I-girder superstructures.
2. All tension components in the positive moment region of two box superstructures. Negative moment regions over the piers have four top flanges and are therefore considered redundant.
3. All tension components of straddle and integral piers.

D. Avoid fracture critical members. Fracture critical requirements are expensive due to the intensive welding procedures, base metal and weld tests, and inspections after fabrication. Two I-girder systems on non-movable structures are undesirable and must be approved by the State Structures Design Engineer.

E. Delete the 2nd paragraph of ***LRFD*** 6.6.2.2 related to System Redundant Members.

Commentary: The identification and use of System Redundant Members are not allowed by FDOT.

F. Designate on the plans, all:

1. Primary (main) members. Also, identify areas of primary members that are subject to tension and stress reversal, and designate that CVN testing is required.

*Commentary: Primary members have additional fabrication and inspection criteria per Section 460 of the **Specifications** and AASHTO/AWS D1.5 Bridge Welding Code which includes but not limited to material traceability, material testing, bolt hole fabrication, welding procedures, and inspection.*

2. Fracture critical components. In addition, for steel girders that are composite along any part of its length, and whose webs are subject to tension, the entire web depth shall be designated a Fracture Critical Member (FCM). Do not show tension zone limits for the web depth.
3. Splice plate testing requirements. Splice plates are to be tested to the requirements of the tension components to which they are attached.

Attachment ‘E’

SDG 5.13 – Straight Steel I-Girder Units

5.13 STRAIGHT STEEL I-GIRDER UNITS

5.13.1 Design Cases

A. Design straight steel I-girder (SSI-G) units in accordance with the criteria in Table 5.13.1-1. Configure steel units to meet either Case 1 or 2. If Case 1 or 2 cannot be met due to roadway geometry constraints and limitations, meet the lowest case number possible shown in the Table. See ***SDM*** 2.14 and ***LRFD*** 4.6.3.3.2 for definitions of skew angle, θ, and skew index, I_s. See ***SDG*** 5.3.2 for the designation of primary and secondary members for cross-frames.

Commentary: It is preferrable that a steel unit is configured to meet either Case 1 or 2. These cases allow for a simpler analysis method (LGA) for design and load rating which decreases design costs and the potential for design errors. It also improves economy by designating cross-frames as secondary members with bearing connections. Although roadway geometry may dictate a higher case number, the EOR should attempt to meet the lowest case number possible. For further information regarding the development of Case 1 and 2 criteria, see FDOT Research Projects BE535 and BEB13. Criteria was developed by FDOT and the referenced research reports.

B. For phase construction, analyze and design each phase separately as Case 1 or Case 2 as applicable when the following criteria is met:

1. All phases meet either Case 1 or 2 criteria per Table 5.13.1-1.
2. The skew index for the entire unit at final condition (entire cross-section) is less than or equal to 0.45.
3. The construction sequence follows the requirements of ***SDG*** 4.2.11 and ***SDG*** 5.1.B.

Otherwise, use the skew index for the final condition (entire cross-section) of the unit to select Case 3, 4, or 5 per Table 5.13.1-1.

C. Steel framing systems that have the following configurations or elements, but not limited to: haunched girders, non-continuous longitudinal girders (i.e., bifurcated), or substringers are to be designated as either Case 4 or 5.

Table 5.13.1-1 Design Criteria for Straight Steel I-Girder Units

Case	SDG Section	Skew Angle (θ)	Skew Index (I_s)	Cross-Frame Configuration	Required Method(s) of Analysis[3]	Cross-Frame Connection Type	Calculate Cross-Frame Rating Factor
1[1]	5.13.2 & 5.13.3	$\theta \leq 20°$	$I_s \leq 0.45$	contiguous[2] and parallel to skew angle	LGA [***LRFD*** 4.6.2.2]	Bearing	No
2[1]	5.13.2 & 5.13.4	$20° < \theta \leq 50°$	$I_s \leq 0.3$	contiguous[2] and normal to girders	LGA [***LRFD*** 4.6.2.2]	Bearing	No
3	5.13.5	$\theta \leq 50°$	$I_s \leq 0.45$	Any[5]	LGA and RMA [***LRFD*** 4.6.2.2 & 4.6.3]	Bearing	No
4	5.13.6	$\theta \leq 50°$	$I_s \leq 0.6$	Any[5]	RMA [***LRFD*** 4.6.3]	Slip-critical	No[6]
5	5.13.7	$\theta \leq 60°$[4]	Any	Any[5]	RMA 3D-FEA [***LRFD*** 4.6.3]	Slip-critical	Yes

1. Staggered cross-frame arrangements (similar to the term "discontinuous" as defined in ***LRFD*** 6.2) are not permitted.
2. As defined in ***LRFD*** 6.2. This configuration may consist of removed nuisance cross-frames from a contiguous line in the vicinity of a support as discussed in ***LRFD*** C6.7.4.2. There must be an end cross-frame line located along each skewed support.
3. LGA refers to a line girder analysis defined as analyzing an individual straight girder from the rest of the superstructure system using classical force and displacement methods or finite element model in which the live load forces are determined using the LLDF defined in ***LRFD*** 4.6.2.2. RMA refers to a refined method of analysis as defined in ***LRFD*** 4.6.3.
4. See ***SDG*** 1.10 for skew angle limitation.
5. See ***SDG*** 5.7.B.
6. Cross-frame(s) (or any structural element) can be load rated at the discretion of the EOR.

5.13.2 Design Criteria for Case 1 and 2

This section covers the criteria for both Case 1 and 2. For additional requirements for Case 1 or Case 2, see ***SDG*** 5.13.3 and ***SDG*** 5.13.4, respectively.

A. Meet the supplemental conditions to ***LRFD*** 4.6.2.2 as shown in Table 5.13.2-1.

Table 5.13.2-1 Supplemental Conditions to *LRFD* 4.6.2.2

Supplemental Conditions[1]	**LRFD 4.6.2.2 Conditions**
1. Width of deck can vary up to 5 degrees[2]	Width of deck is constant
2. Girder spacing can be non-parallel up to 5 degrees[2,3]	Girders are parallel
3. The beam spacing must meet the range of applicability	For beam spacing exceeding the range of applicability as specified in tables in Articles 4.6.2.2.2 and 4.6.2.2.3, the live load on each beam is based on the lever rule.
4. $10{,}000 \le K_g \le 10{,}000{,}000$	$10{,}000 \le K_g \le 7{,}000{,}000$
5. Difference between skew angles of two adjacent supports does not exceed 10 degrees	Not explicitly addressed
6. $d_e / S \le 0.35$	Not explicitly addressed
7. $0.95 \le S/D_w \le 2.00$	Not addressed
8. RDDP < 175 (see ***SDG*** 5.13.2.K)	Not addressed

1. Consider software limitations and methodologies when using Table 5.13.2-1 Supplemental Conditions to comply with these design criteria.
2. The angle is measured between the centerline of bridge (or a line parallel to) and the edge of deck or girder line. If each side of the bridge width varies, the two angles are to be summed and then compared to the limit.
3. Calculate LLDF using the girder spacing at the 2/3 point along the span toward the wider end.

Commentary: Considering that the general geometry for SSI-G units consist of girder spacings between 9-feet and 12-feet and spans between 150-feet and 240-feet, an optimized d_e/S ratio is in the range of 0.10 to 0.20.

B. Distribute all non-composite dead loads equally (i.e., uniformly) to all girders.

C. For girders with variable moment of inertia, compute a single weighted average value for K_g that is used for the entire girder to calculate the moment live load distribution factor (LLDF) in ***LRFD*** Table 4.6.2.2.2b-1.

Commentary: For a continuous girder, K_g may be calculated at evenly spaced points along each span, and then an average K_g value calculated for each span. The single weighted average value of K_g is calculated by multiplying each span K_g value *by its corresponding span length, summing the resulting values for each span together, and then dividing by the total girder length. An alternate weighted average method is using the length of each girder section, between shop or field splices, multiplied by its K_g value and then summing the resulting values and dividing by the total girder length.*

D. For continuous girders, calculate the moment LLDF for each span using the single weighted average value of K_g. Then use the maximum value for the LGA.

E. For continuous girders, calculate the shear LLDF for each span and then use the maximum value for the LGA.

F. Apply the skew correction factor per ***LRFD*** 4.6.2.2.3c including the LLDF determined by the lever rule (***LRFD*** C4.6.2.2.1) and the rigid cross-section method (***LRFD*** 4.6.2.2.2d).

G. The skew correction factor for continuous spans is calculated for each span using the single weighted average value of K_g, and the largest skew correction factor is applied to the shear LLDF. The skew correction factor needs to be applied to each side of an interior support.

H. Do not use the reduction of LLDFs for moment in ***LRFD*** 4.6.2.2.2e.

Commentary: The method described for calculating the moment and shear LLDF (using a single weighted average value for K_g) was the basis for using an LGA (i.e., the AISC/NSBA LRFD Simon Version 10.4.0.0 software was used for LGA in the FDOT Research Project BEB13).

I. If using the optional live load deflection criteria (see ***SDG*** 1.2), calculate the girder deflection due to live load using the moment LLDF per ***LRFD*** 4.6.2.2.

J. All girders in the unit must be the same. Varying girder plate lengths (i.e., field splice locations) or cross-frames spacings is permitted to avoid conflicts or due to non-parallel girder spacings and/or supports (contact a local fabricator for guidance).

Commentary: The criterion for keeping all the girders in the unit the same is a basic assumption in the development of the recommendations for Case 1 and 2 used in the FDOT Research Project BEB13.

K. Calculate the rigid differential deflection parameter, RDDP, as follows:

$$RDDP = DDP \cdot \cos\theta \cdot \left(\frac{L_{eff}}{100}\right) \quad \text{[dimensionless]} \qquad \text{[Eq. 5-1]}$$

Where:

$$DDP = x\left(\frac{L^3 - 2Lx^2 + x^3}{K_g}\right) \quad \text{[dimensionless]} \qquad \text{[Eq. 5-2]}$$

$$x = w_g \cdot \tan\theta \quad \text{[ft]} \qquad \text{[Eq. 5-3]}$$

L = maximum span length in the unit [ft]

K_g = longitudinal stiffness parameter per ***LRFD*** 4.6.2.2.1 [in^4].

L_{eff} = effective span length [ft]. For simple spans, the effective span length is the span length. For continuous spans, the effective span length is equal to the distance between a simple support and a point of non-composite dead load contraflexure, or between points of non-composite dead load contraflexure. For a 2-span unit, use the maximum effective span length. For three or more continuous spans, use the largest of: 1) effective span length of the maximum interior span, 2) 50% of the maximum interior span length or 3) 65% of the effective length of the maximum end span.

Commentary: The results from the FDOT Research Project BEB13 showed that bridges with similar skew indexes had significantly different cross-frame loads and that the proposed recommendations resulted in highly conservative design forces for many of the bridges. FDOT re-evaluated the data and developed the rigid differential deflection parameter, RDDP, as an additional indicator of skew effects. Using this parameter in conjunction with the criteria for Case 2, reveals that bridges with similar skew indexes could have significantly different RDDP values. It was found that bridges with higher RDDP values also had higher bottom chord cross-frame forces.

L. When calculating the exterior girder fatigue moment range, use the procedure below in lieu of the LLDF in ***LRFD*** 4.6.2.2 when the steel unit meets all of the following conditions:

- Applies only to Case 1 continuous spans and Case 2 units,
- Has 4 or more girders,
- All spans in the unit have lengths greater than or equal to 150-feet.,
- The girder spacing is greater than or equal to 9-feet.,
- The d_e/S ratio is less than or equal to 0.26.

1. Calculate the LLDF ($g_{interior}$) per ***LRFD*** Table 4.6.2.2.2b-1 using the "One Design Lane Loaded" equation (do not divide by 1.2 per ***LRFD*** 3.6.1.1.2).
2. The term e_M used below is synonymous with 'e' per ***LRFD*** Table 4.6.2.2.2d-1 with the value not to exceed 1.0.
3. Calculate the fatigue moments (maximum and minimum) using the following equation for the LLDF in the LGA:

$$LLDF_{mfatEXT}=1.2\cdot e_M\cdot g_{interior} \qquad \text{[Eq. 5-4]}$$

4. Calculate the fatigue moment range by taking the difference between the maximum and minimum fatigue moments.
5. In addition, use the following modifications to the fatigue moment range:
    a. For continuous steel units (see Figure 5.13.2-1).

i. Zone 1: In the vicinity of interior supports, use the ***LRFD*** LLDF (lever rule or rigid cross-section analysis) at the location of the support. Transition from the ***LRFD*** LLDF to the $LLDF_{mfatEXT}$ from the point of support to a distance of 10% of the span length. This applies to each side of the support.

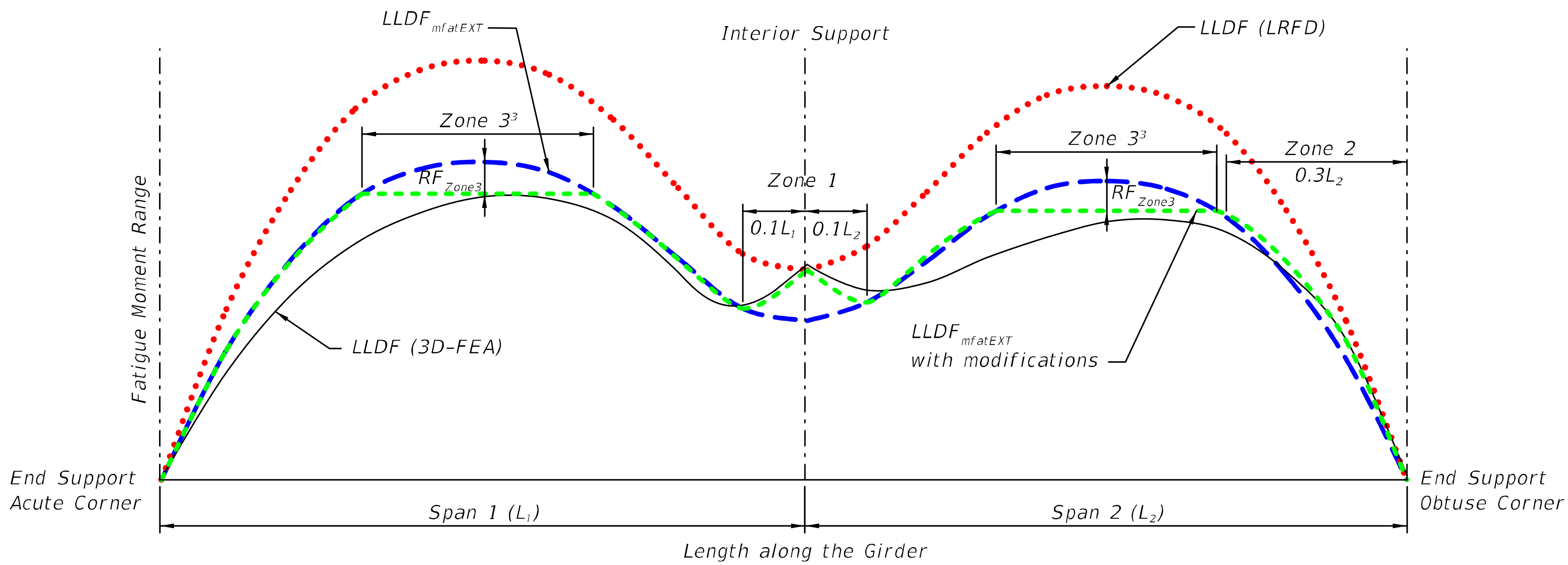
ii. Zone 2: Increase the fatigue moment range by the skew correction factor, SCF_{mfat}, per Table 5.13.2-2, where applicable.

iii. Zone 3: Case 2 only. For areas where the bottom flange is in tension, reduce the maximum value by the reduction factor, RF_{Zone3} (as a "%"). Use the reduced value as shown in Figure 5.13.2-1.

$$RF_{Zone3} = 0.09 * RDDP \qquad \text{[Eq. 5-5]}$$

b. For Case 2 single span steel units, increase the fatigue moment range by the skew correction factor, SCF_{mfat}, per Table 5.13.2-2, where applicable.

Figure 5.13.2-1 Fatigue Moment Range Modifications



Notes:
1. The lines shown represent the fatigue moment range values based on the LLDF determined by the stated method.
2. Case 2 Two span unit shown, three or more spans similar.
3. Limits of Zone 3 determined by horizontal line extension and intersection with fatigue moment range line.

Table 5.13.2-2 Fatigue Moment Range Skew Correction Factor, SCF_{mfat}

	Simple Spans[1,2]	Continuous Spans[1,2]
Length of Applicability	$0.4L_{span}$	$0.3L_{span}$
Case 1	Not Applicable	1.1
Case 2	$1.33(1 + 0.06 * RDDP^{0.3}) – 0.33 \leq 1.35$	$1.5(1 + 0.03 * RDDP^{0.4}) – 0.5 \leq 1.35$

1. Only applicable at end supports at the obtuse corner of the span(s).
2. Decrease the SCF_{mfat} value linearly to a value of 1 at the end of the length of applicability.

*Commentary: The **LRFD** LLDF for determining the fatigue truck maximum and minimum moments (i.e., moment range) for the exterior girder is based on using the lever rule (**LRFD** Table 4.6.2.2.2d-1) or the rigid cross-section analysis (**LRFD** 4.6.2.2.2d). FDOT Research Project BEB13 demonstrated that these formulas are significantly conservative when compared to values determined by a 3D-FEA, and in many cases, Fatigue I can govern the design of the girder. The above procedure is based on using a single LLDF ($LLDF_{mfatEXT}$) that is approximately one standard deviation above the average positive fatigue moment LLDFs determined using a 3D-FEA. Furthermore, the modifications are for specific areas where the single LLDF ($LLDF_{mfatEXT}$) is unconservative or conservative when compared to the results of a 3D-FEA. At end support locations of Case 2 units, the sum of the longitudinal girder fatigue moment stress range and the flange lateral bending fatigue stress range may necessitate a bolted tab plate (see **SDG** 5.8.A and **SDM** 16.6.F) if the cost is justified. If a bolted tab plate is used, it should only be detailed for specific locations and not at all connection plates along the girder length.*

M. For intermediate cross-frames, use only load combinations for Strength I, Fatigue I or II, and Constructability. Use "K" frames for intermediate cross-frames as shown in ***SDM*** Figure 16.7-1. Evenly space cross-frames, with a spacing not to exceed 30-feet.

N. For end cross-frames located at the end of a unit below a free edge of the concrete deck (see ***SDM*** Figure 16.7-1 End Cross-frame) and at interior supports located below a continuous concrete deck, use only load combinations for Strength I, Strength III, Fatigue I or II, and Constructability.

O. For skewed steel units, the girder will layover (twist) at the end support due to geometric compatibility with the intermediate and end cross-frames. The girder layover (the relative horizontal movement of the top flange to the bottom flange) due to the non-composite dead loads (i.e., self-weight of the steel girder and the concrete deck including the SIP forms) can be calculated using the following procedure:

1. Determine the girder vertical deflection at the 1/10th point, $\Delta_{0.1Ls}$ (ft), (usually given in the LGA) along the span under consideration from the end support.
2. Calculate the girder major-axis bending rotation, α (radians), at the end support by the following equation:

   $\alpha = \Delta_{0.1Ls} / (0.1 * L_s)$ [Eq. 5-6]

   where: L_s = the span length under consideration (ft)
3. Calculate the girder major-axis bending twist, φ (radians), at the end support by the following equation:

   $\varphi = (\alpha)(\tan\theta)$ [Eq. 5-7]
4. Calculate the girder layover (inch) at the end support by the following equation:

   Layover = (D)(φ) [Eq. 5-8]

   where: D = depth of the girder (inch).

*Commentary: The girder twist needs to be evaluated in the joint and bearing design per **LRFD** 14.4.1. Refer to **LRFD** 6.7.2 for discussion regarding girder layovers.*

5.13.3 Additional Criteria for Case 1

A. The girder flange lateral bending stresses due to skew effects are taken equal to zero.

B. The forces to design the intermediate and end cross-frame members and connections are shown in Table 5.13.3-1.

Table 5.13.3-1 Cross-Frame Component Forces

Load Case	Top Chord	Diagonals	Bottom Chord
Intermediate Cross-Frames			
Strength I[1] (kips)	40	60	100
Fatigue Range[2] (kips)	10	10	15
Constructibility[3] (kips)	15	10	15
End Cross-Frames			
Strength I[1,4] (kips)	40	35	40
Fatigue Range[2] (kips)	10	10	10
Constructibility[3,4] (kips)	5	5	5

1. The forces shown for Strength I are already factored.
2. Apply load factors for Fatigue I or II loading combinations as applicable. In addition, apply a factor of 0.65.
3. The values are for the unfactored force induced into the cross-frames due to the non-composite weight of the concrete deck and SIP forms. Apply other force effects as applicable.
4. For end cross-frames directly supporting a free edge of the concrete deck, these forces do not include effects of the concrete edge beam weight and a local wheel load.

5.13.4 Additional Criteria for Case 2

A. For exterior girders use the multiplicative factors shown in Table 5.13.4-1. The multiplicative factors are in addition to the ***LRFD*** skew correction factor. The multiplicative factors for the Fatigue Live Load Vertical Shear Force Range are applied at the support and the value decreased linearly to 1.0 at the mid-span of the exterior girder.

Table 5.13.4-1 Factors Applicable to Exterior Girder

	Simple Span	Continuous Spans
Fatigue Live Load Vertical Shear Force Range	1.2	1.3[1]
Absolute Maximum Fatigue Live Load Vertical Shear Force	1.0	
Bearing Reactions	1.0	1.15/1.0 (Uplift)[1]
1. Apply to the obtuse corner of an interior support and at the obtuse corner of an end span support.		

Commentary: The factor for the Absolute Maximum Fatigue Live Load Vertical Shear Force (i.e., used for designing the web per ***LRFD*** *6.10.5.3) is 1.0 because it is reasonably predicted by an LGA. However, because of the lack of conservatism in the LGA analysis for predicting the minimum fatigue live load vertical shear force, it is necessary to augment the Fatigue Live Load Vertical Shear Force Range (i.e., used for designing the shear connectors per* ***LRFD*** *6.10.10) by the specified factors.*

B. Use the girder flange lateral bending stresses due to skew effects as shown in Table 5.13.4-2.

Table 5.13.4-2 Girder Flange Lateral Bending Stresses Due to Skew Effects

Girder	Location[1]	Unfactored f_L[2] (ksi)	Fatigue Range f_L[3] (ksi)	Constructability f_L[5] (ksi)
Exterior	Near Support at the obtuse corner for Simple Spans	7.5	3.5	1
	Near End Supports at the obtuse corner for Continuous Units	7.5 ≤ [2.5+(RDDP/135)*10] ≤ 12	5.5	3
	Near Interior Supports of Continuous Units	4.5	N/A[4]	3
	Within Span	0	0	0
Interior	Near Supports for Simple Spans	7.5	2.5	1
	Near End Supports for Continuous Units	10	4.5	2
	Near Interior Supports of Continuous Units	7.5	N/A[4]	2
	Within Span	0	0	0

1. Near is defined as the first two cross-frame lines adjacent to the support. Transition the flange lateral bending stress from the value in the table at the first cross-frame location to a value of zero at the second cross-frame location.
2. Apply a 1.6 or 1.2 load factor for the Strength I and Service II loading combination, respectively. These are weighted average load factors (i.e., includes dead and live loads) recommended in the FDOT Research Project BEB13.
3. Apply load factors for Fatigue I or II loading combinations as applicable. In addition, apply a factor of 0.65.
4. Does not need to be considered for the top flange due to the contribution of the concrete deck; or the bottom flange since the flange lateral bending stress range is small and the flange is generally in compression.
5. The values are for the unfactored flange lateral bending stress induced into the girder due to the weight of the concrete deck and SIP forms.

C. The forces to design the intermediate and end cross-frames members and connections are shown in Table 5.13.4-3.

Table 5.13.4-3 Cross-Frame Component Forces

Load Case	Top Chord	Diagonals	Bottom Chord
Intermediate Cross-Frames Simple Spans			
Strength I[1] (kips)	40	70	1.3*RDDP + 85*(S/D_w) – 115 ≥ 100
Fatigue Range[2] (kips)	10	12	0.10*RDDP + 3*(S/D_w) + 5 ≥ 15
Constructibility[3] (kips)	15	10	20
Intermediate Cross-Frames Continuous Spans			
Strength I[1] (kips)	50	0.35*RDDP + 20*(S/D_w) +20 ≥ 70	1.3*RDDP + 85*(S/D_w) – 90 ≥ 100
Fatigue Range[2] (kips)	10	0.05*RDDP +11	0.20*RDDP + 8*(S/D_w) - 2 ≥ 15
Constructibility[3] (kips)	40	20	40
End Cross-Frames			
Strength I[1,4] (kips)	100	100	75
Fatigue Range[2] (kips)	10	15	15
Constructibility[3,4] (kips)	10	10	10

1. The forces shown for Strength I are already factored.
2. Apply load factors for Fatigue I or II loading combinations as applicable. In addition, apply a factor of 0.65.
3. The values are for the unfactored force induced into the cross-frames due to the weight of the concrete deck and SIP forms. Apply other forces effects as applicable.
4. For end cross-frames directly supporting a free edge of the concrete deck, these forces do not include effects of the concrete edge beam weight and a local wheel load.

5.13.5 Design Criteria for Case 3

A. Design the girders using a LGA with the LLDF per ***LRFD*** 4.6.2.2 using the criteria in ***SDG*** 5.13.2 A through J. The supplemental conditions 5, 6, 7, and 8 of Table 5.13.2-1 do not apply for Case 3 bridges.

B. Use an RMA to determine fatigue stress ranges, bearing reactions, flange lateral bending stresses, girder cambers, and cross-frame forces.

C. For design of the intermediate cross-frames, use only Load Combinations for Strength I, Fatigue I or II, and Constructability.

D. For end cross-frames located at the end of a unit below a free edge of the concrete deck and at interior supports located below a continuous concrete deck, use only load combinations for Strength I, Strength III, Fatigue I or II, and Constructability.

E. If the requirements of this Section cannot be met, use Case 4 criteria.

Commentary: Case 3 applies to steel I-girder units where the structural systems do not meet Case 1 or 2 criteria, but the girder is designed using a LGA to ensure its capacity is still sufficient in the final condition without relying on the cross-frames to transfer gravity loads. This allows the cross-frames to be designated as secondary members with bearing connections. The force effects stated in 5.13.5.B could not be estimated accurately using either upper value bounds or empirical formulas; therefore, an RMA is required to determine these force effects.

5.13.6 Design Criteria for Case 4

Use a Refined Method of Analysis (***LRFD*** 4.6.3) for Case 4 as defined in Table 5.13.1-1.

5.13.7 Design Criteria for Case 5

Use a 3D-FEA Refined Method of Analysis (***LRFD*** 4.6.3) for Case 5 as defined in Table 5.13.1-1. The 3D-FEA analysis requirements include, but are not limited, to the following:

A. Model the superstructure fully in three dimensions.

B. Model the girder flanges using beam, plate, shell, or solid elements.

C. Model the girder webs using plate, shell, or solid elements.

D. Model the intermediate and end cross-frames components using beam, truss, or plate elements.

E. Model the deck using plate, shell, or solid elements.

Attachment 'F'

SDG 5.14 – Horizontally Curved Steel I-Girder Units

5.14 HORIZONTALLY CURVED STEEL I-GIRDER UNITS

Design horizontally curved steel I-girder units in accordance with ***LRFD*** and the following requirements:

A. For horizontally-curved I-girder units satisfying all the conditions specified in ***LRFD*** 4.6.1.2.4b, design the cross-frame connections as bearing connections. Do not calculate a load rating factor for the intermediate or end cross-frames.

B. For horizontally-curved I-girder units not satisfying all the conditions specified in ***LRFD*** 4.6.1.2.4b, design the cross-frame connections as slip-critical connections. Do not calculate a load rating factor for the intermediate or end cross-frames except as noted in Number C.1 below.

C. Use a 3D-FEA (see ***SDG*** 5.13.5 for requirements) refined method of analysis (***LRFD*** 4.6.3) when:

   1. The ***LRFD*** bridge skew index is greater than 0.6 (see also ***SDG*** 1.10). Calculate a rating factor for the intermediate cross-frames and end cross-frames.

   2. The central angle (any span in the unit) is greater than 0.06 radians and the ***LRFD*** bridge skew index is greater than 0.4 but less than or equal to 0.6. Do not calculate a load rating factor for the intermediate or end cross-frames.

Attachment ‘G’

SDG 7.6 – Widening Rules

7.6 WIDENING RULES

7.6.1 General

A. The transverse reinforcement in the new deck should be spaced to match the existing spacing. Different bar sizes may be used if necessary.

B. Do not mix concrete and steel beams in the same span.

Commentary: This requirement follows the fundamental engineering principal that beams or girders within the same span must have stiffnesses and thermal properties that are the same or similar. Mixing concrete beams and steel girders (different stiffnesses and thermal properties) within the same span will cause detrimental bridge responses that include the following: potential overstress of existing beams (stiffer beams will attract more load); deck overstress due to differential live load induced vertical movement across the deck bay; differential longitudinal thermal movements between existing and new beams or girders due to different coefficients of thermal expansion.

C. Satisfy the vertical clearance requirements of ***FDM*** 260.

Modification for Non-Conventional Projects:
Delete ***SDG*** 7.6.1.C and insert the following: C. Satisfy the vertical clearance requirements of ***FDM*** 260 unless otherwise allowed by the RFP.

D. For widenings of overpass structures, contact the District Maintenance Office for a history of overheight vehicle impacts.

Modification for Non-Conventional Projects:
Delete ***SDG*** 7.6.1.D and see the RFP for requirements.

E. For all widenings, provide back-up documentation that the proposed widening design and details are based on actual field conditions determined by field survey for the following items, at a minimum:

1. Bridge location
2. Vertical and horizontal clearances
3. Stationing
4. Skew angles
5. Substructure locations and elevations
6. Span lengths
7. Deck finish grade elevations
8. Number and type of beams

9. Outside dimensions of all structural components
10. Bridge mounted utilities
11. Bridge mounted signs and support structures

Provide the back-up documentation preferably with the BDR but no later than the 60% Submittal. Notify the Department's Project Manager of any discrepancies that are critical to the continuation of the widening design.

Modification for Non-Conventional Projects:
Delete ***SDG*** 7.6.1.D and insert the following: E. For all widenings, provide back-up documentation with each 90% Component Plan Submittal confirming that the proposed widening design and details are based on actual field conditions determined by field survey for the following items, at a minimum: 1. Bridge location 2. Vertical and horizontal clearances 3. Stationing 4. Skew angles 5. Substructure location and elevations 6. Span lengths 7. Deck finish grade elevations 8. Number and type of beams 9. Outside dimensions of all structural components 10. Bridge mounted utilities 11. Bridge mounted signs and support structures

7.6.2 Concrete Beams

A. For widening AASHTO, Bulb-T, and cast-in-place concrete beam bridges, use Florida-I beams or AASHTO Type II beams. For widening existing AASHTO Type II Beam bridges, investigate the most economical option for using either AASHTO Type II Beams or FIB 36 Beams. For all other widenings, use the same superstructure type and depth where possible.

Commentary: The increased span and load carrying capacity of the Florida-I will generally allow designers to widen bridges using shallower beam depth than existing beams. For example the designer can use FIB 54 to widen an existing AASHTO type V bridge.

B. Coordinate the use of non-standard height prestressed concrete beams with the DSDE.

Commentary: So as to preserve the shape of the side forms used to construct standard beams, the standard beam heights should not be decreased by reducing the web, bottom or top flange heights, or increased by increasing the web or bottom flange heights. The top flange height can be increased or the entire top flange can be eliminated without changing the shape of the standard side forms.

C. Voided-slab bridges require special attention. Contact the DSDE for guidance. The DSDE will coordinate with the SDO to establish recommendations and criteria for the widening of the particular structure.

Modification for Non-Conventional Projects:
Delete ***SDG*** 7.6.2.C and see the RFP for requirements.

D. When widening with AASHTO Type II or Florida-I Beams, squaring beam ends, placing bearing pads orthogonally and eliminating permanent end diaphragms are the preferred options. However, skewed beam ends, skewed bearing pads and end diaphragms may be used at the discretion of the DSDE.

Modification for Non-Conventional Projects:
Delete ***SDG*** 7.6.2.D.

Commentary: Beams with squared off ends are easier to construct and less likely to spall off when the prestressing is released. Skewed ends may be necessary when matching the width of existing bents or pier caps.

E. Where the existing bridge uses end diaphragms and diaphragms are proposed for the widening, connect the new diaphragm to the existing diaphragm. Drill and epoxy rebar into the adjacent existing diaphragm. Do not drill into existing beams.

Commentary: Drilling into the existing beams at their ends is prohibited to prevent damaging the reinforcing steel and strand.

7.6.3 Steel I-Girders

A. Provide concrete closure pour in deck between new and existing structure.

B. Provide diaphragms and cross-frames between new and existing girders, spaced to line up with existing diaphragms and cross-frames.

C. Attach cross-frame connection stiffeners to existing girder webs and flanges by angles or bent plates. Field drill and bolt to existing girders.

D. Field welding to existing girder webs, tension flanges, and flanges subject to stress reversal is prohibited.

E. Field welding to the compression flanges of existing girders is allowed, with approval of the SSDE, but only if the compression flange is embedded in the concrete deck and bolted connections are not easily accommodated.

Modification for Non-Conventional Projects:
Delete ***SDG*** 7.6.3.E and see the RFP for requirements.

F. Field welding to the existing bearing stiffeners is allowed, with approval of the SSDE, but only if a bolted connection to the existing structure cannot be accommodated.

Modification for Non-Conventional Projects:
Delete ***SDG*** 7.6.3.F and see the RFP for requirements.

G. For widenings consisting of 2 or more girders, the end diaphragm located between the existing and proposed girders may consist of only a top strut, unless the design requires otherwise (e.g., stability during erection, transfer of lateral loads, etc.).

H. For major widenings where the existing cross-frame connection plates are not connected to the flanges, the existing connection plates shall be retrofitted by attaching to the flanges by angles or bent plates as per the above procedures.

I. For existing straight steel I-girder units, see Table 7.6.3-1 for criteria regarding the requirements for level of analysis, load rating, and strengthening of the existing cross-frames. Analyze the existing steel I-girder unit and analyze and design the widening separately as Case 1 or Case 2 as applicable when all the following conditions are met:

1. Both the existing and widening meet Case 1 or Case 2 criteria per Table 7.6.3-1.
2. The skew index for the entire unit (existing and proposed widening in the final condition) unit is less than 0.45.
3. The cross-frames are designed and detailed in accordance with ***SDG*** 5.1.B.

Otherwise, use the skew index for the entire unit (i.e., final widened configuration) to select Case 3, 4, or 5 per Table 5.13-1. See ***SDG*** 5.13 for additional information.

Table 7.6.3-1 Cross-Frame Load Rating and Strengthening Requirements

Case[1]	Calculate Cross-Frame RF[2]	Strengthen Existing Cross-Frames
1	No	No[3]
2	No	No[3]
3	No	No[3,4]
4	No	If the demand exceeds the load-carrying capacity and/or Footnote 3.
5	Yes	Strengthen if RF ≤ 1.00

1. See ***SDG*** Table 5.13-1 for cross-frame configuration and required level of analysis.
2. RF = Rating Factor
3. Strengthening of the existing cross-frames is not required unless otherwise determined in the Bridge Analysis during the PD&E phase.
4. Perform a LGA and RMA. If the RMA indicates overstress in the existing cross-frames, perform another RMA without including the existing intermediate cross-frames in the structural model. If the girders have a sufficient load rating for all analyses, then load rating and strengthening of the existing intermediate cross-frames is not required.

Modification for Non-Conventional Projects:
Delete ***SDG*** 7.6.3.I and Table 7.6.3-1 and see the RFP for requirements.