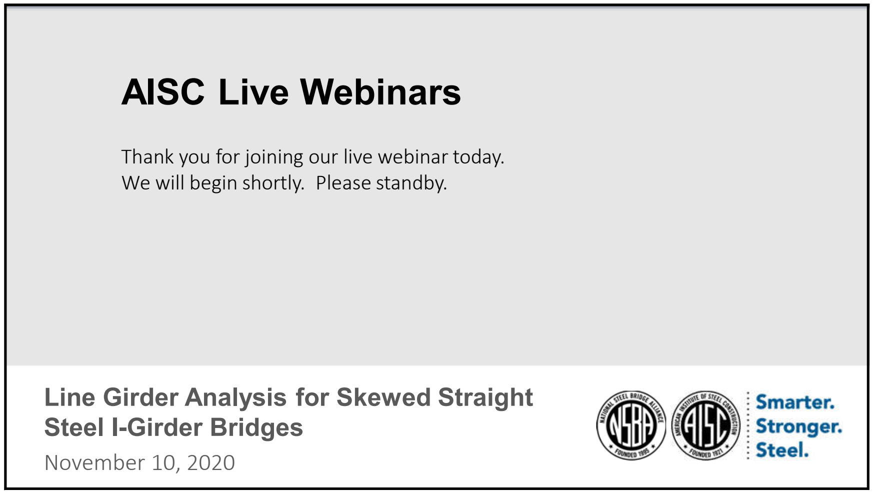

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Line Girder Analysis for Skewed Straight Steel I-Girder Bridges

November 10, 2020

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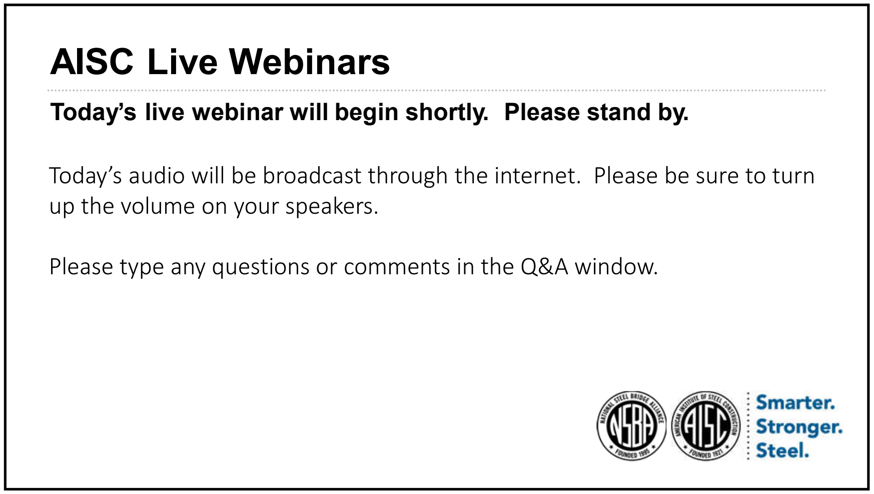

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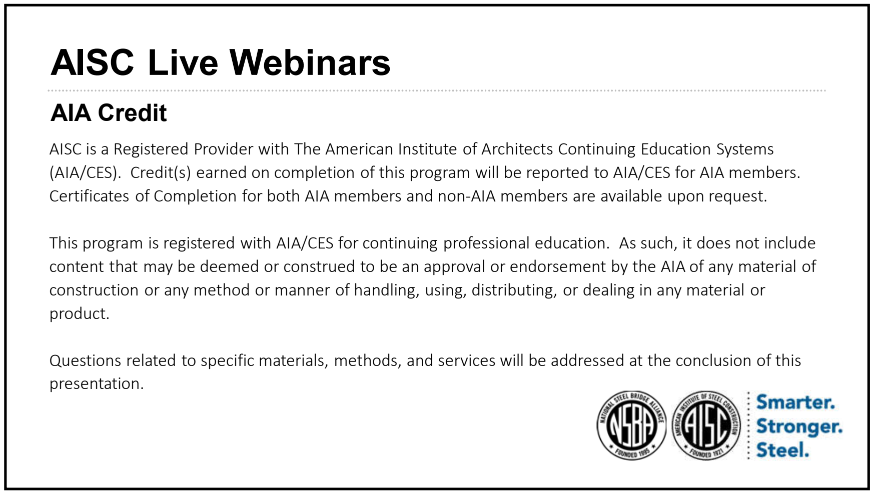

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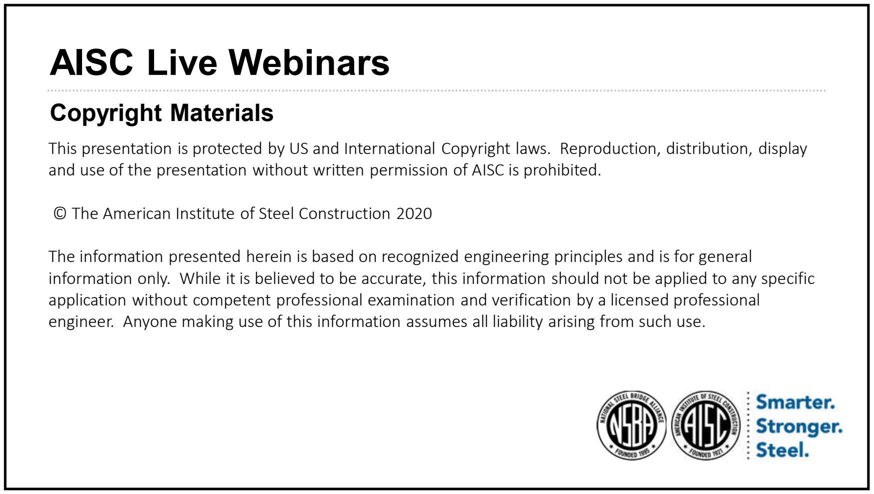

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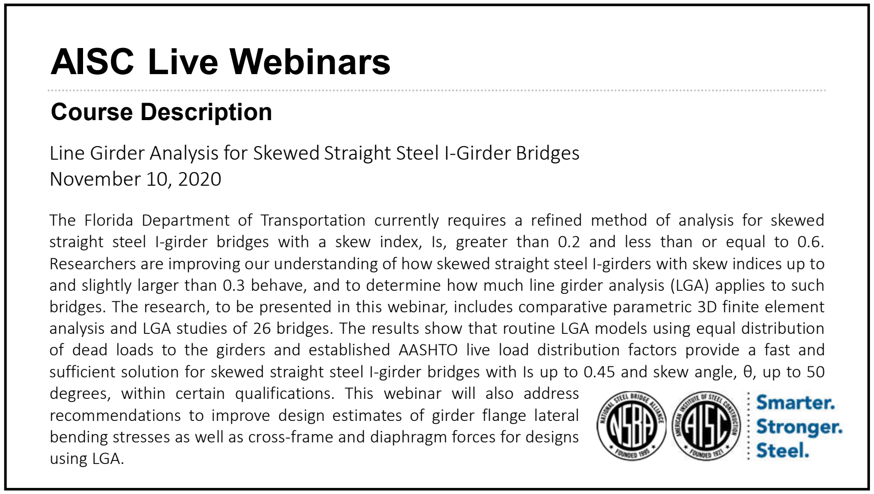

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Course Description

Line Girder Analysis for Skewed Straight Steel I-Girder Bridges
November 10, 2020

The Florida Department of Transportation currently requires a refined method of analysis for skewed straight steel I-girder bridges with a skew index, Is, greater than 0.2 and less than or equal to 0.6. Researchers are improving our understanding of how skewed straight steel I-girders with skew indices up to and slightly larger than 0.3 behave, and to determine how much line girder analysis (LGA) applies to such bridges. The research, to be presented in this webinar, includes comparative parametric 3D finite element analysis and LGA studies of 26 bridges. The results show that routine LGA models using equal distribution of dead loads to the girders and established AASHTO live load distribution factors provide a fast and sufficient solution for skewed straight steel I-girder bridges with Is up to 0.45 and skew angle, θ, up to 50 degrees, within certain qualifications. This webinar will also address recommendations to improve design estimates of girder flange lateral bending stresses as well as cross-frame and diaphragm forces for designs using LGA.

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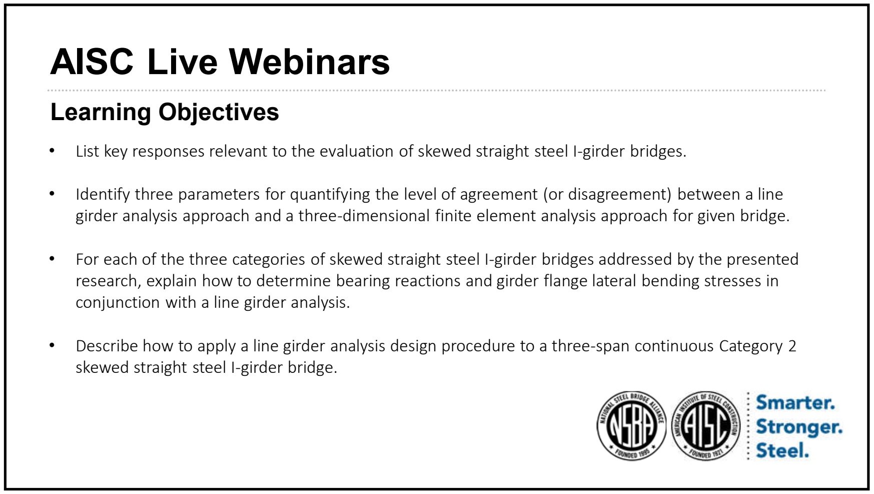

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Learning Objectives

- List key responses relevant to the evaluation of skewed straight steel I-girder bridges.
- Identify three parameters for quantifying the level of agreement (or disagreement) between a line girder analysis approach and a three-dimensional finite element analysis approach for given bridge.
- For each of the three categories of skewed straight steel I-girder bridges addressed by the presented research, explain how to determine bearing reactions and girder flange lateral bending stresses in conjunction with a line girder analysis.
- Describe how to apply a line girder analysis design procedure to a three-span continuous Category 2 skewed straight steel I-girder bridge.

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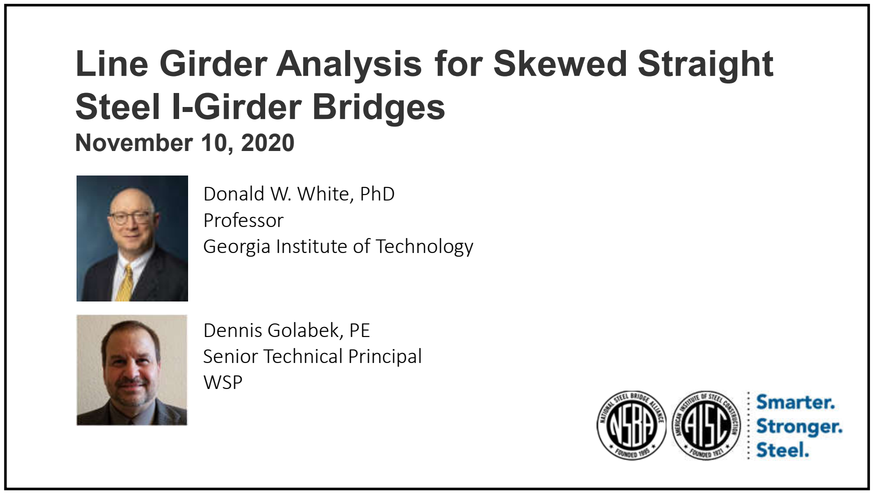

Line Girder Analysis for Skewed Straight Steel I-Girder Bridges

November 10, 2020

Donald W. White, PhD
Professor
Georgia Institute of Technology

Dennis Golabek, PE
Senior Technical Principal
WSP

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Part 1:
Overview of Recent Research

Donald W. White, PhD
Professor
Georgia Institute of Technology

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CREDITS

RESEARCH REPORT:

STRAIGHT STEEL I-GIRDER BRIDGES WITH SKEW INDEX APPROACHING 0.3, Final Report, FDOT Contract No. BE535, Nov. 2020, Florida Department of Transportation, Tallahassee, FL

RESEARCH TEAM:

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FDOT Project Manager:

Vickie Young

FDOT Steering Committee:

Christina Freeman, Ben Goldsberry and Dennis Golabek

FOCUS OF FDOT SUPPORTED RESEARCH

Evaluate the extent to which simple Line Girder Analysis (LGA) methods can be applied for the design of straight steel I-girder bridges having small to moderate skew

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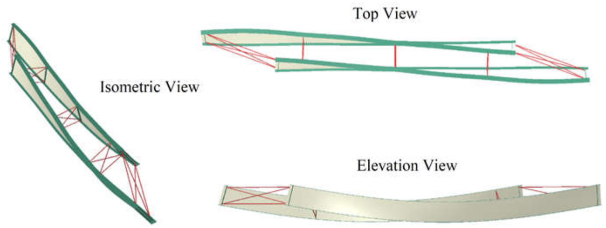
OUTLINE/AGENDA

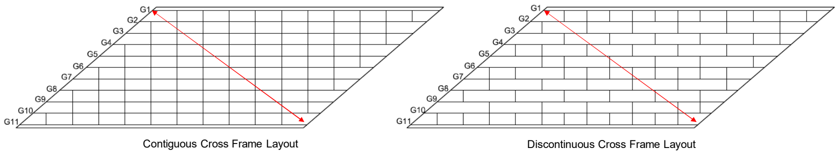
- Background: Behavior of Skewed Steel I-Girder Bridges
- Research Motivation and Objective
- Research Approach
- Research Findings and Recommendations
- COMPREHENSIVE DESIGN EXAMPLE (Dennis)
- Concluding Remarks

SKEWED I-GIRDER BRIDGE BEHAVIOR – TORSION



- The end cross frames at skewed bearing lines twist the girders due to compatibility of deformations
- Intermediate cross frames perpendicular to the girders attach to the girders at different positions along the spans, twisting the girders due to differential vertical deflections

SKEWED I-GIRDER BRIDGE BEHAVIOR – LOAD PATH



Contiguous Cross Frame Layout

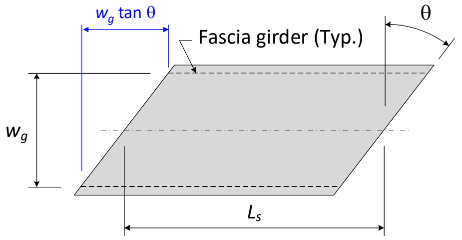
Discontinuous Cross Frame Layout

- Transverse load path effects: cross frames along the shorter diagonal of the bridge plan attract greater forces
- These effects can be mitigated by using discontinuous cross frame layouts with sufficient offsets &/or staggers (NCHRP 725 & NCHRP 20-07/355)

SKEW INDEX I_s AND SKEW ANGLE θ

$$I_s = \frac{w_g \tan\theta}{L_s}$$

I_s is taken as the maximum value determined from each span of a given bridge UNIT



- When $I_s < 0.3$, LGA calculates DC1 bending moments and vertical displacements with a worst-case normalized error ≤ 12% (NCHRP Report 725)
- Local skew effects near bearing lines such as girder layovers, bearing reactions, and girder end shear forces are influenced directly by the skew angle, θ

MOTIVATION FOR THIS RESEARCH

- For straight skewed bridges, the FDOT *Structures Design Guidelines* currently require
  - ➢ A refined method of analysis (2D grid, satisfying the NCHRP 725 requirements, or 3D FEA) for bridges with $0.2 < I_s \leq 0.6$
  - ➢ 3D FEA for bridges with $I_s > 0.6$
- This implies LGA is sufficient only for $I_s \leq 0.2$
- Between the years of 2000 and 2014, FDOT built approximately 250 steel I-girder bridges with $I_s \leq 0.3$
- The NCHRP 725 findings suggest LGA potentially can be applicable for bridges having I_s up to at least 0.3
- The use of discontinuous cross frame layouts with sufficient offsets and/or staggers can potentially extend LGA applicability even further

RESEARCH OBJECTIVE

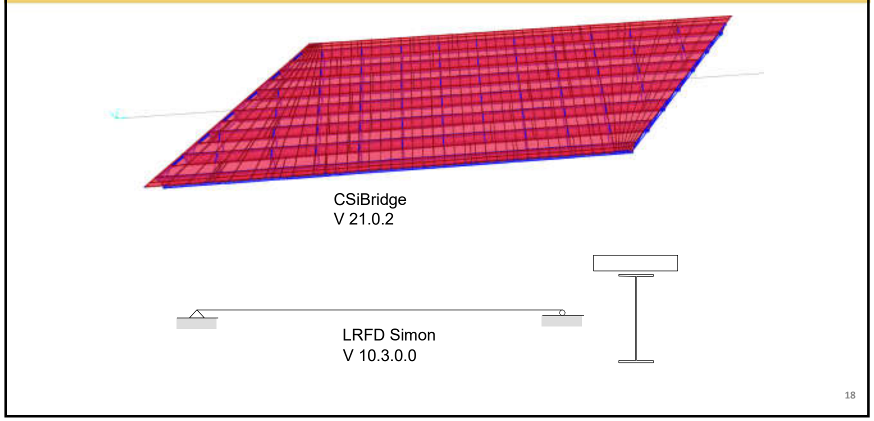
- Understand more fully the behavior of steel I-girder bridges with skew indices up to and slightly larger than 0.3
- Determine when LGA can be used in lieu of a refined analysis for these types of bridges
- Develop recommendations for application of LGA to straight-skewed I-girder bridges:
  - ➢ Direct estimation of girder major-axis bending moments, vertical shear forces, bearing reactions and vertical displacements (with adjustment where needed)
  - ➢ “Semi-direct” estimation of girder layovers under total dead load
  - ➢ Indirect estimation of girder flange lateral bending stresses and cross-frame forces

RESEARCH APPROACH – COMPARATIVE PARAMETRIC STUDY

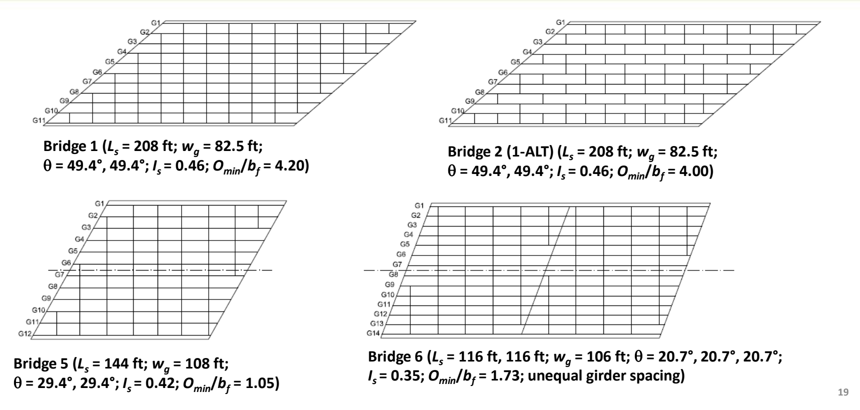
3D FEA VS LGA

- 20 existing bridges were selected from the FDOT inventory
- Six of the FDOT bridges were redesigned using discontinuous cross frame layouts with recommended offsets and/or staggers, giving a total of 26 bridges studied
- Bridge characteristics:
  - ➢ Skew indices from 0.15 to 0.47
  - ➢ Skew angles from 10.0 to 58.7 degrees
  - ➢ 21 parallel skew bridges + 5 nonparallel skew bridges
  - ➢ 16 continuous-span bridges + 10 simple-span bridges
  - ➢ Cross frame layout: contiguous (16 bridges) + discontinuous (8 bridges) + contiguous with cross frames parallel to the skew (2 bridges)
  - ➢ All the bridges used Steel Dead Load Fit (SDLF) detailing of their cross frames
  - ➢ A complete range of the dead & live load responses was considered, including the study of staged deck placement for four of the 26 bridges

3D FEA VS LGA



PLAN SKETCHES OF BRIDGES STUDIED



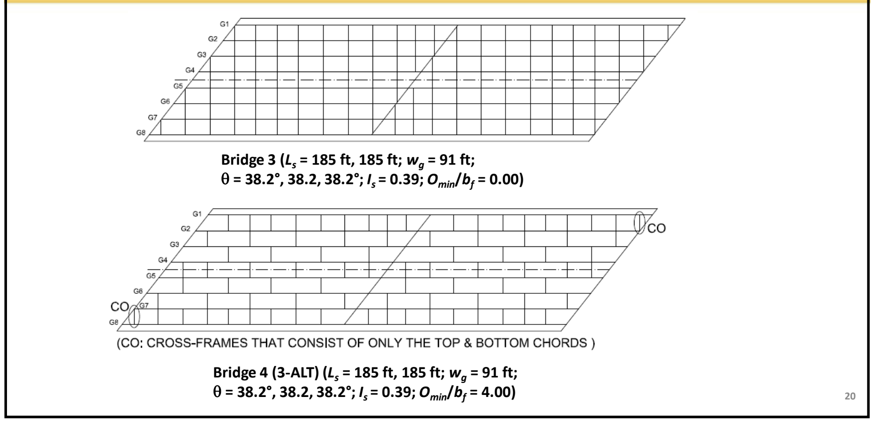
Bridge 1 (L_s = 208 ft; w_g = 82.5 ft; θ = 49.4°, 49.4°; I_s = 0.46; O_{min}/b_f = 4.20)

Bridge 2 (1-ALT) (L_s = 208 ft; w_g = 82.5 ft; θ = 49.4°, 49.4°; I_s = 0.46; O_{min}/b_f = 4.00)

Bridge 5 (L_s = 144 ft; w_g = 108 ft; θ = 29.4°, 29.4°; I_s = 0.42; O_{min}/b_f = 1.05)

Bridge 6 (L_s = 116 ft, 116 ft; w_g = 106 ft; θ = 20.7°, 20.7°, 20.7°; I_s = 0.35; O_{min}/b_f = 1.73; unequal girder spacing)

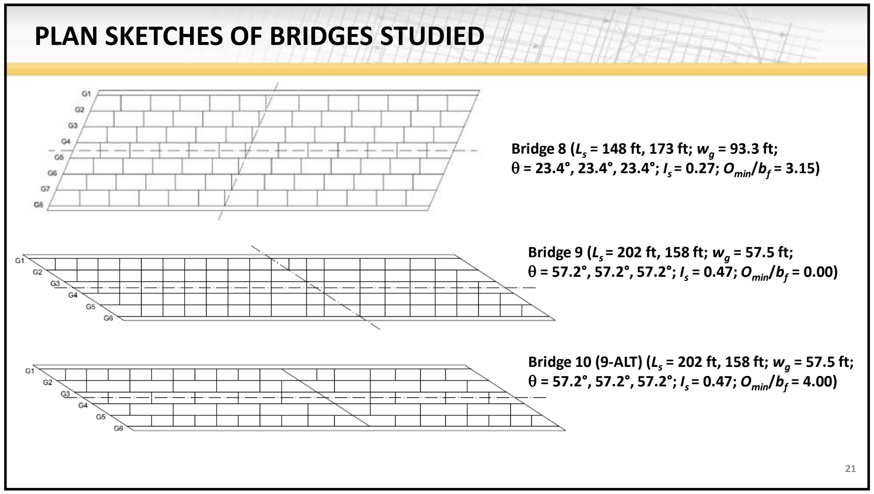
PLAN SKETCHES OF BRIDGES STUDIED



Bridge 3 (L_s = 185 ft, 185 ft; w_g = 91 ft; θ = 38.2°, 38.2, 38.2°; I_s = 0.39; O_{min}/b_f = 0.00)

(CO: CROSS-FRAMES THAT CONSIST OF ONLY THE TOP & BOTTOM CHORDS)

Bridge 4 (3-ALT) (L_s = 185 ft, 185 ft; w_g = 91 ft; θ = 38.2°, 38.2, 38.2°; I_s = 0.39; O_{min}/b_f = 4.00)

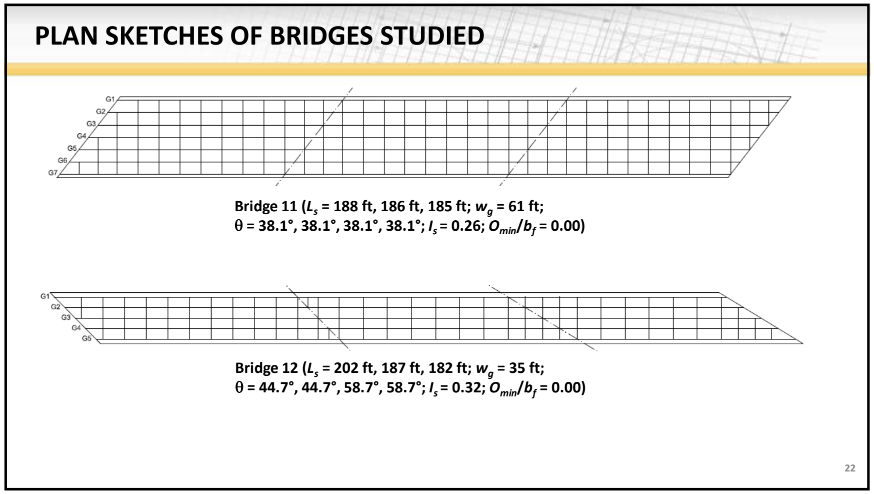


PLAN SKETCHES OF BRIDGES STUDIED

Bridge 8 (L_s = 148 ft, 173 ft; w_g = 93.3 ft;
θ = 23.4°, 23.4°, 23.4°; I_s = 0.27; O_{min}/b_f = 3.15)

Bridge 9 (L_s = 202 ft, 158 ft; w_g = 57.5 ft;
θ = 57.2°, 57.2°, 57.2°; I_s = 0.47; O_{min}/b_f = 0.00)

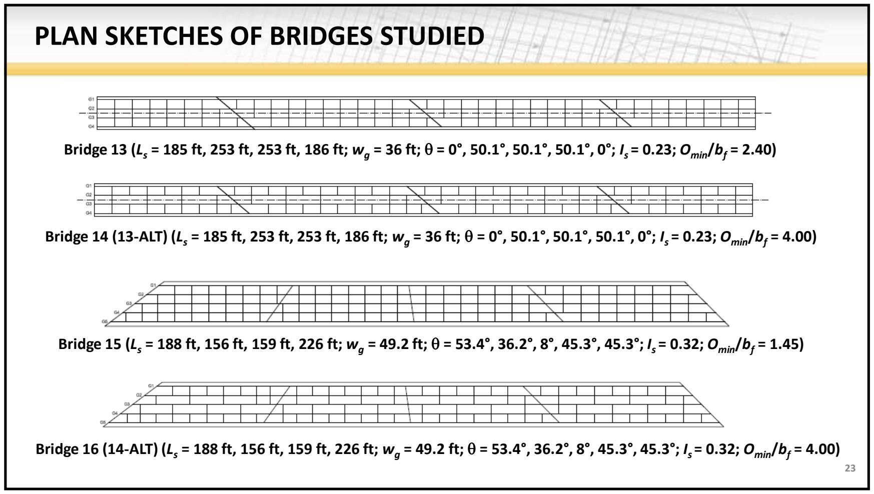
Bridge 10 (9-ALT) (L_s = 202 ft, 158 ft; w_g = 57.5 ft;
θ = 57.2°, 57.2°, 57.2°; I_s = 0.47; O_{min}/b_f = 4.00)



PLAN SKETCHES OF BRIDGES STUDIED

Bridge 11 (L_s = 188 ft, 186 ft, 185 ft; w_g = 61 ft;
θ = 38.1°, 38.1°, 38.1°, 38.1°; I_s = 0.26; O_{min}/b_f = 0.00)

Bridge 12 (L_s = 202 ft, 187 ft, 182 ft; w_g = 35 ft;
θ = 44.7°, 44.7°, 58.7°, 58.7°; I_s = 0.32; O_{min}/b_f = 0.00)



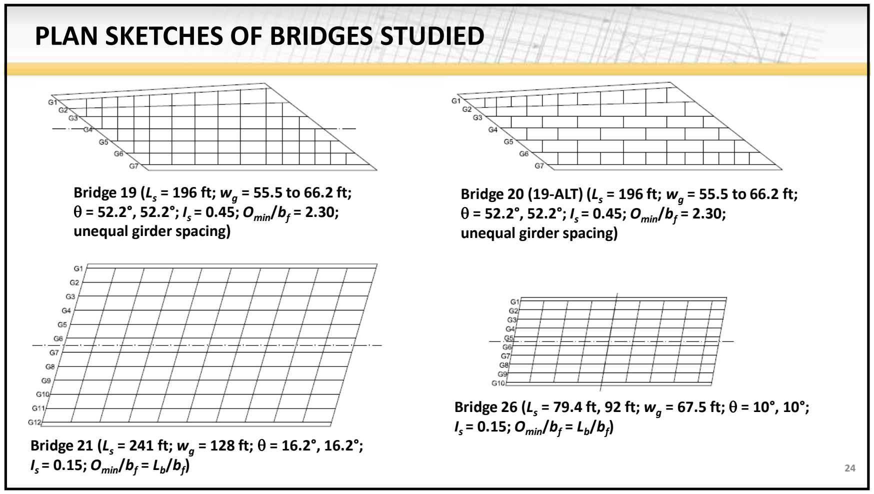
PLAN SKETCHES OF BRIDGES STUDIED

Bridge 13 (L_s = 185 ft, 253 ft, 253 ft, 186 ft; w_g = 36 ft; θ = 0°, 50.1°, 50.1°, 50.1°, 0°; I_s = 0.23; O_{min}/b_f = 2.40)

Bridge 14 (13-ALT) (L_s = 185 ft, 253 ft, 253 ft, 186 ft; w_g = 36 ft; θ = 0°, 50.1°, 50.1°, 50.1°, 0°; I_s = 0.23; O_{min}/b_f = 4.00)

Bridge 15 (L_s = 188 ft, 156 ft, 159 ft, 226 ft; w_g = 49.2 ft; θ = 53.4°, 36.2°, 8°, 45.3°, 45.3°; I_s = 0.32; O_{min}/b_f = 1.45)

Bridge 16 (14-ALT) (L_s = 188 ft, 156 ft, 159 ft, 226 ft; w_g = 49.2 ft; θ = 53.4°, 36.2°, 8°, 45.3°, 45.3°; I_s = 0.32; O_{min}/b_f = 4.00)



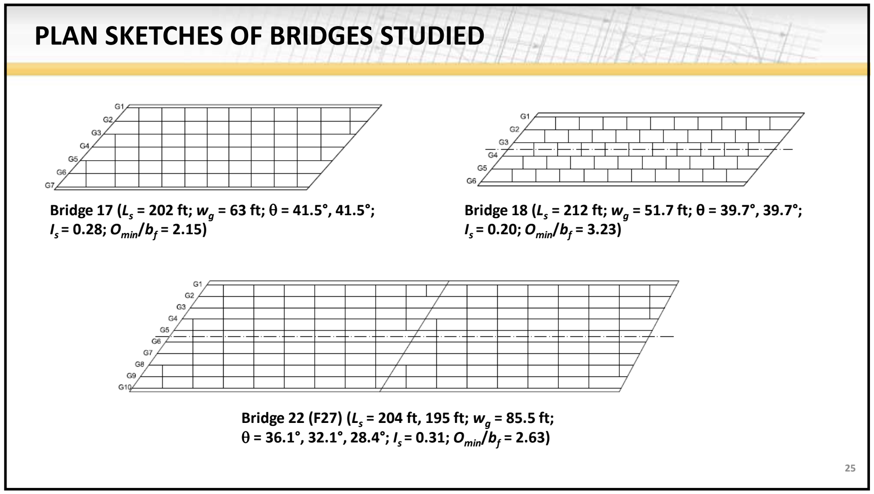
PLAN SKETCHES OF BRIDGES STUDIED

Bridge 19 (L_s = 196 ft; w_g = 55.5 to 66.2 ft;
θ = 52.2°, 52.2°; I_s = 0.45; O_{min}/b_f = 2.30;
unequal girder spacing)

Bridge 20 (19-ALT) (L_s = 196 ft; w_g = 55.5 to 66.2 ft;
θ = 52.2°, 52.2°; I_s = 0.45; O_{min}/b_f = 2.30;
unequal girder spacing)

Bridge 21 (L_s = 241 ft; w_g = 128 ft; θ = 16.2°, 16.2°;
I_s = 0.15; $O_{min}/b_f = L_b/b_f$)

Bridge 26 (L_s = 79.4 ft, 92 ft; w_g = 67.5 ft; θ = 10°, 10°;
I_s = 0.15; $O_{min}/b_f = L_b/b_f$)

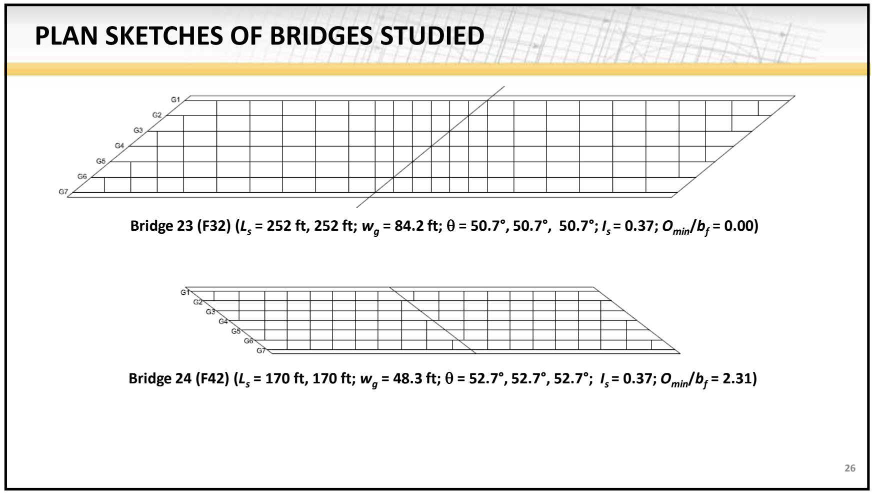


PLAN SKETCHES OF BRIDGES STUDIED

Bridge 17 (L_s = 202 ft; w_g = 63 ft; θ = 41.5°, 41.5°; I_s = 0.28; O_{min}/b_f = 2.15)

Bridge 18 (L_s = 212 ft; w_g = 51.7 ft; θ = 39.7°, 39.7°; I_s = 0.20; O_{min}/b_f = 3.23)

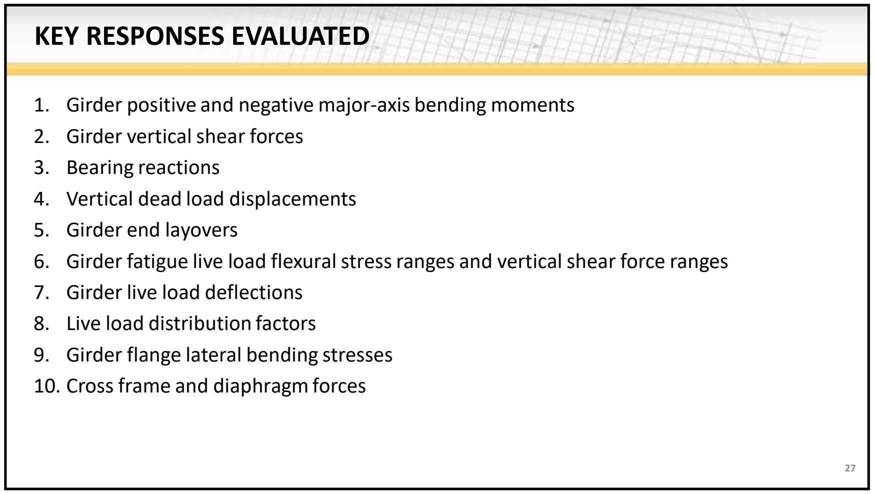
Bridge 22 (F27) (L_s = 204 ft, 195 ft; w_g = 85.5 ft; θ = 36.1°, 32.1°, 28.4°; I_s = 0.31; O_{min}/b_f = 2.63)



PLAN SKETCHES OF BRIDGES STUDIED

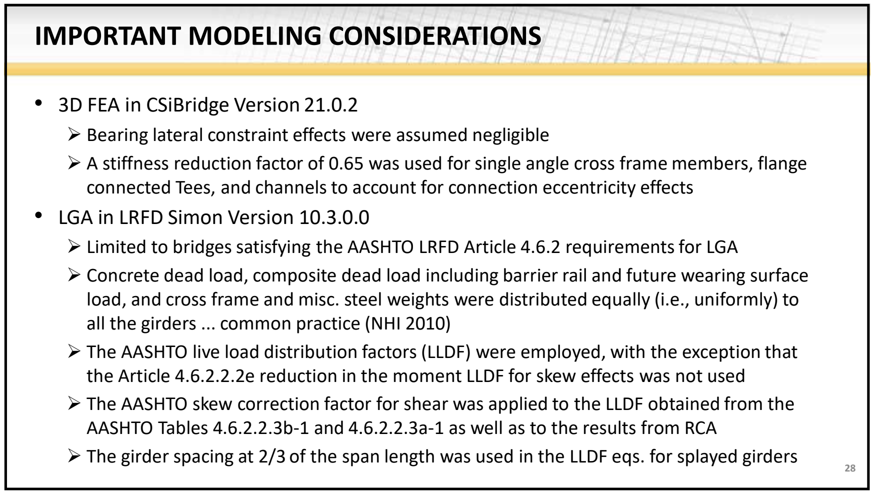
Bridge 23 (F32) (L_s = 252 ft, 252 ft; w_g = 84.2 ft; θ = 50.7°, 50.7°, 50.7°; I_s = 0.37; O_{min}/b_f = 0.00)

Bridge 24 (F42) (L_s = 170 ft, 170 ft; w_g = 48.3 ft; θ = 52.7°, 52.7°, 52.7°; I_s = 0.37; O_{min}/b_f = 2.31)



KEY RESPONSES EVALUATED

1. Girder positive and negative major-axis bending moments
2. Girder vertical shear forces
3. Bearing reactions
4. Vertical dead load displacements
5. Girder end layovers
6. Girder fatigue live load flexural stress ranges and vertical shear force ranges
7. Girder live load deflections
8. Live load distribution factors
9. Girder flange lateral bending stresses
10. Cross frame and diaphragm forces



IMPORTANT MODELING CONSIDERATIONS

- 3D FEA in CSiBridge Version 21.0.2
  - ➢ Bearing lateral constraint effects were assumed negligible
  - ➢ A stiffness reduction factor of 0.65 was used for single angle cross frame members, flange connected Tees, and channels to account for connection eccentricity effects
- LGA in LRFD Simon Version 10.3.0.0
  - ➢ Limited to bridges satisfying the AASHTO LRFD Article 4.6.2 requirements for LGA
  - ➢ Concrete dead load, composite dead load including barrier rail and future wearing surface load, and cross frame and misc. steel weights were distributed equally (i.e., uniformly) to all the girders ... common practice (NHI 2010)
  - ➢ The AASHTO live load distribution factors (LLDF) were employed, with the exception that the Article 4.6.2.2.2e reduction in the moment LLDF for skew effects was not used
  - ➢ The AASHTO skew correction factor for shear was applied to the LLDF obtained from the AASHTO Tables 4.6.2.2.3b-1 and 4.6.2.2.3a-1 as well as to the results from RCA
  - ➢ The girder spacing at 2/3 of the span length was used in the LLDF eqs. for splayed girders

MEASURES OF DIFFERENCES BETWEEN LGA AND 3D FEA

Measure	Mathematical Expression	Recommended Tolerance
Professional Factor	$\rho_{\max} = \dfrac{\lvert 3DFEA \rvert_{\max}}{\lvert LGA \rvert_{\max}}$	1.05
Normalized difference for girder vertical displacements	$\varepsilon_{\max 2} = \dfrac{(\Delta_{LGA})_{\max} - (\Delta_{3DFEA})_{\max}}{L_s}$	±0.0005
Normalized difference for girder differential vertical displacements	$\varepsilon_{\max 3} = \max\left\{ \dfrac{\left[(\Delta_{LGA})_{\max} - (\Delta_{3DFEA})_{\max}\right]_{G3} - \left[(\Delta_{LGA})_{\max} - (\Delta_{3DFEA})_{\max}\right]_{G1}}{w_g}, \dfrac{\left[(\Delta_{LGA})_{\max} - (\Delta_{3DFEA})_{\max}\right]_{G3} - \left[(\Delta_{LGA})_{\max} - (\Delta_{3DFEA})_{\max}\right]_{G4}}{w_g} \right\}$	±0.001

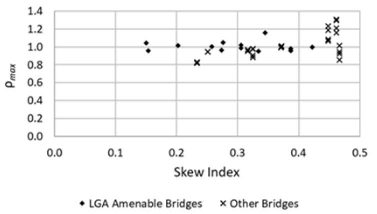
Findings and Recommendations

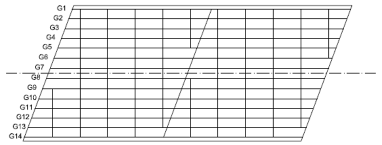
PROPOSED CATEGORIZATION OF BRIDGES

- **Bridges in which LGA is permitted:**
  1. Parallel skew bridges satisfying AASHTO LRFD Article 4.6.2, having $\theta \leq 20°$ & $I_s \leq 0.15$, and having intermediate cross frame lines oriented parallel to skew
  2. Parallel skew bridges satisfying AASHTO LRFD Article 4.6.2, having $\theta \leq 50°$ (cross frames oriented perpendicular to the girders), and having $I_s \leq 0.3$
  3. Parallel skew bridges satisfying AASHTO LRFD Article 4.6.2, having $\theta \leq 50°$ and $0.3 < I_s \leq 0.4$, or having $\theta \leq 30°$ and $0.4 < I_s \leq 0.45$
- **Bridges for which LGA is not recommended:**
  - Parallel skew bridges that do not fall within the three categories listed above
  - Bridges having nonparallel skew, i.e., deviation in skew angles between adjacent support lines larger than 10°
  - Bridges having splayed girders, i.e., girder splay angles larger than 3° or change in the deck width within a span larger than 15%
  - Bridges with any girder having a $D/b_f > 5.0$

GIRDER BENDING MOMENTS AND VERTICAL SHEARS

- $\rho_{\max} \leq 1.02$ for all girders in all the Category 1 and 2 bridges studied, for both positive and negative bending moments ($\rho_{\max} \leq 1.05$ for girder shears)… this is due to the conservatism associated with the AASHTO LLDF used in the LGA
- $\rho_{\max} = 1.06$ for positive bending moment, 0.96 for negative bending moment, & 1.16 for shear in the fascia girders for one of the Category 3 bridges (Bridge 6)





Bridge 6 (L_s = 116 ft, 116 ft; w_g = 106 ft; θ = 20.7°, 20.7°, 20.7°; I_s = 0.35; O_{min}/b_f = 1.73; unequal girder spacing)

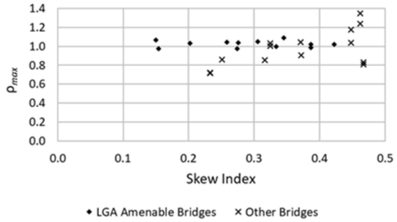
GIRDER BENDING MOMENTS AND VERTICAL SHEARS

- The fascia girders generally have larger ρ_{max} values (for moment and shear) compared to the interior girders
- The girder ρ_{max} values are smaller in bridges having discontinuous cross frame arrangements with recommended offsets &/or staggers
- There is a clear spike in the ρ_{max} values for the STR I vertical shear forces in the fascia girders for the bridges studied that fall outside of Categories 1, 2 and 3

BEARING REACTIONS

- For bridges belonging to Design Categories 2 and 3, multiply the fascia girder bearing reactions by 1.10 at the obtuse corners at abutments and at the piers in continuous-span bridges
- This is in addition to the application of the AASHTO LRFD Article 4.6.2.2.3c live load skew correction factor to these reactions

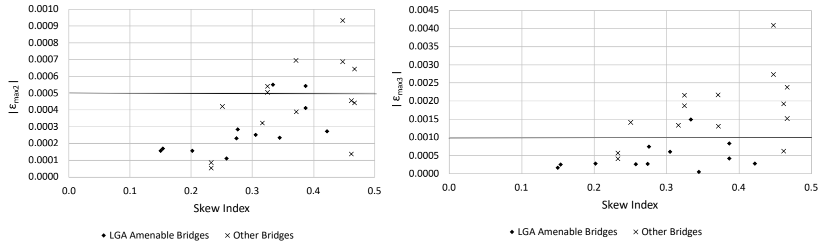
... bearing reaction ρ_{max} values prior to multiplication by 1.10



TOTAL DEAD LOAD (TDL) VERTICAL DISPLACEMENTS

- Important for determining girder cambers
- Conservative displacement estimates can be just as difficult of a problem as unconservative displacement estimates
- The largest relative differences between the 3D FEA and LGA calculations tend to occur for the TDL vertical displacements
  ... the TDL displacement estimates do not have the benefit of offsetting conservative live load estimates, which is the case in evaluating the STR I response quantities
- Bridges that fall outside of Design Categories 1, 2 and 3 may have difficulty in satisfying the recommended tolerances for vertical displacements
- For the bridges in Categories 1, 2 and 3, the LGA predictions of the TDL (SDLF) displacements meet, or only slightly exceed, the target requirements

TOTAL DEAD LOAD VERTICAL DISPLACEMENTS



GIRDER LAYOVER UNDER TOTAL DEAD LOAD

... based on a simplified consideration of compatibility of deformations, assuming steel dead load fit (SDLF) detailing of the cross frames, the ***girder major-axis bending end rotation under the concrete dead load*** may be estimated with good accuracy as

$$\alpha = \frac{\delta_{0.1L_s}}{0.1L_s}$$

where $\delta_{0.1Ls}$ is the LGA estimate of the concrete dead load vertical displacement at the girder 1/10th point within the span

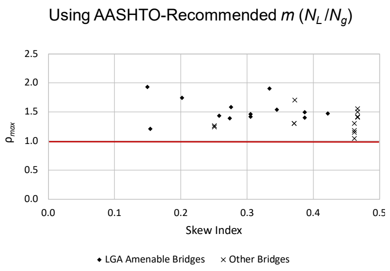
Given α and the bearing line skew angle θ, the corresponding ***girder twist*** at the abutments may be estimated as

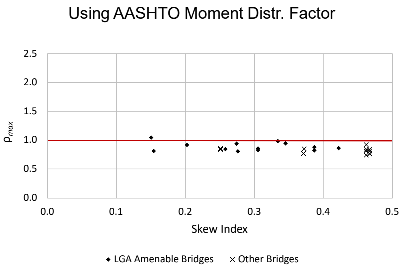
$$\phi = \alpha \tan(\theta)$$

Using the web depth, *D*, the ***girder layover*** at the top flange is

$$Layover = D\phi$$

ESTIMATION OF LIVE LOAD DISPLACEMENTS


Using AASHTO-Recommended m (N_L / N_g)


Using AASHTO Moment Distr. Factor

INDIRECT RESPONSE ESTIMATES

- Girder flange lateral bending stresses
  - ➢ Improvement of the AASHTO LRFD Article C6.10.1 rules
- Cross frame forces
  - ➢ A table of coefficients (expressed as fractions of girder forces) is recommended for estimation of upper-bound cross frame forces caused by the skew effects
  - ➢ These estimated forces are to be added to the other calculated cross frame forces due to wind load, eccentric loads on overhangs during construction, etc. to obtain the required forces for cross frame design
  - ➢ Stability bracing considerations are already addressed in these coefficients ... i.e., there is no need to calculate separate stability bracing cross frame forces
  - ➢ In addition, the cross frame members should satisfy the AASHTO LRFD restrictions on L/r_{min}
  - ➢ Straight skewed bridge cross frame members should be considered as secondary members, but $L/r_{min} \leq 120$ is preferred

CALCULATION OF UNFACTORED f_ℓ (IMPROVEMENT OF AASHTO C6.10.1)

					f_ℓ (ksi)		
Bridge Category	**Cross Frame Layout**	**Orientation of Intermediate Cross Frames**	**Girder**	**Location**	**$O_{min}/b_f < 4$**	**$O_{min}/b_f \geq 4$**	**Current AASHTO C6.10.1**
1	Contiguous	Parallel to skew	Exterior	All	0	0	0
			Interior		0	0	0
2 or 3	Contiguous	Perpendicular to girders	Exterior	At or near supports	8	4	7.5
				Within the span	0	0	0
		Perpendicular to girders	Interior	At or near supports	10	5	10
				Within the span	0	0	0
2 or 3	Discontinuous throughout the span	Perpendicular to girders	Exterior	At or near supports	8	4	7.5
				Within the span	3	2	2
		Perpendicular to girders	Interior	At or near supports	10	5	10
				Within the span	15	10	10

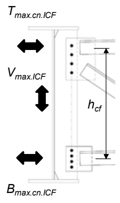
CALCULATION OF FACTORED f_ℓ (IMPROVEMENT OF AASHTO C6.10.1)

Use a weighted average load factor of:

- 1.6 for STR I
- 1.3 for STR III
- 1.2 for SER II
- 1.4 for DC + construction loads
- 1.75 for Fatigue I ... smaller estimated value under consideration
- 0.8 for Fatigue II ... smaller estimated value under consideration

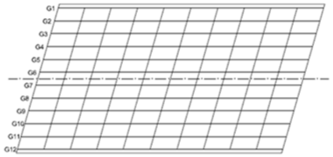
CROSS FRAME AND DIAPHRAGM FORCES – TABLE OF COEFFICIENTS

Cross-Frame Case	Load Effect	DC1 & Constr	DC2	DW	HL-93 LL	STR I & SER II	Fatigue LL
(1i) Intermediate cross-frames using offsets and staggers greater than or equal to $4b_f$ throughout the span	$V_{max.ICF-}/V_{max.g}$	0.02	0.40	0.03	0.06	0.03	0.06
	$B_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.20	0.02	0.06	0.03	0.05
	$T_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.20	0.02	0.05	0.02	0.05
(2i) Contiguous intermediate cross-frames, or intermediate cross-frames with any offsets and staggers less than $4b_f$ within the span	$V_{max.ICF-}/V_{max.g}$	0.06	1.20	0.06	0.20	0.09	0.14
	$B_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.60	0.04	0.14	0.08	0.12
	$T_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.03	0.40	0.02	0.08	0.04	0.08
(1b) Bearing line cross-frames where the offset of intermediate cross-frames relative to the bearing line is greater than or equal to $4b_f$	$V_{max.BCF-}/V_{max.g}$	0.02	0.07	0.02	0.02	0.02	0.02
	$B_{max.cn.BCF}/(M_{max.g}/h_{cf})$	0.02	0.04	0.02	0.02	0.02	0.02
	$T_{max.cn.BCF}/(M_{max.g}/h_{cf})$	0.02	0.10	0.02	0.03	0.02	0.04
(2b) Bearing line cross-frames where the offset of intermediate cross-frames relative to the bearing line is smaller than $4b_f$	$V_{max.BCF-}/V_{max.g}$	0.04	0.12	0.06	0.06	0.04	0.06
	$B_{max.cn.BCF}/(M_{max.g}/h_{cf})$	0.02	0.14	0.02	0.08	0.05	0.07
	$T_{max.cn.BCF}/(M_{max.g}/h_{cf})$	0.02	0.14	0.02	0.03	0.02	0.05



SUMMARY OF LGA GUIDELINES – CATEGORY 1 BRIDGES

- ...AASHTO LRFD Article 4.6.2 requirements satisfied, $\theta \leq 20^o$ & $I_s \leq 0.15$, CFs parallel to bearing lines...
- Calculate girder design demands and cambers directly from LGA without any further adjustment
- Calculate bearing reactions from LGA without any further adjustment
- Calculate girder end layovers under TDL using a simplified compatibility of deformations analysis
- Neglect effects of skew on girder flange lateral bending stresses
- Determine the cross frame forces due to skew effects using the table of coefficients
- Determine live load deflections using LLDF for moment

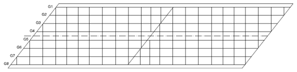


Bridge 21

L_s = 241 ft, w_g = 128 ft,
$\theta = 16.2^o < 20^o$, I_s = 0.15

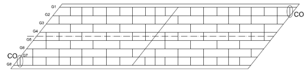
SUMMARY OF LGA GUIDELINES – CATEGORY 2 & 3 BRIDGES

- ... AASHTO LRFD Article 4.6.2 requirements satisfied, larger limits on I_s and θ satisfied...
- Calculate girder design demands and cambers directly from LGA without any further adjustment
- Multiply bearing reactions by 1.10 at obtuse corners at abutments & at fascia girders at the piers in continuous-span bridges, in addition to applying skew correction factors
- Calculate girder end layovers under TDL using a simplified compatibility of deformations analysis
- Estimate the girder flange lateral bending stresses due to skew effects using improvements upon the AASHTO LRFD Article C6.10.1 recommendations
- Determine the cross frame forces due to skew effects using the table of coefficients
- Determine live load deflections using LLDF for moment



Bridge 3

L_s = 185 ft, 185 ft, w_g = 91 ft, θ = 38.2°,
I_s = 0.39, O_{min}/b_f = zero



(CO: CROSS-FRAMES THAT CONSIST OF ONLY THE TOP & BOTTOM CHORDS)

Bridge 4

L_s = 185 ft, 185 ft, w_g = 91 ft, θ = 38.2°,
I_s = 0.39, O_{min}/b_f = 4.0

Part 2:
Comprehensive Design Example

Dennis Golabek, PE
Senior Technical Principal
WSP

Smarter.
Stronger.
Steel.

Line Girder Analysis for Skewed Straight Steel I-Girder Bridges (SSSIG)

- AASHTO LRFD BDS Loading Combinations
- AASHTO LRFD BDS Approximate Methods of Analysis
  - Requirements
  - LLDF
- FDOT BE 535 Recommendations
  - Applicability
  - Live Load Analysis
  - LGA using LRFD Simon V10.3.0.0 (8th Ed. LRFD)
- Design Example
  - 3-span continuous, 140'-180'-140', θ = 30°, I_s = 0.20

AASHTO LRFD BDS Loading Combinations (Strength I, Service I&II, and Fatigue)

HL-93 Loading one or multiple lanes

Strength I
- member strength for normal vehicular use for design life

Service II
- permanent deformation(overload)
- slip-critical connections

Truck only one lane

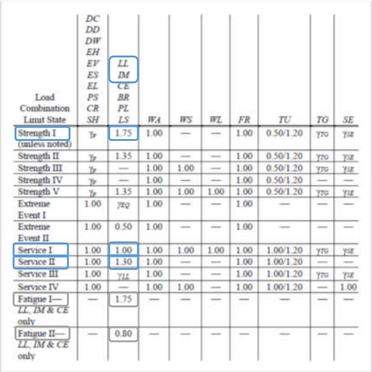
Fatigue I or II
- stress ranges
- shear (fatigue truck) in girder webs

HL-93 Special loading

Service I
- optional live load deflection criteria
- (cambers)

Constructability

IMPORTANT



Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}
Strength III	γ_p	—	1.00	1.00	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—
Strength V	γ_p	1.35	1.00	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}
Extreme Event I	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—
Extreme Event II	1.00	0.50	1.00	—	—	1.00	—	—	—
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—
Service III	1.00	γ_{LL}	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}
Service IV	1.00	—	1.00	1.00	—	1.00	1.00/1.20	—	1.00
Fatigue I—*LL*, *IM* & *CE* only	—	1.75	—	—	—	—	—	—	—
Fatigue II—*LL*, *IM* & *CE* only	—	0.80	—	—	—	—	—	—	—

AASHTO LRFD BDS Loading Combinations (Strength I, Service I&II, and Fatigue)

Strength I and Service II

3.6.1.2.2 Design Truck – axle weights and spacing

3.6.1.3 Design Vehicular Live Loads –truck or tandem and lane; negative moment

3.6.1.1.2 Multiple Presence of Live Load

4.6.2.2.1 MPF, m, incorporated in the distribution factors

Service I

2.5.2.6.2/3.6.1.3.2 Optional LL deflection - design truck alone or 25% DT + lane load and equal distribution

3.6.1.1.2 Multiple Presence of Live Load

Table 3.6.1.1.2-1—Multiple Presence Factors, m

Number of Loaded Lanes	Multiple Presence Factors, m
1	1.20
2	1.00
3	0.85
>3	0.65

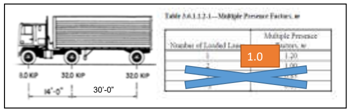
Fatigue I or II

1.3.2.3 Single truck

3.6.1.1.2 MPF – one lane LLDF, <u>other than the lever rule and statical method</u>, divided by 1.20

3.6.1.4.1 Fixed 30' between the 32 kip axle

3.6.1.4.3b LLDF for one lane

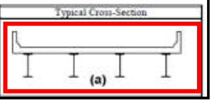


AASHTO LRFD BDS Approximate Methods of Analysis

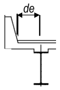
4.6.2 Approximate Methods of Analysis

4.6.2.2—Beam-Slab Bridges

Table 4.6.2.2.1-1—Common Deck Superstructures Covered in Articles 4.6.2.2.2 and 4.6.2.2.3

Supporting Components	Type of Deck	Typical Cross-Section
Steel Beam	Cast-in-place concrete slab, precast concrete slab, steel grid, glued/spiked panels, stressed wood	 (a)

LLDF per **Articles 4.6.2.2.2 and 4.6.2.2.3** – meet the following conditions:



- Width of deck is constant;
- Number of beams is **not less than four** (UNO – is 3 for Type “a”);
- Beams are parallel and have approximately the same stiffness;
- *de* ≤ 5.5’ for Type “a”
- Cross-section applicable to Table 4.6.2.2.1-1 (Type “a” for skewed SSIG)

For beam spacing exceeding the range of applicability, the live load on each beam shall be the reaction of the loaded lanes based on the lever rule unless specified otherwise herein. Bridges not meeting the requirements of this Article shall be analyzed as specified in Article 4.6.3.

Where bridges meet the conditions specified herein, **permanent loads** of and on the deck may be distributed uniformly among the beams and/or stringers.

AASHTO LRFD BDS Approximate Methods of Analysis

LLDF for Moments

Interior Beams (4.6.2.2.2b)

One Lane

$$0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

Two or more Lanes

$$0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,Lt_s^3}\right)^{0.1}$$

$3.5 \le S \le 16.0$
$4.5 \le t_s \le 12.0$
$20 \le L \le 240$
$N_b \ge 4$
$10{,}000 \le K_g \le 7{,}000{,}000$

Strength I & Service II - use greater “g”
Fatigue – use **One Lane** “g” / 1.2

Exterior Beams (4.6.2.2.2d)

1) One Lane — Lever Rule

Number of Lane	*m*
1 lane	m_1 = 1.2

Two or more Lanes

$$g = e\,g_{interior} \qquad e = 0.77 + \frac{d_e}{9.1}$$

$-1.0 \le d_e \le 5.5$

2)

$$R=\frac{N_L}{N_b}+\frac{X_{ext}\sum^{N_L}e}{\sum^{N_b}x^2} \quad \text{(C4.6.2.2.2d-1)}$$

Rigid Cross-section Analysis (RCA)

Number of Lane	*m*
1 lane	m_1 = 1.2
2 lanes	m_2 = 1.0
3 lanes	m_3 = 0.85

4.6.2.2.2e Skewed Bridges
When the line supports are s d an he difference between skew angles of two ad ent s o upports does not exceed 10 degrees, the be g mo in the beams **may be reduced** in accordance wi 4.6.2.2.2e-1.

Strength I and Service II – use the greatest “g”
Fatigue – greater of **One Lane** LR or RCA (m_1 = 1.0)

AASHTO LRFD BDS Approximate Methods of Analysis

LLDF for Shears

Interior Beams (4.6.2.2.3a)

One Lane

$$0.36+\frac{S}{25.0}$$

Two or more Lanes

$$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$$

$3.5 \le S \le 16.0$
$20 \le L \le 240$
$4.5 \le t_s \le 12.0$
$N_b \ge 4$

Strength I & Service II - use greater “g”
Fatigue – use **One Lane** “g” / 1.2

Exterior Beams (4.6.2.2.3b)

1) One Lane — Lever Rule

Number of Lane	*m*
1 lane	m_1 = 1.2

Two or more Lanes

$$g = e\,g_{interior} \qquad e = 0.6 + \frac{d_e}{10}$$

2)

$$R=\frac{N_L}{N_b}+\frac{X_{ext}\sum^{N_L}e}{\sum^{N_b}x^2} \quad \text{(C4.6.2.2.2d-1)}$$

Rigid Cross-section Analysis (RCA)

Number of Lane	*m*
1 lane	m_1 = 1.2
2 lanes	m_2 = 1.0
3 lanes	m_3 = 0.85

Strength I and Service II – use the greatest “g”
Fatigue – greater of **One Lane** LR or RCA (m_1 = 1.0)

AASHTO LRFD BDS Approximate Methods of Analysis

Skewed Bridges (4.6.2.2.3c)

Shear in girders shall be adjusted when the line of support is skewed — Applied to LR & RCA

1. Connected and behave as a unit, only the exterior and first interior beam.
2. SCF applied between the obtuse corner support and mid-span, and decreased linearly to 1.0 at mid-span

Skew Correction Factor (SCF)

Table 4.6.2.2.3c-1

Correction Factor	Range of Applicability
$1.0+0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$	$0° \le \theta \le 60°$ $3.5 \le S \le 16.0$ $20 \le L \le 240$ $N_b \ge 4$

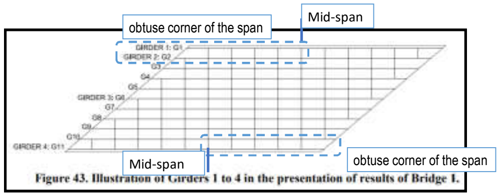
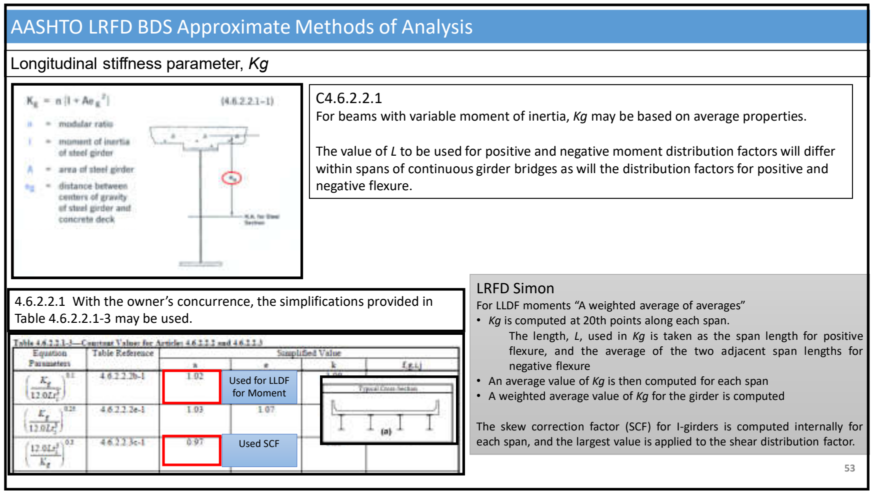


Figure 43. Illustration of Girders 1 to 4 in the presentation of results of Bridge 1.

4.6.2.6 Effective Flange Width (Steel I-Girders with CIP concrete slab - Type “a”)

…the effective flange width may be taken as the tributary width perpendicular to the axis of the member…
…the largest skew angle, θ, in the bridge system shall be less than 75 degrees…



AASHTO LRFD BDS Approximate Methods of Analysis

Longitudinal stiffness parameter, *Kg*

$K_g = n(I + Ae_g^2)$ (4.6.2.2.1-1)

n = modular ratio

I = moment of inertia of steel girder

A = area of steel girder

e_g = distance between centers of gravity of steel girder and concrete deck

C4.6.2.2.1
For beams with variable moment of inertia, *Kg* may be based on average properties.

The value of *L* to be used for positive and negative moment distribution factors will differ within spans of continuous girder bridges as will the distribution factors for positive and negative flexure.

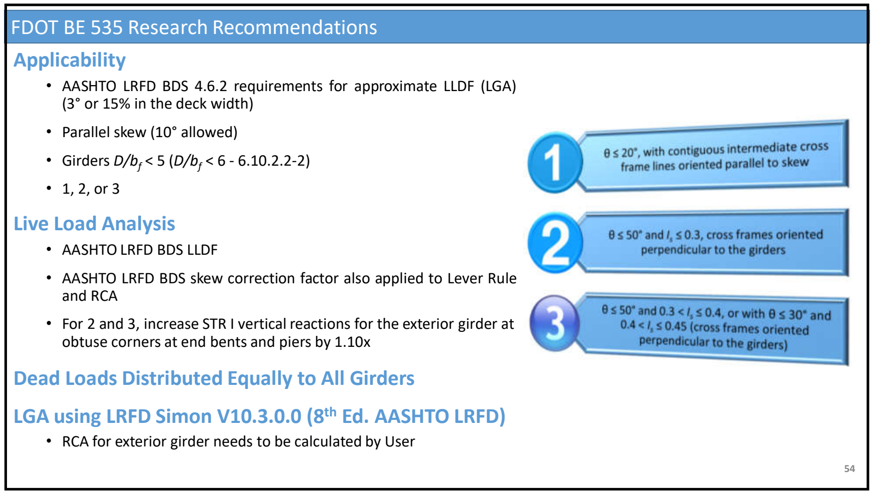
4.6.2.2.1 With the owner's concurrence, the simplifications provided in Table 4.6.2.2.1-3 may be used.

Table 4.6.2.2.1-3—Constant Values for Articles 4.6.2.2.2 and 4.6.2.2.3

Equation Parameters	Table Reference	Simplified Value: a	e	k	f,g,i,j
$\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$	4.6.2.2.2b-1	1.02	Used for LLDF for Moment		
$\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}$	4.6.2.2.2e-1	1.03	1.07		
$\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}$	4.6.2.2.3c-1	0.97	Used SCF		

LRFD Simon
For LLDF moments "A weighted average of averages"
- *Kg* is computed at 20th points along each span.
  The length, *L*, used in *Kg* is taken as the span length for positive flexure, and the average of the two adjacent span lengths for negative flexure
- An average value of *Kg* is then computed for each span
- A weighted average value of *Kg* for the girder is computed

The skew correction factor (SCF) for I-girders is computed internally for each span, and the largest value is applied to the shear distribution factor.



FDOT BE 535 Research Recommendations

Applicability
- AASHTO LRFD BDS 4.6.2 requirements for approximate LLDF (LGA) (3° or 15% in the deck width)
- Parallel skew (10° allowed)
- Girders $D/b_f < 5$ ($D/b_f < 6$ - 6.10.2.2-2)
- 1, 2, or 3

1 θ ≤ 20°, with contiguous intermediate cross frame lines oriented parallel to skew

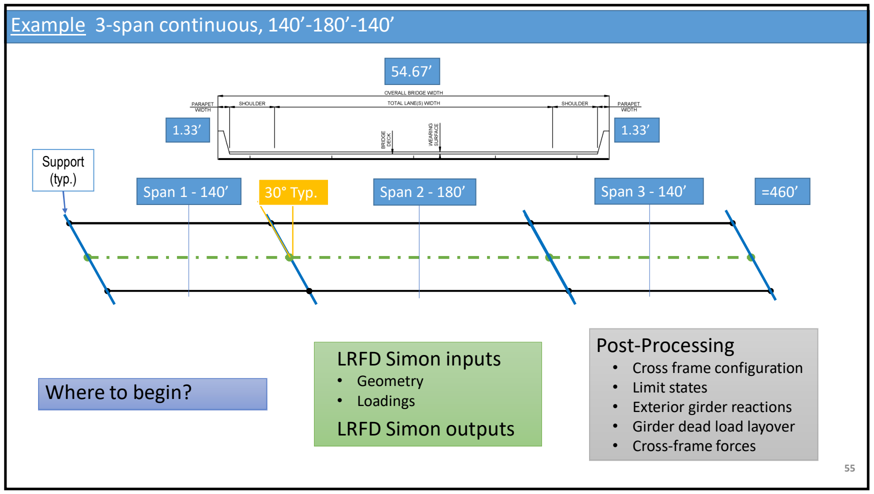
2 θ ≤ 50° and $I_s \le 0.3$, cross frames oriented perpendicular to the girders

3 θ ≤ 50° and $0.3 < I_s \le 0.4$, or with θ ≤ 30° and $0.4 < I_s \le 0.45$ (cross frames oriented perpendicular to the girders)

Live Load Analysis
- AASHTO LRFD BDS LLDF
- AASHTO LRFD BDS skew correction factor also applied to Lever Rule and RCA
- For 2 and 3, increase STR I vertical reactions for the exterior girder at obtuse corners at end bents and piers by 1.10x

Dead Loads Distributed Equally to All Girders

LGA using LRFD Simon V10.3.0.0 (8th Ed. AASHTO LRFD)
- RCA for exterior girder needs to be calculated by User



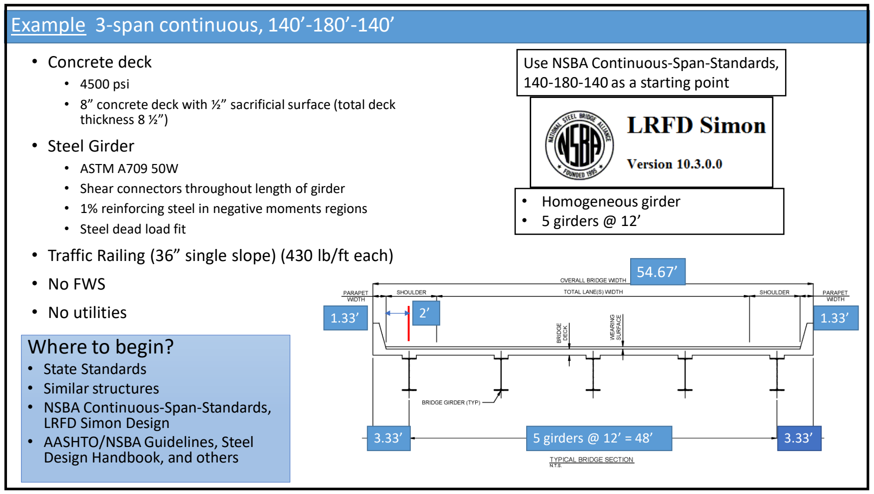
Example 3-span continuous, 140'-180'-140'

Where to begin?

LRFD Simon inputs
- Geometry
- Loadings

LRFD Simon outputs

Post-Processing
- Cross frame configuration
- Limit states
- Exterior girder reactions
- Girder dead load layover
- Cross-frame forces



Example 3-span continuous, 140'-180'-140'

- Concrete deck
  - 4500 psi
  - 8" concrete deck with ½" sacrificial surface (total deck thickness 8 ½")
- Steel Girder
  - ASTM A709 50W
  - Shear connectors throughout length of girder
  - 1% reinforcing steel in negative moments regions
  - Steel dead load fit
- Traffic Railing (36" single slope) (430 lb/ft each)
- No FWS
- No utilities

Use NSBA Continuous-Span-Standards, 140-180-140 as a starting point

LRFD Simon
Version 10.3.0.0

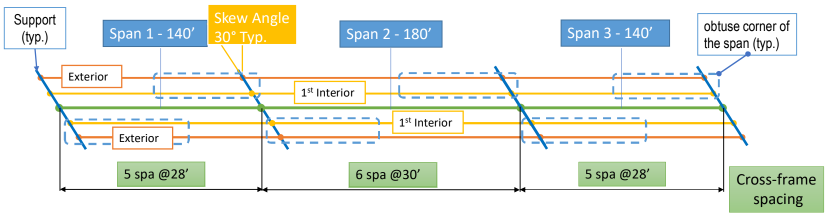
- Homogeneous girder
- 5 girders @ 12'

Where to begin?
- State Standards
- Similar structures
- NSBA Continuous-Span-Standards, LRFD Simon Design
- AASHTO/NSBA Guidelines, Steel Design Handbook, and others

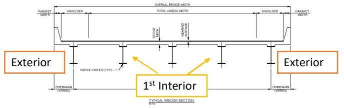
LRFD Simon Inputs - Geometry

$I_s = w_g * \tan \theta / L$
$= 48 * \tan 30 / 140 = 0.20$

2: $\theta < 50°$ and $I_s \leq 0.3$, cross frames oriented perpendicular to the girders


Support (typ.)
Span 1 - 140’
Skew Angle 30° Typ.
Span 2 - 180’
Span 3 - 140’
obtuse corner of the span (typ.)
Exterior
1st Interior
1st Interior
Exterior
5 spa @28’
6 spa @30’
5 spa @28’
Cross-frame spacing

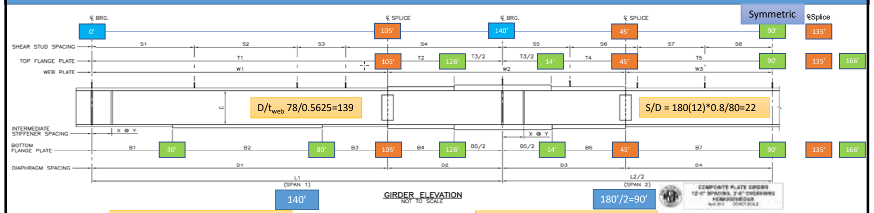
Girder spacing = 12’



Exterior
1st Interior
Exterior

Observations
- Girder design cannot use Appendix A6 (LRFD *6.10.6.2.3)*
- Skew correction factor (SCF) for shear (and reactions) (LRFD 4.6.2.2.3c)
- Cross-frames perpendicular – FLB stresses (LRFD C6.10.1 and FDOT BE 535) (if you so choose)
- FDOT BE 535 Modification factor for exterior girder reaction

LRFD Simon Inputs – Girder



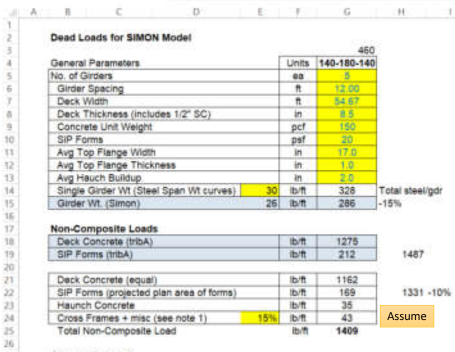
INTERIOR GDR INPUT DATA for SPAN 1

Component	Range (ft)	Yield Strength (ksi)	Approx Weight (tons)	Length (ft)	Thickness (in)	Width (in)
web	105.00	50.0	7.83	105.000	0.5625	78.00
web	140.00	50.0	2.61	35.000	0.5625	78.00
top flange	105.00	50.0	2.14	105.000	0.7500	16.00
top flange	126.00	50.0	0.64	21.000	1.0000	18.00
top flange	140.00	50.0	0.78	14.000	1.8125	18.00
bottom flange	30.00	50.0	0.82	30.000	1.0000	16.00
bottom flange	80.00	50.0	2.04	50.000	1.5000	16.00
bottom flange	105.00	50.0	1.02	25.000	1.5000	16.00
bottom flange	126.00	50.0	1.16	21.000	1.6250	20.00
bottom flange	140.00	50.0	1.25	14.000	2.6250	20.00

INTERIOR GDR INPUT DATA for SPAN 2

Component	Range (ft)	Yield Strength (ksi)	Approx Weight (tons)	Length (ft)	Thickness (in)	Width (in)
web	45.00	50.0	3.36	45.000	0.5625	78.00
web	135.00	50.0	6.71	90.000	0.5625	78.00
web	180.00	50.0	3.36	45.000	0.5625	78.00
top flange	14.00	50.0	0.78	14.000	1.8125	18.00
top flange	45.00	50.0	1.19	31.000	1.2500	18.00
top flange	90.00	50.0	0.92	45.000	0.7500	16.00
top flange	135.00	50.0	0.92	45.000	0.7500	16.00
top flange	166.00	50.0	1.19	31.000	1.2500	18.00
top flange	180.00	50.0	0.78	14.000	1.8125	18.00
bottom flange	14.00	50.0	1.25	14.000	2.6250	20.00
bottom flange	45.00	50.0	1.65	31.000	1.5625	20.00
bottom flange	90.00	50.0	1.55	45.000	1.1250	18.00
bottom flange	135.00	50.0	1.55	45.000	1.1250	18.00
bottom flange	166.00	50.0	1.65	31.000	1.5625	20.00
bottom flange	180.00	50.0	1.25	14.000	2.6250	20.00

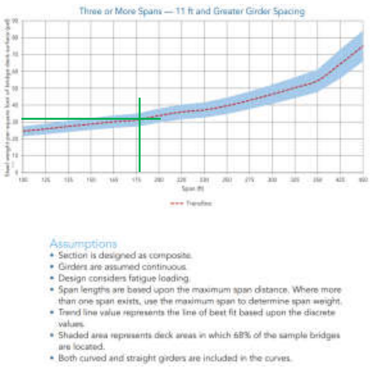
LRFD Simon Inputs - Dead Loads



Dead Loads for SIMON Model

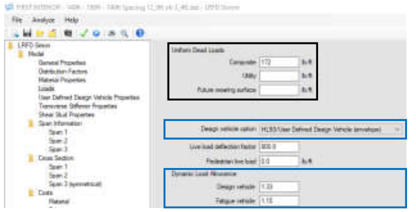
		Units	140-180-140	
			460	
General Parameters				
No. of Girders		ea	5	
Girder Spacing		ft	12.00	
Deck Width		ft	54.67	
Deck Thickness (includes 1/2" SC)		in	8.5	
Concrete Unit Weight		pcf	150	
SIP Forms		psf	20	
Avg Top Flange Width		in	17.0	
Avg Top Flange Thickness		in	1.0	
Avg Hauch Buildup		in	2.0	
Single Girder Wt (Steel Span Wt curves)	30	lb/ft	328	Total steel/gdr
Girder Wt. (Simon)	26	lb/ft	286	-15%
Non-Composite Loads				
Deck Concrete (tribA)		lb/ft	1275	
SIP Forms (tribA)		lb/ft	212	1487
Deck Concrete (equal)		lb/ft	1162	
SIP Forms (projected plan area of forms)		lb/ft	169	1331 -10%
Haunch Concrete		lb/ft	35	
Cross Frames + misc (see note 1)	15%	lb/ft	43	Assume
Total Non-Composite Load		lb/ft	1409	
Composite Loads				
36-inch single slope railings (distrib. evenly)	430	lb/ft	172	

All Girders


Three or More Spans — 11 ft and Greater Girder Spacing

Assumptions
- Section is designed as composite.
- Girders are assumed continuous.
- Design considers fatigue loading.
- Span lengths are based upon the maximum span distance. Where more than one span exists, use the maximum span to determine span weight.
- Trend line value represents the line of best fit based upon the discrete values.
- Shaded area represents deck areas in which 68% of the sample bridges are located.
- Both curved and straight girders are included in the curves.

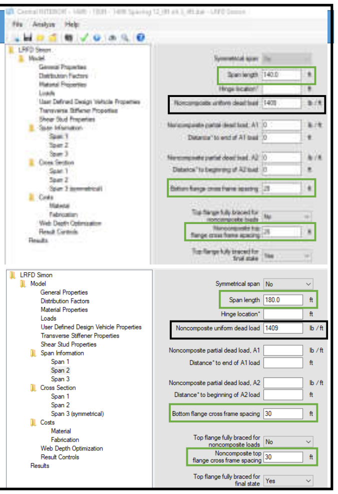
LRFD Simon Inputs

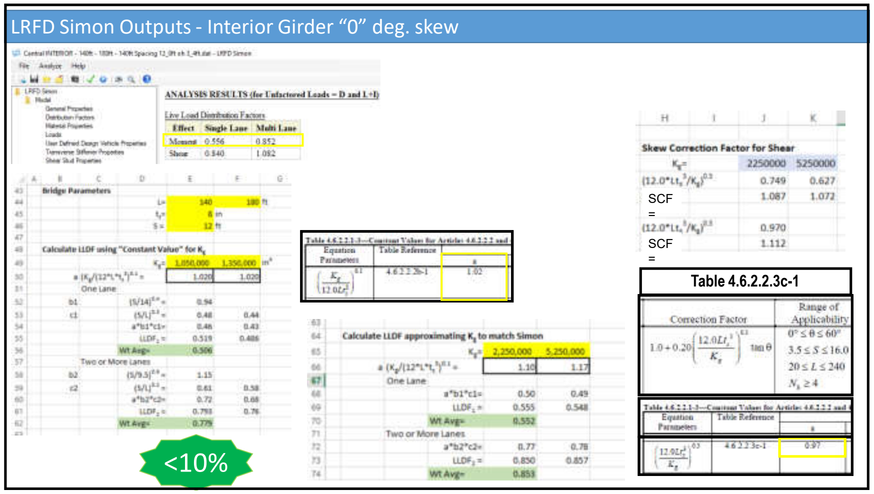
Live Loads

Geometry

Dead Loads



LRFD Simon Program Assumptions:
1. Flange lateral bending stresses are not considered at the service, fatigue or strength limit states.
2. The optional provisions of Appendix A6 are not considered for composite I-sections in negative flexure.
3. The optional moment redistribution provisions of Appendix B6 are not considered.



LRFD Simon Outputs - Interior Girder "0" deg. skew

Central INTERIOR - 140ft - 180ft - 140ft Spacing 12_0ft oh 3_4ft.dat - LRFD Simon

ANALYSIS RESULTS (for Unfactored Loads = D and L+I)

Live Load Distribution Factors

Effect	Single Lane	Multi Lane
Moment	0.556	0.852
Shear	0.840	1.082

Bridge Parameters

	L=	140	180 ft
	t_s=	8 in	
	S =	12 ft	

Calculate LLDF using "Constant Value" for K_g

	K_g=	1,050,000	1,350,000 in^4
	a $(K_g/(12*L*t_s^3))^{0.1}$ =	1.020	1.020
One Lane			
b1	$(S/14)^{0.4}$ =	0.94	
c1	$(S/L)^{0.3}$ =	0.48	0.44
	a*b1*c1=	0.46	0.43
	$LLDF_1$ =	0.519	0.486
	Wt Avg=	0.506	
Two or More Lanes			
b2	$(S/9.5)^{0.6}$ =	1.15	
c2	$(S/L)^{0.2}$ =	0.61	0.58
	a*b2*c2=	0.72	0.68
	$LLDF_2$ =	0.793	0.76
	Wt Avg=	0.779	

<10%

Table 4.6.2.2.1-3—Constant Values for Articles 4.6.2.2.2 and

Equation Parameters	Table Reference	a
$\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$	4.6.2.2.2b-1	1.02

Calculate LLDF approximating K_g to match Simon

	K_g=	2,250,000	5,250,000
	a $(K_g/(12*L*t_s^3))^{0.1}$ =	1.10	1.17
One Lane			
	a*b1*c1=	0.50	0.49
	$LLDF_1$ =	0.555	0.548
	Wt Avg=	0.552	
Two or More Lanes			
	a*b2*c2=	0.77	0.78
	$LLDF_2$ =	0.850	0.857
	Wt Avg=	0.853	

Skew Correction Factor for Shear

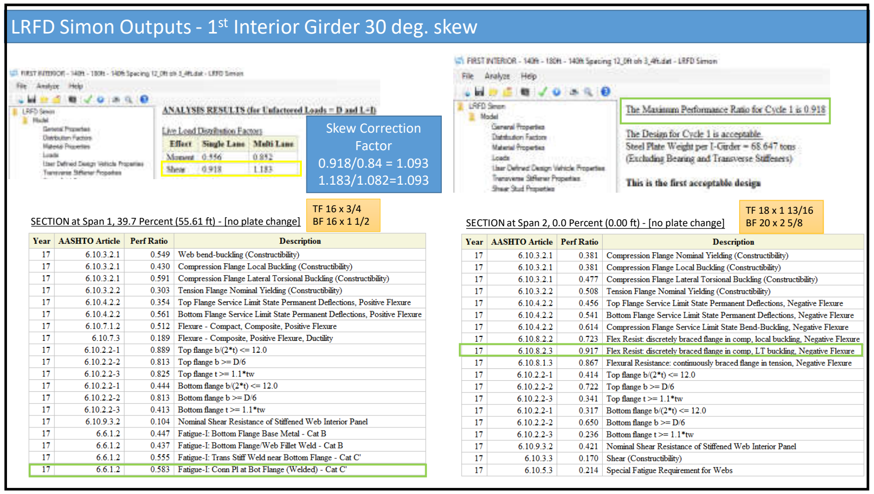
K_g=	2250000	5250000
$(12.0*L t_s^3/K_g)^{0.3}$	0.749	0.627
SCF =	1.087	1.072
$(12.0*L t_s^3/K_g)^{0.3}$	0.970	
SCF =	1.112	

Table 4.6.2.2.3c-1

Correction Factor	Range of Applicability
$1.0+0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$	$0° \le \theta \le 60°$ $3.5 \le S \le 16.0$ $20 \le L \le 240$ $N_b \ge 4$

Table 4.6.2.2.1-3—Constant Values for Articles 4.6.2.2.2 and

Equation Parameters	Table Reference	a
$\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}$	4.6.2.2.3c-1	0.97



LRFD Simon Outputs - 1st Interior Girder 30 deg. skew

FIRST INTERIOR - 140ft - 180ft - 140ft Spacing 12_0ft oh 3_4ft.dat - LRFD Simon

ANALYSIS RESULTS (for Unfactored Loads = D and L+I)

Live Load Distribution Factors

Effect	Single Lane	Multi Lane
Moment	0.556	0.852
Shear	0.918	1.183

Skew Correction Factor
0.918/0.84 = 1.093
1.183/1.082=1.093

The Maximum Performance Ratio for Cycle 1 is 0.918

The Design for Cycle 1 is acceptable.
Steel Plate Weight per I-Girder = 68.647 tons
(Excluding Bearing and Transverse Stiffeners)

This is the first acceptable design

SECTION at Span 1, 39.7 Percent (55.61 ft) - [no plate change]

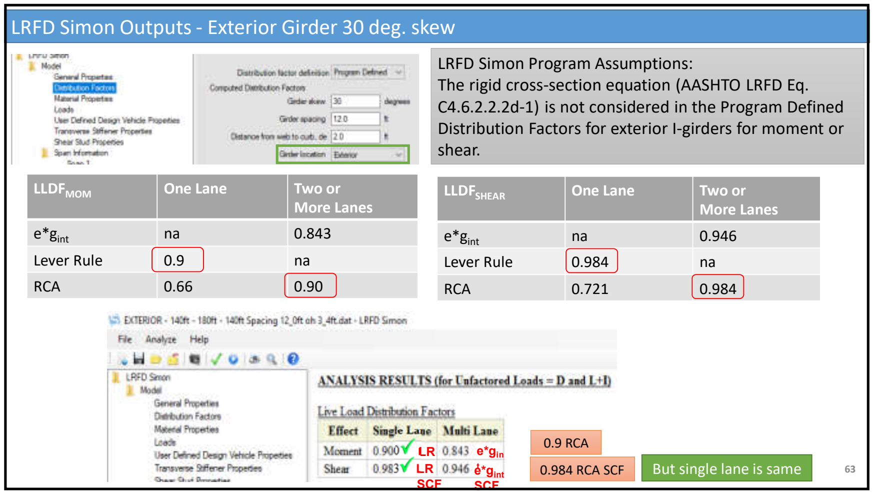
TF 16 x 3/4
BF 16 x 1 1/2

Year	AASHTO Article	Perf Ratio	Description
17	6.10.3.2.1	0.549	Web bend-buckling (Constructibility)
17	6.10.3.2.1	0.430	Compression Flange Local Buckling (Constructibility)
17	6.10.3.2.1	0.591	Compression Flange Lateral Torsional Buckling (Constructibility)
17	6.10.3.2.2	0.303	Tension Flange Nominal Yielding (Constructibility)
17	6.10.4.2.2	0.354	Top Flange Service Limit State Permanent Deflections, Positive Flexure
17	6.10.4.2.2	0.561	Bottom Flange Service Limit State Permanent Deflections, Positive Flexure
17	6.10.7.1.2	0.512	Flexure - Compact, Composite, Positive Flexure
17	6.10.7.3	0.189	Flexure - Composite, Positive Flexure, Ductility
17	6.10.2.2-1	0.889	Top flange b/(2*t) <= 12.0
17	6.10.2.2-2	0.813	Top flange b >= D/6
17	6.10.2.2-3	0.825	Top flange t >= 1.1*tw
17	6.10.2.2-1	0.444	Bottom flange b/(2*t) <= 12.0
17	6.10.2.2-2	0.813	Bottom flange b >= D/6
17	6.10.2.2-3	0.413	Bottom flange t >= 1.1*tw
17	6.10.9.3.2	0.104	Nominal Shear Resistance of Stiffened Web Interior Panel
17	6.6.1.2	0.447	Fatigue-I: Bottom Flange Base Metal - Cat B
17	6.6.1.2	0.437	Fatigue-I: Bottom Flange/Web Fillet Weld - Cat B
17	6.6.1.2	0.555	Fatigue-I: Trans Stiff Weld near Bottom Flange - Cat C'
17	6.6.1.2	0.583	Fatigue-I: Conn Pl at Bot Flange (Welded) - Cat C'

SECTION at Span 2, 0.0 Percent (0.00 ft) - [no plate change]

TF 18 x 1 13/16
BF 20 x 2 5/8

Year	AASHTO Article	Perf Ratio	Description
17	6.10.3.2.1	0.381	Compression Flange Nominal Yielding (Constructibility)
17	6.10.3.2.1	0.381	Compression Flange Local Buckling (Constructibility)
17	6.10.3.2.1	0.477	Compression Flange Lateral Torsional Buckling (Constructibility)
17	6.10.3.2.2	0.508	Tension Flange Nominal Yielding (Constructibility)
17	6.10.4.2.2	0.456	Top Flange Service Limit State Permanent Deflections, Negative Flexure
17	6.10.4.2.2	0.541	Bottom Flange Service Limit State Permanent Deflections, Negative Flexure
17	6.10.4.2.2	0.614	Compression Flange Service Limit State Bend-Buckling, Negative Flexure
17	6.10.8.2.2	0.723	Flex Resist: discretely braced flange in comp, local buckling, Negative Flexure
17	6.10.8.2.3	0.917	Flex Resist: discretely braced flange in comp, LT buckling, Negative Flexure
17	6.10.8.1.3	0.867	Flexural Resistance: continuously braced flange in tension, Negative Flexure
17	6.10.2.2-1	0.414	Top flange b/(2*t) <= 12.0
17	6.10.2.2-2	0.722	Top flange b >= D/6
17	6.10.2.2-3	0.341	Top flange t >= 1.1*tw
17	6.10.2.2-1	0.317	Bottom flange b/(2*t) <= 12.0
17	6.10.2.2-2	0.650	Bottom flange b >= D/6
17	6.10.2.2-3	0.236	Bottom flange t >= 1.1*tw
17	6.10.9.3.2	0.421	Nominal Shear Resistance of Stiffened Web Interior Panel
17	6.10.3.3	0.170	Shear (Constructibility)
17	6.10.5.3	0.214	Special Fatigue Requirement for Webs



LRFD Simon Outputs - Exterior Girder 30 deg. skew

Distribution factor definition: Program Defined
Computed Distribution Factors
Girder skew: 30 degrees
Girder spacing: 12.0 ft
Distance from web to curb, de: 2.0 ft
Girder location: Exterior

LRFD Simon Program Assumptions:
The rigid cross-section equation (AASHTO LRFD Eq. C4.6.2.2.2d-1) is not considered in the Program Defined Distribution Factors for exterior I-girders for moment or shear.

$LLDF_{MOM}$	One Lane	Two or More Lanes
$e*g_{int}$	na	0.843
Lever Rule	0.9	na
RCA	0.66	0.90

$LLDF_{SHEAR}$	One Lane	Two or More Lanes
$e*g_{int}$	na	0.946
Lever Rule	0.984	na
RCA	0.721	0.984

EXTERIOR - 140ft - 180ft - 140ft Spacing 12_0ft oh 3_4ft.dat - LRFD Simon

ANALYSIS RESULTS (for Unfactored Loads = D and L+I)

Live Load Distribution Factors

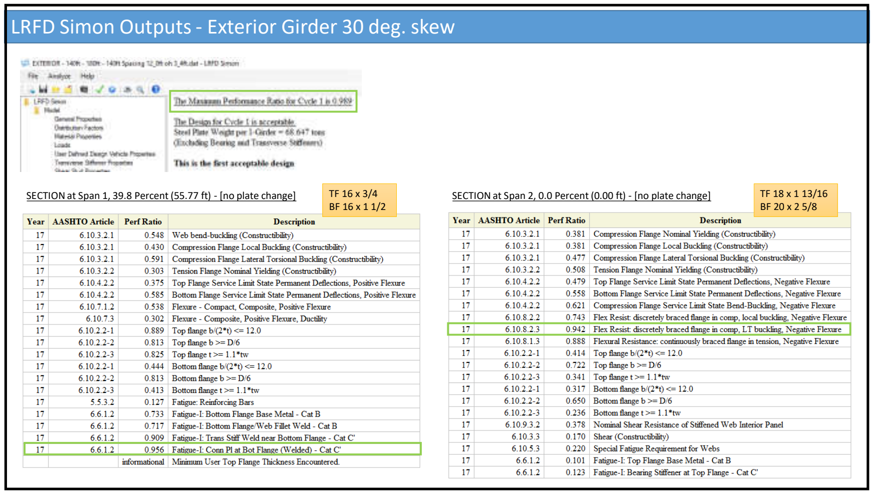
Effect	Single Lane	Multi Lane
Moment	0.900 ✓ LR	0.843 $e*g_{in}$
Shear	0.983 ✓ LR SCF	0.946 $e*g_{int}$ SCF

0.9 RCA

0.984 RCA SCF

But single lane is same

63



LRFD Simon Outputs - Exterior Girder 30 deg. skew

EXTERIOR - 140ft - 180ft - 140ft Spacing 12_0ft oh 3_4ft.dat - LRFD Simon

The Maximum Performance Ratio for Cycle 1 is 0.989

The Design for Cycle 1 is acceptable.
Steel Plate Weight per I-Girder = 68.647 tons
(Excluding Bearing and Transverse Stiffeners)

This is the first acceptable design

SECTION at Span 1, 39.8 Percent (55.77 ft) - [no plate change]

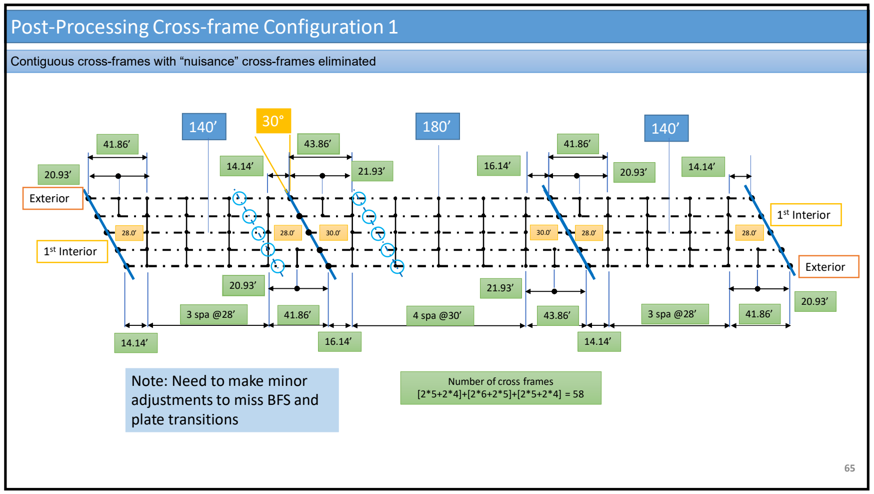
TF 16 x 3/4
BF 16 x 1 1/2

Year	AASHTO Article	Perf Ratio	Description
17	6.10.3.2.1	0.548	Web bend-buckling (Constructibility)
17	6.10.3.2.1	0.430	Compression Flange Local Buckling (Constructibility)
17	6.10.3.2.1	0.591	Compression Flange Lateral Torsional Buckling (Constructibility)
17	6.10.3.2.2	0.303	Tension Flange Nominal Yielding (Constructibility)
17	6.10.4.2.2	0.375	Top Flange Service Limit State Permanent Deflections, Positive Flexure
17	6.10.4.2.2	0.585	Bottom Flange Service Limit State Permanent Deflections, Positive Flexure
17	6.10.7.1.2	0.538	Flexure - Compact, Composite, Positive Flexure
17	6.10.7.3	0.302	Flexure - Composite, Positive Flexure, Ductility
17	6.10.2.2-1	0.889	Top flange b/(2*t) <= 12.0
17	6.10.2.2-2	0.813	Top flange b >= D/6
17	6.10.2.2-3	0.825	Top flange t >= 1.1*tw
17	6.10.2.2-1	0.444	Bottom flange b/(2*t) <= 12.0
17	6.10.2.2-2	0.813	Bottom flange b >= D/6
17	6.10.2.2-3	0.413	Bottom flange t >= 1.1*tw
17	5.5.3.2	0.127	Fatigue: Reinforcing Bars
17	6.6.1.2	0.733	Fatigue-I: Bottom Flange Base Metal - Cat B
17	6.6.1.2	0.717	Fatigue-I: Bottom Flange/Web Fillet Weld - Cat B
17	6.6.1.2	0.909	Fatigue-I: Trans Stiff Weld near Bottom Flange - Cat C'
17	6.6.1.2	0.956	Fatigue-I: Conn Pl at Bot Flange (Welded) - Cat C'
		informational	Minimum User Top Flange Thickness Encountered.

SECTION at Span 2, 0.0 Percent (0.00 ft) - [no plate change]

TF 18 x 1 13/16
BF 20 x 2 5/8

Year	AASHTO Article	Perf Ratio	Description
17	6.10.3.2.1	0.381	Compression Flange Nominal Yielding (Constructibility)
17	6.10.3.2.1	0.381	Compression Flange Local Buckling (Constructibility)
17	6.10.3.2.1	0.477	Compression Flange Lateral Torsional Buckling (Constructibility)
17	6.10.3.2.2	0.508	Tension Flange Nominal Yielding (Constructibility)
17	6.10.4.2.2	0.479	Top Flange Service Limit State Permanent Deflections, Negative Flexure
17	6.10.4.2.2	0.558	Bottom Flange Service Limit State Permanent Deflections, Negative Flexure
17	6.10.4.2.2	0.621	Compression Flange Service Limit State Bend-Buckling, Negative Flexure
17	6.10.8.2.2	0.743	Flex Resist: discretely braced flange in comp, local buckling, Negative Flexure
17	6.10.8.2.3	0.942	Flex Resist: discretely braced flange in comp, LT buckling, Negative Flexure
17	6.10.8.1.3	0.888	Flexural Resistance: continuously braced flange in tension, Negative Flexure
17	6.10.2.2-1	0.414	Top flange b/(2*t) <= 12.0
17	6.10.2.2-2	0.722	Top flange b >= D/6
17	6.10.2.2-3	0.341	Top flange t >= 1.1*tw
17	6.10.2.2-1	0.317	Bottom flange b/(2*t) <= 12.0
17	6.10.2.2-2	0.650	Bottom flange b >= D/6
17	6.10.2.2-3	0.236	Bottom flange t >= 1.1*tw
17	6.10.9.3.2	0.378	Nominal Shear Resistance of Stiffened Web Interior Panel
17	6.10.3.3	0.170	Shear (Constructibility)
17	6.10.5.3	0.220	Special Fatigue Requirement for Webs
17	6.6.1.2	0.101	Fatigue-I: Top Flange Base Metal - Cat B
17	6.6.1.2	0.123	Fatigue-I: Bearing Stiffener at Top Flange - Cat C'



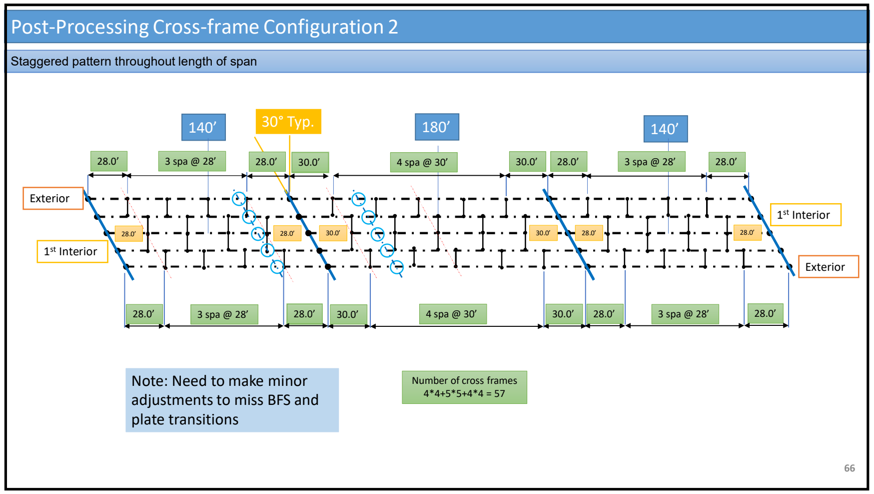
Post-Processing Cross-frame Configuration 1

Contiguous cross-frames with "nuisance" cross-frames eliminated

Note: Need to make minor adjustments to miss BFS and plate transitions

Number of cross frames
[2*5+2*4]+[2*6+2*5]+[2*5+2*4] = 58

65



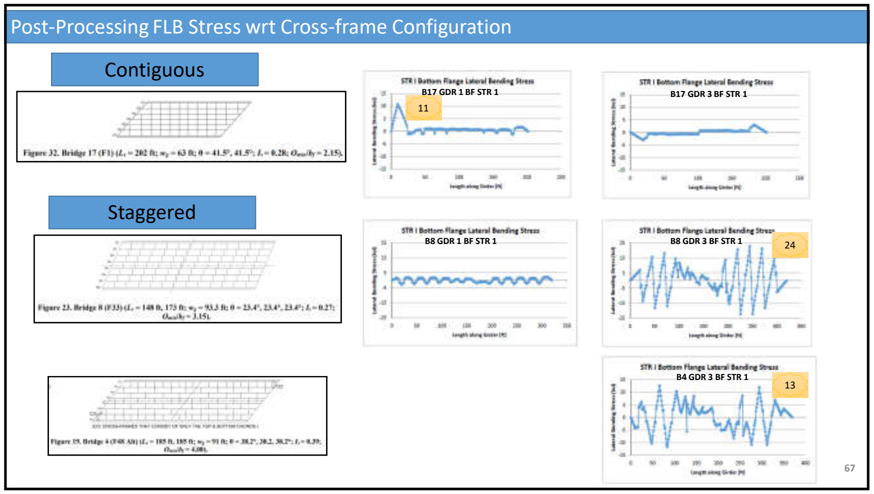
Post-Processing Cross-frame Configuration 2

Staggered pattern throughout length of span

Note: Need to make minor adjustments to miss BFS and plate transitions

Number of cross frames
4*4+5*5+4*4 = 57

66



Post-Processing FLB Stress wrt Cross-frame Configuration

Contiguous

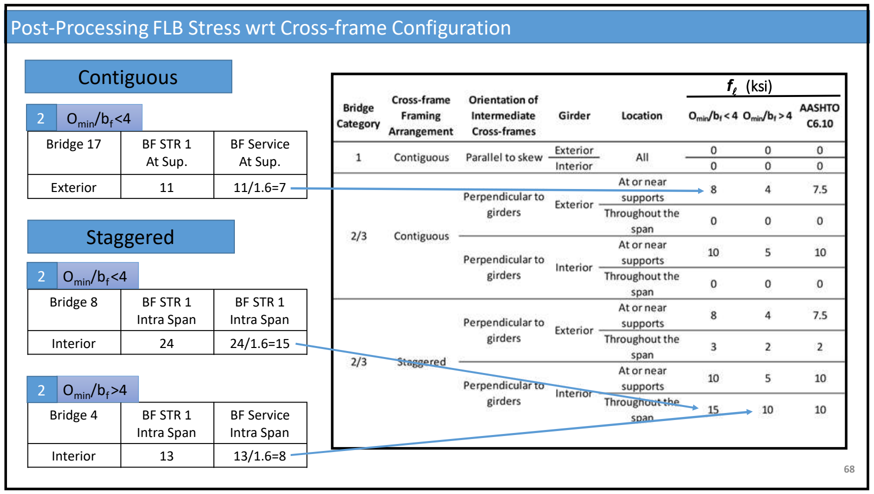
Figure 32. Bridge 17 (F1) (L_s = 202 ft; w_g = 63 ft; θ = 41.5°, 41.5°; I_s = 0.28; O_{min}/b_f = 2.15).

Staggered

Figure 23. Bridge 8 (F33) (L_s = 148 ft, 173 ft; w_g = 93.3 ft; θ = 23.4°, 23.4°, 23.4°; I_s = 0.27; O_{min}/b_f = 3.15).

Figure 19. Bridge 4 (F48 A8) (L_s = 185 ft, 185 ft; w_g = 91 ft; θ = 38.2°, 38.2, 38.2°; I_s = 0.39; O_{min}/b_f = 4.00).

67



Post-Processing FLB Stress wrt Cross-frame Configuration

Contiguous

2	O_{min}/b_f<4	
Bridge 17	BF STR 1 At Sup.	BF Service At Sup.
Exterior	11	11/1.6=7

Staggered

2	O_{min}/b_f<4	
Bridge 8	BF STR 1 Intra Span	BF STR 1 Intra Span
Interior	24	24/1.6=15

2	O_{min}/b_f>4	
Bridge 4	BF STR 1 Intra Span	BF Service Intra Span
Interior	13	13/1.6=8

Bridge Category	Cross-frame Framing Arrangement	Orientation of Intermediate Cross-frames	Girder	Location	f_ℓ (ksi) O_{min}/b_f < 4	f_ℓ (ksi) O_{min}/b_f > 4	f_ℓ (ksi) AASHTO C6.10
1	Contiguous	Parallel to skew	Exterior	All	0	0	0
			Interior		0	0	0
2/3	Contiguous	Perpendicular to girders	Exterior	At or near supports	8	4	7.5
				Throughout the span	0	0	0
		Perpendicular to girders	Interior	At or near supports	10	5	10
				Throughout the span	0	0	0
2/3	Staggered	Perpendicular to girders	Exterior	At or near supports	8	4	7.5
				Throughout the span	3	2	2
		Perpendicular to girders	Interior	At or near supports	10	5	10
				Throughout the span	15	10	10

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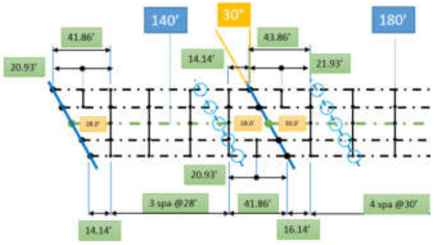
Post-Processing FLB Stress wrt Cross-frame Configuration

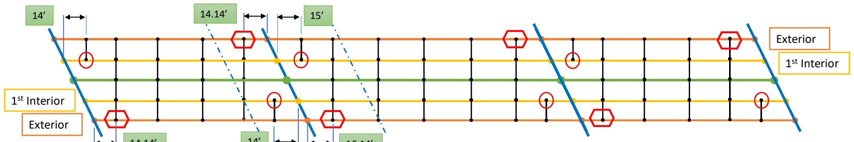


Bridge Category	Cross-frame Framing Arrangement	Orientation of Intermediate Cross-frames	Girder	Location	f_ℓ (ksi) $O_{min}/b_f < 4$	$O_{min}/b_f > 4$	AASHTO C6.10
2/3	Contiguous	Perpendicular to girders	Exterior	At or near supports	8	4	7.5
				Throughout the span	0	0	0
		Perpendicular to girders	Interior	At or near supports	10	5	10
				Throughout the span	0	0	0

Cross-frame Configuration 1 - O_{min}/b_f

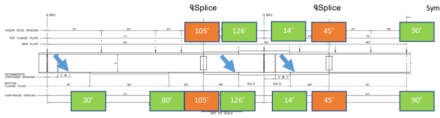
	O_{min}/b_f		
Support	EB 1	P2	
Girder		Back	Ahead
Exterior	14.14(12)/16= 11	9	16.14(12)/20=10
1st Interior	14(12)/16=10.5	15(12)/20=9	9
Other Interior	n/a	n/a	n/a

Post-Processing FLB Stress wrt Cross-frame Configuration



LEGEND

- ⬡ 4 ksi FLB Stress (Exterior Girder)
- ○ 5 ksi FLB Stress (Interior Girder)



Bridge Category	Cross-frame Framing Arrangement	Orientation of Intermediate Cross-frames	Girder	Location	f_ℓ (ksi) $O_{min}/b_f < 4$	$O_{min}/b_f > 4$	AASHTO C6.10
2/3	Contiguous	Perpendicular to girders	Exterior	At or near supports	8	4	7.5
				Throughout the span	0	0	0
		Perpendicular to girders	Interior	At or near supports	10	5	10
				Throughout the span	0	0	0

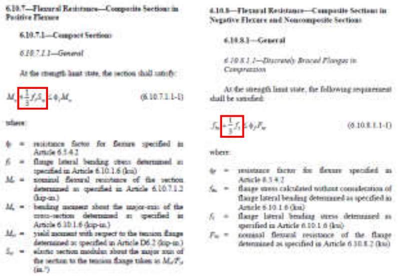
Strength, Service and Fatigue

Strength I
Service II
Fatigue I or II

Table 12. Recommended weighted average load factors for estimation of girder flange lateral bending stresses due to skew effects in straight I-girder bridges.

Load Combination Limit State	Weighted Average Load Factor Applied to Article C6.10.1 Coarse Estimate of f_ℓ
Strength I	1.6
Strength II	1.3
Strength III	1.3
Strength V	1.3
Additional load combination for DC + construction loads, Article 3.4.2.1	1.4
Service I	1.0
Service II	1.2
Fatigue I	1.75
Fatigue II	0.8

Limit State Strength I



6.10.7—Flexural Resistance—Composite Sections in Positive Flexure

6.10.7.1—Compact Sections

6.10.7.1.1—General

At the strength limit state, the section shall satisfy:

$$M_u + \boxed{\frac{1}{3} f_\ell S_{xt}} \le \phi_f M_n \qquad (6.10.7.1.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)
M_n = nominal flexural resistance of the section determined as specified in Article 6.10.7.1.2 (kip-in.)
M_u = bending moment about the major-axis of the cross-section determined as specified in Article 6.10.1.6 (kip-in.)
M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)
S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.³)

6.10.8—Flexural Resistance—Composite Sections in Negative Flexure and Noncomposite Sections

6.10.8.1—General

6.10.8.1.1—Discretely Braced Flanges in Compression

At the strength limit state, the following requirement shall be satisfied:

$$f_{bu} + \boxed{\frac{1}{3} f_\ell} \le \phi_f F_{nc} \qquad (6.10.8.1.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)
f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)
F_{nc} = nominal flexural resistance of the flange determined as specified in Article 6.10.8.2 (ksi)

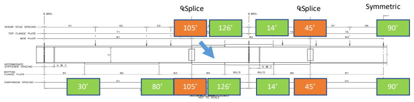
EXTERIOR GIRDER

SECTION at Span 1, 90.0 Percent (126.00 ft) - [left side checked]

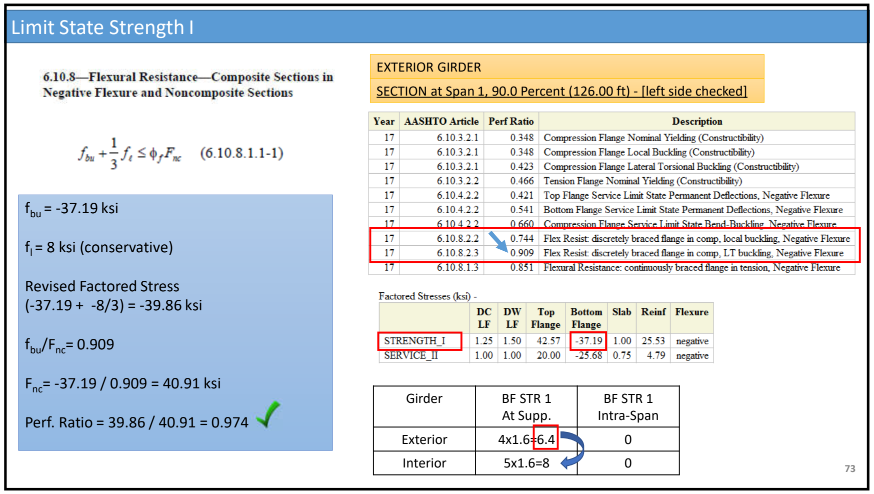
Factored Stresses (ksi) -

	DC LF	DW LF	Top Flange	Bottom Flange	Slab	Reinf	Flexure
STRENGTH_I	1.25	1.50	42.57	-37.19	1.00	25.53	negative

Year	AASHTO Article	Perf Ratio	Description
17	6.10.3.2.1	0.348	Compression Flange Nominal Yielding (Constructibility)
17	6.10.3.2.1	0.348	Compression Flange Local Buckling (Constructibility)
17	6.10.3.2.1	0.423	Compression Flange Lateral Torsional Buckling (Constructibility)
17	6.10.3.2.2	0.466	Tension Flange Nominal Yielding (Constructibility)
17	6.10.4.2.2	0.421	Top Flange Service Limit State Permanent Deflections, Negative Flexure
17	6.10.4.2.2	0.541	Bottom Flange Service Limit State Permanent Deflections, Negative Flexure
17	6.10.4.2.2	0.660	Compression Flange Service Limit State Bend-Buckling, Negative Flexure
17	6.10.8.2.2	0.744	Flex Resist: discretely braced flange in comp, local buckling, Negative Flexure
17	6.10.8.2.3	0.909	Flex Resist: discretely braced flange in comp, LT buckling, Negative Flexure
17	6.10.8.1.3	0.851	Flexural Resistance: continuously braced flange in tension, Negative Flexure



Girder	BF STR 1 At Supp.	BF STR 1 Intra-Span
Exterior	4x1.6=6.4	0
Interior	5x1.6=8	0



Limit State Strength I

6.10.8—Flexural Resistance—Composite Sections in Negative Flexure and Noncomposite Sections

$$f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nc} \quad (6.10.8.1.1\text{-}1)$$

f_{bu} = -37.19 ksi

f_l = 8 ksi (conservative)

Revised Factored Stress
(-37.19 + -8/3) = -39.86 ksi

f_{bu}/F_{nc}= 0.909

F_{nc}= -37.19 / 0.909 = 40.91 ksi

Perf. Ratio = 39.86 / 40.91 = 0.974 ✓

EXTERIOR GIRDER

SECTION at Span 1, 90.0 Percent (126.00 ft) - [left side checked]

Year	AASHTO Article	Perf Ratio	Description
17	6.10.3.2.1	0.348	Compression Flange Nominal Yielding (Constructibility)
17	6.10.3.2.1	0.348	Compression Flange Local Buckling (Constructibility)
17	6.10.3.2.1	0.423	Compression Flange Lateral Torsional Buckling (Constructibility)
17	6.10.3.2.2	0.466	Tension Flange Nominal Yielding (Constructibility)
17	6.10.4.2.2	0.421	Top Flange Service Limit State Permanent Deflections, Negative Flexure
17	6.10.4.2.2	0.541	Bottom Flange Service Limit State Permanent Deflections, Negative Flexure
17	6.10.4.2.2	0.660	Compression Flange Service Limit State Bend-Buckling, Negative Flexure
17	6.10.8.2.2	0.744	Flex Resist: discretely braced flange in comp, local buckling, Negative Flexure
17	6.10.8.2.3	0.909	Flex Resist: discretely braced flange in comp, LT buckling, Negative Flexure
17	6.10.8.1.3	0.851	Flexural Resistance: continuously braced flange in tension, Negative Flexure

Factored Stresses (ksi) -

	DC LF	DW LF	Top Flange	Bottom Flange	Slab	Reinf	Flexure
STRENGTH_I	1.25	1.50	42.57	-37.19	1.00	25.53	negative
SERVICE_II	1.00	1.00	20.00	-25.68	0.75	4.79	negative

Girder	BF STR 1 At Supp.	BF STR 1 Intra-Span
Exterior	4x1.6=6.4	0
Interior	5x1.6=8	0

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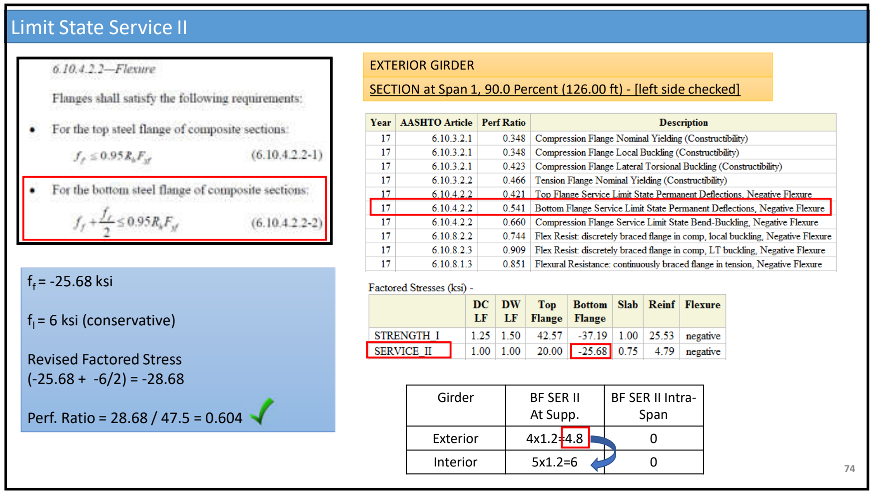

Limit State Service II

6.10.4.2.2—Flexure

Flanges shall satisfy the following requirements:

- For the top steel flange of composite sections:

$$f_f \le 0.95 R_h F_{yf} \quad (6.10.4.2.2\text{-}1)$$

- For the bottom steel flange of composite sections:

$$f_f + \frac{f_\ell}{2} \le 0.95 R_h F_{yf} \quad (6.10.4.2.2\text{-}2)$$

f_f = -25.68 ksi

f_l = 6 ksi (conservative)

Revised Factored Stress
(-25.68 + -6/2) = -28.68

Perf. Ratio = 28.68 / 47.5 = 0.604 ✓

EXTERIOR GIRDER

SECTION at Span 1, 90.0 Percent (126.00 ft) - [left side checked]

Year	AASHTO Article	Perf Ratio	Description
17	6.10.3.2.1	0.348	Compression Flange Nominal Yielding (Constructibility)
17	6.10.3.2.1	0.348	Compression Flange Local Buckling (Constructibility)
17	6.10.3.2.1	0.423	Compression Flange Lateral Torsional Buckling (Constructibility)
17	6.10.3.2.2	0.466	Tension Flange Nominal Yielding (Constructibility)
17	6.10.4.2.2	0.421	Top Flange Service Limit State Permanent Deflections, Negative Flexure
17	6.10.4.2.2	0.541	Bottom Flange Service Limit State Permanent Deflections, Negative Flexure
17	6.10.4.2.2	0.660	Compression Flange Service Limit State Bend-Buckling, Negative Flexure
17	6.10.8.2.2	0.744	Flex Resist: discretely braced flange in comp, local buckling, Negative Flexure
17	6.10.8.2.3	0.909	Flex Resist: discretely braced flange in comp, LT buckling, Negative Flexure
17	6.10.8.1.3	0.851	Flexural Resistance: continuously braced flange in tension, Negative Flexure

Factored Stresses (ksi) -

	DC LF	DW LF	Top Flange	Bottom Flange	Slab	Reinf	Flexure
STRENGTH_I	1.25	1.50	42.57	-37.19	1.00	25.53	negative
SERVICE_II	1.00	1.00	20.00	-25.68	0.75	4.79	negative

Girder	BF SER II At Supp.	BF SER II Intra-Span
Exterior	4x1.2=4.8	0
Interior	5x1.2=6	0

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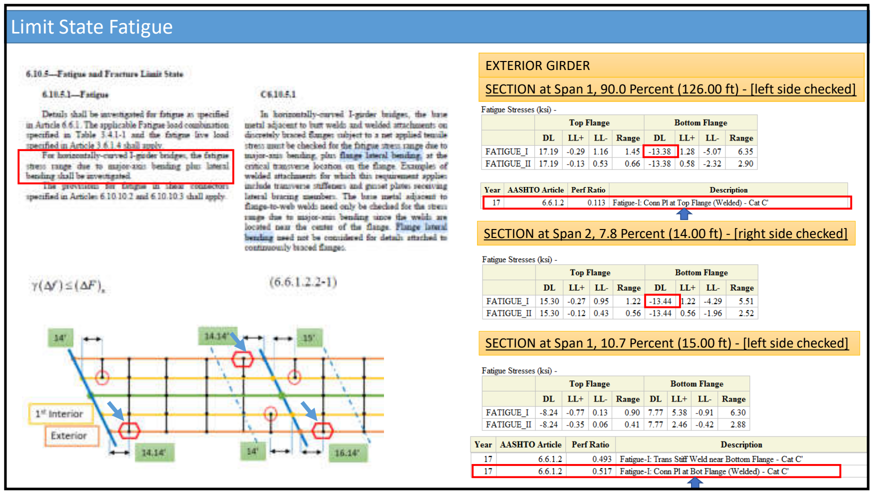


Limit State Fatigue

6.10.5—Fatigue and Fracture Limit State

6.10.5.1—Fatigue

Details shall be investigated for fatigue as specified in Article 6.6.1. The applicable Fatigue load combination specified in Table 3.4.1-1 and the fatigue live load specified in Article 3.6.1.4 shall apply.

For horizontally-curved I-girder bridges, the fatigue stress range due to major-axis bending plus lateral bending shall be investigated.

The provisions for fatigue in shear connectors specified in Articles 6.10.10.2 and 6.10.10.3 shall apply.

C6.10.5.1

In horizontally-curved I-girder bridges, the base metal adjacent to butt welds and welded attachments on discretely braced flanges subject to a net applied tensile stress must be checked for the fatigue stress range due to major-axis bending, plus flange lateral bending, at the critical transverse location on the flange. Examples of welded attachments for which this requirement applies include transverse stiffeners and gusset plates receiving lateral bracing members. The base metal adjacent to flange-to-web welds need only be checked for the stress range due to major-axis bending since the welds are located near the center of the flange. Flange lateral bending need not be considered for details attached to continuously braced flanges.

$$\gamma(\Delta f) \le (\Delta F)_n \quad (6.6.1.2.2\text{-}1)$$

EXTERIOR GIRDER

SECTION at Span 1, 90.0 Percent (126.00 ft) - [left side checked]

Fatigue Stresses (ksi) -

	Top Flange				Bottom Flange			
	DL	LL+	LL-	Range	DL	LL+	LL-	Range
FATIGUE_I	17.19	-0.29	1.16	1.45	-13.38	1.28	-5.07	6.35
FATIGUE_II	17.19	-0.13	0.53	0.66	-13.38	0.58	-2.32	2.90

Year	AASHTO Article	Perf Ratio	Description
17	6.6.1.2	0.113	Fatigue-I: Conn Pl at Top Flange (Welded) - Cat C'

SECTION at Span 2, 7.8 Percent (14.00 ft) - [right side checked]

Fatigue Stresses (ksi) -

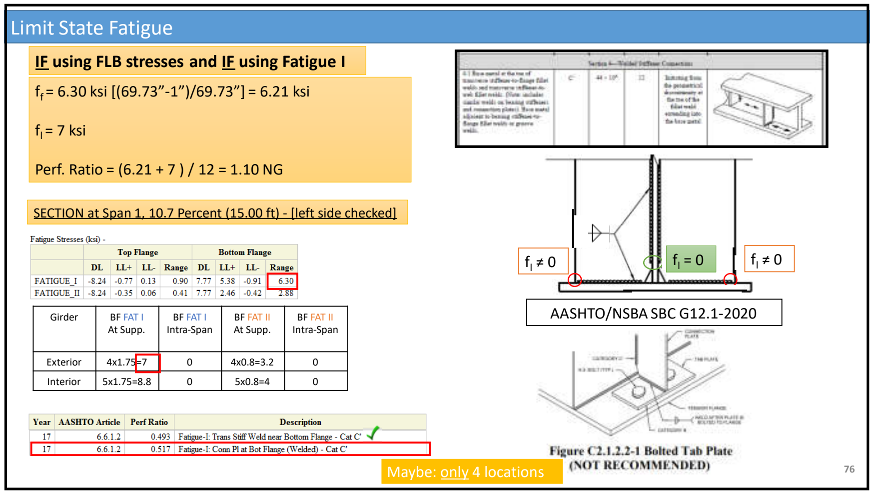
	Top Flange				Bottom Flange			
	DL	LL+	LL-	Range	DL	LL+	LL-	Range
FATIGUE_I	15.30	-0.27	0.95	1.22	-13.44	1.22	-4.29	5.51
FATIGUE_II	15.30	-0.12	0.43	0.56	-13.44	0.56	-1.96	2.52

SECTION at Span 1, 10.7 Percent (15.00 ft) - [left side checked]

Fatigue Stresses (ksi) -

	Top Flange				Bottom Flange			
	DL	LL+	LL-	Range	DL	LL+	LL-	Range
FATIGUE_I	-8.24	-0.77	0.13	0.90	7.77	5.38	-0.91	6.30
FATIGUE_II	-8.24	-0.35	0.06	0.41	7.77	2.46	-0.42	2.88

Year	AASHTO Article	Perf Ratio	Description
17	6.6.1.2	0.493	Fatigue-I: Trans Stiff Weld near Bottom Flange - Cat C'
17	6.6.1.2	0.517	Fatigue-I: Conn Pl at Bot Flange (Welded) - Cat C'



Limit State Fatigue

IF using FLB stresses and IF using Fatigue I

f_f = 6.30 ksi [(69.73”-1”)/69.73”] = 6.21 ksi

f_l = 7 ksi

Perf. Ratio = (6.21 + 7) / 12 = 1.10 NG

SECTION at Span 1, 10.7 Percent (15.00 ft) - [left side checked]

Fatigue Stresses (ksi) -

	Top Flange				Bottom Flange			
	DL	LL+	LL-	Range	DL	LL+	LL-	Range
FATIGUE_I	-8.24	-0.77	0.13	0.90	7.77	5.38	-0.91	6.30
FATIGUE_II	-8.24	-0.35	0.06	0.41	7.77	2.46	-0.42	2.88

Girder	BF FAT I At Supp.	BF FAT I Intra-Span	BF FAT II At Supp.	BF FAT II Intra-Span
Exterior	4x1.75=7	0	4x0.8=3.2	0
Interior	5x1.75=8.8	0	5x0.8=4	0

Year	AASHTO Article	Perf Ratio	Description
17	6.6.1.2	0.493	Fatigue-I: Trans Stiff Weld near Bottom Flange - Cat C' ✓
17	6.6.1.2	0.517	Fatigue-I: Conn Pl at Bot Flange (Welded) - Cat C'

Maybe: only 4 locations

AASHTO/NSBA SBC G12.1-2020

Figure C2.1.2.2-1 Bolted Tab Plate (NOT RECOMMENDED)

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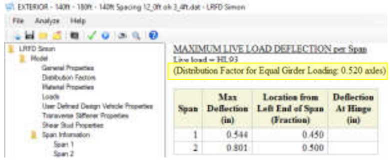
Limit State Service I Optional Live Load Deflection



MAXIMUM LIVE LOAD DEFLECTION per Span
Live load = HL93
(Distribution Factor for Equal Girder Loading: 0.520 axles)

Span	Max Deflection (in)	Location from Left End of Span (Fraction)	Deflection At Hinge (in)
1	0.544	0.450	
2	0.801	0.500	

LLDF = 0.65*4 lanes/ 5 girder
=0.52

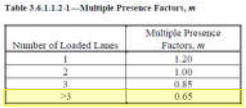
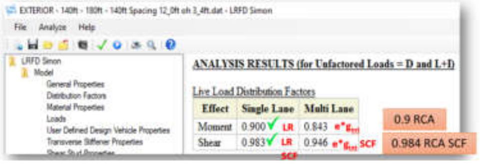


Table 3.6.1.1.2-1—Multiple Presence Factors, *m*

Number of Loaded Lanes	Multiple Presence Factors, *m*
1	1.20
2	1.00
3	0.85
>3	0.65



ANALYSIS RESULTS (for Unfactored Loads = D and L+I)

Live Load Distribution Factors

Effect	Single Lane	Multi Lane
Moment	0.900 ✓ LR	0.843 e*g_{int}
Shear	0.983 ✓ LR SCF	0.946 e*g_{int} SCF

0.9 RCA

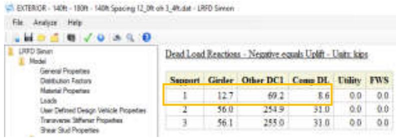
0.984 RCA SCF

Live Load Deflection
Span 1
0.544” * (0.9/0.52) = 1”
L/800 = 2.1”

Span 2
0.801” * (0.9/0.52) = 1.4”
L/800 = 2.7”

Exterior Girder Reactions

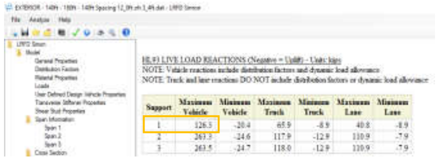
For Cases 2 and 3, increase STR I vertical reactions for the exterior girder at obtuse corners at end bents and at piers (in continuous bridges) by a multiplicative factor of 1.10 (this increase is in addition to the skew correction factor)



Dead Load Reactions - Negative equals Uplift - Units: kips

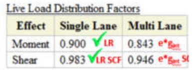
Support	Girder	Other DC1	Comp DL	Utility	FWS
1	12.7	69.2	8.6	0.0	0.0
2	56.0	254.9	31.0	0.0	0.0
3	56.1	255.0	31.0	0.0	0.0

90.5



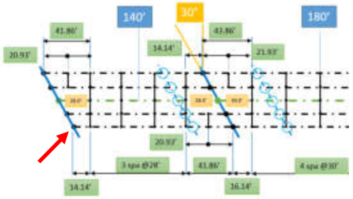
HL93 LIVE LOAD REACTIONS (Negative = Uplift) - Units: kips
NOTE: Vehicle reactions include distribution factors and dynamic load allowance
NOTE: Truck and lane reactions DO NOT include distribution factors or dynamic load allowance

Support	Maximum Vehicle	Minimum Vehicle	Maximum Truck	Minimum Truck	Maximum Lane	Minimum Lane
1	126.3	-20.4	65.9	-8.9	40.8	-8.9
2	263.3	-24.6	117.9	-12.9	110.9	-7.9
3	263.5	-24.7	118.0	-12.9	110.9	-7.9



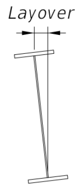
Live Load Distribution Factors

Effect	Single Lane	Multi Lane
Moment	0.900 ✓ LR	0.843 e*g_{int}
Shear	0.983 ✓ LR SCF	0.946 e*g_{int} S



At End Bent 1 (Obtuse corner)
Strength I
Rx_{STRI} = 1.10[1.25 DL + 1.75 (LL+I)]
= 1.10[1.25(90.5) + 1.75(126.3)]
= 1.10[334]
= 368

Girder Dead Load Layover



Layover

Steel dead load fit

Simplified Method to Determine Layovers at Girder Ends:

1. From the line girder analysis, determine the vertical deflections for the concrete deck.
2. Using the vertical deflection, calculate the Girder In-Plane Rotation Angle.

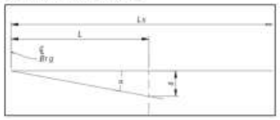
$$\alpha = \tan^{-1}\frac{\delta}{L}$$

3. Calculate the Girder Out-of-Plane Rotation

$$\varphi = \alpha \tan\theta$$

4. Calculate the Layover

$$Layover = D\tan\varphi$$



L_s = Span Length c/c Bearings

L = Length from CL Brg at End Bent to point where Deflection value is taken

δ = Total Deflection at L

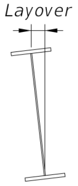
α = Girder In-Plane Rotation Angle

θ = Skew Angle

φ = Girder Out-of-Plane Rotation Angle

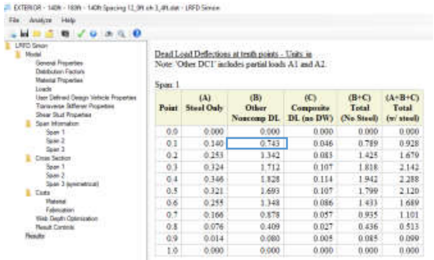
D = Web Depth

Girder Dead Load Layover



Layover

Steel dead load fit



Dead Load Deflections at tenth points - Units: in
Note: 'Other DC1' includes partial loads A1 and A2.

Span 1

Point	(A) Steel Only	(B) Other Noncomp DL	(C) Composite DL (no DW)	(B+C) Total (No Steel)	(A+B+C) Total (w/ steel)
0.0	0.000	0.000	0.000	0.000	0.000
0.1	0.140	0.743	0.046	0.789	0.928
0.2	0.253	1.342	0.083	1.425	1.679
0.3	0.324	1.712	0.107	1.818	2.142
0.4	0.346	1.828	0.114	1.942	2.288
0.5	0.321	1.693	0.107	1.799	2.120
0.6	0.255	1.348	0.086	1.433	1.689
0.7	0.166	0.878	0.057	0.935	1.101
0.8	0.076	0.409	0.027	0.436	0.513
0.9	0.014	0.080	0.005	0.085	0.099
1.0	0.000	0.000	0.000	0.000	0.000

$\alpha = \tan^{-1}\frac{\delta}{L}$ $\alpha = \tan^{-1}$ 0.743/(14x12) α = 0.25 deg.

$\varphi = \alpha\tan\theta$ $\varphi = \alpha\tan 30$ φ = 0.146 deg.

$Layover = D\tan\varphi$ $Layover = 78\tan 0.146$ $Layover$ = 0.2 inch

Cross-frame Forces

What are your State’s successful past practices?

- Standard member sizes
- Minimum Kl/r
- Wind
- Stability force (e.g. % flange force)
- Overhang bracket loads
- 20 kip force
  - AASHTO Article 6.6.1.3.1 D-I Fatigue, welded or bolted connection
- Forces derived from a refined analysis
- Others?
- Is fatigue considered?

Cross-frame Forces

Table 13. Cross-frame shears and chord level connection horizontal forces due to skew effects.

Cross-Frame Case	Load Effect	DC1 & Constr	DC2	DW	HL-93 LL	STR I& SER II	Fatigue LL*
(1i) Intermediate cross-frames using offsets and staggers greater than or equal to $4b_f$ throughout the span	$V_{max.ICF\text{-}}/V_{max.g}$	0.02	0.40	0.03	0.06	0.03	0.06
	$B_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.20	0.02	0.06	0.03	0.05
	$T_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.20	0.02	0.05	0.02	0.05
(2i) Contiguous intermediate cross-frames, or intermediate cross-frames with any offsets and staggers less than $4b_f$ within the span	$V_{max.ICF\text{-}}/V_{max.g}$	0.06	1.20	0.06	0.20	0.09	0.14
	$B_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.60	0.04	0.14	0.08	0.12
	$T_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.03	0.40	0.02	0.08	0.04	0.08
(1b) Bearing line cross-frames where the offset of intermediate cross-frames relative to the bearing line is greater than or equal to $4b_f$	$V_{max.BCF\text{-}}/V_{max.g}$	0.02	0.07	0.02	0.02	0.02	0.02
	$B_{max.cn.BCF}/(M_{max.g}/h_{cf})$	0.02	0.04	0.02	0.02	0.02	0.02
	$T_{max.cn.BCF}/(M_{max.g}/h_{cf})$	0.02	0.10	0.02	0.03	0.02	0.04
(2b) Bearing line cross-frames where the offset of intermediate cross-frames relative to the bearing line is smaller than $4b_f$	$V_{max.BCF\text{-}}/V_{max.g}$	0.04	0.12	0.06	0.06	0.04	0.06
	$B_{max.cn.BCF}/(M_{max.g}/h_{cf})$	0.02	0.14	0.02	0.08	0.05	0.07
	$T_{max.cn.BCF}/(M_{max.g}/h_{cf})$	0.02	0.14	0.02	0.03	0.02	0.05

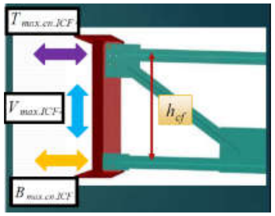
Cross-frame Forces

Cross-Frame Case	Load Effect	DC1 & Constr	DC2	DW	HL-93 LL	STR I& SER II	Fatigue LL*
(2i) Contiguous intermediate cross-frames, or intermediate cross-frames with any offsets and staggers less than $4b_f$ within the span	$V_{maxICF\text{-}}/V_{max.g}$	0.06	1.20	0.06	0.20	0.09	0.14
	$B_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.60	0.04	0.14	0.08	0.12
	$T_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.03	0.40	0.02	0.08	0.04	0.08

$V_{max.g}$ = maximum magnitude of the girder vertical shear force throughout the bridge span or spans under consideration, due to the force effect under consideration.

$M_{max.g}$ = maximum magnitude of the girder major-axis bending moments (positive or negative) throughout the bridge span or spans under consideration, due to the force effect under consideration.

h_{cf} =
- noncomposite condition: the distance between the cross-frame chords.
- composite condition: the distance between the mid-thickness of the bridge deck and the centroid of the cross-frame bottom chord



$V_{max.ICF}$ = maximum magnitude of the intermediate cross-frame shear force throughout the bridge span.

$B_{max.cn.ICF}$ = maximum magnitude horizontal force at the level of the bottom chord throughout the bridge span,

$T_{max.cn.ICF}$ = maximum magnitude horizontal force at the level of the top chord throughout the bridge span,

- where no cross-frame diagonals frame into the connection plates, equal to the maximum magnitude chord force
- where cross-frame diagonals frame into the connection plates, equal to the sum of the maximum magnitude chord force plus the horizontal component of the maximum magnitude diagonal axial force

Cross-frame Forces

Cross-Frame Case	Load Effect	DC1 & Constr	DC2	DW	HL-93 LL	STR I& SER II	Fatigue LL*
(2i) Contiguous intermediate cross-frames, or intermediate cross-frames with any offsets and staggers less than $4b_f$ within the span	$V_{maxICF\text{-}}/V_{max.g}$	0.06	1.20	0.06	0.20	0.09	0.14
	$B_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.60	0.04	0.14	0.08	0.12
	$T_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.03	0.40	0.02	0.08	0.04	0.08

EXTERIOR: SECTION at Span 2, 0.0 Percent (0.00 ft) - [no plate change]

Detailed Information -

		Rh	Rb	Cb	Dc (in)	My (kips-ft)	Mu (kips-ft)	Vu, max (kips)	Vu, min (kips)
STRENGTH_I	(negative)	1.000	1.000	1.000	37.1	19754.76	-13994.7	481.10	126.24

INTERIOR: SECTION at Span 2, 0.0 Percent (0.00 ft) - [no plate change]

Detailed Information -

		Rh	Rb	Cb	Dc (in)	My (kips-ft)	Mu (kips-ft)	Vu, max (kips)	Vu, min (kips)
STRENGTH_I	(negative)	1.000	1.000	1.000	37.1	19754.77	-13617.5	535.35	120.63

Strength I

V_{maxICF}/V_{maxg} = 0.09

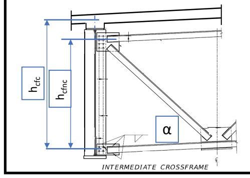
V_{maxICF}=0.09*535 = 48 kips

$B_{maxICF}/(M_{maxg}(12)/h_{cfc})$ = 0.08

B_{maxICF}= 0.08*(13995*12/80) = 168 kips

$T_{maxICF}/(M_{maxg}(12)/h_{cfc})$ = 0.04

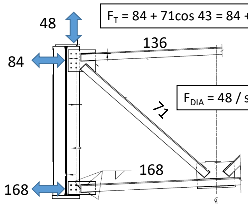
T_{maxICF}=0.04*(13995*12/80) = 84 kips



Web depth 78”
h_{cfnc} = 78-12 = 66 in
h_{cfc} = 78 – 6 + 4 + 8/2 = 80 in

α = $\tan^{-1}$ (66/72)
α = 43 deg.

F_T = 84 + 71cos 43 = 84 + 52 = 136 kips



F_{DIA} = 48 / sin 43 = 71 kips

Cross-frame Forces

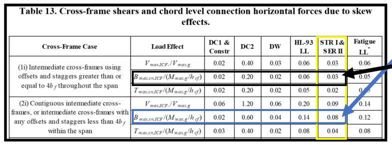


Table 13. Cross-frame shears and chord level connection horizontal forces due to skew effects.

Cross-Frame Case	Load Effect	DC1 & Constr	DC2	DW	HL-93 LL	STR I & SER II	Fatigue LL*
(1i) Intermediate cross-frames using offsets and staggers greater than or equal to $4b_f$ throughout the span	$V_{max.ICF-}/V_{max.g}$	0.02	0.40	0.03	0.06	0.03	0.06
	$B_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.20	0.02	0.06	0.03	0.05
	$T_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.20	0.02	0.05	0.02	[illegible]
(2i) Contiguous intermediate cross-frames, or intermediate cross-frames with any offsets and staggers less than $4b_f$ within the span	$V_{max.ICF-}/V_{max.g}$	0.06	1.20	0.06	0.20	0.09	0.14
	$B_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.02	0.60	0.04	0.14	0.08	0.12
	$T_{max.cn.ICF}/(M_{max.g}/h_{cf})$	0.03	0.40	0.02	0.08	0.04	0.08

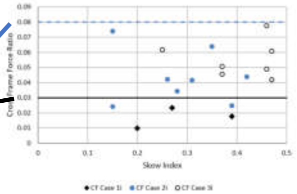


Figure 100. STR I bottom chord connection horizontal force ratio $B_{max.cn.ICF}/(M_{max.g}/h_{cf})$ for the intermediate cross-frames of the parallel skew bridges for cross-frame Cases 1i, 2i, and 3i versus the skew index.

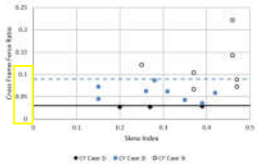
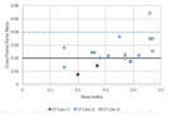


Figure 99. STR I shear force ratio $V_{max.ICF}/V_{max.g}$ for the intermediate cross-frames of the parallel skew bridges for cross-frame Cases 1i, 2i, and 3i versus the skew index.



FDOT Safety Message

Thank You

Wrap-up

CONCLUDING REMARKS

- LGA can be employed to obtain a fast, efficient and effective design solution for a wide range of straight skewed I-girder bridges with small to moderate skew
- The subject FDOT-supported research has clarified the behavior of this special class of bridges, and has provided recommendations that can facilitate the design and rating of these structures

THANK YOU FOR YOUR ATTENTION !

QUESTIONS?

PDH Certificates

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

Smarter.
Stronger.
Steel.

PDH Certificates

- Reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.

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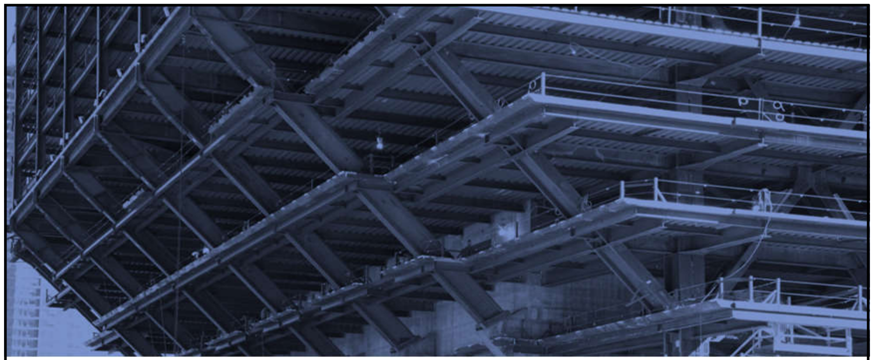

AISC | Thank you

Smarter.
Stronger.
Steel.