Precast, Prestress Bridge Girder Design 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 example 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 Centroid	< 0
Strand Eccentricity below 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
10+00	N 34° 45' 32" W	12° 34' 15" L	6000 ft

Profile

PVI Station	PVI Elevation	Grade in (g_1)	Grade out (g_2)	Length
9+00	100.00	-2%	-1.5%	600 ft

Superelevations

Left	Right
$-0.04\frac{ft}{ft}$	$0.04\frac{ft}{ft}$

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2.3 Bridge Layout

Back of Pavement Seat, Abutment 1, 7+00

Back of Pavement Seat, Abutment 2, 8+30

Abutments are oriented at $S\ 51°\ 47'\ 43"\ W$

Abutment 1, Skew Angle 0°

Abutment 2, Skew Angle $1°\ 14'29.54"\ R$

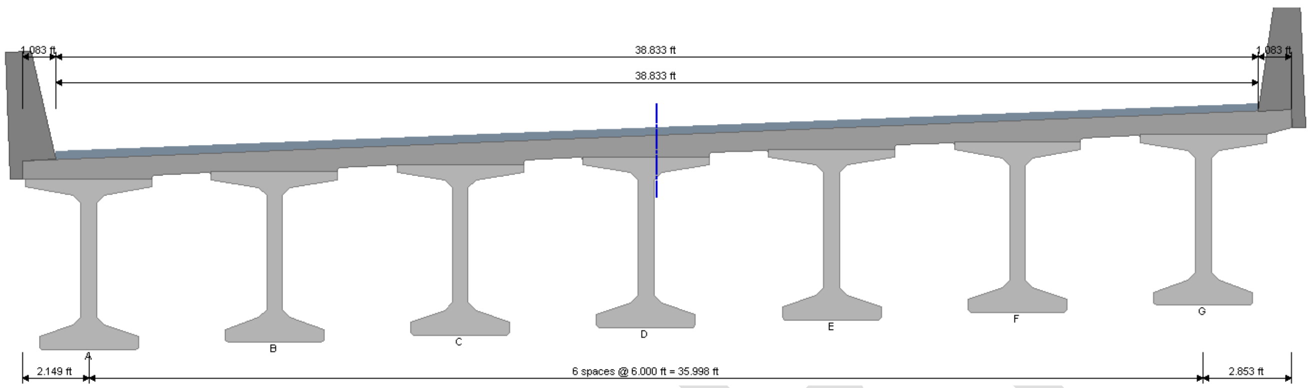

Figure 2-1: Bridge Section at Station 7+65

Girders

7 WF66G @ 6'-0" measured radial at BPS

$A = 874.531\ in^2$
$I_x = 556339.2\ in^4$
$I_y = 71865.3\ in^4$
$Y_t = 34.196\ in$
$Y_b = 31.804\ in$
$S_t = 16268.9\ in^3$
$S_b = 17493.0\ in^3$
Perimeter = 273.284 in

$W_{tf} = 49.0\ in$
$W_{bf} = 38.375\ in$
$t_{web} = 6.125\ in$

f'_{ci} = to be determined

f'_c = to be determined

$\gamma_c = 155\ lb/ft^3$
$\gamma_c = 165\ lb/ft^3$ (including rebar)

Prestressing = to be determined

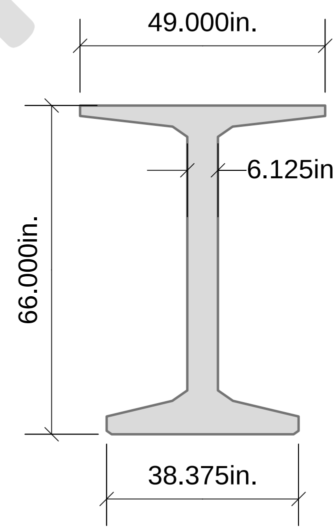

Figure 2-2: Girder Dimensions

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

Interior Diaphragms

Rectangular – Between girders only. H = 31.5 in
T = 8.00 in
Located at $0.25L_s$, $0.5L_s$, and $0.75L_s$.

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Slab

Gross Depth = 7.5 in
Overhang = 2'-6"
Slab Offset ("A" Dimension) = To be determined
Fillet = ¾"
Sacrificial Depth = ½"
f'c = 4 ksi
γ_c = 150 lb/ft^3
γ_c = 155 lb/ft^3 (including rebar)
Future Wearing Surface, 0.035 k/ft^2

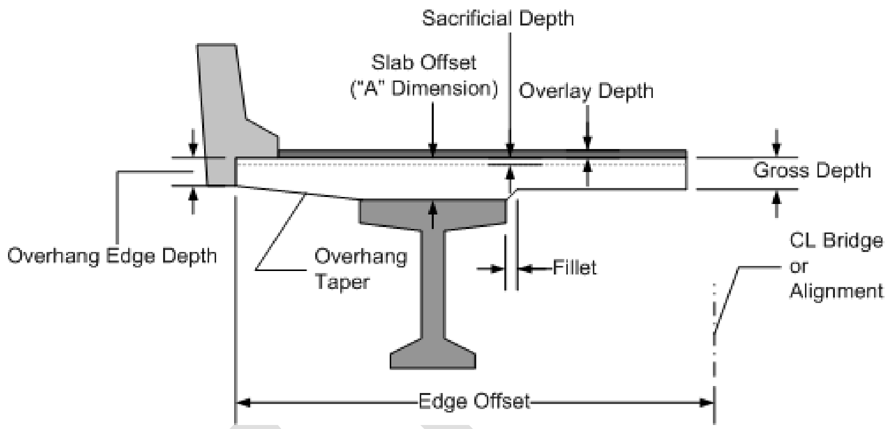

Figure 2-3: Slab Detail

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

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.

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3 Design Preliminaries

Design and load rate the first interior girder (Girder B).

3.1 Construction Sequence

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

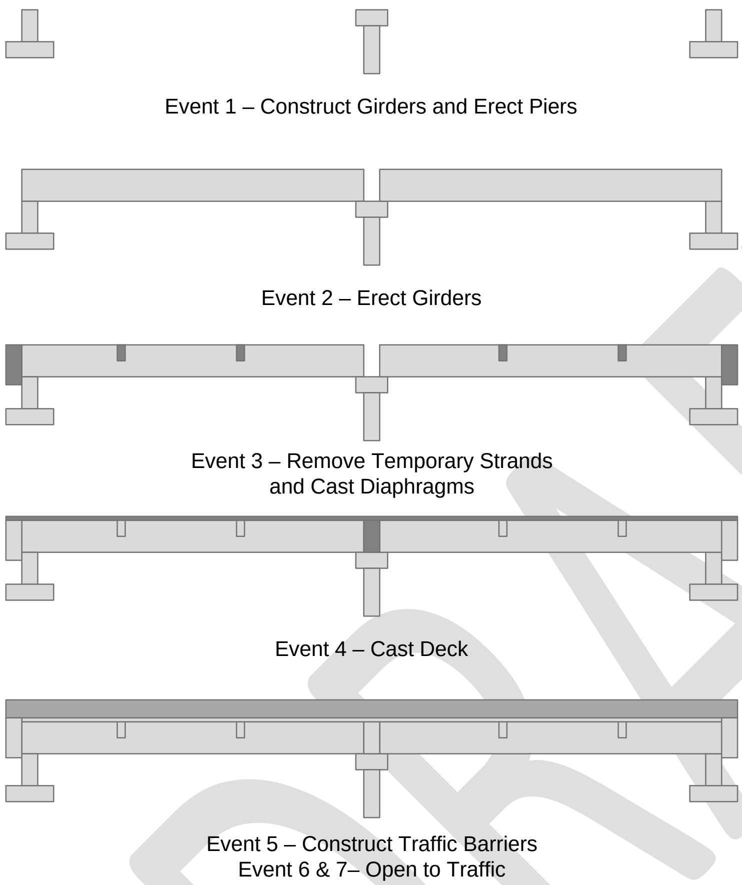

Figure 3-1 Assumed Construction Sequence

3.2 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.

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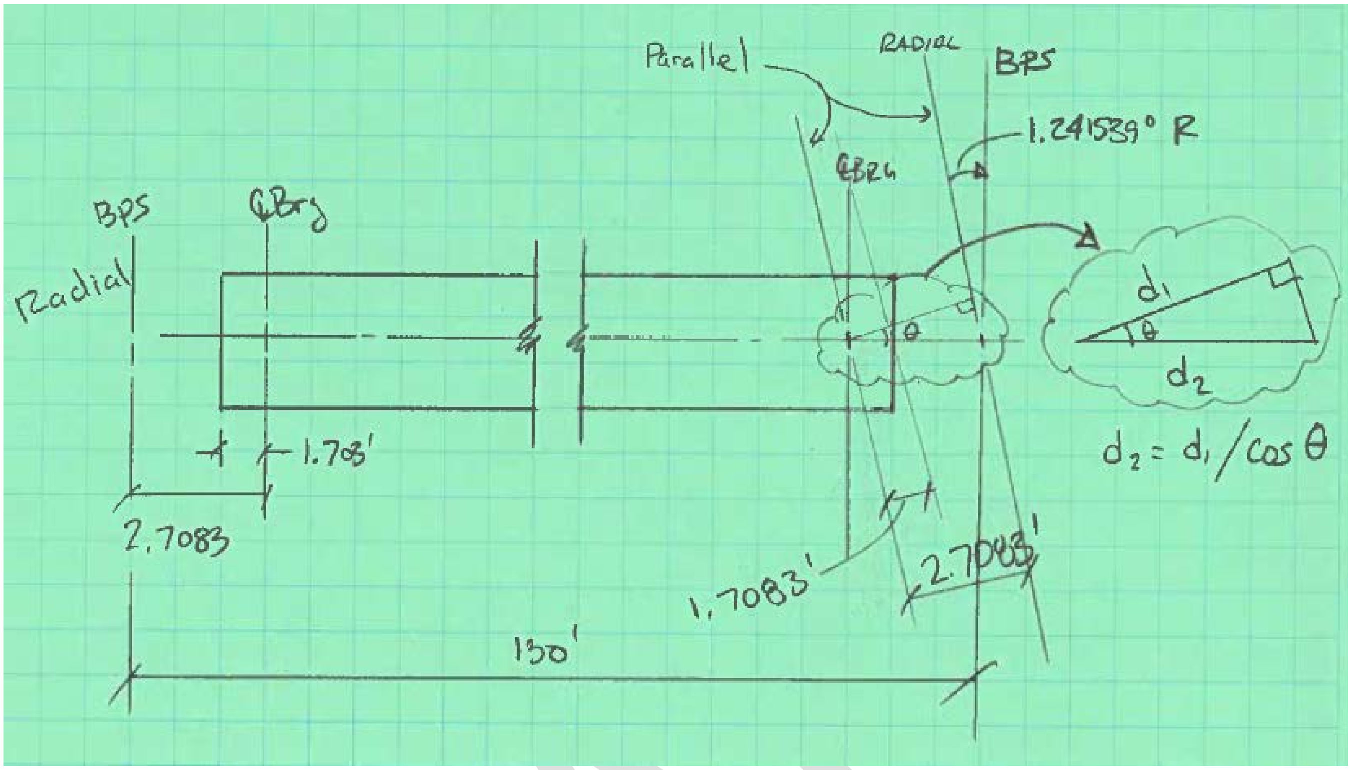

Figure 3-2 Connection Geometry

The bearing-to-bearing span length is $L_s = 130ft - 2.7083ft - (2.7083ft)/\cos(1.241539) = 124.58ft$.

The overall girder length is $L_g = 124.58ft + 1.7083ft + (1.7083ft)/\cos(1.241539) = 127.997ft$.

3.3 Section Properties

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

3.3.1 Effective Flange Width

The effective flange width of a composite concrete deck slab is the tributary width of the member (LRFD 4.6.2.6.1).

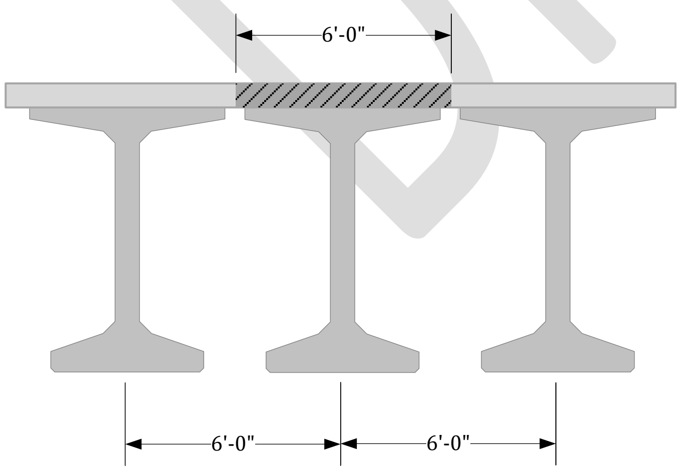

Figure 3-3 Effective Flange Width

$$w_{eff} = 6.0ft = 72in$$

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3.3.2 Composite Girder Properties

Transform the slab to equivalent girder material and use the parallel axis theorem to compute the composite girder properties. At mid-span the bottom of the slab is above the top of the girder by the fillet amount (¾ “). If the actual camber exceeds the predicted camber, the ¾” fillet can be easily lost. Assume the bottom of the slab is directly on top of the girder. This provides the least stiff section where the maximum demand occurs. For simplicity, use this section model at all locations (BDM 5.6.2.B.1).

PGSuper has options to include the haunch depth in the section properties calculations. Each section can use the minimum haunch depth (fillet dimension) or the actual haunch depth. Using the actual haunch depth means there is a different set of section properties at every cross section. Using more precise section properties may be desirable for load rating.

Modulus of elasticity of slab concrete

$$E_c = 120{,}000 K_1 w_c^2 f_c'^{0.33} = (120{,}000)(1.0)(0.150)^2(4.0)^{0.33} = 4266.223\ ksi$$

Modulus of elasticity of girder concrete assuming a concrete strength of $f_c' = 6.3ksi$

$$E_c = 120{,}000 K_1 w_c^2 f_c'^{0.33} = (120{,}000)(1.0)(0.155)^2(6.3)^{0.33} = 5292.088\ ksi$$

$$n = \frac{E_{c\ slab}}{E_{c\ girder}} = \frac{4266.223ksi}{5292.088ksi} = 0.806$$

The sacrificial wearing surface is not part of the structural section. Use the structural slab depth for computing section properties.

$$t_{slab} = t_{gross\ slab\ depth} - t_{sacrifical\ depth} = 7.5in - 0.5in = 7.0in$$

	Area	*Y_b*	*(Area)(Y_b)*
Slab	$(0.806)(72in)(7.0in) = 406.300in^2$	$66.0in + \frac{7.0in}{2} = 69.5in$	$28237.85in^3$
Girder	$874.531in^2$	$31.804in$	$27813.584in^3$
Total	$A_c = 1280.832in^2$		$56051.434in^3$

$$Y_{bc} = \frac{\sum(Area)(Y_b)}{\sum(Area)} = \frac{56051.434in^3}{1280.832in^2} = 43.762in$$

$$Y_{tc\ girder} = H_g - Y_{bc} = 66.0in - 43.762in = 22.238in$$

	Area	*d*	*(Area)(d^2)*	*I_o*	*I_o + (Area)(d^2)*
Slab	$406.300in^2$	$66.0in + \frac{7.0in}{2} - 43.762in = 25.768in$	$269151.259in^4$	$\frac{1}{12}(0.806)(72in)(7.0in)^3 = 1658.748in^4$	$270810in^4$
Girder	$874.531in^2$	$31.804in - 43.762in = -11.958in$	$125052.479in^4$	$556339.2in^4$	$681391.68in^4$
					$I_x = 952201.679in^4$

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$$S_{bc} = \frac{I_x}{Y_{bc}} = \frac{952201.679in^4}{43.762in} = 21758.642in^3$$

$$S_{tc\ girder} = \frac{I_x}{Y_{tc\ girder}} = \frac{952201.679in^4}{22.238in} = 42818.674in^3$$

3.3.3 First Moment of Area of deck slab,

$$Q_{slab} = A_{slab}\left(Y_{tc\ girder} + \frac{t_{slab}}{2}\right) = 406.3in^2\left(22.238in + \frac{7in}{2}\right) = 10457.349in^3$$

3.3.4 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.

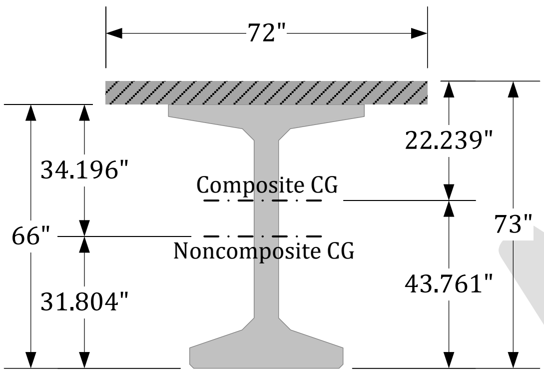

Figure 3-4 Centroid of Non-composte and Composite Section

Table 3-1: Section Properties from PGSuper

	Girder	*Composite Girder*
*Area, **A***	$874.531\ in^2$	$1280.808\ in^2$
$\boldsymbol{I_x}$	$556339.2\ in^4$	$952196.3in^4$
$\boldsymbol{I_y}$	$71865.3\ in^4$	-
$\boldsymbol{Y_{top\ girder}}$	$34.196\ in$	$22.239\ in$
$\boldsymbol{Y_{top\ slab}}$	-	$29.239\ in$
$\boldsymbol{Y_b}$	$31.804\ in$	$43.761\ in$
$\boldsymbol{S_{top\ girder}}$	$16268.9\ in^3$	$42816.4\ in^3$
$\boldsymbol{S_{top\ slab}}$	-	$40396.8\ in^3$
$\boldsymbol{S_b}$	$17493.0\ in^3$	$21759.0\ in^3$
$\boldsymbol{Q_{slab}}$	-	$10457.2\ in^3$
Effective Flange Width, $\boldsymbol{W_{eff}}$	-	$71.996\ in$
Perimeter	$273.284\ in$	-

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3.4 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 diaphragms and deck (dead load applied to non-composite girder section)
4) Install railing system (traffic barriers, sidewalks, etc). (dead load applied to composite section)
5) Final without Live Load (includes future overlay if applicable)
6) 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.

3.4.1 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.

$$w_{girder} = \gamma_c A_g = (0.165kcf)(874.531in^2)\left(\frac{1ft^2}{144in^2}\right) = 1.002\ klf$$

where:

A_g = Gross cross sectional area of the girder

γ_c = Unit weight of concrete

$$M_g = \frac{wx}{2}(l - x)$$

Moment at point of prestress transfer (PSXFR)

Prestress transfer occurs over 60 strand diameters (LRFD 5.9.4.3.1)

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

$$M_g = \frac{(1.002klf)(3ft)}{2}(127.997ft - 3ft) = 187.87k \cdot ft$$

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Moment at harp point (HP)

Harp point is 0.4L from the end of the girder $(0.4)(127.997ft) = 51.199ft$

$$M_g = \frac{(1.002klf)(51.199ft)}{2}(127.997ft - 51.199ft) = 1969.92k \cdot ft$$

Moment at mid-span (0.5L)

$$M_g = \frac{(1.002klf)\left(\frac{127.997ft}{2}\right)}{2}\left(127.997ft - \frac{127.997ft}{2}\right) = 2052.0k \cdot ft$$

3.4.2 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 and roadway slab casting. Installation of the railing system occurs after the roadway slab gains adequate strength.

3.4.2.1 *Diaphragm and Deck Placement*

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

3.4.2.1.1 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

$$P = HW\gamma_c(S - t_{web}) = (31.5in)(8in)(0.155kcf)(72in - 6.125in)\left(\frac{1ft^3}{1728in^3}\right) = 1.49kip$$

Diaphragms are located at $31.145\ ft\ (0..25L)$, $62.290\ ft\ (0.5L)$ and $93.435\ ft\ (0.75L)$ from the left bearing.

3.4.2.1.2 Slab Loads

The slab load consists of the main slab and the slab haunch.

3.4.2.1.2.1 Main Slab Load

The main slab load is

$$w_{slab} = W_{trib}t_{slab}\gamma_c = (72in)(7.5in)(0.155kcf)\left(\frac{1ft^2}{144in^2}\right) = 0.581klf$$

3.4.2.1.2.2 Slab Haunch Load

The slab haunch load accounts for the buildup of concrete between the top of the girder and the bottom of the main slab. This concrete element has a width equal to the top flange width (W_{tf}) and varies in depth along the length of the girder because of camber and variations in the roadway surface.

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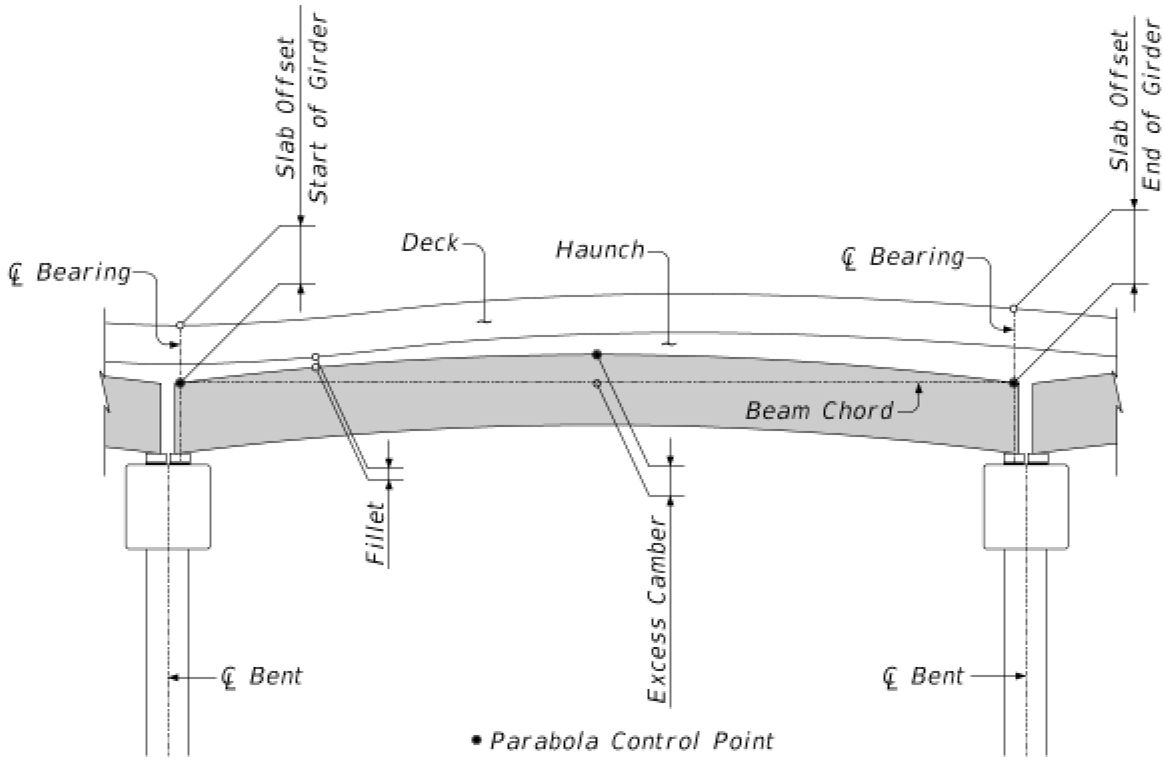

Figure 3-5: Slab Haunch

WSDOT's design policy is to assume the top of the girder is flat (no camber) for purposes of determining the slab haunch load (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

$$w_{haunch} = W_{tf} t_{haunch} \gamma_c$$

Assuming a slab offset ("A" dimension) of $12.25in$, the slab haunch load at the start of the span is

$$t_{haunch} = \mathrm{A} - t_{slab} = 12.25in - 7.5in = 4.75in$$

$$w_{haunch} = (49in)(4.75in)(0.155kcf)\left(\frac{1ft^2}{144in^2}\right) = 0.251\ klf$$

The vertical curve causes the haunch depth to vary along the length of the girder. The table below lists the haunch loading for half the span. This load is modelled as linear load segments.

Location	$t_{haunch}(in)$	$w_{haunch}(klf)$
0.0L	4.750	0.251
0.1L	4.624	0.244
0.2L	4.526	0.239

0.3L	4.456	0.235
0.4L	4.414	0.233
0.5L	4.400	0.232

3.4.2.2 *Superimposed Dead Loads*

Application of superimposed dead loads occurs after the deck has reached adequate strength. The superimposed dead loads consist of the traffic barrier and the overlay, if present. The composite section is resisting these loads.

3.4.2.2.1 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, $n = 3$.

$$2n = 6, N = 7, 2n < N$$

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

AASHTO permits equal distribution for barrier loads to all girders.

3.4.2.3 *Open to Traffic*

3.4.2.3.1 Future Overlay

Evenly distribute the weight of the future wearing surface to all girders. The curb to curb width of the deck is $38.833ft$.

$$w_o = \frac{(38.833ft)(0.035ksf)}{7\ girder} = 0.194\frac{klf}{girder}$$

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.

3.4.2.3.2 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.

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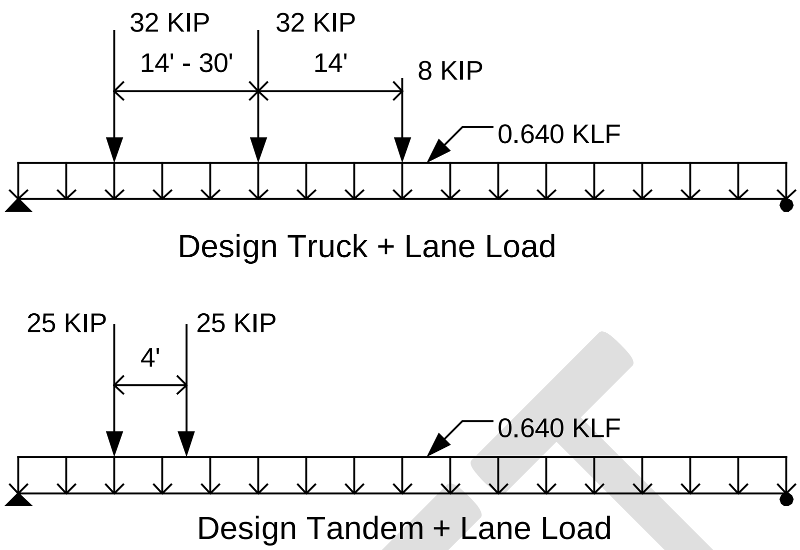

Figure 3-6: HL93 Live Load Model

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%.

3.4.3 Analysis Results Summary

3.4.3.1 *At Release*

Loading	Transfer Point	Harp Point	Mid-Span
Girder	$187.88\ k \cdot ft$	$1970.06\ k \cdot ft$	$2052.15\ k \cdot ft$

3.4.3.2 *At Bridge Site*

Loading	$0.5L_s$
Girder after erection	$1944.05\ k \cdot ft$
Diaphragm	$92.75\ k \cdot ft$
Slab	$1127.58\ k \cdot ft$
Haunch	$456.40\ k \cdot ft$
Traffic Barrier	$446.21\ k \cdot ft$
Future Overlay	$376.69\ k \cdot ft$
Design LLIM (HL-93)	$3851.68\ k \cdot ft$
Fatigue LLIM	$1962.42\ k \cdot ft$

Live loads are per lane

3.4.4 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

3.4.5 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 I-beam with cast-in-place concrete deck corresponds to cross section k.

WSDOT deviates from the LRFD BDS for exterior girders in type k 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)

$$n = \frac{5292.088ksi}{4266.223ksi} = 1.240$$

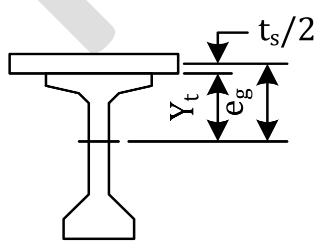

Figure 3-7: e_g Detail

$$e_g = Y_t + \frac{t_s}{2} = 34.196in + \frac{7.0in}{2} = 37.696in$$

$$K_g = 1.24[556339.2in^2 + (874.531in^2)(37.696in)^2] = 2230806in^4$$

3.4.5.1 *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).

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$$N_L = \left\lfloor \frac{38.833ft}{12ft} \right\rfloor = 3\ Design\ Lanes$$

3.4.5.2 *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.

3.4.5.2.1 Compute Distribution Factor for Moment

Check the range of applicability for live load distribution factors.

$3.5\ ft \le S \le 16\ ft$	$S = 6.0\ ft$	**OK**
$4.5\ in \le t_s \le 12\ in$	$t_s = 7.5\ in$	**OK**
$20\ ft \le L \le 240\ ft$	$L = 124.58\ ft$	**OK**
$N_b \ge 4$	$N_b = 7$	**OK**
$10{,}000 in^4 \le K_g \le 7{,}000{,}000\ in^4$	$K_g = 2230806\ in^4$	**OK**

3.4.5.2.1.1 One Design Lane Loaded

The live load distribution factor for one loaded lane is

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

$$gM_1^i = 0.06 + \left(\frac{6}{14}\right)^{0.4}\left(\frac{6}{124.58}\right)^{0.3}\left(\frac{2230806}{12.0 \cdot 124.58 \cdot 7^3}\right)^{0.1} = 0.392$$

3.4.5.2.1.2 Two or More Design Lanes Loaded

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

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

$$gM_{2+}^i = 0.075 + \left(\frac{6}{9.5}\right)^{0.6}\left(\frac{6}{124.58}\right)^{0.2}\left(\frac{2230806}{12.0 \cdot 124.58 \cdot 7^3}\right)^{0.1} = 0.554$$

3.4.5.3 *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.

3.4.5.3.1 Compute Distribution Factor for Shear

Check the range of applicability for live load distribution factors.

$3.5\ ft \le S \le 16\ ft$	$S = 6.0\ ft$	**OK**
$4.5\ in \le t_s \le 12\ in$	$t_s = 7.5\ in$	**OK**
$20\ ft \le L \le 240\ ft$	$L = 124.58\ ft$	**OK**
$N_b \ge 4$	$N_b = 7$	**OK**

3.4.5.3.1.1 One Design Lane Loaded

The live load distribution factor for one design lane loaded is

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

$$gV_1^i = 0.36 + \frac{6}{25.0} = 0.600$$

3.4.5.3.1.2 Two or More Design Lanes Loaded

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

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

$$gV_{2+}^i = 0.2 + \frac{6}{12} - \left(\frac{6}{35}\right)^{2.0} = 0.671$$

3.4.5.4 *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)	0.392	0.554	0.554
Shear (gV)	0.600	0.671	0.671

3.4.5.5 *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)	$0.392/1.2 = 0.327$
Shear (gV)	$0.600/1.2 = 0.500$

4 Flexure Design

WSDOT and local girder fabricators developed the design methodology used by PGSuper. The primary concept is to design for optimized fabrication of precast, prestressed concrete bridge girders. The primary goal is to determine the least required concrete strength at release and lifting while simultaneously placing the least possible demand on the stressing system and achieving adequate stability of the girder during handling operations. A detailed description and numerical example of optimized fabrication design is available in the PCI Journal[2]. Figure 4-1 gives a high-level summary of the design procedure. Girder stresses and stability at initial lifting and hauling are integral elements of the design process. Lifting and hauling conditions often govern the design.

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.

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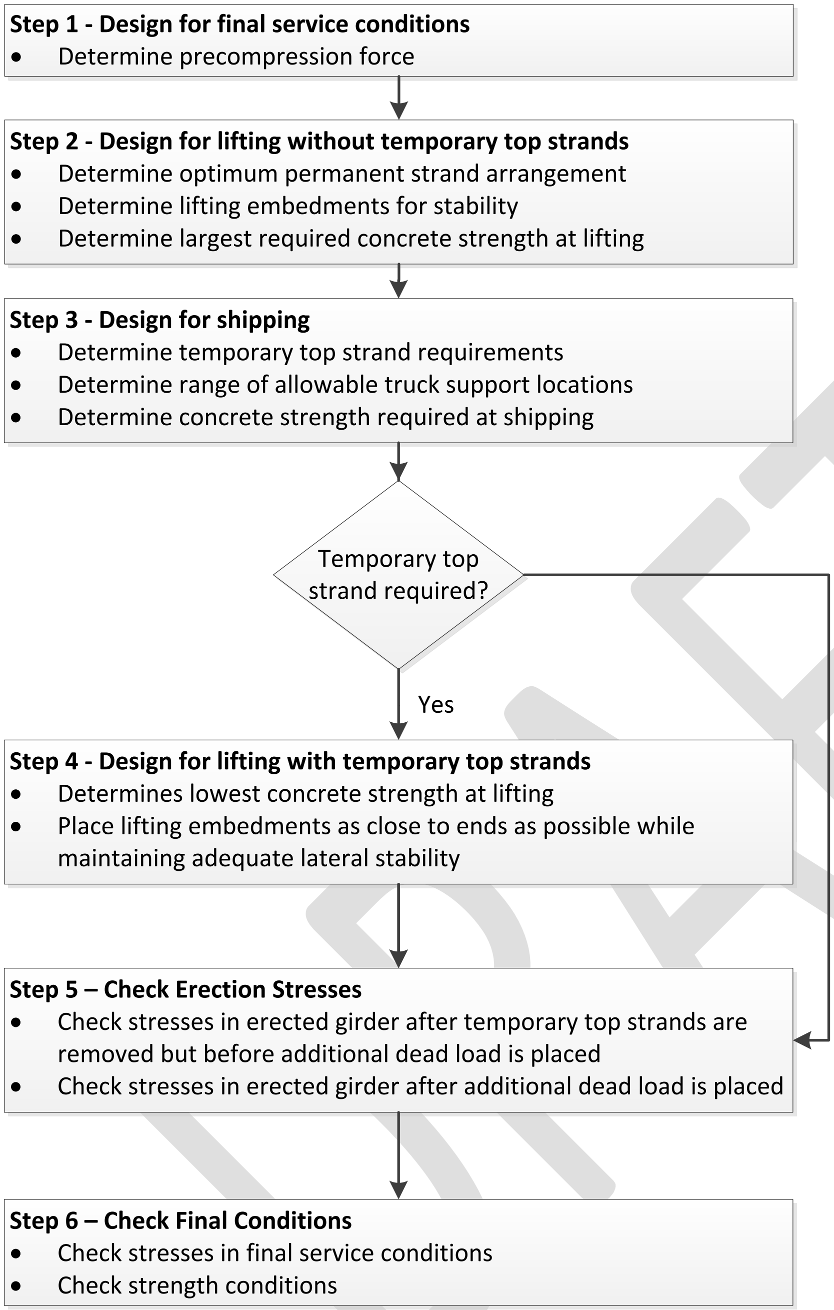

Figure 4-1: Optimized Fabrication Girder Design Procedure

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4.1 Step 1 Design for Final Service Conditions

Design for the Service III limit state for tensile stresses at the bottom of the girder at mid-span.

4.1.1 Stresses due to loads on non-composite section

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

$$S_t = -16268.9 in^3$$

$$S_b = 17493.0 in^3$$

Load	Moment (k-ft)	$f_t(ksi)$	$f_b(ksi)$
Girder	1944.05	-1.434	1.334
Diaphragms	92.75	-0.068	0.064
Slab	1127.58	-0.832	0.774
Haunch	456.40	-0.337	0.313

4.1.2 Stresses due to loads on the composite section

4.1.2.1 *Stress due to dead loads*

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

$$S_{tgc} = -42391.8 in^3$$

$$S_{bc} = 21726.3 in^3$$

Load	Moment (k-ft)	$f_t(ksi)$	$f_b(ksi)$
Barrier	446.21	-0.126	0.246
Future Overlay	376.69	-0.107	0.208

4.1.2.2 *Stress due to slab shrinkage*

Girder stresses must include those due to slab shrinkage elastic gains due to slab shrinkage are included in the effective prestress force.

$$f_{ss} = \frac{-\varepsilon_{ddf} A_d E_{cdeck}}{\left[1 + 0.7\psi_d(t_f, t_d)\right]}\left(\frac{1}{A_c} - \frac{e_d}{S}\right)$$

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

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

$$\frac{V}{S} = \frac{A}{P} = \frac{W_{trib} t_{gross\ slab\ depth}}{2W_{trib} - W_{tf}} = \frac{(72in)(7.5in)}{2(72in) - 49in} = 5.684in$$

Use the gross slab depth when computing slab shrinkage effects. Shrinkage is an early age effect; therefore, the sacrificial depth is part of the deck slab that is shrinking.

$$k_s = 1.45 - 0.13(5.684) = 0.711 < 1.0 \ \therefore 1.0$$

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

Slab concrete age at time of initial loading is $f'_{ci} = 0.8f'_c$. (LRFD 5.4.2.3.1)

$$f'_{ci} = 0.8f'_c = 0.8(4ksi) = 3.2\ ksi$$

$$k_f = \frac{5}{1 + f'_{ci}} = \frac{5}{1 + 3.2} = 1.19$$

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$$t = t_f - t_d = 2000 - 120 = 1880\ days$$

$$k_{td} = \frac{t}{12\left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20}\right) + t} = \frac{1880}{12\left(\frac{100 - 4(3.2)}{3.2 + 20}\right) + 1880} = 0.977$$

$K_{sh} = 0.5$ (BDM 5.1.4.3.D – use 50% slab shrinkage strain)

$$\varepsilon_{ddf} = (0.5)(1.0)(0.95)(1.19)(0.978)(0.48 \times 10^{-3}) = 0.265 \times 10^{-3}$$

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

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

$$t_i = 1\ days$$

$$\psi_d(t_f, t_d) = 1.9(1.0)(0.96)(1.19)(0.978)(1)^{-0.118} = 2.12$$

$$A_d = (72in)(7.5in) = 540in^2$$

$$A_c = 1274.618in^2$$

$$e_d = Y_{tc} + \frac{t_{gross\ slab\ depth}}{2} = 22.239in + \frac{7.5in}{2} = 25.989in$$

$$S_{tgc} = -42391.8in^3$$

$$S_{bc} = 21726.3in^3$$

$$f_{top} = \frac{(-0.265 \times 10^{-3})(540in^2)(4266.223ksi)}{[1 + 0.7(2.12)]}\left(\frac{1}{1280.808in^2} - \frac{25.989in}{-42391.8in^3}\right) = -0.343\ ksi$$

$$f_{bot} = \frac{(-0.265 \times 10^{-3})(540in^2)(4266.223ksi)}{[1 + 0.7(2.12)]}\left(\frac{1}{1280.808in^2} - \frac{25.989in}{21726.3in^3}\right) = 0.102\ ksi$$

4.1.2.3 *Live Load Stresses*

The live load moments computed above are for a full lane of load. These values must be scaled with the live load distribution factor to get a per girder moment

$$f = \frac{gM_{LLIM}}{S}$$

$$S_{tgc} = -42391.8in^3$$

$$S_{bc} = 21726.3in^3$$

Load	Moment (k-ft per lane)	gM	Moment (k-ft per girder)	$f_t(ksi)$ Min	$f_t(ksi)$ Max	$f_b(ksi)$ Min	$f_b(ksi)$ Max
Design (HL93)	3851.68	0.555	2138.24	-0.598	0	0	1.178
Fatigue	1962.42	0.327	642.37	-0.179	0	0	0.352

Typically, the governing limit state for final service conditions is Service III

$$Service\ III = 1.0DC + 1.0DW + 0.8(LL + IM)$$

$$f_b = 1.334ksi(girder) + 0.064ksi(diaphragm) + 0.774ksi(slab) + 0.313ksi(haunch) + 0.246ksi(barrier) + 0.208ksi(future\ overlay) + 0.102ksi(slab\ shrinkage) + (0.8)(1.178ksi)(Design\ LLIM) = 3.983ksi$$

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The stress due to prestressing is

$$f_{ps} = \frac{P}{A} + \frac{Pe}{S}$$

The tension limit is 0.0 ksi per BDM 5.2.1.C.

$$f_b + f_{ps} = 0.0\ ksi$$

For load rating, use the tension stress limit from the LRFD Bridge Design Specifications (BDM 13.2.4)

Assume the resultant prestress force is 3” above the bottom of the girder.

$$e = Y_b - 3in = 31.804in - 3in = 28.804in$$

$$3.983ksi + \frac{P}{874.531in^2} + \frac{P(28.804in)}{17493.0in^3} = 0.0ksi$$

$$P = -1427.56kip$$

Assume a final effective prestress of 85% of the initial prestress

$$P = N(0.217in^2)(0.85)(0.75f_{pu}) = 1427.56kip$$

$$1427.56kip = N(0.217in^2)(0.85)(0.75)(270ksi)$$

$$N = 38.2$$

Use 38 strands.

$$P = 38(0.217in^2)(0.85)(0.75)(270ksi) = 1419.3\ kip$$

4.1.3 Check Estimate of Final Concrete Strength

Compression at the top of girder, Service I limit state is limited to $0.6f'_c$.

Service I limit state for compression stresses at the top of the girder at mid-span.

$$Service\ I = 1.0DC + 1.0DW + 1.0(LL + IM)$$

$$f_t = -1.434ksi(girder) - 0.068ksi(diaphragm) - 0.832ksi(slab) - 0.337ksi(haunch) - 0.126ksi(barrier) - 0.107ksi(futre\ overlay) - 0.343ksi(slab\ shrinkage) - 0.598ksi(Design\ LLIM) = -3.845\ ksi$$

The stress due to prestressing is

$$f_{ps} = \frac{P}{A} + \frac{Pe}{S}$$

$$f_{ps} = \frac{-1419.3kip}{874.531in^2} + \frac{(-1419.3kip)(28.804in)}{-16268.9in^3} = 0890ksi$$

$$-0.6f'_c = 0.890ksi - 3.845ksi = -2.955ksi$$

$$f'_c = 4.93ksi$$

Compression at the top of girder, Fatigue I limit state. Stress is limited to $0.4f'_c$..

Fatigue I limit state for compression stresses at the top of the girder at mid-span.

$$Fatigue\ I = 0.5DC + 0.5DW + 1.5(LL + IM)$$

DRAFT

$$f_t = 0.5(-1.434ksi(girder) - 0.068ksi(diaphragm) - 0.832ksi(slab) - 0.337ksi(haunch) - 0.126ksi(barrier) - 0.107ksi(future\ overlay) - 0.343ksi(slab\ shrinkage)) + 1.5(-0.179ksi)(Fatigue\ LLIM) = -1.847ksi$$

The stress due to prestressing is

$$f_{ps} = \frac{P}{A} + \frac{Pe}{S}$$

$$f_{ps} = \frac{-1419.3kip}{874.531in^2} + \frac{(-1419.3kip)(28.804in)}{-16268.9in^3} = 0.890ksi$$

$$-0.4f'_c = 0.5(0.890)ksi - 1.847ksi = -1.402ksi$$

$$f'_c = 3.51ksi$$

The required concrete strength does not exceed our assumed value. **OK**

4.2 Step 2 - Design for Lifting without Temporary Top Strands

Temporary top strands are generally not required for lifting of most girders. This step identifies the lifting locations that provide adequate lateral stability and the corresponding concrete strength. The optimum strand arrangement is also determined.

Assume lifting devices are located 3 ft from the ends of the girder (BDM 5.6.2.C and Standard Specifications 6-02.3(25)).

4.2.1 Proportion Strands

The optimum exit location for the permanent pretensioning results in the compressive stress at the transfer point or lift point being approximately equal to the compressive stress at the harp point. This provides the lowest exit eccentricity that does not increase the required concrete strength at lifting. Finding this location is an iterative procedure. Experience has shown that a good starting point is a ratio of straight-to-harped strands of approximately 2:1. Manipulation of the harped strands varies the exit eccentricity.

The harped strand exit location can be manipulated in one of two ways: 1) by lowering a fixed number of harped strands at the ends or, 2) by leaving the harped strands at their highest exit location, and dropping pairs of strands from the harped strand group into the straight strand group.

The optimal arrangement of strands for this girder is 30 harped stands with an eccentricity of $28.870\ in$ and 8 harped strands with end eccentricity of $-27.196\ in$ and harp point eccentricity of $27.804\ in$. The permanent strand eccentricity at 0.5L is $28.646\ in$.

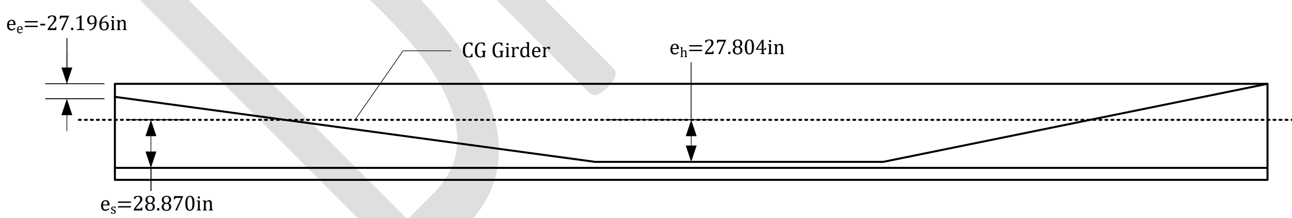

Figure 4-2: Optimum Strand Arrangement

4.2.2 Prestress losses

4.2.2.1 *Initial relaxation before transfer*

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 a 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.

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

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

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

$$t = 1\ day$$

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

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.

4.2.2.2 *Elastic Shortening*

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

$$f_{cgp} = \frac{P}{A} + \frac{Pe^2}{I} - \frac{M_g e}{I}$$

$$P = N(0.217in^2)\left(f_{pj} - \Delta f_{pR0} - \Delta f_{pES}\right)$$

Solve this equation iteratively for P.

Assume 9% initial loss and $f'_{ci} = 5.1ksi$

$$E_{ci} = 120000(1.0)(0.155)^2(5.1)^{0.33} = 4935.632\ ksi$$

$$P = 38(0.217)(1 - 0.09)(202.5) = 1519.5\ kip$$

$$f_{cgp} = \frac{1519.5kip}{874.531in^2} + \frac{(1519.5kip)(28.646in)^2}{556339.2in^4} - \frac{(2052.15k \cdot ft)\left(\frac{12in}{1ft}\right)(28.646in)}{556339.2in^4} = 2.711ksi$$

$$\Delta f_{pES} = \frac{28500ksi}{4935.632ksi}(2.711ksi) = 15.654ksi$$

$$P = (38)(0.217in^2)(202.5ksi - 1.98ksi - 15.654ksi) = 1524.4\ kip$$

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	1532.20 kip
HP	1521.73 kip
0.5Lg	1523.87 kip

4.2.3 Check girder stability

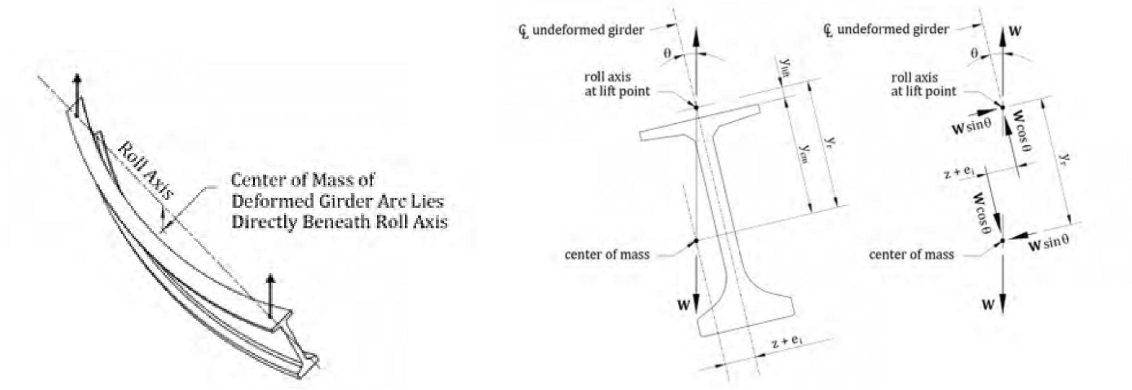

Figure 4-3: Equilibrium of Hanging Girder

4.2.3.1 *Vertical Location of Center of Gravity*

4.2.3.1.1 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.

4.2.3.1.1.1 Girder

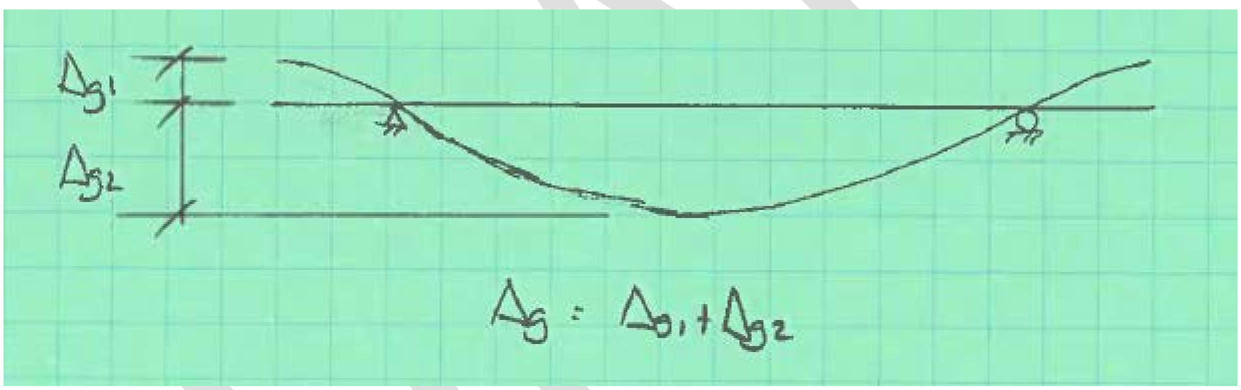

Figure 4-4: Girder Self-Weight Deflection during Lifting

$$L_s = L_g - 2a = 127.997ft - 2(3ft) = 121.997ft$$

At girder ends

$$\Delta_{g1} = \frac{w_g a}{24E_{ci}I_x}\left[3a^2(a + 2L_s) - L_s^3\right]$$
$$= \frac{(-1.002klf)(3ft)}{24(4935.632ksi)(556339.2in^4)}\left[3(3ft)^2\big(3ft + 2(121.997ft)\big) - (121.997ft)^3\right]\left(\frac{1728in^3}{1ft^3}\right)$$
$$= 0.141\ in$$

Mid-span

$$\Delta_{g2} = \frac{5w_g L_s^4}{384E_{ci}I_x} - \frac{w_g a^2 L_s^2}{16E_{ci}I_x} = \left[\frac{5(-1.002klf)(121.997ft)^4}{384(4935.632ksi)(556339.2in^4)} - \frac{(-1.002klf)(3ft)^2(121.997ft)^2}{16(4935.632ksi)(556339.2in^4)}\right]\left(\frac{1728in^3}{1ft^3}\right)$$
$$= -1.819in + 0.005in = -1.813in$$

4.2.3.1.1.2 Prestressing

4.2.3.1.1.2.1 Straight Strands

$$P = \left(\frac{30}{38}\right)(1523.87kip) = 1203.055kip$$

$$\Delta_{ss} = \frac{PeL^2}{8E_{ci}I_x} = \frac{(1203.055kip)(28.870in)(127.997ft)^2}{8(4935.632ksi)(556339.2in^4)}\left(\frac{144in^2}{1ft^2}\right) = 3.730in$$

4.2.3.1.1.2.2 Harped Strands

$$P = \left(\frac{8}{38}\right)(1523.87kip) = 320.814\,kip$$

$$e' = e_{hp} - e_e = 27.804in - (-27.196in)) = 55\,in$$

$$b = 0.4$$

$$N = \frac{Pe'}{bL} = \frac{(320.814kip)(55in)}{(0.4)(127.997ft)}\left(\frac{1ft}{12in}\right) = 28.719kip$$

$$\Delta_{hs} = \frac{b(3-4b^2)NL^3}{24E_{ci}I_x} + \frac{Pe_eL^2}{8E_{ci}I_x}$$

$$= \frac{0.4(3-4(0.4)^2)(28.719kip)(127.997ft)^3}{24(4935.632ksi)(556339.2in^4)}\left(\frac{1728in^3}{1ft^3}\right)$$

$$+ \frac{(320.814kip)(-27.196in)(127.997ft)^2}{8(4935.632ksi)(556339.2in^4)}\left(\frac{144in^2}{1ft^2}\right) = 1.491in - 0.937in = 0.554in$$

4.2.3.1.1.3 Initial Camber

$$\Delta_{ps} = \Delta_{ss} + \Delta_{hs} = 3.730in + 0.554in = 4.284in$$

$$\Delta_{camber} = \Delta_{g1} - \Delta_{g2} + \Delta_{ps} = -1.813in - 0.141in + 4.284in = 2.330in$$

4.2.3.1.2 Offset factor

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

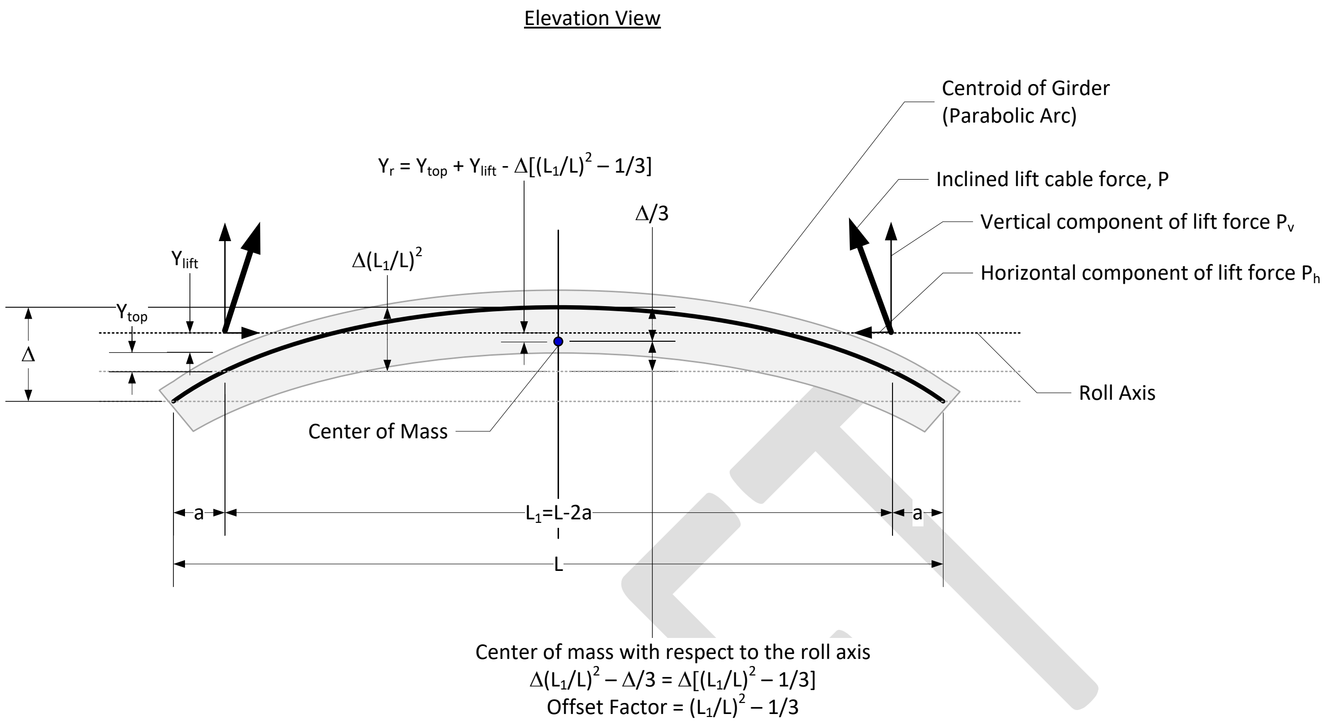

Figure 4-5: Offset Factor

$$F_o = \left(\frac{L_s}{L_g}\right)^2 - \frac{1}{3} = \left(\frac{121.997ft}{127.997ft}\right)^2 - \frac{1}{3} = 0.575$$

4.2.3.1.3 Location the roll axis above the top of girder

$$y_{rc} = 0in$$

4.2.3.1.4 Location of CG below roll axis

$$y_r = Y_{top} - F_o(\Delta_{camber}) + y_{rc} = 34.196in - (0.575)(2.080in) - 0in = 33.0in$$

4.2.3.2 *Lateral Deflection Parameters*

4.2.3.2.1 Lateral Sweep

Sweep tolerance is 1/8” per 10 ft

$$e_{sweep} = \frac{127.997ft}{10ft}\left(\frac{1}{8}in\right) = 1.6in$$

4.2.3.2.2 Initial Lateral Eccentricity

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

$$e_{lift} = 0.25in$$

$$e_i = F_o e_{sweep} + e_{lift} = (0.575)(1.6in) + 0.25in = 1.17in$$

4.2.3.2.3 Lateral Deflection of CG

Lateral deflection of center of gravity due to total girder weight applied to weak axis

$$W_g = w_g L_g = (1.002klf)(127.997ft) = 128.253kip$$

$$a = 3ft$$

$$L_s = L_g - 2a = 127.997ft - 2(3ft) = 121.997ft$$

$$z_o = \left(\frac{W_g}{12E_{ci}I_{yy}L_g^2}\right)\left(\frac{L_s^5}{10} - a^2L_s^3 + 3a^4L_s + \frac{6a^5}{5}\right)$$
$$= \left(\frac{128.253kip}{12(4935.632ksi)(71865.3in^4)(127.997ft)^2}\right)\left(\frac{(121.997ft)^5}{10} - (3ft)^2(121.997ft)^3 + 3(3ft)^4(121.997ft) + \frac{6}{5}(3ft)^5\right)\left(\frac{1728in^3}{1ft^3}\right) = 8.537\ in$$

4.2.3.3 *Equilibrium tilt angle*

$$\theta_{eq} = \frac{e_i}{y_r - z_o} = \frac{1.17in}{32.856in - 8.537in} = 0.04809\ rad$$

4.2.3.4 *Girder Stresses in Hanging Girder*

4.2.3.4.1 Direct stress at Prestress Transfer Point and Harp Point

4.2.3.4.1.1 Prestressing

$$f_{ps} = \frac{P}{A} + \frac{Pe}{S}$$

From PGSuper, the effective prestress force at the prestress transfer is $P = 1209.63\ kip$ straight strands and $P = 322.57kip$ harped strands. The strand eccentricities are $28.870in$ and $-23.974in$.

$$f_t = \frac{-(1209.63kip + 322.57kip)}{874.531in^2} + \frac{(-1209.63kip)(28.870in) + (-322.57kip)(-23.974in)}{-16268.9in^3} = -0.081ksi$$

$$f_b = \frac{-(1209.63kip + 322.57kip)}{874.531in^2} + \frac{(-1209.63kip)(28.870in) + (-322.57kip)(-23.974in)}{17493.0in^3} = -3.306ksi$$

From PGSuper, the effective prestress force at the harp point is $P = 1201.52\ kip$ straight strands and $P = 320.40kip$ harped strands. The strand eccentricities are 28.870in and 27.804in

$$f_t = \frac{-(1201.52kip + 320.40kip)}{874.531in^2} + \frac{(-1201.52kip)(28.870in) + (-320.40kip)(27.804in)}{-16268.9in^3} = 0.939ksi$$

$$f_b = \frac{-(1201.52kip + 320.40kip)}{874.531in^2} + \frac{(-1201.52kip)(28.870in) + (-320.40kip)(27.804in)}{17493.0in^3} = -4.232ksi$$

4.2.3.4.1.2 Girder self-weight

$$M_g = \frac{w_g}{2}(L_s x - x^2 - a^2)$$

At Transfer point

$$x = l_t - a = 3ft - 3ft = 0.0ft$$

$$M_g = \frac{(1.002klf)}{2}\left((121.997ft)(0ft) - (0ft)^2 - (3ft)^2\right) = -4.509k \cdot ft$$

$$f_t = \frac{-4.509k \cdot ft}{-16268.9in^3}\left(\frac{12in}{1ft}\right) = 0.003ksi$$

$$f_b = \frac{-4.509k \cdot ft}{17493.0in^3}\left(\frac{12in}{1ft}\right) = -0.003ksi$$

At Harp Point

$$x = 0.4L_g - a = 0.4(127.997ft) - 3ft = 48.199ft$$

$$M_g = \frac{(1.002klf)}{2}\left((121.997ft)(48.199ft) - (48.199ft)^2 - (3ft)^2\right) = 1777.67k \cdot ft$$

$$f_t = \frac{1777.67k \cdot ft}{-16268.9in^3}\left(\frac{12in}{1ft}\right) = -1.311ksi$$

$$f_b = \frac{1777.67k \cdot ft}{17493.0in^3}\left(\frac{12in}{1ft}\right) = 1.219ksi$$

4.2.3.4.2 Tilt induced stresses

Top left flange tip at Transfer Point

$$f_{tl} = \frac{M_g W_{tf}}{2I_{yy}}\theta_{eq}$$

$$f_{tl} = \frac{(-4.509k \cdot ft)(49in)}{2(71865.3in^4)}(0.04809rad)\left(\frac{12in}{1ft}\right) = 0.001ksi$$

Bottom right flange tip at Transfer Point

$$f_{br} = -\frac{M_g W_{bf}}{2I_{yy}}\theta_{eq}$$

$$f_{br} = -\frac{(-4.509k \cdot ft)(38.375in)}{2(71865.3in^4)}(0.04809rad)\left(\frac{12in}{1ft}\right) = -0.001ksi$$

Top left flange tip at Harp Point

$$f_{tl} = \frac{M_g W_{tf}}{2I_{yy}}\theta_{eq}$$

$$f_{tl} = \frac{(1777.67k \cdot ft)(49in)}{2(71865.3in^4)}(0.04809rad)\left(\frac{12in}{1ft}\right) = 0.349ksi$$

Bottom right flange tip at Harp Point

$$f_{br} = -\frac{M_g W_{bf}}{2I_{yy}}\theta_{eq}$$

$$f_{br} = -\frac{(1777.67k \cdot ft)(38.375in)}{2(71865.3in^4)}(0.04809rad)\left(\frac{12in}{1ft}\right) = -0.274ksi$$

4.2.3.4.3 Total stress

Top left flange tip at Transfer Point

$$f_{tl} = -0.081ksi + 0.003ksi + 0.001ksi = -0.078ksi$$

Bottom left flange tip at Transfer Point

$$f_{br} = -3.306ksi - 0.003ksi - 0.001ksi = -3.310ksi$$

Top left flange tip at Harp Point

$$f_{tl} = 0.939ksi - 1.311ksi + 0.349ksi = -0.024ksi$$

Bottom left flange tip at Harp Point

$$f_{br} = -4.323ksi + 1.219ksi - 0.274ksi = -3.285ksi$$

4.2.3.5 *Factor of Safety Against Cracking*

Lateral cracking moment

$$M_{cr} = \frac{(f_r - f_{direct})2I_{yy}}{W_{top}}$$

Tilt angle at first crack

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

Cracking moment at Transfer Point

$$f_{direct} = f_{ps} + f_g = -0.081ksi + 0.003ksi = -0.077ksi$$

$$M_{cr} = \frac{\big(0.547ksi - (-0.077ksi)\big)2(71865.3in^4)}{49in}\left(\frac{1in}{12ft}\right) = 152.7\,k \cdot ft$$

Tilt angle at first crack at Transfer Point

$$\theta_{cr} = \frac{152.7k \cdot ft}{4.51k \cdot ft} = 33.8\,rad \therefore 0.4\,rad$$

Factor of Safety against Cracking at Transfer Point

$$FS_{cr} = \frac{y_r\theta_{cr}}{e_i + z_o\theta_{cr}} = \frac{(32.856in)(0.4)}{1.17in + (8.537in)(0.4)} = 2.867$$

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

Cracking moment at Harp Point

$$M_{cr} = \frac{\big(0.547ksi - (0.939ksi - 1.311ksi)\big)2(71865.3in^4)}{49in}\left(\frac{1in}{12ft}\right) = 224.64\,k \cdot ft$$

Tilt angle at first crack at Harp Point

$$\theta_{cr} = \frac{224.64k \cdot ft}{1777.67k \cdot ft} = 0.126rad$$

Factor of Safety against Cracking at Harp Point

$$FS_{cr} = \frac{y_r\theta_{cr}}{e_i + z_o\theta_{cr}} = \frac{(32.856in)(0.126)}{1.17in + (8.537in)(0.126)} = 1.842$$

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

4.2.3.6 *Factor of Safety against Failure*

$$\theta_{max} = \sqrt{\frac{e_i}{2.5z_o}} \leq 0.4\,rad = \sqrt{\frac{1.17}{2.5(8.537)}} = 0.234\,rad$$

$$FS_f = \frac{y_r\theta_{max}}{e_i + (1 + 2.5\theta_{max})(z_o\theta_{max})} = \frac{(32.856in)(0.234)}{1.17in + \big(1 + 2.5(0.234)\big)(8.537in)(0.234)} = 1.773$$

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

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$$FS_f = 1.841$$

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

4.2.4 Determine Concrete Strength at Lifting

For the general stress case, compression stress is limited to $-0.65f'_{ci}$.

Bottom centerline at point of prestress transfer $-0.65f'_{ci} = -3.309ksi, f'_{ci} = 5.1\ ksi$

Bottom centerline at harp point $-0.65f'_{ci} = -3.104ksi, f'_{ci} = 4.8\ ksi$

Peak compression stress at the extremities of the section are limited to $-0.7f'_{ci}$

Bottom left flange tip at point of prestress transfer $-0.70f'_{ci} = -3.310ksi, f'_{ci} = 4.8\ ksi$

Bottom left flange time at harp point $-0.70f'_{ci} = -3.285ksi, f'_{ci} = 4.7\ ksi$

The controlling initial strength matches our original assumption.

4.3 Step 3 - Design for Shipping

Assume temporary top strands are not required for shipping

Girders can be shipped as early as 10 days and as late as 90 days. The effective prestress force and camber can vary significantly in this time range. Perform this analysis with the effective prestress force at 10 days and the camber at 90 days (BDM 5.6.3.D.6).

4.3.1 Estimate Prestress Losses at Shipping

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

4.3.1.1 *Shrinkage of Girder Concrete*

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

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

$$K_{ih} = \frac{1}{1 + \frac{E_p}{E_{ci}}\frac{A_{ps}}{A_g}\left(1 + \frac{A_g e^2}{I_g}\right)\left[1 + 0.7\psi_b(t_f, t_i)\right]}$$

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

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

$$\frac{V}{S} = \frac{AL_g}{PL_g + 2A} = \frac{(874.531in^2)(127.997ft)(\frac{12in}{1ft})}{(273.284in)(127.997ft)\left(\frac{12in}{1ft}\right) + 2(874.531in^2)} = 3.187in$$

$$k_s = 1.45 - 0.13(3.187) = 1.04$$

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

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

$$k_f = \frac{5}{1 + f'_{ci}} = \frac{5}{1 + 5.1} = 0.82$$

$$k_{td}(t = 9days) = \frac{9}{12\left(\frac{100 - 4(5.1)}{5.1 + 20}\right) + 9} = 0.191$$

$$k_{td}(t = 1999days) = \frac{1999}{12\left(\frac{100 - 4(5.1)}{5.1 + 20}\right) + 1999} = 0.981$$

$$\psi_b(t_f, t_i) = 1.9(1.04)(0.96)(0.82)(0.981)(1)^{-0.118} = 1.52$$

$$\varepsilon_{bih} = (1.04)(0.95)(0.82)(0.191)(0.48 \times 10^{-3}) = 0.000074$$

$$A_{ps} = 38(0.217in^2) = 8.246in^2$$

$$K_{ih} = \frac{1}{1 + \frac{28500ksi}{4935.632ksi}\frac{8.246in^2}{874.532in^3}\left(1 + \frac{874.532in^2(28.646in)^2}{556339.2in^4}\right)[1 + 0.7(1.52)]} = 0.795$$

$$\Delta f_{pSRH} = (0.000074)(28500ksi)(0.795) = 1.677ksi$$

4.3.1.2 *Creep of Girder Concrete*

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

$$\psi_b(t_h, t_i) = 1.9(1.04)(0.96)(0.82)(0.191)(1)^{-0.118} = 0.297$$

$$\Delta f_{CRH} = \frac{28500ksi}{4935.632ksi}(2.767ksi)(0.297)(0.795) = 3.773ksi$$

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.

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

$$K_L' = 45$$

$$f_{pt} = \frac{1523.87kip}{8.246in^2} = 184.802ksi$$

$$\Delta f_{pR1H} = \left[\frac{184.802ksi}{45}\frac{\log(24 \cdot 10)}{\log(24 \cdot 1)}\left(\frac{184.802ksi}{243ksi} - 0.55\right)\right]\left[1 - \frac{3(1.667ksi + 3.506ksi)}{184.802ksi}\right](0.795) = 1.080\ ksi$$

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 Shipping*

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

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

$$\Delta f_{pLTH} = 1.677ksi + 3.773ksi + 1.080ksi = 6.530ksi$$

$$\Delta_{pH} = 1.98ksi + 15.642ksi + 6.530ksi = 24.152ksi$$

4.3.2 Check Girder Stability

Assume bunk points at 5.5 ft (H away from the ends of the girder) and hauling will occur with the HT40-72 haul truck configuration. This is the least stiff hauling configuration in the region.

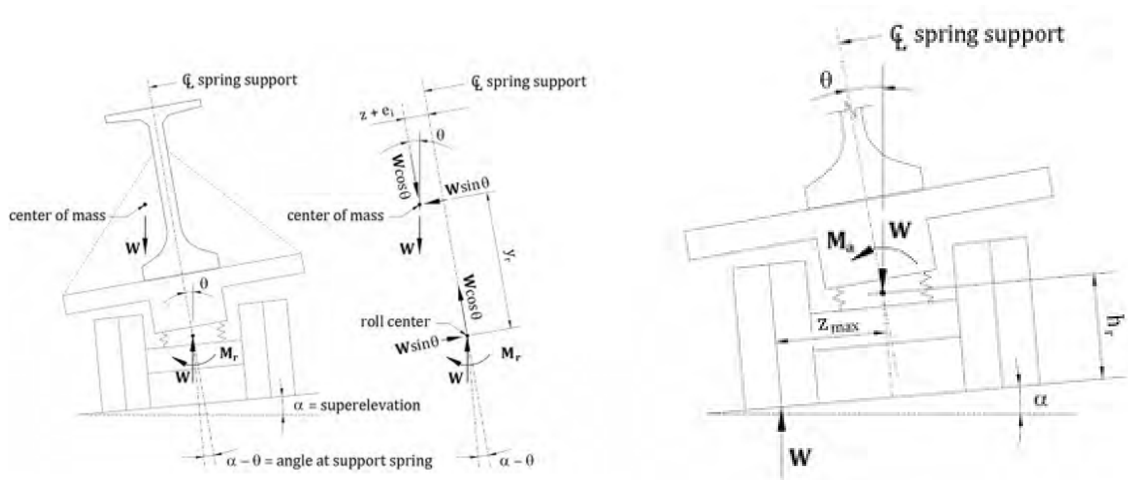

Figure 4-6: Equilibrium during Hauling

4.3.2.1 *Stability Analysis Parameters*

Parameter	Value
Rotational Stiffness	$K_\theta = 40000 \dfrac{k \cdot in}{rad}$
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	$f_r = 0.24\lambda\sqrt{f_c'} = (0.24)(1.0)\sqrt{6.3ksi} = 0.602ksi$

4.3.2.2 *Vertical Location of Center of Gravity*

4.3.2.2.1 Camber at Hauling

Assume girder transportation occurs as late as possible to maximum 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

$$\Delta_{ps} = 4.284in$$

Changing the datum to the storage support location

$$\Delta_{ps1} = 4.076in \text{ at mid-span}$$

$$\Delta_{ps2} = -0.208in \text{ at girder end}$$

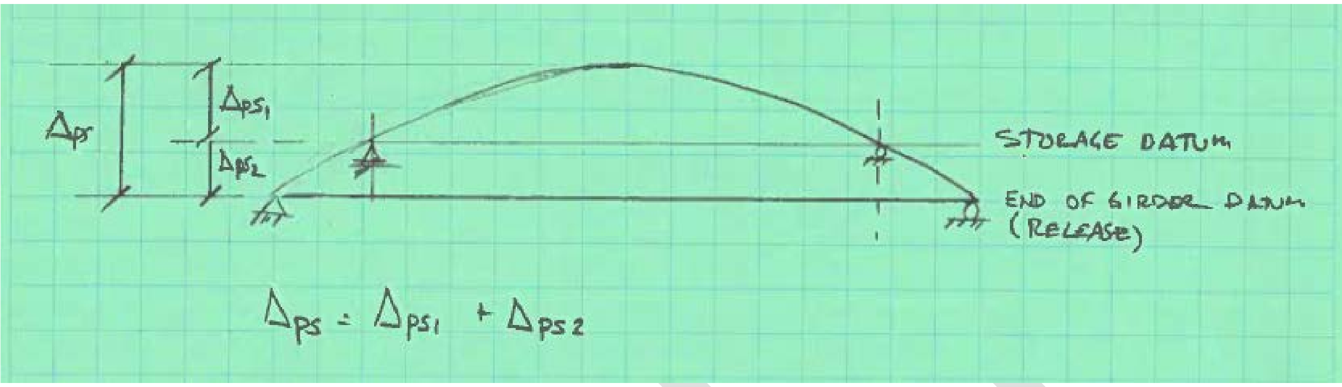

Figure 4-7: Prestress induced Deflection based on Storage Datum

The dead load deflection at mid-span during storage is

$$L_s = L_g - 2a = 127.997ft - 2(1.708ft) = 124.58ft$$

The dead load deflection at the girder ends during storage is

$$\Delta_{g1} = \frac{w_g a}{24E_{ci}I_x}[3a^2(a + 2L_s) - L_s^3]$$
$$= \frac{(-1.002klf)(1.708ft)}{24(4935.632ksi)(556339.2in^4)}[3(1.708ft)^2(1.708ft + 2(124.58ft)) - (124.58ft)^3]\left(\frac{1728in^3}{1ft^3}\right)$$
$$= 0.087\ in$$

$$\Delta_{g2} = \frac{5w_g L_s^4}{384E_{ci}I_x} - \frac{w_g a^2 L_s^2}{16E_{ci}I_x} = \left[\frac{5(-1.002klf)(124.58ft)^4}{384(4935.632ksi)(556339.2in^4)} - \frac{(-1.002klf)(1.708ft)^2(124.58ft)^2}{16(4935.632ksi)(556339.2in^4)}\right]\left(\frac{1728in^3}{1ft^3}\right)$$
$$= -1.978in + 0.002in = -1.976in$$

Creep deflection during storage is

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

$$k_{td}(t = 89days) = \frac{89}{12\left(\frac{100 - 4(5.1)}{5.1 + 20}\right) + 89} = 0.700$$

$$\psi_b(t_h, t_i) = 1.9(1.04)(0.96)(0.82)(0.700)(1)^{-0.118} = 1.088$$

At mid-span

$$\Delta_{creep} = (1.088)(4.076in - 1.976in) = 2.285in$$

At end of girder

$$\Delta_{creep} = (1.088)(-0.208in + 0.087in) = -0.132in$$

Girder deflection in the hauling configuration

$$L_s = 127.997ft - 2(5.5ft) = 116.997ft$$

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Mid-span deflection

$$\Delta_g = \frac{5w_g L_s^4}{384E_c I_x} - \frac{w_g a^2 L_s^2}{16E_c I_x} = \left[\frac{5(-1.002klf)(116.997ft)^4}{384(5292.088ksi)(556339.2in^4)} - \frac{(-1.002klf)(5.5ft)^2(116.997ft)^2}{16(5292.008ksi)(556339.2in^4)}\right]\left(\frac{1728in^3}{1ft^3}\right)$$
$$= -1.435in + 0.015in = -1.420in$$

Deflection at girder ends

$$\Delta_g = \frac{w_g a}{24E_c I_x}\left[3a^2(a + 2L_s) - L_s^3\right]$$
$$= \frac{(-1.002klf)(5.5ft)}{24(5292.088ksi)(556339.2in^4)}\left[3(5.5ft)^2\left(5.5ft + 2(116.997ft)\right) - (116.997ft)^3\right]\left(\frac{1728in^3}{1ft^3}\right)$$
$$= 0.199\ in$$

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

$$\Delta_{camber} = \left(\Delta_g + \Delta_{ps} + \Delta_{creep}\right)_{mid-span} - \left(\Delta_g + \Delta_{ps} + \Delta_{creep}\right)_{end}$$
$$= (-1.420in + 4.076in + 2.285in) - (0.199in - 0.208in - 0.132in) = 5.082in$$

4.3.2.2.2 Offset Factor

$$F_o = \left(\frac{L_s}{L_g}\right)^2 - \frac{1}{3} = \left(\frac{116.997ft}{127.997ft}\right)^2 - \frac{1}{3} = 0.502$$

4.3.2.2.3 Location of roll axis below top of girder

$$y_{rc} = H_{bg} + H_g - H_{rc} = 72.0in + 66.0in - 24.0in = 114.0in$$

4.3.2.2.4 Location of center of gravity above roll axis

$$y_r = y_{rc} - Y_{top} + F_o(\Delta_{camber}) = 114.0in - 34.196in + 0.502(5.082in) = 82.345in$$

4.3.2.3 *Lateral Deflection Parameters*

4.3.2.3.1 Lateral Sweep

Sweep tolerance = 1/8" per 10 ft

$$e_{sweep} = \left(\frac{127.997ft}{10ft}\right)\left(\frac{1}{8}in\right) = 1.600in$$

4.3.2.3.2 Initial Lateral Eccentricity

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

$$e_i = F_o e_{sweep} + e_{bunk} = (0.502)(1.600in) + 1.000in = 1.803in$$

4.3.2.3.3 Lateral Deflection of CG

Lateral deflection of center of gravity due to total weight of girder applied to the weak axis

$$z_o = \frac{W_g}{12E_c I_y L_g^2}\left(\frac{L_s^5}{10} - a^2 L_s^3 + 3a^4 L_s + \frac{6}{5}a^5\right)$$

$$z_o = \frac{128.253kip}{12(5292.088ksi)(71865.3in^4)(127.997ft)^2}\left(\frac{(116.997ft)^5}{10} - (5.5ft)^2(116.997ft)^3 + 3(5.5ft)^4(116.997ft) + \frac{6}{5}(5.5ft)^5\right)\left(\frac{1728in^3}{1ft^3}\right) = 6.355in$$

Impact up

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$$z_o = (0.8)(6.355in) = 5.084in$$

Impact down

$$z_o = (1.2)(6.355in) = 7.626in$$

4.3.2.3.4 Girder Stresses at Harping Point

4.3.2.3.4.1 Stress due to prestressing

$$f_t = \frac{-(1159.28kip + 309.14kip)}{874.531in^2} + \frac{(-1159.28kip)(28.870in) + (-309.14kip)(27.804in)}{-16268.9in^3} = 0.906ksi$$

$$f_b = \frac{-(1159.28kip + 309.14kip)}{874.531in^2} + \frac{(-1159.28kip)(28.870in) + (-309.14kip)(27.804in)}{17493.0in^3} = -4.083ksi$$

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

$$M_g = \frac{w_g}{2}(L_s x - x^2 - a^2)$$

$$x = 0.4L_g - a = 0.4(127.997ft) - 5.5ft = 45.699ft$$

$$M_g = \frac{1.002klf}{2}\left((116.997ft)(45.699ft) - (45.699ft)^2 - (5.5ft)^2\right) = 1617.23k \cdot ft$$

$$f_t = \frac{1617.23k \cdot ft}{-16268.9in^3}\left(\frac{12in}{1ft}\right) = -1.193ksi$$

$$f_b = \frac{1617.23k \cdot ft}{17493.0in^3}\left(\frac{12in}{1ft}\right) = 1.109ksi$$

4.3.2.4 *Analyze normal crown slope, no impact case*

4.3.2.4.1 Equilibrium tilt angle

$$\theta_{eq} = \frac{(K_\theta \alpha + (IM)W_g e_i)}{K_\theta - (IM)W_g(y_r + z_o)} = \frac{\left(\left(40000\frac{k \cdot in}{rad}\right)\left(0.02\frac{ft}{ft}\right) + (1.0)(128.253kip)(1.803in)\right)}{\left(40000\frac{k \cdot in}{rad}\right) - (1.0)(128.253kip)(82.129in + 6.355in)} = 0.036\ rad$$

4.3.2.4.2 Stress due to lateral loading from tilt

Top left flange tip

$$f_{tl} = \frac{(IM)(M_g)\theta_{eq}W_{top}}{2I_y} = \frac{(1.0)(1617.23k \cdot ft)(0.036)(49in)}{(2)(71865.3in^4)}\left(\frac{12in}{1ft}\right) = 0.238\ ksi$$

Bottom right flange tip

$$f_{br} = -\frac{(IM)(M_g)\theta_{eq}W_{bot}}{2I_y} = -\frac{(1.0)(1617.23k \cdot ft)(0.036)(38.375in)}{(2)(71865.3in^4)}\left(\frac{12in}{1ft}\right) = -0.187\ ksi$$

4.3.2.4.3 Factor of Safety against Cracking

Lateral cracking moment

$$f_{direct} = f_{ps} + (IM)f_g = 0.906ksi + (1.0)(-1.193ksi) = -0.282ksi$$

$$M_{cr} = \frac{(f_r - f_{direct})2I_y}{W_{top}} = \frac{\left(0.602ksi - (-0.282ksi)\right)(2)(71865.3in^4)}{49in}\left(\frac{1ft}{12in}\right) = 216.08k \cdot ft$$

Tilt angle at first crack

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

$$\theta_{cr} = \frac{216.08\ k\cdot ft}{1617.23\ k\cdot ft} = 0.1344\ rad$$

Factor of Safety against Cracking

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

$$FS_{cr} = \frac{\left(\left(40000\,\frac{k\cdot in}{rad}\right)\left(0.1344\ rad - 0.02\frac{ft}{ft}\right)\right)}{(1.0)(128.253kip)[(6.355in + 82.345in)(0.1344rad) + 1.803in]} = 2.605$$

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

4.3.2.4.4 Factor of Safety against Failure

$$\theta_{max} = \alpha + \sqrt{\alpha^2 + \frac{e_i + \left((IM)z_o + y_r\right)\alpha}{2.5(IM)z_o}} \leq 0.4\ rad$$

$$\theta_{max} = 0.02\frac{ft}{ft} + \sqrt{\left(0.02\ \frac{ft}{ft}\right)^2 + \frac{1.803in + \left((1.0)(6.355in) + (82.345in)\right)\left(0.02\frac{ft}{ft}\right)}{2.5(1.0)(6.355in)}} = 0.495\ rad \therefore 0.4\ rad$$

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

$$FS_f = \frac{\left(40000\,\frac{k\cdot in}{rad}\right)\left(0.4 - 0.02\frac{ft}{ft}\right)}{(1.0)(128.253kip)[(1.0)(6.355in)(0.4)\left(1 + 2.5(0.4)\right) + (82.345in)(0.4) + 1.803in} = 2.976$$

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

$$FS_f = 2.976$$

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

4.3.2.4.5 Factor of Safety against Rollover

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

$$\theta_{ro} = \frac{(1.0)(128.253kip)\left(\frac{72in}{2} - (24in)(0.02) - 1.0in\right)}{\left(40000\,\frac{k\cdot in}{rad}\right)} + 0.02 = 0.1339\ rad$$

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

$$FS_r = \frac{\left(40000\,\frac{k\cdot in}{rad}\right)(0.1339 - 0.02)}{(1.0)(128.253kip)\left[\left((6.355in)\left(1 + 2.5(0.1339)\right) + 82.345in\right)(0.1339) + 1.803in\right]} = 2.544$$

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

4.3.2.5 *Analyze normal crown slope, impact up*

4.3.2.5.1.1 Equilibrium tilt angle

$$\theta_{eq} = \frac{\left(K_\theta \alpha + (IM)W_g e_i\right)}{K_\theta - (IM)W_g(y_r + z_o)} = \frac{\left(\left(40000\frac{k\cdot in}{rad}\right)\left(0.02\frac{ft}{ft}\right) + (0.8)(128.253kip)(1.803in)\right)}{\left(40000\frac{k\cdot in}{rad}\right) - (0.8)(128.253kip)(82.345in + (0.8)(6.355in))} = 0.0317\ rad$$

4.3.2.5.1.2 Stress due to lateral loading from tilt

Top left flange tip

$$f_{tl} = \frac{(IM)\left(M_g\right)\theta_{eq}W_{top}}{2I_y} = \frac{(0.8)(1617.23k\cdot ft)(0.0317)(49in)}{(2)(71865.3in^4)}\left(\frac{12in}{1ft}\right) = 0.168\ ksi$$

Bottom right flange tip

$$f_{br} = -\frac{(IM)\left(M_g\right)\theta_{eq}W_{bot}}{2I_y} = -\frac{(0.8)(1617.23k\cdot ft)(0.0317)(38.375in)}{(2)(71865.3in^4)}\left(\frac{12in}{1ft}\right) = -0.132\ ksi$$

4.3.2.5.2 Factor of Safety against Cracking

Cracking moment

$$f_{direct} = f_{ps} + (IM)f_g = 0.906ksi + (0.8)(-1.193ksi) = -0.044ksi$$

$$M_{cr} = \frac{(f_r - f_{direct})2I_y}{W_{top}} = \frac{\left(0.602ksi - (-0.044ksi)\right)(2)(71865.3in^4)}{49in}\left(\frac{1ft}{12in}\right) = 159.9k\cdot ft$$

Tilt angle at first crack

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

$$\theta_{cr} = \frac{159.9\ k\cdot ft}{(0.8)1617.23\ k\cdot ft} = 0.123\ rad$$

Factor of Safety against Cracking

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

$$FS_{cr} = \frac{\left(\left(40000\ \frac{k\cdot in}{rad}\right)\left(0.123\ rad - 0.02\frac{ft}{ft}\right)\right)}{(0.8)(128.253kip)[(5.084in + 82.129in)(0.123rad) + 1.803in]} = 3.204$$

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

4.3.2.5.3 Factor of Safety against Failure

$$\theta_{max} = \alpha + \sqrt{\alpha^2 + \frac{e_i + \left((IM)z_o + y_r\right)\alpha}{2.5z_o}} \leq 0.4\ rad$$

$$\theta'_{max} = 0.02\frac{ft}{ft} + \sqrt{\left(0.02\ \frac{ft}{ft}\right)^2 + \frac{1.803in + \left((0.8)(6.355in) + 82.345in\right)0.02}{2.5(0.8)(6.355in)}} = 0.549\ rad \therefore 0.4\ rad$$

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

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$$FS_f = \frac{\left(40000\,\frac{k\cdot n}{rad}\right)\left(0.4 - 0.02\,\frac{ft}{ft}\right)}{(0.8)(128.253kip)[(0.8)(6.355in)(0.4)\left(1 + 2.5(0.4)\right) + (82.345in)(0.4) + 1.803in} = 3.817$$

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

4.3.2.5.4 Factor of Safety against Rollover

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

$$\theta_{ro} = \frac{(0.8)(128.253kip)\left(\frac{72in}{2} - (24in)(0.02)\right)}{\left(40000\,\frac{k-in}{rad}\right)} + 0.02 = 0.1111\,rad$$

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

$$FS_r = \frac{\left(40000\,\frac{k\cdot in}{rad}\right)(0.1111 - 0.02)}{(0.8)(128.253kip)\left[\left((0.8)(6.355in)\left(1 + 2.5(0.1111)\right) + 82.345in\right)(0.1111) + 1.803in\right]} = 3.042$$

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

4.3.2.6 *Analyze normal crown slope, impact down*

4.3.2.6.1 Equilibrium tilt angle

$$\theta_{eq} = \frac{\left(K_\theta\alpha + (IM)W_g e_i\right)}{K_\theta - (IM)W_g(y_r + (IM)z_o)} = \frac{\left(\left(40000\frac{k\cdot in}{rad}\right)\left(0.02\,\frac{ft}{ft}\right) + (1.2)(128.253kip)(1.803in)\right)}{\left(40000\frac{k\cdot in}{rad}\right) - (1.2)(128.253kip)(82.129in + (1.2)(6.355in))} = 0.0412\,rad$$

4.3.2.6.2 Stress due to lateral loading from tilt

Top left flange tip

$$f_{tl} = \frac{(IM)\left(M_g\right)\theta_{eq}W_{top}}{2I_y} = \frac{(1.2)(1617.23k\cdot ft)(0.0412)(49in)}{(2)(71865.3in^4)}\left(\frac{12in}{1ft}\right) = 0.327\,ksi$$

Bottom right flange tip

$$f_{br} = -\frac{(IM)\left(M_g\right)\theta_{eq}W_{bot}}{2I_y} = -\frac{(1.2)(1617.23k\cdot ft)(0.0412)(38.375in)}{(2)(71865.3in^4)}\left(\frac{12in}{1ft}\right) = -0.256\,ksi$$

4.3.2.6.3 Factor of Safety against Cracking

Cracking moment

$$f_{direct} = f_{ps} + (IM)f_g = 0.906ksi + (1.2)(-1.193ksi) = -0.519ksi$$

$$M_{cr} = \frac{(f_r - f_{direct})2I_y}{W_{top}} = \frac{\left(0.602ksi - (-0.519ksi)\right)(2)(71865.3in^4)}{49in}\left(\frac{1ft}{12in}\right) = 275.67k\cdot ft$$

Tilt angle at first crack

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

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$$\theta_{cr} = \frac{275.67\ k \cdot ft}{(1.2)1617.23\ k \cdot ft} = 0.142\ rad$$

Factor of Safety against Cracking

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

$$FS_{cr} = \frac{\left(\left(40000\,\frac{k\cdot in}{rad}\right)\left(0.142\ rad - 0.02\,\frac{ft}{ft}\right)\right)}{(1.2)(128.253kip)[((0.8)(6.355in) + 82.345in)(0.142rad) + 1.803in]} = 2.180$$

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

4.3.2.6.4 Factor of Safety against Failure

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

$$\theta_{max} = 0.02\frac{ft}{ft} + \sqrt{\left(0.02\ \frac{ft}{ft}\right)^2 + \frac{1.803in + ((0.8)(6.355in) + 82.345in)\left(0.02\frac{ft}{ft}\right)}{2.5(1.2)(6.355in)}} = 0.455\ rad \therefore 0.4\ rad$$

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

$$FS_f = \frac{\left(40000\,\frac{k\cdot in}{rad}\right)\left(0.4 - 0.02\frac{ft}{ft}\right)}{(1.2)(128.253kip)[(1.2)(6.355in)(0.4)(1 + 2.5(0.4)) + (82.345in)(0.4) + 1.803in} = 2.418$$

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

4.3.2.6.5 Factor of Safety against Rollover

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

$$\theta_{ro} = \frac{(1.2)(128.253kip)\left(\frac{72in}{2} - (24in)(0.02)\right)}{\left(40000\,\frac{k\cdot n}{rad}\right)} + 0.02 = 0.1567\ rad$$

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

$$FS_r = \frac{\left(40000\,\frac{k\cdot in}{rad}\right)(0.1567 - 0.02)}{(1.2)(128.253kip)\left[\left((1.2)(6.355in)(1 + 2.5(0.1567)) + 82.345in\right)(0.1567) + 1.803in\right]} = 2.170$$

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

4.3.2.7 *Analyze at maximum superelevation, no impact*

4.3.2.7.1 Equilibrium tilt angle

$$\theta_{eq} = \frac{(K_\theta\alpha + (IM)W_g e_i)}{K_\theta - (IM)W_g(y_r + z_o)} = \frac{\left(\left(40000\frac{k\cdot in}{rad}\right)\left(0.06\frac{ft}{ft}\right) + (1.0)(128.253kip)(1.803in)\right)}{\left(40000\frac{k\cdot in}{rad}\right) - (1.0)(128.253kip)(82.345in + 6.355in)} = 0.092\ rad$$

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4.3.2.7.2 Stress due to lateral loading from tilt

$$f_{tl} = \frac{(IM)\left(M_g\right)\theta_{eq}W_{top}}{2I_y} = \frac{(1.0)(1617.23k\cdot ft)(0.092)(49in)}{(2)(71865.3in^4)}\left(\frac{12in}{1ft}\right) = 0.608\ ksi$$

$$f_{br} = -\frac{(IM)\left(M_g\right)\theta_{eq}W_{bot}}{2I_y} = -\frac{(1.0)(1617.23k\cdot ft)(0.036)(38.375in)}{(2)(71865.3in^4)}\left(\frac{12in}{1ft}\right) = -0.476\ ksi$$

4.3.2.7.3 Factor of Safety against Cracking

Cracking moment

$$f_{direct} = f_{ps} + (IM)f_g = 0.906ksi + (1.0)(-1.193ksi) = -0.044ksi$$

$$M_{cr} = \frac{(f_r - f_{direct})2I_y}{W_{top}} = \frac{\left(0.602ksi - (0.906ksi - 1.193ksi)\right)(2)(71865.3in^4)}{49in}\left(\frac{1ft}{12in}\right) = 217.3k\cdot ft$$

Tilt angle at first crack

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

$$\theta_{cr} = \frac{217.3\ k\cdot ft}{1617.23\ k\cdot ft} = 0.1344\ rad$$

Factor of Safety against Cracking

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

$$FS_{cr} = \frac{\left(\left(40000\,\frac{k\cdot n}{rad}\right)\left(0.1344\ rad - 0.06\,\frac{ft}{ft}\right)\right)}{(1.0)(128.253kip)[((1.0)(6.355in) + 82.129in)(0.1344rad) + 1.803in]} = 1.694$$

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

4.3.2.7.4 Factor of Safety against Failure

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

$$\theta_{max} = 0.06\frac{ft}{ft} + \sqrt{\left(0.06\ \frac{ft}{ft}\right)^2 + \frac{1.803in + \left((1.0)(6.355in) + 82.345in\right)\left(0.06\frac{ft}{ft}\right)}{2.5(1.0)(6.355in)}} = 0.732\ rad \therefore 0.4\ rad$$

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

$$FS_f = \frac{\left(40000\,\frac{k\cdot in}{rad}\right)\left(0.4 - 0.06\frac{ft}{ft}\right)}{(1.0)(128.253kip)[(1.0)(6.355in)(0.4)\left(1 + 2.5(0.4)\right) + (82.345in)(0.4) + 1.803in} = 2.662$$

$FS_f > 1.5$ **OK**

4.3.2.7.5 Factor of Safety against Rollover

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

$$\theta_{ro} = \frac{(1.0)(128.253kip)\left(\frac{72in}{2} - (24in)(0.06)\right)}{\left(40000\,\frac{k\cdot in}{rad}\right)} + 0.06 = 0.1708\,rad$$

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

$$FS_r = \frac{\left(40000\,\frac{k\cdot in}{rad}\right)(0.1708 - 0.06)}{(1.0)(128.253kip)\left[\left((1.0)(6.355in)\left(1 + 2.5(0.1708)\right) + 82.345in\right)(0.1708) + 1.803in\right]} = 1.965$$

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

4.3.3 Check concrete strength

4.3.3.1 *Compression*

For the general case, compression stress is limited to $-0.65f_c'$.

Bottom centerline, impact up

$$f_b = f_{ps} + (IM)f_g$$

$$f_b = -4.083ksi + (0.8)(1.109ksi) = -3.196ksi$$

$$-0.65f_c' = -3.196ksi, f_c' = 4.9\ ksi$$

Peak compression stress at the extremities of the section are limited to $-0.70f_c'$.

$$f_b = f_{ps} + (IM)(f_g + f_{tilt})$$

$$f_b = -4.083ksi + (0.8)(1.109ksi - 0.187ksi)) = -3.345ksi$$

$$-0.70f_c' = -3.345ksi, f_c' = 4.8\ ksi$$

Our assumption for f'_c is ok.

4.3.3.2 *Tension*

Check tension stress limit

Maximum tension stress occurs at harp point top left corner of girder on normal crown slope with impact up

$$f_t = 0.906ksi + (0.8)(-1.193ksi + 0.238ksi) = 0.142\ ksi$$

Stress limit

$$0.0948\lambda\sqrt{f_c'} = 0.0948(1.0)\sqrt{6.3ksi} = 0.238\ ksi$$

Tension stress is within limit

Because stability requirements and stress limits are satisfied, temporary top strands are not required.

4.4 Step 4 - Design for Lifting with Temporary Top Strands

This step can be skipped because temporary stop strands are not required.

However, when temporary top strands are required for shipping, it is advantageous to take advantage of them at initial lifting. With the increased tension resistance provided by the temporary strands, move the lift point locations towards the ends of the girder. The dead load provides more balancing of the prestressing, reducing the concrete strength required at initial lifting. The fabricator can move to the next production cycle more quickly when removing girders from the forms at an earlier age.

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4.5 Step 5 – Check Erection Stresses

4.5.1 Losses between Transfer to Deck Placement

4.5.1.1 *Shrinkage of Girder Concrete*

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

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

$$K_{id} = \frac{1}{1 + \frac{E_p}{E_{ci}} \frac{A_{ps}}{A_g} \left(1 + \frac{A_g e^2}{I_g}\right) \left[1 + 0.7\psi_b(t_f, t_i)\right]}$$

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

$$k_s = 1.45 - 0.13\left(\frac{V}{S}\right) \geq 1.0 = 1.04$$

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

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

$$k_f = \frac{1}{1 + f'_{ci}} = 0.82$$

$$k_{td} = \frac{t}{12\left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20}\right) + t} = \begin{matrix} 0.758 \; with \; t = (t_d - t_i) = 119 \; day \\ 0.981 \; with \; t = (t_f - t_i) = 1999 \; day \end{matrix}$$

$$t_i = 1 \; day$$

$$t_d = 120 \; day$$

$$t_f = 2000 \; day$$

$$\varepsilon_{bid} = (1.04)(0.95)(0.82)(0.758)(0.48 \times 10^{-3}) = 0.000293$$

$$\psi_b(t_f, t_i) = 1.9(1.04)(0.96)(0.82)(0.981)(1)^{-0.118} = 1.52$$

$$K_{id} = \frac{1}{1 + \left(\frac{28500ksi}{4935.632ksi}\right)\left(\frac{8.246in^2}{874.531in^2}\right)\left(1 + \frac{(874.531in^2)(28.646in)^2}{556339.2in^4}\right)(1 + 0.7(1.52))} = 0.795$$

$$\Delta f_{pSR} = (0.000293)(28500ksi)(0.795) = 6.649 \; ksi$$

4.5.1.2 *Creep of Girder Concrete*

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

$$\psi_b(t_d, t_i) = 1.9(1.04)(0.96)(0.82)(0.758)(1)^{-0.118} = 1.17$$

$$\Delta f_{pCR} = \frac{28500ksi}{4935.632ksi}(2.722ksi)(1.17)(0.795) = 14.668 \; ksi$$

4.5.1.3 *Relaxation of Prestressing Strands*

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

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$$f_{pt} = f_{pj} - \Delta f_{pR0} - \Delta f_{pES} = 202.5ksi - 1.98ksi - 15.72ksi = 184.8ksi$$

$$\Delta f_{pR1} = \left[\frac{184.8\ ksi}{45}\frac{\log(24 \cdot 120)}{\log(24 \cdot 1)}\left(\frac{184.8ksi}{243ksi} - 0.55\right)\right]\left[1 - \frac{3(6.649ksi + 14.668ksi)}{184.8\ ksi}\right](0.795) = 1.127\ ksi$$

4.5.1.4 *Time dependent losses*

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

$$\Delta f_{pLT_{id}} = 6.649ksi + 14.668ksi + 1.127ksi = 22.44\ ksi$$

4.5.1.5 *Elastic gains*

Added dead load on noncomposite section

$$M_{adl} = M_{diaphragm} + M_{slab} + M_{haunch}$$

$$M_{adl} = 92.75k \cdot ft + 1127.58k \cdot ft + 456.45k \cdot ft = 1676.78k \cdot ft$$

$$\Delta f'_{cd} = \frac{M_{adl}e}{I_g}$$

$$\Delta f'_{cd} = (1676.78k \cdot ft)\left(\frac{12in}{1ft}\right)\left(\frac{28.646in}{556339.2in^4}\right) = 1.036\ ksi$$

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

$$\Delta f_{pED} = \left(\frac{28500ksi}{5292.088ksi}\right)(1.036ksi) = 5.579\ ksi$$

4.5.2 Stresses

Effective prestress

$$P = A_{ps}\left(0.75f_{pu} - \Delta f_{pR0} - \Delta f_{pES} - \Delta f_{pLT_{id}} + \Delta f_{pED}\right)$$

$$P = 8.246in^2(202.5ksi - 1.98ksi - 15.72ksi - 22.44ksi + 5.579ksi) = 1384.8\ kip$$

Stress in girder due to effective prestress

$$f_{ps} = \frac{P}{A} + \frac{Pe}{S}$$

$$f_t = \frac{-1384.8kip}{874.531in^2} + \frac{(-1834.8kip)(28.646in)}{-16268.9in^3} = 0.855ksi$$

$$f_b = \frac{-1384.8kip}{874.531in^2} + \frac{(-1834.8kip)(28.646in)}{17493.0in^3} = -3.851ksi$$

Stress due to loading

$$f = \frac{M_{girder} + M_{diaphragm} + M_{slab} + M_{haunch}}{S}$$

$$f_t = \frac{(1944.05k \cdot ft + 92.75k \cdot ft + 1127.58k \cdot ft + 456.45k \cdot ft)}{-16268.9in^3}\left(\frac{12in}{1ft}\right) = -2.671\ ksi$$

$$f_b = \frac{(1944.05k \cdot ft + 92.75k \cdot ft + 1127.58k \cdot ft + 456.45k \cdot ft)}{17493.0in^3}\left(\frac{12in}{1ft}\right) = 2.483\ ksi$$

Stress Limits

Compression

$$-0.45f'_c = -0.45(6.3ksi) = -2.835ksi$$

Tension

$$0.19\lambda\sqrt{f'_c} = 0.19(1.0)\sqrt{6.3\,ksi} = 0.477ksi$$

Stress Demand

$$f_t = 0.855ksi - 2.671ksi = -1.816\,ksi$$

$$f_b = -3.851ksi + 2.483ksi = -1.368\,ksi$$

Repeating these calculations for PSXFR location, the stress demand on the bottom of the girder is $-2.795ksi$. A concrete strength of 6.212 ksi satisfies the compression stress limit. This is the controlling point for compression (See PGSuper output)

4.6 Step 6 – Check Final Conditions

4.6.1 Losses from Deck Placement to Final

4.6.1.1 *Shrinkage of Girder Concrete*

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

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

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

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

From before

$$k_s = 1.04$$

$$k_{hs} = 0.95$$

$$k_{hc} = 0.96$$

$$k_f = 0.82$$

$$\psi_b(t_f, t_i) = 1.52$$

$$\varepsilon_{bid} = 0.000293$$

$$k_{td}(t = t_f - t_i) = 0.981$$

$$\varepsilon_{bif} = (1.04)(0.95)(0.82)(0.981)(0.48 \times 10^{-3}) = 0.000380$$

$$\varepsilon_{bdf} = 0.000380 - 0.000293 = 0.000087$$

$$e_c = e + y_{bc} - y_b = 28.646in + 43.761in - 31.804in = 40.603in$$

$$K_{df} = \frac{1}{1 + \left(\frac{28500ksi}{4935.632ksi}\right)\left(\frac{8.246in^2}{1280.808in^2}\right)\left(1 + \frac{(1280.808in^2)(40.603in)^2}{952196.3in^4}\right)(1 + 0.7(1.52))} = 0.802$$

$$\Delta f_{pSD} = (0.000087)(28500ksi)(0.802) = 1.988\,ksi$$

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4.6.1.2 *Creep of Girder Concrete*

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

$$\Delta f_{cd} = -\left(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}\right)\left(\frac{A_{ps}}{A_g}\right)\left(1 + \frac{A_g e^2}{I_g}\right) - (\Delta f'_{cd} + \Delta f''_{cd})$$

$$\Delta f'_{cd} = \frac{M_{adl} e}{I_g} = 1.036\ ksi$$

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

$$M_{sidl} = M_{barrier} + M_{overlay}$$

$$M_{sidl} = 444.35k \cdot ft + 375.12k \cdot ft = 819.47k \cdot ft$$

$$\Delta f''_{cd} = \frac{(819.47k \cdot ft)(43.761in - 31.804in + 28.646in)}{952196.3in^4}\left(\frac{12in}{1ft}\right) = 0.419ksi$$

$$\Delta f_{cd} = -(22.44\ ksi)\left(\frac{8.246in^2}{874.531in^3}\right)\left(1 + \frac{(874.531in^2)(28.646in)^2}{556339.2in^4}\right) - (1.036ksi + 0.419ksi) = -1.940\ ksi$$

$$k_{td} = \frac{t}{12\left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20}\right) + t} = 0.868\ with\ t = \left(t_f - t_d\right) = 1886\ day$$

$$\psi_b\left(t_f, t_d\right) = 1.9(1.04)(0.96)(0.82)(0.98)(120)^{-0.118} = 0.866$$

$$\Delta f_{pCD} = \left(\frac{28500ksi}{4935.632ksi}\right)(2.722ksi)(1.52 - 1.17)(0.802) + \left(\frac{28500ksi}{5292.088ksi}\right)(-1.942ksi)(0.866)(0.802) = -2.868\ ksi$$

4.6.1.3 *Relaxation of Prestressing Strands*

$$\Delta f_{pR2} = \Delta f_{pR1} = 1.127\ ksi$$

4.6.1.4 *Shrinkage of Deck Concrete*

$$\Delta f_{pSS} = \frac{E_p}{E_c}\Delta f_{cdf} K_{df}\left[1 + 0.7\psi_b\left(t_f, t_d\right)\right]$$

$$\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{c\ deck}}{\left[1 + 0.7\psi_d\left(t_f, t_d\right)\right]}\left(\frac{1}{A_c} - \frac{e_c\ e_d}{I_c}\right)$$

$$\Delta f_{cdf} = \frac{(0.000265)(540in^2)(4266.223ksi)}{1 + 0.7(2.12)}\left(\frac{1}{539.968in^2} - \frac{(40.603in)(25.739in)}{952196.3in^4}\right) = 0.078\ ksi$$

$$\Delta f_{pSS} = \left(\frac{28500ksi}{5292.088ksi}\right)(0.078ksi)(0.802)\left(1 + 0.7(0.868)\right) = 0.541\ ksi$$

4.6.1.5 *Time Dependent Losses*

$$\Delta f_{pLT_{df}} = \Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR1} - \Delta f_{pSS} = 1.988ksi - 2.868ksi + 1.127ksi - 0.541ksi = -0.294\ ksi$$

$$\Delta f_{pLT} = \Delta f_{pLT_{id}} + \Delta f_{pLT_{df}} = 22.44ksi - 0.294ksi = 22.15ksi$$

$$\Delta f_{pT} = \Delta f_{pR0} + \Delta f_{pES} + \Delta f_{pLT} - \Delta f_{pED} - \Delta f_{pSIDL} = 1.98ksi + 15.719ksi + 22.15ksi - 5.579ksi - 2.268ksi = 32.0ksi$$

DRAFT

4.6.1.6 *Elastic Gains*

4.6.1.6.1 Superimposed dead loads

$$\Delta f_{pSIDL} = \frac{E_p}{E_c}\Delta f''_{cd} = \left(\frac{28500ksi}{5292.088ksi}\right)(0.421ksi) = 2.268\ ksi$$

4.6.1.6.2 Live Loads

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

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

$$\Delta f'''_{cd} = \begin{cases} \dfrac{(2135.40k \cdot ft)(43.761in - 31.804in + 28.646in)}{952196.3in^4}\left(\dfrac{12in}{1ft}\right) = 1.093\ ksi\ (Design\ Live\ Load) \\ \dfrac{(641.54k \cdot ft)(43.761in - 31.804in + 28.646in)}{952196.3in^4}\left(\dfrac{12in}{1ft}\right) = 0.328\ ksi\ (Fatigue\ Live\ Load) \end{cases}$$

$$\Delta f_{pLL} = \begin{cases} \left(\dfrac{28500ksi}{5292.088ksi}\right)(1.093ksi) = 5.855ksi\ (Design\ Live\ Load) \\ \left(\dfrac{28500ksi}{5292.088ksi}\right)(0.328ksi) = 1.768ksi\ (Fatigue\ Live\ Load) \end{cases}$$

4.6.2 Stresses

4.6.2.1 *Final Stresses without Live Load*

Effective Prestress

$$P = A_{ps}\left(0.75f_{pu} - \Delta f_{pR0} - \Delta f_{pES} - \Delta f_{pLT_{id}} + \Delta f_{pED} - \Delta f_{pLT_{df}} + \Delta f_{pSIDL}\right)$$

$$P = 8.246in^2(202.5ksi - 1.98ksi - 15.72ksi - 22.44ksi + 5.579ksi - (-0.349ksi) + 2.268ksi)$$
$$= (8.246in^2)(170.556ksi) = 1406.4\ kip$$

Stress in girder due to effective prestress

$$f_{ps} = \frac{P}{A} + \frac{Pe}{S}$$

$$f_t = \frac{-1406.4kip}{874.531in^2} + \frac{(-1406.4kip)(28.646in)}{-16268.9in^3} = 0.868ksi$$

$$f_b = \frac{-1406.4kip}{874.531in^2} + \frac{(-1406.4kip)(28.646in)}{17493.0in^3} = -3.911ksi$$

Stress due to loading

$$f = \frac{(M_{girder} + M_{diaphragm} + M_{slab} + M_{haunch})}{S} + \frac{\left(M_{barrier} + M_{overlay}\right)}{S_c} + f_{ss}$$

$$f_t = \frac{(1944.05 + 92.75 + 1127.58 + 456.45k \cdot ft)}{-16268.9in^3}\left(\frac{12in}{1ft}\right) + \frac{(446.21 + 376.69k \cdot ft)}{-42816.4in^3}\left(\frac{12in}{1ft}\right) - 0.343ksi = -3.244\ ksi$$

$$f_b = \frac{(1944.05 + 92.75 + 1127.58 + 456.45k \cdot ft)}{17493.0in^3}\left(\frac{12in}{1ft}\right) + \frac{(446.21 + 0.0k \cdot ft)}{21759.0in^3}\left(\frac{12in}{1ft}\right) + 0.102ksi = 2.832\ ksi$$

This limit state evaluation is for compression stress. Minimize the stress at the bottom of the girder by considering the case before future overlay installation.

Stress Limits

Compression

$$-0.45f'_c = -0.45(6.3ksi) = -2.835ksi$$

Stress Demand

$$f_t = 0.868ksi - 3.244ksi = -2.376\ ksi$$

Stresses are within allowable limits

4.6.2.2 *Final Stresses with Live Load*

4.6.2.2.1 Service III Limit State

The design calculations cover the Service III limit state. Validate the assumed final effective prestress of 85% of the initial prestress.

$$1 - \frac{202.5ksi - 170.556ksi - 0.8(5.885ksi)}{202.5ksi} = 0.866 = 86.5\%\ \textbf{OK}$$

4.6.2.2.2 Service I Limit State

Effective Prestress with Service I live load

$$P = A_{ps}\left(0.75f_{pu} - \Delta f_{pR0} - \Delta f_{pES} - \Delta f_{pLT_{id}} + \Delta f_{pED} - \Delta f_{pLT_{df}} + \Delta f_{pSIDL} + \Delta f_{pLL}\right)$$

$$P = 8.246in^2(202.5ksi - 1.98ksi - 15.72ksi - 22.44ksi + 5.579ksi - (-0.349ksi) + 2.268ksi + 5.855ksi)$$
$$= (8.246in^2)(176.411ksi) = 1454.68\ kip$$

Stress in girder due to effective prestress

$$f_{ps} = \frac{P}{A} + \frac{Pe}{S}$$

$$f_t = \frac{-1454.68kip}{874.531in^2} + \frac{(-1454.68kip)(28.646in)}{-16268.9in^3} = 0.898ksi$$

$$f_b = \frac{-1454.68kip}{874.531in^2} + \frac{(-1454.68kip)(28.646in)}{17493.0in^3} = -4.046ksi$$

Stress due to loading

$$f = \frac{(M_{girder} + M_{diaphragm} + M_{slab} + M_{haunch})}{S} + \frac{(M_{barrier} + M_{overlay} + M_{llim})}{S_c} + f_{ss}$$

$$f_t = \frac{(1944.05 + 92.75 + 1127.58 + 456.45k \cdot ft)}{-16268.9in^3}\left(\frac{12in}{1ft}\right) + \frac{(446.21 + 376.69 + 2135.4k \cdot ft)}{-42816.4in^3}\left(\frac{12in}{1ft}\right) - 0.343ksi$$
$$= -3.839\ ksi$$

$$f_b = \frac{(1944.05 + 92.75 + 1127.58 + 456.45k \cdot ft)}{17493.0in^3}\left(\frac{12in}{1ft}\right) + \frac{(446.21 + 0.00 + 0.0k \cdot ft)}{21759.0in^3}\left(\frac{12in}{1ft}\right) + 0.102ksi$$
$$= 2.830ksi$$

DRAFT

This limit state evaluation is for compression stress. Minimize the stress at the bottom of the girder by considering the case before future overlay installation and live load not on the structure.

Stress Limits

Compression

$$-0.6f_c' = -0.6(6.3ksi) = -3.780ksi$$

Stress Demand

$$f_t = (0.898ksi - 3.839ksi) = -2.941\ ksi$$

Stresses are within limits

4.6.2.2.3 Fatigue I Limit State

Effective Prestress with Fatigue I live load

$$P = A_{ps}\left(0.75f_{pu} - \Delta f_{pR0} - \Delta f_{pES} - \Delta f_{pLT_{id}} + \Delta f_{pED} - \Delta f_{pLT_{df}} + \Delta f_{pSIDL} + 1.5\Delta f_{pLL}\right)$$

$$P = 8.246in^2(202.5ksi - 1.98ksi - 15.72ksi - 22.44ksi + 5.579ksi - (-0.349ksi) + 2.268ksi + 1.5(1.768ksi)) = (8.246in^2)(173.208ksi) = 1428.27\ kip$$

Stress in girder due to effective prestress

$$f_{ps} = \frac{P}{A} + \frac{Pe}{S}$$

$$f_t = \frac{-1428.27kip}{874.531in^2} + \frac{(-1428.27kip)(28.646in)}{-16268.9in^3} = 0.882ksi$$

$$f_b = \frac{-1428.27kip}{874.531in^2} + \frac{(-1428.27kip)(28.646in)}{17493.0in^3} = -3.947ksi$$

Stress Limits

Compression

$$-0.4f_c' = -0.4(6.3ksi) = -2.520ksi$$

Stress Demand

$$f = \frac{0.5(M_{girder} + M_{diaphragm} + M_{slab} + M_{haunch})}{S} + \frac{0.5M_{barrier} + 0.5M_{overlay}}{S_c} + 0.5f_{ss} + \frac{1.5M_{LLIM}}{S_c}$$

$$f_t = \frac{0.5(1944.05 + 92.75 + 1127.58 + 456.45k \cdot ft)}{-16268.9in^3}\left(\frac{12in}{1ft}\right) + \frac{(0.5)(446.21 + 376.69k \cdot ft)}{-42816.4in^3}\left(\frac{12in}{1ft}\right) + 0.5(-0.343ksi) + \frac{1.5(641.54k \cdot ft)}{-42816.4in^3}\left(\frac{12in}{1ft}\right) = 0.5(-2.671ksi - 0.230ksi - 0.343ksi) + 1.5(-0.180ksi) = -1.892ksi$$

$$f_t = 0.5(0.882ksi) - 1.892ksi = -1.451\ ksi$$

Stresses are within limits

4.6.3 Moment Capacity

4.6.3.1 *Compute Nominal Moment Capacity at 0.5L$_g$.*

Strength I limit state

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

$$M_u = 1.25(1944.05 + 92.75 + 1127.58 + 456.45 + 446.21) + 1.50(376.69) + 1.75(0.554)(3851.68) = 9383.04k \cdot ft$$

$$c = \frac{A_{ps}f_{pu} - \alpha_1 f_c'(b - b_w)h_f}{\alpha_1 f_c' \beta_1 b_w + kA_{ps}\dfrac{f_{pu}}{d_p}}$$

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

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

$$\alpha_1 = 0.85$$

$$d_p = Y_t + e + t_s = 34.196in + 28.646in + 7in = 69.836in$$

$$c = \frac{(8.246in^2)(270ksi) - 0.85(4ksi)(72in - 6.125in)(7in)}{0.85(4ksi)(0.85)(6.125in) + (0.28)(8.246in^2)\left(\dfrac{270ksi}{69.836in}\right)} = \frac{658.62kip}{17.7\dfrac{k}{in} + 8.926\dfrac{k}{in}} = 24.74in$$

$$f_{ps} = 270ksi\left(1 - 0.28\frac{24.74in}{69.836in}\right) = 243.22ksi$$

$$a = \beta_1 c = 0.85(24.74in) = 21.03in$$

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + \alpha_1 f_c'(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

$$M_n = (8.246in^2)(243.22ksi)\left(69.836in - \frac{21.03in}{2}\right) + 0.85(4ksi)(72in - 6.125in)(7in)\left(\frac{21.03in}{2} - \frac{7in}{2}\right)$$
$$= 129972k \cdot in = 10831k \cdot ft$$

$$d_t = 73in - 2in = 71in$$

$$\phi = 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) = 0.583 + 0.25\left(\frac{71in}{21.03in} - 1\right) = 1.177, use\ 1.0$$

$$M_r = \phi M_n = 1.0(10831k \cdot ft) = 10831k \cdot ft$$

$M_r > M_u$ **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:

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

where

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

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$$k = 0.67 + \frac{f'_c}{9000}$$

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

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

$$\varepsilon'_c x1000 = \frac{f'_c}{E_c}\frac{n}{n-1}$$

Effective prestress, $f_{pe} = f_{pj} - \Delta f_{pT} = 202.5ksi - 31.943ksi = 170.557ksi$

Initial strain in prestressing strand, $\varepsilon_{psi} = \frac{f_{pe}}{E_p} = \frac{170.557ksi}{28500ksi} = 5.984x10^{-3}$

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.

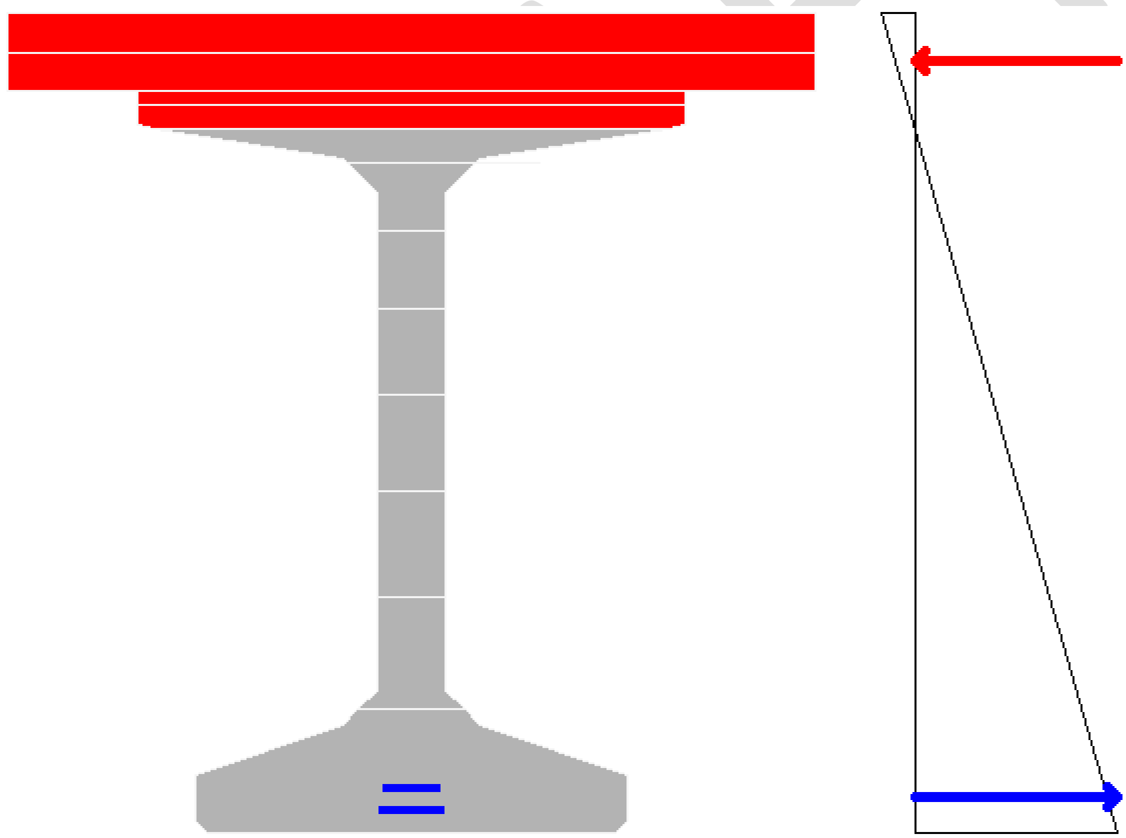

Figure 4-8: Discretized Girder Section for Strain Compatibility Analysis

Slice	Area (in^2)	Y_{cg} (in)	Strain	Stress (KSI)	δF = (Area)(Stress) (kip)	δM = (δF)(Y_{cg}) (kip-ft)
1	262.785	39.371	-0.00246618	-3.701	-972.51	-3190.75
2	241.186	35.871	-0.00144242	-3.773	-909.90	-2719.96
3	54.444	33.641	-0.000789968	-3.248	-176.85	-495.80
4	104.709	32.017	-0.000314846	-1.314	-137.54	-366.95
5	84.035	29.736	0.000352098	0.000	0.00	0.00
6	44.107	25.108	0.00170587	0.000	0.00	0.00
7	42.228	18.283	0.00370241	0.000	0.00	0.00
8	47.197	10.983	0.00583769	0.000	0.00	0.00
9	52.165	2.871	0.00821022	0.000	0.00	0.00
10	57.133	-6.051	0.01082	0.000	0.00	0.00
11	64.906	-15.979	0.0137241	0.000	0.00	0.00
12	323.607	-27.341	0.0170475	0.000	0.00	0.00
13	1.736	-27.804	0.0231671	266.187	462.10	-1070.67
14	3.038	-27.804	0.0231671	266.187	808.68	-1873.67
15	3.472	-29.804	0.0237521	266.711	926.02	-2299.89

Resultant Force = ∑(δF) = 0.00 kip

Resultant Moment = ∑(δM) = -12017.7 kip-ft

Depth to neutral axis, *c = 10.256 in*
Compression Resultant, *C = -2196.8 kip*
Depth to Compression Resultant, d_c =*4.196 in*
Tension Resultant, *T = 2196.80 kip*
Depth to Tension Resultant, d_e =*69.843 in*
Nominal Capacity, M_n = *12017.7 kip-ft*
Moment Arm, $d_e - d_c = M_n/T$ = *65.647 in*

The capacity reduction factor is $0.75 \le \phi = 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \le 1.0$

$$\phi = 0.583 + 0.25\left(\frac{71in}{10.256in} - 1\right) = 2.06 \;\therefore\; \phi = 1.0$$

$$M_r = 12017.7k \cdot ft \ge M_u = 9385.79k \cdot ft \;\; \textbf{OK}$$

4.6.3.2 *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

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

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

4.6.3.2.1 Compute cracking moment at 0.5L_g.

$f_r = 0.24\sqrt{f_c'} = 0.24\sqrt{6.3ksi} = 0.602ksi$

$f_{cpe} = 3.911ksi$

$S_c = 21759.0in^3$

$S_{nc} = 17493.0in^3$

$M_{dnc} = M_{girder} + M_{diaphragms} + M_{slab} + M_{haunch} = 3620.83k \cdot ft$

$$M_{cr} = 1.0\left[(1.6 \cdot 0.602ksi + 1.0 \cdot 3.911ksi)(21759.0in^3)\left(\frac{1ft}{12in}\right) - (3620.831k \cdot ft)\left(\frac{21729.0in^3}{17493.0in^3} - 1\right)\right] = 8665.81\ k \cdot ft$$

4.6.3.2.2 Evaluate Minimum Reinforcement Requirement

$M_u = 9385.79k \cdot ft$

$$M_{r\,min} = lesser\ of \begin{cases} M_{cr} = 8665.81k \cdot ft \\ 1.33M_u = 1.33 \cdot 9385.79k \cdot ft = 12483.1k \cdot ft \end{cases} = 8665.81k \cdot ft$$

$M_r = 12017.1k \cdot ft \geq M_{r\,min} = 8665.81k \cdot ft$ **OK**

4.7 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 $P = 0.04A_{ps}(f_{pj} - \Delta f_{pR0} - \Delta f_{pES}) = 0.04(8.246in^2)(202.5ksi - 1.98ksi - 14.709ksi) = 61.29ksi$

The splitting zone is $\frac{h}{4} = \frac{5.5ft}{4} = 1.375ft$. The vertical reinforcement in the splitting zone is $3.277in^2$.

The splitting resistance is $P_r = f_s A_s = (20\ ksi)(3.277in^2) = 65.54\ kip$

$P < P_r$ **OK**

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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"

4.8 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 $1.5d = 1.5(66\ in) = 99\ in = 8.25\ ft$.

Provide #3 bars spaced at 6" for the end 8.25ft of the girder.

5 Shear Design

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.

5.1 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 5-1 illustrates this technique.

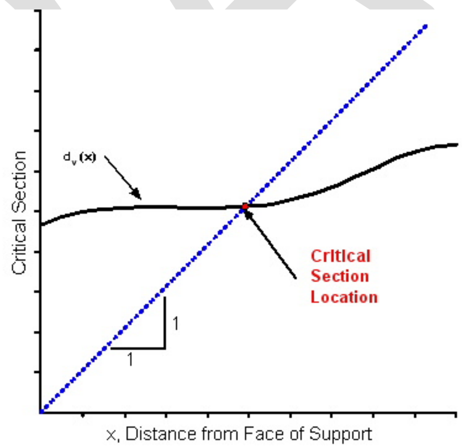

Figure 5-1: Graphical method to Determine Critical Section Location

For this girder, the critical sections are located *5.755 ft* and *118.825 ft* from the left support. The tables that follow show the details for finding the critical sections.

Table 5-1: Critical Section Calculation Details for Abutment 1

Location from Left Support (ft)	Assumed C.S. Location (in)	d_v (in)	CS Intersects?

DRAFT

(FoS) 0.500	0.000	63.060	No
(Bar Develop.) 1.280	9.356	63.060	No
(PSXFR) 1.292	9.499	63.060	No
3.792	39.499	63.060	No
4.125	43.500	63.060	No
5.755	63.060	63.060	*Yes
(H) 6.000	66.000	63.060	No
(1.5H) 8.750	99.000	63.061	No
11.091	127.096	63.061	No
(SZB) 12.170	140.038	63.061	No
(0.1L_e) 12.458	143.497	63.061	No
(SZB) 12.920	149.038	63.061	No
13.491	155.896	63.061	No

* - Intersection values are linearly interpolated

Table 5-2: Critical Section Calculation Details for Abutment 2

Location from Left Support (ft)	**Assumed C.S. Location (in)**	**d_v (in)**	**CS Intersects?**
111.089	155.896	63.061	No
(SZB) 111.661	149.038	63.061	No
(0.9L_e) 112.122	143.497	63.061	No
(SZB) 112.411	140.038	63.061	No
113.489	127.096	63.061	No
(1.5H) 115.830	99.000	63.061	No
(H) 118.580	66.000	63.060	No
118.825	63.060	63.060	*Yes
120.455	43.500	63.060	No
120.789	39.499	63.060	No
(PSXFR) 123.289	9.499	63.060	No
(Bar Develop.) 123.301	9.356	63.060	No
(FoS) 124.080	0.000	63.060	No

* - Intersection values are linearly interpolated

5.2 Check Ultimate Shear Capacity

5.2.1 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

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

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

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.

5.2.1.1 *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 =57.206 in*, d_e *=70.067 in*, and *h =73 in*.

$$d_v = greatest\ of \begin{cases} Moment\ Arm = 57.206in \\ 0.9d_e = 0.9(70.067in) = 63.06in \\ 0.72h = 0.72(73in) = 52.56in \end{cases}$$

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

$$M_u = 1674.53\ k \cdot ft$$

$$N_u = 0\ kip$$

$$|V_u - V_p| = 281\ kip$$

$$d_v = 63.060\ in$$

$$A_s = 0\ in^2$$

$$E_s = 29000\ ksi$$

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$$A_{ps} = 6.030\ in^2$$
$$E_{ps} = 28500\ ksi$$
$$A_{ct} = 482.906\ in^2$$
$$E_c = 5292.088ksi$$

$$\varepsilon_s = \frac{\left(\frac{|1674.63k\cdot ft|\left(\frac{12in}{1ft}\right)}{63.06in} + 0.5(0) + 281kip - (6.030in^2)(202.5ksi)\right)}{(29000ksi)(0in^2) + (28500ksi)(6.030in^2) + (5292.088\text{ksi})(482.906in^2)} = -0.228x10^{-3} < 0$$

Step 5: Compute β and θ

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} = \frac{4.8}{\left(1 + (750)(-0.228x10^{-3})\right)} = 5.79$$
$$\theta = 29 + 3500\varepsilon_s = 29 + (3500)(-0.228x10^{-3}) = 28.21°$$

5.2.1.2 *Compute Shear Capacity of Concrete*

$$V_c = 0.0316\beta\lambda\sqrt{f_c'}b_v d_v = 0.0316(5.79)(1.0)\sqrt{6.3ksi}(6.125in)(63.06in) = 177.36\ kip$$

5.2.1.3 *Compute Shear Capacity of Transverse Reinforcement*

For #5 stirrups, $A_v = 0.62\ in^2$.

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(0.62\ in^2)(60\ ksi)(63.06in)\cot 28.21}{6\ in} = 729.08\ kip$$

5.2.1.4 *Compute Nominal Shear Capacity of Section*

$$V_n = V_c + V_p + V_s = 177.36\ kip + 24.5\ kip + 729.08\ kip = 930.93\ kip$$
$$V_n = 0.25f_c' b_v d_v + V_p = 0.25(6.3\ ksi)(6.125\ in)(63.06\ in) + 24.5\ kip = 632.83kip$$
$$V_r = \phi V_n = 0.9(632.83\ kip) = 569.55\ kip$$

5.2.1.5 *Check Ultimate Shear Capacity*

$V_u = 305.51\ kip \leq V_r = 569.55\ kip$ **OK**

Repeat these calculations at all locations where stirrup size or spacing changes or where the applied shear abruptly changes.

5.2.2 Check Requirement for Transverse Reinforcement

Transverse reinforcement is required when $V_u > 0.5\phi(V_c + V_p)$. (LRFD 5.8.2.4)

$$0.5\phi(V_c + V_p) = 0.5(0.9)(177.36\ kip + 24.5\ kip) = 90.837\ kip < 305.51\ kip$$

V_u exceeds the limiting value; therefore, transverse reinforcement is required at this section. Transverse reinforcement is provided. **OK**

5.2.3 Check Minimum Transverse Reinforcement

Where transverse reinforcement is required, as specified in LRFD 5.8.2.4, the area of steel shall not be less than $A_{v\,min} = 0.0316\sqrt{f_c'}\frac{b_v s}{f_y} = 0.0316\sqrt{6.3\ ksi}\frac{(6.125\ in)(6\ in)}{60\ ksi} = 0.0486\ in^2 < 0.62\ in^2$

OK

This can also be represented as $\frac{A_v}{s}min = 0.0316\sqrt{f_c'}\frac{b_v}{f_y} = 0.0316\sqrt{6.3\ ksi}\frac{6.125\ in}{60\ ksi} = 0.0081\frac{in^2}{in} = 0.097\frac{in^2}{ft}$.

5.2.4 Check Maximum Spacing of Transverse Reinforcement

The spacing of the transverse reinforcement shall not exceed the following:

- If $V_u < 0.1f_c'b_vd_v$ then $s \leq 0.8d_v \leq 24\ in$
- If $V_u \geq 0.1f_c'b_vd_v$ then $s \leq 0.4d_v \leq 12\ in$

$$0.1f_c'b_vd_v = 0.1(6.3\ ksi)(6.125\ in)(63.06\ in) = 243.33\ kip < 305.51\ kip$$

$$s_{max} = 0.4d_v = 0.4(63.06\ in) = 25.2\ in\ > 12\ in \rightarrow\ s_{max} = 12\ in$$

The actual spacing is 6.0 in. **OK**

5.3 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:

$$A_sf_y + A_{ps}f_{ps} \geq \left[\frac{M_u}{d_v\phi_f} + 0.5\frac{N_u}{\phi_a} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta\right]$$

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:

$$A_sf_y +\ A_{ps}f_{ps} \geq \left(\frac{V_u}{\phi_v} - 0.5V_s - V_p\right)\cot\theta$$

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)

$$A_{ps} = (30)(0.217in^2) = 6.510in^2$$

From the moment capacity analysis, $f_{ps,avg} = 219.532ksi$. 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).

$M_u = 1674.53k \cdot ft$

$d_v = 63.06in$

$V_u = 305.51kip$

$V_s = 339.46kip$

$V_p = 24.5kip$

$\theta = 28.21°$

$$\frac{M_u}{d_v\phi_f} + 0.5\frac{N_u}{\phi_a} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta$$

$$= \frac{1674.53k \cdot ft\left(\frac{12in}{1ft}\right)}{(63.06in)(1.0)} + 0.5\frac{(0)}{1.0} + \left(\left|\frac{305.51kip}{0.9} - 24.5kip\right| - 0.5(339.46kip)\right)\cot 28.21° = 589.48kip$$

$A_{ps}f_{ps} = (6.510in^2)(219.532ksi) = 1429.15kip$

$1429.15kip \geq\ 589.48kip$ **OK**

5.4 Check Horizontal Interface Shear

This entire design is based on the assumption that the slab and girder work together to form a composite section. Verify the slab-girder interface has adequate capacity to develop this composite action.

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5.4.1 Check Nominal Capacity

The critical section for shear location is used to demonstrate these calculations. A complete design will verify the slab-girder interface capacity at various sections along the girder.

5.4.1.1 *Compute Nominal Capacity*

The nominal shear resistance at the slab-girder interface is $V_{ni} = cA_{cv} + \mu[A_{vf}f_y + P_c] \leq minimum\begin{cases}K_1 f'_c A_{cv} \\ K_2 A_{cv}\end{cases}$

where

V_{ni}	=	Nominal shear resistance (kip)
A_{cv}	=	Area of concrete engaged in shear transfer (in^2)
A_{vf}	=	Area of shear reinforcement crossing the shear plane (in^2)
f_y	=	Yield strength of reinforcement (ksi)
c	=	Cohesion factor
μ	=	Friction factor
P_c	=	Permanent net compressive force normal to the shear plane, or 0.0 kip if tensile (kip)
f'_c	=	Specified 28-day strength of the weaker concrete (ksi)
K_1	=	0.3
K_2	=	1.8

The top flange of the girder, which is a roughened surface, supports the deck slab. For this situation $c = 0.280$ *ksi* and $\mu = 1.0$.

The area of concrete engaged in the shear transfer: $A_{cv} = b_{vi}L_{vi} = (49\ in)\left(1\frac{in}{in}\right) = 49\frac{in^2}{in}$.

The area of shear reinforcement consists of the stirrups extending from the web into the slab (#5 @ 6.0 in): $A_{vf} = \frac{0.62\ in^2}{6\ in} = 0.103\frac{in^2}{in} = 1.24\frac{in^2}{ft}$.

It is conservative to take $P_c = 0\ klf$.

$V_{ni} = cA_{cv} + \mu[A_{vf}f_y + P_c] = (0.280\ ksi)\left(49\frac{in^2}{in}\frac{12in}{1ft}\right) + 1.0\left[\left(1.24\frac{in^2}{ft}\right)(60ksi) + 0\ klf\right] = 239.04\ \ kip/ft$

$K_1 f'_c A_{cv} = 0.3(4ksi)\left(49\frac{in^2}{in}\frac{12in}{ft}\right) = 705.6\ \ kip/ft$

$K_2 A_{cv} = 1.8\left(49\frac{in^2}{in}\frac{12in}{ft}\right) = 1058.4\ \ kip/ft$

$V_{ni} = 239.04\ \ kip/ft$

$V_r = \phi V_{ni} = 0.9(239.04\ \ kip/ft) = 215.1\ kip/ft$

5.4.1.2 *Compute Demand*

The factored interface shear stress for a concrete girder/slab bridge may be determined as $v_{ui} = \frac{V_u}{b_{vi}d_{vi}}$. The factored interface shear force for a concrete girder/slab bridge may be determined as $V_{ui} = v_{ui}A_{cv}$. Substituing Equation 5.8.4.2-1 into 5.8.4.2-2 the interface shear force is $V_{uh} = \frac{V_u}{d_{vi}}$.

At the critical section, $V_u = 305.51$ *kip*.

$$V_{uh} = \frac{V_u Q}{I} = \frac{(305.51kip)(10457.2in^3)}{(952196.3in^4)} = 40.262\frac{k}{ft}$$

$$V_{uh} \leq V_r$$

OK

5.4.2 Check Minimum Reinforcement

The LRFD specification requires a minimum amount of shear reinforcement in the slab-girder interface. Check to make sure this requirement is satisfied.

The cross-sectional area, A_{vf}, of the reinforcement per unit length should not be less than $\frac{0.05 b_v}{f_y}$.

For a cast-in-place concrete slab on clean concrete girder surface free of laitance:

- The minimum interface shear reinforcement need not exceed the lessor of the amount determined using Eqn. 5.8.4.4-1 and the amount needed to resist $\frac{1.33 V_{ui}}{\phi}$ as determined using Eqn 5.8.4.1-3
- The minimum reinforcement provisions shall be waived for girder/slab interfaces with surface roughened to an amplitude of 0.25 in where the factored interface shear stress, v_{ui} of Eqn 5.8.4.2-1 is less than 0.210 ksi, and all vertical (transverse) shear reinforcement required by the provisions of Article 5.8.1.1 is extended across the interface and adequately anchored into the slab.

$v_{ui} = \frac{V_n}{A_{cv}} = \frac{239.582 \frac{kip}{ft}}{49 \frac{in^2}{in} \frac{12in}{1ft}} = 0.068 \frac{ksi}{ft} < 0.100 \frac{ksi}{ft}$. This requirement is waived.

OK

The maximum allowable spacing of the transverse reinforcement is *24.0 in*. The actual spacing at this section is 6.*0 in*. The maximum spacing along the length of the girder is *18.0 in.* **OK**

6 Check Haunch Dimension

The slab offset was assumed to be $12.25 in$. 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.

The haunch depth is to be such that at the mid-span the distance between the bottom of the slab and the top of the girder is equal to the slab fillet dimension, $0.75 in$. Account for geometric effects due to the roadway and camber. The haunch depth at the bearing is $A_{haunch} = A_{slab+fillet} + A_{profile\ effect} + A_{girder\ orientation\ effect} + A_{excess\ camber}$.

6.1 Slab and Fillet

The slab and fillet is the gross slab depth plus the fillet dimension. If the actual camber is exactly equal to the predicted value, and all deflections are as predicted, the top of the girder will be exactly t_{fillet} below the bottom of the deck as its closest point.

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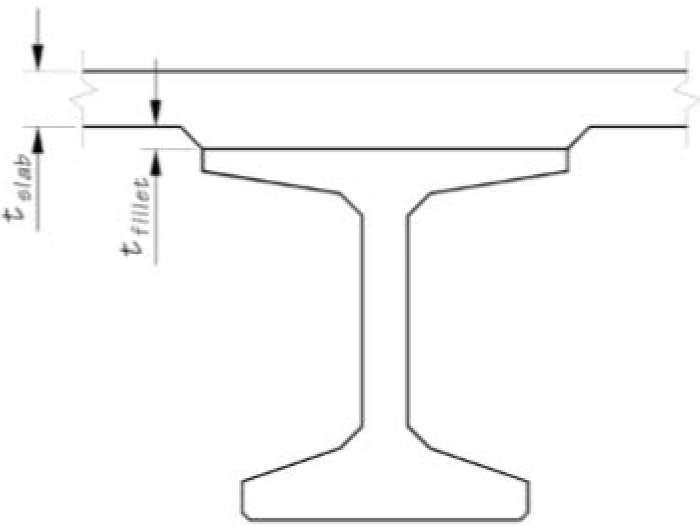

Figure 6-1: Slab + Fillet Effect

$$A_{slab+fillet} = 7.5\ in + 0.75\ in = 8.25in$$

6.2 Profile Effect

PGSuper uses a general approach to determine the profile effect. Draw a chord line from the point where a vertical line passing through the CL Bearings intersect the deck. Then the profile effect is the maximum difference in elevation between this chord line and the roadway surface.

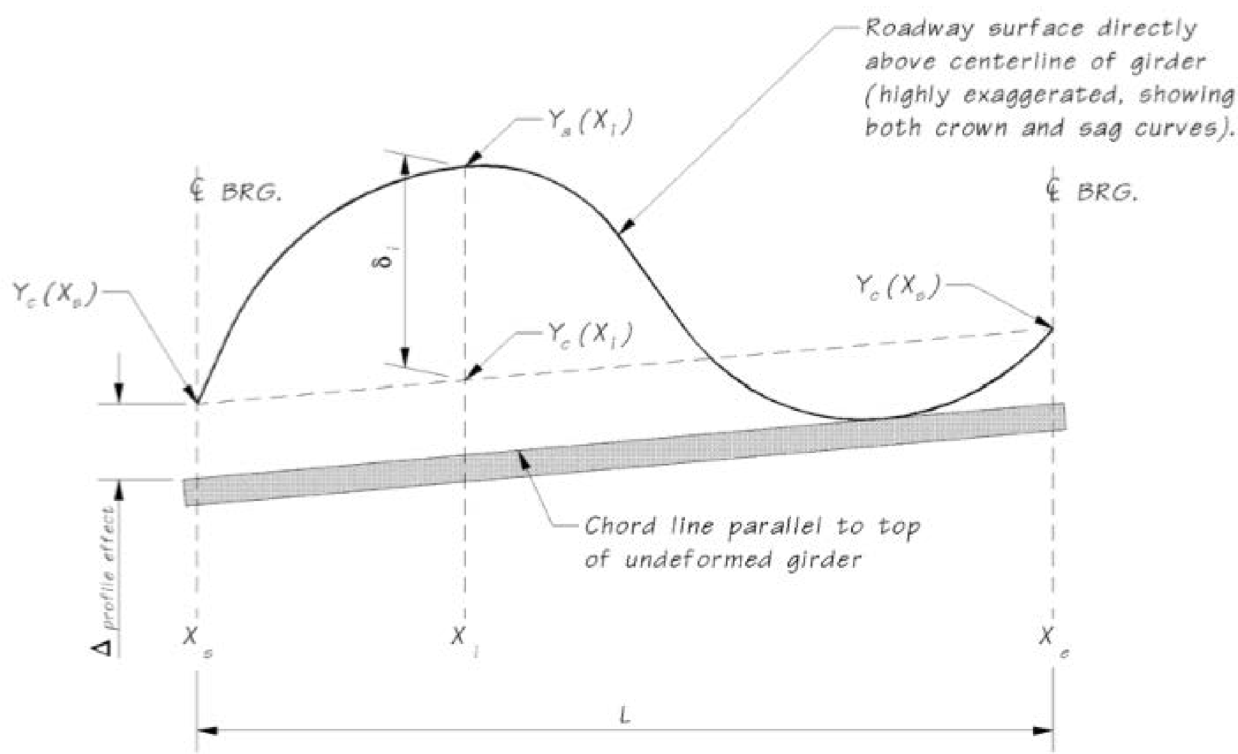

Figure 6-2: General Method for Profile Effect

The entire span of the bridge is within the limits of the horizontal and vertical curves. Use the simplified method of computing the profile effect. See BDM Appendix 5-B1 for additional information.

6.2.1 Vertical Curve

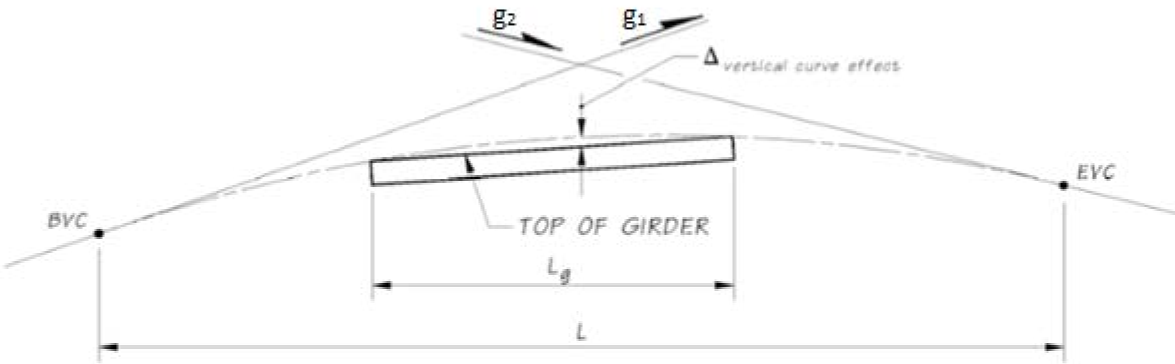

Figure 6-3: Vertical Curve Effect

$$A_{vc} = \frac{1.5(g_2 - g_1)L_g^2}{100L_{vc}}(in) = \frac{1.5\left(-1.5\% - (-2\%)\right)(124.58ft)^2}{100(600ft)} = 0.194\ in$$

6.2.2 Horizontal Curve

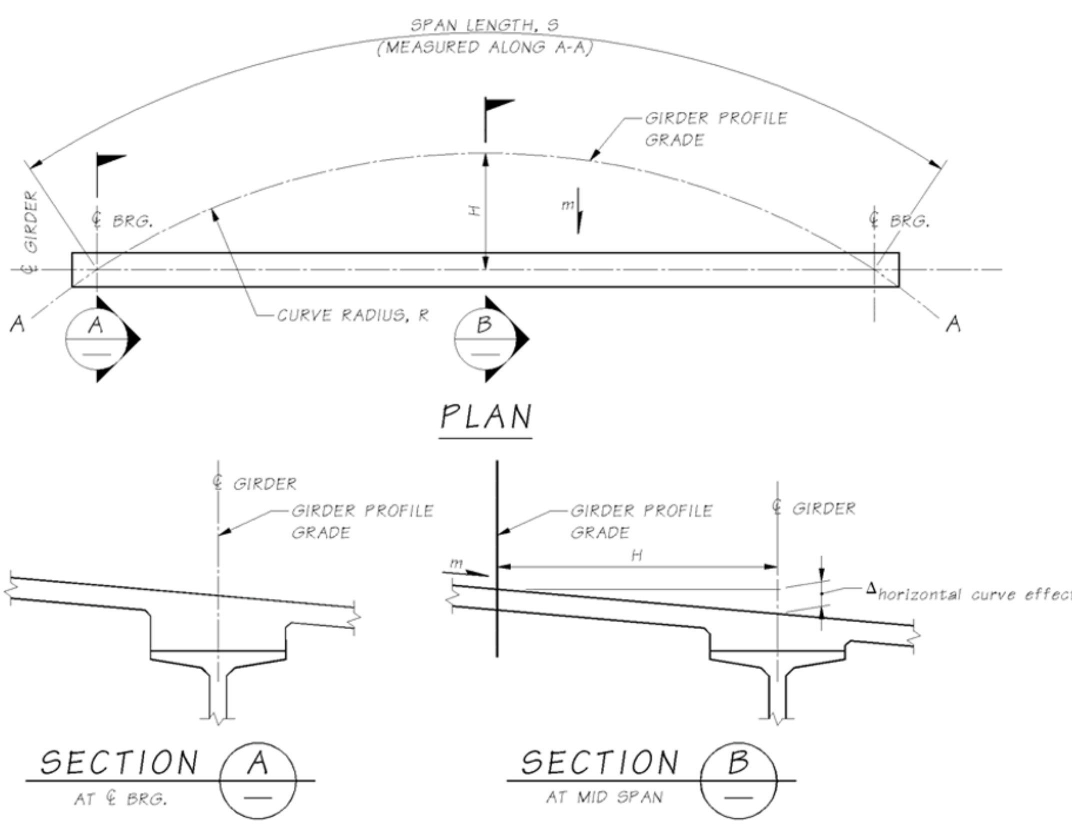

Figure 6-4: Horizontal Curve Effect

$$A_{hc} = \frac{1.5S^2m}{R}(in) = \frac{1.5(827.42 - 702.71ft)^2\left(0.04\frac{ft}{ft}\right)}{6000\ ft} = 0.156in$$

6.2.3 Profile Effect

$$A_{profile} = A_{vc} + A_{hc} = \ 0.194in + 0.156in = 0.350in$$

6.3 Girder Orientation Effect

The girder orientation effect accounts for the crown slope and the orientation of the girder. $A_{girder\ orientation\ effect} = m\frac{w_{tf}}{2}$.

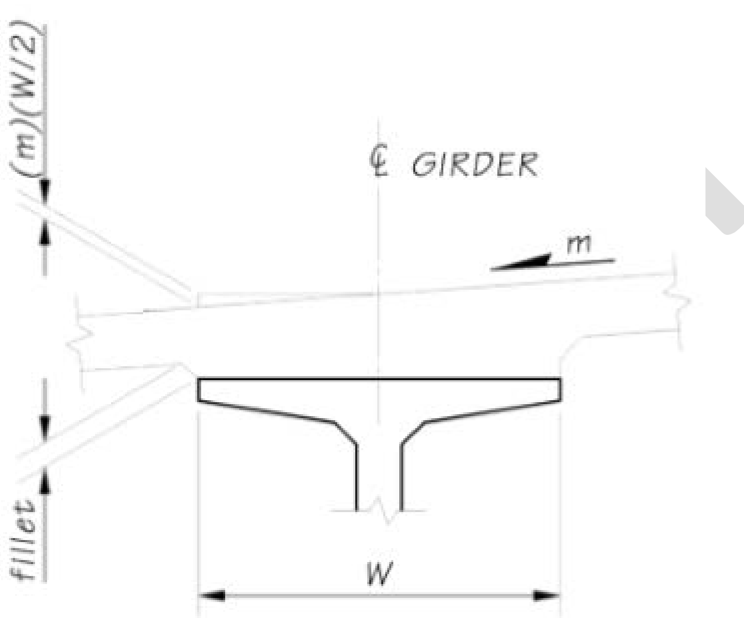

Figure 6-5: Top Flange Effect

$$A_{top\ flange\ effect} = 0.04\frac{49in}{2} = 0.980in$$

6.4 Excess Camber

The excess camber is the camber that remains in the girder after all of the loads are applied.

Figure 6-6: Camber Effect

The graphic below illustrates how the girder deflects over time.

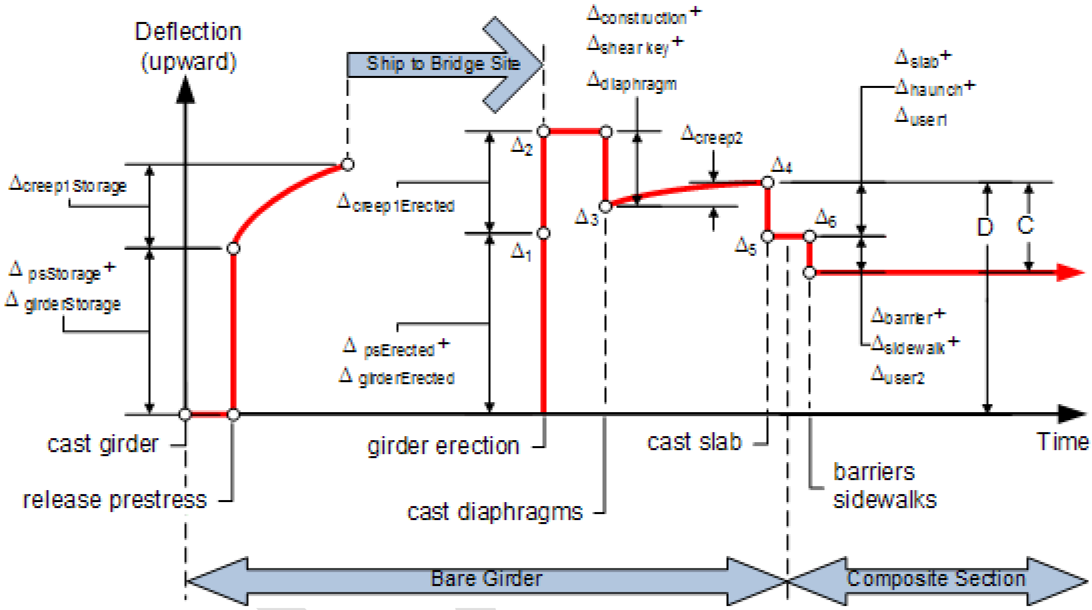

Figure 6-7: Camber Diagram

Assume time-dependent deformations end after deck casting

$$\Delta_{girder} = deflection\ due\ to\ girder\ self$$

$$\Delta_{ps} = deflection\ due\ to\ permanent\ prestressing, based\ on\ inplace\ span\ length$$

$$\Delta_{creep1} = \psi(t_e, t_i)(\Delta_{girder} + \Delta_{ps})$$

$$\Delta_{dia} = deflection\ due\ to\ diaphragm\ self\ weight$$

$$\delta_{girder} = incremental\ girder\ deflection\ due\ to\ change\ in\ support\ location\ between\ storage\ and\ erection$$

$$\Delta_{creep2} = [\psi(t_d, t_i) - \psi(t_e, t_i)](\Delta_{girder} + \Delta_{ps}) + \psi(t_d, t_e)(\Delta_{dia} + \delta_{girder})$$

$$\Delta_{deck} = deflection\ due\ to\ deck\ self\ weight$$

$$\Delta_{haunch} = deflection\ due\ to\ haunch\ self\ weight$$

$$\Delta_{barrier} = deflection\ due\ to\ traffic\ barrier\ self\ weight$$

$$\Delta_{excess} = excess\ camber$$

$$\Delta_1 = (\Delta_{girder} + \Delta_{ps})$$

$$\Delta_2 = \Delta_1 + \Delta_{creep1}$$
$$\Delta_3 = \Delta_2 + \Delta_{dia}$$

$$\Delta_4 = \Delta_3 + \Delta_{creep2}$$

$$\Delta_5 = \Delta_4 + \Delta_{deck} + \Delta_{haunch}$$

$$\Delta_6 = \Delta_{excess} = \Delta_5 + \Delta_{barrier}$$

6.4.1 Compute Creep Coefficients

The creep coefficients for release until erection and deck casting are computed above.

Prestress release until erection $\psi(t_h = 90, t_i = 1) = \psi(t_e = 90, t_i = 1) = 1.088$

Prestress release until deck casting $\psi(t_d = 120, t_e = 1) = 1.17$

Compute creep coefficient for erection to deck casting

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.

$$f'_{ci} = 6.3\ ksi$$

$$k_f = \frac{5}{1 + 6.3} = 0.685$$

$$k_{td} = \frac{(120 - 90)}{12\left(\frac{100 - 4(6.3)}{6.3 + 20}\right) + (120 - 90)} = 0.468$$

$$\psi(t_d = 120, t_e = 90) = 1.9(1.04)(0.96)(0.685)(0.468)(90)^{-0.118} = 0.356$$

6.4.2 Compute Deflections

Girder Deflection, for the erected girder

$$\Delta_g = \frac{5wL^4}{384E_{ci}I_x} = \frac{5(-1.002klf)(124.58ft)^4}{384(4935.632ksi)(556339.2in^4)}\left(\frac{1728in^3}{1ft^3}\right) = -1.978in$$

Prestress Deflection, $\Delta_{ps} = 4.284in$. This is the deflection measured relative to the ends of the girder. The deflection at the CL Bearing based on the release datum is $\Delta_{psbrg} = 0.208in$. The prestress deflection measured relative to the bearings is $\Delta_{ps} = 4.284in - 0.208in = 4.076in$

Creep Deflection during Storage, $\Delta_{creep1} = 1.088(4.076in - 1.978in) = 2.282in$

Diaphragm Deflection, $\Delta_{diaphragm} = -0.084in$

Slab Deflection, $\Delta_{slab} = -1.070in$

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Haunch Deflection, $\Delta_{haunch} = -0.434in$

Creep Deflection between diaphragm and deck casting, $\Delta_{creep2} = (1.17 - 1.088)(4.076in - 1.978in) + (0.356)(-0.084in) = 0.142in$

Traffic Barrier Deflection, $\Delta_{tb} = -0.247in$

$$\Delta_1 = -1.978in + 4.076in = 2.098in$$

$$\Delta_2 = 2.098in + 2.282in = 4.380in$$

$$\Delta_3 = 4.380 - 0.084in = 4.296in$$

$$\Delta_4 = 4.296in + 0.142in = 4.438in = D_{120}$$

$$\Delta_5 = 4.438in - 1.070in - 0.434in = 2.934in$$

$$\Delta_6 = 2.934 - 0.247in = 2.687in = \Delta_{excess}$$

6.5 Check Required Haunch

The required haunch is $A_{haunch} = A_{slab+fillet} + A_{top\ flange\ effect} + + A_{profile\ effect} + A_{excess\ camber}$

$$A_{haunch} = 8.25in + 0.98in + 0.35in + 2.687in = 12.267\ in$$

The provided haunch is $12.25\ in$. **OK**

6.6 Compute Lower Bound Camber at 40 days

6.6.1 Creep Coefficients

Creep coefficients are computed the same as before, assuming erection at 10 days and deck casting at 40 days.

$$\psi_b(t_d = 10, t_i = 1) = 0.296$$

$$\psi_b(t_d = 40, t_e = 10) = 0.461$$

$$\psi_b(t_f = 40, t_1 = 1) = 0.783$$

6.6.2 Compute Deflections

Creep Deflection, $\Delta_{creep1} = 0.296(4.076in - 1.978in) = 0.621in$

Additional Creep Deflection, $\Delta_{creep2} = (0.783 - 0.296)(4.076in - 1.978in) + (0.461)(-0.084in + 0.0in) = 0.989in$

Traffic Barrier Deflection, $\Delta_{tb} = -0.247in$

$$\Delta_1 = -1.978in + 4.076in = 2.098in$$

$$\Delta_2 = 2.098in + 0.621in = 2.719in$$

$$\Delta_3 = 2.719 - 0.084in = 2.635in$$

$$\Delta_4 = 2.635in + 0.989in = 3.624in = D_{40}$$

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 $= 0.5D_{40} = 1.81\ in$

6.7 Check for Possible Girder Sag

When the screed camber, C, exceeds the deflection at slab casting, D, the girder will have a net downward deflection, also known as sag. The sag condition is most likely to occur for rapidly constructed bridges.

Compare the screed camber to the average value of D_{40} to determine the potential for sag. The average value is 75% $D_{40} = (0.75)(3.624in) = 2.718in$

$$\Delta_{excess} = D - C$$

$$\Delta_5 = 3.624in - 1.070in - 0.434in = 2.12in$$

$$\Delta_6 = 2.12 - 0.247in = 1.873in = \Delta_{excess}$$

$$C = 3.624in - 1.873in = 1.75in$$

$$C < 75\%D_{40} \textbf{ OK}$$

7 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. 7+02.71, Offset 12.029ft L, Elev. 103.509ft

Abutment 2, Sta. 8+27.55, Offset 12.026ft L, Elev. 101.184ft

The slope of the girder is $\frac{101.184ft - 103.509ft}{124.58ft} = -0.01866\frac{ft}{ft}$

The slope-adjusted height of the girder is $66in\left(\sqrt{(-0.01866)^2 + (1)^2}\right) = 66.011in$

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 = $103.509ft - 66.011in\left(\frac{1ft}{12in}\right) - 12.25in\left(\frac{1ft}{12in}\right) = 96.987ft$

Bottom of girder elevation at Abutment 2: Elev = $101.184ft - 66.011in\left(\frac{1ft}{12in}\right) - 12.25in\left(\frac{1ft}{12in}\right) = 94.662ft$

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.

8 Design Summary

$$f'_{ci} = 5.1ksi$$

$$f'_c = 6.3ksi$$

$$\#\,Strands = 38\ (30\ Straight, 8\ Harped)$$

$$Slab\ Offset\ (\text{"A" } Dimension) = 12.25in$$

$$Pick\ Point = 3ft$$

$$Bunk\ Point = 5.5\ ft$$

$$Haul\ Truck\ Configuration = HT40 - 72$$

Use stirrups per WSDOT standard

9 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.

9.1 Inventory Rating

9.1.1 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

$$\phi_c = \phi_s = \phi_n = 1.0$$
$$M_n = 12017.64k \cdot ft$$
$$M_{DC} = 4067.04k \cdot ft$$
$$M_{DW} = 0.0k \cdot ft$$
$$M_{LLIM} = 2135.40 \frac{k \cdot ft}{girder}$$
$$M_{cr} = 8663.74k \cdot ft$$
$$M_u = 8820.75k \cdot ft$$
$$M_{min} = min\begin{cases} M_{cr} \\ 1.33M_u \end{cases} = 8663.74k \cdot ft$$
$$K = \frac{12017.64k \cdot ft}{8663.74k \cdot ft} = 1.387 \therefore 1.0$$
$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.75$$
$$RF = \frac{(1)(1)(1)(1)(12017.65k \cdot ft) - (1.25)(4067.04k \cdot ft) - (1.5)(0k \cdot ft)}{(1.75)(2135.4k \cdot ft)} = 1.86$$

9.1.2 Shear

$$RF = \frac{\phi_c \phi_s \phi_n V_n - \gamma_{DC} V_{DC} - \gamma_{DW} V_{DW}}{\gamma_{LL} V_{LLIM}}$$

At 22.67ft (location where stirrup spacing increases)

$$\phi_c = \phi_s = 1.0, \phi_n = 0.9$$
$$V_n = 409.77kip$$
$$V_{DC} = 83.38kip$$
$$V_{DW} = 0.0k$$
$$V_{LLIM} = 65.62 \frac{kip}{girder}$$
$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.75$$
$$RF = \frac{(1)(1)(0.9)(409.77kip) - (1.25)(83.38kip) - (1.5)(0kip)}{(1.75)(65.62kip)} = 2.31$$

9.1.3 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

$$f_R = f_{limit} - f_{ps} = 0.19\lambda\sqrt{f_c'} - f_{ps}$$
$$f_R = 0.19(1.0)\sqrt{6.3}ksi - (-4.045ksi) = 0.477ksi - (-4.045ksi) = 4.522ksi$$
$$\gamma_{LL} = 1.0$$
$$RF = \frac{4.522ksi - (1.0)(2.73ksi) - 1.0(0ksi)}{(1.0)(1.178ksi)} = 1.52$$

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9.2 Operating Rating

9.2.1 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

$$\phi_c = \phi_s = \phi_n = 1.0$$

$$M_n = 12017.64 k \cdot ft$$

$$M_{DC} = 4067.04 k \cdot ft$$

$$M_{DW} = 0.0 k \cdot ft$$

$$M_{LLIM} = 2135.40 \frac{k \cdot ft}{girder}$$

$$M_{cr} = 8663.74 k \cdot ft$$

$$M_u = 8820.75 k \cdot ft$$

$$M_{min} = min \begin{cases} M_{cr} \\ 1.33 M_u \end{cases} = 8663.74 k \cdot ft$$

$$K = \frac{12017.64 k \cdot ft}{8663.74 k \cdot ft} = 1.387 \therefore 1.0$$

$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.35$$

$$RF = \frac{(1)(1)(1)(1)(12017.65 k \cdot ft) - (1.25)(4067.04 k \cdot ft) - (1.5)(0 k \cdot ft)}{(1.35)(2135.4 k \cdot ft)} = 2.41$$

9.2.2 Shear

$$RF = \frac{\phi_c \phi_s \phi_n V_n - \gamma_{DC} V_{DC} - \gamma_{DW} V_{DW}}{\gamma_{LL} V_{LLIM}}$$

At 22.67ft (location where stirrup spacing increases)

$$\phi_c = \phi_s = 1.0, \phi_n = 0.9$$

$$V_n = 409.77 kip$$

$$V_{DC} = 83.38 kip$$

$$V_{DW} = 0.0 kip$$

$$V_{LLIM} = 65.62 \frac{kip}{girder}$$

$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.75$$

$$RF = \frac{(1)(1)(0.9)(409.77 kip) - (1.25)(83.38 kip) - (1.5)(0 kip)}{(1.35)(65.62 kip)} = 3.07$$

9.3 Legal Loads

Type 3, $M_{LLIM} = 855.77 k \cdot ft$

Type 3S2, $M_{LLIM} = 1076.05k \cdot ft$

Type 3-3, $M_{LLIM} = 1117.01k \cdot ft$

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.

9.3.1 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

$$\phi_c = \phi_s = \phi_n = 1.0$$

$$M_n = 12017.64k \cdot ft$$

$$M_{DC} = 4067.04k \cdot ft$$

$$M_{DW} = 0.0k \cdot ft$$

$$M_{LLIM} = 1117.01 \frac{k \cdot ft}{girder}$$

$$M_{cr} = 8663.74k \cdot ft$$

$$M_u = 8820.75k \cdot ft$$

$$M_{min} = min \begin{Bmatrix} M_{cr} \\ 1.33M_u \end{Bmatrix} = 8663.74k \cdot ft$$

$$K = \frac{12017.64k \cdot ft}{8663.74k \cdot ft} = 1.387 \therefore 1.0$$

$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.45$$

$$RF = \frac{(1)(1)(1)(1)(12017.65k \cdot ft) - (1.25)(4067.04k \cdot ft) - (1.5)(0k \cdot ft)}{(1.45)(1117.01k \cdot ft)} = 4.28$$

9.3.2 Shear

$$RF = \frac{\phi_c \phi_s \phi_n V_n - \gamma_{DC} V_{DC} - \gamma_{DW} V_{DW}}{\gamma_{LL} V_{LLIM}}$$

At 22.67ft (location where stirrup spacing increases)

$$\phi_c = \phi_s = 1.0, \phi_n = 0.9$$

$$V_n = 409.77kip$$

$$V_{DC} = 83.38kip$$

$$V_{DW} = 0.0kip$$

$$V_{LLIM} = 44.97 \frac{kip}{girder}$$

$$\gamma_{DC} = 1.25, \gamma_{DW} = 1.50, \gamma_{LL} = 1.45$$

$$RF = \frac{(1)(1)(0.9)(409.77kip) - (1.25)(83.38kip) - (1.5)(0k)}{(1.45)(44.97kip)} = 6.47$$

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9.3.3 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 loading.

$$\Delta f_{pLL} = \frac{E_p}{E_c} \frac{M_{LLIM}(Y_{bc} - Y_{bg} + e)}{I_c} = \frac{28500 ksi}{5292.088\ ksi} \frac{(1117.01 k \cdot ft)(43.761 in - 31.804 in + 28.646 in)}{952196.3 in^4} \left(\frac{12 in}{1 ft}\right)$$
$$= 3.078 ksi$$

$$P = (8.246 in^2)(202.5 ksi - 31.992 ksi + 3.078 ksi) = 1431.39 kip$$

$$f_{ps} = -\frac{1431.39 kip}{874.531 in^2} - \frac{(1431.39 kip)(28.646 in)}{17493.0 in^3} = -3.981 ksi$$

$$f_R = 0.19(1.0)\sqrt{6.3} ksi - (-3.981 ksi) = 0.477 ksi - (-3.981 ksi) = 4.458 ksi$$

$$\gamma_{LL} = 1.0$$

$$RF = \frac{4.458 ksi - (1.0)(2.73 ksi) - 1.0(0 ksi)}{(1.0)(0.616 ksi)} = 2.80$$

9.4 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 $0.9 f_y$. The analysis method used by PGSuper follows MBE A3.13.4.2b.

$$f_r = 0.9 f_y = (0.9)(0.9) f_{pu} = (0.9)(0.9)(270 ksi) = 218.7 ksi$$

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, $M_{LLIM} = 1570.93 k \cdot ft$ per girder.

For OL2, $M_{LLIM} = 2727.88 k \cdot ft$ per girder

$$M_{bcr} = (1.0)(4067.04 k \cdot ft) + (1.0)(0) + (1.0)(2727.88 k \cdot ft) - 8663.74 k \cdot ft = -1868.08 k \cdot ft$$

Because $M_{bcr} < 0$, the loads aren't enough to cause cracking, so take $M_{bcr} = 0.0 k \cdot ft$

The additional stress transferred to the reinforcement due to cracking is

$$f_{bcr} = \frac{E_s}{E_g} \frac{M_{bcr}(d_s - c)}{I_{cr}} = 0.0 ksi$$

$$f_s = f_{pe} + f_{bcr}$$

Compute the effective prestress

For OL1

$$\Delta f_{pLL} = \frac{E_p}{E_c}\frac{M_{LLIM}\left(Y_{bc} - Y_{bg} + e\right)}{I_c} = \frac{28500ksi}{5292.088\ ksi}\frac{(1570.93k \cdot ft)(43.761in - 31.804in + 28.646in)}{952196.3in^4}\left(\frac{12in}{1ft}\right)$$
$$= 4.329ksi$$

$$f_{pe} = 202.5ksi - 31.992ksi + 4.329ksi = 174.837\text{ ksi}$$

$$f_s = f_{pe} + f_{brc} = 174.837ksi + 0ksi = 174.837ksi$$

For OL2

$$\Delta f_{pLL} = \frac{E_p}{E_c}\frac{M_{LLIM}\left(Y_{bc} - Y_{bg} + e\right)}{I_c} = \frac{28500ksi}{5292.088\ ksi}\frac{(2727.88k \cdot ft)(43.761in - 31.804in + 28.646in)}{952196.3in^4}\left(\frac{12in}{1ft}\right)$$
$$= 7.517ksi$$

$$f_{pe} = 202.5ksi - 31.992ksi + 7.517ksi = 178.025\text{ ksi}$$

$$f_s = 178.025ksi$$

Yield stress ratio

$$SR = \frac{f_r}{f_s}$$

OL1

$$SR = \frac{218.7ksi}{174.837ksi} = 1.25$$

OL2

$$SR = \frac{218.7ksi}{178.025ksi} = 1.23$$

10 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

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