Deck Bulb Tee Girder Example

PGSuper Training

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1 Introduction

The purpose of this document is to illustrate how the PGSuper computer program performs its computations. PGSuper is a computer program for the design, analysis, and load rating of precast, prestressed concrete girder bridges.

A design evaluation followed by a load rating analysis illustrates the engineering computations performed by PGSuper. PGSuper uses a state-of-the-art iterative design algorithm and other iterative computational procedures. Only the final iterative steps are of interest. To avoid lengthy iterations in this document, trial variables are “guessed” based on the final iterations produced by the software.

PGSuper uses 16 decimals of precision. There will be minor differences between these “hand” calculations and numbers reported by PGSuper. When noted, these calculations adopt numeric values reported by PGSuper.

1.1 Sign Convention

This document and PGSuper use the following sign convention.

Item	Value
Compression	< 0
Tension	> 0
Upward Deflection	> 0
Downward Deflection	< 0
Top Section Modulus	< 0
Bottom Section Modulus	> 0
Strand Eccentricity above and right of Centroid	< 0
Strand Eccentricity below and left of Centroid	> 0

2 Bridge Description

2.1 Site Conditions

Normal Exposure

Average Ambient Relative Humidity: 75%

2.2 Roadway

Alignment

PI Station	Back Tangent	Delta	Radius
	N 90 E		

Profile

PVI Station	PVI Elevation	Grade in (g_1)	Grade out (g_2)	Length
95+00	100.00	1%		

Superelevations

Left	Right
$-0.02\frac{ft}{ft}$	$-0.02\frac{ft}{ft}$

2.3 Bridge Layout

This bridge is on a straight alignment with a slight uphill grade. There is a normal 2% roadway crown.

Back of Pavement Seat, Abutment 1, 95+00

Back of Pavement Seat, Abutment 2, 96+60

Abutments are Normal to the alignment

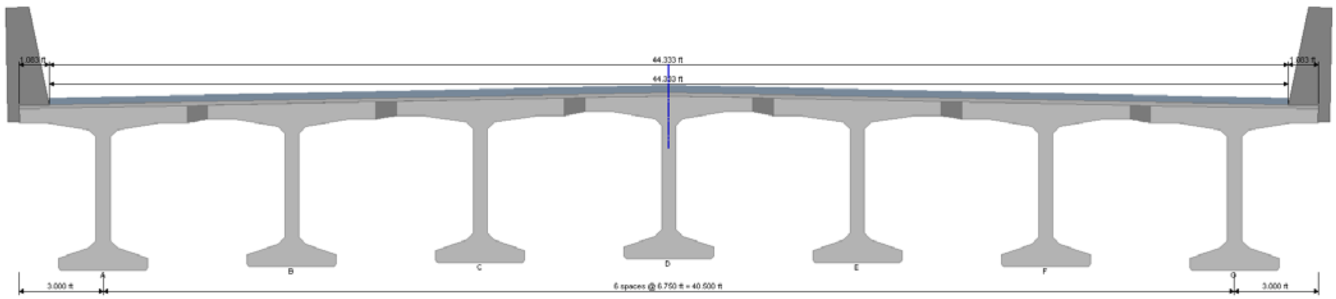


Figure 2-1: Bridge Section at Station 95+80.00

Girders

7 WF69DG with 6’-0” symmetric top width and 9” Joint Spacing

6.5” longitudinal top flange thickening at girder ends

Properties vary (given at CL Span)

$A = 1211.371\ in^2$
$I_x = 861860.5\ in^4$
$I_y = 251152.4\ in^4$
$I_{xy} = 17465.9\ in^4$
$X_l = 36.514$ in
$X_r = 35.486$ in
$Y_t = 28.797$ in
$Y_b = 41.643$ in
Perimeter = 326.738 in

$W_{tf} = 72.0$ in
$W_{bf} = 38.375$ in
$t_{web} = 6.125$ in

$f'_{ci} = 6.0$ ksi

$f'_c = 6.8$ ksi

Lightweight Concrete

$\gamma_c = 125\ lb/ft^3$
$\gamma_c = 130\ lb/ft^3$ (including rebar)

$K_1 = 0.9$

Pick Points 10.0ft

Bunk Points 9ft

Haul Configuration: HT60-72

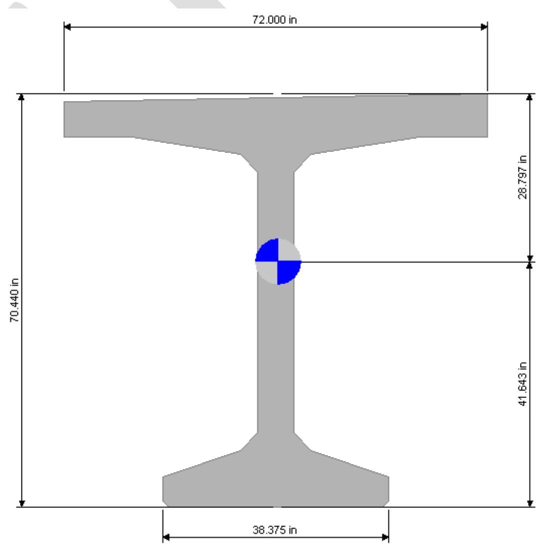


Figure 2-2: Girder Dimensions

Harping points at 0.4L from the end of the girder.

Interior Diaphragms

Rectangular – Between girders only.	H = 23 in T = 8.00 in	Located at $0.25L_s$, $0.50L_s$ and $0.75L_s$.

Non-structural Overlay

Gross Depth = 1.5 in
Slab Offset ("A" Dimension) = 2"
Sacrificial Depth = ½"
f'c = 4 ksi
γ_c = 140 lb/ft^3
Future Wearing Surface, 0.035 k/ft^2

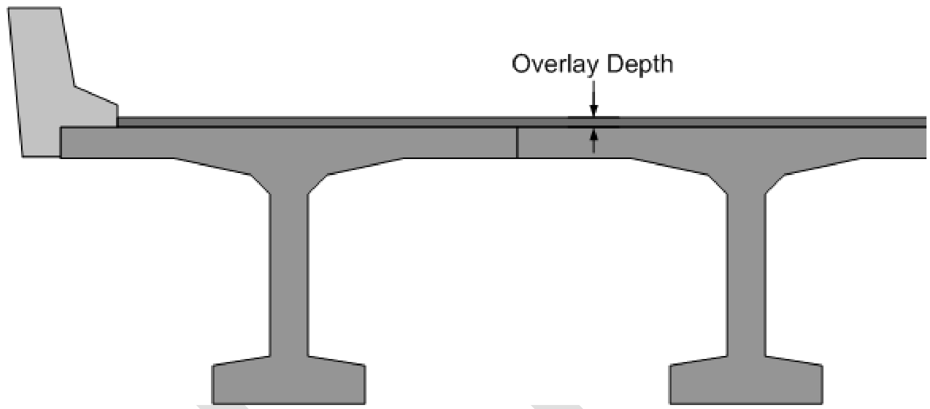


Figure 2-3: Non-structural Overlay Detail

Strands

0.6" Diameter	f_{pu} = 270.0 ksi
Grade 270	f_{py} = 243.0 ksi
Low Relaxation	E_{ps} = 28500 ksi
	a_{ps} = 0.217 in^2/per strand
	Straight Strands = 36
	Harped Strands = 14
	Temporary Strands = 2

Traffic Barrier

42" Single Slope

Design weight = 0.690 kip/ft/barrier

Load is distributed to 3 exterior girders

Load Modifiers

Ductility	Redundancy	Importance
η_D = 1.0	η_R = 1.0	η_I = 1.0

Criteria

Design in accordance with the AASHTO LRFD Bridge Design Specification, Eighth Edition, 2017 and the WSDOT Bridge Design Manual

Load Rate in accordance with AASHTO, The Manual for Bridge Evaluation, Second Edition, 2011 with 2015 interim revisions and the WSDOT Bridge Design Manual

WSDOT policy is to design using gross section properties (BDM 5.6.2.I) using refined estimate of prestress losses (BDM 5.4.1.C). PGSuper supports stress analysis with transformed section properties, the LRFD approximate method for estimating prestress losses, and a non-linear time-step analysis.

3 Design Preliminaries

Evaluate the first interior girder (Girder B).

3.1 Analysis of Asymmetric Girders

The WF69DG Girders used in this bridge are asymmetric. There is no geometric symmetry about the axes passing through the centroid of the section. The transverse thickening of the top flange to accommodate the roadway crown slope is the source of the asymmetry.

Asymmetry is of concern for two reasons, biaxial stresses and lateral deflections

The girder undergoes biaxial bending caused by the shape of the cross section and the lateral eccentricity of the precompression force from the centroid of the section. Depending on the shape of the girder, these biaxial effects can be appreciable, especially in long span girders and girders with large cross slope in the top flange.

The asymmetry and lateral eccentricity of the precompression force cause lateral deflections. Accurate fit-up of the UHPC joint connection between girders is essential. The UHPC joints provide a high-strength tension connection over a very short distance. Lateral deflections can cause the joint between adjacent girders to widen, reducing lap splices, or narrow, reducing bar embedment.

Construction specifications limit initial and long-term lateral deflections. The design engineer must evaluate the lateral deflections to ensure girder fabrication within tolerance is achievable.

PGSuper has special features for analysis of asymmetric girders. The program uses biaxial stress analysis while a girder is an independent unit and assumes uniaxial bending once the UHPC joints join the girder into the composite bridge system. In addition, while a girder is an independent unit, the program computes lateral deflections. Engineers should evaluate the lateral deflections and, if excessive, should mitigate their effect.

The features of interest are:

1) Deck bulb tee girders can have a variety of top flange overhang arrangements including equal overhangs, unequal overhangs specified by the engineer, and unequal overhangs automatically proportioned such that the CG of the girder coincides with the CL web,
2) Prestressing strands can be arranged with an unsymmetrical placement. This is often used to make the resultant precompression force coincident with the CG of the girder.

3.2 Construction Sequence

Figure 3-1 shows the assumed construction sequence. PGSuper models the various construction stages with Construction Events.

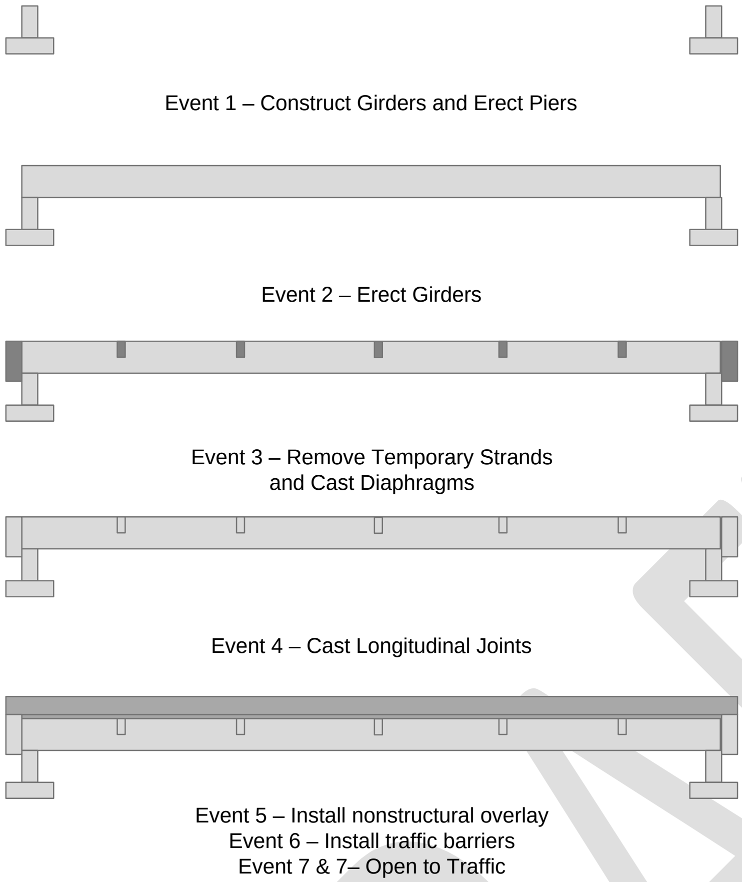


Figure 3-1 Assumed Construction Sequence

3.3 Girder Length

For a typical stub abutment with a Type A connection, the centerline of bearing is located 2’-8.5” from, and measured normal to, the back of pavement seat. The distance from the centerline bearing to the end of the girder is 1’-8.5” measured normal to the CL Bearing, which is parallel to the back of pavement seat.

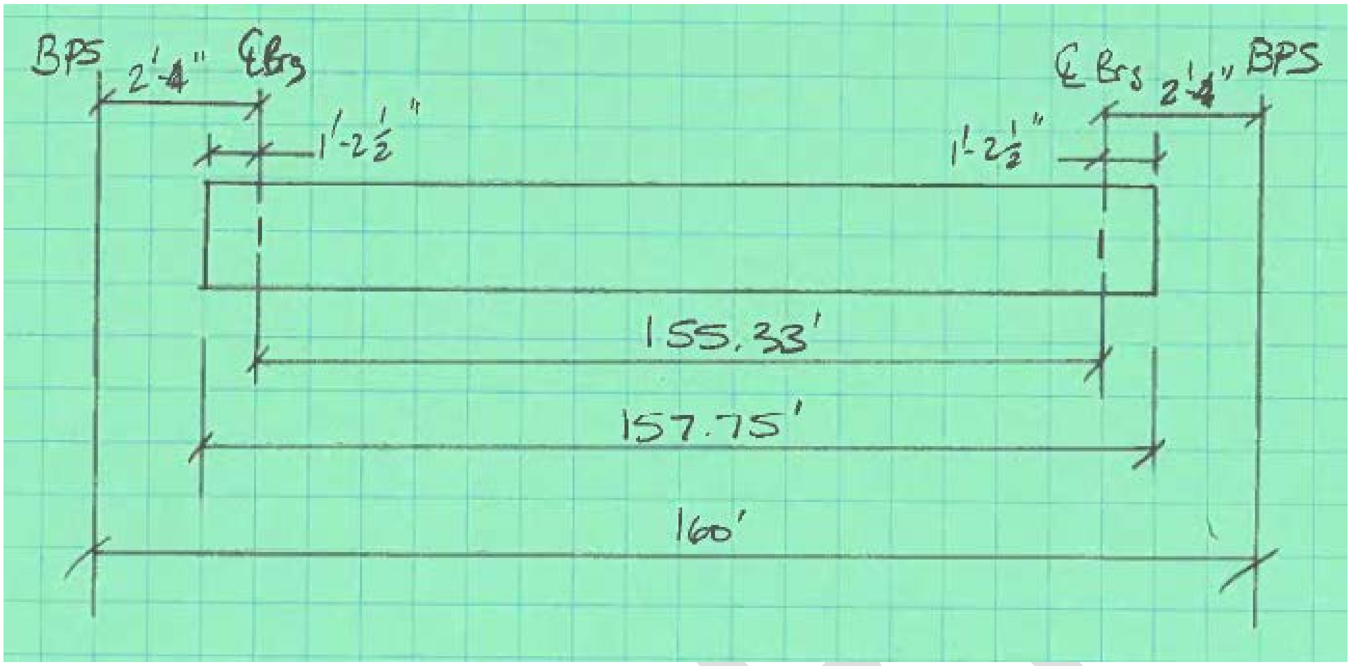


Figure 3-2 Girder Length Geometry

The bearing-to-bearing span length is $L_s = 160ft - 2(2.333ft) = 155.33ft$.

The overall girder length is $L_g = 155.33ft + 2(1.2083ft) = 157.75ft$.

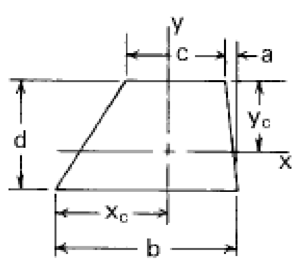
3.4 Section Properties

Compute the composite section properties. The basic girder section properties are in the bridge description.

3.4.1 Composite Girder Properties

Transform the structural joint to equivalent girder material and use the parallel axis theorem to compute the composite girder properties. The longitudinal joint is a trapezoid in section.

14. Trapezoid



$$A = \frac{d}{2}(b + c)$$

$$y_c = \frac{d}{3}\frac{2b + c}{b + c}$$

$$x_c = \frac{2b^2 + 2bc - ab - 2ac - c^2}{3(b + c)}$$

$$I_x = \frac{d^3}{36}\frac{b^2 + 4bc + c^2}{b + c}$$

$$I_y = \frac{d}{36(b + c)}[b^4 + c^4 + 2bc(b^2 + c^2) - a(b^3 + 3b^2c - 3bc^2 - c^3) + a^2(b^2 + 4bc + c^2)]$$

$$I_{xy} = \frac{d^2}{72(b + c)}[c(3b^2 - 3bc - c^2) + b^3 - a(2b^2 + 8bc + 2c^2)]$$

Modulus of elasticity of longitudinal joint concrete

$$E_c = 120{,}000K_1 w_c^2 f_c'^{0.33} = (120{,}000)(1.0)(0.160)^2(14.0)^{0.33} = 7339.111\ ksi$$

Modulus of elasticity of girder concrete

$$E_c = 120{,}000K_1 w_c^2 f_c'^{0.33} = (120{,}000)(0.9)(0.125)^2(6.8)^{0.33} = 3176.667\ ksi$$

$$n = \frac{E_{c\ joint}}{E_{c\ girder}} = \frac{7339.111ksi}{3176.667ksi} = 2.31$$

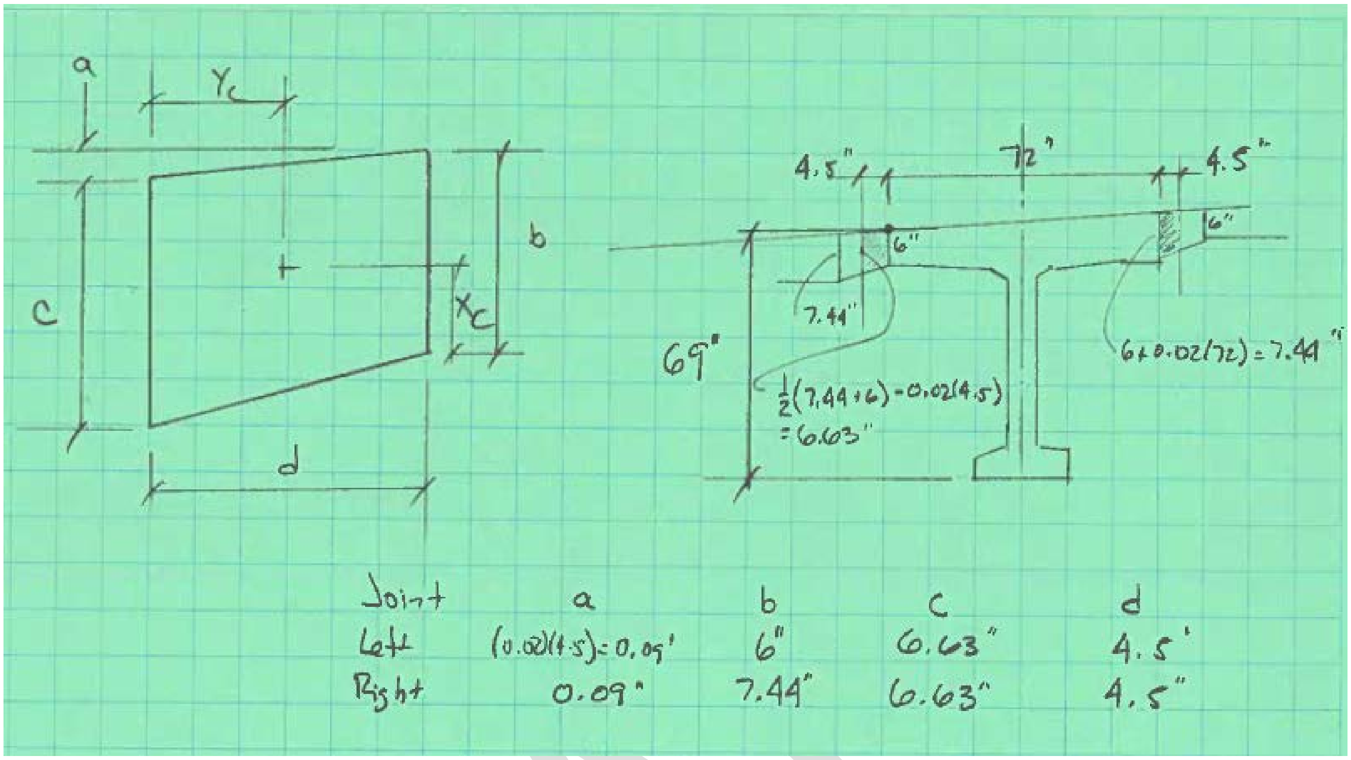


Figure 3-3 Longitudinal Joint Geometry at Mid-span

Left Joint

$$Y_b = 69in - 6.0in + \frac{2(6in)^2 + 2(6in)(6.63in) - (0.09in)(6in) - 2(0.09in)(6.63in) - (6.63in)^2}{3(6in + 6.63in)}$$
$$= 69in - 6in + 2.794in = 65.794in$$

Right Joint

$$69in + 0.02(72in + 4.5in) - 6.63n + \frac{2(6.63in)^2 + 2(6.63in)(7.44in) - (0.09in)(6.63in) - 2(0.09in)(7.44in) - (7.44in)^2}{3(6.63in + 7.44in)}$$
$$= 69in + 1.53in - 6.63in + 3.063in = 66.963in$$

	Area	Y_b	*(Area)(Y_b)*
Left Joint	$(2.31)\left(\frac{1}{2}\right)(4.5in)(6.0in + 6.63in)$ $= (2.31)(28.4175n^2)$ $= 65.644in^2$	$65.794in$	$4319in^3$
Right Joint	$(2.31)\left(\frac{1}{2}\right)(4.5in)(6.63in + 7.44in)$ $= (2.31)(31.6575in^2)$ $= 73.129in^2$	$66.963in$	$4896.9in^3$
Girder	$1211.371in^2$	$41.643in$	$50445.123in^3$
Total	$A_c = 1350.1in^2$		$59661.0in^3$

$$Y_{bc} = \frac{\sum(Area)(Y_b)}{\sum(Area)} = \frac{59661.0in^3}{1350.1in^2} = 44.19in$$

$$Y_{tc} = H_g + 0.02(72in) - Y_{bc} = 69.0in + 0.02(72in) - 44.19in = 26.25in$$

Left Joint

$$I_x = (2.31)\left(\frac{4.5in}{36(6in + 6.63in)}\right)[(6in)^4 + (6.63in)^4 + 2(6in)(6.63in)((6in)^2 + (6.63in)^2) - (0.09in)((6in)^3 + 3(6in)^2(6.63in) - 3(6in)(6.63in)^2 - (6.63in)^3) + (0.09in)^2((6in)^2 + 4(6in)(6.63in) + (6.63in)^2)] = (2.31)(95.06in^4) = 219.59in^4$$

Right Joint

$$I_x = (2.31)\left(\frac{4.5in}{36(6.63in + 7.44in)}\right)[(6.63in)^4 + (7.44in)^4 + 2(6.63in)(7.44in)((6.63in)^2 + (7.44in)^2) - (0.09in)((6.63in)^3 + 3(6.63in)^2(7.44in) - 3(6.63in)(7.44in)^2 - (7.44in)^3) + (0.09in)^2((6.63in)^2 + 4(6.63in)(7.44in) + (7.44in)^2)] = (2.31)(131.64n^4) = 304.1in^4$$

	Area	*d*	*(Area)(d²)*	*Iₓ*	*Iₓ + (Area)(d²)*
Left Joint	$65.644in^2$	$65.794in - 44.19in = 21.6in$	$30626.86in^4$	$219.59in^4$	$30846.5in^4$
Right Joint	$73.129in^2$	$66.963in - 44.19in = 22.77in$	$37915.4in^4$	$204.1in^4$	$38219.5in^4$
Girder	$1211.371in^2$	$41.64in - 44.19in = -2.55in$	$7876.94in^4$	$861860.5in^4$	$869737.4in^4$
					$I_x = 938803in^4$

3.4.2 Section Property Summary

Below are the section properties from PGSuper. They are slightly different then the properties computed above. Use the section properties reported by PGSuper for better agreement between these calculations and the software.

Table 3-1: Section Properties from PGSuper at Mid-Span

	Girder	*Composite Girder*
*Area, **A***	$1211.371\ in^2$	$1351.106\ in^2$
I_x	$861860.5\ in^4$	$938938.9in^4$
I_y	$251152.4\ in^4$	-
I_{xy}	$17465.9\ in^4$	-
X_l	$36.514\ in$	-
X_r	$35.486\ in$	-
Y_t	$28.797\ in$	$26.241\ in$
Y_b	$41.643\ in$	$44.199\ in$

$\boldsymbol{S_t}$	-	$35781.2n^3$
$\boldsymbol{S_b}$	-	$21243.5\ in^3$
$Perimeter$	$326.738\ in$	-

3.5 Structural Analysis

There are several significant stages during the life of a prestressed girder. PGSuper automatically models these stages as Construction Events. The events are:

1) Construct girders (aka Casting Yard Stage)
    a) Tension strands, form girders, cast concrete, concrete curing. Initial relaxation of the prestressing strand occurs.
    b) Strip forms and impart the precompression force into the girder (aka Release)
    c) Move girders into storage area (Initial lifting)
    d) Elapsed time during storage (creep, shrinkage, and relaxation losses occur)
2) Erect girders
    a) Prior to erection, the girders must be transported from the fabrication facility to the bridge site
    b) Erect and brace girders
    c) De-tension temporary strands (if applicable)
3) Cast longitudinal joints (dead load applied to non-composite girder section)
4) Install nonstructural overlay. (dead load applied to composite section)
5) Install railing system (traffic barriers, sidewalks, etc). (dead load applied to composite section)
6) Final without Live Load (includes future overlay if applicable)
7) Final with Live Load

PGSuper models the individual steps within a Construction Event with Analysis Intervals. For example, Event 1 – Construct Girders, models five analysis intervals: Tension Strands and Cast Concrete, Elapsed Time during Curing, Prestress Release, Lifting, Placement into Storage, and Elapsed Time during Storage.

The analysis intervals are a general modelling approach associated with time-step analysis. Precast girder design normally uses a pseudo time-step analysis. However, the PGSuper can perform a refined non-linear time-step analysis. PGSplice uses the non-linear time-step analysis as well.

3.5.1 Girder Construction (Casting Yard)

Girder construction at the casting yard consists of tensioning strands, placing mild reinforcement, installing girder forms, and placing concrete. Stripping of girder forms occurs after the concrete reaches adequate strength to accommodate the stresses and stability of the girder. The strands are the detensioned but because of bond with the girder concrete, the precompression force imparts into the girder. If the prestress force is eccentric to the centroid of the girder and it is sufficient to overcome the self-weight of the girder, the girder cambers upwards. In this condition, the girder bears on its ends and bending stresses develop.

Because the girder has 8" of longitudinal top flange thickening, the dead load varies continuously along its length. The dead load at mid-span is

$$w_{girder} = \gamma_c A_g = (0.130kcf)(1211.371in^2)\left(\frac{1ft^2}{144in^2}\right) = 1.094\ klf$$

where:

A_g = Gross cross sectional area of the girder

γ_c = Unit weight of concrete

From PGSuper, the self-weight loading is

Load Start, From Left End of Girder (ft)	Load End, From Left End of Girder (ft)	Start Weight (kip/ft)	End Weight (kip/ft)
0.000	15.775	1.516	1.364
15.775	31.550	1.364	1.246
31.550	47.325	1.246	1.161
47.325	63.100	1.161	1.110
63.100	78.875	1.110	1.094
78.875	94.650	1.094	1.110
94.650	110.425	1.110	1.161
110.425	126.200	1.161	1.246
126.200	141.975	1.246	1.364
141.975	157.750	1.364	1.516

This load is analyzed as a series of linear load segments along the girder.

Figure 3-4 - Self weight loading

Moment at point of prestress transfer (PSXFR)

Prestress transfer occurs over 60 strand diameters (LRFD 5.9.4.3.1)

$$l_t = 60d_b = (60)(0.6in) = 36in = 3ft$$

$$M_g = 285.98k \cdot ft$$

Moment at harp point (HP)

Harp point is 0.4L from the end of the girder $(0.4)(157.75ft) = 63.1ft$

$$M_g = 3492.81k \cdot ft$$

Moment at mid-girder (0.5L)

$$M_g = 3629.59k \cdot ft$$

3.5.2 Erected Girder

Substructure elements support the girder at permanent bearing locations once erected. Bracing stabilizes the girder. Temporary top strands are detensioned, followed by diaphragm casting and installation of the longitudinal joints. Installation of the railing system and the nonstructural overlay occurs after the longitudinal joints gain adequate strength to form a composite section with all of the girders.

3.5.2.1 *Diaphragm and Longitudinal Joint*

In this stage, the girder supports its self-weight along with the weight of the diaphragms and the longitudinal joints.

3.5.2.1.1 Diaphragm Loads

The diaphragm load for an interior girder is $P = HW\gamma_c(S - t_{web})$, where:

H	=	Height of the interior diaphragm
W	=	Width of the interior diaphragm
t_{web}	=	Width of the girder web
S	=	Spacing of the girders

$$P = HW\gamma_c(S - t_{web}) = (23.0in)(8in)(0.140kcf)(72in + 9in - 6.125in)\left(\frac{1ft^3}{1728in^3}\right) = 1.12kip$$

PGSuper uses the unit weight of the nonstructural overlay for the diaphragms

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Diaphragms are located at 38.833 ft $(0.25L)$, 77.667 ft $(0.50L)$, and 116.50 ft $(0.75L)$ from the left bearing.

3.5.2.1.2 Longitudinal Joint

Like the girder self weight, the longitudinal joint weight varies along the length of the girder due to the longitudinal top flange thickening. The longitudinal joint load at mid-span is

$$(28.4175in^2 + 31.6575in^2)(0.160kcf)\left(\frac{1ft^2}{144in^2}\right) = 0.067klf$$

From PGSuper, the longitudinal joint loading is along the length of the girder is

Load Start, From Left Bearing (ft)	Load End, From Left Bearing (ft)	Start Weight (kip/ft)	End Weight (kip/ft)
-1.208	0.000	0.132	0.131
0.000	14.567	0.131	0.109
14.567	15.533	0.109	0.108
15.533	30.342	0.108	0.091
30.342	31.067	0.091	0.090

31.067	46.117	0.090	0.078
46.117	46.600	0.078	0.077
46.600	61.892	0.077	0.070
61.892	62.133	0.070	0.070
62.133	77.667	0.070	0.067
77.667	93.200	0.067	0.070
93.200	93.442	0.070	0.070
93.442	108.733	0.070	0.077
108.733	109.217	0.077	0.077
109.217	124.267	0.077	0.090
124.267	124.992	0.090	0.090
124.992	139.800	0.090	0.107
139.800	140.767	0.107	0.108
140.767	155.333	0.108	0.130
155.333	156.542	0.130	0.132

3.5.2.2 *Superimposed Dead Loads*

Application of superimposed dead loads occurs after the longitudinal joints have reached adequate strength. The superimposed dead loads consist of the nonstructural overlay, traffic barrier and the future overlay. The composite section is resisting these loads.

3.5.2.2.1 Nonstructural Overlay

The nonstructural overlay load consists of the main overlay and the overlay haunch.

3.5.2.2.1.1 Main Nonstructural Overlay Load

The main nonstructural overlay load is

$$w_{slab} = W_{tf} t_{nso} \gamma_c = (81in)(1.5in)(0.140kcf)\left(\frac{1ft^2}{144in^2}\right) = 0118klf$$

3.5.2.2.1.2 Nonstructural Overlay Haunch Load

The nonstructural overlay haunch load accounts for buildup between the top of the girder and the bottom of the main nonstructural overlay. This concrete element has a width equal to the top flange width plus the width of one longitudinal joint. The haunch depth varies along the girder because of camber, longitudinal top flange thickening, and variations in the roadway surface.

While the goal of longitudinal top flange thickening is to minimize the haunch some buildup will be required because of the natural variation of camber.

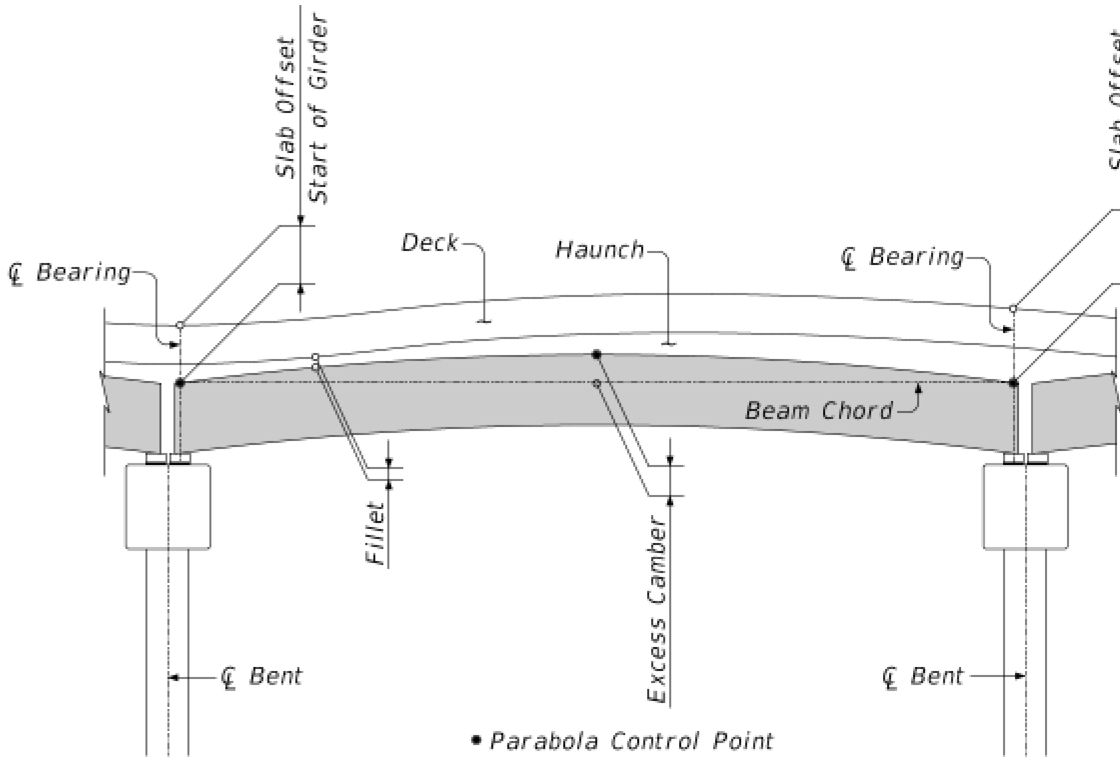


Figure 3-5: Slab Haunch

WSDOT's design policy is to assume zero natural camber for purposes of determining the haunch loads (BDM 5.6.2.D.3.iv).

PGSuper provides the option to consider excess camber when determining loading. This option may be desirable for load rating as it reduces the haunch dead load.

The basic haunch dead load at any given section is

$$w_{haunch} = (W_{tf} + W_{lj})t_{haunch}\gamma_c$$

The slab offset ("A" dimension) is 1.75 in. The nonstructural overlay haunch load at the start of the span is

$$t_{haunch} = \text{A} - t_{nso} + t_{tft} = 1.75in - 1.5in + 8in = 8.25in$$

$$w_{haunch} = (72in + 9in)(8.25in)(0.140kcf)\left(\frac{1ft^2}{144in^2}\right) = 0.650\ klf$$

From PGSuper, the nonstructural overlay haunch loading is

Location From Left Bearing (ft)	Station	Offset (ft)	Top Slab Elevation (ft)	Girder Chord Elevation (ft)	Top Girder Elevation (ft)	Nonstructural Overlay Thickness (in)	Assumed Haunch Depth (in)	Haunch Load (kip/ft)
-1.208	95+01.13	13.500 L	99.741	99.575	99.575	1.500	0.500	0.039
0.000	95+02.33	13.500 L	99.753	99.587	99.570	1.500	0.698	0.055

14.567	95+16.90	13.500 L	99.899	99.732	99.537	1.500	2.840	0.224
15.533	95+17.87	13.500 L	99.909	99.742	99.536	1.500	2.966	0.234
30.342	95+32.68	13.500 L	100.057	99.890	99.543	1.500	4.660	0.367
31.067	95+33.40	13.500 L	100.064	99.897	99.545	1.500	4.731	0.373
46.117	95+48.45	13.500 L	100.215	100.048	99.593	1.500	5.960	0.469
46.600	95+48.93	13.500 L	100.219	100.053	99.595	1.500	5.992	0.472
61.892	95+64.23	13.500 L	100.372	100.206	99.686	1.500	6.740	0.531
62.133	95+64.47	13.500 L	100.375	100.208	99.687	1.500	6.748	0.531
77.667	95+80.00	13.500 L	100.530	100.363	99.822	1.500	7.000	0.551
93.200	95+95.53	13.500 L	100.685	100.519	99.998	1.500	6.748	0.531
93.442	95+95.77	13.500 L	100.688	100.521	100.001	1.500	6.740	0.531
108.733	96+11.07	13.500 L	100.841	100.674	100.216	1.500	5.992	0.472
109.217	96+11.55	13.500 L	100.845	100.679	100.224	1.500	5.960	0.469
124.267	96+26.60	13.500 L	100.996	100.829	100.477	1.500	4.731	0.373
124.992	96+27.33	13.500 L	101.003	100.837	100.490	1.500	4.660	0.367
139.800	96+42.13	13.500 L	101.151	100.985	100.779	1.500	2.966	0.234
140.767	96+43.10	13.500 L	101.161	100.994	100.799	1.500	2.840	0.224
155.333	96+57.67	13.500 L	101.307	101.140	101.124	1.500	0.698	0.055
156.542	96+58.88	13.500 L	101.319	101.152	101.152	1.500	0.500	0.039

Note that the haunch loads increases in the middle of the girder. This is due to the longitudinal top flange thickening and the policy to assume natural camber to be zero for purposes of determining loading.

3.5.2.2.2 Traffic Barrier

The traffic barrier weight is distributed over n exterior girders, if there are $2n$ or more girders, otherwise the weight of the traffic barrier per girders is $w_{tb} = \frac{W_{tb\,left} + W_{tb\,right}}{N}$, where N is the number of girders in the span. From BDM 5.6.3.2.B.2.d, $n = 3$.

$$2n = 7, N = 6, 2n \leq N$$

$$w_{tb} = \frac{W_{tb}}{n} = \frac{0.690 klf}{3\ girders} = 0.230 \frac{klf}{girder}$$

AASHTO permits equal distribution for barrier loads to all girders.

3.5.2.3 *Open to Traffic*

3.5.2.3.1 Future Overlay

Evenly distribute the weight of the future wearing surface to all girders. The curb to curb width of the deck is $44.333ft$.

$$w_o = \frac{(44.333ft)(0.035ksf)}{7\ girder} = 0.222\frac{klf}{girder}$$

Take care when applying the future overlay loading. Certain stress conditions are worse before the overlay is applied and others are worse after it is applied.

3.5.2.3.2 Live Load

The design live load is the HL93 notional model defined in the AASHTO LRFD BDS.

The vehicular live loading is the combination of the:

- design truck or design tandem, and (LRFD 3.6.1.1)
- design lane load (LRFD 3.6.1.2.1)

The design truck consists of three axles. Axle weights and spacing are, 8.0 kip, 14.0 ft, 32.0 kip, 14.0 to 30.0 ft, 32.0 kip. See Figure 3-6 below.

The design tandem consists of a pair of 25.0 kip axles spaced 4.0 ft apart.

The design lane load is 0.640 klf, uniformly distributed along the length of the span.

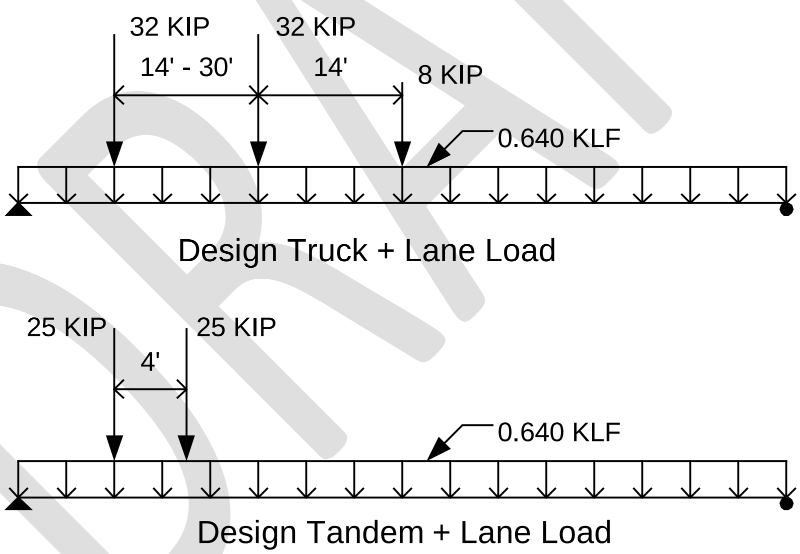


Figure 3-6: HL93 Live Load Model

Apply a dynamic load allowance (impact) of 33% to the design truck and design tandem portions of the live load response.

The fatigue live load is the design truck with the rear axle spacing fixed at 30 ft. The dynamic load allowance for fatigue is 15%.

3.5.3 Analysis Results Summary

3.5.3.1 *At Release*

Loading	Transfer Point	Harp Point	Mid-Span

Girder	$\mathbf{285.98\ k \cdot ft}$	$\mathbf{3492.81\ k \cdot ft}$	$\mathbf{3629.59\ k \cdot ft}$

3.5.3.2 *At Bridge Site*

Loading	0.5L_s
Girder after erection	$\mathbf{3512.77\ k \cdot ft}$
Diaphragm	$\mathbf{86.69\ k \cdot ft}$
Longitudinal Joint	$\mathbf{235.56\ k \cdot ft}$
Nonstructural Overlay	$\mathbf{356.27\ k \cdot ft}$
Nonstructural Overlay Haunch	$\mathbf{1403.73\ k \cdot ft}$
Traffic Barrier	$\mathbf{693.69\ k \cdot ft}$
Future Overlay	$\mathbf{668.56\ k \cdot ft}$
Design LLIM (HL-93)	$\mathbf{5276.56\ k \cdot ft}$
Fatigue LLIM	$\mathbf{2599.00\ k \cdot ft}$

Live loads are per lane

3.5.4 Limit State Responses

Group the structural responses into load cases and compute limit state responses. The total factored load, or limit state response, is $Q = \sum \eta_i \gamma_i q_i$. (LRFD Eqn. 3.4.1-1)

LRFD Table 3.4.1-1 gives the load factors. The limit states of importance are:

- Service I, Q = 1.0DC + 1.0DW + 1.0(LL+IM)
- Service III, Q = 1.0DC + 1.0DW + 0.8(LL+IM)
- Strength I, Q = 1.25DC + 1.50DW + 1.75(LL+IM)
- Fatigue I, Q = 0.5DC + 0.5DW + 1.5(LL+IM)

The live load factor for Service III is 0.8 for design and 1.0 for load rating. See BDM 3.5.2

3.5.5 Live Load Distribution Factors

Compute the live load distribution factors. Select the appropriate cross section type from LRFD Table 4.6.2.2.1-1. A precast concrete tee section with shear keys corresponds to cross section j. The longitudinal joint has sufficient strength to make the girders “sufficiently connected to act as a unit”.

WSDOT deviates from the LRFD BDS for exterior girders in type i sections as described in BDM 3.9.3.A.

Compute the longitudinal stiffness parameter K_g.

$$K_g = n\left(I + Ae_g^2\right)$$

where:

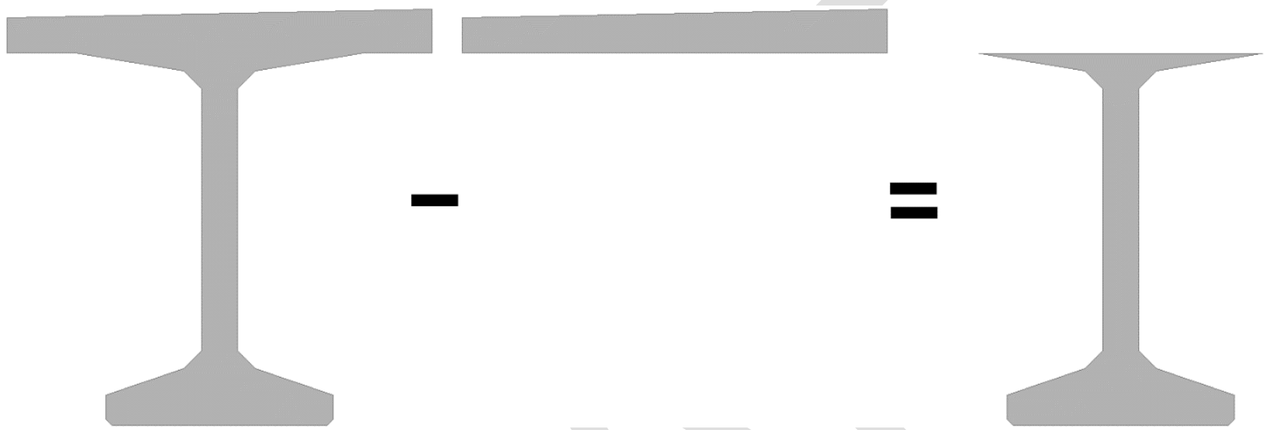
n = modular ratio between beam and deck material $n = \frac{E_{beam}}{E_{slab}}$

I = moment of inertia of the beam (in^4)

A = area of beam (in^2)

e_g = distance between the centers of gravity of the basic beam and deck (in)

The properties A and I are those of the girder stem for a deck bulb tee girder.



$$a = 6\ in, b = 7.44in, h = 72in$$

$$A_{flange} = 0.5(72in)(6in + 7.44in) = 483.84in^2$$

$$Y_{flange} = b - \frac{a^2 + ab + b^2}{3(a+b)} = 7.44in - \frac{(6in)^2 + (6in)(7.44in) + (7.44in)^2}{3(6in + 7.44in)} = 4.067in$$

$$I_{flange} = \frac{h}{36(a+b)}\left(a^4 + b^4 + 2a^3b + 2ab^3\right)$$
$$= \frac{72in}{36(6in + 7.44in)}\left((6in)^4 + (7.44in)^4 + 2(6in)^3(7.44in) + 2(6in)(7.44in)^3\right) = 1862.5in^4$$

$$A_{stem} = A_g - A_{flange} = 1211.371in^2 - 483.840in^2 = 727.531in^2$$

$$Y_{stem} = \frac{A_gY_g - A_{flange}Y_{flange}}{A_{stem}} = \frac{(1211.371in^2)(28.787in) - (483.840in^2)(4.067in)}{727.531in^2} = 45.243in$$

Note that this Y_{stem} is measured from the top of the girder.

$$I_{stem} = I_g + A_g\left(Y_g - Y_{stem}\right)^2 - I_{flange} - A_{flange}\left(Y_{flange} - Y_{stem}\right)^2$$

$$I_{stem} = 861860.5in^4 + (1211.371in^2)(28.787in - 45.243in)^2 - 1862.5in^4 - (483.40in^2)(4.067in - 45.243in)^2 = 368450in^4$$

Y_{stem} measured from the top of the stem is $Y_{stem} = 45.243in - 7.440in = 37.803in$

Take t_s to be the average top flange thickness

$$t_s = \frac{6in + 7.44in}{2} = 6.72in$$

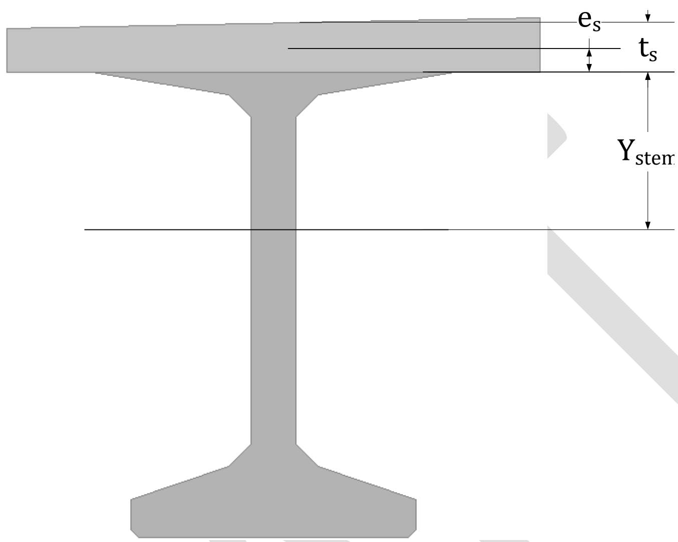


Figure 3-7: e_g Detail

$$e_g = Y_t + \frac{t_s}{2} = 37.803in + \frac{6.720in}{2} = 41.163in$$

$$n = \frac{3176.667ksi}{3176.667ksi} = 1.0$$

$$K_g = 1.0[368450in^4 + (727.531in^2)(41.163in)^2] = 1600574in^4$$

3.5.5.1 *Number of Design Lanes*

The number of design lanes is equal to the integer portion of the roadway width divided by 12 ft (LRFD 3.6.1.1.1).

$$N_L = \left\lfloor \frac{44.333ft}{12ft} \right\rfloor = 3\ Design\ Lanes$$

3.5.5.2 *Distribution of Live Loads per Lane for Moments in Interior Beams*

LRFD Table 4.6.2.2.2b-1 gives the live load distribution factors for moments in interior beams.

3.5.5.2.1 Compute Distribution Factor for Moment

Check the range of applicability for live load distribution factors.

$3.5\ ft \le S \le 16\ ft$	$S = 6.75\ ft$	**OK**
$4.5\ in \le t_s \le 12\ in$	$t_s = 6.72\ in$	**OK**
$20\ ft \le L \le 240\ ft$	$L = 155.33\ ft$	**OK**
$N_b \ge 4$	$N_b = 7$	**OK**
$10{,}000 in^4 \le K_g \le 7{,}000{,}000\ in^4$	$K_g = 1600574in^4\ in^4$	**OK**

3.5.5.2.1.1 One Design Lane Loaded

The live load distribution factor for one loaded lane is

$$gM_1^i = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

$$gM_1^i = 0.06 + \left(\frac{6.75}{14}\right)^{0.4}\left(\frac{6.75}{155.33}\right)^{0.3}\left(\frac{1600574}{12.0 \cdot 155.33 \cdot 6.72^3}\right)^{0.1} = 0.383$$

3.5.5.2.1.2 Two or More Design Lanes Loaded

The live load distribution factor for two or more design lanes loaded is

$$gM_{2+}^i = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

$$gM_{2+}^i = 0.075 + \left(\frac{6.75}{9.5}\right)^{0.6}\left(\frac{6.75}{155.33}\right)^{0.2}\left(\frac{1600574}{12.0 \cdot 155.33 \cdot 6.72^3}\right)^{0.1} = 0.558$$

3.5.5.3 *Distribution of Live Loads per Lane for Shear in Interior Beams*

LRFD Table 4.6.2.2.3a-1 gives the live load distribution factors for shear in interior beams.

3.5.5.3.1 Compute Distribution Factor for Shear

Check the range of applicability for live load distribution factors.

$3.5\ ft \le S \le 16\ ft$	$S = 6.75\ ft$	**OK**
$4.5\ in \le t_s \le 12\ in$	$t_s = 6.72\ in$	**OK**
$20\ ft \le L \le 240\ ft$	$L = 155.33\ ft$	**OK**
$N_b \ge 4$	$N_b = 7$	**OK**

3.5.5.3.1.1 One Design Lane Loaded

The live load distribution factor for one design lane loaded is

$$gV_1^i = 0.36 + \frac{S}{25.0}$$

$$gV_1^i = 0.36 + \frac{6.75}{25.0} = 0.630$$

3.5.5.3.1.2 Two or More Design Lanes Loaded

The live load distribution factor for two or more loaded lanes is

$$gV_{2+}^i = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

$$gV_{2+}^i = 0.2 + \frac{6.75}{12} - \left(\frac{6.75}{35}\right)^{2.0} = 0.725$$

3.5.5.4 *Live Load Distribution Factor Summary*

Distribution Factor Summary for Strength and Service Limit States

Distribution Factor	1 Loaded Load	2+ Loaded Lanes	Controlling Factor

Moment (gM)	0.383	0.558	0.558
Shear (gV)	0.630	0.725	0.725

3.5.5.5 *Live Load Distribution Factor for Fatigue Limit State*

The fatigue live load distribution uses the factor for one loaded lane (LRFD 3.6.1.4.3b). The single lane distribution factors include a multiple presence factor of 1.2. The multiple presence factor for fatigue loading is 1.0 (LRFD 3.6.1.1.2). Divide the one loaded lane distribution factors by 1.2 to get the fatigue distribution factors.

Distribution Factor Summary for Fatigue Limit States

Distribution Factor	1 Loaded Load
Moment (gM)	$0.383/1.2 = 0.320$
Shear (gV)	$0.630/1.2 = 0.525$

4 Losses and Effective Prestress

Effective prestress is the stress or force remaining in prestressing steel after time dependent losses and elastic effects have occurred. Time dependent losses consist of concrete shrinkage, concrete creep, and prestressing steel relaxation. Elastic effects are changes in the prestress due to externally applied loads or internal restraining forces. Elastic effects are often called elastic gains.

Girder stresses used for prestress loss computations, while the girder is an independent unit, are computed with a biaxial stress analysis. This is done so that all stresses are computed on a consistent bases. Note, however, that the stresses computed for prestress losses are those at the location of the CG of the prestressing strand, which is near the vertical centerline of the girder. Biaxial bending effects are negligible and these stresses are essentially equal to stresses computed assuming uniaxial bending.

4.1 Losses before Prestress Transfer

Losses before prestress transfer are due to relaxation of the strand. Prior to the 2005 interim revisions to the LRFD 3rd Edition, relaxation before prestress transfer was included in prestress loss calculations. Since the 2005 interim revisions, this is no longer required based on the idea that fabricators can overstress strands to achieve an effective prestress of $0.75f_{pu}$ at release. However, WSDOT retains the practice of including relaxation prior to prestress transfer because it reflects the production practices used by local fabricators.

$$\Delta f_{pR0} = \frac{\log(24.0t)}{40.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$$

$$f_{pj} = 0.75f_{pu} = 0.75(270) = 202.5ksi$$

$$f_{py} = 0.9f_{pu} = 243ksi$$

$$t = 1\ day$$

$$\Delta f_{pR0} = \frac{\log(24.0 \cdot 1day)}{40}\left[\frac{202.5ksi}{243.0ksi} - 0.55\right](202.5ksi) = 1.980\ ksi$$

This calculation is for intrinsic relaxation of the strand. Intrinsic relaxation is associated with strand tensioned between two stationary points such as in a testing machine or between tensioning bulkheads.

4.2 Losses immediate after transfer

As the force in the pretensioned strands is released from the stressing equipment, it is transferred to the girder as a compression force. This force is typically eccentric and causes axial shortening and bending in the girder. The shortening causes a reduction in the elongation of the strand and a reduction in the precompression force. This is known as the elastic shortening losses.

The permanent and temporary strands are at different elevations. The elastic shortening is computed for each type of strand.

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$

$$f_{cgp}(x, y) = \frac{M_y I_{xx} + M_x I_{xy}}{I_{xx} I_{yy} - I_{xy}^2} x - \frac{M_x I_{yy} + M_y I_{xy}}{I_{xx} I_{yy} - I_{xy}^2} y + \frac{P}{A}$$

$$M_x = (Pe_{psy} - M_g)$$

$$M_y = Pe_{psx}$$

$$x = -e_{px} \; or - e_{tx}$$

$$y = -e_{py} \; or - e_{ty}$$

$$f_{cgp}\ (Permanent\ Strands) = \frac{\left(Pe_{psy} - M_g\right) I_{yy} + Pe_{psx} I_{xy}}{I_{xx} I_{yy} - I_{xy}^2} (e_{py}) - \frac{Pe_{psx} I_{xx} + \left(Pe_{psy} - M_g\right) I_{xy}}{I_{xx} I_{yy} - I_{xy}^2} (e_{px}) + \frac{P}{A}$$

$$f_{cgp}\ (Temporary\ Strands) = \frac{\left(Pe_{psy} - M_g\right) I_{yy} + Pe_{psx} I_{xy}}{I_{xx} I_{yy} - I_{xy}^2} (e_{ty}) - \frac{Pe_{psx} I_{xx} + \left(Pe_{psy} - M_g\right) I_{xy}}{I_{xx} I_{yy} - I_{xy}^2} (e_{tx}) + \frac{P}{A}$$

$$P = N_p\left(a_{ps}\right)\left(f_{pj} - \Delta f_{pR0} - \Delta f_{pES}\ Permanent\ Strands\right) + N_t\left(a_{ps}\right)\left(f_{pj} - \Delta f_{pR0} - \Delta f_{pES}\ Temporary\ Strands\right)$$

Solve this equation iteratively for P and Δf_{pES} of both the permanent and temporary strands.

$$E_{ci} = 120000(0.9)(0.125)^2(6.0)^{0.33} = 3048.131\ ksi$$

$$e_{psx} = 0.514in, e_{psy} = 35.566in$$

$$e_{px} = 0.514in, e_{py} = 38.003in$$

$$e_{tx} = 0.514in, e_{ty} = -25.357in$$

Assume $P = 1975\ kip$

Permanent Strands

$$f_{cgp} = \frac{\left((1975kip)(35.566in) - (3629.59k \cdot ft)\left(\frac{12in}{1ft}\right)\right) 251152.4in^4 + (1975kip)(0.514in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2} (38.003in)$$
$$- \frac{(1975kip)(0.514in)(861860.5in^4) + \left((1975kip)(35.565in) - (3629.59k \cdot ft)\left(\frac{12in}{1ft}\right)\right) 17465.9in^4}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2} (0.514in)$$
$$+ \frac{1975kip}{1211.371in^2} = 2.808ksi$$

$$\Delta f_{pES} = \frac{28500ksi}{3048.131ksi} (2.808ksi) = 26.262ksi$$

Temporary Strands

$$f_{cgp} = \frac{\left((1975kip)(35.566in) - (3629.59k \cdot ft)\left(\frac{12in}{1ft}\right)\right)251152.4in^4 + (1975kip)(0.514in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(-25.357in) - \frac{(1975kip)(0.514in)(861860.5in^4) + \left((1975kip)(35.566in) - (3629.59k \cdot ft)\left(\frac{12in}{1ft}\right)\right)17465.9in^4}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(0.514in) + \frac{1975kip}{1211.371in^2} = 0.839ksi$$

$$\Delta f_{pES} = \frac{28500ksi}{3048.131ksi}(0.839ksi) = 7.843ksi$$

$$P = 52(0.217in^2)(202.5ksi - 1.98ksi - 26.262ksi) + 2(0.217in^2)(202.5ksi - 1.98ksi - 7.843ksi) = 1974.3kip$$

PGSuper performs this calculation with a very small convergence tolerance and at many points along the girder. The effective prestress force at release and initial lifting for various points (as determined by PGSuper) are given below.

Location	Effective Prestress after release
PSXFR	$2003.96\ kip$
HP	$1968.34\ kip$
0.5Lg	$1974.45\ kip$

4.3 Losses at Hauling

Assume hauling to occur as soon as possible (10 days).

4.3.1.1 *Shrinkage of Girder Concrete*

$$\Delta f_{SRH} = \varepsilon_{bih} E_p K_{ih}$$

$$\varepsilon_{bih} = k_s k_{hs} k_f k_{td} 0.48 \times 10^{-3}$$

$$K_{ih} = \frac{1}{1 + \frac{E_p}{E_{ci}}\frac{A_{ps}}{A_g}\left(1 + A_g \frac{\left((e_{psy}I_{yy} + e_{psx}I_{xy})e_y - (e_{psx}I_{xx} + e_{psy}I_{xy})e_x\right)}{I_{xx}I_{yy} - I_{xy}^2}\right)\left[1 + 0.7\psi_b(t_f, t_i)\right]}$$

$$\psi_b(t_f, t_i) = 1.9 k_s k_{hc} k_f k_{td} t_i^{-0.118}$$

$$k_s = 1.45 - 0.13\left(\frac{V}{S}\right) \geq 1.0$$

Because the girder has longitudinal top flange thickening a numerical procedure is used to compute the volume to surface ratio.

$$V_i = \frac{1}{2}(A_i + A_{i+1})(x_{i+1} - x_i)$$

$$S_i = \frac{1}{2}(P_i + P_{i+1})(x_{i+1} - x_i)$$

Location from End of Girder (ft)	Area (in^2)	Perimeter (in)	$x_{i+1} - x_i$ (in)	$\frac{1}{2}(A_i + A_{i+1})$	V_i (in^3)	$\frac{1}{2}(P_i + P_{i+1})$	S_i (in^2)

				(in²)		(in)	
0	1679.371	339.738					
15.775	1510.891	335.058	189.3	1595.131	301958.3	337.398	63869.44
31.55	1379.851	331.418	189.3	1445.371	273608.7	333.238	63081.95
47.325	1286.251	328.818	189.3	1333.051	252346.6	330.118	62491.34
63.1	1230.091	327.258	189.3	1258.171	238171.8	328.038	62097.59
78.875	1211.371	326.738	189.3	1220.731	231084.4	326.998	61900.72
94.65	1230.091	327.258	189.3	1220.731	231084.4	326.998	61900.72
110.425	1286.251	328.818	189.3	1258.171	238171.8	328.038	62097.59
126.2	1379.851	331.418	189.3	1333.051	252346.6	330.118	62491.34
141.975	1510.891	335.058	189.3	1445.371	273608.7	333.238	63081.95
157.75	1679.371	339.738	189.3	1595.131	301958.3	337.398	63869.44
				$\sum V_i =$	2594339 in³	$\sum S_i =$	626882in²

$$\frac{V}{S} = \frac{\sum V_i}{\sum S_i + A_0 + A_n} = \frac{2594339in^3}{1679.371in^2 + 1679.371in^2 + 626882in^2} = 4.116in$$

$$k_s = 1.45 - 0.13(4.116) = 0.915 \therefore 1.0$$

$$k_{hs} = 2.00 - 0.014H = 2.00 - 0.014(75) = 0.95$$

$$k_{hc} = 1.56 - 0.008H = 1.56 - 0.005(75) = 0.96$$

$$k_f = \frac{5}{1 + f'_{ci}} = \frac{5}{1 + 6.0} = 0.714$$

$$k_{td}(t = 9days) = \frac{9}{12\left(\frac{100 - 4(6.0)}{6.0 + 20}\right) + 9} = 0.204$$

$$k_{td}(t = 1999days) = \frac{1999}{12\left(\frac{100 - 4(6.0)}{6.0 + 20}\right) + 1999} = 0.983$$

$$\psi_b(t_f, t_i) = 1.9(1.0)(0.96)(0.714)(0.983)(1)^{-0.118} = 1.28$$

$$\varepsilon_{bih} = (1.0)(0.95)(0.714)(0.204)(0.48 \times 10^{-3}) = 0.000066$$

$$e_{psx} = 0.514in, e_{psy} = 35.566in$$

$$e_{px} = 0.514in, e_{py} = 38.003in$$

$$e_{tx} = 0.514in, e_{ty} = -25.357in$$

Permanent Strands

$$A_{ps} = N(a_{ps}) = 52(0.217in^2) = 11.284in^2$$

$$(e_{psy}I_{yy} + e_{psx}I_{xy})e_{py} = \left((35.556in)(251152.4in^4) + (0.514in)(17465.9in^4)\right)(38.003in) = 339707000in^6$$

$$(e_{psx}I_{xx} + e_{psy}I_{xy})e_{px} = \left((0.514in)(861860.5in^4) + (35.556in)(17465.9in^4)\right)(0.514in) = 546903in^6$$

$$K_{ih} = \frac{1}{1+\frac{28500ksi}{3048.131ksi}\frac{11.284in^2}{1211.371in^2}\left(1+(1211.371in^2)\frac{(339707000in^6-546903in^6)}{(861860.5in^4)(251152.4in^4)-(17465.9in^4)^2}\right)[1+0.7(1.30)]} = 0.676$$

$$\Delta f_{pSRH} = (0.000066)(28500ksi)(0.676) = 1.281ksi$$

Temporary Strands

$$A_{ps} = N\left(a_{ps}\right) = 2(0.217in^2) = 0.434in^2$$

$$\left(e_{psy}I_{yy}+e_{psx}I_{xy}\right)e_{ty} = \left((35.556in)(251152.4in^4)+(0.514in)(17465.9in^4)\right)(-25.357in) = -226665011n^6$$

$$\left(e_{psx}I_{xx}+e_{psy}I_{xy}\right)e_{tx} = \left((0.514in)(861860.5in^4)+(35.556in)(17465.9in^4)\right)(0.514in) = 546903in^6$$

$$K_{ih} = \frac{1}{1+\frac{28500ksi}{3048.131ksi}\frac{11.718in^2}{1211.371in^2}\left(1+(1211.371in^2)\frac{(-226665011in^6-546903in^6)}{(861860.5in^4)(251152.4in^4)-(17465.9in^4)^2}\right)[1+0.7(1.3)]} = 1.05$$

$$\Delta f_{pSRH} = (0.000066)(28500ksi)(1.05) = 1.985ksi$$

4.3.1.2 *Creep of Girder Concrete*

$$\Delta f_{pCRH} = \frac{E_P}{E_{ci}} f_{cgp}\psi_b(t_h,t_i)K_{ih}$$

$$\psi_b(t_h,t_i) = 1.9(1.0)(0.96)(0.714)(0.204)(1)^{-0.118} = 0.266$$

Permanent Strands

$$\Delta f_{CRH} = \frac{28500ksi}{3048.131ksi}(2.808ksi)(0.266)(0.676) = 4.721ksi$$

Temporary Strands

$$\Delta f_{CRH} = \frac{28500ksi}{3048.131ksi}(0.839ksi)(0.266)(1.05) = 2.186si$$

4.3.1.3 *Relaxation of Prestressing Strands*

The girder concrete holds the prestressing strand in tension. The concrete undergoes creep and shrinkage deformations. The strands are between two points that move toward one another. Relaxation occurs at a reduced rate compared to intrinsic relaxation. The relaxation equations given by the AASHTO LRFD BDS are for reduced relaxation.

$$\Delta f_{pR1H} = \left[\frac{f_{pt}}{K_L'}\frac{\log(24t_h)}{\log(24t_i)}\left(\frac{f_{pt}}{f_{py}}-0.55\right)\right]\left[1-\frac{3\left(\Delta f_{pSRH}+\Delta f_{pCRH}\right)}{f_{pt}}\right]K_{ih}$$

$$K_L' = 45$$

Permanent Strands

$$f_{pt} = 202.5ksi - 1.98ksi - 26.250ksi = 174.270ksi$$

$$\Delta f_{pR1H} = \left[\frac{174.270ksi}{45}\frac{\log(24\cdot 10)}{\log(24\cdot 1)}\left(\frac{174.270ksi}{243ksi}-0.55\right)\right]\left[1-\frac{3(1.281ksi+4.721ksi)}{174.270ksi}\right](0.676) = 0.677ksi$$

Temporary Strands

$$f_{pt} = 202.5ksi - 1.98ksi - 7.844ksi = 192.676ksi$$

$$\Delta f_{pR1H} = \left[\frac{192.676ksi}{45}\frac{\log(24\cdot 10)}{\log(24\cdot 1)}\left(\frac{192.676ksi}{243ksi} - 0.55\right)\right]\left[1 - \frac{3(1.985ksi + 2.186ksi)}{192.676ksi}\right](1.05) = 1.757ksi$$

PGSuper supports all three methods of computing relaxation described in the AASHTO LRFD BDS (LRFD 5.9.3.4.2c, C5.9.3.4.2c)

4.3.1.4 *Losses at Hauling*

$$\Delta f_{pLTH} = \Delta f_{pSRH} + \Delta f_{pCRH} + \Delta f_{pR1H}$$

$$\Delta f_{pH} = \Delta f_{pR0} + \Delta f_{pES} + \Delta f_{pLTH}$$

Permanent Strands

$$\Delta f_{pLTH} = 1.281ksi + 4.721ksi + 0.677ksi = 6.679ksi$$

$$\Delta_{pH} = 1.98ksi + 26.250ksi + 6.679ksi = 34.909ksi$$

Temporary Strands

$$\Delta f_{pLTH} = 1.985ksi + 2.186ksi + 1.757ksi = 5.927ksi$$

$$\Delta_{pH} = 1.98ksi + 7.844ksi + 5.927ksi = 15.751ksi$$

4.4 Losses between prestress transfer and installation of precast members

4.4.1.1 *Shrinkage of Girder Concrete*

$$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id}$$

$$\varepsilon_{bid} = k_s k_{hs} k_f k_{td} 0.48 \times 10^{-3}$$

$$K_{id} = \frac{1}{1 + \frac{E_p}{E_{ci}}\frac{A_{ps}}{A_g}\left(1 + \frac{A_g}{I_{xx}I_{yy} - I_{xy}^2}\left(\left(e_{psy}I_{yy} + e_{psx}I_{xy}\right)e_y - \left(e_{psx}I_{xx} + e_{psy}I_{xy}\right)e_x\right)\right)\left[1 + 0.7\psi_b(t_f, t_i)\right]}$$

$$\psi_b(t_f, t_i) = 1.9 k_s k_{hc} k_f k_{td} t_i^{-0.118}$$

$$k_s = 1.45 - 0.13\left(\frac{V}{S}\right) \geq 1.0 = 1.0$$

$$k_{hs} = 2.00 - 0.014H = 0.95$$

$$k_{hc} = 1.56 - 0.008H = 0.96$$

$$k_f = \frac{1}{1 + f'_{ci}} = 0.714$$

$$k_{td} = \frac{t}{12\left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20}\right) + t} = \begin{matrix} 0.772\ with\ t = (t_d - t_i) = 199\ day \\ 0.983\ with\ t = (t_f - t_i) = 1999\ day \end{matrix}$$

$$t_i = 1\ day$$

$$t_d = 120\ day$$

$$t_f = 2000\ day$$

$$\varepsilon_{bid} = (1.0)(0.95)(0.714)(0.772)(0.48 \times 10^{-3}) = 0.000252$$

$$\psi_b(t_f, t_i) = 1.9(1.0)(0.96)(0.714)(0.983)(1)^{-0.118} = 1.28$$

$$K_{id} = K_{ih} = 0.676$$

$$\Delta f_{pSR} = (0.000252)(28500ksi)(0.676) = 4.847\ ksi$$

4.4.1.2 *Creep of Girder Concrete*

$$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp} \psi_b(t_d, t_i) K_{id}$$

$$\psi_b(t_d, t_i) = 1.9(1.0)(0.96)(0.714)(0.772)(1)^{-0.118} = 1.01$$

$$\Delta f_{pCR} = \frac{28500ksi}{3048.131ksi}(2.807ksi)(1.01)(0.676) = 17.857\ ksi$$

4.4.1.3 *Relaxation of Prestressing Strands*

$$\Delta f_{pR1} = \left[\frac{f_{pt}}{K_L'} \frac{\log(24t_d)}{\log(24t_i)} \left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\left[1 - \frac{3(\Delta f_{pSRH} + \Delta f_{pCRH})}{f_{pt}}\right] K_{id}$$

$$f_{pt} = 202.5ksi - 1.98ksi - 26.250ksi = 174.270ksi$$

$$\Delta f_{pR1} = \left[\frac{174.270ksi}{45} \frac{\log(24 \cdot 120)}{\log(24 \cdot 1)} \left(\frac{174.270ksi}{243ksi} - 0.55\right)\right]\left[1 - \frac{3(4.847ksi + 17.857ksi)}{174.270ksi}\right](0.676) = 0.668ksi$$

4.4.1.4 *Time dependent losses*

$$\Delta f_{pLT_{id}} = \Delta f_{pSH} + \Delta f_{pCR} + \Delta f_{pR1}$$

$$\Delta f_{pLT_{id}} = 4.847ksi + 17.857ksi + 0.668ksi = 23.373\ ksi$$

4.5 Effect of temporary strand removal on permanent strands

$$P_{tr} = A_t(f_{pj} - \Delta f_{pH})$$

$$f_{ptr} = -\frac{P_{tr}}{A_g} - \frac{P_{tr}(e_{tx}I_{xx} - e_{ty}I_{xy})}{I_{xx}I_{yy} - I_{xy}^2} e_{px} + \frac{P_{tr}(e_{ty}I_{yy} - e_{tx}I_{xy})}{I_{xx}I_{yy} - I_{xy}^2} e_{py}$$

$$\Delta f_{ptr} = \frac{E_p}{E_c} f_{ptr}$$

$$A_t = (2)(0.217in^2) = 0.434in^2$$

$$P_{tr} = (0.434in^2)(202.5ksi - 7.907ksi) = 81.05\ kip$$

$$f_{ptr} = -\frac{81.05kip}{1211.371in^2} - \frac{81.05kip((0.514in)(861860.5in^4) - (-25.357in)(17465.9in^4))}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2} + \frac{81.05kip((-25.357in)(251152.4in^4) - (0.514in)(17465.9in^4))}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2} = 0.024ksi$$

$$\Delta f_{ptr} = \left(\frac{28500ksi}{3176.667ksi}\right)(0.024ksi) = 0.213ksi$$

4.6 Losses between precast member installation and final

4.6.1.1 *Shrinkage of Girder Concrete*

$$\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df}$$

$$\varepsilon_{bdf} = \varepsilon_{bif} - \varepsilon_{bid}$$

$$\varepsilon = k_s k_{hs} k_f k_{td} 0.48 \times 10^{-3}$$

$$K_{df} = \frac{1}{1 + \frac{E_p}{E_{ci}} \frac{A_{ps}}{A_c} \left(1 + \frac{A_C e_c^2}{I_c}\right) \left[1.0 + 0.7\psi_b\left(t_f, t_i\right)\right]}$$

From before

$$k_s = 1.0$$

$$k_{hs} = 0.95$$

$$k_{hc} = 0.96$$

$$k_f = 0.714$$

$$\psi_b\left(t_f, t_i\right) = 1.28$$

$$\varepsilon_{bid} = 0.000252$$

$$k_{td}\left(t = t_f - t_i = 1999 day\right) = 0.983$$

$$\varepsilon_{bif} = (1.0)(0.95)(0.714)(0.983)(0.48 \times 10^{-3}) = 0.000320$$

$$\varepsilon_{bdf} = 0.000320 - 0.000252 = 0.000069$$

$$e_c = e + y_{bc} - y_b = 38.003 in + 44.199 in - 41.643 in = 40.559 in$$

$$K_{df} = \frac{1}{1 + \left(\frac{28500 ksi}{3048.131 ksi}\right)\left(\frac{11.284 in^2}{1351.106 in^2}\right)\left(1 + \frac{(1351.106 in^2)(40.559 in)^2}{938938.9 in^4}\right)\left(1 + 0.7(1.28)\right)} = 0.676$$

$$\Delta f_{pSD} = (0.000069)(28500 ksi)(0.676) = 1.320\ ksi$$

4.6.1.2 *Creep of Girder Concrete*

$$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp}\left[\psi_b\left(t_f, t_i\right) - \psi_b\left(t_d, t_i\right)\right] K_{df} + \frac{E_p}{E_c}\left(\Delta f_{cd} + \Delta f_{ptr}\right)\psi_b\left(t_f, t_d\right) K_{df}$$

$$\Delta f_{cd} = -\left(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}\right)\left(\frac{A_{ps}}{A_g}\right)\left(1 + \frac{\left(e_{psy} I_{yy} + e_{psx} I_{xy}\right) e_{psy} - \left(e_{psx} I_{xx} + e_{psy} I_{xy}\right) e_{psx}}{I_{xx} I_{yy} - I_{xy}^2}\right) - \left(\Delta f'_{cd} + \Delta f''_{cd}\right)$$

$$M_{adl} = M_{diaphragm} + M_{longitudinal\ joint}$$

$$M_{adl} = 86.69 k \cdot ft + 235.56 k \cdot ft = 322.25 k \cdot ft$$

$$\Delta f'_{cd} = \frac{M_{adl}\left(I_{yy} e_{py} - I_{xy} e_{px}\right)}{I_{xx} I_{yy} - I_{xy}^2}$$

$$\Delta f'_{cd} = \frac{(322.254 k \cdot ft)\left(\frac{12 in}{1 ft}\right)\left((251152.4 in^4)(38.003 in) - (17465.9 in^4)(0.514 in)\right)}{(861860.5 in^4)(251152.4 in^4) - (17465.9 in^4)^2} = 0.171\ ksi$$

$$\Delta f''_{cd} = \frac{M_{sidl}\left(Y_{bc} - Y_{bg} + e\right)}{I_c}$$

$$M_{sidl} = M_{nonstructural\ overlay} + M_{nonstructural\ haunch} + M_{barrier} + M_{future\ overlay}$$

$$M_{sidl} = 356.27 k \cdot ft + 1403.73 k \cdot ft + 693.69 k \cdot ft + 668.56 k \cdot ft = 3122.25 k \cdot ft$$

$$\Delta f''_{cd} = \frac{(3122.25 k \cdot ft)(44.199 in - 41.643 in + 38.003 in)}{938938.9 in^4}\left(\frac{12 in}{1 ft}\right) = 1.618 ksi$$

$$\left(e_{psy}I_{yy} + e_{psx}I_{xy}\right)e_{psy} = \left((38.003in)(251152.4in^4) + (0.514in)(17465.9in^4)\right)(38.003in) = 363062502in^6$$

$$\left(e_{psx}I_{xx} + e_{psy}I_{xy}\right)e_{psx} = \left((0.514in)(861860.5in^4) + (38.003in)(17465.9in^4)\right)(0.514in) = 363939in^6$$

$$\Delta f_{cd} = -(4.847ksi + 17.857ksi + 0.668ksi)\left(\frac{11.284in^2}{1211.371in^3}\right)\left(1 + \frac{363062502in^6 - 363939in^6}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}\right) - (0.171ksi + 1.618)ksi = -2.424\ ksi$$

$$k_{td} = \frac{t}{12\left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20}\right) + t} = 0.982\ with\ t = \left(t_f - t_d\right) = 1880\ day$$

$$\psi_b\left(t_f, t_d\right) = 1.9(1.0)(0.96)(0.714)(0.982)(120)^{-0.118} = 0.727$$

$$\Delta f_{pCD} = \left(\frac{28500ksi}{3048.131ksi}\right)(2.807ksi)(1.28 - 1.01)(0.676) + \left(\frac{28500ksi}{3176.667ksi}\right)(-2.424ksi + 0.213ksi)(0.727)(0.676) = -4.882\ ksi$$

4.6.1.3 *Relaxation of Prestressing Strands*

$$\Delta f_{pR2} = \Delta f_{pR1} = 0.668\ ksi$$

4.6.1.4 *Shrinkage of Deck Concrete*

There is not a deck

$$\Delta f_{pSS} = 0.0\ ksi$$

4.6.1.5 *Time Dependent Losses*

$$\Delta f_{pLT_{df}} = \Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR1} - \Delta f_{pSS} = 1.320ksi - 4.882ksi + 0.668ksi - 0.0ksi = -2.894\ ksi$$

4.7 Elastic Gains

4.7.1.1 *Dead load on noncomposite section*

$$\Delta f_{pED} = \frac{E_p}{E_c}\Delta f'_{cd}$$

$$\Delta f_{pED} = \left(\frac{28500ksi}{3176.667ksi}\right)(0.171ksi) = 1.533\ ksi$$

4.7.1.1.1 Superimposed dead loads

$$\Delta f_{pSIDL} = \frac{E_p}{E_c}\Delta f''_{cd} = \left(\frac{28500ksi}{3176.667ksi}\right)(1.618ksi) = 14.520\ ksi$$

4.7.1.1.2 Live Loads

$$\Delta f_{pLL} = \frac{E_P}{E_c}\Delta f'''_{cd}$$

$$\Delta f'''_{cd} = \frac{M_{llim}\left(Y_{bc} - Y_{bg} + e\right)}{I_c}$$

$$\Delta f_{cd}''' = \begin{cases} \dfrac{(2942.95k\cdot ft)(44.199in - 41.643in + 38.003in)}{938938.9in^4}\left(\dfrac{12in}{1ft}\right) = 1.526\ ksi\ (Design\ Live\ Load) \\ \dfrac{(830.54k\cdot ft)(44.199in - 41.643in + 38.003in)}{938938.9in^4}\left(\dfrac{12in}{1ft}\right) = 0.431\ ksi\ (Fatigue\ Live\ Load) \end{cases}$$

$$\Delta f_{pLL} = \begin{cases} \left(\dfrac{28500ksi}{3176.667ksi}\right)(1.526ksi) = 13.676ksi = \Delta f_{pLL-Design}(Design\ Live\ Load) \\ \left(\dfrac{28500ksi}{3176.667ksi}\right)(0.431ksi) = 3.862ksi = \Delta f_{pLL-Fatigue}(Fatigue\ Live\ Load) \end{cases}$$

4.8 Effective Prestress Summary

$$\Delta f_{pLT} = \Delta f_{pLT_{id}} + \Delta f_{pLT_{df}} = 23.632ksi - 2.894ksi = 20.749ksi$$

$$\Delta f_{pT} = \Delta f_{pR0} + \Delta f_{pES} + \Delta f_{ptr} + \Delta f_{pLT} - \Delta f_{pED} - \Delta f_{pSIDL}$$
$$= 1.98ksi + 26.250ksi + 0.213ksi + 20.749ksi - 1.533si - 14.520ksi = 32.868ksi$$

$$f_{pe} = f_{pj} - \Delta f_{pT} + \begin{cases} 1.0\Delta f_{pLL-Design}\ (Service\ I) \\ 0.8\Delta f_{pLL-Design}\ (Service\ III) \\ 1.5\Delta f_{pLL-Fatigue}\ (Fatigue\ I) \end{cases}$$

$$Service\ I\ f_{pe} = 202.5ksi - 32.868ksi + 1.0(13.676ksi) = 183.318\ ksi$$
$$Service\ III\ f_{pe} = 202.5ksi - 32.868ksi + 0.8(13.676ksi) = 180.581\ ksi$$
$$Fatigue\ I\ f_{pe} = 202.5ksi - 32.868ksi + 1.5(3.862ksi) = 175.426\ ksi$$

5 Stresses

5.1 Final Stresses

Check the final stress conditions first. If the final stresses exceed the limiting stresses, there is not point evaluating the remainder of the design.

The final stress is computed as the sum of the stress on the non-composite section and the composite section. Because the section is asymmetric, a biaxial stress analysis is performed for the non-composite section. The uniaxial analysis is used for composite section stresses.

$$f = f_{nc} + f_c$$

$$f_{nc}(x,y) = \frac{M_y I_{xx} + M_x I_{xy}}{I_{xx}I_{yy} - I_{xy}^2}x - \frac{M_x I_{yy} + M_y I_{xy}}{I_{xx}I_{yy} - I_{xy}^2}y + \frac{P}{A}$$

$$f_c = \frac{M_{sidl} + \gamma_{llim}M_{llim}}{S_c} + f_{ss}$$

$$M_y = Pe_x$$

$$M_x = Pe_y + M_g + M_{adl}$$

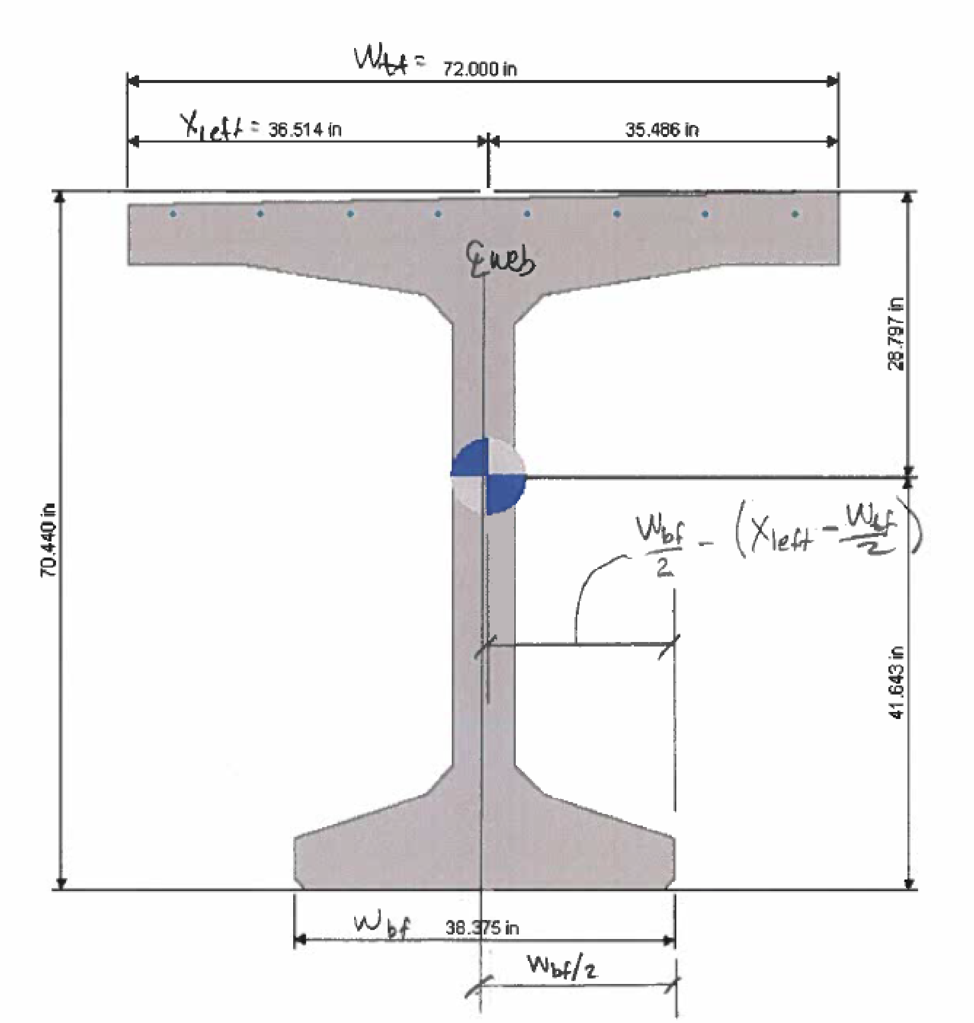
5.1.1 Stress due to slab shrinkage

There is no slab to shrink

$$f_{ss} = 0.0\ ksi$$

5.1.2 Service III

Stress at right corner of bottom flange



$$x = \frac{W_{bf}}{2} - \left(X_{left} - \frac{W_{tf}}{2}\right) = \frac{38.375in}{2} - \left(36.514in - \frac{72in}{2}\right) = 18.6735in$$

$$y = -41.643in$$

Stress due to prestressing

$$P = -(50)(0.217in^2)(180.581ksi) = -1959.3kip$$

$$M_x = (-1959.3kip)(38.003in) = -74459.3k \cdot in$$

$$M_y = (-1959.3kip)(0.514in) = -1007.1k \cdot in$$

$$f_{ps}(x,y) = \frac{(-1007.1k \cdot in)(861860.5in^4) + (-74459.3k \cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(18.6735in)$$
$$- \frac{(-74459.3k \cdot in)(251152.4in^4) + (-1007.1k \cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(-41.643in) - \frac{1959.3kip}{1211.371in^2}$$
$$= 0.187ksi - 3.606ksi - 1.617ksi = -5.411ksi$$

Stress due to gravity loads on non-composite section

$$M_x = M_g + M_{adl} = (3512.77k \cdot ft + 86.69k \cdot ft + 235.59k \cdot ft)\left(\frac{12in}{1ft}\right) = 46020.6k \cdot in$$

$$M_y = 0\ k \cdot in$$

$$f(x,y) = \frac{(0k\cdot in)(861860.5in^4) + (46020.6k\cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(18.6735in) - \frac{(46020.6k\cdot in)(251152.4in^4) + (0k\cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(-41.643in) = 0.069ksi + 2.227ksi = 2.296ksi$$

Stress due to gravity loads on composite section

$$M_x = M_{sidl} = (356.27k\cdot ft + 1403.73k\cdot ft + 693.69k\cdot ft + 668.56k\cdot ft)\left(\frac{12in}{1ft}\right) = 37467.24k\cdot in$$

$$f = \frac{M}{S} = \frac{37467.24k\cdot in}{21243.5in^3} = 1.764ksi$$

Stress due to live load on composite section

$$M = 2942.95k\cdot ft\left(\frac{12in}{1ft}\right) = 35315.4k\cdot in$$

$$f = \frac{M}{S} = \frac{35315.4k\cdot in}{21243.5in^3} = 1.662ksi$$

Total stress

$$f = -5.411ksi + 2.296ksi + 1.764ksi + 0.8(1.662ksi) = -5.411ksi + 5.389ksi = -0.022ksi$$

$$-0.022ksi < 0ksi\ \boldsymbol{OK}$$

To illustrate the effect of the girder asymmetry on stresses, flexural stresses computed based on a uniaxial bending assumption are shown here and in the stress computations that follow. The percent error is computed

$$f = \frac{P}{A} + \frac{Pe}{S} + \frac{M_g}{S} + \frac{M}{S_c}$$

$$f_b = -\frac{1959.3kip}{1211.37in^2} + \frac{(-1959.3kip)(38.003in)}{20696.2in^3} + \frac{46020.6k\cdot in}{20696.2in^3} + \frac{37467.24k\cdot in + 0.8(35315.4k\cdot in)}{21243.5in^3} = -5.215ksi + 5.317ksi = 0.102si$$

$$\frac{0.102ksi - (-0.022ksi)}{-0.022ksi}100\% = -564\%$$

5.1.3 Service I

Stress at left corner of top flange

$$x = -X_{left} = -36.514in$$

$$y = y_t - 0.02(W_{tf}) = 28.797in - 0.02(72in) = 27.357in$$

Stress due to prestressing

$$P = -(50)(0.217in^2)(183.318ksi) = -1989.0kip$$

$$M_x = (-1989.0kip)(38.003in) = -75587.9k\cdot in$$

$$M_y = (-1989.0kip)(0.514in) = -1022.3k\cdot in$$

$$f_{ps}(x,y) = \frac{(-1022.3k\cdot in)(861860.5in^4) + (-75587.9k\cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(-36.514in) - \frac{(-75587.9k\cdot in)(251152.4in^4) + (-1022.3k\cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(27.357in) - \frac{1989.0kip}{1211.371in^2} = 0.372ksi + 2.405ksi - 1.642ksi = 1.135ksi$$

Stress due to gravity loads on non-composite section

$$M_x = M_g + M_{adl} = (3512.77k\cdot ft + 86.69k\cdot ft + 235.59k\cdot ft)\left(\frac{12in}{1ft}\right) = 46020.6k\cdot in$$

$$M_y = 0\ k\cdot in$$

$$f(x,y) = \frac{(0k\cdot in)(861860.5in^4) + (46020.6k\cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(-36.514in) - \frac{(46020.6k\cdot in)(251152.4in^4) + (0k\cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(27.357in) = -0.136ksi - 1.463ksi = -1.599ksi$$

Stress due to gravity loads on composite section

$$M_x = M_{sidl} = (356.27k\cdot ft + 1403.73k\cdot ft + 693.69k\cdot ft + 668.56k\cdot ft)\left(\frac{12in}{1ft}\right) = 37467.24k\cdot in$$

$$f = \frac{M}{S} = \frac{37467.24k\cdot in}{-35781.3in^3} = -1.047ksi$$

Stress due to live load on composite section

$$M = 2942.95k\cdot ft\left(\frac{12in}{1ft}\right) = 35315.4k\cdot in$$

$$f = \frac{M}{S} = \frac{35315.4k\cdot in}{-35781.3n^3} = -0.987ksi$$

Total stress

$$f = 1.015ksi - 1.599ksi - 1.047ksi + 1.0(-0.987ksi) = 1.135ksi - 3.633ksi = -2.498ksi$$

Stress limit $-0.6f'_c = -0.6(6.8ksi) = -4.080ksi$

$$-4.080ksi < -2.498ksi\ \boldsymbol{OK}$$

$$f = \frac{P}{A} + \frac{Pe}{S} + \frac{M_g}{S} + \frac{M}{S_c}$$

$$f_t = -\frac{1989.0kip}{1211.37in^2} + \frac{(-1989.0kip)(38.003in)}{-29929.2in^3} + \frac{46020.6k\cdot in}{-29929.2in^3} + \frac{37467.24k\cdot in + 35315.4k\cdot in}{-35781.3in^3} = 0.884ksi - 3.572ksi = -2.688ksi$$

$$\frac{-2.688ksi - (-2.498ksi)}{-2.498ksi}100\% = 8\%$$

5.1.4 Fatigue I

Stress at left corner of top flange

$$x = -X_{left} = -36.514in$$

$$y = y_t - 0.02(W_{tf}) = 28.797in - 0.02(72in) = 27.357in$$

Stress due to prestressing

$$P = -(50)(0.217in^2)(175.426ksi) = -1903.4kip$$

$$M_x = (-1903.4kip)(38.003in) = -72334.9k\cdot in$$

$$M_y = (-1903.4kip)(0.514in) = -978.3k\cdot in$$

$$f_{ps}(x,y) = \frac{(-978.3k \cdot in)(861860.5in^4) + (-72334.9k \cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(-36.514in) - \frac{(-72334.9k \cdot in)(251152.4in^4) + (-978.3k \cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(27.357in) - \frac{1903.4kip}{1211.371in^2} = 0.356ksi + 2.301ksi - 1.571ksi = 1.086ksi$$

Stress due to gravity loads on non-composite section

$$M_x = M_g + M_{adl} = (3512.77k \cdot ft + 86.69k \cdot ft + 235.59k \cdot ft)\left(\frac{12in}{1ft}\right) = 46020.6k \cdot in$$

$$M_y = 0\ k \cdot in$$

$$f(x,y) = \frac{(0k \cdot in)(861860.5in^4) + (46020.6k \cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(-36.514in) - \frac{(46020.6k \cdot in)(251152.4in^4) + (0k \cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(27.357in) = -0.136ksi - 1.463ksi = -1.599ksi$$

Stress due to gravity loads on composite section

$$M_x = M_{sidl} = (356.27k \cdot ft + 1403.73k \cdot ft + 693.69k \cdot ft + 668.56k \cdot ft)\left(\frac{12in}{1ft}\right) = 37467.24k \cdot in$$

$$f = \frac{M}{S} = \frac{37467.24k \cdot in}{-35781.3in^3} = -1.047ksi$$

Stress due to live load on composite section

$$M = 830.54k \cdot ft\left(\frac{12in}{1ft}\right) = 9966.48k \cdot in$$

$$f = \frac{M}{S} = \frac{9966.48k \cdot in}{-35781.3in^3} = -0.279ksi$$

Total stress

$$f = 0.5(1.086ksi) + 0.5(-1.599ksi) + 0.5(-1.047ksi) + 1.5(-0.279ksi) = 0.543ksi - 1.741ksi = -1.198ksi$$

Stress limit $-0.4f'_c = -0.4(6.8ksi) = -2.720ksi$

$$-2.720ksi < -1.198ksi\ \boldsymbol{OK}$$

$$f = \frac{P}{A} + \frac{Pe}{S} + \frac{M_g}{S} + \frac{M}{S_c}$$

$$f_t = -\frac{(0.5)(1903.4kip)}{1211.37in^2} + \frac{(0.5)(-1903.4kip)(38.003in)}{-29929.2in^3} + \frac{0.5(46020.6k \cdot in)}{-29929.2in^3} + \frac{0.5(37467.24k \cdot in) + 1.5(9966.48k \cdot in)}{-35781.3in^3} = 0.423ksi - 1.710ksi = -1.287ksi$$

$$\frac{-1.287ksi - (-1.198ksi)}{-1.198ksi}100\% = 8\%$$

5.2 Initial Stresses

Evaluate stresses immediately after release.

$$f(x,y) = \frac{M_y I_{xx} + M_x I_{xy}}{I_{xx}I_{yy} - I_{xy}^2}x - \frac{M_x I_{yy} + M_y I_{xy}}{I_{xx}I_{yy} - I_{xy}^2}y + \frac{P}{A}$$

The governing stress immediately after release occurs at the point of prestress transfer. From PGSuper, the effective prestress is $P = -1918.77kip$ in the permanent strands and $P = -85.19kip$ in the temporary strands. The permanent strand eccentricity is $e_{px} = 0.378in, e_{py} = 29.519in$. The temporary strand eccentricity is $e_{tx} = 0.378in, e_{ty} = -17.170in$

$$M_x = Pe_y = -1918.77kip(29.519in) - 85.19kip(-17.170in) = -55177.5k \cdot in$$

$$M_y = Pe_x = (-1918.77kip - 85.19kip)(0.378in) = -757.5k \cdot in$$

Bottom right

$$x = \frac{W_{bf}}{2} - \left(X_{left} - \frac{W_{tf}}{2}\right) = \frac{38.375in}{2} - \left(36.378in - \frac{72in}{2}\right) = 18.8095in$$

$$y = -Y_b = -49.830in$$

$$\begin{aligned} f_{ps}(x,y) = {} & \frac{(-757.5k \cdot in)(117488.7in^4) + (-55177.5k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(18.8095in) \\ & - \frac{(-55177.5k \cdot in)(438325.6in^4) + (-757.5k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(-49.830in) \\ & - \frac{1918.77kip + 85.19kip}{1644.48in^2} = -0.065ksi - 2.349ksi - 1.219ksi = -3.633ksi \end{aligned}$$

Top left

$$x = -X_{left} = -36.378in$$

$$y = Y_t - 0.02(W_{tf}) = 26.625n - 0.02(72in) = 25.185in$$

$$\begin{aligned} f_{ps}(x,y) = {} & \frac{(-757.5k \cdot in)(117488.7in^4) + (-55177.5k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(-36.378in) \\ & - \frac{(-55177.5k \cdot in)(438325.6in^4) + (-757.5k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(25.185in) \\ & - \frac{1918.77kip + 85.19kip}{1644.48in^2} = 0.126ksi + 1.187ksi - 1.219ksi = 0.095ksi \end{aligned}$$

Dead load stresses

$$M_x = M_g = 285.98k \cdot ft = 3431.76k \cdot in$$

Bottom right

$$\begin{aligned} f_g(x,y) = {} & \frac{(0k \cdot in)(117488.7in^4) + (3431.76k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(18.8095in) \\ & - \frac{(3431.76k \cdot in)(438325.6in^4) + (-757.0k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(-49.830in) = 0.002ksi + 0.146ksi \\ & = 0.148ksi \end{aligned}$$

Top left

$$\begin{aligned} f_g(x,y) = {} & \frac{(0k \cdot in)(117488.7in^4) + (3431.76k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(-36.378in) \\ & - \frac{(3431.76k \cdot in)(438325.6in^4) + (-757.0k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(25.185in) = -0.004ksi - 0.074ksi \\ & = -0.078ksi \end{aligned}$$

Stress Limit Evaluation

Bottom right

$$f = -3.633ksi + 0.148ksi = -3.485ksi$$

Stress limit $-0.65f'_{ci} = -0.65(6.0ksi) = -3.900ksi$

$$-3.900ksi < -3.485\ ksi\ \boldsymbol{OK}$$

Top left

$$f = 0.095ksi - 0.078ksi = 0.017ksi$$

Concrete density modification factor (LRFD 5.4.2.8)

$$0.75 \le \lambda = 7.5w_c \le 1.0\ when\ f_{ct} is\ not\ specified$$

$$\lambda = 7.5(0.125kcf) = 0.9375$$

Stress limit $0.0948\lambda\sqrt{f'_{ci}} \le 0.200ksi = 0.0948(0.9375)\sqrt{6.0ksi} = 0.217ksi \rightarrow 0.200ksi$

$$0.017ksi < 0.200ksi\ \boldsymbol{OK}$$

$$f = \frac{P}{A} + \frac{Pe}{S} + \frac{M_g}{S}$$

$$f_b = -\frac{1918.77kip + 85.19kip}{1644.448in^2} + \frac{-1918.77kip(29.519in) - 85.19kip(-17.170in)}{23509.9in^3} + \frac{3431.76k \cdot in}{23509.9in^3}$$
$$= -3.566ksi + 0.146ksi = -3.420ksi$$

$$\frac{-3.420ksi - (-3.485ksi)}{-3.485ksi}100\% = -2\%$$

$$f_t = -\frac{1918.77kip + 85.19kip}{1644.448in^2} + \frac{-1918.77kip(29.519in) - 85.19kip(-17.170in)}{-43998.9in^3} + \frac{3431.76k \cdot in}{-43998.9in^3}$$
$$= 0.035ksi - 0.078ksi = -0.043ksi$$

$$\frac{-0.043ksi - (0.017ksi)}{0.017ksi}100\% = -353\%$$

5.3 Cast Longitudinal Joints

Evaluate stresses immediately after the longitudinal joints are cast. Controlling point is at the point of prestress transfer.

This is not an AASHTO LRFD requirement. BDM 5.2.1C provides stress limits at erection. The governing erection stress case for the noncomposite girder is after the longitudinal joints are cast.

$$f(x,y) = \frac{M_y I_{xx} + M_x I_{xy}}{I_{xx}I_{yy} - I_{xy}^2}x - \frac{M_x I_{yy} + M_y I_{xy}}{I_{xx}I_{yy} - I_{xy}^2}y + \frac{P}{A}$$

Stress due to prestressing

$$P = -1645.27kip$$

$$M_x = (-1645.27kip)(29.519in) = -48566.7k \cdot in$$

$$M_y = (-1645.27kip)(0.378in) = -621.9k \cdot in$$

Bottom right

$$x = \frac{W_{bf}}{2} - \left(X_{left} - \frac{W_{tf}}{2}\right) = \frac{38.375in}{2} - \left(36.378in - \frac{72in}{2}\right) = 18.8095in$$

$$y = -Y_b = -49.830in$$

$$\begin{aligned} f_{ps}(x,y) = &\frac{(-621.9k \cdot in)(117488.7in^4) + (-48566.7k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(18.8095in) \\ &- \frac{(-48566.7k \cdot in)(438325.6in^4) + (-621.9k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(-49.830in) - \frac{1645.27kip}{1644.48in^2} \\ &= -0.055ksi - 2.068ksi - 1.000ksi = -3.124ksi \end{aligned}$$

Top left

$$x = -X_{left} = -36.378in$$

$$y = Y_t - 0.02\left(W_{tf}\right) = 26.625n - 0.02(72in) = 25.185in$$

$$\begin{aligned} f_{ps}(x,y) = &\frac{(-621.9 \cdot in)(117488.7in^4) + (-48566.7k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(-36.378in) \\ &- \frac{(-48566.7k \cdot in)(438325.6in^4) + (-621.9k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(25.185in) - \frac{1645.27kip}{1644.48in^2} \\ &= 0.107ksi + 1.044ksi - 1.000ksi = 0.152ksi \end{aligned}$$

Stress due to Dead Loads

$$M_x = M_g + M_{diaphragm} + M_{lj} = (169.17k \cdot ft + 3.00k \cdot ft + 12.14k \cdot ft)\left(\frac{12in}{1ft}\right) = 2211.72k \cdot in$$

Bottom right

$$\begin{aligned} f_{nc}(x,y) = &\frac{(0k \cdot in)(117488.7in^4) + (2211.72k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(18.8095in) \\ &- \frac{(2211.72k \cdot in)(438325.6in^4) + (-757.0k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(-49.830in) = 0.001ksi + 0.094ksi \\ &= 0.095ksi \end{aligned}$$

Top left

$$\begin{aligned} f_{nc}(x,y) = &\frac{(0k \cdot in)(117488.7in^4) + (2211.72k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(-36.378in) \\ &- \frac{(2211.72k \cdot in)(438325.6in^4) + (-757.0k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(25.185in) = -0.003ksi - 0.048ksi \\ &= -0.050ksi \end{aligned}$$

Stress Limit Evaluation

Bottom right

$$f = -3.124ksi + 0.095ksi = -3.029ksi$$

Stress limit $-0.45f'_c = -0.45(6.8ksi) = -3.060ksi$

$$-3.060\ ksi < \ -3.029ksi\ \boldsymbol{OK}$$

Top left

$$f = 0.152ksi - 0.050ksi = 0.102ksi$$

Stress limit $0.19\lambda\sqrt{f'_c} = 0.19(0.9375)\sqrt{6.8ksi} = 0.464ksi$

$$0.102ksi < 0.464ksi\ \boldsymbol{OK}$$

$$f = \frac{P}{A} + \frac{Pe}{S} + \frac{M_g}{S}$$

$$f_b = -\frac{1645.27kip}{1644.448in^2} + \frac{-1645.27kip(29.519in)}{23509.9in^3} + \frac{2211.72k \cdot in}{23509.9in^3} = -3.066ksi + 0.094ksi = -2.972ksi$$

$$\frac{-2.972ksi - (-3.029ksi)}{-3.029ksi} 100\% = -2\%$$

$$f_t = -\frac{1645.27kip}{1644.448in^2} + \frac{-1645.27kip(29.519in)}{-43998.9in^3} + \frac{2211.72k \cdot in}{-43998.9in^3} = 0.103ksi - 0.050ksi = 0.053ksi$$

$$\frac{0.053ksi - 0.102ksi}{0.102ksi} 100\% = -48\%$$

5.4 After Superimposed Dead Loads (Permanent Loads Only)

$$f_{nc}(x,y) = \frac{M_y I_{xx} + M_x I_{xy}}{I_{xx} I_{yy} - I_{xy}^2} x - \frac{M_x I_{yy} + M_y I_{xy}}{I_{xx} I_{yy} - I_{xy}^2} y + \frac{P}{A}$$

$$f_c = \frac{M}{S}$$

$$f = f_{nc} + f_c$$

Stress due to Prestressing

At point of prestress transfer

From PGSuper, the effective prestress is $P = -1598.46kip$.

$$M_x = Pe_y = -1598.46kip(29.519in) = -47184.9k \cdot in$$

$$M_y = Pe_x = (-1598.46kip)(0.378in) = -604.2k \cdot in$$

Bottom right

$$x = \frac{W_{bf}}{2} - \left(X_{left} - \frac{W_{tf}}{2}\right) = \frac{38.375in}{2} - \left(36.378in - \frac{72in}{2}\right) = 18.8095in$$

$$y = -Y_b = -49.830in$$

$$\begin{aligned} f_{ps}(x,y) = &\frac{(-604.2k \cdot in)(117488.7in^4) + (-47184.9k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(18.8095in) \\ &- \frac{(-47184.9k \cdot in)(438325.6in^4) + (-604.2k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(-49.830in) - \frac{1598.46kip}{1644.48in^2} \\ &= -0.054ksi - 2.009ksi - 0.972ksi = -3.036ksi \end{aligned}$$

Dead load stresses

$$M_x = M_g + M_{diaphragm} + M_{lj} = (169.17k \cdot ft + 3.00k \cdot ft + 12.14k \cdot ft)\left(\frac{12in}{1ft}\right) = 2211.72k \cdot in$$

$$\begin{aligned} f_{nc}(x,y) = &\frac{(0k \cdot in)(117488.7in^4) + (2211.72k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(18.8095in) \\ &- \frac{(2211.72k \cdot in)(438325.6in^4) + (-757.0k \cdot in)(16115.2in^4)}{(1171488.7in^4)(438325.6in^4) - (16115.2in^4)^2}(-49.830in) = 0.001ksi + 0.094ksi \\ &= 0.095ksi \end{aligned}$$

$$\begin{aligned} f_c = &\frac{\left(M_{nonstructural\ overlay} + M_{nonstructural\ haunch} + M_{barrier}\right)}{S_{bc}} = \frac{(16.25k \cdot ft + 53.17k \cdot ft + 31.64k \cdot ft)\left(\frac{12in}{1ft}\right)}{24021.8in^3} \\ &= \frac{1212.72k \cdot in}{24021.8in^3} = 0.050ksi \end{aligned}$$

Stress Limit Evaluation

$$f = -3.036ksi + 0.095ksi + 0.050ksi = -2.891ksi$$

Stress limit $-0.45f_c' = -0.45(6.8ksi) = -3.606ksi$

$$-3.060\ ksi < -2.891ksi\ \boldsymbol{OK}$$

$$f = \frac{P}{A} + \frac{Pe}{S} + \frac{M_g}{S} + \frac{M}{S_c}$$

$$f_b = -\frac{1598.46kip}{1644.448in^2} + \frac{(-1598.46kip)(29.519in)}{23509.9in^3} + \frac{2211.72k \cdot in}{23509.9in^3} + \frac{1212.72k \cdot in}{24021.8in^3} = -2.979ksi + 0.145ksi = -2.834ksi$$

$$\frac{-2.834ksi - (-2.891ksi)}{-2.891ksi}100\% = -2\%$$

5.5 Lifting

5.5.1 Check girder stability

Designing precast, prestressed concrete bridge girders for lateral stability ensures safety and constructability. PCI's Aspire Magazine[3] presents WSDOT's perspective on stability design.

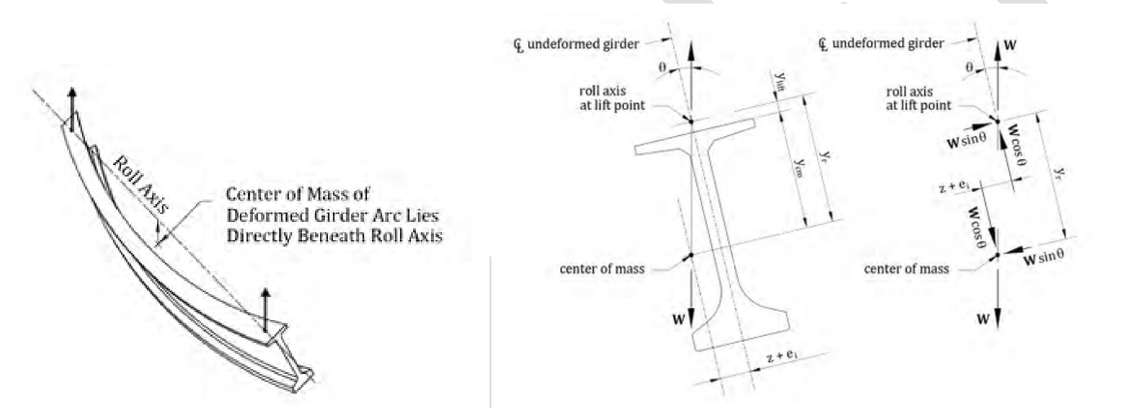


Figure 5-1: Equilibrium of Hanging Girder

Because of the asymmetric of the girder section and the longitudinal top flange thickening, it is easier to solve this problem numerically than with closed form equations.

5.5.1.1 *Vertical Location of Center of Gravity*

5.5.1.1.1 Estimate Camber

Compute camber for the girder in the hanging configuration. However, the stability analysis procedure needs the camber measured from a datum at the ends of the girder, not the lift points.

5.5.1.1.1.1 Girder

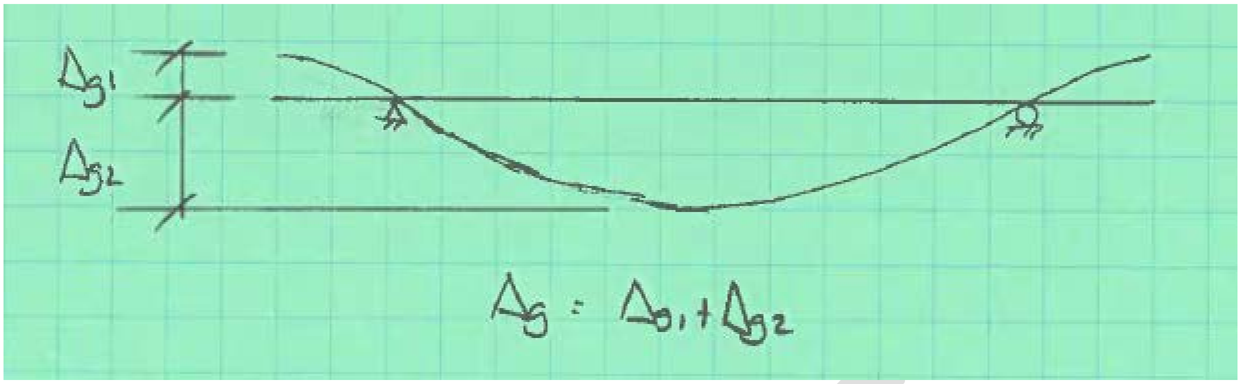


Figure 5-2: Girder Self-Weight Deflection during Lifting

The self-weight deflections are computed using a finite element model.

At girder ends

$$\Delta_{g1} = 0.798in$$

Mid-span

$$\Delta_{g2} = -3.456in$$

Total

$$\Delta_{gy} = -3.456n - 0.798in = -4.254in$$

The associated lateral deflection due to asymmetry is

$$\Delta_{gx} = -\frac{I_{xy}}{I_{yy}}\Delta_{gy} = -\frac{17465.9in^4}{251152.4in^4}(-4.254in) = 0.296in$$

5.5.1.1.1.2 Prestressing

The customary equations for prestress induced deflections must be modified for girders with longitudinal top flange thickening. See Appendix A for a derivation of the equations.

5.5.1.1.1.2.1 Straight Strands

$$P = \left(\frac{36}{36+14}\right)(1890.83kip) = 1361.40kip$$

$$\Delta_{ssy1} = \frac{P\left(e_y + \frac{2}{3}(Y_{bm} - Y_{be})\right)L^2}{8E_{ci}}\left(\frac{I_{yy}}{I_{xx}I_{yy} - I_{xy}^2}\right)$$
$$= \left[\frac{(1361.40kip)\left(47.040in + \frac{2}{3}(41.643in - 50.373in)\right)(157.75ft)^2}{8(3048.131ksi)}\right]\left(\frac{251152.4in^4}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}\right)\left(\frac{144in^2}{1ft^2}\right)$$
$$= 9.582in$$

$$\Delta_{ssx1} = \frac{P\left(e_x + \frac{2}{3}(X_{lm} - X_{le})\right)L^2}{8E_{ci}}\left(\frac{I_{xx}}{I_{xx}I_{yy} - I_{xy}^2}\right)$$
$$= \frac{(1361.40kip)\left(0.370in + \frac{2}{3}(36.514in - 36.370in)\right)(157.75ft)^2}{8(3048.131ksi)}\left(\frac{861860.5in^4}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}\right)\left(\frac{144in^2}{1ft^2}\right)$$
$$= 0.372in$$

$$\Delta_{ssy2} = -\frac{I_{xy}}{I_{xx}}\Delta_{ssx1} = -\left(\frac{17465.9in^4}{8618605in^4}\right)(0.372in) = -0.008in$$

$$\Delta_{ssx2} = -\frac{I_{xy}}{I_{yy}}\Delta_{ssy1} = -\left(\frac{17465.9in^4}{251152.4n^4}\right)(9.582in) = -0.666in$$

$$\Delta_{ssy} = 9.582in - 0.008in = 9.574in$$

$$\Delta_{ssx} = 0.372in - 0.666in = -0.295in$$

5.5.1.1.1.2.2 Harped Strands

$$P = \left(\frac{14}{36+14}\right)(1361.40kip) = 529.43kip$$

$$e' = e_{hp} - e_e - (Y_{be} - Y_{bh}) = 37.644in - (-16.567in) - (50.373in - 42.073in) = 62.511in$$

$$b = 0.4$$

$$N = \frac{Pe'}{bL} = \frac{(529.43kip)(62.511in)}{(0.4)(157.75ft)}\left(\frac{1ft}{12in}\right) = 43.71kip$$

$$\Delta_{hsy1} = \frac{b(3-4b^2)NL^3}{24E_{ci}}\left(\frac{I_{yy}}{I_{xx}I_{yy} - I_{xy}^2}\right) + \frac{P\left(e_e + \frac{2}{3}(Y_{bm} - Y_{be})\right)L^2}{8E_{ci}}\left(\frac{I_{yy}}{I_{xx}I_{yy} - I_{xy}^2}\right)$$
$$= \frac{0.4(3-4(0.4)^2)(43.71kip)(157.75ft)^3}{24(3048.131ksi)}\left(\frac{251152.4in^4}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}\right)\left(\frac{1728in^3}{1ft^3}\right)$$
$$+ \frac{(529.43kip)\left(-16.567in + \frac{2}{3}(41.643in - 50.373in)\right)(157.75ft)^2}{8(3048.131ksi)}\left(\frac{251152.4.in^4}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}\right)\left(\frac{144in^2}{1ft^2}\right)$$
$$= 4.446in - 2.024in = 2.422in$$

$$\Delta_{hsx1} = \frac{P\left(e_x + \frac{2}{3}(X_{lm} - X_{le})\right)}{8E_{ci}}\left(\frac{I_{xx}}{I_{xx}I_{yy} - I_{xy}^2}\right)$$
$$= \frac{(529.43kip)\left(0.370in + \frac{2}{3}(36.514in - 36.370in)\right)(157.75ft)^2}{8(3048.131ksi)}\left(\frac{861860.5in^4}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}\right)\left(\frac{144in^2}{1ft^2}\right)$$
$$= 0.145in$$

$$\Delta_{hsy2} = -\frac{I_{xy}}{I_{xx}}\Delta_{hsx1} = -\left(\frac{17465.9in^4}{861860.5in^4}\right)(0.145in) = -0.003in$$

$$\Delta_{hsx2} = -\frac{I_{xy}}{I_{yy}}\Delta_{hsy1} = -\left(\frac{17465.9in^4}{251152.4n^4}\right)(2.422in) = -0.168in$$

$$\Delta_{hsy} = 2.422in - 0.003in = 2.419in$$

$$\Delta_{hsx} = 0.145in - 0.168in = -0.023in$$

5.5.1.1.1.2.3 Temporary Strands

$$P = 83.62kip$$

$$\Delta_{tsy1} = \frac{P\left(e + \frac{2}{3}(Y_{bm} - Y_{be})\right)L^2}{8E_{ci}}\left(\frac{I_{yy}}{I_{xx}I_{yy} - I_{xy}^2}\right)$$
$$= \left[\frac{(83.62kip)\left(-16.627in + \frac{2}{3}(41.643in - 50.373in)\right)(157.75ft)^2}{8(3048.131ksi)}\right]\left(\frac{251152.4in^4}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}\right)\left(\frac{144in^2}{1ft^2}\right)$$
$$= -0.320in$$

$$\Delta_{tsx1} = \frac{P\left(e_x + \frac{2}{3}(X_{lm} - X_{le})\right)L^2}{8E_{ci}}\left(\frac{I_{xx}}{I_{xx}I_{yy} - I_{xy}^2}\right)$$
$$= \frac{(83.62kip)\left(0.370in + \frac{2}{3}(36.514in - 36.370in)\right)(157.75ft)^2}{8(3048.131ksi)}\left(\frac{861860.5in^4}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}\right)\left(\frac{144in^2}{1ft^2}\right)$$
$$= 0.023in$$

$$\Delta_{tsy2} = -\frac{I_{xy}}{I_{xx}}\Delta_{tsx1} = -\left(\frac{17465.9in^4}{861860.5in^4}\right)(0.023in) = -0.000in$$

$$\Delta_{tsx2} = -\frac{I_{xy}}{I_{yy}}\Delta_{tsy1} = -\left(\frac{17465.9in^4}{251152.4in^4}\right)(-0.320in) = 0.022in$$

$$\Delta_{tsy} = -0.320in - 0.000in = -0.320in$$

$$\Delta_{tsx} = 0.023in + 0.022in = 0.045in$$

5.5.1.1.1.2.4 Total

$$\Delta_{psy} = 9.574in + 2.419in - 0.320in = 11.673in$$

$$\Delta_{psx} = -0.295in - 0.023in + 0.045in = -0.273in$$

5.5.1.1.1.3 Initial Camber

$$\Delta_{camber} = \Delta_{gy} + \Delta_{psy} = -4.254in + 11.673in = 7.419in$$

$$\Delta_{camber} = \Delta_{gx} + \Delta_{psx} = 0.296in - 0.273in = 0.023in$$

5.5.1.1.2 Offset factor

The offset factor locates the center of mass of the girder with respect to the roll axis.

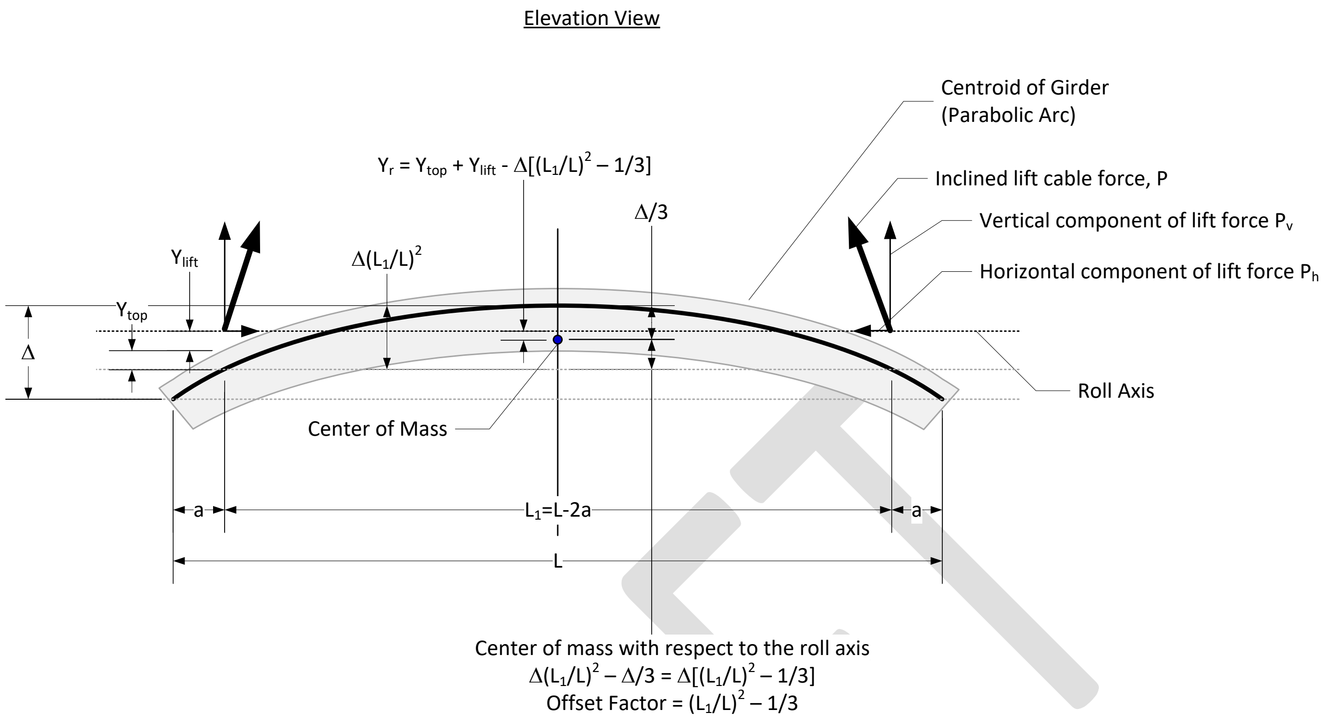


Figure 5-3: Offset Factor

$$F_o = \left(\frac{L_s}{L_g}\right)^2 - \frac{1}{3} = \left(\frac{137.75ft}{157.75ft}\right)^2 - \frac{1}{3} = 0.429$$

5.5.1.1.3 Location the roll axis above the top of girder

There are a variety of rigging schemes available to the contractor. Since the center of gravity is offset from the centerline of the web, a secondary lifting line may be utilized. This will raise the location of the roll axis to the point where the lifting lines are connected as shown in Figure 5-4

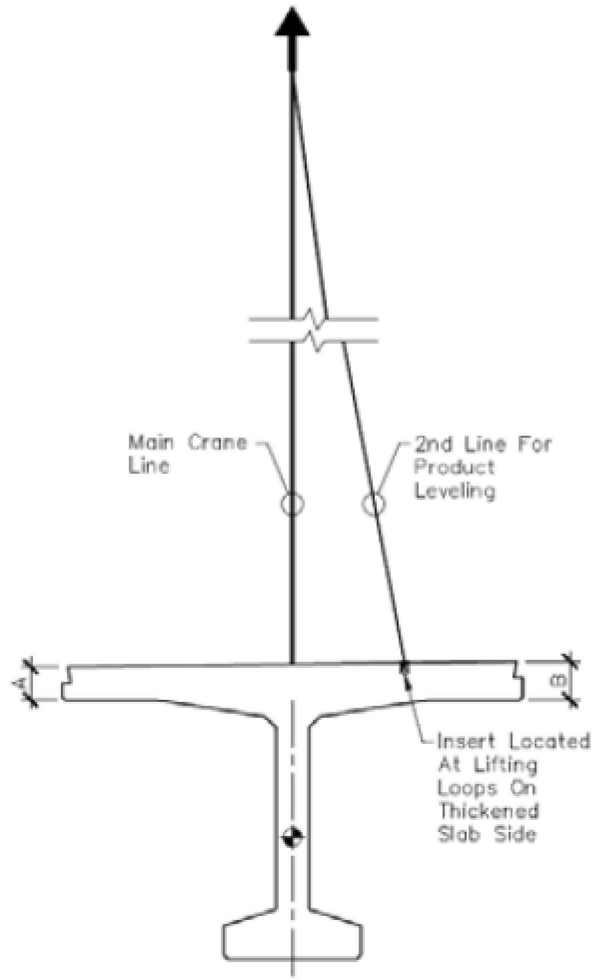


Figure 5-4: Lifting with secondary line

At design-time the actual lifting scheme is unknown so assume that the girder will be lifted by a single line.

$$y_{rc} = 0in$$

5.5.1.1.4 Location of CG below roll axis

$$Y_{top} = 25.847in + \frac{26.258in - 25.847in}{15.775ft}(10ft) = 26.108in$$

$$y_r = Y_{top} - F_o\Delta_{camber} + y_{rc} = 26.108in - (0.429)(7.419in) + 0in = 22.925in$$

5.5.1.2 *Lateral Deflection Parameters*

5.5.1.2.1 Lateral Sweep

Sweep tolerance is 1/8" per 10 ft

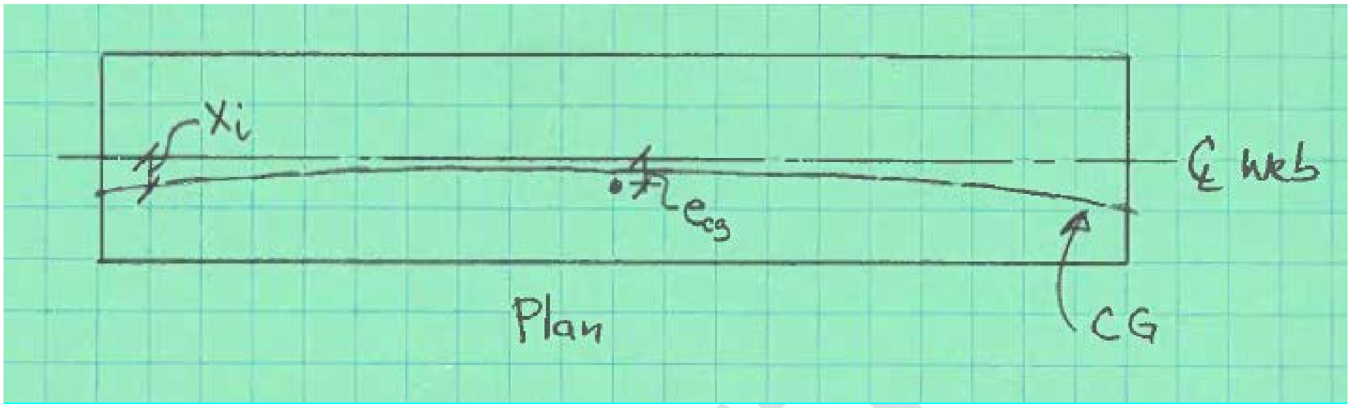
$$e_{sweep} = \frac{157.75ft}{10ft}\left(\frac{1}{8}in\right) = 1.972in$$

5.5.1.2.2 Initial Lateral Eccentricity

Initial lateral eccentricity of center of gravity of girder due to lateral sweep and eccentricity of lifting devices from CL girder

$$e_{lift} = 0.25in$$

Eccentricity of CG from roll axis



$$e_{cg} = \frac{\int w(z)X(z)dz}{\int w(z)dz = W_g} \cong \frac{\sum 0.5(wA_iX_i + wA_{i+1}X_{i+1})(dL)}{W_g} = 0.454in$$

$$e_i = F_o(e_{sweep} + \Delta_{lc}) + e_{lift} + e_{cg} = (0.429)(1.972in + 0.023in) + 0.25in + 0.454in = 1.560in$$

5.5.1.2.3 Lateral Deflection of CG

z_o is solved numerically because the weight and stiffness of the girder are non-uniform.

$$z_o = \frac{\int w(z)Z(z)dz}{\int w(z)dz = W_g} \cong \frac{\sum 0.5(wA_iZ_i + wA_{i+1}Z_{i+1})(dL)}{W_g} = 5.721in$$

5.5.1.3 *Equilibrium tilt angle*

$$\theta_{eq} = \frac{e_i}{y_r - z_o} = \frac{1.560in}{22.925in - 5.721in} = 0.0907\ rad$$

5.5.1.4 *Girder Stresses in Hanging Girder*

5.5.1.4.1 Direct stress at Pick Point

5.5.1.4.1.1 Prestressing

$$f_{ps}(x,y) = \frac{M_yI_{xx} + M_xI_{xy}}{I_{xx}I_{yy} - I_{xy}^2}x - \frac{M_xI_{yy} + M_yI_{xy}}{I_{xx}I_{yy} - I_{xy}^2}y + \frac{P}{A}$$

$$M_x = Pe_y, M_y = Pe_x$$

From PGSuper, the effective prestress force at the pick points are $P = 1382.34\ kip$ straight strands, $P = 537.58kip$ harped strands, and $P = 84.92kip$ temporary strands. The strand eccentricities are $e_{sy} = 45.235in$, $e_{hy} = -8.465in$, and $e_{ty} = -18.431in$ for the straight, harped, and temporary strand, respectively. The lateral strand eccentricity for all strand types is $e_x = 0.397in$

$$M_x = (-1382.34kip)(45.235in) + (-537.58kip)(-8.465in) + (-84.93kip)(-18.431in) = -56414.2k \cdot in$$

$$M_y = (-1382.34kip - 537.58kip - 84.92kip)(0.397in) = -795.9\ k \cdot in$$

$$A_g = 1679.371in^2 + \frac{1510.891in^2 - 1679.371in^2}{15.775ft}(10ft) = 1572.57in^2$$

$$I_{xx} = 1194879.7in^4 + \frac{1080736.3in^4 - 1194879.7in^4}{15.775ft}(10ft) = 1122522.7in^4$$

$$I_{yy} = 453417.5in^4 + \frac{380608.4in^4 - 453417.5in^4}{15.775ft}(10ft) = 407262.8in^4$$

$$I_{xy} = 16078.7in^4 + \frac{16334.8in^4 - 16078.7in^4}{15.775ft}(10ft) = 16241.0in^4$$

$$X_{left} = 36.370in + \frac{36.412in - 36.370in}{15.775ft}(10ft) = 36.397in$$

$$Y_{top} = 26.567in + \frac{26.978in - 26.567in}{15.775ft}(10ft) = 26.828in$$

$$H_g = 76.220in + \frac{73.880in - 76.220in}{15.775ft}(10ft) = 74.737in$$

Top Left $(x, y) = \left(-36.397in, 26.828in - 0.02(72in)\right) = (-36.397in, 25.388in)$

$$f_{ps} = \frac{(-795.9k \cdot in)(122522.7in^4) + (-56414.2k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-36.397in) - \frac{(-56414.2k \cdot in)(407262.8in^4) + (-795.9k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(25.388in) + \frac{(-1382.34kip - 537.58kip - 84.92kip)}{1572.57in^2} = 0.144ksi + 1.277ksi - 1.275ksi = 0.147ksi$$

Top Right $(x, y) = (72in - 36.397in, 26.828in) = (35.603in, 26.828in)$

$$f_{ps} = \frac{(-795.9k \cdot in)(122522.7in^4) + (-56414.2k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(35.603in) - \frac{(-56414.2k \cdot in)(407262.8in^4) + (-795.9k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(26.828in) + \frac{(-1382.34kip - 537.58kip - 84.92kip)}{1572.57in^2} = -0.141ksi + 1.350ksi - 1.275ksi = -0.066ksi$$

Bottom Left $(x, y) = \left(\frac{72in}{2} - 36.397in - \frac{38.375in}{2}, 26.828in - 74.737in - \frac{72in}{2}\left(0.02\frac{ft}{ft}\right)\right) = (-19.584in, -48.629in)$

$$f_{ps} = \frac{(-795.9k \cdot in)(122522.7in^4) + (-56414.2k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-19.584in) - \frac{(-56414.2k \cdot in)(407262.8in^4) + (-795.9k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-48.629in) + \frac{(-1382.34kip - 537.58kip - 84.92kip)}{1572.57in^2} = 0.078ksi - 2.447ksi - 1.275ksi = -3.644ksi$$

Bottom Right $(x, y) = \left(\frac{38.375in}{2} - \left(36.397in - \frac{72in}{2}\right), 26.828in - 74.737in - \frac{72in}{2}\left(0.02\frac{ft}{ft}\right)\right) = (18.791in, -48.629in)$

$$f_{ps} = \frac{(-795.9k \cdot in)(122522.7in^4) + (-56414.2k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(18.791in) - \frac{(-56414.2k \cdot in)(407262.8in^4) + (-795.9k \cdot in)(16237.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-48.629in) + \frac{(-1382.34kip - 537.58kip - 84.92kip)}{1572.57in^2} = -0.074ksi - 2.447ksi - 1.275ksi = -3.796ksi$$

5.5.1.4.1.2 Girder self-weight

$$M_x = M_g = -74.20k \cdot ft = -890.4k \cdot in$$

$$M_y = 0$$

Top Left $(x, y) = \left(-36.397in, 26.828in - 0.02(72in)\right) = (-36.397in, 25.388in)$

$$f_g = \frac{(-0k \cdot in)(122522.7in^4) + (-890.4k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-36.397in) - \frac{(-890.4k \cdot in)(407262.8in^4) + (-0k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(25.388in) = 0.001ksi + 0.020ksi = 0.021ksi$$

Top Right $(x, y) = (72in - 36.397in, 26.828in) = (35.603in, 26.828in)$

$$f_g = \frac{(-0k \cdot in)(122522.7in^4) + (-890.4k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(35.603in) - \frac{(-890.4k \cdot in)(407262.8in^4) + (-0k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(26.828in) = -0.001ksi + 0.021ksi = 0.020ksi$$

Bottom Left $(x, y) = \left(\frac{72in}{2} - 36.397in - \frac{38.375in}{2}, 26.828in - 74.737in - \frac{72in}{2}\left(0.02\frac{ft}{ft}\right)\right) = (-19.584in, -48.629in)$

$$f_g = \frac{(-0k \cdot in)(122522.7in^4) + (-890.4k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-19.584in) - \frac{(-890.4k \cdot in)(407262.8in^4) + (-0k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-48.629n) = 0.001ksi - 0.038ksi = -0.037ksi$$

Bottom Right $(x, y) = \left(\frac{38.375in}{2} - \left(36.397in - \frac{72in}{2}\right), 26.828in - 74.737in - \frac{72in}{2}\left(0.02\frac{ft}{ft}\right)\right) = (18.791in, -48.629in)$

$$f_g = \frac{(-0k \cdot in)(122522.7in^4) + (-890.4k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(18.791in) - \frac{(-890.4k \cdot in)(407262.8in^4) + (-0k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-48.629in) = -0.001ksi - 0.038ksi = -0.039ksi$$

5.5.1.4.2 Tilt induced stresses

$$M_x = 0;$$

$$M_y = -M_g\theta_{eq} = -(-890.4k \cdot in)(0.0907rad) = 80.76k \cdot in$$

Top Left $(x, y) = \left(-36.397in, 26.828in - 0.02(72in)\right) = (-36.397in, 25.388in)$

$$f_{tilt} = \frac{(80.76k \cdot in)(122522.7in^4) + (0k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-36.397in) - \frac{(0k \cdot in)(407262.8in^4) + (80.76k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(25.388in) = -0.007ksi + 0.000ksi = -0.007ksi$$

Top Right $(x, y) = (72in - 36.397in, 26.828in) = (35.603in, 26.828in)$

$$f_{tilt} = \frac{(80.76k \cdot in)(122522.7in^4) + (0k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(35.603in) - \frac{(0k \cdot in)(407262.8in^4) + (80.76k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(26.828in) = 0.007ksi + 0.000ksi = 0.007ksi$$

Bottom Left $(x, y) = \left(\frac{72in}{2} - 36.397in - \frac{38.375in}{2}, 26.828in - 74.737in - \frac{72in}{2}\left(0.02\frac{ft}{ft}\right)\right) = (-19.584in, -48.629in)$

$$f_{tilt} = \frac{(80.76k \cdot in)(122522.7in^4) + (0k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-19.584in)$$
$$- \frac{(0k \cdot in)(407262.8in^4) + (80.76k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-48.629in) = -0.004ksi + 0.000ksi$$
$$= -0.004ksi$$

Bottom Right $(x, y) = \left(\frac{38.375in}{2} - \left(36.397in - \frac{72in}{2}\right), 26.828in - 74.737in - \frac{72in}{2}\left(0.02\frac{ft}{ft}\right)\right) = (18.791in, -48.629in)$

$$f_{tilt} = \frac{(80.76k \cdot in)(122522.7in^4) + (0k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(18.791in)$$
$$- \frac{(0k \cdot in)(407262.8in^4) + (80.76k \cdot in)(16241.0in^4)}{(122522.7in^4)(407262.8in^4) - (16241.0in^4)^2}(-48.629in) = 0.004ksi + 0.000ksi$$
$$= 0.004ksi$$

5.5.1.4.3 Total stress

$$f_t = f_{ps} + f_g + f_{tilt}$$

Top left flange tip

$$f_{tl} = 0.147ksi + 0.021ksi - 0.007ksi = 0.161ksi$$

Top right flange tip

$$f_{tr} = -0.066ksi + 0.020ksi + 0.007ksi = -0.039ksi$$

Bottom left flange tip

$$f_{bl} = -3.644ksi - 0.037ksi - 0.004ksi = -3.685ksi$$

Bottom right flange tip

$$f_{br} = -3.796ksi - 0.039ksi + 0.004ksi = -3.831ksi$$

5.5.1.5 *Factor of Safety Against Cracking*

Lateral cracking moment

$$M_{cr} = \frac{(f_r - f_{direct})(I_{xx}I_{yy} - I_{xy}^2)}{I_{xx}x - I_{xy}y}$$

Tilt angle at first crack

$$\theta_{cr} = \frac{M_{cr}}{M_g} \leq 0.4\ rad$$

Cracking moment at Harp Point, top left (controlling point, see PGSuper)

$$f_r = 0.24\lambda\sqrt{f'_{ci}} = (0.24)(0.9375)\sqrt{6.0ksi} = 0.551ksi$$

$$f_{direct} = f_{ps} + f_g = 0.923ksi - 1.022ksi = -0.099ksi$$

$$M_{cr} = \frac{(0.551ksi - (-0.099ksi))(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2)}{(876531.1in^4)(-36.506in) - (17360.6in^4)(27.187in)}\left(\frac{1ft}{12in}\right) = 378.6\ k \cdot ft$$

Tilt angle at first crack at Transfer Point

$$\theta_{cr} = \frac{378.6k \cdot ft}{2516.93k \cdot ft} = 0.150\ rad$$

Factor of Safety against Cracking at Transfer Point

$$FS_{cr} = \frac{y_r\theta_{cr}}{e_i + z_o\theta_{cr}} = \frac{(22.925in)(0.150)}{1.560in + (5.721in)(0.150)} = 1.422$$

$$FS_{cr} > 1.0 \ \textbf{OK}$$

5.5.1.6 *Factor of Safety against Failure*

$$\theta_{max} = \sqrt{\frac{e_i}{2.5z_o}} \leq 0.4\,rad = \sqrt{\frac{1.560in}{2.5(5.721in)}} = 0.330\,rad$$

$$FS_f = \frac{y_r\theta_{max}}{e_i + (1 + 2.5\theta_{max})(z_o\theta_{max})} = \frac{(22.925in)(0.330)}{1.560in + \big(1 + 2.5(0.330)\big)(5.721in)(0.330)} = 1.511$$

If $FS_f < FS_{cr}, FS_f = FS_{cr}$

$$FS_f = 1.511$$

$$FS_f > 1.5 \ \textbf{OK}$$

5.5.2 Check Girder Stresses

5.5.2.1 *Compression stress*

Stress limit

$$-0.65f'_{ci} = -0.65(6.0ksi) = -3.900\ ksi$$

Bottom right at pick point

$$-3.831ksi < -3.900ksi \ \textbf{OK}$$

Bottom right at harp point

$$-3.868ksi < -3.900\ ksi \ \textbf{OK}$$

The stress at the prestress pick point and the harp point are approximately the same. The required concrete strength at these locations is also the same. The girder is optimized for fabrication. See Reference 2 for more information about designing for optimized fabrication.

5.5.2.2 *Tension stress*

$$0.0948\lambda\sqrt{f'_{ci}} \leq 0.200ksi = 0.0948(0.9375)\sqrt{6.0ksi} = 0.217ksi \ \therefore 0.200ksi$$

The maximum tension occurs at the top left corner at the harp point

$$0.293ksi > 0.200ksi \ \textbf{No Good}$$

Check if there is sufficient reinforcement to use the increased tension limit

$$0.19\lambda\sqrt{f'_{ci}} = 0.19(0.9375)\sqrt{6.0ksi} = 0.436ksi\ with\ sufficient\ reinforcement$$

The neutral axis (zero stress) line is determined using the stress at the corners of the girder section. The area of the girder section in tension is then determined as illustrated in Figure 5-5.

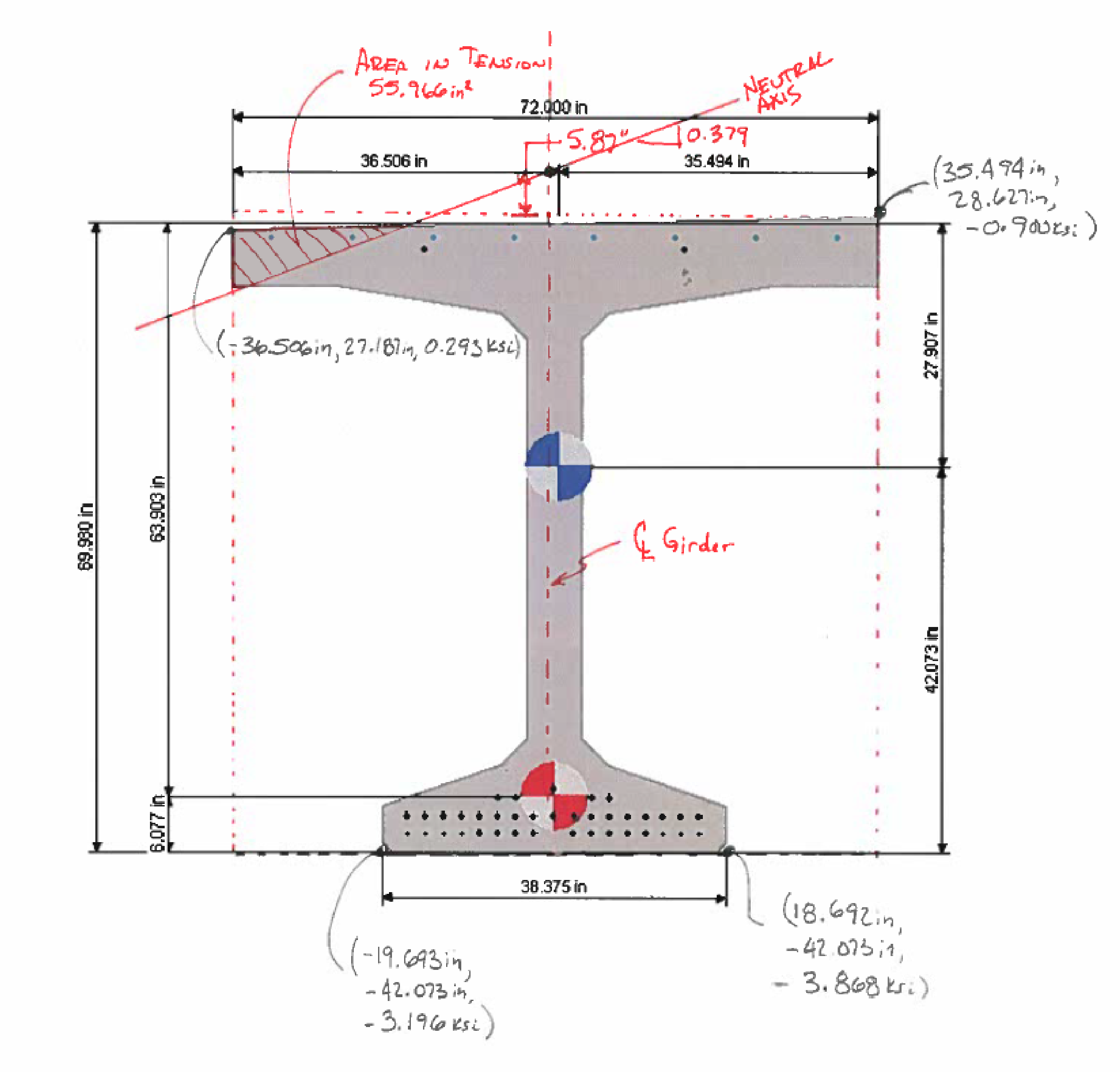


Figure 5-5: Location of neutral axis and area of girder in tension

The tensile force in the concrete is

$$T = \frac{f_t}{2}A_t = \frac{0.293ksi}{2}(55.966in^2) = 8.2kip$$

The required reinforcement is

$$A_{s\,required} = \frac{T}{f_s}\ where\ f_s = 0.5f_y \leq 30ksi = \frac{8.2kip}{30ksi} = 0.273in^2$$

As can be seen in Figure 5-5, two #6 bars are providing tensile resistance

$$A_{s\,provided} = 2(0.44in^2) = 0.88in^2$$

$$A_{s\,provided} > A_{s\,required} \text{ the higher tension limit can be used}$$

$$0.293ksi < 0.436\ ksi\ \textbf{OK}$$

5.6 Hauling

5.6.1 Check girder stability

Bunk points are H away from the ends of the girder ($9ft$) and hauling is assumed to occur with the HT60-72 haul truck configuration.

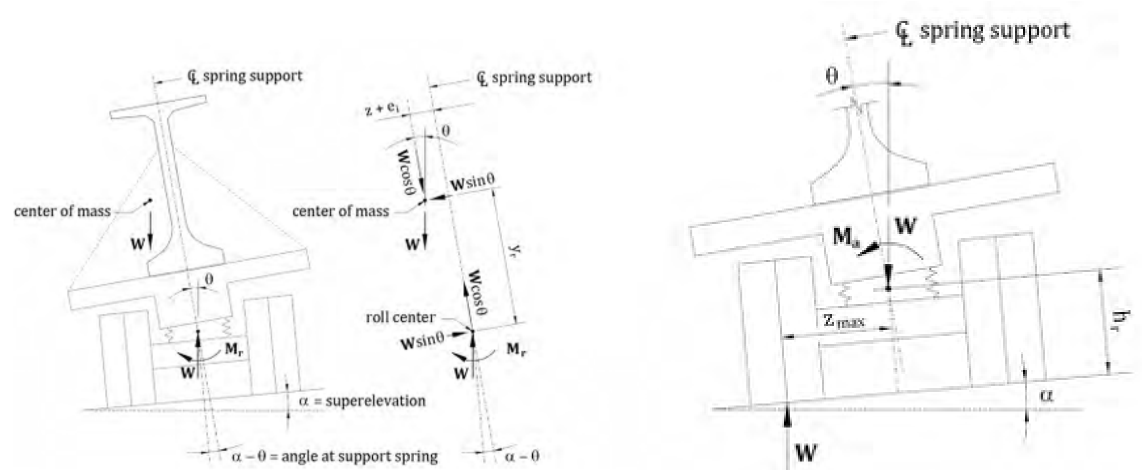


Figure 5-6: Equilibrium during Hauling

5.6.1.1 *Stability Analysis Parameters*

Parameter	**Value**
Rotational Stiffness	$K_\theta = 60000\dfrac{k\cdot in}{rad}$
Center-to-center wheel spacing	$W_{cc} = 72\ in$
Height of the roll center above the roadway surface	$H_{rc} = 24in$
Height of the bottom of the girder above roadway	$H_{bg} = 72\ in$
Bunk placement tolerance	$e_{bunk} = 1.0\ in$
Normal Crown Slope	$\alpha = 0.02\dfrac{ft}{ft}$
Maximum Superelevation	$\alpha = 0.06\dfrac{ft}{ft}$
Impact for Normal Crown Slope Case	$IM = \pm 20\%$
Impact for Superelevation Case	$IM = 0\%$
Modulus of Rupture	$f_r = 0.24\lambda\sqrt{f_c'} = (0.24)(0.9375)\sqrt{6.8ksi} = 0.586ksi$

5.6.1.2 *Vertical Location of Center of Gravity*

5.6.1.2.1 Camber at Hauling

Assume girder transportation occurs as late as possible to maximize camber grown while in storage. Assume transportation occurs at 90 days.

The camber at hauling is equal to the camber at the end of storage plus the change in dead load deflection due to the different support conditions between storage and hauling.

From before, the prestress deflection measured from the ends of the girder is

$$\Delta_{psy} = 11.673in$$

$$\Delta_{psx} = -0.273in$$

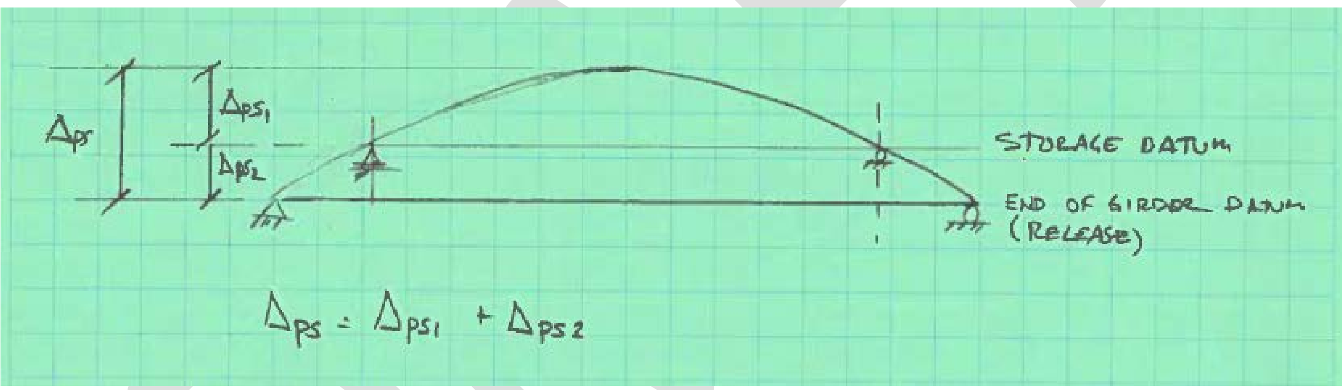
Changing the datum to the storage support location

$$\Delta_{ps1y} = 11.348in \text{ at mid-span}$$

$$\Delta_{ps2y} = -0.324in \text{ at girder end}$$

$$\Delta_{ps1x} = -0.267in \text{ at mid-span}$$

$$\Delta_{ps2x} = 0.006in \text{ at girder end}$$



5.6.1.2.1.1 Girder deflection in storage configuration

The self-weight deflections are computed using a finite element model.

At girder ends

$$\Delta_{gy1} = 0.146in$$

Mid-span

$$\Delta_{gy2} = -5.854in$$

The associated lateral deflection due to asymmetry is

At girder ends

$$\Delta_{gx1} = -\frac{I_{xy}}{I_{yy}}\Delta_{gy1} = -\frac{17465.9in^4}{251152.4in^4}(0.146in) = -0.010in$$

Mid-span

$$\Delta_{gx2} = -\frac{I_{xy}}{I_{yy}}\Delta_{gy2} = -\frac{17465.9in^4}{251152.4in^4}(-5.854in) = 0.407in$$

Creep deflection during storage is

$$\Delta_{creep} = \psi_b(t_h, t_i)(\Delta_{ps} + \Delta_g)$$

$$k_{td}(t = 89days) = \frac{89}{12\left(\frac{100 - 4(6.0)}{6.0 + 20}\right) + 89} = 0.717$$

$$\psi_b(t_h, t_i) = 1.9(1.0)(0.96)(0.714)(0.717)(1)^{-0.118} = 0.935$$

At mid-span

$$\Delta_{creepy} = (0.935)(11.348in - 5.854in) = 5.137in$$

$$\Delta_{creepx} = (0.935)(-0.267in + 0.407in) = 0.131in$$

At end of girder

$$\Delta_{creepy} = (0.935)(-0.324in + 0.146in) = -0.166in$$

$$\Delta_{creepx} = (0.935)(0.006in - 0.010\ in) = -0.004in$$

5.6.1.2.1.2 Girder deflection in the hauling configuration

The self-weight deflections are computed using a finite element model.

At girder ends

$$\Delta_{gy1} = 0.727in$$

Mid-span

$$\Delta_{gy2} = -3.543in$$

The associated lateral deflection due to asymmetry is

At girder ends

$$\Delta_{gx1} = -\frac{I_{xy}}{I_{yy}}\Delta_{gy1} = -\frac{17465.9in^4}{251152.4in^4}(0.727n) = -0.051in$$

Mid-span

$$\Delta_{gx} = -\frac{I_{xy}}{I_{yy}}\Delta_{gy} = -\frac{17465.9in^4}{251152.4in^4}(-3.543in) = 0.246in$$

We want the total camber measured between the girder ends and mid-span

$$\Delta_{cambery} = \left(\Delta_g + \Delta_{ps} + \Delta_{creep}\right)_{mid-span} - \left(\Delta_g + \Delta_{ps} + \Delta_{creep}\right)_{end}$$
$$= (-3.543in + 11.348in + 5.137in) - (0.727n - 0.324in - 0.166in) = 12.705in$$

$$\Delta_{camberx} = \left(\Delta_g + \Delta_{ps} + \Delta_{creep}\right)_{mid-span} - \left(\Delta_g + \Delta_{ps} + \Delta_{creep}\right)_{end}$$
$$= (0.246in - 0.267in + 0.131in) - (-0.051in + 0.006in - 0.004in) = 0.159in$$

5.6.1.2.2 Offset Factor

$$F_o = \left(\frac{L_s}{L_g}\right)^2 - \frac{1}{3} = \left(\frac{139.75ft}{157.75t}\right)^2 - \frac{1}{3} = 0.451$$

5.6.1.2.3 Location of roll axis below top of girder

$$y_{rc} = H_{bg} + H_g - H_{rc} = 72.0in + 76.94in - 24.0in = 124.94in$$

5.6.1.2.4 Location of center of gravity above roll axis

$$Y_{top} at\ roll\ axis = 26.567in + \frac{26.978in - 26.567in}{15.775ft} 9ft = 26.801in$$

$$y_r = y_{rc} - Y_{top} + F_o(\Delta_{camber}) = 124.94in - 26.801in + 0.451(12.705in) = 103.87in$$

5.6.1.3 *Lateral Deflection Parameters*

5.6.1.3.1 Lateral Sweep

Sweep tolerance = 1/8” per 10 ft

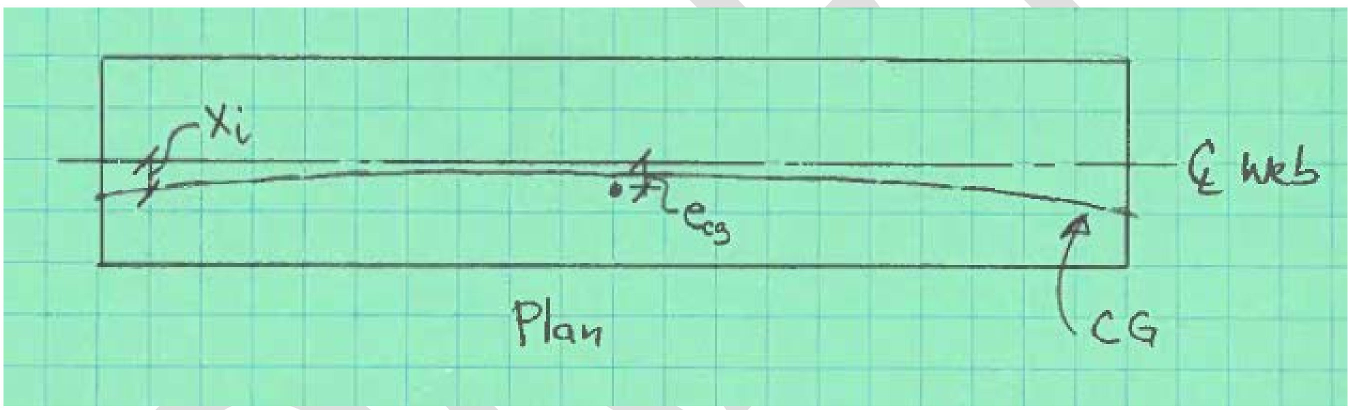
$$e_{sweep} = \left(\frac{157.75ft}{10ft}\right)\left(\frac{1}{8}in\right) = 1.972in$$

5.6.1.3.2 Initial Lateral Eccentricity

Initial lateral eccentricity of center of gravity of girder due to lateral sweep and eccentricity of bunking devices from CL girder

$$e_{bunk} = 1.000in$$

Eccentricity of CG from roll axis



$$e_{cg} = \frac{\int w(z)X(z)dz}{\int w(z)dz = W_g} \cong \frac{\sum 0.5(wA_iX_i + wA_{i+1}X_{i+1})(dL)}{W_g} = 0.454n$$

$$e_i = F_o(e_{sweep} + \Delta_{lc}) + e_{bunk} + e_{cg} = (0.451)(1.972in - 0.159in) + 1.000in + 0.454in = 2.272in$$

5.6.1.3.3 Lateral Deflection of CG

z_o is solved numerically because the weight and stiffness of the girder are non-uniform.

$$z_o = \frac{\int w(z)Z(z)dz}{\int w(z)dz = W_g} \cong \frac{\sum 0.5(wA_iZ_i + wA_{i+1}Z_{i+1})(dL)}{W_g} = 5.990in$$

5.6.1.3.4 Girder Stresses at Harping Point

5.6.1.3.4.1 Stress due to prestressing

$$f_{ps}(x,y) = \frac{M_yI_{xx} + M_xI_{xy}}{I_{xx}I_{yy} - I_{xy}^2}x - \frac{M_xI_{yy} + M_yI_{xy}}{I_{xx}I_{yy} - I_{xy}^2}y + \frac{P}{A}$$

$$M_x = Pe_y, M_y = Pe_x$$

From PGSuper, the effective prestress force at the harping points are $P = 1303.88\ kip$ straight strands, $P = 507.06kip$ harped strands, and $P = 81.39kip$ temporary strands. The strand eccentricities are $e_{ys} = 38.739in$, $e_{yh} = 37.644in$, and $e_{yt} = -24.927in$. The lateral strand eccentricity is $e_x = 0.506in$

$$M_x = (-1303.88kip)(38.739in) + (-507.06kip)(37.644in) + (-81.39kip)(-24.927in) = -67569.9k \cdot in$$

$$M_y = (-1303.88kip - 507.06kip - 81.39kip)(0.506in) = -957.52\ k \cdot in$$

Top Left $(x, y) = (-36.506in, 27.187in)$

$$f_{ps} = \frac{(-957.52k \cdot in)(876531.1in^4) + (-67569.9k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-36.506in) - \frac{(-67569.9k \cdot in)(259244.3in^4) + (-957.52k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(27.187in) + \frac{(-1303.88kip - 507.06kip - 81.39kip)}{1230.091in^2} = 0.324ksi + 2.101ksi - 1.538ksi = 0.886ksi$$

Top Right $(x, y) = (35.494in, 28.627in)$

$$f_{ps} = \frac{(-957.52k \cdot in)(876531.1in^4) + (-67569.9k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(35.494in) - \frac{(-67569.9k \cdot in)(259244.3in^4) + (-957.52k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(28.627in) + \frac{(-1303.88kip - 507.06kip - 81.39kip)}{1230.091in^2} = -0.315ksi + 2.212ksi - 1.538ksi = 0.359ksi$$

Bottom Left $(x, y) = (-19.693in, -42.073in)$

$$f_{ps} = \frac{(-957.52k \cdot in)(876531.1in^4) + (-67569.9k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-19.693in) - \frac{(-67569.9k \cdot in)(259244.3in^4) + (-957.52k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-42.073in) + \frac{(-1303.88kip - 507.06kip - 81.39kip)}{1230.091in^2} = 0.175ksi - 3.251ksi - 1.538ksi = -4.614ksi$$

Bottom Right $(x, y) = (18.682in, -42.073in)$

$$f_{ps} = \frac{(-957.52k \cdot in)(876531.1in^4) + (-67569.9k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(18.682in) - \frac{(-67569.9k \cdot in)(259244.3in^4) + (-957.52k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-42.073in) + \frac{(-1303.88kip - 507.06kip - 81.39kip)}{1230.091in^2} = -0.166ksi - 3.251ksi - 1.538ksi = -4.955ksi$$

5.6.1.3.4.2 Stress due to girder self-weight (without impact)

$$M_x = 2614.52k \cdot ft = 31374.24k \cdot in$$

$$M_y = 0\ k \cdot in$$

Top Left $(x, y) = (-36.506in, 27.187in)$

$$f_g = \frac{(0k \cdot in)(876531.1in^4) + (31374.24k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-36.506in) - \frac{(31374.24k \cdot in)(259244.3in^4) + (0k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(27.187in) = -0.088ksi - 0.974ksi = -1.062ksi$$

Top Right $(x, y) = (35.494in, 28.627in)$

$$f_g = \frac{(0k \cdot in)(876531.1in^4) + (31374.24k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(35.494in) - \frac{(31374.24k \cdot in)(259244.3in^4) + (0k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(28.627in) = 0.085ksi - 1.026ksi = -0.941ksi$$

Bottom Left $(x, y) = (-19.693in, -42.073in)$

$$f_g = \frac{(0k \cdot in)(876531.1in^4) + (31374.24k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-19.693in) - \frac{(31374.24k \cdot in)(259244.3in^4) + (0k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-42.073in) = -0.047si + 1.508ksi = 1.461ksi$$

Bottom Right $(x, y) = (18.682in, -42.073in)$

$$f_g = \frac{(0k \cdot in)(876531.1in^4) + (31374.24k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(18.682in) - \frac{(31374.24k \cdot in)(259244.3in^4) + (0k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-42.073in) = 0.045ksi + 1.508ksi = 1.553ksi$$

5.6.1.4 *Analyze normal crown slope, no impact case*

5.6.1.4.1 Equilibrium tilt angle

$$\theta_{eq} = \frac{(K_\theta \alpha + (IM)W_g e_i)}{K_\theta - (IM)W_g(y_r + (IM)z_o)} = \frac{\left(\left(60000\frac{k \cdot in}{rad}\right)\left(0.02\frac{ft}{ft}\right) + (1.0)(195.18kip)(2.415in)\right)}{\left(60000\frac{k \cdot in}{rad}\right) - (1.0)(195.18kip)(103.873in + (1.0)5.990in)} = 0.04335\ rad$$

5.6.1.4.2 Stress due to lateral loading from tilt

$$M_x = 0;$$

$$M_y = -M_g\theta_{eq} = -31374.24(0.04335rad) = -1360.07k \cdot in$$

Top Left $(x, y) = (-36.506in, 27.187in)$

$$f_{tilt} = \frac{(-1360.07k \cdot in)(876531.1in^4) + (0k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-36.506in) - \frac{(0k \cdot in)(259244.3in^4) + (-1360.07k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(27.187in) = 0.192ksi + 0.003ksi = 0.195ksi$$

Top Right $(x, y) = (35.494in, 28.627in)$

$$f_{tilt} = \frac{(-1360.07k \cdot in)(876531.1in^4) + (0k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(35.494in) - \frac{(0k \cdot in)(259244.3in^4) + (-1360.07k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(28.627in) = -0.186ksi + 0.003ksi = -0.183ksi$$

Bottom Left $(x, y) = (-19.693in, -42.073in)$

$$f_{tilt} = \frac{(-1360.07k \cdot in)(876531.1in^4) + (0k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-19.693in) - \frac{(0k \cdot in)(259244.3in^4) + (-1360.07k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-42.073in) = 0.103ksi - 0.004ksi = 0.099ksi$$

Bottom Right $(x, y) = (18.682in, -42.073in)$

$$f_{tilt} = \frac{(-1360.07k \cdot in)(876531.1in^4) + (0k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(18.682in) - \frac{(0k \cdot in)(259244.3in^4) + (-1360.07k \cdot in)(17360.6in^4)}{(876531.1in^4)(259244.3in^4) - (17360.6in^4)^2}(-42.073in) = -0.098ksi - 0.004ksi = -0.103ksi$$

5.6.1.4.3 Factor of Safety against Cracking

Lateral cracking moment

$$f_{direct} = f_{ps} + (IM)f_g = 0.886ksi + (1.0)(-1.062ksi) = -0.176ksi$$

$$M_{cr} = \frac{(f_r - f_{direct})(I_{xx}I_{yy} - I_{xy}^2)}{I_{xx}x - I_{xy}y}$$

$$M_{cr} = \frac{(0.587ksi - (-0.176ksi))((876531.1in^4)(259244.3in^4) - (17360.6in^4)^2)}{(876531.1in^4)(-36.506in) - (17360.6in^4)(27.187in)}\left(\frac{1ft}{12in}\right) = 444.4k \cdot ft$$

Tilt angle at first crack

$$\theta_{cr} = \frac{M_{cr}}{(IM)M_g} \leq 0.4$$

$$\theta_{cr} = \frac{444.4\ k \cdot ft}{(1.0)(2614.52k \cdot ft)} = 0.1699\ rad$$

Factor of Safety against Cracking

$$FS_{cr} = \frac{K_\theta(\theta_{cr} - \alpha)}{(IM)W_g[((IM)z_o + y_r)\theta_{cr} + e_i]}$$

$$FS_{cr} = \frac{\left(\left(60000\frac{k \cdot in}{rad}\right)\left(0.1699\ rad - 0.02\frac{ft}{ft}\right)\right)}{(1.0)(195.18kip)\left[\left((1.0)5.990in + 103.873in\right)(0.1699rad) + 2.415in\right]} = 2.186$$

$FS_{cr} > 1.0$ **OK**

5.6.1.4.4 Factor of Safety against Failure

$$\theta'_{max} = \sqrt{\alpha^2 + \frac{e_i + ((IM)z_o + y_r)\alpha}{2.5(IM)z_o}} + \alpha \leq 0.4\ rad$$

$$\theta'_{max} = \sqrt{0.02^2 + \frac{2.415in + ((1.0)(5.990in) + 103.873in)0.02}{2.5(1.0)(5.990in)}} + 0.02 = 0.575\ rad\ \therefore 0.4\ rad$$

$$FS_f = \frac{K_\theta(\theta'_{max} - \alpha)}{(IM)W_g[((IM)z_o\theta'_{max})(1 + 2.5\theta'_{max}) + y_r\theta'_{max} + e_i]}$$

$$FS_f = \frac{60000\,\frac{k \cdot in}{rad}(0.4 - 0.02)}{(1.0)(195.18kip)[((1.0)(5.990in)(0.4)(1 + 2.5(0.40)) + (103.873in)(0.4) + 2.415in]} = 2.396$$

$$FS_f > 1.5 \textbf{ OK}$$

5.6.1.4.5 Factor of Safety against Rollover

$$\theta_{ro} = \frac{(IM)W_g\left(\frac{W_{cc}}{2} - H_{rc}\alpha\right)}{K_\theta} + \alpha$$

$$\theta_{ro} = \frac{(1.0)(195.18kip)\left(\frac{72in}{2} - (24in)(0.02)\right)}{\left(60000\,\frac{k \cdot in}{rad}\right)} + 0.02 = 0.1355\,rad$$

$$FS_r = \frac{K_\theta(\theta_{ro} - \alpha)}{(IM)W_g[((IM)z_o(1 + 2.5\theta_{ro}) + y_r)\theta_{ro} + e_i]}$$

$$FS_r = \frac{\left(60000\,\frac{k \cdot in}{rad}\right)(0.1355 - 0.02)}{(1.0)(195.18kip)\left[\left((1.0)(5.990in)\left(1 + 2.5(0.1355)\right) + 103.873in\right)(0.1355) + 2.415in\right]} = 2.020$$

$$FS_r > 1.5 \textbf{ OK}$$

5.6.1.5 *Analyze normal crown slope, impact up*

5.6.1.5.1 Equilibrium tilt angle

$$\theta_{eq} = \frac{\left(K_\theta\alpha + (IM)W_g e_i\right)}{K_\theta - (IM)W_g(y_r + (IM)z_o)} = \frac{\left(\left(60000\frac{k \cdot in}{rad}\right)\left(0.02\frac{ft}{ft}\right) + (0.8)(195.18kip)(2.415in)\right)}{\left(60000\frac{k \cdot in}{rad}\right) - (0.8)(195.18kip)(103.873in + (0.8)5.990in)} = 0.03665\,rad$$

5.6.1.5.2 Factor of Safety against Cracking

Lateral cracking moment

$$f_{direct} = f_{ps} + (IM)f_g = 0.886ksi + (0.8)(-1.062ksi) = 0.036ksi$$

$$M_{cr} = \frac{(f_r - f_{direct})\left(I_{xx}I_{yy} - I_{xy}^2\right)}{I_{xx}x - I_{xy}y}$$

$$M_{cr} = \frac{\left(0.587ksi - (0.036ksi)\right)\left((876531.1in^4)(259244.3in^4) - (17360.6in^4)^2\right)}{(876531.1in^4)(-36.506in) - (17360.6in^4)(27.187in)}\left(\frac{1ft}{12in}\right) = 320.9k \cdot ft$$

Tilt angle at first crack

$$\theta_{cr} = \frac{M_{cr}}{(IM)M_g} \leq 0.4$$

$$\theta_{cr} = \frac{320.9\,k \cdot ft}{(0.8)(2614.52k \cdot ft)} = 0.1534\,rad$$

Factor of Safety against Cracking

$$FS_{cr} = \frac{K_\theta(\theta_{cr} - \alpha)}{(IM)W_g[((IM)z_o + y_r)\theta_{cr} + e_i]}$$

$$FS_{cr} = \frac{\left(\left(60000\,\frac{k\cdot in}{rad}\right)\left(0.1534\,rad - 0.02\frac{ft}{ft}\right)\right)}{(0.8)(195.18kip)\left[\left((0.8)5.990in + 103.873in\right)(0.1534rad) + 2.415in\right]} = 2.686$$

$$FS_{cr} > 1.0\ \textbf{OK}$$

5.6.1.5.3 Factor of Safety against Failure

$$\theta'_{max} = \sqrt{\alpha^2 + \frac{e_i + ((IM)z_o + y_r)\alpha}{2.5(IM)z_o}} + \alpha \le 0.4\,rad$$

$$\theta'_{max} = \sqrt{0.02^2 + \frac{2.415in + ((0.8)(5.990in) + 103.873in)0.02}{2.5(0.8)(5.990in)}} + 0.02 = 0.639\,rad\ \therefore 0.4\,rad$$

$$FS_f = \frac{K_\theta(\theta'_{max} - \alpha)}{(IM)W_g[((IM)z_o\theta'_{max})(1 + 2.5\theta'_{max}) + y_r\theta'_{max} + e_i]}$$

$$FS_f = \frac{60000\,\frac{k\cdot in}{rad}(0.4 - 0.02)}{(0.8)(195.18kip)[((0.8)(5.990in)(0.4)(1 + 2.5(0.40)) + (103.873in)(0.4) + 2.415in]} = 3.055$$

$$FS_f > 1.5\ \textbf{OK}$$

5.6.1.5.4 Factor of Safety against Rollover

$$\theta_{ro} = \frac{(IM)W_g\left(\frac{W_{cc}}{2} - H_{rc}\alpha\right)}{K_\theta} + \alpha$$

$$\theta_{ro} = \frac{(0.8)(195.18kip)\left(\frac{72in}{2} - (24in)(0.02)\right)}{\left(60000\,\frac{k\cdot in}{rad}\right)} + 0.02 = 0.1124\,rad$$

$$FS_r = \frac{K_\theta(\theta_{ro} - \alpha)}{(IM)W_g[((IM)z_o(1 + 2.5\theta_{ro}) + y_r)\theta_{ro} + e_i]}$$

$$FS_r = \frac{\left(60000\,\frac{k\cdot in}{rad}\right)(0.1124 - 0.02)}{(0.8)(195.18kip)\left[\left((0.8)(5.990in)\left(1 + 2.5(0.1124)\right) + 103.873in\right)(0.1124) + 2.415in\right]} = 2.402$$

$$FS_r > 1.5\ \textbf{OK}$$

5.6.1.6 *Analyze normal crown slope, impact down*

5.6.1.6.1 Equilibrium tilt angle

$$\theta_{eq} = \frac{\left(K_\theta\alpha + (IM)W_g e_i\right)}{K_\theta - (IM)W_g(y_r + (IM)z_o)} = \frac{\left(\left(60000\frac{k\cdot in}{rad}\right)\left(0.02\frac{ft}{ft}\right) + (1.2)(195.18kip)(2.415in)\right)}{\left(60000\frac{k\cdot in}{rad}\right) - (1.2)(195.18kip)(103.873in + (1.2)5.990in)} = 0.05195\,rad$$

5.6.1.6.2 Factor of Safety against Cracking

Lateral cracking moment

$$f_{direct} = f_{ps} + (IM)f_g = 0.886ksi + (1.2)(-1.062ksi) = -0.388ksi$$

$$M_{cr} = \frac{(f_r - f_{direct})(I_{xx}I_{yy} - I_{xy}^2)}{I_{xx}x - I_{xy}y}$$

$$M_{cr} = \frac{(0.587ksi - (-0.388ksi))((876531.1in^4)(259244.3in^4) - (17360.6in^4)^2)}{(876531.1in^4)(-36.506in) - (17360.6in^4)(27.187in)}\left(\frac{1ft}{12in}\right) = 567.85k \cdot ft$$

Tilt angle at first crack

$$\theta_{cr} = \frac{M_{cr}}{(IM)M_g} \leq 0.4$$

$$\theta_{cr} = \frac{567.85\ k \cdot ft}{(1.2)(2614.52k \cdot ft)} = 0.18099\ rad$$

Factor of Safety against Cracking

$$FS_{cr} = \frac{K_\theta(\theta_{cr} - \alpha)}{(IM)W_g[((IM)z_o + y_r)\theta_{cr} + e_i]}$$

$$FS_{cr} = \frac{\left(\left(60000\,\frac{k\cdot in}{rad}\right)\left(0.18099\ rad - 0.02\,\frac{ft}{ft}\right)\right)}{(1.2)(195.18kip)\left[\left((1.2)5.990in + 103.873in\right)(0.18099rad) + 2.415in\right]} = 1.832$$

$$FS_{cr} > 1.0 \textbf{ OK}$$

5.6.1.6.3 Factor of Safety against Failure

$$\theta'_{max} = \sqrt{\alpha^2 + \frac{e_i + ((IM)z_o + y_r)\alpha}{2.5(IM)z_o}} + \alpha \leq 0.4\ rad$$

$$\theta'_{max} = \sqrt{0.02^2 + \frac{2.415in + ((1.2)(5.990in) + 103.873in)0.02}{2.5(1.2)(5.990in)}} + 0.02 = 0.528\ rad\ \therefore 0.4\ rad$$

$$FS_f = \frac{K_\theta(\theta'_{max} - \alpha)}{(IM)W_g[((IM)z_o\theta'_{max})(1 + 2.5\theta'_{max}) + y_r\theta'_{max} + e_i]}$$

$$FS_f = \frac{60000\,\frac{k\cdot in}{rad}(0.4 - 0.02)}{(1.2)(195.18kip)[((1.2)(5.990in)(0.4)(1 + 2.5(0.40)) + (103.873in)(0.4) + 2.415in]} = 1.958$$

$$FS_f > 1.5 \textbf{ OK}$$

5.6.1.6.4 Factor of Safety against Rollover

$$\theta_{ro} = \frac{(IM)W_g\left(\frac{W_{cc}}{2} - H_{rc}\alpha\right)}{K_\theta} + \alpha$$

$$\theta_{ro} = \frac{(1.2)(195.18kip)\left(\frac{72in}{2} - (24in)(0.02)\right)}{\left(60000\,\frac{k\cdot in}{rad}\right)} + 0.02 = 0.1586\ rad$$

$$FS_r = \frac{K_\theta(\theta_{ro} - \alpha)}{(IM)W_g[((IM)z_o(1 + 2.5\theta_{ro}) + y_r)\theta_{ro} + e_i]}$$

$$FS_r = \frac{\left(60000\,\frac{k\cdot in}{rad}\right)(0.1586 - 0.02)}{(1.2)(195.18kip)\left[\left((1.2)(5.990in)\left(1 + 2.5(0.1586)\right) + 103.873in\right)(0.1586) + 2.415in\right]} = 1.734$$

$$FS_r > 1.5 \textbf{ OK}$$

5.6.1.7 *Analyze at maximum superelevation, no impact*

5.6.1.7.1 Equilibrium tilt angle

$$\theta_{eq} = \frac{\left(K_\theta \alpha + (IM)W_g e_i\right)}{K_\theta - (IM)W_g\left(y_r + (IM)z_o\right)} = \frac{\left(\left(60000\frac{k\cdot in}{rad}\right)\left(0.06\frac{ft}{ft}\right) + (1.0)(195.18kip)(2.415in)\right)}{\left(60000\frac{k\cdot in}{rad}\right) - (1.0)(195.18kip)(103.873in + (1.0)5.990in)} = 0.10559\ rad$$

5.6.1.7.2 Factor of Safety against Cracking

Lateral cracking moment

$$f_{direct} = f_{ps} + (IM)f_g = 0.886ksi + (1.0)(-1.062ksi) = -0.176ksi$$

$$M_{cr} = \frac{\left(f_r - f_{direct}\right)\left(I_{xx}I_{yy} - I_{xy}^2\right)}{I_{xx}x - I_{xy}y}$$

$$M_{cr} = \frac{\left(0.587ksi - (-0.176ksi)\right)\left((876531.1in^4)(259244.3in^4) - (17360.6in^4)^2\right)}{(876531.1in^4)(-36.506in) - (17360.6in^4)(27.187in)}\left(\frac{1ft}{12in}\right) = 444.4k \cdot ft$$

Tilt angle at first crack

$$\theta_{cr} = \frac{M_{cr}}{(IM)M_g} \le 0.4$$

$$\theta_{cr} = \frac{444.4\ k \cdot ft}{(1.0)(2614.52k \cdot ft)} = 0.1699\ rad$$

Factor of Safety against Cracking

$$FS_{cr} = \frac{K_\theta(\theta_{cr} - \alpha)}{(IM)W_g\left[\left((IM)z_o + y_r\right)\theta_{cr} + e_i\right]}$$

$$FS_{cr} = \frac{\left(\left(60000\,\frac{k\cdot in}{rad}\right)\left(0.1699\ rad - 0.06\frac{ft}{ft}\right)\right)}{(1.0)(195.18kip)\left[\left((1.0)5.990in + 103.873in\right)(0.1699rad) + 2.415in\right]} = 1.603$$

$$FS_{cr} > 1.0 \textbf{ OK}$$

5.6.1.7.3 Factor of Safety against Failure

$$\theta'_{max} = \sqrt{\alpha^2 + \frac{e_i + \left((IM)z_o + y_r\right)\alpha}{2.5(IM)z_o}} + \alpha \le 0.4\ rad$$

$$\theta'_{max} = \sqrt{0.06^2 + \frac{2.415in + \left((1.0)(5.990in) + 103.873in\right)0.06}{2.5(1.0)(5.990in)}} + 0.06 = 0.838\ rad\ \therefore 0.4\ rad$$

$$FS_f = \frac{K_\theta(\theta'_{max} - \alpha)}{(IM)W_g\left[\left((IM)z_o\theta'_{max}\right)(1 + 2.5\theta'_{max}) + y_r\theta'_{max} + e_i\right]}$$

$$FS_f = \frac{60000\,\frac{k\cdot in}{rad}(0.4 - 0.06)}{(1.0)(195.18kip)\left[\left((1.0)(5.990in)(0.4)(1 + 2.5(0.40)\right) + (103.873in)(0.4) + 2.415in\right]} = 2.144$$

$$FS_f > 1.5 \textbf{ OK}$$

5.6.1.7.4 Factor of Safety against Rollover

$$\theta_{ro} = \frac{(IM)W_g\left(\frac{W_{cc}}{2} - H_{rc}\alpha\right)}{K_\theta} + \alpha$$

$$\theta_{ro} = \frac{(1.0)(195.18kip)\left(\frac{72in}{2} - (24in)(0.02)\right)}{\left(60000\,\frac{k\cdot in}{rad}\right)} + 0.06 = 0.1724\,rad$$

$$FS_r = \frac{K_\theta(\theta_{ro} - \alpha)}{(IM)W_g[((IM)z_o(1 + 2.5\theta_{ro}) + y_r)\theta_{ro} + e_i]}$$

$$FS_r = \frac{\left(60000\,\frac{k\cdot in}{rad}\right)(0.1724 - 0.06)}{(1.0)(195.18kip)\left[\left((1.0)(5.990in)\left(1 + 2.5(0.1724)\right) + 103.873in\right)(0.1724) + 2.415in\right]} = 1.585$$

$$FS_r > 1.5\ \textbf{OK}$$

5.6.2 Check Girder Stresses

5.6.2.1 *Compression stress*

Stress limit

$$-0.65f_c' = -0.65(6.8ksi) = -4.420ksi$$

Maximum compression stress occurs at bottom right corner of girder on normal crown slope with impact up at the harp point

$$f_b = f_{direct} + f_{tilt}$$

$$f_b = -3.712si + (-0.069ksi) = -3.782\ ksi$$

$$-4.420\ ksi < -3.782ksi\ \textbf{OK}$$

5.6.2.2 *Tension stress*

Stress limit

$$0.0948\lambda\sqrt{f_c'} = 0.0948(0.9375)\sqrt{6.8ksi} = 0.232ksi$$

Maximum tension stress occurs at top left corner of girder on max superelevation slope at the harp point

$$f_t = -0.176ksi + 0.474ksi = 0.298\ ksi$$

$$0.298\ ksi > 0.232\ ksi\ \boldsymbol{No\ Good}$$

Check if there is sufficient reinforcement to use the increased tension limit

$$0.24\lambda\sqrt{f_c'} = 0.24(0.9375)\sqrt{6.8ksi} = 0.587ksi\ with\ sufficient\ reinforcement$$

The neutral axis (zero stress) line is determined using the stress at the corners of the girder section. The area of the girder section in tension is then determined as illustrated in Figure 5-7.

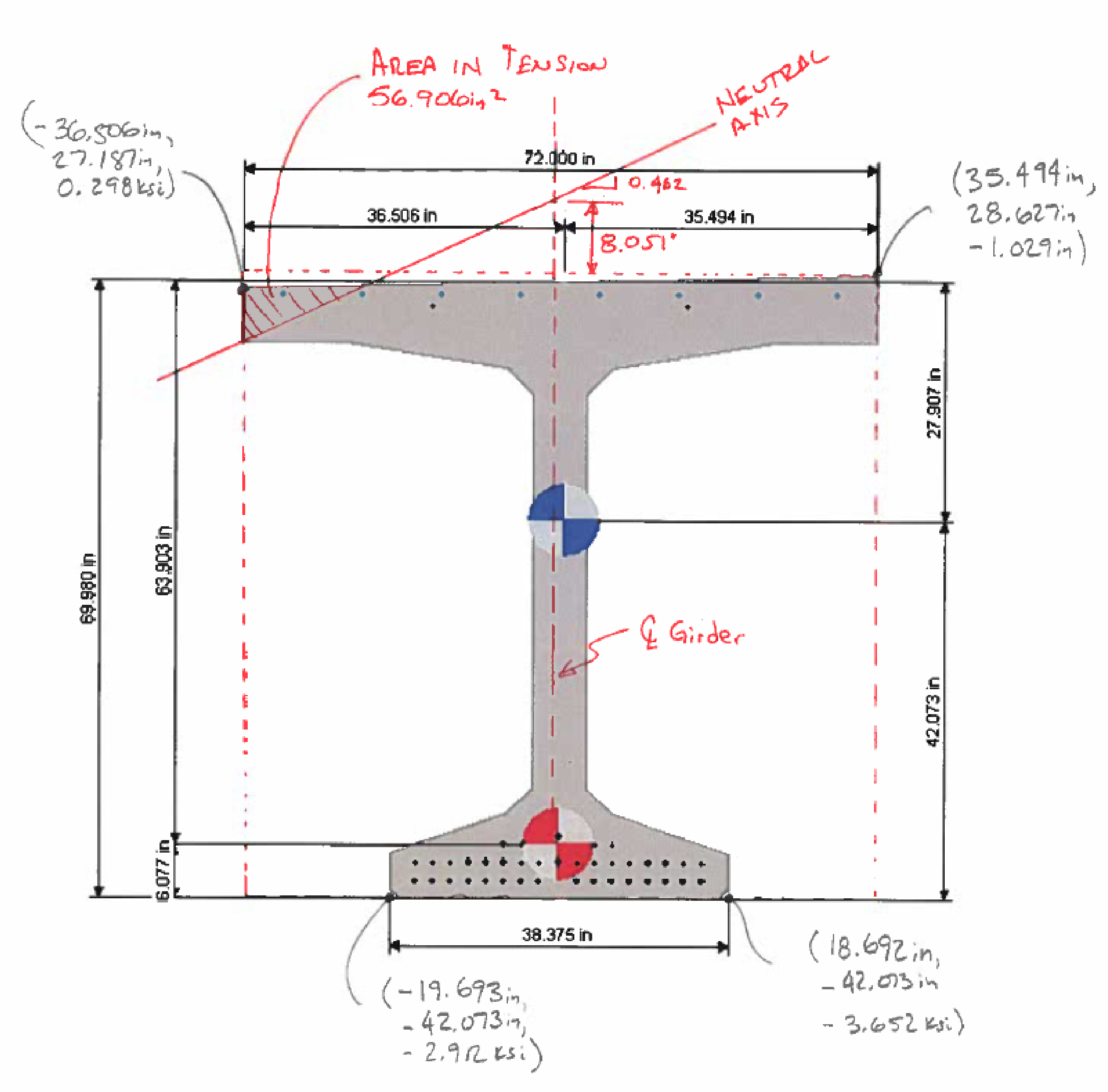


Figure 5-7: Location of neutral axis and area of girder in tension

The tensile force in the concrete is

$$T = \frac{f_t}{2} A_t = \frac{0.298ksi}{2}(56.906in^2) = 8.47kip$$

The required reinforcement is

$$A_{s\,required} = \frac{T}{f_s} \; where \; f_s = 0.5f_y \leq 30ksi = \frac{8.47kip}{30ksi} = 0.282in^2$$

As can be seen in Figure 5-7, one #6 bars are providing tensile resistance

$$A_{s\,provided} = 0.44in^2$$

$A_{s\,provided} > A_{s\,required}$ the higher tension limit can be used

$0.298\ ksi < 0.587\ ksi$ **OK**

6 Flexural Capacity

6.1.1.1 *Compute Nominal Moment Capacity at 0.5Lg.*

Strength I limit state

$$Strength\ I = 1.25DC + 1.5DW + 1.75(LL + IM)$$

$$M_u = 1.25(3512.77 + 86.69 + 243.15 + 356.27 + 1403.73 + 693.69) + 1.50(668.56) + 1.75(2942.95) = 14013.90k \cdot ft$$

$$c = \frac{A_{ps} f_{pu}}{\alpha_1 f_c' \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$$

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 2\left(1.04 - \frac{243}{270}\right) = 0.28$$

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$

$$\alpha_1 = 0.85$$

$$\beta_1 = 0.85 - 0.05(f_c' - 4ksi) \le 0.65 = 0.85 - 0.05(6.8 - 4) = 0.71$$

$$d_p = Y_t + e + t_s = 28.797in + 38.003in + 0in = 66.800in$$

$$c = \frac{(10.850in^2)(270ksi)}{0.85(6.8ksi)(0.71)(81in) + (0.28)(10.850in^2)\left(\frac{270ksi}{66.80in}\right)} = \frac{2929.5kip}{332.41\frac{k}{in} + 12.28\frac{k}{in}} = 8.499in$$

$$a = \beta_1 c = 0.71(8.499in) = 6.034in$$

$$f_{ps} = 270ksi\left(1 - 0.28\frac{8.499in}{66.8in}\right) = 260.44ksi$$

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$

$$M_n = (10.850in^2)(260.44ksi)\left(66.8in - \frac{6.034in}{2}\right) = 180236k \cdot in = 15019k \cdot ft$$

$$d_t = 70.44in - \left(\frac{72in}{2}\right)(0.02) - 2in = 67.72in$$

$$\varepsilon_t = 0.003\left(\frac{d_t}{c} - 1\right) = 0.003\left(\frac{67.72in}{8.449in} - 1\right) = 0.021$$

$$0.75 \le \phi = 0.75 + \frac{0.25(\varepsilon_t - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}} \le 1.0 = 0.75 + \frac{0.25(0.021 - 0.005)}{0.005 - 0.002} = 2.08 \therefore \phi = 1.0$$

$$M_r = \phi M_n = 1.0(15019 \cdot ft) = 15019k \cdot ft$$

$M_r > M_u$ **OK**

The AASHTO method for computing moment capacity does not account for the large compression flange in the girder or the higher strength of the girder concrete. See Reference 7 for more information. PGSuper uses strain compatibility analysis to compute the moment capacity.

Stress-strain relationship for prestressing strands:

$$f_{ps} = \varepsilon_{ps}\left[877 + \frac{27{,}613}{\left(1 + \left(112.4\varepsilon_{ps}\right)^{7.36}\right)^{\frac{1}{7.36}}}\right] \leq 270ksi$$

Stress-strain relationship for concrete:

$$f_c = f_c' \frac{n\left(\frac{\varepsilon_{cf}}{\varepsilon_c'}\right)}{n - 1 + \left(\frac{\varepsilon_{cf}}{\varepsilon_c'}\right)^{nk}}$$

where

$$n = 0.8 + \frac{f_c'}{2500}$$

$$k = 0.67 + \frac{f_c'}{9000}$$

$$if\ \frac{\varepsilon_{cf}}{\varepsilon_c'} < 1.0, k = 1.0$$

$$E_c = \frac{40{,}000\sqrt{f_c'} + 1{,}000{,}000}{1000}$$

$$\varepsilon_c' x1000 = \frac{f_c'}{E_c}\frac{n}{n-1}$$

Effective prestress, $f_{pe} = f_{pj} - \Delta f_{pT} = 202.5ksi - 32.868si = 169.632ksi$

Initial strain in prestressing strand, $\varepsilon_{psi} = \frac{f_{pe}}{E_p} = \frac{170.396ksi}{28500ksi} = 5.95210^{-3}$

Discretize the composite girder section into “slices”. Compute the strain at the centroid of each slice. The stress in the slice is determined from the stress-strain relationship for the slice material. Finally, compute the axial force and moment contribution for each slice. Sum the contribution of each slice to determine the capacity of the section.

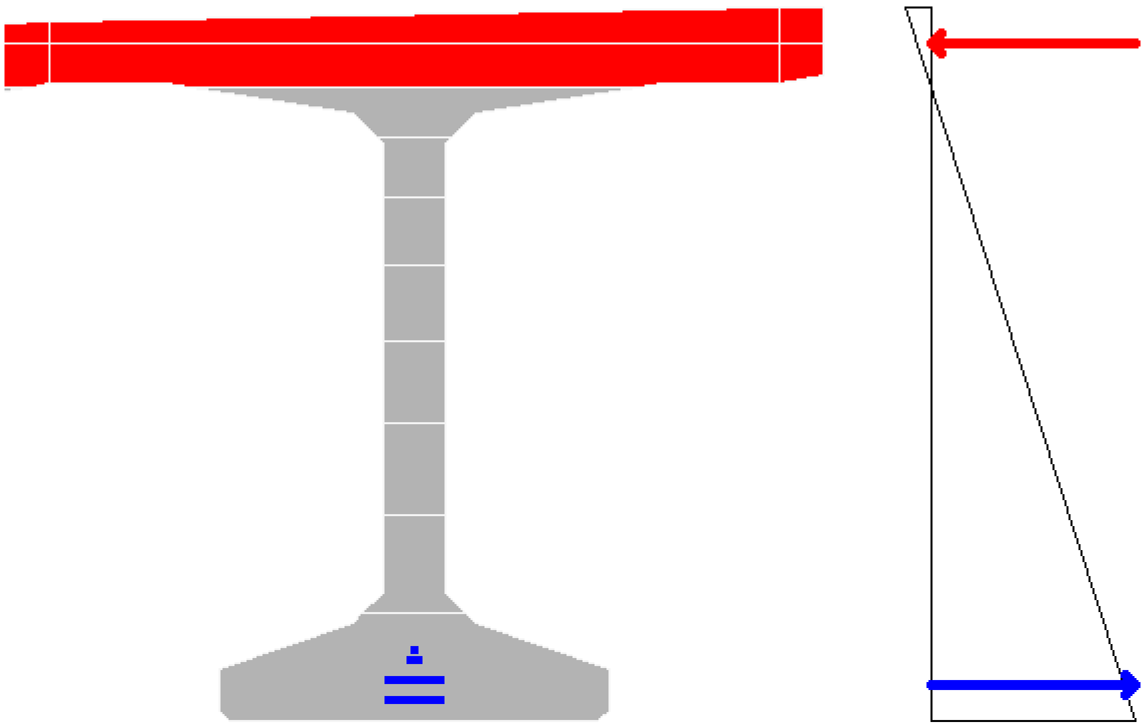


Figure 6-1: Discretized Girder Section for Strain Compatibility Analysis

Slice	Area (in^2)	Y_{cg} (in)	Strain	Stress (KSI)	δF = (Area)(Stress) (kip)	δM = (δF)(Y_{cg}) (kip-ft)
1	15.804	27.052	-0.00232	-12.742	-201.36	-453.94
2	200.209	26.717	-0.0022	-6.8	-1361.35	-3030.97
3	8.919	26.287	-0.00203	-11.423	-101.88	-223.17
4	16.058	23.505	-0.00097	-5.535	-88.88	-174.09
5	300.774	23.204	-0.00085	-3.605	-1084.21	-2096.49
6	19.033	23.18	-0.00084	-4.821	-91.75	-177.23
7	0.017	20.987	-1.28E-06	-0.007	0	0
8	0.653	20.815	6.44E-05	0	0	0
9	0.298	20.987	-1.28E-06	-0.006	0	0
10	97.992	19.269	0.000656	0	0	0
11	36.262	12.988	0.003061	0	0	0
12	40.761	6.702	0.005468	0	0	0
13	45.556	-0.344	0.008165	0	0	0
14	50.351	-8.174	0.011163	0	0	0
15	55.147	-16.786	0.01446	0	0	0
16	64.168	-26.458	0.018163	0	0	0
17	0.434	-34.643	0.027249	269.828	117.11	-338.08

18	0.868	-35.643	0.027632	270	234.36	-696.11
19	319.855	-37.254	0.022297	0	0	0
20	2.604	-37.643	0.028398	270	703.08	-2205.52
21	3.472	-37.643	0.028398	270	937.44	-2940.7
22	3.472	-39.643	0.029163	270	937.44	-3096.94

Resultant Force = ∑(δF) = 0.00 kip

Resultant Moment = ∑(δM) = 15433.23 kip-ft

Depth to neutral axis, *c = 7.830 in*
Compression Resultant, *C = -2929.43 kip*
Depth to Compression Resultant, d_c *=4.486 in*
Tension Resultant, *T = 2929.43 kip*
Depth to Tension Resultant, d_e *=67.706 in*

Depth to Tension Resultant (for shear), d_e *=66.80 in*
Nominal Capacity, M_n *= 15433.23 kip-ft*
Moment Arm, $d_e - d_c = M_n/T = 63.220$ *in*

The capacity reduction factor is

$$\varepsilon_t = 0.003\left(\frac{d_t}{c} - 1\right) = 0.003\left(\frac{67.72in}{7.830in} - 1\right) = 0.023$$

$$0.75 \le \phi = 0.75 + \frac{0.25(\varepsilon_t - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}} \le 1.0 = 0.75 + \frac{0.25(0.023 - 0.005)}{0.005 - 0.002} = 2.25 \;\therefore\; \phi = 1.0$$

$$M_r = 15433.23k \cdot ft \ge M_u = 14013.90k \cdot ft \;\; \textbf{OK}$$

6.1.1.2 *Minimum Reinforcement and the Cracking Moment*

In order to insure there is sufficient reinforcement in the section to achieve ductile behavior, a minimum amount of reinforcement is required. The minimum reinforcement is such that any section in the girder shall have adequate prestressed reinforcement to develop a factored flexural resistance, M_r, which is at least the lesser of the cracking strength or 133% of the ultimate moment. (LRFD 5.6.3.3)

$$M_{r\,min} = lesser\ of\ \begin{cases} M_{cr} \\ 1.33M_u \end{cases}$$

The cracking moment is

$$M_{cr} = \gamma_3\left[(\gamma_1 f_r + \gamma_2 f_{cpe})S_c - M_{dnc}\left(\frac{S_c}{S_b} - 1\right)\right]$$

where:

f_r	=	Modulus of rupture
f_{cpe}	=	Compressive stress due to prestressing at the bottom of the girder
S_c	=	Bottom section modulus of the composite section
S_b	=	Bottom section modulus of the non-composite section
M_{dnc}	=	Dead load moment resisted by the non-composite section
γ_1	=	Flexural cracking variability factor = 1.6
γ_2	=	Prestress variability factor = 1.1

γ_3 = Ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement = 1.0 for prestressed concrete

6.1.1.2.1 Compute cracking moment at $0.5L_g$.

$$f_r = 0.24\lambda\sqrt{f_c'} = 0.24(0.9375)\sqrt{6.8ksi} = 0.587ksi$$

$$f_{cpe} = 5.009ksi$$

$$S_c = 21243.5in^3$$

$$S_{nc} = 20696.2in^3$$

$$M_{dnc} = M_{girder} + M_{diaphragms} = 3599.46k \cdot ft$$

$$M_{cr} = 1.0\left[(1.6 \cdot 0.587ksi + 1.1 \cdot 5.009ksi)(21243.5in^3)\left(\frac{1ft}{12in}\right) - (3599.46k \cdot ft)\left(\frac{21243.5in^3}{20696.2in^3} - 1\right)\right]$$
$$= 11321.39k \cdot ft$$

6.1.1.2.2 Evaluate Minimum Reinforcement Requirement

$$M_u = 14013.90k \cdot ft$$

$$M_{r\,min} = lesser\ of \begin{cases} M_{cr} = 11321.39k \cdot ft \\ 1.33M_u = 1.33 \cdot 14013.9k \cdot ft = 18638.48k \cdot ft \end{cases} = 11321.39k \cdot ft$$

$M_r = 15433.23k \cdot ft \geq M_{r\,min} = 11321.39k \cdot ft$ **OK**

6.2 Check Splitting Resistance

Compute the splitting resistance of the pretensioned anchorage zone provided by the vertical reinforcement in the ends of the girder at the service limit states as $P_r = f_s A_s$ (5.10.10.1) where,

f_s = the stress in the steel not exceeding *20 ksi*

A_s = total area of vertical reinforcement located within the distance h/4 from the end of the beam (*in*^{2})

h = overall depth of the girder (*in*)

The resistance shall not be less than 4% of the prestressing force at transfer.

The splitting force at PSXFR is

Permanent Strands

$$P = 0.04A_{ps}(f_{pj} - \Delta f_{pR0} - \Delta f_{pES}) = 0.04(10.850in^2)(202.5ksi - 1.98ksi - 26.250ksi) = 76.75kip$$

Temporary Strands

$$P = 0.04A_{ps}(f_{pj} - \Delta f_{pR0} - \Delta f_{pES}) = 0.04(0.434in^2)(202.5ksi - 1.98ksi - 7.844ksi) = 3.41kip$$

$$P = 76.75kip + 3.41kip = 80.16\ kip$$

The splitting zone is $\frac{h}{4} = \frac{6.311ft}{4} = 1.578ft$. The vertical reinforcement in the splitting zone is $4.971in^2$.

The splitting resistance is $P_r = f_s A_s = (20\ ksi)(4.971in^2) = 99.41\ kip$

$$P < P_r\ \boldsymbol{OK}$$

If the splitting reinforcement does not fit within H/4 from the end of the girder, BDM 5.6.2F permits the total splitting reinforcement to extend beyond H/4 at a spacing not greater than 2.5"

6.3 Check Confinement Zone Reinforcement

For the distance of *1.5d* from the ends of the girder, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than #3 deformed bars with spacing not exceeding *6 in*.

The length of the confinement zone is $1.5d = 1.5(6.412ft)\ in = 9.617\ ft$.

Provide #3 bars spaced at 6" for the end $13.875ft$ of the girder.

7 Shear Capacity

Ensure the girder has sufficient capacity to resist shear in the Strength I limit state. Verify that shear reinforcement is adequately detailed.

These computations and checks demonstrate shear design at the critical section (LRFD 5.7.3.2 and 5.7.3.3). A complete design will also evaluated shear locations where abrupt changes to the shear force diaphragm occur and at changes in reinforcement size and spacing.

7.1 Locate Critical Section for Shear

The critical section for shear is located at d_v from the face of support where d_v is from the critical section. For purposes of design, the ultimate shear between the support and the critical section is equal to the shear at the critical section.

Determining the location of the critical section can be challenging because d_v varies with position along the girder. To find the critical section plot d_v along the length of the girder and draw a 45° line from the face of support towards the center of the girder. The intersection point of the 45° line and the d_v curve is the location of the critical section. Figure 7-1 illustrates this technique.

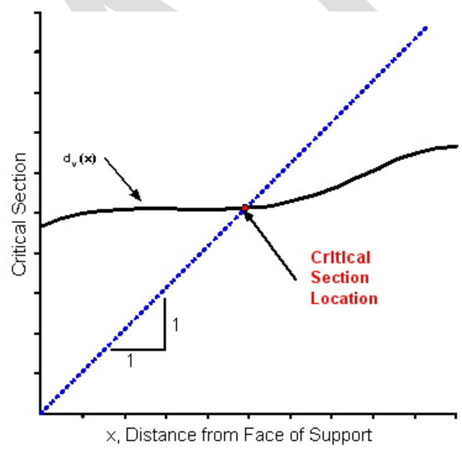


Figure 7-1: Graphical method to Determine Critical Section Location

For this girder, the critical sections are located $5.936\ ft$ and $149.397\ ft$ from the left support. The tables that follow show the details for finding the critical sections.

Table 7-1: Critical Section Calculation Details for Abutment 1

Location from Left Support (ft)	Assumed C.S. Location (in)	d_v (in)	CS Intersects?
(FoS) 0.500	0.000	65.995	No
(SZB) 0.670	2.039	65.971	No
(PSXFR) 1.792	15.500	65.809	No

(Bar Develop.) 2.473	23.677	65.713	No
4.796	51.557	65.389	No
5.936	65.235	65.235	*Yes
(H) 6.895	76.742	65.106	No
7.792	87.500	64.987	No
8.542	96.500	64.889	No
(1.5H) 10.093	115.114	64.690	No
(SZB) 12.670	146.039	64.368	No
(SZB) 13.420	155.039	64.277	No
(ST) 14.567	168.800	64.140	No
(0.1L$_e$) 15.533	180.400	64.026	No

* - Intersection values are linearly interpolated

Table 7-2: Critical Section Calculation Details for Abutment 2

Location from Left Support (ft)	Assumed C.S. Location (in)	d_v (in)	CS Intersects?
(0.9L$_e$) 139.800	180.400	64.026	No
(ST) 140.767	168.800	64.140	No
(SZB) 141.913	155.039	64.277	No
(SZB) 142.663	146.039	64.368	No
(1.5H) 145.241	115.114	64.690	No
146.792	96.500	64.889	No
147.542	87.500	64.987	No
(H) 148.438	76.742	65.106	No
149.397	65.235	65.235	*Yes
150.537	51.557	65.389	No
(Bar Develop.) 152.860	23.677	65.713	No
(PSXFR) 153.542	15.500	65.809	No
(SZB) 154.663	2.039	65.971	No
(FoS) 154.833	0.000	65.995	No

* - Intersection values are linearly interpolated

7.2 Check Ultimate Shear Capacity

7.2.1 Compute Nominal Shear Resistance

The nominal shear resistance, V_n, is the lesser of:

$$V_n = V_c + V_p + V_s$$

$$V_n = 0.25 f'_c b_v d_v + V_p$$

for which

$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v$

$V_s = \frac{A_v f_y d_v \cot\theta}{s}$

where

b_v	=	Effective web width taken as the minimum web width within the depth d_v.
d_v	=	Effective shear depth
s	=	Stirrup spacing
β	=	Factor indicating ability of diagonally cracked concrete to transmit tension
θ	=	Angle of inclination of diagonal compressive stresses
A_v	=	Area of shear reinforcement within a distance s
V_p	=	Component in the direction of the applied shear of the effective prestressing force, positive if resisting the applied shear.

7.2.1.1 *Determination of β and θ*

Step 1: Determine b_v

b_v is the effective web width. For this girder $b_v = 6.125in$.

Step 2: Determine d_v

d_v is the distance measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure (internal moment arm), but it need not be taken less than the greater of 0.9d_e or 0.72h.

From a flexural capacity analysis at the critical section the *Moment Arm =55.376 in*, d_e *=72.482 in*, and *h =75.906 in*.

$$d_v = greatest\ of \begin{cases} Moment\ Arm = 55.376in \\ 0.9d_e = 0.9(72.482in) = 65.235in \\ 0.72h = 0.72(75.906in) = 54.069in \end{cases} = 65.235in$$

Step 3: Compute stress in prestrssing steel when the stress in the surrounding concrete is 0.0 ksi

Per PCI BDM MNL-133-11 8.4.1.1.4

$$f_{po} = 0.75 f_{pu} = 202.5ksi$$

Step 4: Compute the longitudinal strain on the flexural tension side of the beam

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{E_s A_s + E_p A_{ps} + E_c A_{ct}}\ for\ \varepsilon_s < 0$$

At the critical section

$$f_{pe} = 149.190\ ksi$$

$$P_{eh} = (14)(0.217in^2)(149.190ksi) = 453.24\ kip$$

$$V_p = \frac{P_{eh}}{\sqrt{1^2 + \left(\frac{0.4L}{e'}\right)^2}}$$

$$e' = e_{hp} - e_e - (Y_{be} - Y_{bh}) = 37.644in - (-16.567in) - (50.373in - 42.073in) = 62.511in$$

$$0.4L = 63.1ft = 757.2in$$

$$V_p = \frac{(453.24 kip)}{\sqrt{1^2 + \left(\frac{757.2 in}{62.511 in}\right)^2}} = 37.29\ kip$$

$$M_u = 2075.23\ k \cdot ft$$

$$N_u = 0\ kip$$

$$V_u = 378.05 kip$$

$$|V_u - V_p| = 340.76\ kip$$

$$d_v = 65.235\ in$$

$$A_s = 0\ in^2$$

$$E_s = 29000\ ksi$$

Development length

$$l_d = \kappa\left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b = 1.6\left(213.837 ksi - \frac{2}{3} 150.206 ksi\right)(0.6 in) = 109.152 in$$

Location of critical section from end of girder

$$l_{px} = (5.936 ft + 1.208 ft)\left(\frac{12 in}{1 ft}\right) = 85.728 in$$

The effective prestress must be reduced to account for the lack of full development

$$\frac{f_{px}}{f_{ps}} = \frac{f_{pe} + \frac{(l_{px} - 60 d_b)}{(l_d - 60 d_b)}(f_{ps} - f_{pe})}{f_{ps}} = \frac{150.206 ksi + \frac{(85.728 in - 60(0.6 in))}{(109.152 in - 60(0.6 in))}(213.837 ksi - 150.206 ksi)}{213.837 ksi} = 0.905$$

$$A_{ps} = (36)(0.217 in^2)(0.905) = 7.068 in^2$$

$$E_{ps} = 28500\ ksi$$

$$A_{ct} = 491.530\ in^2$$

$$E_c = 3176.667 ksi$$

$$\varepsilon_s = \frac{\left(\frac{|2075.23 k \cdot ft|\left(\frac{12 in}{1 ft}\right)}{65.234 in} + 0.5(0) + 340.76 kip - (7.068 in^2)(202.5 ksi)\right)}{(29000 ksi)(0 in^2) + (28500 ksi)(7.068 in^2) + (3176.667 \text{ksi})(491.530 in^2)} = -0.4x10^{-3} < 0$$

Step 5: Compute β and θ

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} = \frac{4.8}{(1 + (750)(-0.4x10^{-3}))} = 6.86$$

$$\theta = 29 + 3500\varepsilon_s = 29 + (3500)(-0.4x10^{-3}) = 27.6°$$

7.2.1.2 *Compute Shear Capacity of Concrete*

$$V_c = 0.0316\beta\lambda\sqrt{f'_c} b_v d_v = 0.0316(6.86)(0.9375)\sqrt{6.8 ksi}(6.125 in)(65.234 in) = 211.66\ kip$$

7.2.1.3 *Compute Shear Capacity of Transverse Reinforcement*

For #5 stirrups, $A_v = 0.62\ in^2$.

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(0.62\ in^2)(60\ ksi)(65.237 in)\cot 27.6}{6\ in} = 773.65\ kip$$

7.2.1.4 *Compute Nominal Shear Capacity of Section*

$$V_n = V_c + V_p + V_s = 211.66\ kip + 37.29\ kip + 773.65\ kip = 1022.60\ kip$$

$$V_n = 0.25f_c' b_v d_v + V_p = 0.25(6.8\ ksi)(6.125\ in)(65.237in) + 37.29kip = 716.54kip$$

$$V_r = \phi V_n = 0.9(716.54\ kip) = 644.89\ kip$$

7.2.1.5 *Check Ultimate Shear Capacity*

$$V_u = 378.05\ kip \le V_r = 644.89\ kip\ \boldsymbol{OK}$$

Repeat these calculations at all locations where stirrup size or spacing changes or where the applied shear abruptly changes.

7.2.2 Check Requirement for Transverse Reinforcement

Transverse reinforcement is required when $V_u > 0.5\phi(V_c + V_p)$. (LRFD 5.8.2.4)

$$0.5\phi(V_c + V_p) = 0.5(0.9)(211.66\ kip + 37.29\ kip) = 112.03\ kip < 378.05\ kip$$

V_u exceeds the limiting value; therefore, transverse reinforcement is required at this section. Transverse reinforcement is provided. **OK**

7.2.3 Check Minimum Transverse Reinforcement

Where transverse reinforcement is required, as specified in LRFD 5.7.2.5, the area of steel shall not be less than $A_{v\,min} = 0.0316\lambda\sqrt{f_c'}\frac{b_v s}{f_y} = 0.0316(0.9375)\sqrt{6.8\ ksi}\frac{(6.125\ in)(6\ in)}{60\ ksi} = 0.047\ in^2 < 0.62\ in^2$ **OK**

This can also be represented as $\frac{A_v}{s}min = 0.0316\lambda\sqrt{f_c'}\frac{b_v}{f_y} = 0.0316(0.9375)\sqrt{6.8\ ksi}\frac{6.125\ in}{60\ ksi}\left(\frac{12in}{1ft}\right) = 0.095\frac{in^2}{ft}$.

7.2.4 Check Maximum Spacing of Transverse Reinforcement

The spacing of the transverse reinforcement shall not exceed the following:

- If $v_u < 0.125f_c'$ then $s \le 0.8d_v \le 24\ in$
- If $v_u \ge 0.125f_c'$ then $s \le 0.4d_v \le 12\ in$

$$v_u = \frac{|V_u - \phi V_p|}{\phi b_v d_v} = \frac{|378.05kip - 0.9(39.29kip)|}{0.9(6.125in)(65.234in)} = 0.958ksi$$

$$0.125f_c' = 0.125(6.8\ ksi) = 0.850ksi < 0.958ksi$$

$$s_{max} = 0.4d_v = 0.4(65.234\ in) = 26.09\ in > 12\ in \rightarrow s_{max} = 12\ in$$

The actual spacing is 6.0 in. **OK**

7.3 Check Longitudinal Reinforcement for Shear

At each section, the tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be proportioned to satisfy:

$$A_s f_y + A_{ps} f_{ps} \ge \left[\frac{M_u}{d_v \phi_f} + 0.5\frac{N_u}{\phi_a} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta\right]$$

At the inside edge of the bearing area of simple end supports to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \ge \left(\frac{V_u}{\phi_v} - 0.5V_s - V_p\right)\cot\theta$$

At the critical section, all of the harped strands are above the mid-height of the girder. The harped strands are not on the flexural tension side (See LRFD Figure 5.7.3.4.2-2)

$$A_{ps} = (36)(0.217in^2) = 7.812in^2$$

From the moment capacity analysis, $f_{ps,avg} = 98.036ksi$. The stress in the strands adjusted for lack of full development in the moment capacity analysis. Do not apply the reduction again in these calculations (See LRFD 5.9.4.3.2).

$M_u = 181.19 \cdot ft$

$d_v = 65.995in$

$V_u = 378.05kip$

$V_s = \frac{V_u}{\phi} = 420.05kip$

$V_p = 37.29kip$

$\theta = 27.6°$

$$\frac{M_u}{d_v \phi_f} + 0.5\frac{N_u}{\phi_a} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta$$
$$= \frac{181.19k \cdot ft\left(\frac{12in}{1ft}\right)}{(65.995in)(1.0)} + 0.5\frac{(0)}{1.0} + \left(\left|\frac{378.05kip}{0.9} - 37.29kip\right| - 0.5(420.05kip)\right)\cot 27.6° = 330.41kip$$

$A_{ps}f_{ps} = (7.812in^2)(98.036ksi) = 765.86kip$

$765.86kip \geq 330.41kip$ **OK**

7.4 Check Horizontal Interface Shear

This structure type does not have a composite deck. Horizontal interface shear analysis is not applicable.

8 Check Haunch Dimension

The slab offset is $2.0in$. Verify the haunch is large enough to accommodate the camber, but not too large that the girder has to carry unnecessary dead load. For such a large girder, an extra inch of concrete over the top flange can add up to a considerable amount of weight.

Because this girder has longitudinal top flange thickening, the goal is to have as uniform a haunch depth as possible. Ideally, at mid-span, the bottom of the nonstructural overlay will be directly on top of the girder with no extra buildup.

Account for geometric effects due to the roadway and camber.

The slab offset at the bearing is $A_{so} = A_{nonstructral\ overlay} + A_{profile\ effect} + A_{girder\ orientation\ effect} + A_{excess\ camber} + A_{top\ flange\ shape\ effect}$.

8.1 Nonstructural Overlay

The nonstructural overlay effect is simply the installed depth of the material.

$$A_{nonstructral\ overlay} = 1.5in$$

8.2 Profile Effect

The alignment does not have vertical or horizontal curves.

$$A_{profile} = 0.0in$$

8.3 Girder Orientation Effect

The roadway cross slope is built into the top flange of the girder.

$$A_{top\ flange\ effect} = 0.0in$$

8.4 Excess Camber

The excess camber is the camber that remains in the girder after all of the loads are applied.

Figure 8-1: Camber Effect

The graphic below illustrates how the girder deflects over time.

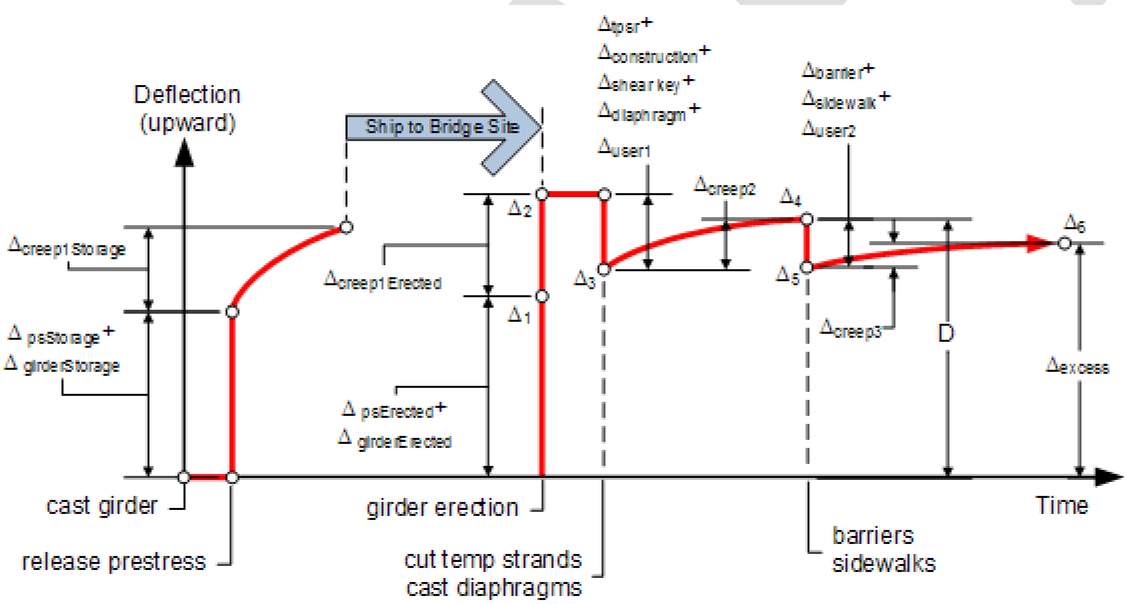


Figure 8-2: Camber Diagram

$$\Delta_{girder} = deflection\ due\ to\ girder\ self$$

$$\Delta_{ps} = deflection\ due\ to\ permanent\ prestressing, based\ on\ inplace\ span\ length$$

$$\Delta_{creep1} = \psi(t_e, t_i)(\Delta_{girder} + \Delta_{ps})$$

$$\Delta_{dia} = deflection\ due\ to\ diaphragm\ self\ weight$$

$$\Delta_{tsr} = deflection\ due\ to\ temporary\ strand\ removal$$

$$\delta_{girder} = incremental\ girder\ deflection\ due\ to\ change\ in\ support\ location\ between\ storage\ and\ erection$$

$$\Delta_{creep2} = [\psi(t_d, t_i) - \psi(t_e, t_i)](\Delta_{girder} + \Delta_{ps}) + \psi(t_d, t_e)(\Delta_{dia} + \delta_{girder})$$

$$\Delta_{nonstructural\ overlay} = deflection\ due\ to\ nonstructural\ overlay\ self\ weight$$

$$\Delta_{haunch} = deflection\ due\ to\ nonstructral\ overlay\ haunch\ self\ weight$$

$$\Delta_{barrier} = deflection\ due\ to\ traffic\ barrier\ self\ weight$$

$$\Delta_{creep3} = [\psi(t_f, t_i) - \psi(t_d, t_i)](\Delta_{girder} + \Delta_{ps}) + [\psi(t_f, t_e) - \psi(t_d, t_e)](\Delta_{dia} + \Delta_{tsr}) + \psi(t_f, t_d)(\Delta_{nonstructural\ overlay} + \Delta_{haunch} + \Delta_{barrier})$$

$$\Delta_{excess} = excess\ camber$$

$$\Delta_1 = (\Delta_{girder} + \Delta_{ps})$$

$$\Delta_2 = \Delta_1 + \Delta_{creep1}$$

$$\Delta_3 = \Delta_2 + \Delta_{dia} + \Delta_{tsr}$$

$$\Delta_4 = \Delta_3 + \Delta_{creep2}$$

$$\Delta_5 = \Delta_4 + \Delta_{nonstructural\ overlay} + \Delta_{haunch} + \Delta_{barrier}$$

$$\Delta_6 = \Delta_{excess} = \Delta_5 + \Delta_{creep3}$$

8.4.1 Compute Creep Coefficients

The creep coefficients for release until erection and deck casting are computed above.

Prestress release until erection $\psi(t_h = 90, t_i = 1) = \psi(t_e = 90, t_i = 1) = 0.935$

Prestress release until deck casting $\psi(t_d = 120, t_e = 1) = 1.006$

Compute creep coefficient for erection to deck casting

$$k_{td} = \frac{(2000 - 1)}{12\left(\frac{100 - 4(6.0)}{6.0 + 20}\right) + (2000 - 1)} = 0.983$$

$$\psi(t_d = 2000, t_e = 1) = 1.9(1.0)(0.96)(0.714)(0.983)(1)^{-0.118} = 1.28$$

f'_{ci} is the girder concrete strength at the time of load application to the erected girder and not the initial concrete strength at release.

$$f'_{ci} = 6.8\ ksi$$

$$k_f = \frac{5}{1 + 6.8} = 0.641$$

$$k_{td} = \frac{(120 - 90)}{12\left(\frac{100 - 4(6.8)}{6.8 + 20}\right) + (120 - 90)} = 0.479$$

$$\psi(t_d = 120, t_e = 90) = 1.9(1.0)(0.96)(0.641)(0.479)(90)^{-0.118} = 0.329$$

$$k_{td} = \frac{(2000 - 90)}{12\left(\frac{100 - 4(6.8)}{6.8 + 20}\right) + (2000 - 90)} = 0.983$$

$$\psi(t_d = 2000, t_e = 90) = 1.9(1.0)(0.96)(0.641)(0.983)(90)^{-0.118} = 0.676$$

$$k_{td} = \frac{(2000 - 120)}{12\left(\frac{100 - 4(6.8)}{6.8 + 20}\right) + (2000 - 120)} = 0.983$$

$$\psi(t_d = 2000, t_e = 120) = 1.9(1.0)(0.96)(0.641)(0.983)(120)^{-0.118} = 0.653$$

8.4.2 Compute Deflections

Girder Deflection, for the erected girder

$$\Delta_{gy} = -5.854in$$

Prestress Deflection, $\Delta_{ps} = 11.672in$. This is the deflection measured relative to the ends of the girder. The deflection at the CL Bearing based on the release datum is $\Delta_{psbrg} = 0.324n$. The prestress deflection measured relative to the bearings is $\Delta_{ps} = 11.672in - 0.324in = 11.348in$

Creep Deflection during Storage, $\Delta_{creep1} = 0.936(11.672in - 5.854in) = 5.134in$

Diaphragm Deflection, $\Delta_{diaphragm} = -0.131in$

Temporary strand removal deflection, $\Delta_{tsr} = 0.289in$

Creep Deflection between diaphragm and nonstructural overlay casting, $\Delta_{creep2} = (1.006 - 0.936)(11.348in - 5.857in) + (0.329)(-0.131in + 0.289in) = 0.469in$

Nonstructural Overlay Deflection, $\Delta_{nonstructural\ overlay} = -0.519in$

Nonstructural Haunch Deflection, $\Delta_{haunch} = -2.000in$

Traffic Barrier Deflection, $\Delta_{tb} = -1.010in$

Creep Deflection from nonstructural overlay casting to final, $\Delta_{creep3} = (1.28 - 0.936)(11.348in - 5.857in) + (0.676 - 0.329)(-0.131in + 0.289in) + (0.653)(-0.519in - 2.000in - 1.010in) = -1.295in$

$$\Delta_1 = 11.348in - 5.857in = 5.492in$$

$$\Delta_2 = 5.492in + 5.134in = 10.626in$$

$$\Delta_3 = 10.626 - 0.131in = 10.785in$$

$$\Delta_4 = 10.785in + 0.469in = 11.254in = D_{120}$$

$$\Delta_5 = 11.254in - 0.519in - 2.000in - 1.010in = 7.724in$$

$$\Delta_6 = 7.724in - 1.295in = 6.429in = \Delta_{excess}$$

$$A_{excess\ camber} = \Delta_{excess} = 6.429in$$

8.5 Top Flange Shape Effect

There is 6.5” of longitudinal top flange thickening measured from the ends of the girder. Measured relative to the CL Bearings, the longitudinal top flange thickening is

$$A_{top\ flange\ shape\ effect} = -6.302in$$

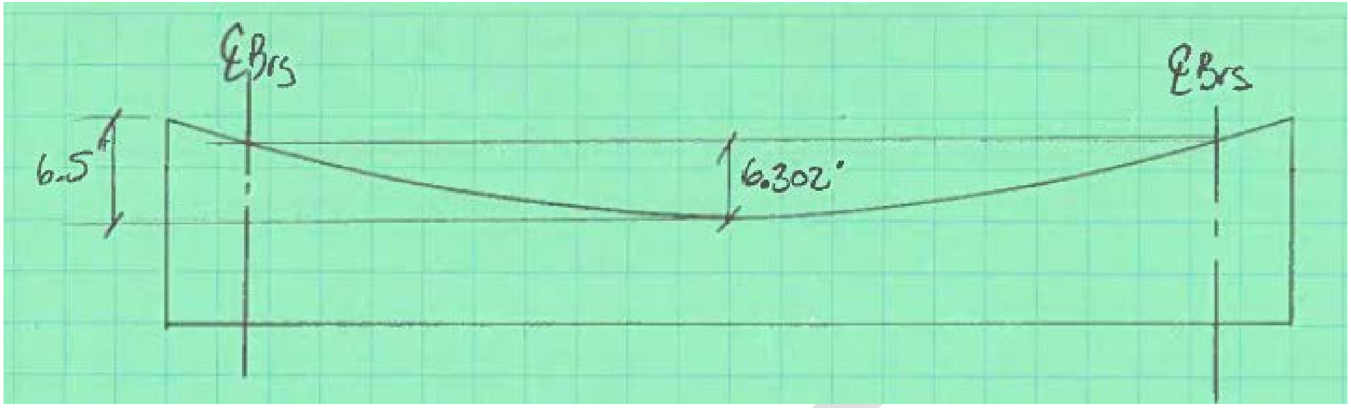


Figure 8-3: Top Flange Shape Effect

8.6 Check Required Slab Offset

The required slab offset is $A_{so} = A_{nonstructural\ overlay} + A_{top\ flange\ effect} + A_{profile\ effect} + A_{excess\ camber} + A_{top\ flange\ shape\ effect}$

At CL Span

$$A_{so} = 1.5in + 0.0in + 0.0in + 6.429in - 6.302in = 1.627\ in$$

At CL Bearing

$$A_{so} = 1.5in + 0.0in + 0.0in + 0in - 0in = 1.5\ in$$

The provided slab offset ("A" Dimensions) is 2.0 in. This is within the 0.25 in tolerance of the required value. **OK**

8.7 Longitudinal Top Flange Thickening

The purpose of the longitudinal top flange thickening is to eliminate the effect of camber so that the nonstructural overlay depth is essential constant. Figure 8-4 shows depth of the nonstructural overlay if longitudinal top flange thickening is not used.

Figure 8-4: Nonstructural overlay without longitudinal top flange thickening

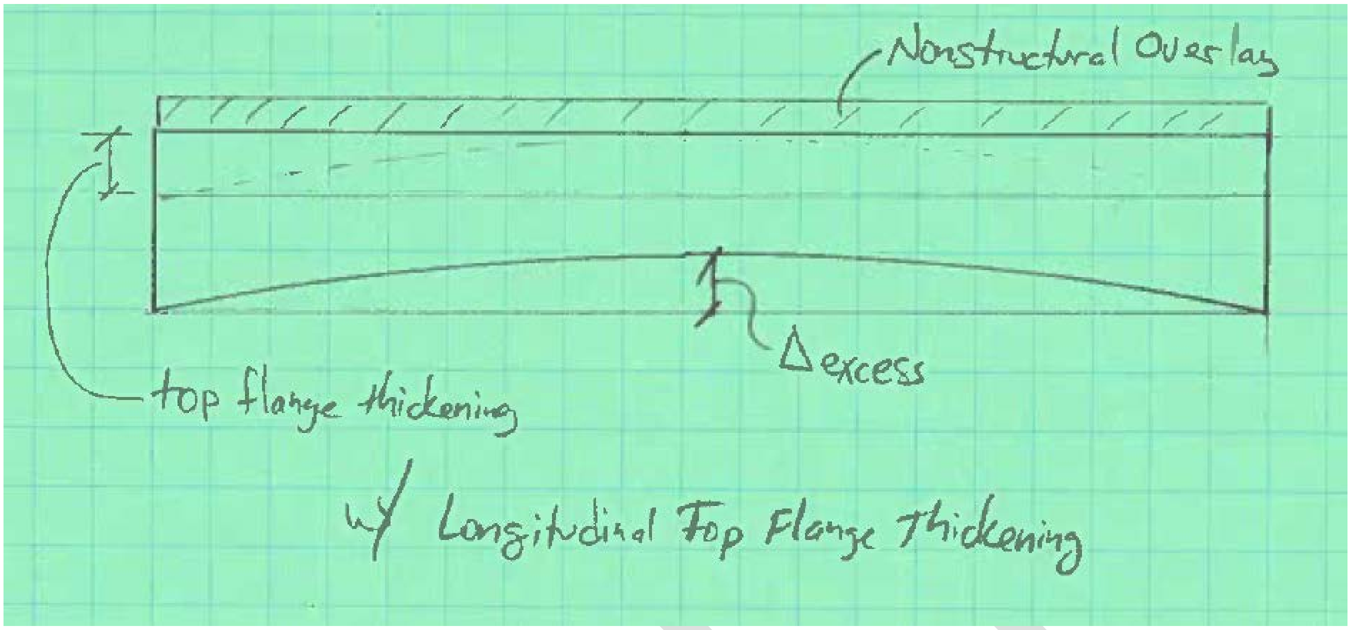


Figure 8-5: Nonstructural overlay with longitudinal top flange thickening

The top flange thickening should be slightly less than the excess camber. The excess camber of $6.429in$ is measured relative to the CL Bearings. From Figure 8-3 shows a $0.2\ in$ difference in top flange thickness from the CL Bearing to the end of the girder the longitudinal top flange thickening must accommodate $6.429in + 0.2in = 6.629in$. The longitudinal top flange thickening of $6.5in$ is adequate.

8.8 Compute Lower Bound Camber at 40 days

8.8.1 Creep Coefficients

Creep coefficients are computed the same as before, assuming erection at 10 days and deck casting at 40 days.

$$\psi_b(t_d = 10, t_i = 1) = 0.266$$

$$\psi_b\left(t_f = 40, t_1 = 1\right) = 0.686$$

$$\psi_b\left(t_f = 2000, t_1 = 1\right) = 1.280$$

$$\psi_b(t_d = 40, t_e = 10) = 0.427$$

$$\psi_b(t_d = 2000, t_e = 10) = 0.877$$

$$\psi_b(t_d = 2000, t_e = 40) = 0.744$$

8.8.2 Compute Deflections

$$\Delta_{creep1} = 0.266(11.348in - 5.854in) = 1.462in$$

$$\Delta_{creep2} = (0.686 - 0.222)(11.348in - 5.854in) + (0.427)(-0.131in + 0.289in) = 2.505in$$

$$\Delta_{creep3} = (1.28 - 0.686)(11.348in - 5.857in) + (0.877 - 0.427)(-0.131in + 0.289in) + (0.744)(-0.519in - 2.000in - 1.010in) = 0.171in$$

$$\Delta_1 = 11.348in - 5.854in = 5.492in$$

$$\Delta_2 = 5.492in + 1.462in = 6.954in$$

$$\Delta_3 = 6.954 - 0.131in = 7.112in$$

$$\Delta_4 = 7.112in + 2.505in = 9.617in = D_{40}$$

This is an upper bound value for D_{40}. There is a $\pm 25\%$ natural variation in camber from the mean value. Therefore, lower bound camber at 40 days = $0.5D_{40} = 0.5(9.617in) = 4.809in$.

8.9 Check for Possible Girder Sag

When the net deflection of the girder is downward, this is known as sag. The sag condition is most likely to occur for rapidly constructed bridges.

Compare the deflections after erection to the average value of D_{40} to determine the potential for sag. The average value is $75\% D_{40} = (0.75)(9.716in) = 7.287in$

$$\Delta_{SIDL} = \Delta_{nonstructural\ overlay} + \Delta_{haunch} + \Delta_{barrier} + \Delta_{creep3} = -0.519in - 2.000in - 1.010in + 0.171in = -3.358in$$

$$C_{SIDL} = -\Delta_{SIDL} = 3.358in$$

$$C_{SIDL} < Average\ D_{40}\ \textbf{OK}$$

For this bridge, the creep deflections are downwards for the longer construction scenario. Check for sag in this case also.

$$\Delta_{SIDL} = \Delta_{nonstructural\ overlay} + \Delta_{haunch} + \Delta_{barrier} + \Delta_{creep3} = -0.519in - 2.000in - 1.010in - 1.295in = -4.824in$$

$$C_{SIDL} = 4.824in$$

$$Average\ D_{120} = 0.75(11.254in) = 8.44in$$

$$C_{SIDL} < Average\ D_{120}\ \textbf{OK}$$

9 Check Lateral Deflections

For precast deck girder systems that utilize a UHPC closure joint, the alignment of adjacent girder top flanges is crucial. Lateral deflections affect the straightness of the joint as well as the width of the joint along the length of the girder. Figure 9-1 shows a typical longitudinal joint with good top flange fit up.

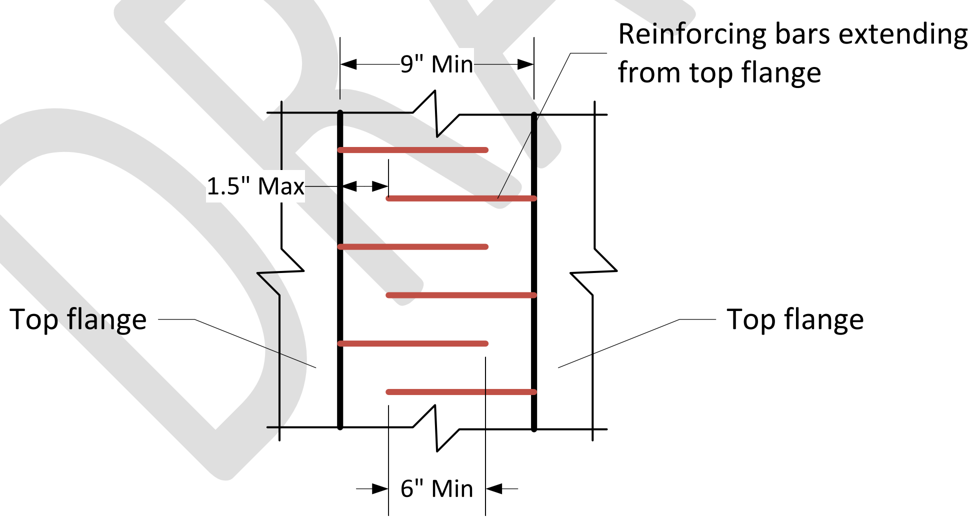


Figure 9-1: Longitudinal joint with good top flange fit up

Two items are of concern. Transverse reinforcement extends from the top flange of deck girders into the longitudinal closure joint. This reinforcement must have sufficient embedment in the joint to develop its tensile capacity. The reinforcement must also have sufficient overlap with the transverse bars extending from the adjacent girder to transfer tensile forces through non-contact lap splices.

Girders are not straight when lateral deflections are present. When lateral deflections of adjacent girders are in opposite directions from one another, joint are narrower or wider than designed. Figure 9-2 and Figure 9-3illustrate these conditions. If a joint becomes too narrow, the transverse bars may need shortening in the field to achieve adequate fit up. Joint bars may not develop their full capacity with less development then designed. If a joint becomes too wide, the length of the non-contact lap splices reduces and the ability transfer force across the joint is adversely affected.

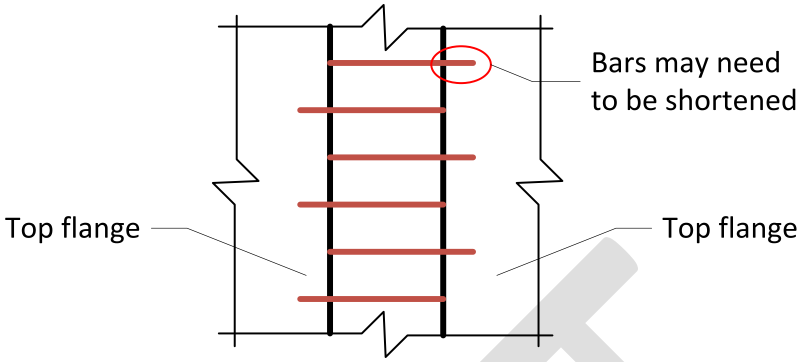


Figure 9-2: Narrow longitudinal joint

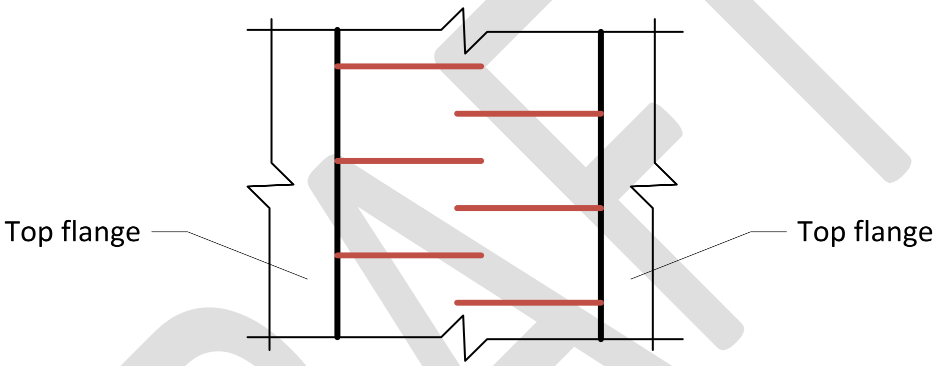


Figure 9-3: Wide longitudinal joint

Girders on either side of the crown point have cross sections that are mirror images of one another. Their lateral deflections will be in opposite directions. Typically, the crown point is along the centerline of the bridge and the greatest lateral deflection occurs at mid-span. The most poorly aligned location in the longitudinal joint will occur at the location of greatest stress.

Lateral deflection due to prestress will occur when the resultant prestress force is eccentric to the center of gravity of the girder. This is the case with asymmetric girder sections. WSDOT permits a sweep of 1/8" per 10 ft of girder length not to exceed ½". The lateral deflections due to prestressing may exceed the permissible sweep and girders could potentially be rejected. The rejection would not be warranted because the sweep is due to a design issue, not a fabrication issue. This lateral deflection will contribute to joint fit-up issues and to the instability of the girder during handling.

Using the lateral deflections computed earlier, the initial lateral deflection is

$$\Delta_{girder} = 0.434in$$

$$\Delta_{ps} = -0.274in$$

$$\Delta_{i} = 0.434in - 0.274in = 0.161in$$

This is about 30% of the permissible sweep.

Creep will occur during storage. The creep deflection, assuming shipping at 90 days is,

$$\Delta_{creep} = 0.135in$$

The lateral deflection at the end of storage and at the time of shipping is

$$\Delta_{es} = 0.161in + 0.135in = 0.296in$$

This is more than 50% of the permissible sweep.

The girders on either side of the bridge centerline will have lateral deflection in opposite direction. Since the center girder (Girder C) is symmetric it will be theoretically straight and the longitudinal joint connection with Girders B and D will be reduced by $0.296in$ at the mid span.

If there is an odd number of girders, the girders on either side of the bridge centerline will deflection in opposite directions. The longitudinal joint connection between these girders could be reduced by nearly $0.6\ in$.

WSDOT does not currently have a policy for limiting or accommodating excess lateral deflections.

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10 Bearing Seat Elevations

From the PGSuper Bridge Geometry Report, the roadway surface elevations at the CL Bearing points for Girder B are:

Abutment 1, Sta. 95+02.33, Offset 13.5ft L, Elev. 99.753ft

Abutment 2, Sta. 96+57.67, Offset 13.5ft L, Elev. 101.307ft

The slope of the girder is $\frac{101.307ft - 99.753ft}{155.33ft} = 0.0100\frac{ft}{ft}$

The slope-adjusted height of the girder is $76.022in\left(\sqrt{(0.01)^2 + (1)^2}\right) = 76.026in$

Deduct the sloped adjusted girder height and the slab offset from the roadway surface elevation to get the bottom of girder elevation.

Bottom of girder elevation at Abutment 1: Elev = $99.753ft - 76.026in\left(\frac{1ft}{12in}\right) - 2.0in\left(\frac{1ft}{12in}\right) = 93.251ft$

Bottom of girder elevation at Abutment 2: Elev =$101.307ft - 76.026in\left(\frac{1ft}{12in}\right) - 2.0in\left(\frac{1ft}{12in}\right) = 94.804ft$

After designing the bearings, add the bearing recess (typically ½”) and deduct the bearing depth from the bottom of girder elevation to get the bearing seat elevation.

11 Load Rating

The bridge opens for traffic without the future overlay installed. For this reason, take the DW force effects associated with the overlay as zero. Installing the overlay necessitates updating the load rating analysis.

11.1 Inventory Rating

11.1.1 Moment

$$RF = \frac{\phi_c\phi_s\phi_n K M_n - \gamma_{DC}M_{DC} - \gamma_{DW}M_{DW}}{\gamma_{LL}M_{LLIM}}$$

$$\phi_c\phi_s \geq 0.85$$

$$K = \frac{M_r}{M_{min}} \leq 1.0$$

At 0.5L

$$\phi_c = \phi_s = \phi_n = 1.0$$

$$M_n = 15433.23k \cdot ft$$

$$M_{DC} = 6288.72k \cdot ft$$

$$M_{DW} = 0.0k \cdot ft$$

$$M_{LLIM} = 2942.95\frac{k \cdot ft}{girder}$$

$$M_{cr} = 11321.39k \cdot ft$$

$$M_u = 14013.90k \cdot ft$$

$$M_{min} = min\begin{cases} M_{cr} \\ 1.33M_u \end{cases} = 11321.39k \cdot ft$$

$$K = \frac{15433.23k \cdot ft}{11321.39k \cdot ft} = 1.36 \therefore K = 1.0$$

$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.75$$

$$RF = \frac{(1)(1)(1)(1)(15433.23k \cdot ft) - (1.25)(6288.72k \cdot ft) - (1.5)(0k \cdot ft)}{(1.75)(2942.95k \cdot ft)} = 1.47$$

11.1.2 Shear

$$RF = \frac{\phi_c\phi_s\phi_n V_n - \gamma_{DC}V_{DC} - \gamma_{DW}V_{DW}}{\gamma_{LL}V_{LLIM}}$$

At 17.17ft (location where stirrup spacing increases)

$$\phi_c = \phi_s = 1.0, \phi_n = 0.9$$

$$V_n = 575.46kip$$

$$V_{DC} = 126.03kip$$

$$V_{DW} = 0.0k$$

$$V_{LLIM} = 86.13\frac{kip}{girder}$$

$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.75$$

$$RF = \frac{(1)(1)(0.9)(575.46kip) - (1.25)(126.03kip) - (1.5)(0kip)}{(1.75)(86.13kip)} = 2.39$$

11.1.3 Bending Stress – Service III limit state

$$RF = \frac{f_R - \gamma_{DC}f_{DC} - \gamma_{DW}f_{DW}}{\gamma_{LL}f_{LLIM}}$$

For load rating we use the AASHTO specified tension limit and live load factor

$$f_R = f_{limit} - f_{ps} = 0.19\lambda\sqrt{f_c'} - f_{ps}$$

$$f_R = 0.19(1.0)\sqrt{6.8ksi} - (-5.493ksi) = 5.957ksi$$

$$\gamma_{LL} = 1.0$$

$$RF = \frac{5.957ksi - (1.0)(3.676ksi) - 1.0(0ksi)}{(1.0)(1.662ksi)} = 1.37$$

11.2 Operating Rating

11.2.1 Moment

$$RF = \frac{\phi_c\phi_s\phi_n KM_n - \gamma_{DC}M_{DC} - \gamma_{DW}M_{DW}}{\gamma_{LL}M_{LLIM}}$$

$$\phi_c \phi_s \geq 0.85$$

$$K = \frac{M_r}{M_{min}} \leq 1.0$$

At 0.5L

$$\phi_c = \phi_s = \phi_n = 1.0$$

$$M_n = 15433.23k \cdot ft$$

$$M_{DC} = 6288.72k \cdot ft$$

$$M_{DW} = 0.0k \cdot ft$$

$$M_{LLIM} = 2942.95 \frac{k \cdot ft}{girder}$$

$$M_{cr} = 11321.39k \cdot ft$$

$$M_u = 11833.9k \cdot ft$$

$$M_{min} = min \begin{cases} M_{cr} \\ 1.33M_u \end{cases} = 11321.39k \cdot ft$$

$$K = \frac{15433.23k \cdot ft}{11321.39k \cdot ft} = 1.36 \therefore K = 1.0$$

$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.35$$

$$RF = \frac{(1)(1)(1)(1)(15433.23k \cdot ft) - (1.25)(6288.72k \cdot ft) - (1.5)(0k \cdot ft)}{(1.35)(2942.95k \cdot ft)} = 1.91$$

11.2.2 Shear

$$RF = \frac{\phi_c \phi_s \phi_n V_n - \gamma_{DC} V_{DC} - \gamma_{DW} V_{DW}}{\gamma_{LL} V_{LLIM}}$$

At 17.17ft (location where stirrup spacing increases)

$$\phi_c = \phi_s = 1.0, \phi_n = 0.9$$

$$V_n = 575.46kip$$

$$V_{DC} = 126.03kip$$

$$V_{DW} = 0.0k$$

$$V_{LLIM} = 86.13 \frac{kip}{girder}$$

$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.35$$

$$RF = \frac{(1)(1)(0.9)(575.46kip) - (1.25)(126.03kip) - (1.5)(0kip)}{(1.35)(86.13kip)} = 3.21$$

11.3 Legal Loads

Type 3, $M_{LLIM} = 1096.76k \cdot ft$

Type 3S2, $M_{LLIM} = 1422.13k \cdot ft$

Type 3-3, $M_{LLIM} = 1501.07k \cdot ft$

Type 3-3 rating will govern so we will show calculations of the rating factors for this loading. The rating factor calculations for the other loadings will be similar. The rating factor calculations for NRL, EV2, and EV3 are similar.

11.3.1 Moment

$$RF = \frac{\phi_c \phi_s \phi_n K M_n - \gamma_{DC} M_{DC} - \gamma_{DW} M_{DW}}{\gamma_{LL} M_{LLIM}}$$

$$\phi_c \phi_s \geq 0.85$$

$$K = \frac{M_r}{M_{min}} \leq 1.0$$

At 0.5L

$$\phi_c = \phi_s = \phi_n = 1.0$$

$$M_n = 15433.23k \cdot ft$$

$$M_{DC} = 6288.72k \cdot ft$$

$$M_{DW} = 0.0k \cdot ft$$

$$M_{LLIM} = 1501.07 \frac{k \cdot ft}{girder}$$

$$M_{cr} = 11321.39k \cdot ft$$

$$M_u = 10037.45k \cdot ft$$

$$M_{min} = min \begin{cases} M_{cr} \\ 1.33 M_u \end{cases} = 11321.39k \cdot ft$$

$$K = \frac{15433.23k \cdot ft}{11321.39k \cdot ft} = 1.36 \therefore K = 1.0$$

$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.45$$

$$RF = \frac{(1)(1)(1)(1)(15433.23k \cdot ft) - (1.25)(6288.72k \cdot ft) - (1.5)(0k \cdot ft)}{(1.45)(1501.07k \cdot ft)} = 3.48$$

11.3.2 Shear

$$RF = \frac{\phi_c \phi_s \phi_n V_n - \gamma_{DC} V_{DC} - \gamma_{DW} V_{DW}}{\gamma_{LL} V_{LLIM}}$$

At 17.17ft (location where stirrup spacing increases)

$$\phi_c = \phi_s = 1.0, \phi_n = 0.9$$

$$V_n = 614.9kip$$

$$V_{DC} = 126.03kip$$

$$V_{DW} = 0.0kip$$

$$V_{LLIM} = 46.95 \frac{kip}{girder}$$

$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.45$$

$$RF = \frac{(1)(1)(0.9)(614.9kip) - (1.25)(126.03kip) - (1.5)(0k)}{(1.45)(46.95kip)} = 5.81$$

11.3.3 Bending Stress – Service III limit state

This is a WSDOT requirement, not in MBE

$$RF = \frac{f_R - \gamma_{DC} f_{DC} - \gamma_{DW} f_{DW}}{\gamma_{LL} f_{LLIM}}$$

For load rating we use the AASHTO specified tension limit and live load factor

$$f_R = f_{limit} - f_{ps} = 0.19\lambda\sqrt{f'_c} - f_{ps}$$

Before we can compute the stress in the girder due to the prestressing, we must compute the effective prestress accounting for the elastic gain for to the Type 3-3 loading.

$$\Delta f_{pLL} = \frac{E_p}{E_c}\frac{M_{LLIM}\left(Y_{bc} - Y_{bg} + e\right)}{I_c} = \frac{28500ksi}{3176.667\ ksi}\frac{(1501.07k\cdot ft)(44.199in - 41.643in + 38.003in)}{938938.9in^4}\left(\frac{12in}{1ft}\right) = 6.98ksi$$

$$P = -(10.850in^2)(202.5ksi - 22.459ksi - 10.410ksi + 6.98ksi) = -1916.23kip$$

$$M_x = (-1916.23kip)(38.003in) = -72822.49k\cdot in$$

$$M_y = (-1916.23kip)(0.514in) = -984.94k\cdot in$$

$$\begin{aligned} f_{ps}(x,y) = {} & \frac{(-984.94k\cdot in)(861860.5in^4) + (-72822.49k\cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(18.6735in) \\ & - \frac{(-72822.49k\cdot in)(251152.4in^4) + (-984.94k\cdot in)(17465.9in^4)}{(861860.5in^4)(251152.4in^4) - (17465.9in^4)^2}(-41.643in) - \frac{1916.23kip}{1211.371in^2} \\ = {} & -0.183ksi - 3.527ksi - 1.582ksi = -5.292ksi \end{aligned}$$

$$f_R = 0.19(1.0)\sqrt{6.8}ksi - (-5.292ksi) = 5.757ksi$$

$$\gamma_{LL} = 1.0$$

$$RF = \frac{5.757ksi - (1.0)(3.676ksi) - 1.0(0ksi)}{(1.0)(0.558ksi)} = 2.45$$

11.4 Permit Loads

The load ratings for the permit loads are the same as the legal loads (with the obvious exception of the live load effects and load factors being different).

WSDOT also evaluates the optional reinforcement yielding check (MBE 6A.5.4.2.2b). The stress in the prestressing steel nearest the extreme tension fiber should not exceed $0.9f_y$. The analysis method used by PGSuper follows MBE A3.13.4.2b.

$$f_r = 0.9f_y = (0.9)(0.9)f_{pu} = (0.9)(0.9)(270ksi) = 218.7ksi$$

Moment beyond cracking

$$M_{bcr} = \gamma_{DC}M_{DC} + \gamma_{DW}M_{DW} + \gamma_{LL}M_{LLIM} - M_{cr}$$

Unlike the other permit rating cases where the one loaded lane live load distribution factor is used (MBE 6A.4.5.4.2b), use the governing of one loaded lane and two or more loaded lanes for these calculations (MBE C6A.5.4.2.2b).

For OL1, $M_{LLIM} = 2033.19k\cdot ft$ per girder.

For OL2, $M_{LLIM} = 3720.66k\cdot ft$ per girder

$$M_{bcr} = (1.0)(6288.72k\cdot ft) + (1.0)(0) + (1.0)(3720.66k\cdot ft) - 11321.39k\cdot ft = -1312k\cdot ft$$

Because $M_{bcr} < 0$, the loads aren't enough to cause cracking, so take $M_{bcr} = 0.0k\cdot ft$

The additional stress transferred to the reinforcement due to cracking is

$$f_{bcr} = \frac{E_s}{E_g}\frac{M_{bcr}(d_s - c)}{I_{cr}} = 0.0ksi$$

$$f_s = f_{pe} + f_{bcr}$$

Compute the effective prestress

For OL1

$$\Delta f_{pLL} = \frac{E_p}{E_c}\frac{M_{LLIM}\left(Y_{bc} - Y_{bg} + e\right)}{I_c} = \frac{28500ksi}{3176.667\ ksi}\frac{(2033.19k \cdot ft)(44.199in - 41.643in + 38.003in)}{938938.9in^4}\left(\frac{12in}{1ft}\right)$$
$$= 9.455ksi$$

$$f_{pe} = 202.5ksi - 22.459ksi - 10.410ksi + 9.455si = 179.086\text{ ksi}$$

$$f_s = f_{pe} + f_{brc} = 179.086ksi + 0ksi = 179.086ksi$$

For OL2

$$\Delta f_{pLL} = \frac{E_p}{E_c}\frac{M_{LLIM}\left(Y_{bc} - Y_{bg} + e\right)}{I_c} = \frac{28500ksi}{3176.667\ ksi}\frac{(3720.66k \cdot ft)(44.199in - 41.643in + 38.003in)}{938938.9in^4}\left(\frac{12in}{1ft}\right)$$
$$= 17.303ksi$$

$$f_{pe} = 202.5ksi - 22.459ksi - 10.410ksi + 17.303ksi = 186.934\text{ ksi}$$

$$f_s = f_{pe} + f_{brc} = 186.934ksi + 0ksi = 186.934\text{ksi}$$

Yield stress ratio

$$SR = \frac{f_r}{f_s}$$

OL1

$$SR = \frac{218.7ksi}{179.086ksi} = 1.22$$

OL2

$$SR = \frac{218.7ksi}{186.934ksi} = 1.17$$

12 Software

PGSuper is precast-prestressed girder design, analysis, and load rating software. PGSuper is part of the BridgeLink Bridge Engineering Application Suite jointly developed by the Washington State and Texas Departments of Transportation.

Download from http://www.wsdot.wa.gov/eesc/bridge/software

13 References

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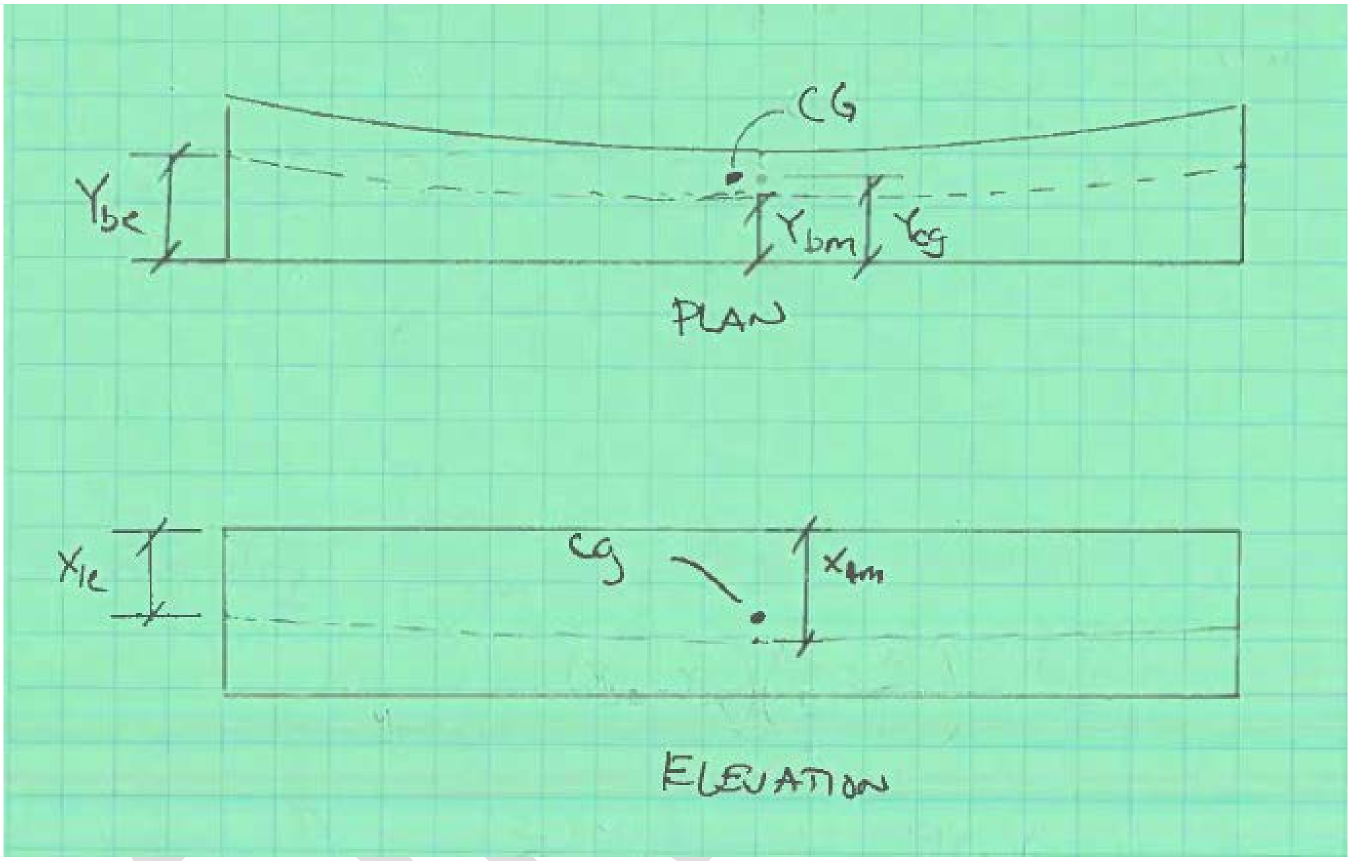
8. WSDOT, *Bridge Design Manual*, Washington State Department of Transportation
9. WSDOT, *PGSuper Theoretical Manual*, Washington State Department of Transportation

14 Appendix A

14.1 Girder center of mass

The girder is unrestrained and free to deform due to the precompression force. Take the center of stiffness of the girder to be the point of zero deformation. The center of gravity along the girder is in the shape of a parabolic curve. The center of stiffness is at the center of mass of the girder.



$$Y_{cg} = \frac{\int_0^L y_{cg}(z)dz}{L}$$

$$y_{cg}(z) = Y_{be} - 4(Y_{be} - Y_{bm})\left(\frac{z}{L} - \frac{z}{L^2}\right)$$

$$\int_0^L Y_{be} dz = Y_{be} L$$

$$\int_0^L 4(Y_{be} - Y_{bm})\left(\frac{z}{L} - \frac{z^2}{L^2}\right) dz = \frac{2}{3}(Y_{be} - Y_{bm})L$$

$$Y_{cg} = \frac{Y_{be}L - \frac{2}{3}(Y_{be} - Y_{bm})L}{L} = Y_{be} - \frac{2}{3}(Y_{be} - Y_{bm}) = Y_{be} + \frac{2}{3}(Y_{bm} - Y_{be})$$

Similarly, the lateral location center of mass is

$$X_{cg} == \frac{\int_0^L x_{cg}(z)dz}{L}$$

$$x_{cg}(z) = X_{le} - 4(X_{le} - X_{lm})\left(\frac{z}{L} - \frac{z}{L^2}\right)$$

$$\int_0^L X_{le}dz = X_{le}L$$

$$\int_0^L 4(X_{le} - X_{lm})\left(\frac{z}{L} - \frac{z^2}{L^2}\right)dz = \frac{2}{3}(X_{le} - X_{lm})L$$

$$X_{cg} = \frac{X_{le}L - \frac{2}{3}(X_{le} - X_{lm})L}{L} = X_{le} - \frac{2}{3}(X_{le} - X_{lm}) = X_{le} + \frac{2}{3}(X_{lm} - X_{le})$$

14.2 Deflections of Asymmetric Beams

Figure 4 shows an arbitrary asymmetric section with moments M_x and M_y about the x and y-axes.

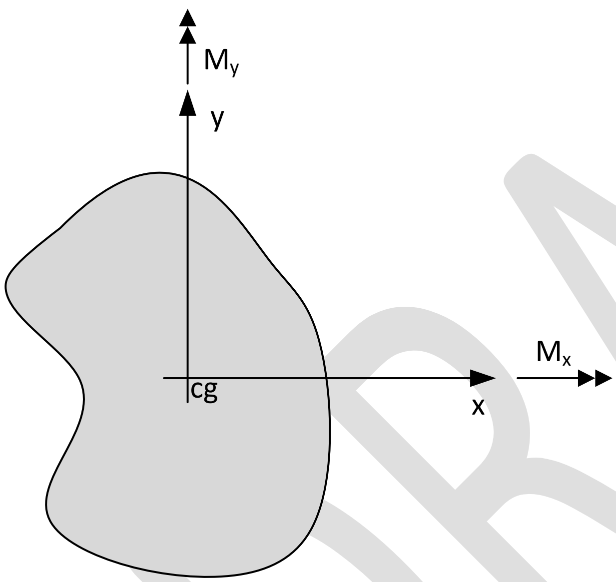


Figure 4 Arbitrary asymmetric section

Let ϕ_x and ϕ_y be the curvatures about the x-axis and y-axis respectively.

Equation 1 relates moments and curvatures.

$$\begin{bmatrix} M_x \\ M_y \end{bmatrix} = E\begin{bmatrix} I_{xx} & I_{xy} \\ I_{xy} & I_{yy} \end{bmatrix}\begin{bmatrix} \phi_x \\ \phi_y \end{bmatrix} \tag{1}$$

Solving Equation 1 for curvature gives,

$$\begin{bmatrix} \phi_x \\ \phi_y \end{bmatrix} = \frac{1}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)}\begin{bmatrix} I_{yy} & -I_{xy} \\ -I_{xy} & I_{xx} \end{bmatrix}\begin{bmatrix} M_x \\ M_y \end{bmatrix} \tag{2}$$

From classical Bernoulli beam theory, curvature is equal to the second derivate of deflection, therefore

$$\phi_x = \frac{d^2y}{dz^2} = \frac{M_xI_{yy} - M_yI_{xy}}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)} \tag{3}$$

$$\phi_y = \frac{d^2x}{dz^2} = \frac{M_y I_{xx} - M_x I_{xy}}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)} \tag{4}$$

Assuming superposition is applicable, the following two cases are independent and the summation of the deflections result in the total deflection.

14.2.1 Case 1 - $M_x \neq 0, M_y = 0$

M_x is caused by gravity loads (self-weight of girder) and prestressing that is eccentric with respect to the x-axis. Substituting $M_y = 0$ into Equations 3 and 4 yields

$$\phi_x = \frac{d^2y}{dz^2} = \frac{M_x I_{yy}}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)} \tag{5}$$

$$\phi_y = \frac{d^2x}{dz^2} = \frac{-M_x I_{xy}}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)} \tag{6}$$

The deflections in the x and y directions are found by integrating Equations 5 and 6 two times.

$$\Delta_{y1} = \iint \frac{d^2y}{dz^2} = \frac{I_{yy}}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)} \iint M_x(z) \tag{7}$$

$$\Delta_{x1} = \iint \frac{d^2x}{dz^2} = \frac{-I_{xy}}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)} \iint M_x(z) \tag{8}$$

Re-arranging Equations 7 and 8 gives

$$\iint M_x(z) = \frac{E\left(I_{xx}I_{yy} - I_{xy}^2\right)}{I_{yy}} \Delta_{y1} \tag{9}$$

$$\iint M_x(z) = \frac{E\left(I_{xx}I_{yy} - I_{xy}^2\right)}{-I_{xy}} \Delta_{x1} \tag{10}$$

Setting Equation 9 equal to Equation 10 and simplifying gives

$$\Delta_{x1} = -\frac{I_{xy}}{I_{yy}} \Delta_{y1} \tag{11}$$

Equation 11 gives the lateral deflection of an asymmetric section as a function of the vertical deflection.

14.2.2 Case 2 - $M_x = 0, M_y \neq 0$

M_y is caused by prestressing that is eccentric with respect to the y-axis. Substituting $M_x = 0$ into Equations 3 and 4 yields

$$\phi_x = \frac{d^2y}{dz^2} = \frac{-M_y I_{xy}}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)} \tag{12}$$

$$\phi_y = \frac{d^2x}{dz^2} = \frac{M_y I_{xx}}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)} \tag{13}$$

The deflections in the x and y directions are found by integrating Equations 12 and 13 two times.

$$\Delta_{y2} = \iint \frac{d^2y}{dz^2} = \frac{-I_{xy}}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)} \iint M_y(z) \tag{14}$$

$$\Delta_{x2} = \iint \frac{d^2x}{dz^2} = \frac{I_{xx}}{E\left(I_{xx}I_{yy} - I_{xy}^2\right)} \iint M_y(z) \tag{15}$$

Re-arranging Equations 14 and 15 gives

$$\iint M_x(z) = \frac{E\left(I_{xx}I_{yy} - I_{xy}^2\right)}{-I_{xy}} \Delta_{y2} \tag{16}$$

$$\iint M_x(z) = \frac{E\left(I_{xx}I_{yy} - I_{xy}^2\right)}{I_{xx}} \Delta_{x2} \tag{17}$$

Setting Equation 16 equal to Equation 17 and simplifying gives

$$\Delta_{y2} = -\frac{I_{xy}}{I_{xx}} \Delta_{x2} \tag{18}$$

Equation 18 gives the vertical deflection of an asymmetric section as a function of lateral deflection.

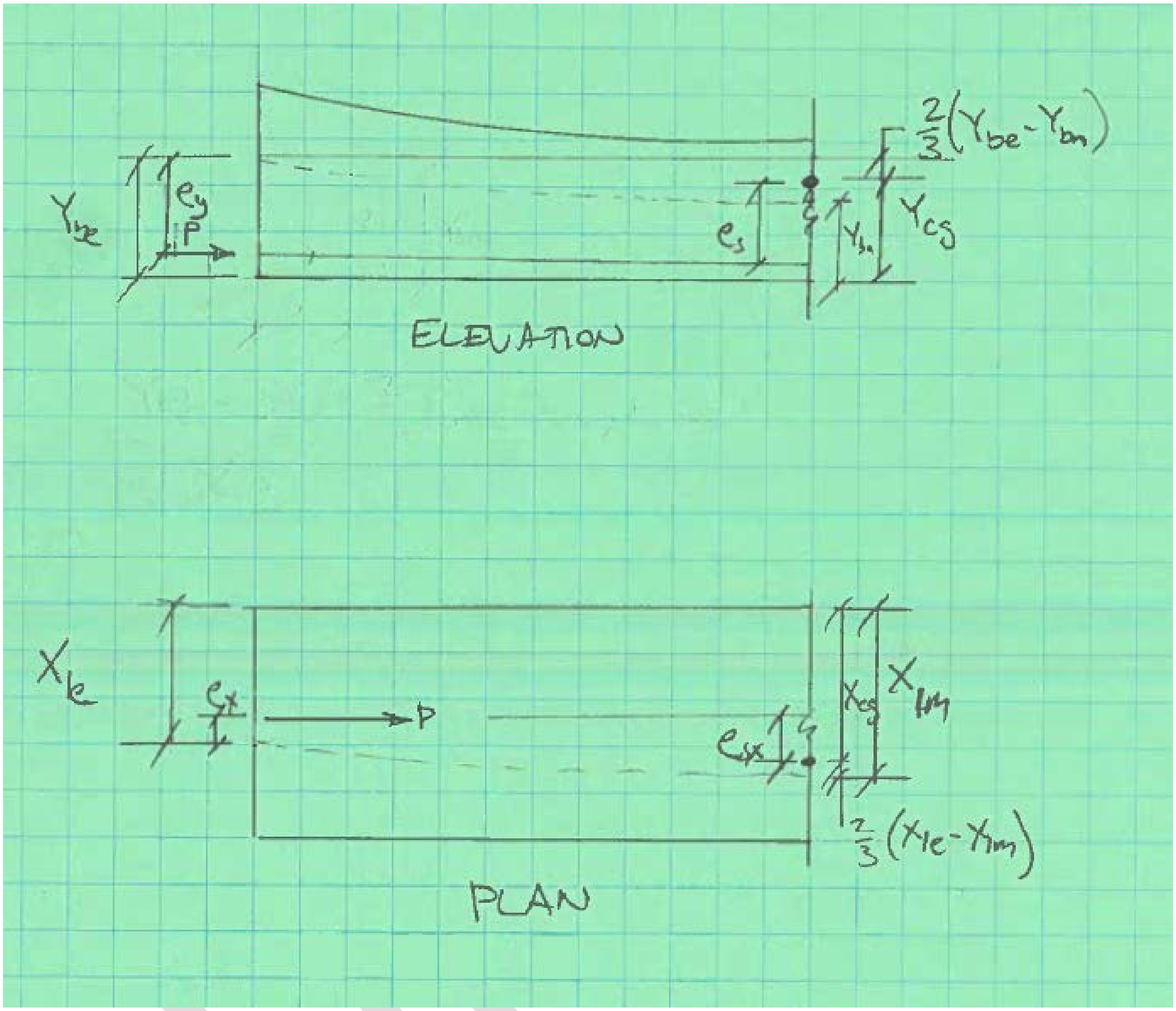
Superimposing the deflections from Case 1 and Case 2 gives the total deflection

$$\Delta_x = \Delta_{x1} + \Delta_{x2} \tag{19}$$

$$\Delta_y = \Delta_{y1} + \Delta_{y2} \tag{20}$$

14.3 Deflection due to straight strands

The free body diagrams below show the vertical and lateral eccentricity of a straight strand with respect to the center of mass of the girder.



$$e_y = e_s + \frac{2}{3}(Y_{be} - Y_{bm})$$

$$e_s = e_y + \frac{2}{3}(Y_{bm} - Y_{be})$$

$$e_x = e_{sx} + \frac{2}{3}(X_{le} - X_{lm})$$

$$e_{sx} = e_x + \frac{2}{3}(X_{lm} - X_{le})$$

The vertical deflection is

$$M_x = Pe_s$$

$$\Delta_{y1} = \frac{M_x L^2}{8} \frac{I_{yy}}{E(I_{xx} I_{yy} - I_{xy}^2)}$$

$$\Delta_{y2} = -\frac{I_{xy}}{I_{xx}} \Delta_{x2}$$

$$\Delta_y = \Delta_{y1} + \Delta_{y2}$$

Similarly the lateral deflection is,

$$M_y = Pe_{sx}$$

$$\Delta_{x2} = \frac{M_y L^2}{8} \frac{I_{xx}}{E(I_{xx} I_{yy} - I_{xy}^2)}$$

$$\Delta_{x1} = -\frac{I_{xy}}{I_{yy}} \Delta_{y1}$$

$$\Delta_x = \Delta_{x1} + \Delta_{x2}$$

These deflections are also applicable to part of the harped strand deflection and temporary strand deflection.

14.4 Deflection due to harped strands

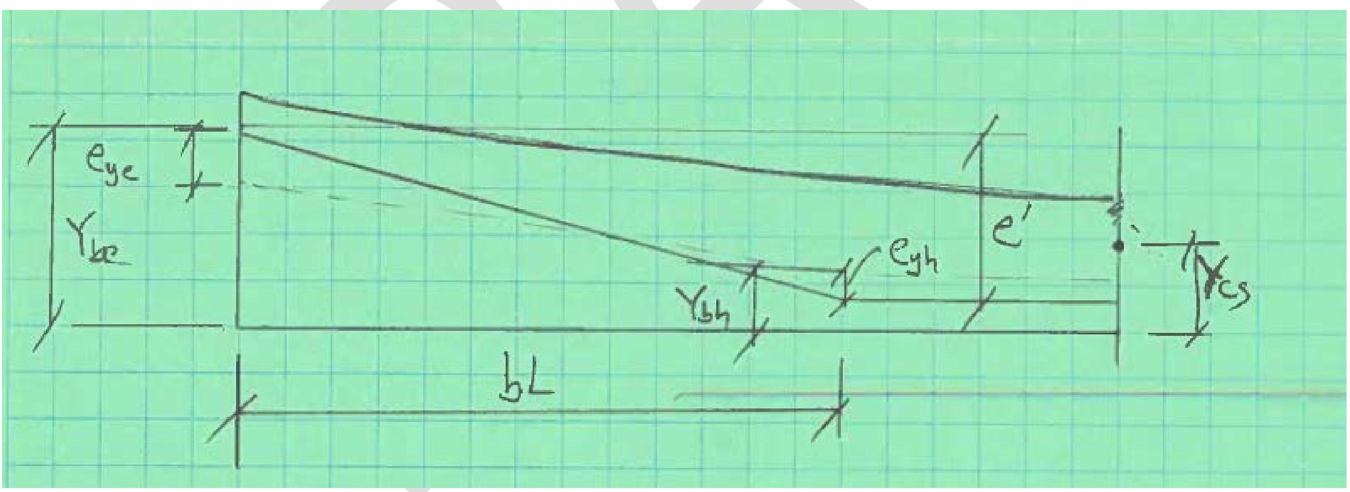
The deflection due to the harp strand consists of two parts. The first part is due to the eccentricity of the strand. The deflections due to this part are the same as for the straight strands. The second part is due to the angular change of the strand at the harp point.

The deflection due to the angular change at the harp point is

$$\Delta = \frac{b(3 - 4b^2) N L^3}{24} \frac{I_{yy}}{E(I_{xx} I_{yy} - I_{xy}^2)}$$

$$N = \frac{Pe'}{bL}$$

$$e' = (e_{yh} - e_{ye}) + (Y_{be} - Y_{bh})$$



The total vertical harped strand deflection is

$$M_x = Pe_s = Pe_{ye}$$

$$\Delta_{y1} = \left(\frac{M_x L^2}{8} + \frac{b(3 - 4b^2) N L^3}{24} \right) \frac{I_{yy}}{E(I_{xx} I_{yy} - I_{xy}^2)}$$

$$\Delta_{y2} = -\frac{I_{xy}}{I_{xx}} \Delta_{x2}$$

$$\Delta_y = \Delta_{y1} + \Delta_{y2}$$

14.5 Modifications to LRFD prestress loss equations for asymmetric beams

NCHRP Report 495 is the basis for the prestress loss equations in the AASHTO LRFD Bridge Design Specifications. The prestress loss equations where developed assuming a symmetric beam. Transverse and longitudinal top flange thickening results in an asymmetric beam. The following derives the creep and shrinkage loss equations for an asymmetric beam.

Refer to NCHRP Report 495 for further information.

14.5.1 Prestress loss due to shrinkage for independent girder element

$$\Delta\varepsilon_p = \Delta\varepsilon_c$$

$$\Delta\varepsilon_p = \frac{\Delta P}{A_{ps}E_{ps}}$$

$$\Delta\varepsilon_c = \varepsilon_{bid} - \left(\frac{\Delta P}{E_{ci}A_n} - \frac{\Delta P e_x I_{xx} + \Delta P e_y I_{xy}}{E_{ci}\left(I_{xx}I_{yy} - I_{xy}^2\right)} e_x + \frac{\Delta P e_y I_{yy} + \Delta P e_x I_{xy}}{E_{ci}\left(I_{xx}I_{yy} - I_{xy}^2\right)} e_y\right)\left(1 + 0.7\psi_{bif}\right)$$

$$E_{ps}\varepsilon_{bid} = \frac{\Delta P}{A_{ps}}\left(1 + \left(1 + 0.7\psi_{bif}\right)\left(\frac{E_{ps}}{E_c}\right)\left(\frac{A_{ps}}{A_n}\right)\left(1 + \frac{A_n}{I_{xx}I_{yy} - I_{xy}^2}\left(\left(e_y I_{yy} + e_x I_{xy}\right)e_y - \left(e_x I_{xx} + e_y I_{xy}\right)e_x\right)\right)\right)$$

$$\Delta f_{pSR} = \frac{\Delta P}{A_{ps}} = E_{ps}\varepsilon_{bid}K_{id}$$

$$K_{id} = \frac{1}{1 + \left(\frac{E_{ps}}{E_{ci}}\right)\left(\frac{A_{ps}}{A_n}\right)\left(1 + \frac{A_n}{I_{xx}I_{yy} - I_{xy}^2}\left(\left(e_y I_{yy} + e_x I_{xy}\right)e_y - \left(e_x I_{xx} + e_y I_{xy}\right)e_x\right)\right)\left(1 + 0.7\psi_{bif}\right)}$$

14.5.2 Prestress loss due to creep for independent girder element

$$\Delta\varepsilon_p = \Delta\varepsilon_c$$

$$\frac{\Delta P}{A_{ps}E_{ps}} = \frac{f_{cgp}}{E_{ci}}\psi_{bid} - \left(\frac{\Delta P}{E_{ci}A_n} - \frac{\Delta P e_x I_{xx} + \Delta P e_y I_{xy}}{E_{ci}\left(I_{xx}I_{yy} - I_{xy}^2\right)} e_x + \frac{\Delta P e_y I_{yy} + \Delta P e_x I_{xy}}{E_{ci}\left(I_{xx}I_{yy} - I_{xy}^2\right)} e_y\right)\left(1 + 0.7\psi_{bif}\right)$$

$$\frac{E_{ps}}{E_{ci}} f_{cgp} = \frac{\Delta P}{A_{ps}}\left(1 + \left(1 + 0.7\psi_{bif}\right)\left(\frac{E_{ps}}{E_c}\right)\left(\frac{A_{ps}}{A_n}\right)\left(1 + \frac{A_n}{I_{xx}I_{yy} - I_{xy}^2}\left(\left(e_y I_{yy} + e_x I_{xy}\right)e_y - \left(e_x I_{xx} + e_y I_{xy}\right)e_x\right)\right)\right)$$

$$\Delta f_{pCR} = \frac{\Delta P}{A_{ps}} = \frac{E_{ps}}{E_{ci}} f_{cgp}\psi_{bid}K_{id}$$