CHAPTER 16.1 STRENGTHENING STEEL GIRDERS FOR LIVE LOADS

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16.1.1 INTRODUCTION

With the introduction of the California permit design live load P-13 in the early 1970s, and California permit load P-15 in 2006, many steel girder bridges built before that time are now rated structurally deficient in live load capacity for permit trucks. Steel girder bridges on the newly designated freight corridor routes are in the process of being strengthened to full permit, P-15 capacity. To meet this great challenge, Caltrans published Structure Technical Policy (STP) 16.6: *Design Criteria for Strengthening Steel Girders for Live Loads* in 2021 (Caltrans, 2021). This Chapter provides guidelines and several examples for typical strengthening of composite and noncomposite steel girder bridges for live loads. For further detailed discussions, references may be made elsewhere (Klaiber, et al., 1987; Silano, 1993; Xanthakos, 1995; Dorton and Reel, 1997; Klaiber and Wipf, 2000; Reid, Milne, and Craig, 2001; Khan, 2010; Newman, 2012; Cheng, Duan and Najiar, 2014).

Before starting any detailed analysis for strengthening, the designer should access the *Bridge Inspection Records Information System* (BIRIS) to obtain as-built bridge plans, shop drawings, and bridge inspection and maintenance information such as the present and past condition of the bridge and the current live load ratings. The designer should be aware that; however, the load rating report not only lists one of the worst deficiencies, but deficiencies that may exist in other different locations. The Structure Maintenance and Investigation (SM&I) office may have identified additional locations in their backup calculations.

16.1.2 STRENGTHENING METHODS

The strengthening of existing steel girders shall be designed in accordance with STP 16.6 Caltrans, 2021) or the project-specific design criteria.

The following typical strengthening methods should be selected based on the project status.

16.1.2.1 Flexural Strengthening

- ***Making A Composite Section:*** For a noncomposite section, the most effective method is to install shear studs to increase moment resistance by making a composite section. This method can be used when a noncomposite deteriorated concrete deck is to be replaced by a new deck or a noncomposite superstructure is to be replaced. If a concrete deck is still sound, composite action can be added by simply removing strips of the concrete deck, adding shear connectors on the girder top flanges, and grouting the strips, as shown in Figure 16.1.2-1. Shear studs can be installed by double-nut bolt, high-tension friction grip bolt, expansion anchor, or undercut anchor, as shown in Figure 16.1.2-2.

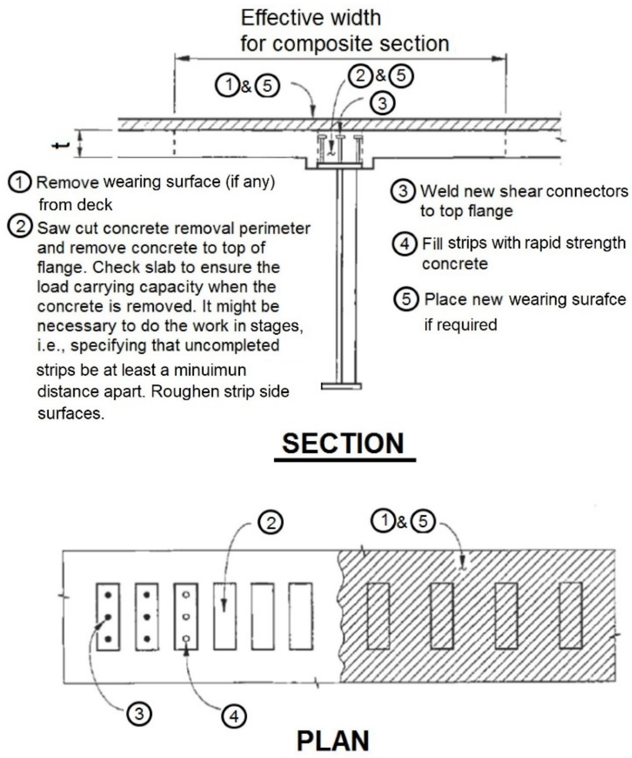


Figure 16.1.2-1 Making A Composite Section

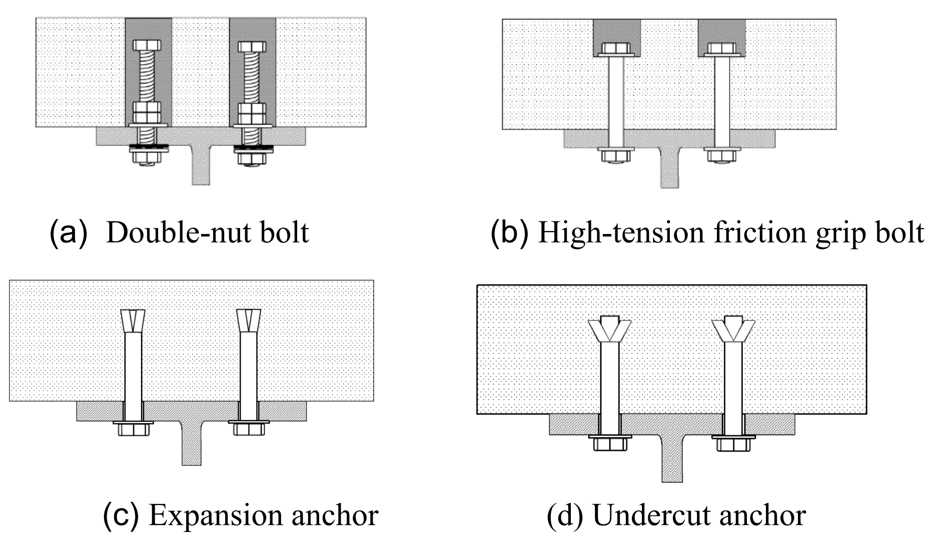
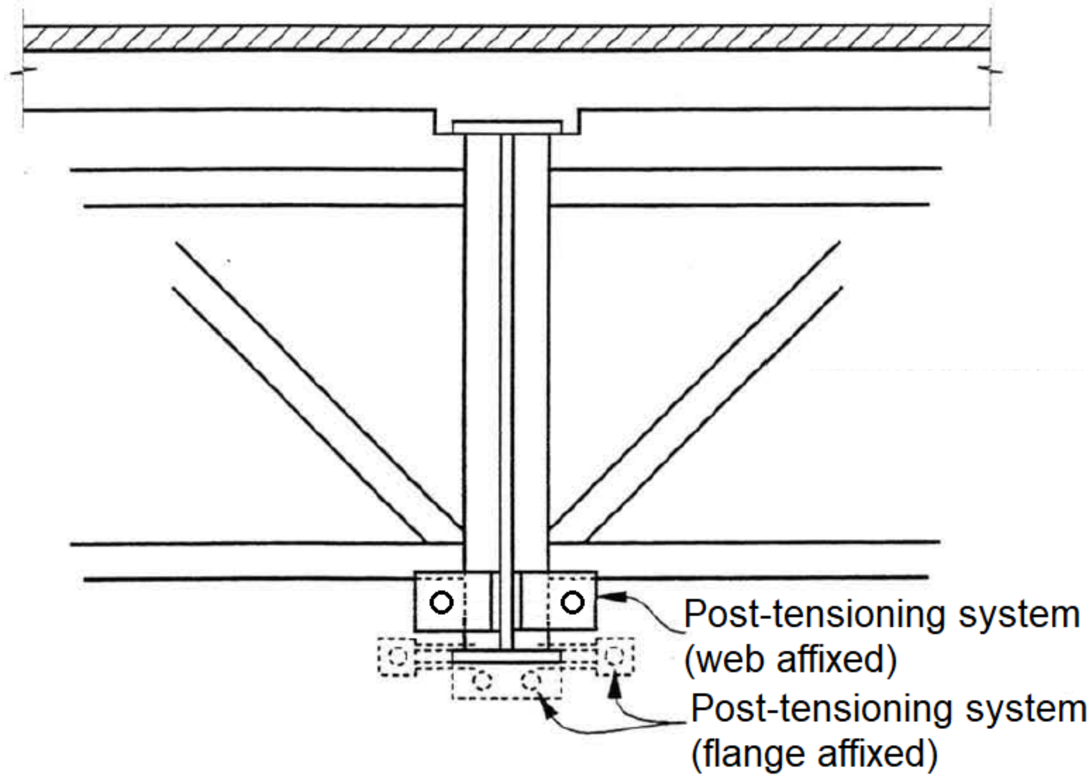


Figure 16.1.2-2 Bolted Shear Studs (Kwon, et al., 2007)

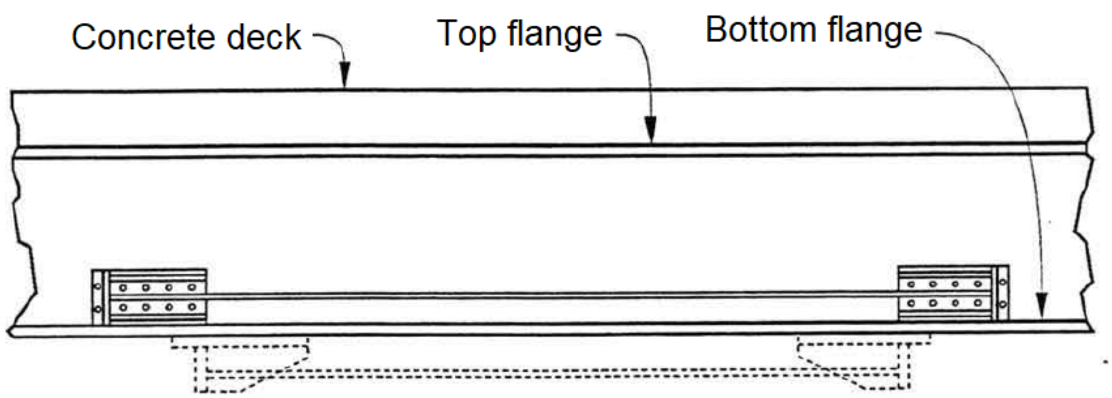
- ***Adding Transverse Bracings**:* For a noncomposite girder or composite girders in negative moment regions, adding discrete bracings, such as cross frames and diaphragms, may be a cost effective method to increase the flexural resistance of an existing girder that is controlled by lateral torsional buckling. In other words, there is no value for adding transverse bracings if unbraced length has no effect on the flexural resistance.
- ***Adding Cover Plates:*** Adding steel cover plates to the steel girder flange to increase the flexural resistance is effective when an existing steel girder is controlled by either flange yielding or flange local buckling. To avoid difficulties and potentially poor quality of field overhead welding, welded cover plates are not permitted per STP 16.6. While it is very easy to bolt the new cover plate to the existing flange, one major disadvantage of using the bolted cover plate in the tension flange is that the moment capacity is limited to the yielding moment as specified in Article 6.10.1.8 and Caltrans STP 16.6. This yield moment limitation may result in a larger cover plate, while Article 6.10.12.1 specifies that the maximum thickness of a single cover plate on a flange shall not be greater than two times the thickness of the flange to which the cover plate is attached.
- ***Post-tensioning:*** Using post-tensioning to increase the flexural resistance of an existing steel girder may be a better solution when the steel girder is controlled by tension flange yielding or compression flange overstress. Prestressing steel should be placed in the tension zone at the strength limit state, as far as possible from the neutral axis. For continuous steel girders, secondary effects due to post-tensioning shall be considered. Prestressing steel shall be anchored beyond the point at which it is no longer required. Strands should not be used in short lengths as the anchor set losses will be excessive. High strength rods have negligible anchor set losses. Figures 16.1.2-3 and 16.1.2-4 show two typical post-tensioning system layouts. The king post system shown in Figure 16.1.2-4 should not be used when vertical clearance is an issue.
- ***Combination of Two or More Methods***: Two or more methods may be combined on a project-by-project basis.

16.1.2.2 Shear Strengthening

- ***Adding Transverse Stiffeners:*** Transverse stiffeners may be added to increase the shear resistance of an existing steel girder that is controlled by shear buckling. There is no value in adding transverse stiffeners when the steel girder is controlled by shear yielding.
- ***Adding Web Plates.*** Web plates can be added to increase the shear resistance of an existing steel girder that is controlled by either shear yielding or shear buckling.

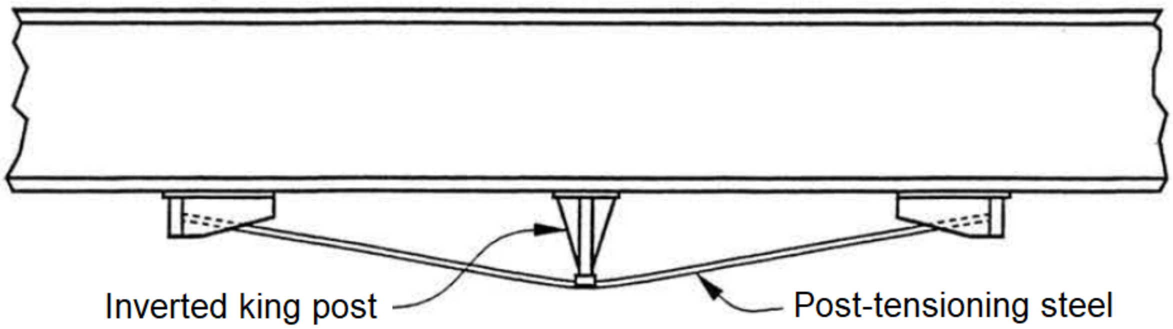


(a) Girder Section

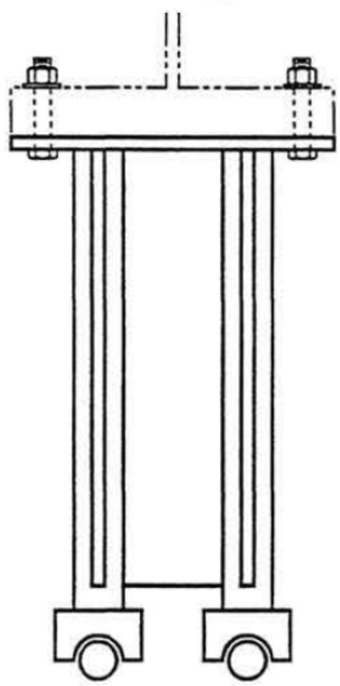


(b) Girder Elevation

Figure 16.1.2-3 Post-tensioning System Affixed at Web or Flange



(a) Girder Elevation



(b) King Post Section

Figure 16.1.2-4 Post-tensioning System Using King Post

16.1.2.3 Strengthening Using Fiber Reinforced Polymer (FRP)

Although the methods discussed in Sections 16.1.2.1 and 16.1.2.2 have often been used in bridge strengthening, they do not cover all the strengthening methods. Depending on the special situation of each project, other methods may be used.

A new technology using FRP materials for strengthening steel girders has been under development for the past 20 years (Miller, et al., 2001; Sen, et al., 2001; Tavakkolizadeh and Saadatmanesh, 2003a,b; Phares, et al., 2003). Technological advancements, such as the development of high modulus carbon FRP (HM CFRP) (Rizkalla, et al., 2007; Schnerch, et al. 2007) and carbon FRP (CFRP) strand sheets (Tabrizi, et al., 2015) further improved the effectiveness of using CFRP materials to strengthen steel bridge girders. Selvaraj and Madhavan (2019) reported a study on strengthening steel girders with low-modulus CFRP. To apply this new technology in practice, several key issues including the bounding surface preparation requirements, effective methods for inspecting FRP-strengthened steel beams, the integrity of the bonded interface, the durability and the life-cycle cost, and LRFD compatible design criteria still need to be fully developed.

16.1.3 FLEXURAL STRENGTHENING DESIGN EXAMPLE 1 – COVER PLATES

The following is an example of the flexural strengthening of a simple span composite steel girder bridge using cover plates to increase flexural capacity due to increased live loads.

16.1.3.1 Existing Steel Girder Bridge Data

Bridge Type:	Simple span, multi steel-concrete composite girder bridge		
Span Length:	90 ft between the center line of bearings		
Bridge Width:	33'-8"		
Year Built:	1950		
Girder:	Composite steel girder		
Live Load:	H20-S16-44		
Reinforced Concrete:	f_s =	20,000	psi
	f_c =	1,200	psi
Structural Steel:	f_s =	18,000	psi
	F_y =	33,000	psi

Typical section and girder data are shown in Figures 16.1.3-1 and 16.1.3-2.

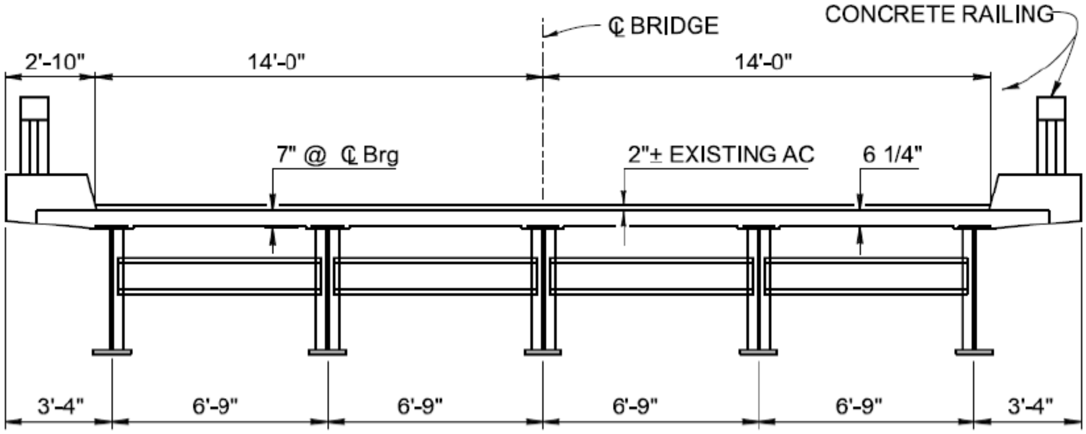


Figure 16.1.3-1 Typical Section

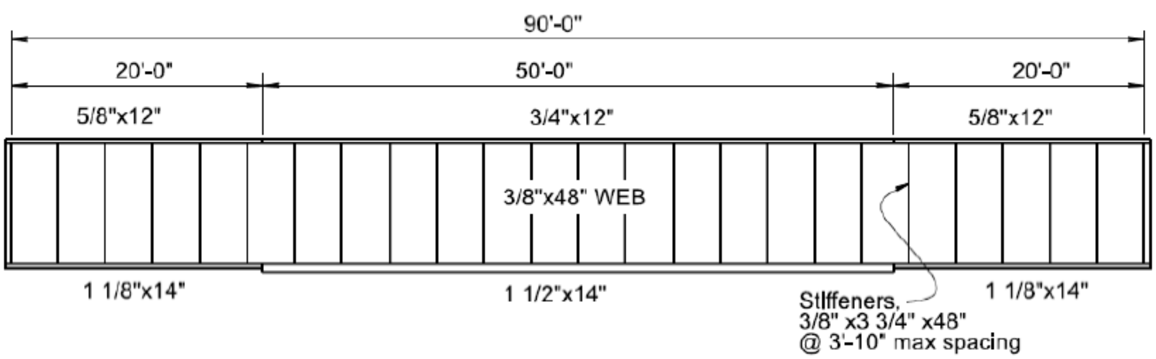


Figure 16.1.3-2 Interior Girder Span

16.1.3.2 Design Requirement

Perform the following strengthening design portions for an interior plate girder in accordance with STP 16.6 (Caltrans 2021) and AASHTO-CA BDS-8 (AASHTO, 2017; Caltrans, 2019). A similar procedure can be used for strengthening an exterior girder, but it is not illustrated here.

Step 1: Determine Material Properties

Step 2: Perform Load and Structural Analysis

Step 3: Calculate Live Load Distribution Factors

Step 4: Determine Load and Resistance Factors and Load Combinations

Step 5: Calculate Factored Moments and Shears - Strength Limit States

Step 6: Calculate Factored Moments and Shears - Fatigue Limit States

Step 7: Calculate Factored Moments and Shears - Service Limit State II

Step 8: Check Flexural Resistances of Composite Sections of Existing Girder

Step 9: Design Cover Plate

Step 10: Design Cover Plate Connection

The following notations are used in this example:

"AASHTO xxx-x" denotes "AASHTO Equation xxx-x"

"CA xxx" denotes "California Amendment Article xxx"

"CA xxx-x" denotes "California Amendment Equation xxx-x"

"STP xxx" denotes "Caltrans Structure Technical Policy Article xxx"

16.1.3.3 Determine Material Properties

Per STP 16.6.5, actual material properties for existing structures, F_{ya}, F_{ua}, and f'_{ca} should be obtained from physical tests if feasible. In the absence of test results for this example, they are determined as follows:

As-built concrete compressive strength: $f'_c = 2.5f_c = (2.5)(1{,}200) = 3{,}000 \text{ psi} = 3.0 \text{ ksi}$

Actual concrete compressive strength: $f'_{ca} = 1.2f'_c = (1.2)(3.0) = 3.6 \text{ ksi}$

Unit weight of concrete: $w_c =$ 0.15 kcf

Modulus of elasticity of concrete:

$$E_{ca} = 33{,}000K_1 w_c^{1.5}\sqrt{f'_{ca}} = (33{,}000)(1.0)(0.15)^{1.5}\sqrt{3.6} = 3{,}637 \text{ ksi}$$

(AASHTO C5.4.2.4-2)

Actual yield strength of existing steel: $F_{ya} =$ 33 ksi

Actual tensile strength of existing steel: $F_{ua} =$ 60 ksi

Modulus of elasticity of steel: E_s = 29,000 ksi

Modular Ratio $n = \dfrac{E_s}{E_{ca}} = \dfrac{29{,}000}{3{,}637} = 7.97$, Use n = 8.

New steel - ASTM A709 Grade 36. Since Caltrans Standard Specifications (Caltrans, 2018) 55-1.02D(3) specifies that unless otherwise described, structural steel plates, shapes, and bars must comply with ASTM A709/A709M, Grade 50, ASTM A709 Grade 36 shall be shown clearly on the plans. The material properties are as follows:

Specified minimum yield strength of steel: $F_y =$ 36 ksi

Unit weight of steel: $w_s =$ 0.49 kcf

16.1.3.4 Perform Load and Structural Analysis

16.1.3.4.1 Calculate Permanent Loads for an Interior Girder

The permanent load or dead load of an interior girder includes *DC* and *DW*. *DC* is dead load of structural components and nonstructural attachments. *DW* is dead load of the wearing surface.

DC1 - Structural dead load, acting on the noncomposite section

- Concrete Slab

  Concrete slab thickness: t_s = 6.25 in.

  Girder spacing: S = 6.75 ft

  Weight of deck slab: $W_s = t_s S w_c = (6.25/12)(6.75)(150) = 527$ lb/ft

- Girder Self Weight

  Section at ends:

  Top flange PL 5/8X12

  Top flange width: b_{fc} = 12 in.

  Top flange thickness: t_{fc} = 0.625 in.

  Bottom flange PL 1 1/8X14

  Bottom flange width: b_{ft} = 14 in.

  Bottom flange thickness: t_{ft} = 1.125 in.

  Web PL 3/8X48

  Web thickness: t_w = 0.375 in.

Web depth: D = 48 in.

Gross section area:

$$A_{ge} = (12 \times 0.625) + (48 \times 0.375) + (14 \times 1.125) = 41.25 \text{ in.}^2$$

Weight of steel girder: $W_{ge} = A_{ge} w_s = (41.25)(490/144) = 140$ lb/ft

Section at midspan:

Top flange PL 3/4X12

Top flange width: b_{fc} = 12 in.

Top flange thickness: t_{fc} = 0.75 in.

Bottom flange PL 1 1/2X14

Bottom flange width: b_{ft} = 14 in.

Bottom flange thickness: t_{ft} = 1.50 in.

Web PL 3/8X48

Web thickness: t_w = 0.375 in.

Web depth: D = 48 in.

Gross section area:

$$A_{gm} = (12 \times 0.75) + (48 \times 0.375) + (14 \times 1.5) = 48 \text{ in.}^2$$

Weight of steel girder: $W_{gm} = A_{gm} w_s = (48)(490/144) = 163$ lb/ft

- Stiffener Weight

Transverse Stiffener: 3/8x3 3/4x48 @3'-10" two sides. Total 50 stiffeners.

Stiffener width: b_t = 3.75 in.

Stiffener thickness: t_t = 0.375 in.

Stiffener volume: $V_{st} = (0.375)(3.75)(48) = 67.5 \text{ in.}^3$

Total stiffener weight:

$$W_{st} = \frac{(67.5)(50)(490)}{12^3 (90)} = 11 \text{ lb/ft}$$

- Bracing Weight

Assume W_{br} = 10 lb/ft

- Miscellaneous Dead Load for Haunch, Welds, etc.

Assume W_{misc} = 10 lb/ft

Total DC1

End Span

$$DC1_{end} = W_s + W_{ge} + W_{st} + W_{br} + W_{misc}$$
$$= 527 + 140 + 11 + 10 + 10 = 698 \text{ lb/ft} = 0.698 \text{ k/ft}$$

Midspan

$$DC1_{mid} = W_s + W_{gm} + W_{st} + W_{br} + W_{misc}$$
$$= 527 + 163 + 11 + 10 + 10 = 721 \text{ lb/ft} = 0.721 \text{ k/ft}$$

DC1 is shown in Figure 16.1.3-3.

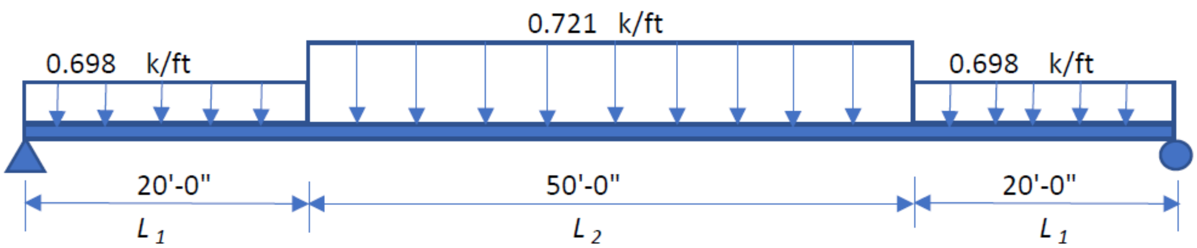


Figure 16.1.3-3 DC1 Dead Load

DC2 - Nonstructural dead load, acting on the long-term composite section

Assume one side barrier: $W_{barrier}$ = 250 lb/ft

Assume one side railing: $W_{railing}$ = 100 lb/ft

$$DC2_{total} = 2\left(W_{barrier} + W_{railing}\right) = (2)(250 + 100) = 700 \text{ lb/ft} = 0.7 \text{ k/ft}$$

Assume *DC2* is distributed equally to all girders and *DC2* for an interior girder as:

$$DC2 = DC2_{total} / 5 = 0.7 / 5 = 0.14 \text{ k/ft}$$

DC2 is shown in Figure 16.1.3-4.

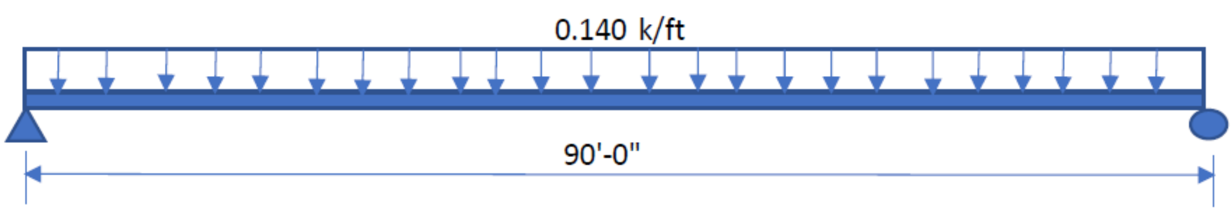


Figure 16.1.3-4 DC2 Dead Load

DW - Considering Future wearing surface 35 psf (Ignore the weight of existing AC overlay)

Deck width from curb to curb = 28 ft

$$DW_{total} = (35)(28) = 980 \text{ lb/ft} = 0.98 \text{ k/ft}$$

Assume *DW* is distributed equally to all girders and *DW* for an interior girder is as:

$$DW = 0.98 / 5 = 0.196 \text{ k/ft}$$

DW is shown in Figure 16.1.3-5.

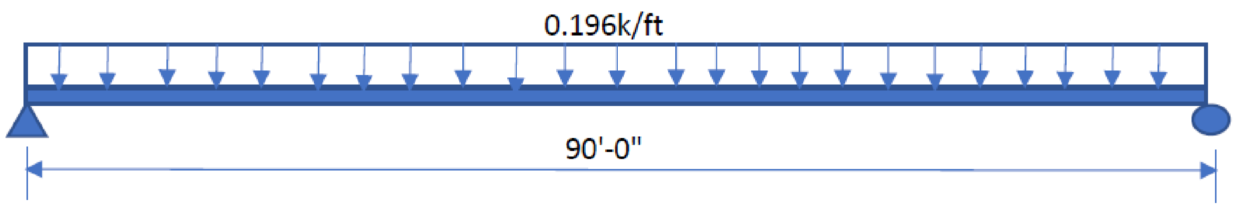


Figure 16.1.3-5 DW Load

16.1.3.4.2 Live Load and Dynamic Load Allowance

For live load upgrade, HL-93 (Article 3.6.1.2) and Caltrans P15 (CA Article 3.6.1.8) are considered for this example. To consider the wheel load impact from moving vehicles, the dynamic load allowance is as follows:

IM	=	33%	for the strength I limit state	(CA Table 3.6.2.1-1)
IM	=	25%	for the strength II limit state	
IM	=	15%	for the fatigue limit states	

Unfactored dead load moments and shears for an interior girder are calculated and shown in Table 16.1.3-1.

Table 16.1.3-1 Unfactored Dead Load Moments and Shears for an Interior Girder

Point x/L	Moment			Shear		
	DC1	*DC2*	*DW*	*DC1*	*DC2*	*DW*
	M_{DC1} (kip-ft)	M_{DC2} (kip-ft)	M_{DW} (kip-ft)	V_{DC1} (kip)	V_{DC2} (kip)	V_{DW} (kip)
0.0	0	0	0	32.0	6.3	8.8
0.1	260	51	71	25.7	5.0	7.1
0.2	463	91	127	19.4	3.8	5.3
0.3	609	119	167	13.0	2.5	3.5
0.4	697	136	191	6.5	1.3	1.8
0.5	726	142	198	0	0.0	0.0
0.6	697	136	191	-6.5	-1.3	-1.8
0.7	609	119	167	-13.0	-2.5	-3.5
0.8	463	91	127	-19.4	-3.8	-5.3
0.9	260	51	71	-25.7	-5.0	-7.1
1.0	0	0	0	-32.0	-6.3	-8.8

In this design example, a live load analysis is performed by the CTBridge computer program. Unfactored live load moments and shears for one lane with the dynamic load allowance are shown in Table 16.1.3-2.

Table 16.1.3-2 Unfactored Live Load Moments and Shears for One Lane with Dynamic Load Allowance

Point x/*L*	Moment (*LL+IM*)		Fatigue Moment (*LL+IM*)		Shear (*LL+IM*)		Fatigue Shear (*LL+IM*)	
	HL93	*P15*	*HL93*	*P9*	*HL93*	*P15*	*HL93*	*P9*
	M_{HL93} (k-ft)	M_{P15} (k-ft)	M_{FHL93} (k-ft)	M_{FP9} (k-ft)	V_{HL93} (kip)	V_{P15} (kip)	V_{FHL93} (kip)	V_{FP9} (kip)
0.0	0	0	0	0.0	112.1	195.8	64.4	174.3
0.1	920	1519	520	1368	100.1	168.8	57.8	152.0
0.2	1615	2430	891	2236	85.5	135.0	49.5	124.2
0.3	2086	3341	1144	2987	71.5	108.0	41.2	99.4
0.4	2363	3645	1270	3353	58.1	81.0	32.9	74.5
0.5	2430	3949	1247	3488	-45.3	-60.8	-24.6	-55.9
0.6	2363	3645	1270	3353	-58.1	-81.0	-32.9	-74.5
0.7	2086	3341	1144	2987	-71.5	-108.0	-41.2	-99.4
0.8	1615	2430	891	2236	-85.5	-135.0	-49.5	-124.2
0.9	920	1519	520	1368	-100.1	-168.8	-57.8	-152.0
1.0	0	0	0	0	-112.1	-195.8	-64.4	-174.3

16.1.3.5 Calculate Live Load Distribution Factors

To calculate the live load distribution factors, we need to calculate the longitudinal stiffness parameter, K_g, as follows.

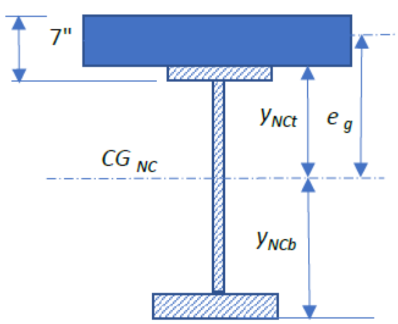
16.1.3.5.1 Existing Section Properties at Midspan Section

The longitudinal stiffness parameter, K_g is estimated per AASHTO Equation 4.6.2.2.1-1, as shown in Table 16.1.3-3.

Note: In determining the stiffness parameter, K_g, some factors may be appropriately varied along the span to include consideration of the average properties of the girder (Article C4.6.2.2.1). It is not recommended to use simplifications in Table 4.6.2.2.1-3 for a final design.

Table 16.1.3-3 Existing Section Properties at Midspan Section

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	$y_i - y_{NCb}$ (in.)	$A_i(y_i - y_{NCb})^2$ (in.^{4})	I_o (in.^{4})
Top Flange (3/4x12)	9.00	49.88	448.88	30.63	8,445	0.42
Web (3/8x48)	18	25.50	459.00	6.26	705	3,456
Bottom Flange (1 1/2x14)	21.00	0.75	15.75	-18.49	7,181.18	3.94
Total (Σ)	48.00		923.63		16,331	3,460



y_i = Component CG to the bottom of bottom flange

I_o = Moment of inertia of component about its CG

$y_{NCb} = \Sigma A_i y_i / \Sigma A_i$ = 19.24 in.

$y_{NCt} = (0.75+48+1.5)-19.24$ = 31.01 in.

$I_{NC} = \Sigma I_0 + \Sigma A_i (y_i - y_{NCb})^2$ = 19,792 in.^{4}

$e_g = 31.01+7-6.25/2-0.75$ = 34.1 in.

n = Modular Ratio = 8

$K_g = n(I_{NC} + Ae_g^2)$ = 605,713 in.^{4}

(AASHTO 4.6.2.2.1-1)

16.1.3.5.2 Calculate Live Load Distribution Factors

From AASHTO Table 4.6.2.2.1-1, the cross-section of this example is Type "*a*" structure and the number of girders N_b = 5.

Strength Limit States - Live Load Moment Distribution Factors (AASHTO Tables 4.6.2.2.2b-1)

One design lane loaded: $$DF_m = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} = 0.433$$

Two or more design lanes loaded :

$$DF_m = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} = 0.602 \qquad \text{Control}$$

Strength Limit States - Live Load Shear Distribution Factors (AASHTO Table 4.6.2.2.3a-1)

One design lane loaded: $DF_v = 0.36 + \frac{S}{25} = 0.63$

Two or more design lanes loaded:

$$DF_v = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 = 0.725 \qquad \text{Control}$$

where:

L = Span length for moment is being calculated = 90 ft

S = Girder spacing = 6.75 ft

t_s = Concrete slab thickness = 6.25 in.

K_g = Stiffness parameter = 605,713 in.^{4}

Note: the above equations have the multiple presence factor, *m*, included in them (Article C3.6.1.1.2).

Fatigue Limit States - Live Load Moment Distribution Factors

For the fatigue limit states, the live load is one HL-93 or one P9 truck as specified in CA 3.6.1.4.1, a multiple presence factor of 1.2 should be divided from above one lane factors (Article 3.6.1.1.2).

Fatigue Limit States - Live Load Moment Distribution Factor

$$DF_m = \frac{0.433}{1.2} = 0.361$$

Fatigue Limit States - Live Load Shear Distribution Factors

$$DF_m = \frac{0.63}{1.2} = 0.525$$

Live load distribution factors are summarized in Table 16.1.3-4.

Table 16.1.3-4 Summary of Live Load Distribution Factors

Limit States	DF_m	DF_v
Strength Limit States	0.602	0.725
Service Limit States	0.602	0.725
Fatigue Limit States	0.361	0.525

16.1.3.6 Determine Load and Resistance Factors and Load Combinations

16.1.3.6.1 Design Equation

$$\Sigma \eta_i \gamma_i Q_i \leq \phi R_n = R_r \quad \text{(AASHTO 1.3.2.1-1)}$$

where:

η_i = load modifier factor = 1.0

γ_i = load factor

Q_i = force effect

ϕ = resistance factor

R_n = nominal resistance

R_r = factored resistance

16.1.3.6.2 Determine Applicable Resistance Factors for Strength Limit State

According to Article 6.5.4.2, the following resistance factors are used for the strength limit states in this example.

For flexure ϕ_f = 1.00

For tension, fracture in net section ϕ_u = 0.80

For tension, yielding in gross section ϕ_y = 0.95

For shear connector ϕ_{sc} = 0.85

For ASTM F3125 bolts in shear ϕ_s = 0.80

For bolt bearing on material ϕ_{bb} = 0.80

For weld metal in fillet weld

– shear in the throat of weld metal ϕ_{e2} = 0.80

16.1.3.6.3 Determine Applicable Load Factors and Load Combinations

According to CA Table 3.4.1-1, the following five load combination group considered for this example:

Strength I: $1.25DC+1.5DW+1.75(DF)(LL+IM)_{HL\text{-}93}$

Strength II: $1.25DC+1.5DW+1.35(DF)(LL+IM)_{P15}$

Service II: $1.0DC+1.0DW+1.3(DF)(LL+IM)_{HL\text{-}93}$

Fatigue I: $1.75(DF)(LL+IM)_{HL\text{-}93}$

Fatigue II: $1.0(DF)(LL+IM)_{P9}$

16.1.3.7 Calculate Factored Moments and Shears – Strength Limit States

Using load combinations as discussed in Section 16.1.3.6.3, factored moments and shears for strength limit states I and II are calculated and listed in Tables 16.1.3-5 and 16.1.3-6, respectively.

Table 16.1.3-5 Factored Moment Envelopes for Interior Girder

Point x/L	Dead Load			Live Load (LL+IM)		Load Combination	
	DC1	*DC2*	*DW*	*HL93*	*P15*	Strength I	Strength II
	$1.25M_{DC1}$ (kip-ft)	$1.25M_{DC2}$ (kip-ft)	$1.5M_{DW}$ (kip-ft)	$1.75DFM_{HL93}$ (kip-ft)	$1.35DFM_{P15}$ (kip-ft)	M_u (kip-ft)	M_u (kip-ft)
0.0	0	0	0	0.0	0.0	0.0	0.0
0.1	325	64	107	969	1235	1465	1731
0.2	579	113	191	1702	1976	2585	2858
0.3	761	149	250	2199	2717	3359	3877
0.4	871	170	286	2491	2964	3817	4290
0.5	907	177	298	2562	3211	3944	4593
0.6	871	170	286	2491	2964	3817	4290
0.7	761	149	250	2199	2717	3359	3877
0.8	579	113	191	1702	1976	2585	2858
0.9	325	64	107	969	1235	1465	1731
1.0	0	0	0	0.0	0.0	0.0	0.0

Table 16.1.3-6 Factored Shear Envelopes for Interior Girder

Point x/L	Dead Load			Live Load (LL+IM)		Load Combination	
	DC1	DC2	DW	*HL93*	*P15*	Strength I	Strength II
	$1.25V_{DC1}$ (kip)	$1.25V_{DC2}$ (kip)	$1.5V_{DW}$ (kip)	$1.75DFV_{HL93}$ (kip)	$1.75DFV_{P15}$ (kip)	V (kip)	V (kip)
0.0	40.0	7.9	13.2	142.3	191.7	203.4	252.8
0.1	32.1	6.3	10.6	127.0	165.2	176.0	214.3
0.2	24.3	4.7	7.9	108.5	132.2	145.5	169.1
0.3	16.2	3.2	5.3	90.8	105.7	115.4	130.4
0.4	8.1	1.6	2.6	73.7	79.3	86.1	91.6
0.5	0.0	0.0	0.0	-57.5	-59.5	-57.5	-59.5
0.6	-8.1	-1.6	-2.6	-73.7	-79.3	-86.1	-91.6
0.7	-16.2	-3.2	-5.3	-90.8	-105.7	-115.4	-130.4
0.8	-24.3	-4.7	-7.9	-108.5	-132.2	-145.5	-169.1
0.9	-32.1	-6.3	-10.6	-127.0	-165.2	-176.0	-214.3
1.0	-40.0	-7.9	-13.2	-142.3	-191.7	-203.4	-252.8

16.1.3.8 Calculate Factored Moments and Shears - Fatigue Limit States

Using load combinations as discussed in Section 16.1.3.6.3, factored moments and shears for fatigue limit states are calculated and listed in Tables 16.1.3-7 and 16.1.3-8, respectively.

Table 16.1.3-7 Fatigue Limit State -Factored Moment and Shear by Live Load only

Point x/L	Live Load Moment *(DF)(LL+IM)*		Fatigue Moment		Live Load Shear *(DF)(LL+IM)*		Factored Fatigue Shear	
	HL93	*P9*	Fatigue I	Fatigue II	*HL93*	*P9*	Fatigue I	Fatigue II
	DFM_{FHL93} (kip-ft)	DM_{FP9} (kip-ft)	$1.75DFM_{FHL93}$ (kip-ft)	DM_{FP9} (kip-ft)	DFV_{FHL93} (kip)	DFV_{FP9} (kip)	$1.75DFV_{FHL93}$ (kip)	DFV_{FP9} (kip)
0.0	0.0	0.0	0.0	0.0	33.8	91.5	59.2	91.5
0.1	188	494	328	494	30.3	79.8	53.1	79.8
0.2	321	807	563	807	26.0	65.2	45.5	65.2
0.3	413	1078	722	1078	21.6	52.2	37.9	52.2
0.4	458	1210	802	1210	17.3	39.1	30.2	39.1
0.5	450	1259	787	1259	-12.9	-29.3	-22.6	-29.3
0.6	458	1210	802	1210	-17.3	-39.1	-30.2	-39.1
0.7	413	1078	722	1078	-21.6	-52.2	-37.9	-52.2
0.8	321	807	563	807	-26.0	-65.2	-45.5	-65.2
0.9	188	494	328	494	-30.3	-79.8	-53.1	-79.8
1.0	0.0	0.0	0.0	0.0	-33.8	-91.5	-59.1	-91.5

V_u, shear due to the unfactored dead load plus the factored fatigue load (Fatigue I) is also calculated in Table 16.1.3.8 for checking the special fatigue requirement for webs as required by Article 6.10.5.3.

$$V_u = V_{DC1} + V_{DC2} + V_{DW} + (1.75)(DF_v)V_{FHL93}$$

Table 16.1.3-8 Special Fatigue Shear Requirement for Web Check

Point x/L	Dead Load Shear			Fatigue I Shear	Special Shear
	DC1	*DC2*	*DW*		
	V_{DC1} (kip)	V_{DC1} (kip)	V_{DW} (kip)	V_u (kip)	V_u (kip)
0.0	32.0	6.3	8.8	59.2	106.3
0.1	25.7	5.0	7.1	53.1	90.9
0.2	19.4	3.8	5.3	45.5	74.0
0.3	13.0	2.5	3.5	37.9	56.9
0.4	6.5	1.3	1.8	30.2	39.8
0.5	0.0	0.0	0.0	-22.6	-22.6
0.6	-6.5	-1.3	-1.8	-30.2	-39.8
0.7	-13.0	-2.5	-3.5	-37.9	-56.9
0.8	-19.4	-3.8	-5.3	-45.5	-74.0
0.9	-25.7	-5.0	-7.1	-53.1	-90.9
1.0	-32.0	-6.3	-8.8	-59.1	-106.3

16.1.3.9 Calculate Factored Moments and Shears - Service Limit State II

Factored moments and shears at service limit state II are calculated in Tables 16.1.3-9 and 16.1.3-10, respectively.

Table 16.1.3-9 Factored Moment Envelopes for Interior Girder

Point x/L	Dead Load			Live Load	Load Combination
	DC1	*DC2*	*DW*	*HL93*	Service II
	M_{DC1} (kip-ft)	M_{DC2} (kip-ft)	M_{DW} (kip-ft)	$1.3\ DF\ M_{HL93}$ (kip-ft)	M_u (kip-ft)
0.0	0	0	0	0.0	0.0
0.1	260	51	71	720	1102
0.2	463	91	127	1265	1945
0.3	609	119	167	1633	2528
0.4	697	136	191	1850	2873
0.5	726	142	198	1903	2969
0.6	697	136	191	1850	2873
0.7	609	119	167	1633	2528
0.8	463	91	127	1265	1945
0.9	260	51	71	720	1102
1.0	0	0	0	0.0	0.0

Table 16.1.3-10 Factored Shear Envelopes for Interior Girder

Point x/L	Dead Load			Live Load	Load Combination
	DC1	*DC2*	*DW*	*HL93*	Service II
	V_{DC1} (kip)	V_{DC1} (kip)	V_{DW} (kip)	$1.3\ DF\ V_{HL93}$ (kip)	V_u (kip)
0.0	32.0	6.3	8.8	105.7	152.9
0.1	25.7	5.0	7.1	94.3	132.2
0.2	19.4	3.8	5.3	80.6	109.1
0.3	13.0	2.5	3.5	67.4	86.4
0.4	6.5	1.3	1.8	54.8	64.3
0.5	0.0	0.0	0.0	-42.7	-42.7
0.6	-6.5	-1.3	-1.8	-54.8	-64.3
0.7	-13.0	-2.5	-3.5	-67.4	-86.4
0.8	-19.4	-3.8	-5.3	-80.6	-109.1
0.9	-25.7	-5.0	-7.1	-94.3	-132.2
1.0	-32.0	-6.3	-8.8	-105.7	-152.9

16.1.3.10 Check Flexural Resistances of Composite Section of Existing Girder

16.1.3.10.1 Illustrate Calculation of Factored Moment - Strength Limit States

For the midspan section, the design is normally governed by the bending moments. Factored force effects at strength limit states are calculated and summarized in Section 16.1.3.7. Table 16.1.3-11 illustrates detailed calculations for factored moments in the positive moment region at the 0.5 Point.

Table 16.1.3-11 Factored Moment at 0.5 Point

Load Type	Unfactored Moment (kip-ft)	Factored Moment (kip-ft)	
DC1	726	907	applied to steel section alone
DC2	142	177	applied to long-term composite section ($3n$ = 24)
DW	198	298	applied to long-term composite section ($3n$ = 24)
$(LL+IM)_{HL\text{-}93}$	2430	2,562	applied to short-term composite section (n = 8)
$(LL+IM)_{P15}$	3949	3,211	applied to short-term composite section (n = 8)
Controlling M_u $DC+DW+(LL+IM)$	Strength II	4,593	M_u = 907+177+298+3211 = 4593

16.1.3.10.2 Calculate Elastic Section Properties of Existing Girder

Determine Effective Flange Width

According to Article 4.6.2.6 and C4.6.2.6.1, the effective flange width may be taken as the tributary width perpendicular to the axis of the member when the girder span to the spacing ratio (L/S) is larger than 3.1. In this design example, S = 6.75 ft and L = 90 ft. For the interior girder, the effective flange width is as:

$$\because \quad L/S = 90/6.75 = 13.33 > 3.1$$

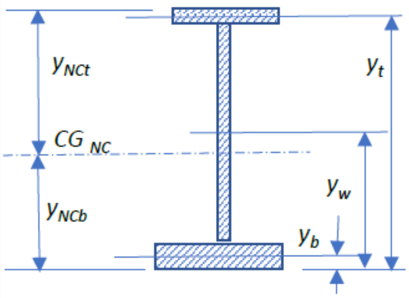
$$\therefore \quad b_{eff} = b = 6.75 \text{ ft} = 81 \text{ in.}$$

Calculate Elastic Section Properties

For the midspan and end span sections, elastic section properties for the existing steel section alone, the existing short-term composite section (n = 8), and the existing long-term composite section ($3n$ = 24) are calculated and shown in Tables 16.1.3-12 to 16.1.3-17. The contributions of the concrete haunch to the moment of inertia and the area are ignored in the following calculations.

Table16.1.3-12 Properties of Existing Steel Midspan Section Alone

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	$y_i - y_{NCb}$ (in.)	$A_i(y_i - y_{NCb})^2$ (in.^{4})	I_o (in.^{4})
Top Flange (3/4x12)	9.00	49.88	448.88	30.63	8,445	0.42
Web (3/8x48)	18.00	25.50	459.00	6.26	705	3,456
Bottom Flange (1 1/2x14)	21.00	0.75	15.75	-18.49	7,181	3.94
Total (Σ)	48.00		924		16,331	3,460



y_i = Component CG to the bottom of bottom flange

I_o = Moment of inertia of component about its CG

$y_{NCb} = \Sigma A_i y_i / \Sigma A_i$ = 19.24 in.

$y_{NCt} = (0.75+48+1.5)-19.24$ = 31.01 in.

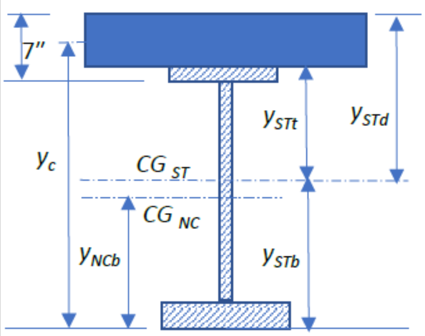
$I_{NC} = \Sigma I_o + \Sigma A_i (y_i - y_{NCb})^2$ = 19,792 in.^{4}

$S_{NCb} = I_{NC} / y_{NCb}$ = 1,029 in.^{3}

$S_{NCt} = I_{NC} / y_{NCt}$ = 638 in.^{3}

Table16.1.3-13 Properties of Existing Short-Term Composite Midspan Section (n = 8)

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	$y_i - y_{STb}$ (in.)	$A_i(y_i - y_{STb})^2$ (in.^{4})	I_o (in.^{4})
Steel Section	48.00	19.24	923.63	-19.41	18,084	19,792
Transformed Conc Slab (81/n x 6.25")	63.28	53.38	3,378	14.72	13,717	205.99
Total (Σ)	111.28		4,301		31,801	19,998



y_i = Component CG to the bottom of bottom flange

I_o = Moment of inertia of component about its CG

$y_{STb} = \Sigma A_i y_i / \Sigma A_i$ = 38.65 in.

$y_{STt} = (0.75+48+1.5)-38.65$ = 11.60 in.

$I_{ST} = \Sigma I_o + \Sigma A_i (y_i - y_{STb})^2$ = 51,799 in.^{4}

$S_{STb} = I_{ST} / y_{STb}$ = 1,340 in.^{3}

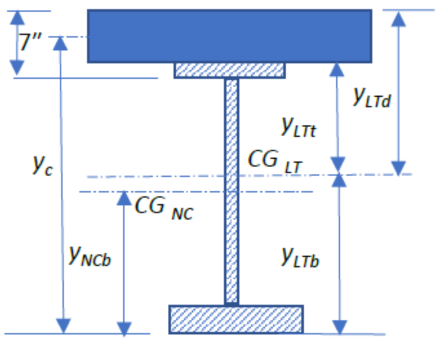
$S_{STt} = I_{st} / y_{STt}$ = 4,466 in.^{3}

$y_{STd} = y_{STt} + 7 - 0.75$ = 17.85 in.

$S_{STd} = I_{ST} / Y_{STd}$ = 2,902 in.^{3}

Table16.1.3-14 Properties of Existing Long-Term Composite Midspan Section (3*n* = 24)

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	y_i-y_{LTb} (in.)	$A_i(y_i$-$y_{LTb})^2$ (in.^{4})	I_0 (in.^{4})
Steel Section	48.00	19.24	923.63	-10.42	5,212	19,792
Transformed Conc Slab (81/3*n* x 6.25")	21.09	53.38	1,126	23.71	11,860	68.66
Total (Σ)	69.09		2,050		17,073	19,860



y_i = Component CG to the bottom of bottom flange

I_0 = Moment of inertia of component about its CG

$y_{LTb} = \Sigma A_i y_i / \Sigma A_i$ = 29.66 in.

$y_{LTt} = (0.75+48+1.5) - 29.66$ = 20.59 in.

$I_{LT} = \Sigma I_0 + \Sigma A_i (y_i - y_{LTb})^2$ = 36,933 in.^{4}

$S_{LTb} = I_{LT} / y_{LTb}$ = 1,245 in.^{3}

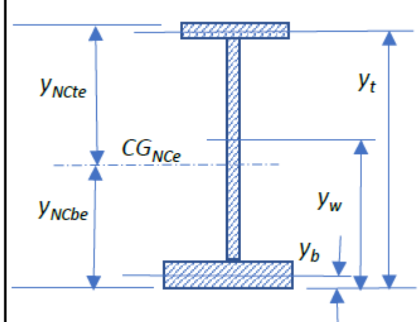
$S_{LTt} = I_{Lt} / y_{LTt}$ = 1,794 in.^{3}

$y_{LTd} = y_{LTt} + 7 - 0.75$ = 26.84 in.

$S_{LTd} = I_{LT} / y_{LTd}$ = 1,376 in.^{3}

Table16.1.3-15 Properties of Existing Steel End Span Section Alone

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	y_i-y_{NCbe} (in.)	$A_i(y_i$-$y_{NCbe})^2$ (in.^{4})	I_0 (in.^{4})
Top Flange (5/8x12)	7.50	49.44	370.78	29.27	6,426	0.24
Web (3/8x48)	18	25.13	452.25	4.96	442	3,456
Bottom Flange (1 1/8x14)	15.75	0.56	8.86	-19.60	6,053	1.66
Total (Σ)	41.25		831.89		12,921	3,458



y_i = Component CG to the bottom of bottom flange

I_0 = Moment of inertia of component about its CG

$y_{NCbe} = \Sigma A_i y_i / \Sigma A_i$ = 20.17 in.

$y_{NCte} = (5/8+48+1.125)-20.17$ = 29.58 in.

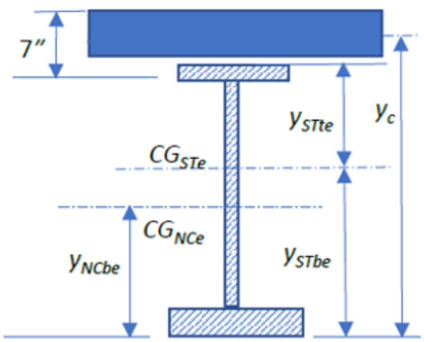
$I_{NCe} = \Sigma I_0 + \Sigma A_i (y_i - y_{NCbe})^2$ = 16,379 in.^{4}

$S_{NCbe} = I_{NCe} / y_{NCbe}$ = 812 in.^{3}

$S_{NCte} = I_{NCe} / y_{NCte}$ = 554 in.^{3}

Table16.1.3-16 Properties of Existing Short-Term Composite End Span Section (n = 8)

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	y_i-y_{STbe} (in.)	$A_i(y_i$-$y_{STbe})^2$ (in.^{4})	I_0 (in.^{4})
Steel Section	41.25	20.17	831.89	-19.88	16,297	16,379
Transformed Conc Slab (81/n x 6.25")	63.28	53.00	3,354	12.96	10,623	205.99
Total (Σ)	104.53		4,186		26,920	16,585



y_i = Component CG to the bottom of bottom flange

I_0 = Moment of inertia of component about its CG

$y_{STbe} = \Sigma A_i y_i / \Sigma A_i$ = 40.04 in.

y_{STte} = (5/8+48+1.125)-40.04 = 9.71 in.

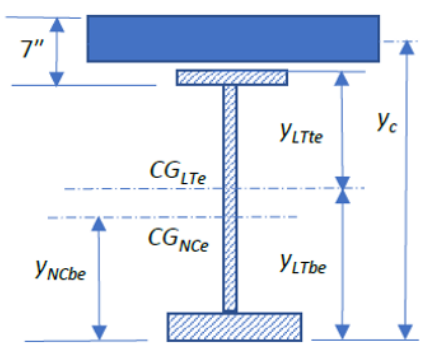
$I_{STe} = \Sigma I_0 + \Sigma A_i (y_i - y_{STbe})^2$ = 43,505 in.^{4}

$S_{STbe} = I_{STe} / y_{STbe}$ = 1,086 in.^{3}

$S_{STte} = I_{STe} / y_{STte}$ = 4,480 in.^{3}

Table16.1.3-17 Properties of Existing Long-Term Composite End Span Section (3n = 24)

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	y_i-y_{LTbe} (in.)	$A_i(y_i$-$y_{LTbe})^2$ (in.^{4})	I_0 (in.^{4})
Steel Section	41.25	20.17	831.89	-11.11	5,091	16,379
Transformed Conc Slab (81/3n x 6.25")	21.09	53.00	1,118	21.72	9,955	68.66
Total (Σ)	62.34		1,950		1,5046	16,448



y_i = Component CG to the bottom of bottom flange

I_0 = Moment of inertia of component about its CG

$y_{LTbe} = \Sigma A_i y_i / \Sigma A_i$ = 31.28 in.

y_{LTte} = (5/8+48+1.125)-31.28 = 18.47 in.

$I_{LTe} = \Sigma I_0 + \Sigma A_i (y_i - y_{LTbe})^2$ = 31,494 in.^{4}

$S_{LTbe} = I_{LTe} / y_{LTbe}$ = 1,007 in.^{3}

$S_{LTte} = I_{LTe} / y_{LTte}$ = 1,705 in.^{3}

16.1.3.10.3 Calculate Flexural Resistance for the Existing Girder

According to STP 16.6.4, for an existing steel girder using ASTM A7 with F_y of 30 ksi, the nominal flexural resistance shall be taken as the yield moment, M_{ys}. Since F_y = 33 ksi for this example, the moment resistance is based on the compactness of the composite section; check section compactness first and then estimate M_n.

Calculate Flexural Resistance, M_n, at Midspan 0.5L

Check Compactness of Section

For composite sections in the positive moment region, it is usually assumed that the top flange is adequately braced by the hardened concrete deck. There is no requirement for the compression flange slenderness and bracing for compact composite sections at the strength limit state. Three requirements (Article 6.10.6.2.2) for a compact composite section in straight bridges are checked as follows:

Specified minimum yield strength of flanges:

$$F_{yf} = 33 \text{ ksi} < 70 \text{ ksi} \qquad \text{O.K.} \qquad \text{(AASHTO 6.10.6.2.2)}$$

Web:

$$\frac{D}{t_w} = \frac{48}{0.375} = 128 < 150 \qquad \text{O.K.} \qquad \text{(AASHTO 6.10.2.1.1-1)}$$

Section:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E_s}{F_{yc}}} \qquad \text{(AASHTO 6.10.6.2.2-1)}$$

where D_{cp} is the depth of the web in compression at the plastic moment state and is determined in the following.

Compressive force in the concrete slab:

$$P_s = 0.85 f'_{ca}\, b_{eff} t_s = 0.85(3.6)(81)(6.25) = 1{,}549 \text{ kip}$$

in which t_s is the thickness of the concrete slab

Yield force in the top compression flange:

$$P_c = A_{fc} F_{yc} = (12 \times 0.75)(33) = 297 \text{ kip}$$

Yield force in the web:

$$P_w = A_w F_{yw} = (48 \times 0.375)(33) = 594 \text{ kip}$$

Yield force in the bottom tension flange:

$$P_t = A_{ft}F_{yt} = (14\times1.5)(33) = 693 \text{ kip}$$

Per AASHTO D6.1, the forces in longitudinal reinforcement may be conservatively neglected. Thus,

$$P_{rt} = P_{rb} = 0$$

$$\because\ P_t + P_w + P_c = 693 + 594 + 297 = 1{,}584 \text{ kip} \quad > P_s = 1{,}549 \text{ kip}$$

$\therefore$ The plastic neutral axis is within the top compression flange (AASHTO Table D6.1-1, Case II), and D_{cp} is equal to zero.

$$\frac{2D_{cp}}{t_w} = 0.0 < 3.76\sqrt{\frac{E}{F_{yc}}} \qquad \text{O.K.} \qquad \text{(AASHTO 6.10.6.2.2-1)}$$

The existing section meets the requirements for the composite compact section in positive flexure. The nominal flexural resistance, M_n, is, therefore, calculated in accordance with Article 6.10.7.1.2 (AASHTO, 2017; Caltrans, 2019).

Calculate Plastic Moment M_p

At the plastic moment state, the compressive stress in the concrete slab of a composite section is assumed equal to $0.85 f_c'$, and tensile stress in the concrete slab is neglected. The yield strength of the steel girder section is assumed equal to F_{ya} = 33 ksi. The reinforcement in the concrete slab is neglected in this example. The plastic moment M_p is determined using equilibrium equations and is the first moment of all forces about the plastic neutral axis (AASHTO D6.1).

Determine Location of Plastic Neutral Axis (PNA)

As calculated above, the plastic neutral axis (PNA) is within the top flange of the steel girder. Denote that $\bar{y}$ is the distance from the top of the compression flange to the PNA as shown in Figure 16.1.3-6, we obtain:

$$P_s + P_{c1} = P_{c2} + P_w + P_t$$

where

$$P_{c1} = \bar{y}\, b_{fc} F_{yc}$$

$$P_{c2} = \left(t_{fc} - \bar{y}\right) b_{fc} F_{yc}$$

in which b_{fc} and t_{fc} are the width and thickness of the top flange of the steel section, respectively; F_{yc} is the yield strength of the top compression flange of the steel section.

Substituting the above expressions into the equilibrium equation for $\bar{y}$, obtain

$$\bar{y} = \frac{t_{fc}}{2}\left(\frac{P_w + P_t - P_s}{P_c} + 1\right)$$

$$\bar{y} = \frac{0.75}{2}\left(\frac{594 + 693 - 1549}{297} + 1\right) = 0.044 \text{ in.} < t_{fc} = 0.75 \text{ in.} \qquad \text{OK}$$

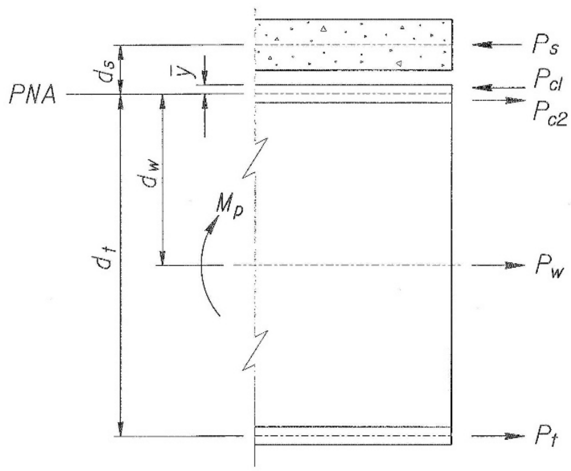


Figure 16.1.3-6 Plastic Moment Capacity State

Calculate Plastic Moment M_p

Summing all forces about the PNA, obtain:

$$M_p = \sum M_{PNA} = P_s d_s + P_{c1}\left(\frac{\bar{y}}{2}\right) + P_{c2}\left(\frac{t_{cf} - \bar{y}}{2}\right) + P_w d_w + P_t d_t$$

$$= P_s d_s + b_{fc} F_{yc}\left(\frac{\left(\bar{y}\right)^2 + \left(t_{cf} - \bar{y}\right)^2}{2}\right) + P_w d_w + P_t d_t$$

where

$$d_s = \left(7 - \frac{6.25}{2}\right) - 0.75 + 0.044 = 3.17 \text{ in.}$$

$$d_w = \frac{48}{2} + 0.75 - 0.044 = 24.71 \text{ in.}$$

$$d_t = \frac{1.5}{2} + 48 + 0.75 - 0.044 = 49.46 \text{ in.}$$

$$M_p = (1{,}549)(3.17) + (12)(33)\left(\frac{0.044^2 + (0.75 - 0.044)^2}{2}\right)$$
$$+ (594)(24.71) + (693)(49.46)$$
$$= 53{,}963 \text{ k-in.} = 4{,}497 \text{ kip-ft}$$

Calculate Yield Moment M_y

The yield moment M_y corresponds to the first yielding of either steel flange. It is obtained by the following formula (AASHTO D6.2):

$$M_y = M_{D1} + M_{D2} + M_{AD} \qquad \text{(AASHTO D6.2.2-2)}$$

where

M_{D1} = moment due to the factored permanent load at strength limit state applied to the steel section of the existing girder before the concrete deck is made composite (kip-in)

M_{D2} = moment due to the factored permanent load at strength limit state applied to the long-term composite section of the existing girder after the concrete deck is made composite (kip-in)

M_{AD} = additional moment applied to the short-term composite section to cause yielding in either the flanges (kip-in.)

For unshored construction of the existing steel girder, M_{AD} can be obtained by solving AASHTO Equation D6.2.2-1 as:

$$M_{AD} = S_{ST}\left(F_{yaf} - \frac{M_{D1}}{S_{NC}} - \frac{M_{D2}}{S_{LT}}\right) \qquad \text{(AASHTO D6.2.2-1)}$$

F_{yaf} = actual yield strength of the existing steel tension flange without holes (ksi)

S_{NC} = noncomposite elastic section (steel section alone) modulus of the existing steel girder (in.^{3})

S_{LT} = long-term composite elastic section modulus of the existing steel girder (in.^{3})

S_{ST} = short-term composite elastic section modulus of the existing steel girder (in.^{3})

From Table 16.1.3-5, factored moments, M_{D1} and M_{D2} are as follows:

$$M_{D1} = 1.25M_{DC1} = 907 \text{ k-ft} = 10{,}884 \text{ kip-in.}$$

$$M_{D2} = 1.25M_{DC2} + 1.5M_{DW} = 177 + 298 = 475 \text{ kip-ft} = 5{,}700 \text{ kip-in.}$$

For the bottom flange, section moduli are obtained from Tables 16.1.3-12, 13, and 14 as:

$$S_{NC} = S_{NCb} = 1{,}029 \text{ in.}^3 \ ; \ S_{LT} = S_{LTb} = 1{,}245 \text{ in.}^3 \ ; S_{ST} = S_{STb} = 1{,}340 \text{ in.}^3$$

$$M_{AD} = (1{,}340)\left(33 - \frac{10{,}884}{1{,}029} - \frac{5{,}700}{1{,}245}\right) \qquad \text{Control}$$
$$= 23{,}912 \text{ kip-in.} = 1{,}993 \text{ kip-ft}$$

For the top flange, section moduli are obtained from Tables 16.1.3-12, 13, and 14 as:

$$S_{NC} = S_{NCt} = 638 \text{ in.}^3 \ ; \ S_{LT} = S_{LTt} = 1{,}794 \text{ in.}^3 \ ; \ S_{ST} = S_{STt} = 4{,}466 \text{ in.}^3$$

$$M_{AD} = (4{,}466)\left(33 - \frac{10{,}884}{638} - \frac{5{,}700}{1{,}794}\right)$$
$$= 57{,}000 \text{ kip-in.} = 4{,}750 \text{ kip-ft}$$

It is obvious that the bottom flange controls. The yield moment of the existing girder midspan section is

$$M_y = M_{D1} + M_{D2} + M_{AD}$$
$$= 907 + 475 + 1{,}993 = 3{,}375 \text{ k-ft}$$

Calculate Flexural Resistance, M_n

The nominal flexural resistance of the composite compact section in the positive flexure is calculated in accordance with AASHTO and CA 6.10.7.1.2:

If $D_p \leq 0.1 D_t$, then:

$$M_n = M_p \qquad \text{(AASHTO 6.10.7.1.2-1)}$$

Otherwise:

$$M_n = \left[1 - \left(1 - \frac{M_y}{M_p}\right)\left(\frac{D_p / D_t - 0.1}{0.32}\right)\right] M_p \qquad \text{(CA 6.10.7.1.2-2)}$$

where D_p is the depth from the top of the concrete deck to the PNA; D_t is the total depth of the composite section.

The compact and noncompact sections shall satisfy the following ductility requirement to ensure that the tension flange of the steel section reaches significant yielding before the crushing strain is reached at the top of the concrete deck.

$D_p \le 0.42 D_t$ (AASHTO 6.10.7.3-1)

$D_p = 3.17 + 6.25/2 = 6.295$ in.

$D_t = 7 + 48 + 1.5 = 56.5$ in.

$D_p = 6.295$ in. $< 0.42 D_t = 0.42(56.5) = 23.73$ in. O.K.

$D_p = 6.295$ in. $> 0.1 D_t = 5.65$ in.

$$M_n = \left[1 - \left(1 - \frac{M_y}{M_p}\right)\left(\frac{D_p / D_t - 0.1}{0.32}\right)\right] M_p$$

$$= \left[1 - \left(1 - \frac{3{,}375}{4{,}497}\right)\left(\frac{6.295 / 56.5 - 0.1}{0.32}\right)\right](4{,}497) = 4{,}457 \text{ kip-ft}$$

$$\phi_f M_n = (1.0)(4{,}457) = 4{,}457 \text{ kip-ft}$$

Calculate Flexural Resistance, M_n, at End Span at 0.22L (x = 20 ft)

The calculation procedure for the flexural resistance for the end span section is similar to the above calculations for the midspan section and is not illustrated here.

Flexural resistance of the end span section of the existing girder is obtained as:

$$\phi M_n = (1.0)(3{,}780) = 3{,}780 \text{ kip-ft}$$

Check Flexural Resistances of Existing Girder

The factored moment at strength limit states for the end span at 0.22*L* (x = 20 ft) with a small section is calculated from Table 16.1.3-5 by interpolation as:

$$M_{u-x=20} = 2{,}858 + (3{,}877 - 2{,}858)\frac{2}{9} = 3{,}085 \text{ kip-ft}$$

Comparisons of flexural moments from Table 16.1.3-5 with factored resistances are shown in Table 16.1.3-18 and Figure 16.1.3-7.

Table16.1.3-18 Factored Moments and Resistances for Existing Girder at Strength Limit State

Point x/L	Location x (ft)	M_u (kip-ft)	$\phi_f Mn$ (kip-ft)	$\phi_f M_n > M_u$	Strengthening Needed
0.0	0	0	3,780	OK	No
0.1	9	1,731	3,780	OK	No
0.2	18	2,858	3,780	OK	No
0.22	20	3,085	3,780	OK	No
0.22	20	3,085	4,457	OK	No
0.3	27	3,877	4,457	OK	No
0.4	36	4,290	4,457	OK	No
0.5	45	4,593	4,457	NG	Yes
0.6	54	4,290	4,457	OK	No
0.7	63	3,877	4,457	OK	No
0.78	70	3,085	4,457	OK	No
0.78	70	3,085	3,780	OK	No
0.8	72	2,858	3,780	OK	No
0.9	81	1,731	3,780	OK	No
1.0	90	0	3,780	OK	No

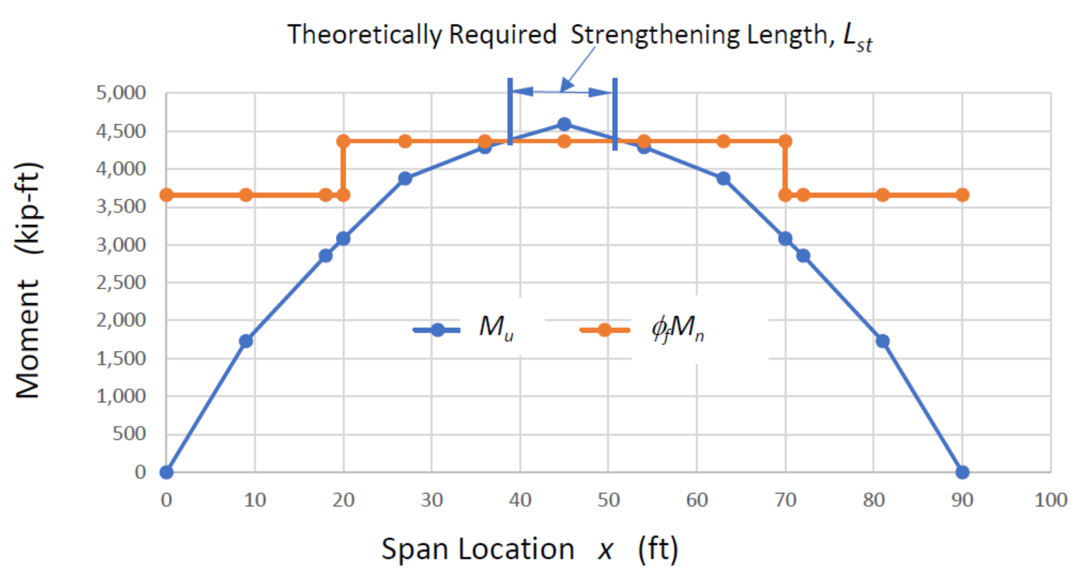


Figure 16.1.3-7 Factored Moments and Resistances for Existing Girder

It is seen that flexural strengthening is needed from Point 0.4 to Point 0.6. Theoretically required strengthening length, L_{st}, as shown in Figure 16.1.3-7, is obtained by $M_u = \phi_f M_n$ = 4,457 kip-ft as follows:

$$L_{st} = (2)\frac{(4{,}593 - 4{,}457)}{(4{,}593 - 4{,}290)}(9) = 8.08 \text{ ft}$$

In this example, the cover plates will be used for flexural strengthening.

16.1.3.11 Design Cover Plates

Try one 1 1/4" x 14" cover plate with the length of 12 ft in the middle of the span, as shown in Figure 16.1.3-8.

$$L_{cp} = 12 \text{ ft} > \frac{d}{6} + 3 = \frac{(1.5 + 48 + 0.75)}{6} + 3 = 11.38 \text{ ft} \quad \text{OK} \quad \text{(AASHTO 6.10.12.1-1)}$$

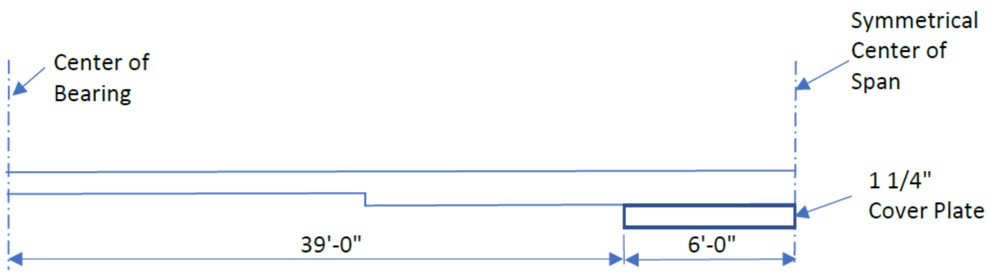


Figure 16.1.3-8 New Cover Plate

16.1.3.11.1 Calculate Factored Moments and Shears due to Additional Cover Plate

For the new cover plate, ASTM A709 Grade 36,

thickness: t_{cp} = 1.25 in.; width b_{cp} = 14 in.;

Weight of cover plate $$W_{cp} = \frac{(1.25)(14)(490)}{12^2} = 59.55 \text{ lb/ft}$$

$$\gamma W_{cp} = (1.25)(59.55) = 74.44 \text{ lb/ft} = 0.074 \text{ k/ft}$$

Factored additional cover plate dead load is shown in Figure 16.1.3-9. Factored moments and shears of the strengthened girder are shown in Table 16.1.3-19.

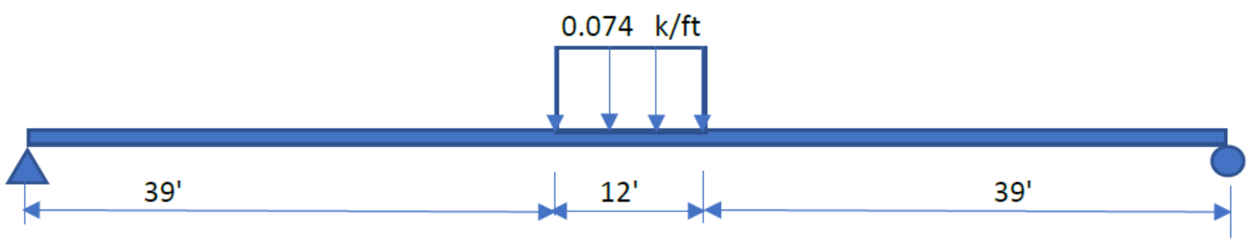


Figure 16.1.3-9 Factored Loads due to New Cover Plate

Table16.1.3-19 Factored Moments and Shears of Strengthened Girders

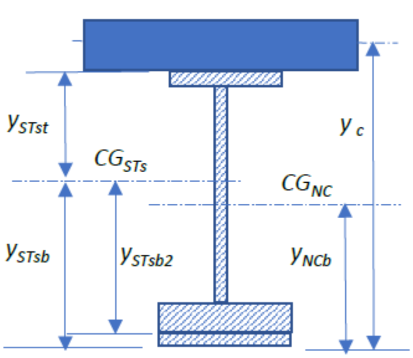
Point x/L	Location x (ft)	Existing Strength II M_u (kip-ft)	Added Moment γM_{CP} (kip-ft)	Total M_u (kip-ft)	Existing Strength II V_u (kip)	Added Shear γV_{CP} (kip)	Total V_u (kip)
0.0	0	0.0	0.0	0.0	252.8	0.4	253.2
0.1	9	1730.6	4.0	1734.6	214.3	0.4	214.7
0.2	18	2858.4	8.0	2866.5	169.1	0.4	169.6
0.3	27	3876.9	12.1	3889.0	130.4	0.4	130.9
0.4	36	4290.4	15.7	4306.2	91.6	0.4	92.1
0.5	45	4592.9	18.8	4611.7	-59.5	0.0	-59.5
0.6	54	4290.4	15.7	4306.2	-91.6	-0.4	-92.1
0.7	63	3876.9	12.1	3889.0	-130.4	-0.4	-130.9
0.8	72	2858.4	8.0	2866.5	-169.1	-0.4	-169.6
0.9	81	1730.6	4.0	1734.6	-214.3	-0.4	-214.7
1.0	90	0.0	0.0	0.0	-252.8	-0.4	-253.2

16.1.3.11.2 Calculate Section Properties of Strengthened Girder Section

Since the new cover plate is usually added in unshored construction, all permanent loads are assumed to be applied on the existing girder, and live loads are applied to the strengthened short-term composite section. The short-term section properties for the strengthened girder section are calculated in Table 16.1.3-20.

Table 16.1.3-20 Properties for Strengthened Short-term Composite Section at Midspan (*n* = 8)

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	$y_i - y_{STsb}$ (in.)	$A_i(y_i - y_{STsb})^2$ (in.^{4})	I_o (in.^{4})
Existing Steel Section	48.00	20.49	983.63	-14.07	9,506	19,792
1 1/4"x14" cover Plate	17.50	0.625	10.94	-33.94	20,158	2.28
Transformed Conc Slab (81/*n* x 6.25")	63.28	54.63	3,457	20.06	25,465	206
Total (Σ)	128.78		4,451		55,129	20,000



y_i = Component CG to the bottom of bottom flange

I_o = Moment of inertia of component about its CG

$y_{STsb} = \Sigma A_i y_i / \Sigma A_i$ = 34.56 in.

y_{STst} = (0.75+48+1.5+1.25)-34.56 = 16.94 in.

$I_{STs} = \Sigma I_0 + \Sigma A_i (y_i - y_{STsb})^2$ = 75,129 in.^{4}

$S_{STsb} = I_{STs} / y_{STsb}$ = 2,174 in.^{3}

$S_{STst} = I_{Sts} / y_{STst}$ = 4,436 in.^{3}

$y_{STsb2} = y_{STsb}$ -1.25 = 33.31 in.

$S_{STsb2} = I_{STs} / y_{STsb2}$ = 2,255 in.^{3}

16.1.3.11.3 Check Strengthened Flexural Resistance - Strength Limit State

According to STP 16.6.6.3.1, for a steel tension flange strengthened by a cover plate, the nominal flexural resistance, M_n, shall be taken as the yield moment, M_{ys}. The yield moment M_{ys} corresponds to the first yielding of either the steel flange or the cover plate. It is obtained by the following formula (AASHTO D6.2):

$$M_{ys} = M_{D1} + M_{D2} + M_{AD} \qquad \text{(AASHTO D6.2.2-2)}$$

$$M_{AD} = S_{STs}\left(F_y^* - \frac{M_{D1}}{S_{NC}} - \frac{M_{D2}}{S_{LT}}\right) \qquad \text{(AASHTO D6.2.2-1)}$$

For existing tension flange,

$$F_y^* = 0.84\left(\frac{A_{nf}}{A_{gf}}\right)F_{uaf} \le F_{yaf} \qquad \text{(AASHTO 6.10.1.8-1)}$$

where

F_{yaf} = actual yield strength of the existing steel tension flange without holes = 33 ksi

F_{uaf} = actual tensile strength of the existing steel tension flange without holes = 60 ksi

A_{gf} = gross section of existing tension flange = (14)(1.5) = 21 in.^{2}

A_{nf} = net section of existing tension flange(in.^{2})

Assume 2-3/4″ bolts, bolt hole diameter is 0.813 in. (AASHTO Table 6.13.2.4.2-1).

$$A_{nf} = 1.5\left(14 - 2\left(0.813\right)\right) = 18.56 \text{ in.}^2$$

$$\because F_y^* = 0.84\left(\frac{18.56}{21}\right)(60) = 44.54 \text{ ksi} > F_{yaf} = 33 \text{ ksi}, \text{ Use } F_y^* = F_{yaf} = 33 \text{ ksi}$$

From Tables 16.1.3-5 and 16.1.3-19, factored moments at the 0.5*L*, M_{D1} and M_{D2} are as follows:

$$M_{D1} = 1.25M_{DC1} = 907 \text{ kip-ft} = 10{,}844 \text{ kip-in.}$$

$$M_{D2} = 1.25M_{DC2} + 1.25M_{cp} + 1.5M_{DW} = 177 + 18.8 + 298 = 493.8 \text{ kip-ft} = 5{,}926 \text{ kip-in.}$$

For the existing bottom flange, section moduli are obtained from Tables 16.1.3-12, 14 and 20 as:

$$S_{NC} = S_{NCb} = 1{,}029 \text{ in.}^3 \text{ ; } S_{LT} = S_{LTb} = 1{,}245 \text{ in.}^3 \text{ ; } S_{STs} = S_{STb2} = 2{,}255 \text{ in.}^3$$

$$M_{AD} = S_{STs}\left(F_y^* - \frac{M_{D1}}{S_{NC}} - \frac{M_{D2}}{S_{LT}}\right)$$

$$=(2,255)\left(33 - \frac{10,844}{1,029} - \frac{5,926}{1,245}\right) = 39,918 \text{ kip-in.} = 3,327 \text{ kip-ft}$$

For the new cover plate, section modulus is obtained from Tables 16.1.3-20.

$$S_{STs} = S_{STsb} = 2,174 \text{ in.}^3$$

$$F_y^* = 0.84\left(\frac{A_{ncp}}{A_{gcp}}\right)F_{ucp} \leq F_{ycp} \qquad \text{(AASHTO 6.10.1.8-1)}$$

where

F_{ycp} = specified minimum yield strength of the cover plate= 36 ksi

F_{ucp} = specified minimum tensile strength of the cover plate = 58 ksi

A_{gcp} = gross section of the cover plate = (14)(1.25) = 17.5 in.^{2}

A_{ncp} = net section of the cover plate (in.^{2})

Assume 2-3/4″ bolts, bolt hole diameter is 0.813 in. (AASHTO Table 6.13.2.4.2-1).

$$A_{ncp} = 1.25\left(14 - 2(0.813)\right) = 15.47 \text{ in.}^2$$

$$\because F_y^* = 0.84\left(\frac{15.47}{17.5}\right)(58) = 43.1 \text{ ksi} > F_{ycp} = 36 \text{ ksi},$$

Use $F_y^* = F_{ycp} = 36$ ksi

Since the cover plate does not carry M_{D1} and M_{D2}, we obtain

$$M_{AD} = S_{STs}F_y^* = (2,174)(36) = 78,264 \text{ kip-in.} = 6,522 \text{ kip-ft}$$

It is seen that the existing tension flange reaches yield first.

$$M_{ys} = M_{D1} + M_{D2} + M_{AD} = 907 + 493.8 + 3,327 = 4,729 \text{ kip-ft}$$

From Table 16.1.3-19, the total factored moment at the 0.5*L*, M_u = 4,612 kip-ft.

$$\phi_f M_n = \phi_f M_{ys} = (1.0)(4,729) = 4,729 \text{ kip-ft} > M_u = 4,612 \text{ kip-ft}$$

OK

A 1 1/4"x14" cover plate is sufficient at the 0.5 point for the strength limit state.

16.1.3.11.4 Check for Stress Limitations at 0.5 Point - Service II Limit State

According to Article 6.10.4.2.2, at the service II limit state, for this example, stresses in flanges and the cover plate shall satisfy the following requirement:

$$f_f \leq 0.95 R_h F_{yaf} \qquad \text{(AASHTO 6.10.4.2.2-1)}$$

where:

R_h = hybrid factor determined as specified in Article 6.10.1.10.1 = 1.0

For composite section unshored construction, the flange stress at service II limit state is obtained as:

$$f_f = \frac{M_{SD1}}{S_{NC}} + \frac{M_{SD2}}{S_{LT}} + \frac{M_{SL}}{S_{STs}} \qquad (16.1.3.11.4\text{-}1)$$

where

M_{SD1} = moment due to the factored permanent load at service II limit state applied to the steel section of the existing girder before the concrete deck is made composite (kip-in)

M_{SD2} = moment due to the factored permanent load at service II limit state applied to the long-term composite section of the existing girder after the concrete deck is made composite (kip-in)

M_{SL} = moment due to the factored transient load at the service II limit state applied to the short-term composite section of strengthened steel girder (kip-in.)

From Tables 16.1.3-9 and 16.1.3-19,

$$M_{SD1} = 726 + (18.8 / 1.25) = 741.0 \text{ kip-ft} = 8{,}892 \text{ kip-in.}$$

$$M_{SD2} = 142 + 198 = 340 \text{ kip-ft} = 4{,}080 \text{ kip-in.}$$

$$M_{SL} = 1{,}903 \text{ kip-ft} = 22{,}836 \text{ kip-in.} \qquad \text{OK}$$

For the existing tension flange:

$$f_f = \frac{8{,}892}{1{,}029} + \frac{4{,}080}{1{,}245} + \frac{22{,}836}{2{,}255} \qquad \text{OK}$$
$$= 22.04 \text{ ksi} < 0.95 R_h F_{yaf} = 31.35 \text{ ksi}$$

For the new cover plate,

$$f_f = \frac{22{,}836}{2{,}174} = 10.5 \text{ ksi} < 0.95 R_h F_{ycp} = 34.2 \text{ ksi}$$

Notice that since the new cover plate does not carry a permanent load moment, it usually has small stress and does not control the design.

16.1.3.11.5 Check Cover Plate at 0.5 Point – Fatigue Limit States

Factored fatigue moments at the 0.5 point from Table 16.1.3-7 are as follows:

Fatigue I M_{u1} = 787 kip-ft

Fatigue II M_{u2} = 1259 kip-ft

Fatigue stress ranges for the new cover plate are obtained as:

Fatigue I $$\gamma(\Delta f) = \frac{787(12)}{2,174} = 4.34 \text{ ksi}$$

Fatigue II $$\gamma(\Delta f) = \frac{1,259(12)}{2,174} = 6.95 \text{ ksi}$$

Per AASHTO Table 6.6.1.2.3-1, for Description 2.2 cover plate, Category B, $(\Delta F)_{TH} = 16$ ksi

It is seen that fatigue stress ranges for both Fatigue I and Fatigue II are less than the constant-amplitude fatigue threshold, $(\Delta F)_{TH} = 16$ ksi; and the new cover plate satisfies fatigue limit state requirements.

16.1.3.12 Design Cover Plate Connection

STP 16.6.6.3.2 specifies that welded cover plates are not permitted, and bolted connections shall be designed as slip-critical.

Try F3125 Grade A325 HS 3/4" bolts with threads excluded from the shear plane to connect the new cover plate to the existing bottom flange.

Determine Bolt Spacing within Development Length

Article 6.10.12.2.3 requires that bolts in the slip-critical connections of the cover plate to the flange between the theoretical and actual ends, i.e., development length, shall be adequate to develop the force due to the factored loads in the cover plate at the theoretical end.

At the theoretical end of the cover plate, $x = 45 - \frac{L_{st}}{2} = 45 - \frac{8.08}{2} = 40.96$ ft

Based on factored live load moments shown in Table 16.1.3-5, the factored live load moment at the theoretical end (x = 40.96 ft) is:

$$M_{u\_LL\_x=40.96} = 2{,}964 + \left(\frac{(3{,}211 - 2{,}964)(9 - 4.04)}{9} \right) = 3{,}100 \text{ kip-ft}$$

The longitudinal force developed in the cover plate is

$$T_{cp} = \frac{M_{u\_LL-X=40.96}}{S_{STsb}} A_{gcp} = \frac{(3{,}100)(12)}{2{,}174}(17.5) = 299.4 \text{ kip}$$

For Grade A325 3/4” bolt,

A_b = cross-sectional area of a bolt = 0.442 in.^{2}

F_{ub} = tensile strength of bolt = 120 ksi (AASHTO 6.4.3.1)

N_s = number of slip plane in connection = 1

The nominal shear resistance of a bolt is obtained as:

$$R_{n1} = 0.56 A_b F_{ub} N_s. = (0.56)(0.442)(120)(1) = 29.7 \text{ kip} \quad \text{(AASHTO 6.13.2.7-1)}$$

The nominal bearing resistance at bolt holes is obtained as:

For Grade A325 3/4” bolt, the nominal diameter of the bolt, d = 0.75 in.; the bolt hole diameter is 0.813 in. (AASHTO Table 6.13.2.4.2-1); use the edge distance = 1.50 in. (AASHTO Table 6.13.2.6.6-1).

$$\because\ L_c = \text{the clear edge distance} = 1.5 - (0.813/2) = 1.09 \text{ in.} < 2d = 1.5 \text{ in.}$$

$$R_{n2} = 1.2 L_c t_{cp} F_u = (1.2)(1.09)(1.25)(58) = 94.8 \text{ kip} \quad \text{(AASHTO 6.13.2.9-2)}$$

It is seen that shear resistance controls, and the nominal shear resistance per bolt is obtained as:

$$R_n = \min(R_{n1,} R_{n2}) = 29.7 \text{ kip}$$

A total number of bolts required between the theoretical and actual ends is obtained as:

$$N_{reqd} = \frac{T_{cp}}{\phi_s R_n} = \frac{299.4}{(0.8)(29.7)} = 12.6$$

The length between the theoretical and actual ends, i.e., the development length = 6 – 4.04 = 1.96 ft., the edge distance = 1.5 inches. Use 4-A325 3/4” bolts in each row; the required bolt rows are obtained:

$$N_{row-reqd} = \frac{N_{reqd}}{4} = \frac{12.6}{4} = 3.15 \qquad \text{Use 4 Rows}$$

Try 4 rows, the required bolt spacing is:

$$S_{reqd} = \frac{(1.96)(12) - 1.5}{4} = \frac{22.02}{4} = 5.51 \text{ in.}$$

Article 6.13.2.6.3 specifies that the maximum pitch of fasteners in mechanically fastened built-up members shall not exceed the lesser of the requirements for sealing or stitch. The maximum sealing spacing is 7.0 in. (Article 6.13.2.6.2). The maximum pitch and gauge for stitch bolts in the tension member is 24 times the thickness of the thinner outside plate = 24(1.25) = 30.0 in. (Article 6.13.2.6.3).

Using 4 rows, 4 - A325 3/4" bolts at a spacing of 4.0 in. satisfies all the above requirements.

The nominal slip resistance per bolt is:

$$R_n = K_h K_s N_s P_t \qquad \text{(AASHTO 6.13.2.8-1)}$$

where

- K_h = the hole size factor and is equal to 1.0 for the standard hole (AASHTO Table 6.13.2.8-2)
- K_s = the surface condition factor and is taken as 0.5 for Class B surface condition (AASHTO Table 6.13.2.8-3)
- N_s = the number of slip planes and is equal to 1.0
- P_t = the minimum required bolt tension and is equal to 28 kips (AASHTO Table 6.13.2.8-1).

$$R_n = K_h K_s N_s P_t = (1.0)(0.5)(1)(28) = 14.0 \text{ kip} \qquad \text{(AASHTO 6.13.2.8-1)}$$

The factored live load moment at service II limit state (Table 16.1.3-9) at the theoretical end,

$$M_{u\_LL\_x=40.96} = 1{,}850 + \left(\frac{(1{,}903 - 1{,}850)(9 - 4.04)}{9} \right) = 1{,}879 \text{ kip-ft}$$

The longitudinal force developed in the cover plate is:

$$T_{cp} = \frac{M_{u\_LL-X=40.96}}{S_{STsb}} A_{gcp} = \frac{(1{,}879)(12)}{2{,}174}(17.5) = 181.5 \text{ kip}$$

Total 4 rows – 16 bolts and the total slip resistance is:

$$R = (16)(R_n) = 16(14.0) = 224 \text{ kip} > T_{cp} = 181.5 \text{ kip} \quad \text{OK}$$

Check Total Bolts Required for Cover Plate

STP 16.6.6.3.2 specifies that at the strength limit state, bolted connections on either side of the location of the maximum stress of the cover plate shall be designed for the plastic force of the cover plate.

The plastic force is:

$$T_{cp} = A_{gcp} F_{ycp} = (17.5)(36) = 630 \text{ kip}$$

The total number of bolts required is:

$$N_{total\_reqd} = \frac{T_p}{\phi_s R_n} = \frac{630}{(0.8)(29.7)} = 26.5$$

Try two bolts each row at a spacing of 7.0 in. inside of the theoretical end, the total number of bolt each side of the location of the maximum stress is:

$$N_{prod} = 16 + 16 = 32 > N_{total-reqd} = 26.5 \qquad \text{OK}$$

Final Cover Plate and Connection

The final cover plate length = $(1.5 + 3(4.0) + 8(7.0) + 2.5)(2) =$ 144 in. and is shown in Figure 16.1.3-10. The final cover plate connection bolt layout is shown in Figure 16.1.3-11.

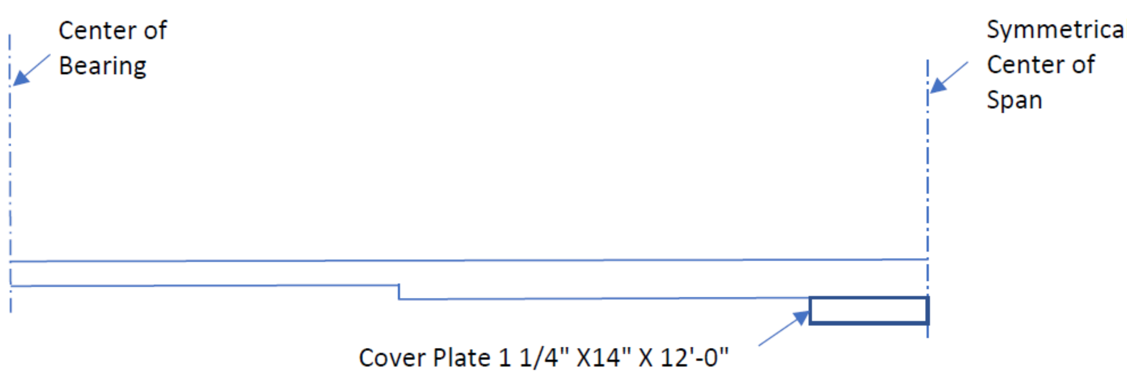


Figure 16.1.3-10 Final Cover Plate Layout

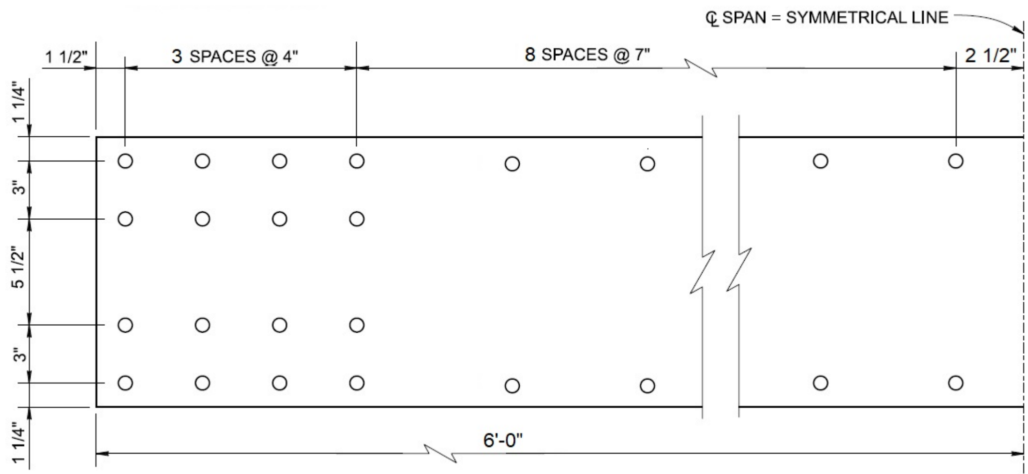


Figure 16.1.3-11 Final Cover Plate Connection Bolt Layout

16.1.4 FLEXURAL STRENGTHENING DESIGN EXAMPLE 2 – POST-TENSIONING

The following is an example of strengthening a simple span composite steel girder bridge using the external post-tensioning prestressing to increase flexural capacity due to increased live loads. The future wearing surface has not been added before the post-tensioning.

16.1.4.1 Existing Steel Girder Bridge Data

Existing steel girder bridge data are the same as Example 1 shown in Section 16.1.3.1

16.1.4.2 Design Requirement

Load and structural analyses, live load distributions, load and load combinations, factored moments and shears and factored flexural resistances are the same as Example 1, as shown in Sections 16.1.3.3 to 16.1.3.10. Weights of prestressing steel and connection brackets are very small compared with the superstructure dead loads and are not considered in this example.

Perform the following strengthening design portions for an interior plate girder in accordance with STP 16.6 (Caltrans 2021) and AASHTO-CA BDS-8 (AASHTO, 2017; Caltrans, 2019). A similar procedure can be used for strengthening exterior girders and is not illustrated here.

Step 1: Design Post-tensioning Prestress Tendon

Step 2: Design Anchorage Bracket Components

Step 3: Design Anchorage Bracket Connection to Girder Web

Step 4: Design Anchorage Bracket Plate Weld Connection

16.1.4.3 Design Post-Tensioning Prestress Tendon

16.1.4.3.1 Determine Material Properties for Prestressing Steel

Try ASTM A416 Grade 270 prestressing strand for post-tensioning tendon. Material properties are obtained from Article 5.4.4 as follows:

Specified tensile strength of prestressing steel: f_{pu} = 270 ksi

Yield strength of prestressing steel: $f_{py} = 0.9\, f_{pu}$ = 243 ksi

Modulus of elasticity of prestressing steel: E_p = 28,500 ksi

Modulus ratio for prestressing steel: $n_{ps} = \dfrac{E_s}{E_p} = \dfrac{29{,}000}{28{,}500} = 1.018$

16.1.4.3.2 Assume Area of Prestressing Steel and Layout

Because this bridge is located on a highway where vertical clearances cannot be reduced, the prestressing tendons are placed along the side of the girder and anchorages attached to the web. The tendons will be placed far enough away from the girder web to clear the girder stiffener plates. Also, clearances are provided for the monostand jacking equipment (assume 5" clearance), as shown in Figure 16.1.4 -1.

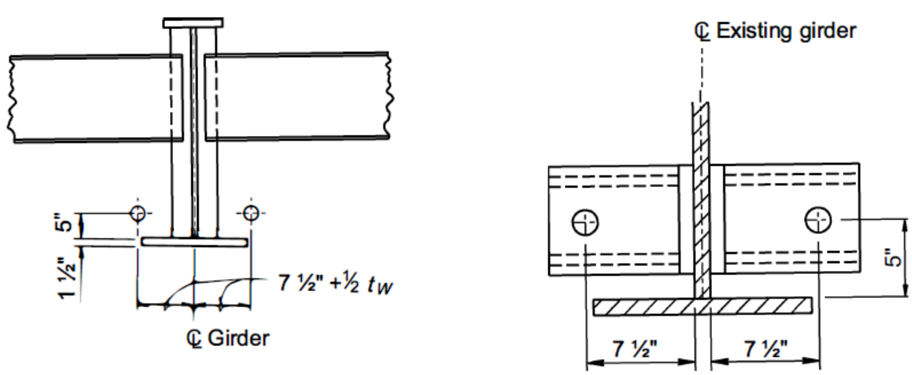


Figure 16.1.4-1 Prestressing Tendon Layout

Try 8-0.6 in. diameter low-relaxation strands with a total length of 48 ft, as shown in Figure 16.1.4-2.

The area of prestressing steel is: $A_{ps} = (8)(0.217) = 1.736 \text{ in.}^2$

The jacking stress in prestressing steel is selected per CA Table 5.9.2.2-1 as:

$$f_j = 0.75 f_{pu} = (0.75)(270) = 202.5 \text{ ksi}$$

The total estimated prestressing force at jacking is:

$$P_j = f_j A_{ps} = (202.5)(1.736) = 351.5 \text{ kip}$$

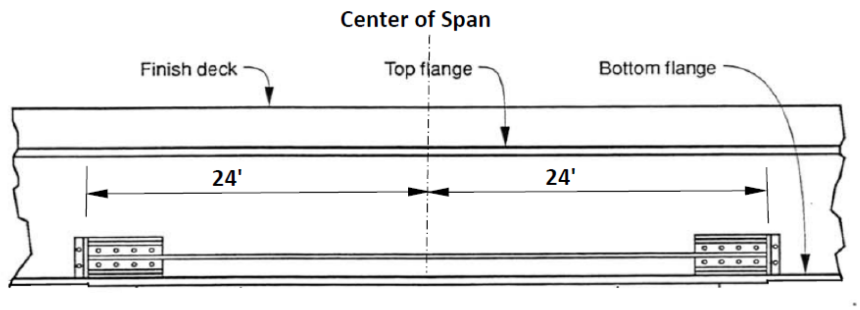


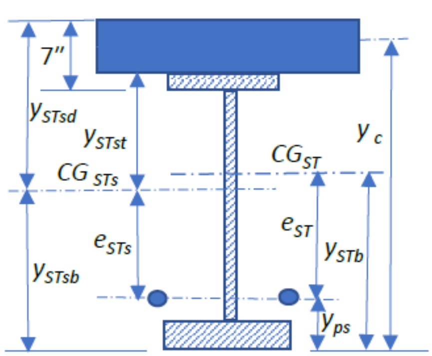
Figure 16.1.4-2 Girder Elevation for Post-tensioning Strengthening

16.1.4.3.3 Calculate Transformed Section Properties

Transformed section properties with strands are calculated in the following tables using information from Tables 16.1.3-13 and 16.1.3-14.

Table 16.1.4-1 Section Properties for Strengthened Short-term Composite Section Midspan (*n* = 8)

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	$y_i - y_{STsb}$ (in.)	$A_i(y_i - y_{STsb})^2$ (in.^{4})	I_o (in.^{4})
Strands (A_{ps}/n_{ps})	1.71	6.50	11.09	-31.67	1,711	0
Existing Short-Term Composite Section	111.28	38.65	4,301	0.49	26	51,799
Total (Σ)	112.99		4,312		1,737	51,799



$n_{ps} = E_s/E_p =$ 29000/28500 = 1.018

y_i = Component CG to the bottom of bottom flange

I_0 = Moment of inertia of component about its CG

$y_{STsb} = \Sigma A_i y_i / \Sigma A_i$ = 38.17 in.

$y_{STst} = (0.75+48+1.5)-38.17$ = 12.08 in.

$I_{STs} = \Sigma I_0 + \Sigma A_i(y_i - y_{STsb})^2$ = 53,536 in.^{4}

$S_{STsb} = I_{STs}/y_{STsb}$ = 1,403 in.^{3}

$S_{STst} = I_{STs}/y_{STst}$ = 4,431 in.^{3}

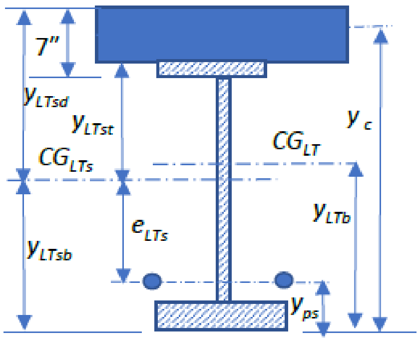
$e_{STs} = y_{STsb} - 5 - 1.5$ = 31.67 in

$y_{STsd} = y_{STst} + 7 - 0.75$ = 18.33 in

$S_{STsd} = I_{STs}/y_{STsd}$ = 2,920 in.^{3}

Table 16.1.4-2 Section Properties for Strengthened Long-Term Composite Section Midspan (3n = 24)

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	y_i-y_{LTsb} (in.)	$A_i(y_i$-$y_{LTsb})^2$ (in.^{4})	I_0 (in.^{4})
Strands (A_{ps}/n_{ps})	1.71	6.50	11.09	-22.60	872	0
Existing Long-Term Composite Section	69.09	29.66	2,050	0.56	22	36,933
Total (Σ)	70.80		2,061		893	36,933



y_i = Component CG to the bottom of bottom flange

I_0 = Moment of inertia of component about its CG

$y_{LTsb} = \Sigma A_i y_i / \Sigma A_i$ = 29.10 in.

y_{LTst} = (0.75+48+1.5)-29.1 = 21.15 in.

$I_{LTs} = \Sigma I_0 + \Sigma A_i (y_i - y_{LTsb})^2$ = 37,826 in.^{4}

$S_{LTsb} = I_{LTs} / y_{LTsb}$ = 1,300 in.^{3}

$S_{LTst} = I_{LTs} / y_{LTst}$ = 1,789 in.^{3}

$e_{LTs} = y_{LTsb}$ - 5 - 1.5 = 22.60 in

$y_{LTsd} = y_{LTst}$ +7-0.75 = 27.40 in

$S_{LTsd} = I_{LTs} / y_{LTsd}$ = 1,381 in.^{3}

16.1.4.3.4 Estimate Prestress Losses and Effective Prestress

Instantaneous Losses

For post-tensioned steel girders, the instantaneous losses include the anchor set and the elastic shortening.

- *Anchor Set Loss, Δf_{pA}*

The anchor set is caused by the movement of the tendon prior to the seating of the anchorage gripping device. This loss occurs prior to the force transfer between wedge (or jaws) and anchor block. The anchor set loss is the reduction in strand force through the loss in the stretched length of the strand. Article 5.9.3.2.1 suggests a common value for the anchor set as Δ_{pA}= 3/8 inch, which represents the amount of the slip in Caltrans approved anchorage systems.

$$\Delta f_{pA} = \frac{\Delta_{pA} E_p}{L_{ps}} = \frac{(0.375)(28{,}500)}{(48)(12)} = 18.55 \text{ ksi} \qquad (16.1.4.3.4\text{-}1)$$

- *Elastic Shortening Loss, Δf_{pES}*

Elastic shortening loss is obtained by replacing E_{ci} with E_s in CA Eq. (5.9.3.2.3b-1) as follows:

$$\Delta f_{pES} = 0.5\frac{E_p}{E_s}f_{cgp} \qquad (16.1.4.3.4\text{-}2)$$

where

f_{cgp} = sum of steel stresses at the center of gravity of prestressing tendons due to the prestressing force after jacking and the self-weight of the member at the section of the maximum moment (ksi)

For post-tensioned structures with unbonded tendons, Article 5.9.3.2.3b permits that the f_{cgp} value is calculated as the stress at the center of gravity of the prestressing steel averaged along the length of the member. However, for this example, the f_{cgp} is calculated at the section of the maximum moment based on the existing short-term composite section properties conservatively.

$$f_{cgp} = \left(\frac{P_j}{A_{ST}} + \frac{P_j e_{ST}^2}{I_{ST}}\right) \qquad (16.1.4.3.4\text{-}3)$$

From Table 16.1.3-13, the existing short-term composite section properties are A_{ST} = 111.28 in.^{2}; I_{ST} = 51,799 in.^{4}; e_{ST} = y_{STb} – 6.5 = 38.65 – 6.5 = 32.15 in. From Section 16.1.4.3.2, P_j = 351.5 kip.

$$f_{cgp} = \left(\frac{351.5}{111.28} + \frac{(351.5)(32.15)^2}{51{,}799}\right) = 10.17 \text{ ksi}$$

$$\Delta f_{pES} = 0.5\frac{28{,}500}{29{,}000}(10.17) = 5.00 \text{ ksi}$$

Long Term Loss, Δf_{pLT}

According to STP 16.6.6.4, lthe ong-term prestress losses are taken as, Δf_{pLT} = 5 ksi.

Total Prestress Loss, Δf_{pT}

$$\Delta f_{pT} = \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT} = 18.55 + 5.00 + 5.0 = 28.55 \text{ ksi}$$

Effective prestress stress, f_{pe}

The effective prestress stress in prestressing steel after losses is obtained as:

$$f_{pe} = f_j - \Delta f_{pT} = 0.75 f_{pu} - \Delta f_{pT} = 202.5 - 28.55 = 173.95 \text{ ksi}$$

The effective prestressing force is as:

$$P_{pe} = A_{ps} f_{pe} = (1.736)(173.95) = 302.0 \text{ kip}$$

16.1.4.3.5 Check Flexural Resistance at Midspan – Strength Limit State

Estimate Stress in the Unbonded Prestressing Steel f_{ps}

From previous property calculations, the neutral axis is within the top flange. Assuming the neutral axis is within the top flange, the distance from the extreme compression fiber to the neutral axis, *c*. is obtained by:

$$c = \left(\frac{t_{fc}}{2}\right)\left[\frac{A_{ps}f_{ps} + A_{ft}F_{yaf} + A_w F_{yaw} - 0.85 f'_{ca}\ b_s t_s}{A_{fc}F_{yaf}} + 1\right] + t_s \qquad \text{(STP 16.6.6.4.2)}$$

where

A_{fc}	=	area of the existing compression flange = 9 in.^{2}
A_{ps}	=	area of the prestressing steel = 1.736 in.^{2}
A_{ft}	=	area of the existing tension flange = 21 in.^{2}
A_w	=	area of the existing web = 18 in.^{2}
b_s	=	effective width of concrete deck = 81 in.
D	=	depth of web = 48 in.
F_{yaf}	=	actual yield strength of the existing flange = 33 ksi
F_{yaw}	=	actual yield strength of the existing web = 33 ksi
t_{fc}	=	thickness of the existing compression flange = 0.75 in.
t_s	=	thickness of concrete deck = 6.25 in.
t_w	=	thickness of the web = 0.375 in.
f'_{ca}	=	actual concrete compressive strength = 3.6 ksi

$$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{l_e}\right) \le f_{py} \qquad \text{(AASHTO 5.6.3.1.2-1)}$$

$$l_e = \frac{2l_i}{2 + N_s} \qquad \text{(AASHTO 5.6.3.1.2-2)}$$

f_{pe}	=	effective stress in prestressing steel = 173.95 ksi
l_e	=	effective tendon length
l_i	=	length of tendon between anchorage = (48)(12) = 576 in.
N_s	=	number of plastic hinges at supports in an assumed failure mechanism crossed by the tendon between anchorages or discretely bonded points assumed as = 0 for simple span

d_p = distance from extreme compression fiber (deck) to the centroid of the prestressing tendons = 7 + 48 – 5 = 50 in.

Above two equations with two unknown (f_{ps} and c) need to be solved simultaneously to achieve a closed-form solution by iterations.

Per Article C5.6.3.1.2, try the average stress in unbonded prestressing steel made as:

$$f_{ps} = f_{pe} + 15 = 173.95 + 15 = 188.95 \text{ ksi} \quad \text{(AASHTO C5.6.3.1.2-1)}$$

Assume the neutral axis is within the top flange:

$$c = \left(\frac{t_{fc}}{2}\right)\left[\frac{A_{ps}f_{ps} + A_{ft}F_{yaf} + A_wF_{yaw} - 0.85f'_{ca}\ b_st_s}{A_{fc}F_{yaf}} + 1\right] + t_s$$

$$= \left(\frac{0.75}{2}\right)\left[\frac{(1.736)(188.95) + (21)(33) + (18)(33) - 0.85(3.6)(81)(6.25)}{(9)(33)} + 1\right] + 6.25$$

$$= 6.71 \text{ in.} < t_s + t_{fc} = 7.0 \text{ in.}$$

$$f_{ps} = 173.95 + 900\left(\frac{50 - 6.71}{576}\right) = 241.59 \text{ ksi} \neq \text{ Assumed } f_{py} = 188.95 \text{ ksi}$$

Try again with $f_{ps} = 241.41$ ksi

Assume the neutral axis is within the top flange:

$$c = \left(\frac{t_{fc}}{2}\right)\left[\frac{A_{ps}f_{ps} + A_{ft}F_{yaf} + A_wF_{yaw} - 0.85f'_{ca}\ b_st_s}{A_{fc}F_{yaf}} + 1\right] + t_s$$

$$= \left(\frac{0.75}{2}\right)\left[\frac{(1.736)(241.41) + (21)(33) + (18)(33) - 0.85(3.6)(81)(6.25)}{(9)(33)} + 1\right] + 6.25$$

$$= 6.823 \text{ in.} < t_s + t_{fc} = 7.0 \text{ in.}$$

$$f_{ps} = 173.95 + 900\left(\frac{50 - 6.823}{576}\right) = 241.41 \text{ ksi} \qquad \text{OK} \qquad \text{(AASHTO 5.6.3.1.2-1)}$$

$$= \text{Assumed } f_{py} = 241.41 \text{ ksi} < f_{py} = 243 \text{ ksi}$$

Use f_{ps} = 241.41 ksi

Calculate Plastic Moment M_p

From the above calculations, the PNA is within the top steel flange. Denote that $\bar{y}$ is the distance from the top of the steel flange to the PNA as shown in Figure 16.1.4-3.

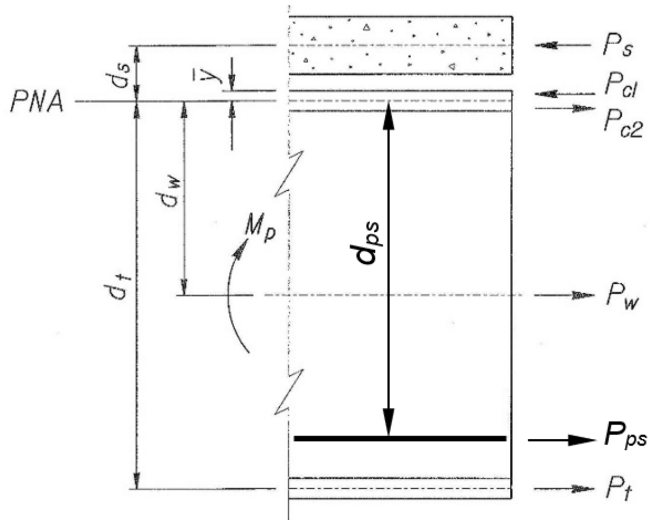


Figure 16.1.4-3 Plastic Moment State at 0.5*L*

From Example 1 shown in Section 16.1.3.10.3, we have:

$$P_s = 0.85 f'_c\, b_{eff} t_s = 0.85(3.6)(81)(6.25) = 1{,}549 \text{ kip}$$

$$P_c = 297 \text{ kip}$$

$$P_w = 594 \text{ kip}$$

$$P_t = 693 \text{ kip}$$

$$P_{ps} = A_{ps} f_{ps} = (1.736)(241.41) = 419 \text{ kip}$$

$$\bar{y} = c - t_s = 6.824 - 6.25 = 0.574 \text{ in}$$

$$d_s = \bar{y} + \frac{t_s}{2} = 0.574 + \frac{6.25}{2} = 3.70 \text{ in.}$$

$$d_w = \frac{D}{2} + t_{fc} - \bar{y} = \frac{48}{2} + 0.75 - 0.574 = 24.18 \text{ in.}$$

$$d_t = D + \frac{t_{ft}}{2} - \bar{y} = 48 + \frac{1.5}{2} - 0.574 = 48.18 \text{ in.}$$

$$d_{ps} = d_t - \frac{t_{ft}}{2} - 5.0 = 48.18 - \frac{1.5}{2} - 5.0 = 42.43 \text{ in.}$$

The plastic moment is obtained by summarizing moments taken from all forces about PNA, or the modified plastic moment equation from AASHTO Table D6.1-1, Case II, as follows:

$$M_p = P_s d_s + b_{fc} F_{yc}\left(\frac{\left(\bar{y}\right)^2 + \left(t_{cf} - \bar{y}\right)^2}{2}\right) + P_w d_w + P_t d_t + P_{ps} d_{ps}$$

$$= (1{,}549)(3.70) + (12)(33)\left[\frac{0.574^2 + (0.75 - 0.574)^2}{2}\right]$$

$$+ (594)(24.18) + (693)(48.18) + (419)(42.43)$$

$$= 71{,}333 \text{ kip-in.} = 5{,}944 \text{ kip-ft}$$

Calculate Yield Moment M_{ys}

For a post-tensioned steel girder, the effective prestressing force and the permanent loads after the post-tensioning, such as future wearing surface dead load, are applied on the long-term composite section of the strengthened steel girder. For the strengthened steel girder, the yield moment M_{ys} corresponds to the first yielding of either steel flange. It is obtained by the modified AASHTO Equation (D6.2.2-2) as follows:

$$M_{ys} = M_{D1} + M_{D2} + M_{D2p} + M_{AD} \tag{16.1.4.3.5-1}$$

$$M_{AD} = S_{STs}\left(F_y^* - \frac{M_{D1}}{S_{NC}} - \frac{M_{D2}}{S_{LT}} - \frac{M_{D2p}}{S_{LTs}}\right) \tag{16.1.4.3.5-2}$$

$$F_y^* = F_{yaf} + f_{fps} \tag{16.1.4.3.5-3}$$

where

f_{fps} = stress in the steel flange due to the effective prestressing force P_{pe}

M_{D2p} = moment due to the factored permanent loads after the post-tensioning at the strength limit state applied to the long-term composite section of the strengthened steel girder (kip-in.)

From Example 1, it is known that the yield moment is controlled by the bottom tension flange; therefore, the bottom tension flange is calculated herein.

$$f_{fps} = \frac{P_{pe}}{A_{LTs}} + \frac{P_{pe} e_{LTs}}{S_{LTsb}} \tag{16.1.4.3.5-4}$$

From Table 16.1.4-2, $A_{LTs} = 70.8 \text{ in.}^2$; $S_{LTsb} = 1{,}300 \text{ in.}^3$; $e_{LTs} = 22.6$ in.

$$f_{fps} = \frac{302}{70.8} + \frac{302(22.6)}{1300} = 9.52 \text{ ksi}$$

$$F_y^* = F_{yaf} + f_{fps} = 33 + 9.52 = 42.52 \text{ ksi}$$

From Tables 16.1.3-5, factored moments at the 0.5*L*, M_{D1} and M_{D2} are as follows:

$$M_{D1} = 1.25M_{DC1} = 907 \text{ kip-ft} = 10{,}884 \text{ kip-in.}$$

$$M_{D2} = 1.25M_{DC2} = 177 \text{ kip-ft} = 2{,}124 \text{ kip-in.}$$

$$M_{D2p} = 1.5M_{DW} = 298 \text{ kip-ft} = 3{,}576 \text{ kip-in.}$$

For the existing bottom flange, section moduli are obtained from Tables 16.1.3-12 and 16.1.3-14 as:

$$S_{NC} = S_{NCb} = 1{,}029 \text{ in.}^3 \ ; \ S_{LT} = S_{LTb} = 1{,}245 \text{ in.}^3$$

For strengthened steel girder (Table 16.1.4-1), $S_{STs} = S_{STsb} = 1{,}403 \text{ in.}^3$

$$M_{AD} = (1{,}403)\left(42.52 - \frac{10{,}844}{1{,}029} - \frac{2{,}124}{1{,}245} - \frac{3{,}576}{1{,}403}\right)$$

$$= 38{,}901 \text{ kip-in.} = 3{,}242 \text{ kip-ft}$$

$$M_{ys} = M_{D1} + M_{D2} + M_{D2p} + M_{AD}$$

$$= 907 + 177 + 298 + 3{,}242 = 4{,}623 \text{ kip-ft}$$

Calculate Flexural Resistance, M_n

The nominal flexural resistance of the strengthened girder section is calculated in accordance with AASHTO and CA 6.10.7.1.2, as discussed in Section 16.1.3.10.3.

For this example, $M_y = M_{ys}$.

The compact and noncompact sections shall satisfy the following ductility requirement to ensure that the tension flange of the steel section reaches significant yielding before the crushing strain is reached at the top of the concrete deck.

$$D_p \leq 0.42D_t \qquad \text{(AASHTO 6.10.7.3-1)}$$

$$D_p = c = 6.824 \text{ in.}$$

$$D_t = 7 + 48 + 1.5 = 56.5 \text{ in.}$$

$$D_p = 6.824 \text{ in.} < 0.42D_t = 0.42(56.5) = 23.73 \text{ in.} \qquad \text{OK}$$

For this example, $M_y = M_{ys}$.

$$\because D_p = 6.824 \text{ in.} > 0.1D_t = 5.65 \text{ in.}$$

$$M_n = \left[1 - \left(1 - \frac{M_y}{M_p}\right)\left(\frac{D_p / D_t - 0.1}{0.32}\right)\right] M_p$$

$$= \left[1 - \left(1 - \frac{4{,}623}{5{,}944}\right)\left(\frac{6.824 / 56.5 - 0.1}{0.32}\right)\right](5{,}944) \qquad \text{(CA 6.10.7.1.2-2)}$$

$$= 5{,}858 \text{ kip-ft}$$

$$\phi_f M_n = (1.0)(5{,}858) = 5{,}858 \text{ kip-ft} > M_u = 4{,}593 \text{ kip-ft} \qquad \text{OK}$$

Note: Since $\phi_f M_n$ is much larger than M_u, prestressing steel weight is ignored.

16.1.4.3.6 Check Stress Limitations at Midspan – Service II Limit State

Determine Stress Limits

Per STP 16.6.6.4.3, stress limits for this example are summarized in Table 16.1.4-3.

Table 16.1.4-3 Stress Limits Summary

Component	Stress Type	Service II Load Case	Stress Limit
Prestressing steel	Tension	Before prestress losses due to the sum of initial prestress and permanent loads After losses due to the sum of effective prestress, permanent loads, and transient loads	$0.75f_{pu}$ = 202.5 ksi (CA Table 5.9.2.2) $0.8f_{py}$ = 194.4 ksi (CA Table 5.9.2.2)
Steel flange	Compression Tension	Before prestress losses due to the sum of initial prestress and permanent loads After losses due to the sum of effective prestress, permanent loads, and transient loads	$0.95R_hF_{yaf}$ = 31.35 ksi (Article 6.10.4.2.2) $0.95R_hF_{yaf}$ = 31.35 ksi (Article 6.10.4.2.2)
Concrete deck	Tension	Before prestress losses due to the sum of initial prestress and permanent loads	$2f_{ra} = 0.48\sqrt{f'_{ca}}$ = 0.91 ksi (Article 6.10.4.2.1)

F_{yaf} = actual yield strength of steel flange (ksi) = 33 ksi
f'_{ca} = actual concrete compressive strength (ksi) = 3.6 ksi
f_{ra} = actual modulus of rupture of concrete (ksi)
R_h = hybrid factor = 1.0 ksi

For calculation of stresses in the post-tensioned steel girders, the following assumptions are made:

- Permanent loads before post-tensioning are resisted by the existing girder section.

- The initial prestressing force is resisted by the existing short-term composite section
- Permanent loads after the post-tensioning and the effective prestressing force are resisted by the strengthened long-term composite section.
- Transient loads are resisted by the strengthened short-term section.

Check Stress in Prestressing Steel

(1) Case I - Before prestress losses due to the sum of initial prestress and permanent loads.

The jacking stress is selected as f_j = 0.75f_{pu} = 202.5 ksi OK

(2) Case II- After losses due to the sum of effective prestress, permanent loads, and transient loads.

Stress in prestressing steel shall satisfy:

$$f_{sps} = f_{pe} + \frac{M_{SDp} e_{LTs}}{n_{ps} I_{LTs}} + \frac{M_{SL} e_{STs}}{n_{ps} I_{STs}} \leq 0.8 f_{py} = 194.4 \text{ ksi} \qquad (16.1.4.3.6\text{-}1)$$

From Tables 16.1.3-9, $M_{SDp} =$ 198 kip-ft $=$ 2,376 kip-in.

M_{SL} = 1,903 kip-ft = 22,836 kip-in.

From Table 16.1.4-1, $I_{STs} = 53{,}536 \text{ in.}^3$; $e_{STs} = 31.67$ in.

From Table 16.1.4-2, $I_{LTs} = 37{,}826 \text{ in.}^3$; $e_{LTs} = 22.6$ in.

From Section 16.1.4.3.4, f_{pe} = 173.95 ksi

$$f_{sps} = 173.95 + \frac{(2{,}376)(22.6)}{(1.018)37{,}826} + \frac{(22{,}836)(31.67)}{(1.018)53{,}536}$$

$$= 188.6 \text{ ksi} < 194.4 \text{ ksi} \qquad \text{OK}$$

Check Stress in Top Steel Flange

(1) Case I - Before prestress losses due to the sum of initial prestress and permanent loads.

For the composite section, since the tension stress in the concrete deck will be checked and mostly controlled, there is no need to check stress in the top steel flange.

(2) Case II- After losses due to the sum of effective prestress, permanent loads, and transient loads.

Compression stress in the top steel flange shall satisfy:

$$f_f = \frac{M_{SD1}}{S_{NCt}} + \frac{M_{SD2}}{S_{LTt}} + \frac{M_{SDp}}{S_{LTst}} + \frac{M_{SL}}{S_{STst}} + \left(\frac{P_{pe}}{A_{LTs}} - \frac{P_{pe} e_{LTs}}{S_{LTst}} \right) \le 0.95 R_h F_{yaf} = 31.35 \text{ ksi} \tag{16.1.4.3.6-2}$$

From Tables 16.1.3-9

$$M_{SD1} = 726 \text{ kip-ft} = 8{,}712 \text{ kip-in.}$$

$$M_{SD2} = 142 \text{ kip-ft} = 1{,}704 \text{ kip-in.}$$

$$M_{SDp} = 198 \text{ kip-ft} = 2{,}376 \text{ kip-in.}$$

$$M_{SL} = 1{,}903 \text{ kip-ft} = 22{,}836 \text{ kip-in.}$$

From Table 16.1.3-12, S_{NCt} = 638 in.^{3} ;

From Table 16.1.3-14, S_{LTt} = 1,794 in.^{3}

From Table 16.1.4-1, S_{STst} = 4,431 in.^{3}

From Table 16.1.4-2, S_{LTst} = 1,789 in.^{3}; e_{LTs}= 22.60 in.; A_{LTs}= 70.8 in.^{2}

From Section 16.1.4.3.4, P_{pe} = 302 kip

$$f_f = \frac{8{,}712}{638} + \frac{1{,}704}{1{,}794} + \frac{2{,}376}{1{,}789} + \frac{22{,}836}{4{,}431} + \left(\frac{302}{70.8} - \frac{(302)(22.6)}{1{,}789} \right)$$

$$= 21.62 \text{ ksi (compression)} < 31.35 \text{ ksi} \qquad \text{OK}$$

Check Stress in Bottom Steel Flange

(1) Case I - Before prestress losses due to the sum of initial prestress and permanent loads.

Compression stress in the bottom steel flange shall satisfy:

$$f_f = \frac{M_{SD1}}{S_{NCb}} + \frac{M_{SD2}}{S_{LTb}} - \left(\frac{P_j}{A_{ST}} + \frac{P_j e_{ST}}{S_{STb}} \right) \le 0.95 R_h F_{yaf} = 31.35 \text{ ksi} \tag{16.1.4.3.6-3}$$

From Table16.1.3-12, S_{NCb} = 1,029 in.^{3}; From Table 16.1.3-14, S_{LTb} = 1,245 in.^{3}

From Table 16.1.3-13, the existing short-term composite section properties are A_{ST} = 111.28 in.^{2}; S_{STb} = 1,340 in.^{3}; e_{ST} = y_{STb} – 6.5 = 38.65 – 6.5 = 32.15 in.

From Section 16.1.4.3.2, P_j = 351.5 kip

$$f_f = \frac{8{,}712}{1{,}029} + \frac{1{,}704}{1{,}245} - \left(\frac{351.5}{111.28} + \frac{(351.5)(32.15)}{1{,}340} \right)$$
$$= -1.76 \text{ ksi (compression)} < 31.35 \text{ ksi} \qquad \text{OK}$$

(2) Case II- After losses due to the sum of effective prestress, permanent loads, and transient loads.

Tension stress in the bottom steel flange shall satisfy:

$$f_f = \frac{M_{SD1}}{S_{NCb}} + \frac{M_{SD2}}{S_{LTb}} + \frac{M_{SDp}}{S_{LTsb}} + \frac{M_{SL}}{S_{STsb}} - \left(\frac{P_{pe}}{A_{LTs}} + \frac{P_{pe} e_{LTs}}{S_{LTsb}} \right) \leq 0.95 R_h F_{yaf} = 31.35 \text{ ksi}$$

(16.1.4.3.6-4)

From Table 16.1.4-2, S_{LTsb}= 1,300 in.^{3}; e_{LTs}= 22.6 in.; A_{LTs}= 70.8 in.^{2}

From Table 16.1.4-1, S_{STsb}= 1,403 in.^{3}

$$f_f = \frac{8{,}712}{1{,}029} + \frac{1{,}702}{1{,}245} + \frac{2{,}376}{1{,}300} + \frac{22{,}836}{1{,}403} - \left(\frac{302}{70.8} + \frac{(302)(22.6)}{1{,}300} \right) \qquad \text{OK}$$
$$= 18.42 \text{ (Tension) ksi} < 31.35 \text{ ksi}$$

Check Stress in Concrete Deck

Case I - Before prestress losses due to the sum of initial prestress and permanent loads.

For this example, the concrete deck is checked for tension. The tension stress in the concrete deck shall satisfy:

$$f_f = \frac{M_{SD2}}{3nS_{LTd}} + \left(\frac{P_j}{nA_{ST}} - \frac{P_j e_{ST}}{nS_{STd}} \right) \leq 2f_{ra} = 0.91 \text{ ksi} \qquad (16.1.4.3.6\text{-}5)$$

From Table 16.1.3-14, S_{LTd} = 1,376 in.^{3}

From Table 16.1.3-13, S_{STd} = 2,902 in.^{3}; e_{ST} = y_{STb} – 6.5 = 38.65 – 6.5 = 32.15 in.; A_{ST} = 111.28 in.^{2}

From Section 16.1.4.3.2, P_j = 351.5 kip

$$f_f = \frac{1{,}702}{(3)(8)(1{,}376)} + \left(\frac{351.5}{(8)(111.28)} - \frac{(351.5)(32.15)}{(8)(2{,}902)} \right)$$
$$= -0.04 \text{ ksi (tension)} < 0.91 \text{ ksi} \qquad \text{OK}$$

16.1.4.4 Design Anchorage Bracket Components

16.1.4.4.1 Select Bracket Layout and Material Properties

The anchorage bracket sketch as shown in Figure 16.1.4-4, is selected for this example.

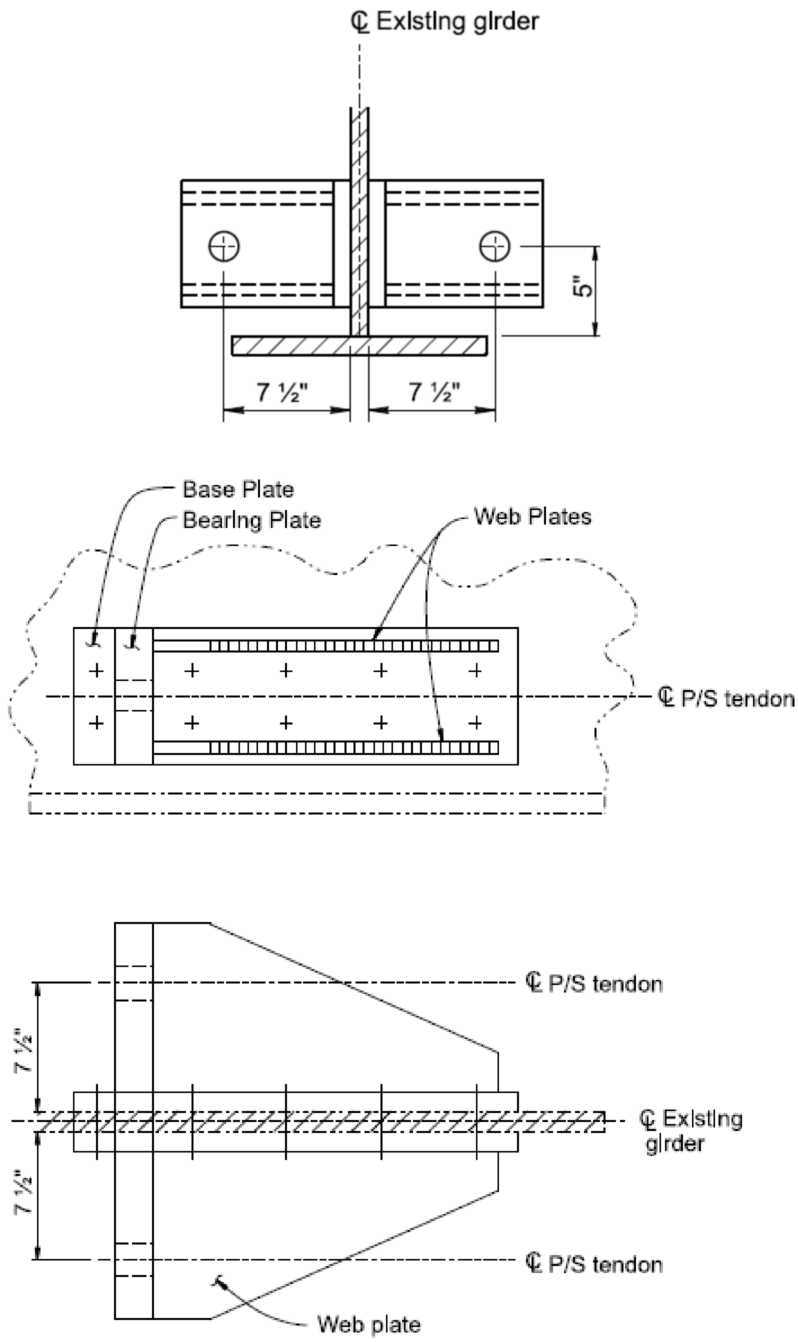


Figure 16.1.4-4 Anchorage Bracket Sketch

Try ASTM A709 Grade 36 for the welded bracket components, F3125 Grade A325 HS 3/4" bolts with threads excluded from the shear plane for the bracket connection to the existing steel girder web.

F_y = 36 ksi; F_u = 58 ksi.

16.1.4.4.2 Calculate Design Forces

In accordance with STP 16.6.6.4.4, at the strength limit state, the design force shall be taken as 1.2 times the maximum jacking force. The maximum jacking force for each bracket is developed when the stress in the prestressing steel reaches f_{ps} as calculated in Section 16.1.4.3.5.

$$P_u = \frac{1.2P_{j\max}}{2} = \frac{(1.2)P_{ps}}{2} = \frac{(1.2)(419)}{2} = 251.4 \text{ kip}$$

At the service II limit state, the maximum jacking force for each bracket is the initial jacking force.

$$P_u = \frac{P_j}{2} = \frac{351.5}{2} = 175.8 \text{ kip}$$

16.1.4.4.3 Determine Base Plate Width

The width of the base plate is determined by the required clearances for the prestressing hardware and bolts.

The minimum distance between the center of 3/4" HS bolts is 2.25", and the minimum edge distance is 1 in., as shown in Figure 16.1.4-5.

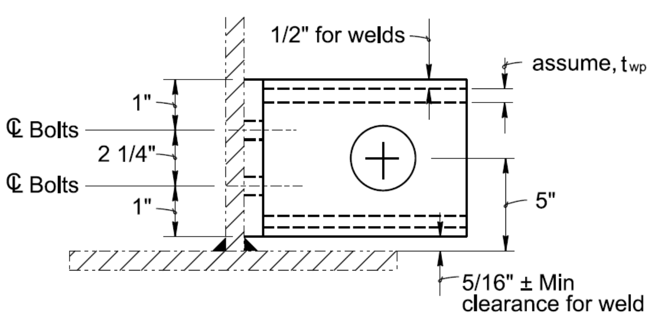


Figure 16.1.4-5 Minimum Clearance Requirements

Assume t_{wp} = 0.75 in. and use two rows of bolts with a minimum edge distance of 1.5 in. The minimum width of the base plate is obtained as:

$$w_{bp} = 2(1.5 + 0.75 + 0.5) + 3 = 8.5 \text{ in.}$$

The required distance of the prestressing tendon to the top of the bottom of the flange = 8.5/2 + 5/16 = 4.6 in. < 5 in. The assumed prestressing tendon layout meets this requirement.

16.1.4.4.4 Determine Bearing Plate Size

The size of the bearing plate is determined by approximating the size of an anchor head for the prestressing and the web plate spacing. For an 8-0.6 diameter strand prestressing system, assume that the anchor head size is 5" in diameter and the hole size is 3 1/2" in diameter.

Thus, two web plates are spaced at 6.75" and try the bearing plate size w_{brp} = 8.5 in. and L_{brp} = 11 in. as shown in Figure 16.1.4-6.

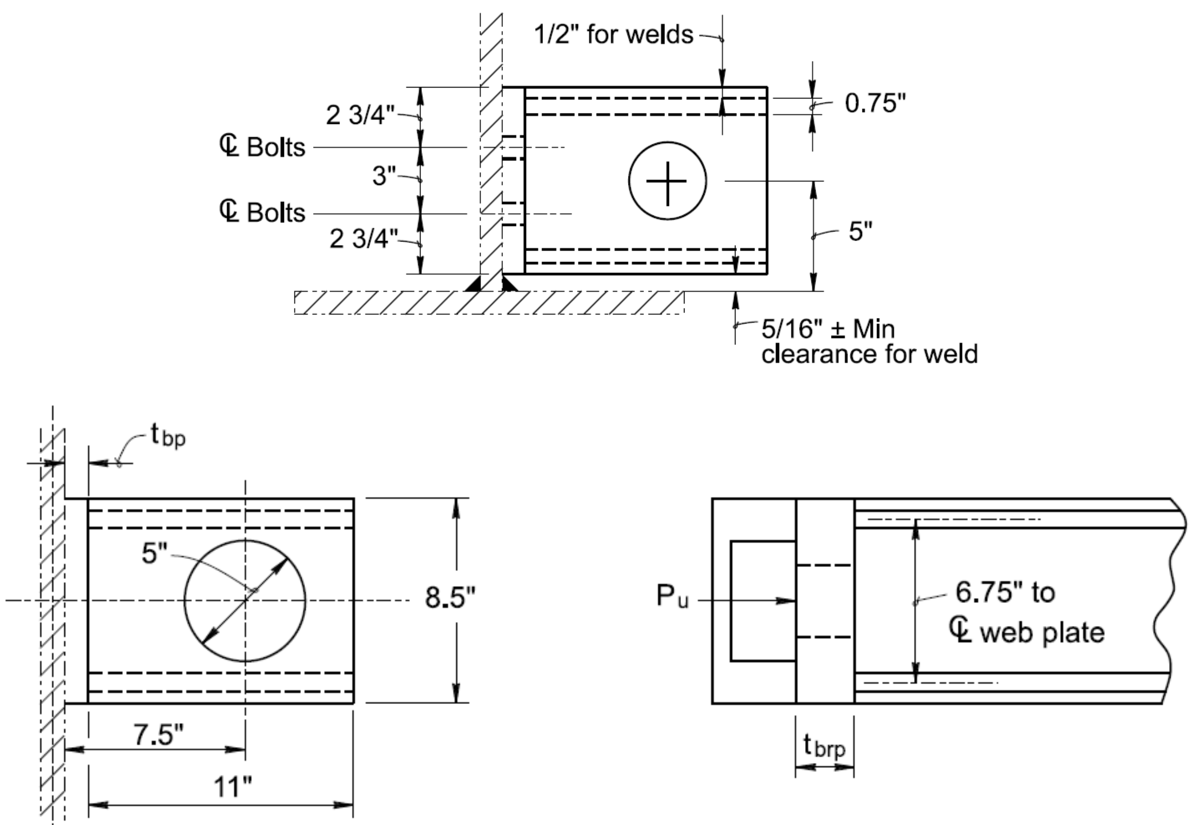


Figure 16.1.4-6 Bearing Plate Sketch

The thickness of the bearing plate is determined by a simple beam analysis conservatively since the bearing plate is rigidly supported by web plates. Assume an approximate equivalent square bearing area 4.25" x 4.25", the uniform load is

$$w = \frac{P_u}{4.25} = \frac{251.4}{4.25} = 59 \text{ k/in.}$$

A simple beam model is shown in Figure 16.1.4-7.

$$R_1 = R_2 = \frac{(59)(4.25)}{2} = 125.4 \text{ kip}$$

$$M_{u\max} = (125.4)\left(\frac{6.75}{2}\right) - \frac{(59)(4.25)^2}{4} = 157 \text{ kip-in.}$$

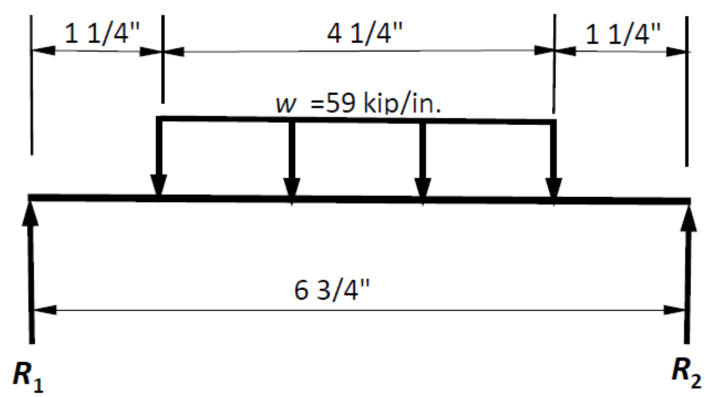


Figure 16.1.4-7 Simple Beam Model

Try t_{brp} = 2 in. The net width of the bearing plate is:

$$b_{brpn} = (11 - 3.5) = 7.5 \text{ in.}$$

$$S_{brp} = \frac{b_{brpn} t_{brp}^2}{6} = \frac{(7.5)(2)^2}{6} = 5.0 \text{ in.}^3$$

$$\phi_f M_n = \phi_f S_{brp} F_y = (1.0)(5.0)(36)$$

$$= 180 \text{ kip-in.} > M_{u\max} = 157 \text{ kip-in.} \qquad \text{OK}$$

Use a bearing plate 2" x 8 1/2" x 11"

16.1.4.4.5 Determine Web Plate Size

The web plate is usually trimmed to a triangular shape since a triangular plate provides the stiffer support than a rectangular shape and minimizes unnecessary weight. The bracket shape should be proportioned so that the load transfer is more in shear than in bending. Generally, an aspect ratio between 1.5 and 2.0 is recommended (ratio of supported edge to loaded edge). The web sketch is shown in Figure 16.1.4-8. The required thickness of the web plate is determined using the procedure presented by Salmon, Johnson, and Malhas (2009).

Strength Requirement - Plastic Strength Method

$$t_{wp} > \frac{P_u}{F_y \sin^2 \alpha \left[\sqrt{4e^2 + b^2} - 2e^2 \right]} \qquad (16.1.4.4.5\text{-}1)$$

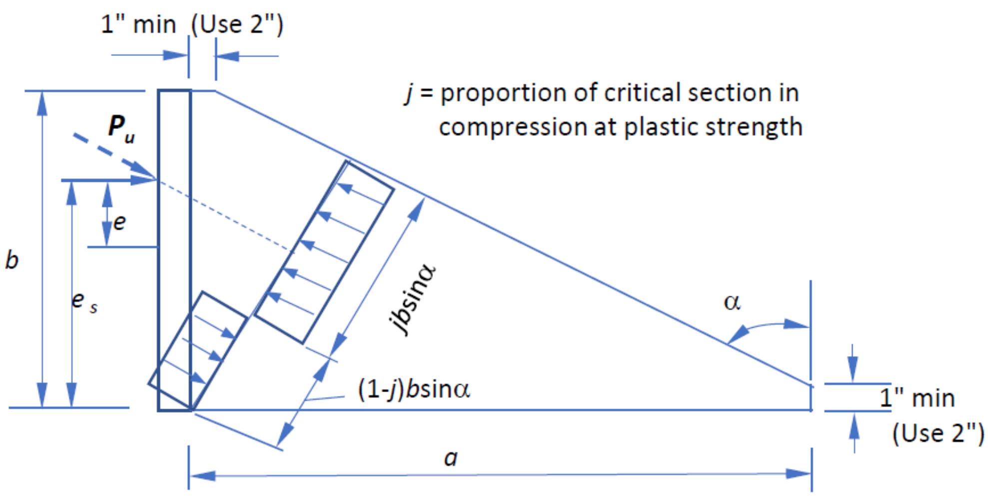


Figure 16.1.4-8 Web Plate Sketch

where:

$b = 11\text{ in.}$

$e_s = 7.5 - 1.0 = 6.5\text{ in.}$ (Assume base plate thickness of 1.0 in.)

$$e = e_s - \frac{b}{2} = 6.5 - \frac{11}{2} = 1.0\text{ in.}$$

Try a = 18 in. and t_{wp} = 0.75 in.

$$\frac{a}{b} = \frac{18}{11} = 1.64 \quad > 1.5 \qquad \text{OK}$$

$$\alpha = \text{Tan}^{-1}\left(\frac{a-2}{b-2}\right) = \text{Tan}^{-1}\left(\frac{18-2}{11-2}\right) = 1.058\text{ (rad)}$$

$$t_{wp} = 0.75 \text{ in. } > \frac{\frac{251.4}{2}}{(36)\sin^2(1.058)\left[\sqrt{4(1.0)^2+(11)^2}-2(1.0)^2\right]} = 0.5 \text{ in. OK}$$

Stability Requirement

$$\text{if} \quad 0.5 \le \frac{b}{a} \le 1.0, \quad t_{wp} \ge \frac{b\sqrt{F_y}}{125} \qquad (16.1.4.4.5\text{-}2)$$

$$\text{if} \quad 1.0 \le \frac{b}{a} \le 12.0, \quad t_{wp} \ge \frac{b\sqrt{F_y}}{125\left(\frac{b}{a}\right)} \qquad (16.1.4.4.5\text{-}3)$$

$$\frac{b}{a} = \frac{11}{18} = 0.61, \quad t_{wp} = 0.75 \text{ in. } > \frac{b\sqrt{F_y}}{125} = \frac{(11)\sqrt{36}}{125} = 0.528 \text{ in.} \qquad \text{OK}$$

Use two web plates 3/4" x 11" x 18"

16.1.4.5 Design Anchorage Bracket Connection to Girder Web

Per STP 16.6.6.4.5, a post-tensioning anchorage bracket shall be connected to the existing girders by high strength bolts. An anchorage bracket connection shall be designed as slip-critical for the combined shear and tension.

16.1.4.5.1 Calculate Shear Resistance Per Bolt

For Grade A325 3/4” bolt,

A_b = cross-sectional area = 0.442 in.^{2}

F_{ub} = tensile strength of bolt = 120 ksi (AASHTO 6.4.3.1)

N_s = number of slip plane in connection = 1

The nominal shear resistance of a bolt is obtained as:

$$R_{n1} = 0.56 A_b F_{ub} N_s. = (0.56)(0.442)(120)(1) = 29.7 \text{ kip} \quad \text{(AASHTO 6.13.2.7-1)}$$

The nominal bearing resistance at bolt holes on the base plate is obtained as:

For Grade A325 3/4” bolt, the nominal diameter of a bolt, d = 0.75 in.; the bolt hole diameter is 0.813 in. (AASHTO Table 6.13.2.4.2-1); Try the edge distance of 2.0 in.

$$\because \; L_c = \text{the clear edge distance} = 2.0 - (0.813/2) = 1.59 \text{ in.} > 2d = 1.5 \text{ in.}$$

$$R_{n2} = 2.4dt_{bp}F_u = (2.4)(0.75)(1.0)(58) = 104.4 \text{ kip} \qquad \text{(AASHTO 6.13.2.9-1)}$$

The nominal bearing resistance at bolt holes on the existing girder web is obtained as:

Since the girder web resists the two brackets, only half of the thickness contributes to the bearing resistance.

$$R_{n3} = 2.4d\frac{t_w}{2}F_u = (2.4)(0.75)\left(\frac{0.375}{2}\right)(58) = 19.6 \text{ kip} \qquad \text{(AASHTO 6.13.2.9-1)}$$

It is seen that the bearing resistance at bolt holes on the existing girder web controls and the nominal shear resistance per bolt is obtained as: .

$$R_n = \min(R_{n1}, R_{n2}, R_{n3}) = \min(29.7, 104.4, 19.6) = 19.6 \text{ kip}$$

16.1.4.5.2 Determine Required Bolt Number and Pattern

$$N_{reqd} = \frac{P_u}{\phi_s R_n} = \frac{251.4}{(0.8)(19.6)} = 16$$

Use two rows of bolts at a spacing of 2.5 in. and an edge distance of 2.0 in. The bolt pattern is shown in Figure 16.1.4-9.

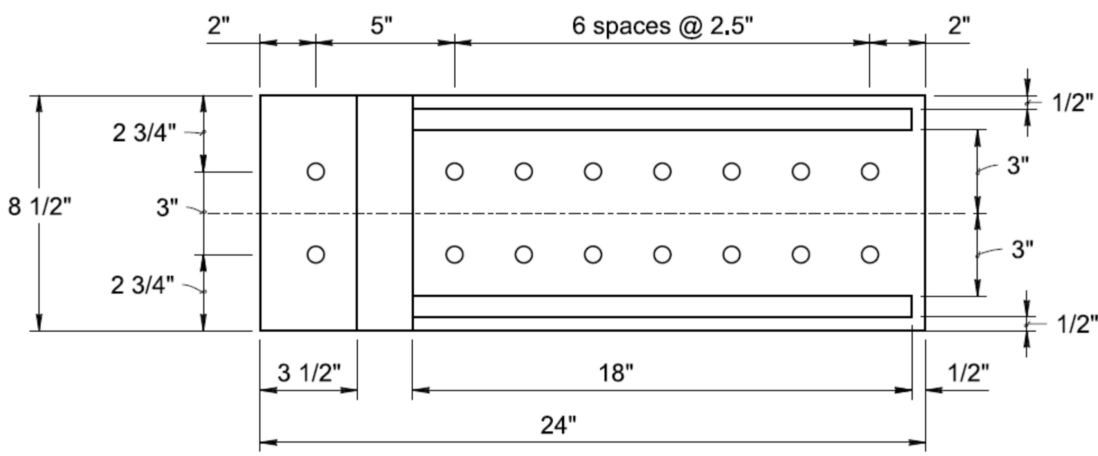


Figure 16.1.4-9 Bolt Pattern

16.1.4.5.3 Check Bolt Connection under Combined Shear and Tension

Calculate Bolt Tension Force

$$M_u = P_u(7.5 - 1.0) = (251.4)(6.5) = 1{,}634.1 \text{ kip-in.}$$

Assume the tension force is elastically distributed in bolts, and the compression force is resisted by the base plate, as shown in Figure 16.1.4-10.

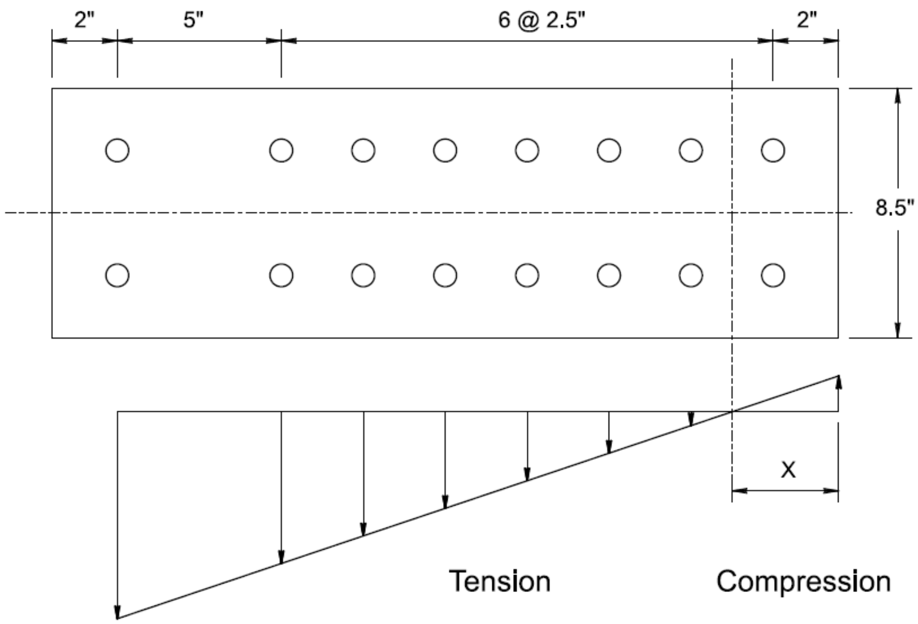


Figure 16.1.4-10 Bolt and Base Plate Force Distribution

The elastic neutral axis is obtained by equating the first moments of the base plate compression area and tension bolt areas about the neutral axis (N.A.).

$$\sum A_b y_i = \frac{w_{bp} x^2}{2}$$

Assume seven rows of bolts resist the tension force

$$\sum A_b y_i = (2)(0.442)\left[(22 - x) + (17 - x) + (14.5 - x) + (12 - x) + (9.5 - x) + (7 - x) + (4.5 - x)\right]$$

$$= (0.884)(86.5 - 7x) = \frac{w_{bp} x^2}{2} = \frac{(8.5)x^2}{2} = 4.25x^2$$

$$4.25x^2 + 6.188x - 76.47 = 0$$

$$x = \frac{-6.188 \pm \sqrt{6.188^2 - 4(4.25)(-76.47)}}{2(4.25)} = \begin{cases} 3.58 \text{ in.} \\ -5.03 \text{ in.} \end{cases}$$

Since the negative value has no practical meaning, x = 3.58 in. meets the assumption of seven rows of bolts in tension. The combined moment of inertia of the bolt group and the compression block area about the neutral axis is obtained as:

$$\begin{aligned} I_x &= \frac{w_{bp} x^3}{3} + \sum A_b y_i^2 \\ &= \frac{(8.5)(3.58)^3}{3} + (0.884)\begin{bmatrix} (22-3.58)^2 + (17-3.58)^2 + (14.5-3.58)^2 \\ (12-3.58)^2 + (9.5-3.58)^2 + (7-3.58)^2 + (4.5-3.58)^2 \end{bmatrix} \\ &= 799 \text{ in.}^4 \end{aligned}$$

The maximum tension force occurs at the most left row of bolts and is obtained as follows:

$$T_u = \frac{M_u(22-3.58)A_b}{I_x} = \frac{(1{,}634.1)(18.42)(0.442)}{799} = 16.65 \text{ kip}$$

Per AASHTO 6.13.2.11, the nominal tensile resistance of a bolt subjected to combined shear and axial tensile, T_n is obtained as:

For one bolt,

$$P_u = \frac{251.4}{16} = 15.71 \text{ kip}$$

$$R_n = 29.7 \text{ kip}$$

$$\because \frac{P_u}{R_n} = \frac{15.71}{29.7} = 0.53 > 0.33$$

$$T_n = 0.76 A_b F_{ub}\sqrt{1-\left(\frac{P_u}{\phi_s R_n}\right)^2} = (0.76)(0.442)(120)\sqrt{1-\left(\frac{15.71}{(0.8)(29.7)}\right)^2} = 30.24 \text{ kip}$$

$$\phi_t T_n = (0.8)(30.24) = 24.19 \text{ kip} > T_u = 16.65 \text{ kip} \qquad \text{OK}$$

16.1.4.5.4 Check Flexural Resistance of Cantilever Portion of Base Plate

The cantilever portion of the base plate is subjected to the bending moment due to tension force in the bolts, as shown in Figure 16.1.4-11.

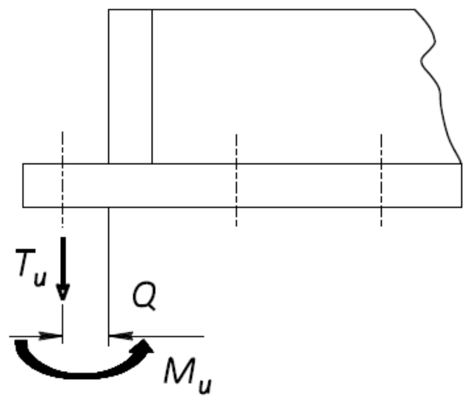


Figure 16.1.4-11 Bending Moment on Base Plate

$$M_u = T_u(1.5) = (2)(16.65)(1.5) = 49.95 \text{ kip-in.}$$

For the base plate, t_{bp} = 1.0 in.

$$\phi_f M_n = F_y S_{bp} = (1.0)(36)\left(\frac{8.5(1.0)^2}{6}\right) = 51 \text{ kip-in.} > M_u = 49.95 \text{ kip-in.} \quad \text{OK}$$

Use base plate 1"x8 1/2"x24"

16.1.4.5.5 Check Slip Resistance at Service II Limit State

Per STP 16.6.6.4.5, at the jacking stage, bolted connections shall be designed to prevent slip for the maximum jacking force.

$$P_u = \frac{P_j}{2} = \frac{351.5}{2} = 175.8 \text{ kip}$$

Calculate Slip Resistance Per Bolt

The nominal slip resistance per bolt is:

$$R_n = K_h K_s N_s P_t \quad \text{(AASHTO 6.13.2.8-1)}$$

where

K_h = the hole size factor and is equal to 1.0 for the standard hole (AASHTO Table 6.13.2.8-2)

K_s = the surface condition factor and is taken as 0.5 for Class B surface condition (AASHTO Table 6.13.2.8-3)

N_s = the number of slip planes and is equal to 1.0

P_t = the minimum required bolt tension and is equal to 28 kips (AASHTO Table 6.13.2.8-1)

$$R_n = K_h K_s N_s P_t = (1.0)(0.5)(1)(28) = 14.0 \text{ kip} \qquad \text{(AASHTO 6.13.2.8-1)}$$

Check Slip Resistance Per Bolt

$$R_n = 14.0 \text{ kip} > R_u = \frac{P_u}{N_b} = \frac{175.8}{16} = 10.99 \text{ kip} \quad \text{OK}$$

16.1.4.6 Design Anchorage Bracket Plate Weld Connection

16.1.4.6.1 Weld Types and Layout

For this example, complete penetration groove welds are used to attach the bearing plate to the web plate, and fillet welds are used to attach the web plates to the base plate and the bearing plate. The welds are subject to shear and bending. The welds on the base plate are shown in Figure 16.1.4-12.

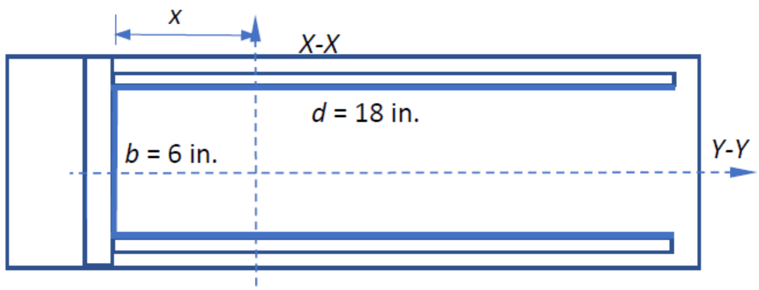


Figure 16.1.4-12 Welds on Base Plate

16.1.4.6.2 Calculate Force Resultants on Welds

Assume that the weld along the bearing plate is equivalent to fillet welds and shear is only resisted by web plate welds on both sides, and then

$$P_u = 251.4 \text{ kip}$$

$$M_u = P_u(7.5 - 1.0) = (251.4)(6.5) = 1{,}634.1 \text{ kip-in.}$$

The location of the neutral axis of the welds shown in Figure 16.1.4-12 is obtained as:

$$x = \frac{d^2}{2d+b} = \frac{18^2}{2(18)+6} = 7.71 \text{ in.}$$

$$I_y = (2)\left[\frac{2d^3}{12} + 2d\left(\frac{d}{2} - x\right)^2 + bx^2\right]$$

$$= (2)\left[\frac{2(18)^3}{12} + (2)(18)\left(\frac{18}{2} - 7.71\right)^2 + (6)(7.71)^2\right]$$

$$= 2{,}777.1 \text{ in.}^4$$

The shear component on the weld is obtained as:

$$s_{uvx} = \frac{P_u}{4d} = \frac{251.4}{4(18)} = 3.49 \text{ kip/in.}$$

The maximum tension/compression component on the weld is obtained as

$$s_{utx} = \frac{M_u x}{I_y} = \frac{(1{,}634.1)(7.71)}{2777.1} = 4.54 \text{ kip/in. (Tension)}$$

$$s_{utx} = \frac{M_u(d-x)}{I_y} = \frac{(1{,}634.1)(18-7.71)}{2777.1} = 6.05 \text{ kip/in. (compression)}$$

The maximum resultant force flow:

$$s_u = \sqrt{3.49^2 + 6.05^2} = 6.98 \text{ kip/in.}$$

16.1.4.6.3 Check Weld Resistance

Use E70XX weld metal, F_{exx} = 70 ksi, the resistance of a fillet weld is:

$$R_r = 0.6\phi_{e2}F_{exx} = (0.6)(0.8)(70) = 33.6 \text{ ksi} \qquad \text{(AASHTO 6.13.3.2.4-1)}$$

Try 2-5/16 in. fillet welds, t_{weld} = 2(5/16) in. The shear flow resistance is

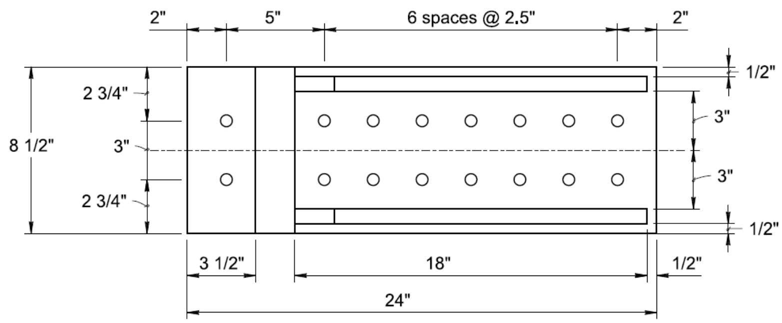
$$s_r = t_{weld}(0.707)R_r = 2\left(\frac{5}{16}\right)(0.707)(33.6)$$

$$= 14.85 \text{ kip/in.} > s_u = 6.98 \text{ kip-in.} \qquad \text{OK}$$

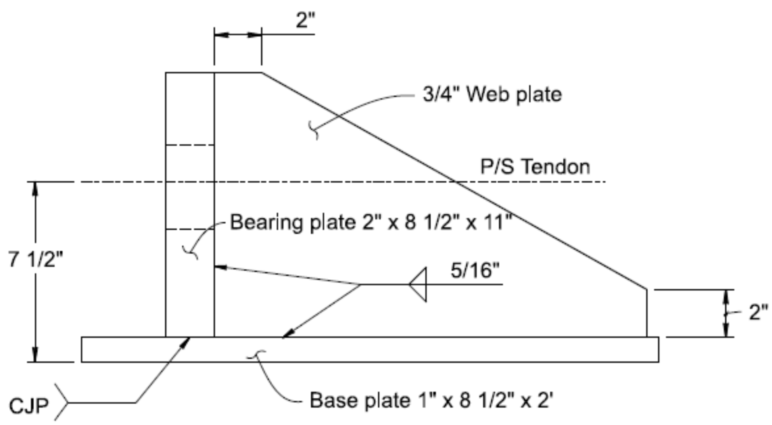
Use 2- 5/16 in. fillet welds for connection between the web plates and base plate, and bearing plate.

16.1.4.6.4 Final Bracket Details

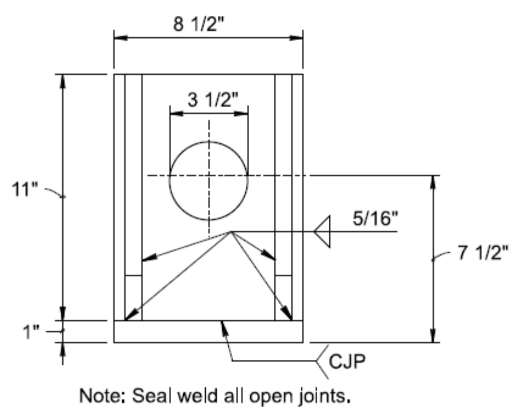
The final bracket details are shown in Figure 16.1.4-13.



(a) Plan View



(b) Elevation View



(c) End View

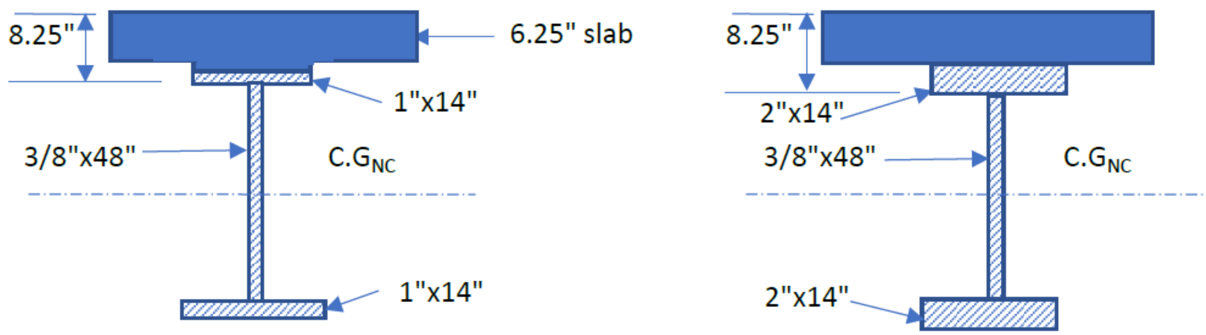
Figure 16.1.4-13 Final Bracket Details

16.1.5 FLEXURAL STRENGTHENING DESIGN EXAMPLE 3 – COMPOSITE ACTION

The following is an example of strengthening a simple span noncomposite steel girder bridge by the composite action by adding shear studs.

16.1.5.1 Existing Steel Girder Bridge Data

Existing steel girder bridge data are the same as Example 1, except the girder is a noncomposite section, and the sections at end spans and the midspan are shown in Figure 16.1.5-1.



(a) End Span (b) Midspan

Figure 16.1.5-1 Existing Girder Sections

16.1.5.2 Design Requirement

Perform the following strengthening design portions for an interior plate girder in accordance with *STP 16.6* (Caltrans 2021) and AASHTO-CA BDS-8 (AASHTO, 2017; Caltrans, 2019). A similar procedure can be used for strengthening exterior girders and is not illustrated here.

Step 1: Calculate Factored Moments

Step 2: Check Flexural Resistance of Noncomposite Section at 0.5 Point

Step 3: Design Composite Section

Step 4: Design Shear Connectors

16.1.5.3 Calculate Factored Moments

Since only *DC1* is changed from Example 1, recalculate *DC1* as follows

- Weight of deck slab: $W_s = 527$ lb/ft (Example 1, Section 16.1.3.4.1)
- Girder Self Weight

  Section at ends:

  Gross section area:

$$A_{ge} = (14 \times 1.0) + (48 \times 0.375) + (14 \times 1.0) = 46.0 \text{ in.}^2$$

  Weight of steel girder: $W_{ge} = A_{ge} w_s = (46.0)(490/144) = 157$ lb/ft

  Section at midspan:

  Gross section area:

$$A_{gm} = (14 \times 2.0) + (48 \times 0.375) + (14 \times 2.0) = 74 \text{ in.}^2$$

  Weight of steel girder: $W_{gm} = A_{gm} w_s = (74)(490/144) = 252$ lb/ft

- Stiffener Weight

  $W_{st} = 11$ lb/ft (Example 1, Section 16.1.3.4.1)

- Bracing Weight

  $W_{br} = 10$ lb/ft (Example 1, Section 16.1.3.4.1)

- Miscellaneous Dead Load for Haunch, Welds, etc.

  $W_{misc} = 10$ lb/ft (Example 1, Section 16.1.3.4.1)

Total DC1

End Span

$$DC1_{end} = W_s + W_{ge} + W_{st} + W_{br} + W_{misc}$$
$$= 527 + 157 + 11 + 10 + 10 = 715 \text{ lb/ft} = 0.715 \text{ k/ft}$$

Midspan

$$DC1_{mid} = W_s + W_{gm} + W_{st} + W_{br} + W_{misc}$$
$$= 527 + 252 + 11 + 10 + 10 = 810 \text{ lb/ft} = 0.810 \text{ k/ft}$$

DC1 is shown in Figure 16.1.5-2.

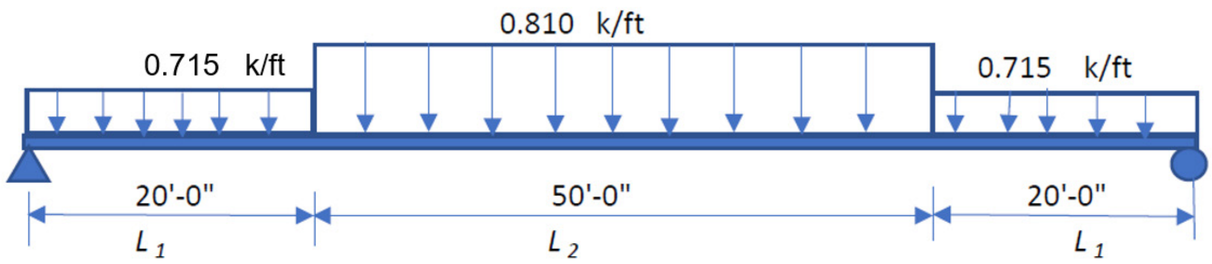


Figure 16.1.5-2 DC1 Dead Load

Unfactored dead load moments for *DC1* are calculated in Table 16.1.5-1. *DC2* and *DW* moments are obtained from Example 1, Table 16.1.3-1.

Table 16.1.5-1 Unfactored Dead Load Moment Summary for an Interior Girder

Point x/L	Moment		
	DC1	*DC2*	*DW*
	M_{DC1} (kip-ft)	M_{DC2} (kip-ft)	M_{DW} (kip-ft)
0.0	0	0	0
0.1	282	51	71
0.2	506	91	127
0.3	670	119	167
0.4	768	136	191
0.5	801	142	198
0.6	768	136	191
0.7	670	119	167
0.8	506	91	127
0.9	282	51	71
1.0	0	0	0

The calculations for live load distribution factors, and live load moments and shears, are similar to the flexural strengthening design Example 1 shown in Section 16.1.3.5 and are not illustrated here. Using factored live load moments obtained from Example 1, Table 16.1.3-5, the factored moment envelope for an interior girder is summarized in Table 16.1.5-2.

Table 16.1.5-2 Factored Moment Envelope for an Interior Girder

Point x/L	Dead Load			Live Load ($LL+IM$)		Load Combination	
	$DC1$	$DC2$	DW	$HL93$	$P15$	Strength I	Strength II
	$1.25M_{DC1}$ (kip-ft)	$1.25M_{DC2}$ (kip-ft)	$1.5M_{DW}$ (kip-ft)	$1.75DFM_{HL93}$ (kip-ft)	$1.35DFM_{P15}$ (kip-ft)	M_u (kip-ft)	M_u (kip-ft)
0.0	0	0	0	0	0	0	0
0.1	352	64	107	990	1261	1513	1784
0.2	632	113	191	1738	2017	2674	2954
0.3	837	149	250	2245	2774	3481	4010
0.4	960	170	286	2543	3026	3959	4442
0.5	1001	177	298	2615	3278	4091	4754
0.6	960	170	286	2543	3026	3959	4442
0.7	837	149	250	2245	2774	3481	4010
0.8	632	113	191	1738	2017	2674	2954
0.9	352	64	107	990	1261	1513	1784
1.0	0	0	0	0	0	0	0

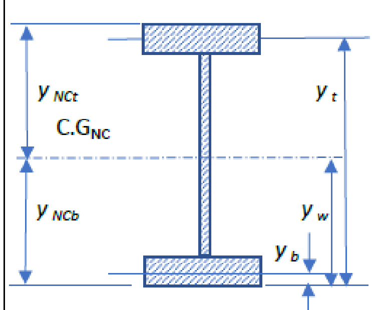
16.1.5.4 Check Flexural Resistance of Noncomposite Section at 0.5*L*

16.1.5.4.1 Calculate Section Properties for Midspan Section at 0.5*L*

Section properties for the noncomposite section at the midspan are calculated in Table 16.1.5-3.

Table 16.1.5-3 Section Properties for Midspan Section at 0.5*L*

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	$y_i - y_{NCb}$ (in.)	$A_i(y_i - y_{NCb})^2$ (in.^{4})	I_o (in.^{4})
Top Flange (2x14)	28.00	51.00	1428.00	25.00	17,500	9.33
Web (3/8x48)	18.00	26.00	468.00	0.00	0	3,456
Bottom Flange (2 x14)	28.00	1.00	28.00	-25.00	17,500	9.33
Total (Σ)	74.00		1,924		35,000	3,475



y_i = Component CG to the bottom of bottom flange

I_o = Moment of inertia of component about its CG

$y_{NCb} = \Sigma A_i y_i / \Sigma A_i$ = 26.00 in.

$y_{NCt} = (2+48+2)-26$ = 26.00 in.

$I_{NC} = \Sigma I_o + \Sigma A_i (y_i - y_{NCb})^2$ = 38,475 in.^{4}

$S_{NCb} = I_{NC} / y_{NCb}$ = 1,480 in.^{3}

$S_{NCt} = I_{NC} / y_{NCt}$ = 1,480 in.^{3}

16.1.5.4.2 Check Flexural Resistance for Midspan Section at 0.5*L*

For this example of the straight girders without skew, the flexural resistance of the noncomposite section at the midspan is calculated in accordance with Article A6.1.

Check Applicability of Article A6.1

- F_{ya} = 33 ksi < 70 Ksi OK
- $\frac{2D_c}{t_w} = \frac{(2)(24)}{0.375} = 128.0 < 5.7\sqrt{\frac{E_s}{F_{yc}}} = 5.7\sqrt{\frac{29,000}{33}} = 169.0$ OK (AASHTO A6.1-1)

where

D_c = depth of web in compression in the elastic range (in.) = 24 in.

t_w = thickness of the web = 0.375 in.

- $\frac{I_{yc}}{I_{yt}} \geq 0.3$ (AASHTO A6.1-2)

For this example, the section is symmetric; moments of inertia of the compression flange and tension flange of the steel section about the vertical axis in the plane of the web are the same, $I_{yc} = I_{yt}$

$$\frac{I_{yc}}{I_{yt}} = 1.0 > 0.3 \quad \text{OK}$$

The midspan section meets all the above requirements, and Article A6.1 is applicable.

Calculate Plastic Moment, M_p

Figure 16.1.5-3 shows the plastic moment state for the midspan section.

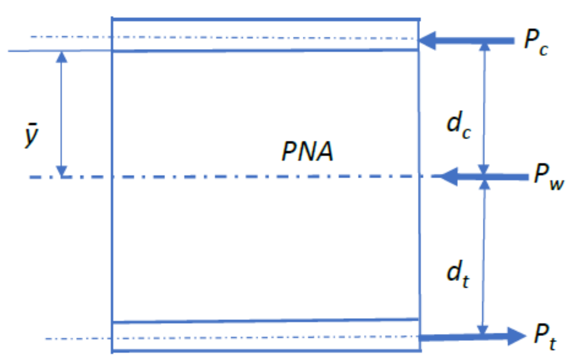


Figure 16.1.5-3 Plastic Moment State at Midspan Section

The yield force in the top compression flange:

$$P_c = A_{fc}F_{yc} = (14 \times 2.0)(33) = 924 \text{ kip}$$

The yield force in the web:

$$P_w = A_w F_{yw} = (48 \times 0.375)(33) = 594 \text{ kip}$$

The yield force in the bottom tension flange:

$$P_t = A_{ft}F_{yt} = (14 \times 2.0)(33) = 924 \text{ kip}$$

$$\because \; P_t + P_w = 924 + 594 = 1{,}518 \text{ kip} \quad > P_c = 924 \text{ kip}$$

$\therefore$ The PNA is within the web (AASHTO Table D6.1-1, Case I)

$$\bar{y} = \left(\frac{D}{2}\right)\left[\frac{P_t - P_c}{P_w} + 1\right] = \left(\frac{48}{2}\right)\left[\frac{924 - 924}{594} + 1\right] = 24 \text{ in.}$$

$$d_c = d_t = \bar{y} + \frac{t_{fc}}{2} = 24 + \frac{2}{2} = 25 \text{ in.}$$

$$d_w = \frac{D}{2} - \bar{y} = \frac{48}{2} - 24 = 0 \text{ in.}$$

From AASHTO Table D6.1-1, the plastic moment is calculated by summarizing moments taking all forces about the PNA as follows:

$$\begin{aligned} M_p &= \frac{P_w}{2D}\left[\bar{y}^2 + \left(D - \bar{y}^2\right)\right] + \left(P_c d_c + P_t d_t\right) \\ &= \frac{594}{2(48)}\left[24^2 + (48 - 24)^2\right] + (924)(25) + (924)(25) \\ &= 53{,}328 \text{ kip-in.} = 4{,}444 \text{ kip-ft} \end{aligned}$$

Calculate Yield Moment, M_y

Since the steel section is symmetric, the yield moment with respect to the tension flange, M_{yt} is the same as the yield moment with respect to the compression flange M_{yc}. Elastic section moduli are the same for both flanges $S_{NCt} = S_{NCb}$ = 1,480 in.^{3}

$$M_{yc} = M_{yt} = M_y = F_y S_{NCb} = (33)(1{,}480) = 48{,}840 \text{ kip-in.} = 4{,}070 \text{ kip-ft}$$

Calculate Web Plastification Factor, R_{pc} and R_{pt}

For this example, the hybrid factor R_h = 1.0, the depth of the web in compression in the elastic range is equal to the depth of the web in compression in the plastic moment state, $D_c = D_{cp}$ = 24 in., the limiting slenderness ratio for a compact web corresponding to $2D_{cp}/t_w$ is $\lambda_{pw(Dcp)}$ and is calculated as follows:

$$\lambda_{rm} = 5.7\sqrt{\frac{E_s}{F_{yc}}} = 5.7\sqrt{\frac{29{,}000}{33}} = 169.0 \qquad \text{(AASHTO A6.2.1-3)}$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} = \frac{\sqrt{\frac{29{,}000}{33}}}{\left(0.54\frac{4{,}444}{(1.0)(4{,}070)} - 0.09\right)^2} \qquad \text{(AASHTO A6.2.1-2)}$$

$$= 118.8 < \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) = (169)\left(\frac{24}{24}\right) = 169.0$$

$$\because \quad \frac{2D_{cp}}{t_w} = \frac{(2)(24)}{0.375} = 128.0 > \lambda_{pw(D_{cp})} = 118.8$$

and

$$\lambda_w = \frac{2D_c}{t_w} = \frac{(2)(24)}{0.375} = 128.0 < \lambda_{rw} = 169.0$$

The steel section qualifies as a noncompact web section (Article A6.2.2).

The limiting slenderness ratio for a compact web corresponding to $2D_c/t_w$ is calculated as:

$$\lambda_{pw(D_c)} = \lambda_{pw(D_{cp})}\left(\frac{D_c}{D_{cp}}\right) = 118.8\left(\frac{24}{24}\right) = 118.8$$

$$R_{pc} = R_{pt} = \left[1 - \left(1 - \frac{R_h M_{yc}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(D_c)}}{\lambda_{rw} - \lambda_{pw(D_c)}}\right)\right]\left(\frac{M_p}{M_{yc}}\right)$$

$$= \left[1 - \left(1 - \frac{(1.0)(4{,}070)}{4{,}444}\right)\left(\frac{128.0 - 118.8}{169.0 - 118.8}\right)\right]\left(\frac{4{,}444}{4{,}070}\right) \qquad \text{(AASHTO A6.2.2-4)}$$

$$= 1.075 < \frac{M_p}{M_{yc}} = \frac{4{,}444}{4{,}070} = 1.092$$

Calculate Flange Local Buckling Resistance, $M_{nc(FLB)}$

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{14}{(2)(2)} = 3.5 \qquad \text{(AASHTO A6.3.2-3)}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000}{33}} = 11.3 \qquad \text{(AASHTO A6.3.2-4)}$$

$$\because \lambda_f = 3.5 < \lambda_{pf} = 11.3$$

$$M_{nc(FLB)} = R_{pc}M_{yc} = (1.075)(4{,}070) = 4{,}375 \text{ kip-ft} \qquad \text{(AASHTO A6.3.2-1)}$$

Calculate Tension Flange Yielding Resistance, M_{nt}

$$M_{nt} = R_{pt}M_{yt} = (1.075)(4{,}070) = 4{,}375 \text{ kip-ft} \qquad \text{(AASHTO A6.4-1)}$$

Calculate Lateral-Torsional Buckling Resistance, $M_{nc(LTB)}$

For this example, there are three intermediate diaphragms at a spacing of 22.5 ft. The unbraced length L_b = 22.5 ft = 270 in. The limiting unbraced length to achieve the nominal flexural resistance $R_{pc}M_{yc}$ under uniform bending, L_p is calculated as:

$$L_p = 1.0r_t\sqrt{\frac{E}{F_{yc}}} \qquad \text{(AASHTO A6.3.3-4)}$$

The effective radius of gyration for lateral torsional buckling, r_t is obtained as:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc}t_{fc}}\right)}} = \frac{14}{\sqrt{12\left(1+\frac{1}{3}\frac{(24)(0.375)}{(14)(2)}\right)}} = 3.84 \qquad \text{(AASHTO A6.3.3-10)}$$

$$L_p = 1.0r_t\sqrt{\frac{E}{F_{yc}}} = (1.0)(3.84)\sqrt{\frac{29{,}000}{33}} = 113.83 \text{ in.}$$

The limiting unbraced length to achieve the nominal onset of yielding in either flange under the uniform bending with consideration of compression-flange residual stress effects, L_r is:

$$L_r = 1.95r_t\frac{E}{F_{yr}}\sqrt{\frac{J}{S_{xc}h}}\sqrt{1+\sqrt{1+6.76\left(\frac{F_{yr}}{E}\frac{S_{xc}h}{J}\right)^2}} \qquad \text{(AASHTO A6.3.3-5)}$$

The depth between the centerline of flanges, h, is:

$$h=\frac{t_{fc}}{2}+D+\frac{t_{ft}}{2}=\frac{2.0}{2}+48+\frac{2.0}{2}=50\text{ in.}$$

St. Venant Torsional constant, J, is:

$$J=\frac{Dt_w^3}{3}+\frac{b_{fc}t_{fc}^3}{3}\left(1-0.63\frac{t_{fc}}{b_{fc}}\right)+\frac{b_{ft}t_{ft}^3}{3}\left(1-0.63\frac{t_{ft}}{b_{ft}}\right)$$

$$=\frac{(48)(0.375)^3}{3}+\frac{(14)2^3}{3}\left(1-0.63\frac{2}{14}\right) \qquad \text{(AASHTO A6.3.3-9)}$$

$$+\frac{(14)2^3}{3}\left(1-0.63\frac{2}{14}\right)=68.79\text{ in.}^4$$

The elastic section modulus about the major axis of the section to the compression flange and the tension flange is the same as:

$$S_{xc}=\frac{M_{yc}}{F_{yc}}=S_{xt}=\frac{M_{yt}}{F_{yt}}=\frac{(4{,}070)(12)}{33}=1{,}480\text{ in.}^3$$

The compression-flange stress at the onset of nominal yielding within the cross section, F_{yr} is:

$$F_{yr}=\text{smaller}\begin{Bmatrix}0.7F_{yc}=(0.7)(33)=23.1\text{ ksi}\\ \dfrac{R_hF_{yt}S_{xt}}{S_{xc}}=\dfrac{(1.0)(33)(1{,}470)}{1{,}470}=33\text{ ksi}\\ F_{yw}=33\text{ ksi}\end{Bmatrix}=23.1\text{ ksi}>0.5F_{yc}=16.5\text{ ksi}$$

$$L_r=1.95r_t\frac{E}{F_{yr}}\sqrt{\frac{J}{S_{xc}h}}\sqrt{1+\sqrt{1+6.76\left(\frac{F_{yr}}{E}\frac{S_{xc}h}{J}\right)^2}}$$

$$=1.95(3.84)\frac{29{,}000}{23.1}\sqrt{\frac{68.79}{(1{,}480)(50)}}\sqrt{1+\sqrt{1+6.76\left(\frac{23.1}{29{,}000}\frac{(1{,}480)(50)}{68.79}\right)^2}}$$

$$=531.7\text{ in.}$$

In this example, since $L_p=113.83$ in. < $L_b=270$ in. < $L_r=531.7$ in. C_b, the moment gradient modifier for the lateral torsional buckling is calculated in accordance with CA Eq. (6.10.8.2.3-7). The factored moment envelope for the unbraced segment at Midspan from 0.2 to 0.5L listed in Table 16.1.5-2 is shown in Figure 16.1.5-4. Moments M_{max}, M_A, M_B, and M_C are estimated from the factored moment envelope shown in Figure 16.1.5-4.

It is obvious that the absolute value of the maximum moment in the unbraced segment is:

$$M_{\max} = 4{,}754 \text{ kip-ft}$$

The absolute values of moments at the quarter point, M_A, at the centerline, M_B, at the three-quarter point, M_C, of the unbraced segment are calculated as follows:

$$M_A = (4{,}442) + \left(\frac{9 - 5.625}{9}\right)(4{,}754 - 4{,}442) = 4{,}559 \text{ kip-ft}$$

$$M_B = (4{,}010) + \left(\frac{1.125 + 5.625}{9}\right)(4{,}442 - 4{,}010) = 4{,}334 \text{ kip-ft}$$

$$M_C = (4{,}010) + \left(\frac{1.125}{9}\right)(4{,}442 - 4{,}010) = 4{,}064 \text{ kip-ft}$$

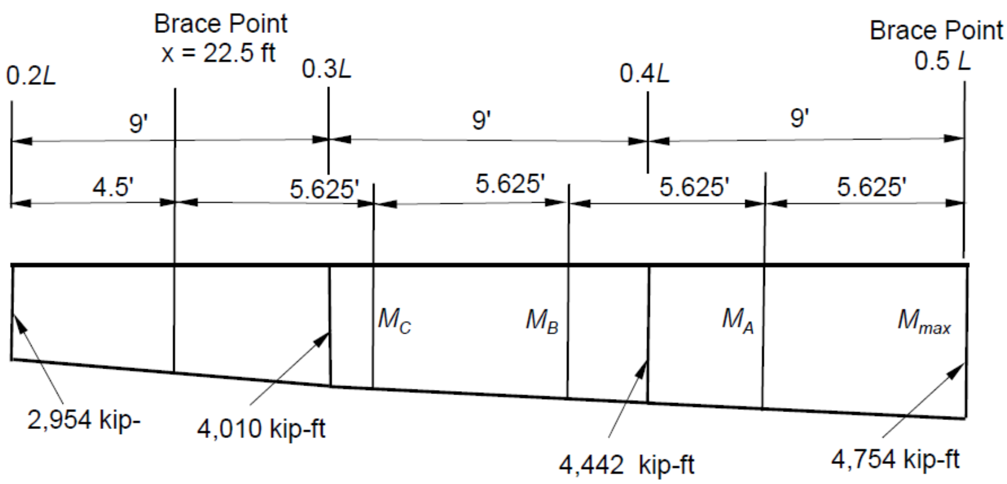


Figure 16.1.5-4 Factored Moment Envelope at Midspan

$$\begin{aligned} C_b &= \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \\ &= \frac{12.5(4{,}754)}{2.5(4{,}754) + 3(4{,}559) + 4(4{,}334) + 3(4{,}064)} \qquad \text{(CA 6.10.8.2.3-7)} \\ &= 1.079 \end{aligned}$$

$$M_{nc(LTB)} = C_b \left[1 - \left(1 - \frac{F_{yr} S_{xc}}{R_{pc} M_{yc}} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] R_{pc} M_{yc}$$

$$= (1.079) \left[1 - \left(1 - \frac{(23.1)(1{,}480)}{(1.075)(4{,}070)(12)} \right) \left(\frac{270 - 113.83}{531.7 - 113.83} \right) \right] (1.075)(4{,}070)$$

$$= 4{,}105 \text{ kip-ft} < R_{pc} M_{yc} = (1.075)(4{,}070) = 4{,}375 \text{ kip-ft}$$

(AASHTO A6.3.3-2)

Determine Flexural Resistance

$$M_n = \min\left(M_{nc(FLB)}, M_{nt}, M_{nc(LTB)} \right) = \min\left(4{,}375,\ 4{,}375,\ 4{,}105\right) = 4{,}105 \text{ kip-ft}$$

$$\phi_f M_n = (1.0)(4{,}105) = 4{,}105 \text{ kip-ft} < M_u = 4{,}754 \text{ kip-ft} \qquad \text{NG}$$

Flexural strengthening is needed at midspan.

16.1.5.5 Design Composite Section

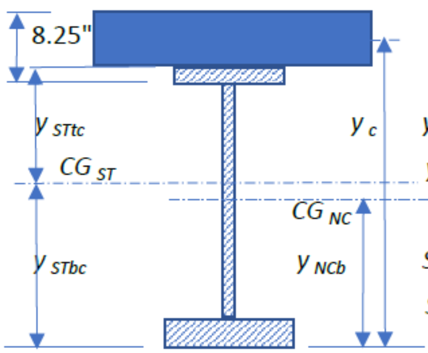
Try to strengthen the steel girder by making a composite section.

16.1.5.5.1 Calculate Section Properties for Midspan Section at 0.5*L*

The short-term and long-term composite section properties for the midspan section are calculated in Tables 16.1.5-4 and 16.1.5-5.

Table 16.1.5-4 Section Properties for Strengthened Short-term Composite Section Midspan (n = 8)

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	$y_i - y_{STbc}$ (in.)	$A_i(y_i - y_{STbc})^2$ (in.^{4})	I_o (in.^{4})
Steel Section	74.00	26.00	1924	-13.43	13,338	38,475
Transformed Conc Slab (81/n x 6.25")	63.28	55.13	3,488	15.70	15,597	205.99
Total (Σ)	137.28		5,412		28,935	38,681



y_i = Component CG to the bottom of bottom flange

I_o = Moment of inertia of component about its CG

$y_{STbc} = \Sigma A_i y_i / \Sigma A_i$ = 39.43 in.

y_{STtc} = (2+48+2)-39.43 = 12.57 in.

$I_{STc} = \Sigma I_o + \Sigma A_i (y_i - y_{STbc})^2$ = 67,616 in.^{4}

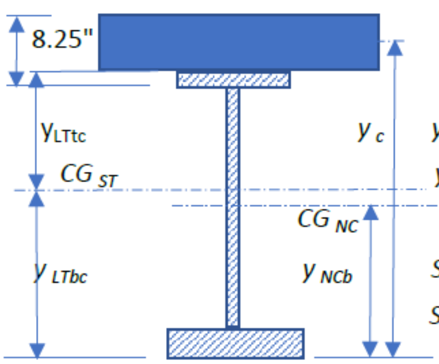
$S_{STbc} = I_{STc} / y_{STbc}$ = 1715 in.^{3}

$S_{STtc} = I_{Stc} / y_{STtc}$ = 5,377 in.^{3}

y_c = (2+48+8.25)-6.25/2 = 55.13 in.

Table 16.1.5-5 Section Properties for Strengthened Long-Term Composite Section Midspan (3n = 24)

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	$y_i - y_{LTbc}$ (in.)	$A_i(y_i - y_{LTbc})^2$ (in.^{4})	I_o (in.^{4})
Steel Section	74.00	26.00	1924	-6.46	3,089	38,475
Transformed Conc Slab (81/3n x 6.25")	21.09	55.13	1,163	22.66	10,835	68.66
Total (Σ)	95.09		3,087		13,924	38,543



y_i = Component CG to the bottom of bottom flange

I_o = Moment of inertia of component about its CG

$y_{LTbc} = \Sigma A_i y_i / \Sigma A_i$ = 32.46 in.

y_{LTtc} = (2+48+2)-32.4 = 19.54 in.

$I_{LTc} = \Sigma I_o + \Sigma A_i (y_i - y_{LTbc})^2$ = 52,467 in.^{4}

$S_{LTbc} = I_{LTc} / y_{LTbc}$ = 1616 in.^{3}

$S_{LTtc} = I_{LTc} / y_{LTtc}$ = 2,685 in.^{3}

16.1.5.5.2 Calculate Flexural Resistance, M_n, at Midspan 0.5L

Check Compactness of Section

For composite sections in the positive moment region, three requirements (Article 6.10.6.2.2) for a compact composite section in straight bridges are checked as follows:

Specified minimum yield strength of flanges:

$$F_{yf} = 33 \text{ ksi} < 70 \text{ ksi} \qquad \text{OK} \qquad \text{(AASHTO 6.10.6.2.2)}$$

Web:

$$\frac{D}{t_w} = \frac{48}{0.375} = 128 < 150 \qquad \text{OK} \qquad \text{(AASHTO 6.10.2.1.1-1)}$$

Section:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}} \qquad \text{(AASHTO 6.10.6.2.2-1)}$$

where D_{cp} is the depth of the web in compression at the plastic moment state and is determined in the following.

The compressive force in the concrete slab:

$$P_s = 0.85 f'_{ca}\, b_{eff} t_s = 0.85(3.6)(81)(6.25) = 1{,}549 \text{ kip}$$

in which t_s is the thickness of the concrete slab

The yield force in the top compression flange:

$$P_c = A_{fc}F_{yc} = (14 \times 2.0)(33) = 924 \text{ kip}$$

The yield force in the web:

$$P_w = A_w F_{yw} = (48 \times 0.375)(33) = 594 \text{ kip}$$

The yield force in the bottom tension flange:

$$P_t = A_{ft}F_{yt} = (14 \times 2.0)(33) = 924 \text{ kip}$$

Per AASHTO D6.1, the forces in longitudinal reinforcement may be conservatively neglected. Thus,

$$P_{rt} = P_{rb} = 0$$

$$\because\ P_t + P_w + P_c = 924 + 594 + 924 = 2{,}442 \text{ kip} > P_s = 1{,}549 \text{ kip}$$

$\therefore$ the PNA is within the top compression flange (AASHTO Table D6.1-1, Case II) and D_{cp} is equal to zero.

$$\frac{2D_{cp}}{t_w} = 0.0 < 3.76\sqrt{\frac{E}{F_{yc}}} \quad \text{OK} \quad \text{(AASHTO 6.10.6.2.2-1)}$$

The existing section meets the requirements for the composite compact section in positive flexure. The nominal flexural resistance, M_n, is, therefore, calculated in accordance with Article 6.10.7.1.2 (AASHTO, 2017; Caltrans, 2019).

Calculate Plastic Moment M_p

Determine Location of the PNA

As calculated above, the PNA is within the top flange of the steel girder. Denote that $\bar{y}$ is the distance from the top of the compression flange to the PNA, as shown in Figure 16.1.5-5, we obtain:

$$\bar{y} = \frac{t_{fc}}{2}\left(\frac{P_w + P_t - P_s}{P_c} + 1\right)$$

$$\bar{y} = \frac{2.0}{2}\left(\frac{594 + 924 - 1549}{924} + 1\right) = 0.97 \text{ in.} < t_{fc} = 2.0 \text{ in.} \quad \text{OK}$$

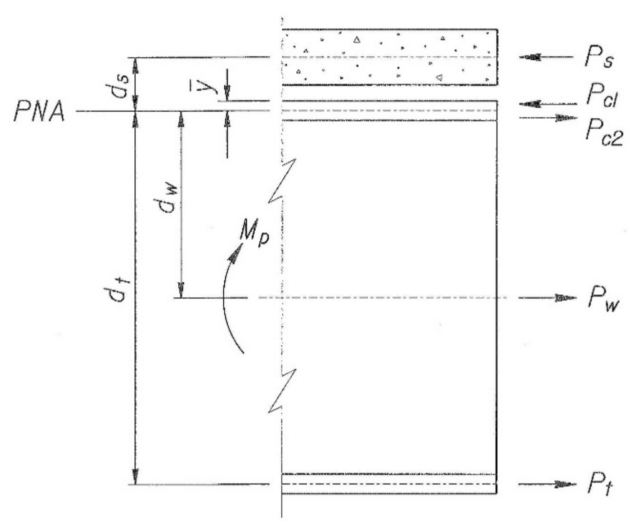


Figure 16.1.5-5 Plastic Moment State at Midspan Section

Calculate Plastic Moment M_p

Summing all forces about the PNA, obtain:

$$M_p = \sum M_{PNA} = P_s d_s + P_{c1}\left(\frac{\bar{y}}{2}\right) + P_{c2}\left(\frac{t_{cf} - \bar{y}}{2}\right) + P_w d_w + P_t d_t$$

$$= P_s d_s + b_{fc} F_{yc}\left(\frac{\left(\bar{y}\right)^2 + \left(t_{cf} - \bar{y}\right)^2}{2}\right) + P_w d_w + P_t d_t$$

where

$$d_s = \left(8.25 - \frac{6.25}{2}\right) - 2.0 + 0.97 = 4.095 \text{ in.}$$

$$d_w = \frac{48}{2} + 2.0 - 0.97 = 25.03 \text{ in.}$$

$$d_t = \frac{2.0}{2} + 48 + 2.0 - 0.97 = 50.03 \text{ in.}$$

$$M_p = (1{,}549)(4.095) + (14)(33)\left(\frac{0.97^2 + (2.0 - 0.97)^2}{2}\right)$$

$$+ (594)(25.03) + (924)(50.03)$$

$$= 67{,}901 \text{ k-in.} = 5{,}658 \text{ kip-ft}$$

Calculate Yield Moment M_y

The yield moment, M_y corresponds to the first yielding of either steel flange. It is obtained by the following formula (AASHTO D6.2):

$$M_{ys} = M_{D1} + M_{D2} + M_{AD} \qquad \text{(AASHTO D6.2.2-2)}$$

$$M_{AD} = S_{ST}\left(F_{yaf} - \frac{M_{D1}}{S_{NC}} - \frac{M_{D2}}{S_{LT}}\right)$$

From Table 16.1.5-2, factored moments, M_{D1} and M_{D2} are as follows:

$$M_{D1} = 1.25 M_{DC1} = 1{,}001 \text{ k-ft} = 12{,}013 \text{ kip-in.}$$

$$M_{D2} = 1.25 M_{DC2} + 1.5 M_{DW} = 177 + 298 = 475 \text{ kip-ft} = 5{,}700 \text{ kip-in.}$$

For the bottom flange, section moduli are obtained from Tables 16.1.5-3, 4, and 5 as:

$$S_{NC} = S_{NCb} = 1{,}480 \text{ in.}^3 \; ; S_{ST} = S_{STbc} = 1{,}715 \text{ in.} \; ; S_{LT} = S_{LTbc} = 1{,}616 \text{ in.}^3$$

$$M_{AD} = (1,715)\left(33 - \frac{12,013}{1,480} - \frac{5,700}{1,616}\right) \qquad \text{Control}$$

$$= 36,625 \text{ kip-in.} = 3,052 \text{ kip-ft}$$

For the top flange, section moduli are obtained from Tables 16.1.5-3, 4, and 5 as:

$S_{NC} = S_{NCt} = 1,480 \text{ in.}^3$; $S_{ST} = S_{STtc} = 5,377 \text{ in.}^3$; $S_{LT} = S_{LTtc} = 2,685 \text{ in.}^3$

$S_{ST} = S_{STtc} = 5,377 \text{ in.}^3$

$$M_{AD} = (5,377)\left(33 - \frac{12,013}{1,480} - \frac{5,700}{2,685}\right)$$

$$= 122,382 \text{ kip-in.} = 10,199 \text{ kip-ft}$$

It is obvious that the bottom flange controls. The yield moment of the strengthened girder midspan section is

$$M_{ys} = M_{D1} + M_{D2} + M_{AD}$$

$$= 1,001 + 475 + 3,052 = 4,528 \text{ k-ft}$$

Calculate Flexural Resistance, M_n

The nominal flexural resistance of the composite compact section in positive flexure is calculated in accordance with AASHTO and CA 6.10.7.1.2.

The compact and noncompact sections shall satisfy the following ductility requirement to ensure that the tension flange of the steel section reaches significant yielding before the crushing strain is reached at the top of the concrete deck.

$$D_p \leq 0.42 D_t \qquad \text{(AASHTO 6.10.7.3-1)}$$

$$D_p = 4.095 + 6.25/2 = 7.22 \text{ in.}$$

$$D_t = 8.25 + 48 + 2.0 = 58.25 \text{ in.}$$

$$D_p = 7.22 \text{ in.} < 0.42 D_t = 0.42(58.25) = 24.47 \text{ in.} \qquad \text{OK}$$

For this example, $M_y = M_{ys}$.

$$\because D_p = 7.22 \text{ in.} > 0.1 D_t = 5.83 \text{ in.}$$

$$M_n = \left[1 - \left(1 - \frac{M_{ys}}{M_p}\right)\left(\frac{D_p / D_t - 0.1}{0.32}\right)\right] M_p$$

$$= \left[1 - \left(1 - \frac{4{,}528}{5{,}658}\right)\left(\frac{7.22 / 58.25 - 0.1}{0.32}\right)\right](5{,}658) \qquad \text{(CA 6.10.7.1.2-2)}$$

$$= 5{,}573 \text{ kip-ft}$$

$$\phi_f M_n = (1.0)(5{,}573) = 5{,}573 \text{ kip-ft} > M_u = 4{,}754 \text{ kip-ft}$$

The strengthened composite section is sufficient.

16.1.5.5.3 Check for Stress Limitations at 0.5 Point - Service II Limit State

According to Article 6.10.4.2.2, at the Service II limit state, for this example, stresses in flanges shall satisfy the following requirement:

$$f_f \le 0.95 R_h F_{yf} = (0.95)(1.0)(33) = 31.35 \text{ ksi} \qquad \text{(AASHTO 6.10.4.2.2-1)}$$

In this example, since all permanent loads are resisted by the existing steel girder and live loads are resisted by the strengthened composite section, the flange stress at the service II limit state is obtained as:

$$f_f = \frac{M_{SD1} + M_{SD2}}{S_{NC}} + \frac{M_{SL}}{S_{STs}} \qquad \text{(16.1.5.5.3-1)}$$

From Table 16.1.5-1, factored moments, M_{D1} and M_{D2} are calculated as follows:

Service II: $1.0DC+1.0DW+1.3(DF)(LL+IM)_{HL\text{-}93}$

$$M_{D1} = M_{DC1} = 801 \text{ k-ft} = 9{,}612 \text{ kip-in}$$

$$M_{D2} = M_{DC2} + M_{DW} = 142 + 198 = 340 \text{ kip-ft} = 4{,}080 \text{ kip-in.}$$

From Table 16.1.3-9:

$$M_{SL} = 1{,}903 \text{ kip-ft} = 22{,}836 \text{ kip-in.}$$

For the top compression flange, section moduli are obtained from Tables 16.1.5-3 and 4 as:

$$S_{NC} = S_{NCt} = 1{,}480 \text{ in.}^3 \text{ ; } S_{ST} = S_{STtc} = 5{,}377 \text{ in.}^3$$

$$f_f = \frac{9{,}612 + 4{,}080}{1{,}480} + \frac{22{,}836}{5{,}377}$$

$$= 13.50 \text{ ksi} < 31.35 \text{ ksi} \qquad \text{OK}$$

For the bottom tension flange, section moduli are obtained from Tables 16.1.5-3 and 4 as:

$$S_{NC} = S_{NCb} = 1{,}480 \text{ in.}^3 \; ; \; S_{ST} = S_{STbc} = 1{,}715 \text{ in.}^3$$

$$f_f = \frac{9{,}612 + 4{,}080}{1{,}480} + \frac{22{,}836}{1{,}715}$$

$$= 22.57 \text{ ksi} < 31.35 \text{ ksi} \qquad \text{OK}$$

16.1.5.6 Design Shear Connectors

In this example, new welded shear connectors are added to the top steel flange to increase flexural resistance by the composite action in accordance with STP 16.6.6.2.1. Shear connectors are designed for fatigue and checked for strength.

16.1.5.6.1 Design for Fatigue

The range of horizontal shear flow, V_{sr}, is as follows:

$$V_{sr} = \frac{V_f Q}{I_{STc}} \qquad (16.1.5.6.1\text{-}1)$$

where V_f is the factored fatigue vertical shear force range as calculated for Example 1 shown in Tables 16.1.3-7 and 16.1.3-8, I_{STc} is the moment of inertia of the transformed short-term composite section, and Q is the first moment of transformed short-term area of the concrete deck about the neutral axis of the short-term composite section.

From Table 16.1.5-4, I_{STc} = 67,616 in.^{4} ; A_c / n = 63.28 in.^{2}

$$Q = (A_c / n)(y_c - y_{STb}) = (63.28)(55.125 - 39.43) = 993.18 \text{ in.}^3$$

$$V_{sr} = \frac{V_f Q}{I_{STc}} = \frac{993.18 V_f}{67{,}616} = 0.0147 V_f$$

Try d = 7/8 inch diameter stud, the fatigue shear resistance of an individual stud shear connector, Z_r is calculated per Article 6.10.10.2 as follows:

Fatigue I: Assume the additional service life required for the project is 50 years.

$ADTT_{SL} = p(ADTT)$ = (0.85)(2,500) = 2,125 > 960(75/50) =1,440

$$Z_r = 5.5d^2 = 5.5(0.875)^2 = 4.21 \text{ kip} \qquad \text{(AASHTO 6.10.10.2-1)}$$

Fatigue II: $(ADTT)_{SL} = p(ADTT)$ = (0.85)(20) = 17, number of fatigue cycles, N, is:

N = (365)(50)(1.0)(17) = 310,250

$$\alpha = 34.5 - 4.28\log N = 34.5 - 4.28\log(310{,}250) = 11.0 \qquad \text{(AASHTO 6.10.10.2-3)}$$

$$Z_r = \alpha d^2 = 11.0(0.875)^2 = 8.42 \text{ kip} \qquad \text{(AASHTO 6.10.10.2-2)}$$

Try 3–d = 7/8 inch diameter studs with F_u = 60 ksi (Article 6.4.4) for the midspan from 0.4L to 0.6L. The required pitch of shear connectors, p, is obtained as:

$$p_{reqd} = \frac{nZ_r}{V_{sr}} = \frac{3Z_r}{V_{sr}} \qquad \text{(AASHTO 6.10.10.1.2-1)}$$

Try 6–d = 7/8 inch diameter studs group with F_u = 60 ksi (Article 6.4.4) for end spans from 0.0 to 0.4L, and from 0.6L to 1.0L, the required pitch of shear connectors, p is obtained as:

$$p_{reqd} = \frac{nZ_r}{V_{sr}} = \frac{6Z_r}{V_{sr}} \qquad \text{(AASHTO 6.10.10.1.2-1)}$$

The detailed calculation is shown in Table 16.1.5-6.

Table 16.1.5-6 Required Pitch of Shear Connectors

	Fatigue I - HL-93			Fatigue II - P-9		
	V_f	V_{sr} =0.0147V_f	p	V_f	V_{sr} =0.0147V_f	p
x/L	(kip)	(kip/in.)	(in.)	(kip)	(kip/in.)	(in.)
0.0	59.2	0.87	14.5	91.5	1.34	18.8
0.1	53.1	0.78	16.2	79.8	1.17	21.5
0.2	45.5	0.67	18.9	65.2	0.96	26.4
0.3	37.9	0.56	22.7	52.2	0.77	33.0
0.4	30.2	0.44	28.4	39.1	0.57	43.9
0.5	22.6	0.33	38.0	29.3	0.43	58.6
0.6	30.2	0.44	28.4	39.1	0.57	43.9
0.7	37.9	0.56	22.7	52.2	0.77	33.0
0.8	45.5	0.67	18.9	65.2	0.96	26.4
0.9	53.1	0.78	16.2	79.8	1.17	21.5
1.0	59.1	0.87	14.5	91.5	1.34	18.8

Select shear stud pitch of 24 in. as shown in Figure 16.1.5-6. A total number of shear studs n =(6)(18)(2) +(3)(9+1) = 246 are provided.

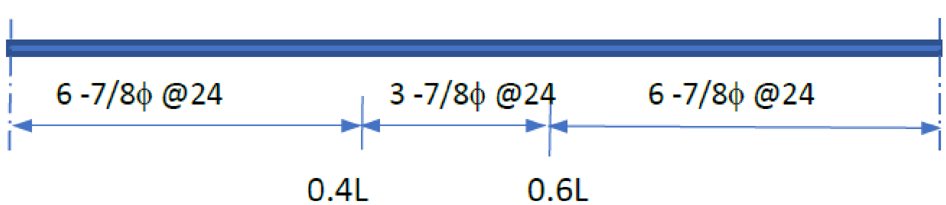


Figure 16.1.5-6 Pitch of Stud Shear Connectors

16.1.5.6.2 Check for Strength

In this example of a straight bridge, the minimum number of shear connectors between the point of the maximum positive moment and each adjacent point of zero moment shall satisfy the following requirement:

$$n = \frac{P}{Q_r} = \frac{P}{\phi_{sc} Q_n} \qquad \text{(AASHTO 6.10.10.4.1-2)}$$

$$P = \text{smaller} \begin{Bmatrix} 0.85 f'_{ca} b_{eff}\, t_s = (0.85)(3.6)(81)(6.25) = 1{,}549 \\ A_s F_y = (74)(33) = 2{,}442 \end{Bmatrix} = 1{,}549 \text{ kip}$$

(AASHTO 6.10.10.4.2-2; 6.10.10.4.2-3)

The factored shear resistance of a single *d* = 7/8 in. stud shear connector is as:

$$E_{ca} = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_{ca}} = (33{,}000)(1.0)(1.5)^{1.5}\sqrt{3.6} = 3{,}637 \text{ ksi}$$

(AASHTO C5.4.2.4-2)

$$Q_n = 0.5 A_{sc} \sqrt{f'_{ca} E_c} = 0.5 \frac{(0.875)^2 \pi}{4} \sqrt{3.6(3{,}637)}$$

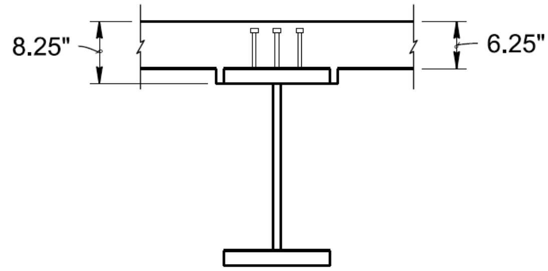
$$= 34.4 \text{ kip} < A_{sc} F_u = \frac{\pi}{4}(0.875)^2 (60) = 36.1 \text{ kip}$$

(AASHTO 6.10.10.4.3-1)

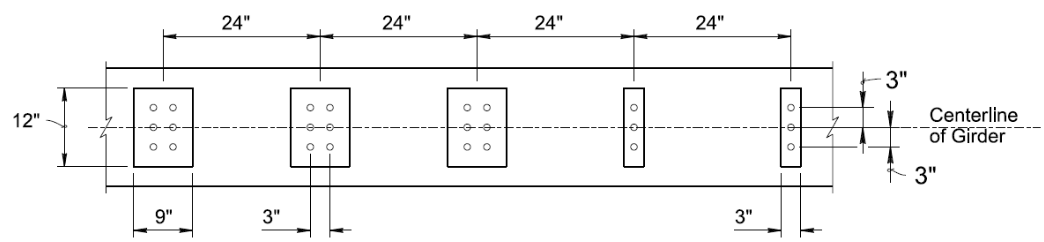
Use Q_n = 34.4 kip, the total number of stud shear connectors between the point of the maximum positive moment and zero moment provided is

$$n_{prod} = \frac{246}{2} = 123 > \frac{P}{\phi_{sc} Q_n} = \frac{1{,}549}{0.85\,(34.4)} = 53 \qquad \text{OK}$$

The designed 6-d =7/8 group and 3-d =7/8 studs are sufficient as shown in Figure 16.1.5-7.



(a) Section



(b) Partial Plan

Figure 16.1.5-7 New Stud Shear Connectors

16.1.6 SHEAR STRENGTHENING DESIGN EXAMPLE

The following is an example of the shear strengthening of a simple span composite steel girder bridge by adding web plates and transverse stiffeners.

16.1.6.1 Existing Steel Girder Bridge Data

Bridge Type:	Simple Span, multi steel girder bridge	
Span Length:	70 ft between the center line of bearings	
Bridge Width:	42'-2"	
Year Built:	1966	
Girder:	Composite steel girder	
Live Load:	H20-44	
Reinforced Concrete:	f_s =	20,000 psi
	f_c =	1,200 psi
Structural Steel:	f_s =	20,000 psi
	F_y =	36,000 psi

Typical section and girder data are shown in Figures 16.1.6-1 and 16.1.6-2.

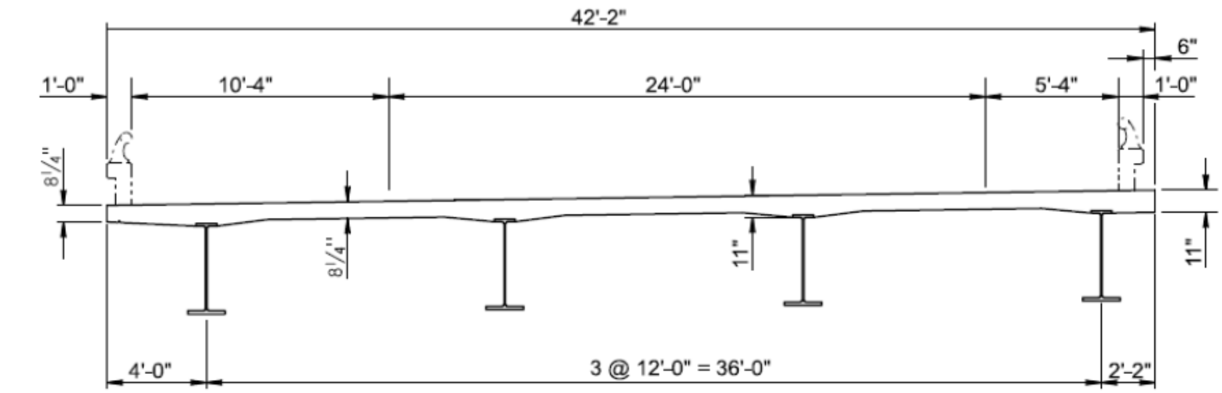


Figure 16.1.6-1 Typical Section

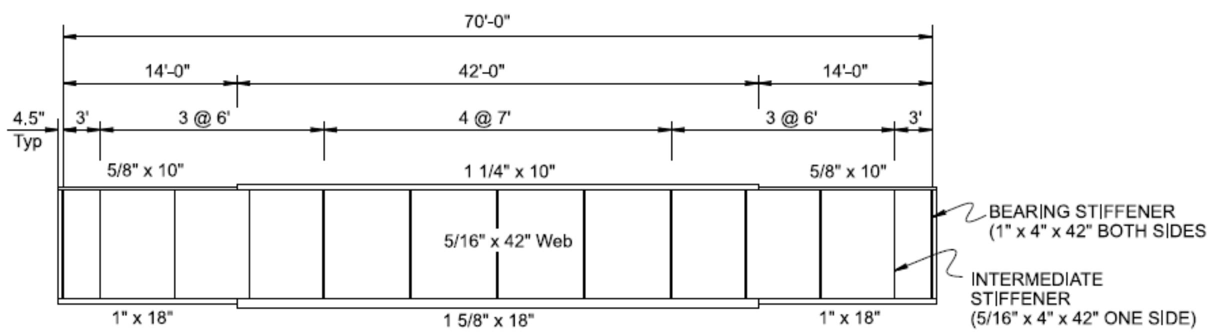


Figure 16.1.6-2 Interior Girder Span

16.1.6.2 Design Requirement

Perform the following strengthening design portions for an interior plate girder in accordance with STP 16.6 (Caltrans 2021) and AASHTO-CA BDS-8 (AASHTO, 2017; Caltrans, 2019). A similar procedure can be used for strengthening exterior girders and is not illustrated here.

Step 1: Determine Material Properties

Step 2: Perform Load and Structural Analysis

Step 3: Calculate Live Load Distribution Factors

Step 4: Determine Load and Resistance Factors and Load Combinations

Step 5: Calculate Factored Shears - Strength Limit States

Step 6: Calculate Factored Shears - Fatigue Limit States

Step 7: Calculate Factored Shears - Service Limit State II

Step 8: Check Shear Resistance for Existing Steel Web Panels

Step 9: Design for Shear Strengthening by Adding Web Plates at End Panel

Step 10: Design for Shear Strengthening by Adding Web Plates at First Interior Panel

Step 11: Design for Shear Strengthening by Adding Transverse Stiffeners at Other Interior Panels

The following notations are used in this example:

"AASHTO xxx-x" denotes "AASHTO Equation xxx-x"

"CA xxx" denotes "California Amendment Article xxx"

"CA xxx-x" denotes "California Amendment Equation xxx-x"

"STP xxx" denotes "Caltrans Structure Technical Policy Article xxx"

16.1.6.3 Determine Material Properties

Per STP 16.6.5, actual material properties for existing structures, F_{ya}, F_{ua}, and f'_{ca} should be obtained from physical tests if feasible. In the absence of test results for this example, they are determined as follows:

As-built concrete compressive strength: $f'_c = 2.5f_c = (2.5)(1{,}200) = 3{,}000 \text{ psi} = 3.0 \text{ ksi}$

Actual concrete compressive strength: $f'_{ca} = 1.2f'_c = (1.2)(3.0) = 3.6 \text{ ksi}$

Unit weight of concrete: $w_c = 0.15 \text{ kcf}$

Modulus of elasticity of concrete:

$$E_{ca} = 33{,}000K_1w_c^{1.5}\sqrt{f'_{ca}} = (33{,}000)(1.0)(1.5)^{1.5}\sqrt{3.6} = 3{,}637 \text{ ksi}$$

(AASHTO C5.4.2.4-2)

Actual yield strength of existing steel: $F_{ya} = 36$ ksi

Actual tensile strength of existing steel: $F_{ua} = 60$ ksi

Modulus of elasticity of steel: $E_s = 29{,}000$ ksi

Modular Ratio $n = \dfrac{E_s}{E_{ca}} = \dfrac{29{,}000}{3{,}637} = 7.97$, Use $n = 8$.

Use ASTM A709 Grade 36, the material properties are as follows:

Specified minimum yield strength of steel: $F_y = 36$ ksi

Unit weight of steel: $w_s = 0.49$ kcf

16.1.6.4 Perform Load and Structural Analysis

16.1.6.4.1 Calculate Permanent Loads for an Interior Girder

The permanent load or dead load of an interior girder includes *DC* and *DW*. *DC* is the dead load of structural components and nonstructural attachments. *DW* is the dead load of wearing surfaces.

DC1 - Structural dead load, acting on the noncomposite section

- Concrete Slab

  Concrete slab thickness: t_s = 8.25 in.

  Girder spacing: S = 12.00 ft

  Weight of deck slab: $W_s = t_s S w_c = (8.25/12)(12)(150) = 1{,}238$ lb/ft

- Girder Self Weight

  Section at ends:

  Top Flange PL 5/8X10

  Top flange width: b_{fc} = 10 in.

  Top flange thickness: t_{fc} = 0.625 in.

  Bottom Flange PL 1X18

  Bottom flange width: b_{ft} = 18 in.

  Bottom flange thickness: t_{ft} = 1.0 in.

  Web PL 5/16X42

  Web thickness: t_w = 0.3125 in.

  Web depth: D = 42 in.

  Gross section area:

  $$A_{ge} = (10 \times 0.625) + (42 \times 0.3125) + (18 \times 1.0) = 37.375 \text{ in.}^2$$

  Weight of steel girder: $W_{ge} = A_{ge} w_s = (37.375)(490/144) = 127$ lb/ft

  Section at midspan:

  Top flange PL 1 1/4X10

  Top flange width: b_{fc} = 10 in.

  Top flange thickness: t_{fc} = 1.25 in.

  Bottom flange PL 1 5/8X18

  Bottom flange width: b_{ft} = 18 in.

  Bottom flange thickness: t_{ft} = 1.625 in.

  Web PL 5/16X42

Web thickness: t_w = 0.3125 in.

Web depth: D = 42 in.

Gross section area:

$$A_{gm} = \left(10 \times 1.25\right) + \left(42 \times 0.3125\right) + \left(18 \times 1.625\right) = 54.875 \text{ in.}^2$$

Weight of steel girder: $W_{gm} = A_{gm} w_s = \left(54.875\right)\left(490 / 144\right) = 187$ lb/ft

- Stiffener Weight

Bearing stiffener: 1"x 42" stiffeners at the end on both sides, total 4 stiffeners for one girder

Stiffener width: b_t = 4 in.

Stiffener thickness: t_t = 1.0 in.

Stiffener volume: $V_{st} = \left(4\right)\left(4\right)\left(1.0\right)\left(42\right) = 672 \text{ in.}^3$

Intermediate Transverse Stiffener: 5/16"x42" " at one side. Total 11 stiffeners.

Stiffener width: b_t = 4 in.

Stiffener thickness: t_t = 0.3125 in.

Stiffener volume: $V_{st} = \left(11\right)\left(4\right)\left(0.3125\right)\left(42\right) = 578 \text{ in.}^3$

Total stiffener weight:

$$W_{st} = \frac{\left(672 + 578\right)\left(490\right)}{12^3\left(70\right)} = 5 \text{ lb/ft}$$

- Bracing Weight

Assume W_{br} = 30 lb/ft

- Miscellaneous Dead Load for Haunch, Welds, etc.

Assume W_{misc} = 85 lb/ft

Total DC1

End Span

$$DC1_{end} = W_s + W_{ge} + W_{st} + W_{br} + W_{misc}$$
$$= 1{,}238 + 127 + 5 + 30 + 85 = 1{,}485 \text{ lb/ft} = 1.485 \text{ k/ft}$$

Midspan

$$DC1_{mid} = W_s + W_{gm} + W_{st} + W_{br} + W_{misc}$$
$$= 1{,}238 + 187 + 5 + 30 + 85 = 1{,}545 \text{ lb/ft} = 1.545 \text{ k/ft}$$

DC1 is shown in Figure 16.1.6-3.

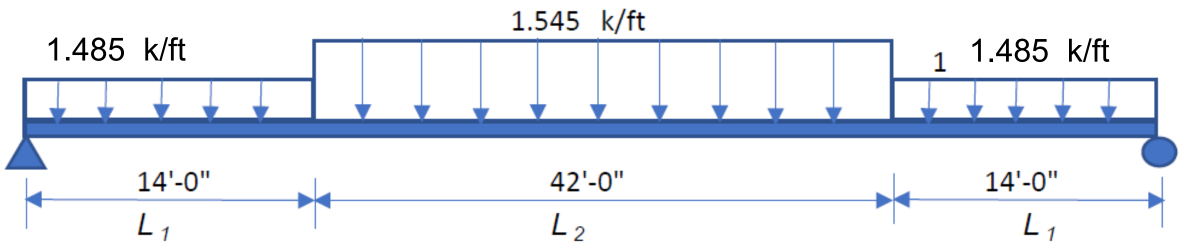


Figure 16.1.6-3 DC1 Dead Load

DC2 - Nonstructural dead load, acting on the long-term composite section

Assume one side barrier: $W_{barrier}$ = 300 lb/ft

Assume one side railing: $W_{railing}$ = 120 lb/ft

$$DC2_{total} = 2\left(W_{barrier} + W_{ralling}\right) = (2)(300 + 120) = 840 \text{ lb/ft} = 0.84 \text{ k/ft}$$

Assume *DC2* is distributed equally to all girders and *DC2* for an interior girder as:

$$DC2 = DC2_{total} / 4 = 0.84 / 4 = 0.210 \text{ k/ft}$$

DC2 is shown in Figure 16.1.6-4.

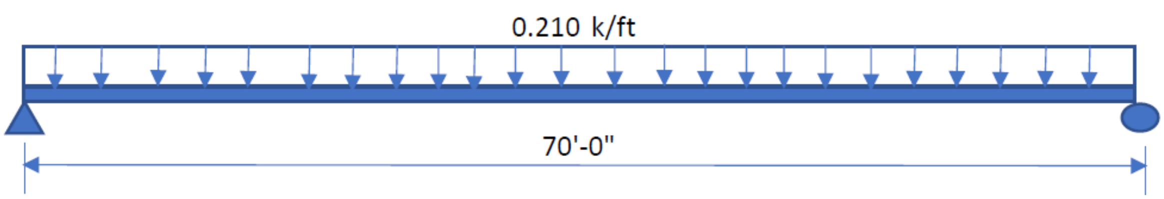


Figure 16.1.6-4 DC2 Dead Load

DW - Considering future wearing surface 35 psf (Ignore the weight of existing AC overlay)

Deck width from curb to curb = 39.67 ft

$$DW_{total} = (35)(39.67) = 1{,}389 \text{ lb/ft} = 1.389 \text{ k/ft}$$

Assume *DW* is distributed equally to all girders, and *DW* for an interior girder is as:

$$DW = 1.389 / 4 = 0.347 \text{ k/ft}$$

DW is shown in Figure 16.1.6-5.

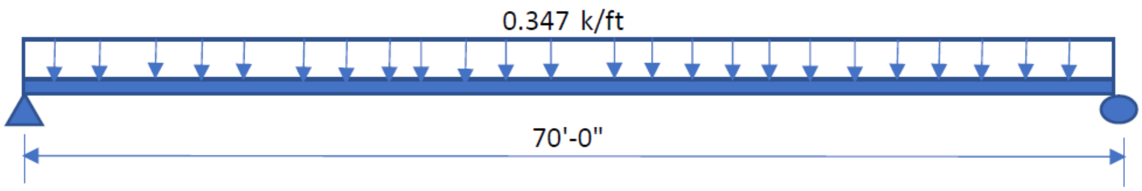


Figure 16.1.6-5 DW Dead Load

16.1.6.4.2 Live Load and Dynamic Load Allowance

For live load upgrade, HL-93 (Article 3.6.1.2) and Caltrans P15 (CA Article 3.6.1.8) are considered for this example. To consider the wheel load impact from moving vehicles, the dynamic load allowances are as follows:

IM = 33% for the strength I limit state (CA Table 3.6.2.1-1)

IM = 25% for the strength II limit state

IM = 15% for the fatigue limit states

16.1.6.4.3 Perform Structural Analysis

Unfactored dead load shears for an interior girder are calculated and shown in Table 16.1.6-1.

Table 16.1.6-1 Unfactored Dead Load Shears for an Interior Girder

Point x/L	Shear		
	DC1	*DC2*	*DW*
	V_{DC1} (kip)	V_{DC2} (kip)	V_{DW} (kip)
0.0	53.2	7.4	12.1
0.1	42.8	5.9	9.7
0.2	32.4	4.4	7.3
0.3	21.6	2.9	4.9
0.4	10.8	1.5	2.4
0.5	0	0.0	0.0
0.6	-10.8	-1.5	-2.4
0.7	-21.6	-2.9	-4.9
0.8	-32.4	-4.4	-7.3
0.9	-42.8	-5.9	-9.7
1.0	-53.2	-7.4	-12.1

In this design example, live load analysis is performed by the CTBridge computer program. Unfactored live load shears for one lane with dynamic load allowance are shown in Table 16.1.6-2.

Table 16.1.6-2 Unfactored Live Load Shears for One Lane with Dynamic Load Allowance

Point x/L	Shear ($LL+IM$)		Fatigue Shear ($LL+IM$)	
	HL93	*P15*	*HL93*	*P9*
	V_{HL93} (kip)	V_{P15} (kip)	V_{FHL93} (kip)	V_{FP9} (kip)
0.0	102.4	159.1	59.2	130.1
0.1	91.9	138.9	53.0	113.6
0.2	77.8	111.9	44.7	91.5
0.3	63.8	86.8	34.8	71.0
0.4	45.5	59.8	26.6	48.9
0.5	-12.3	-33.8	-18.3	-27.6
0.6	-45.5	-59.8	-26.6	-48.9
0.7	-63.8	-86.8	-34.8	-71.0
0.8	-77.8	-111.9	-44.7	-91.5
0.9	-91.9	-138.9	-53.0	-113.6
1.0	-102.4	-159.1	-59.2	-130.1

16.1.6.5 Calculate Live Load Distribution Factors

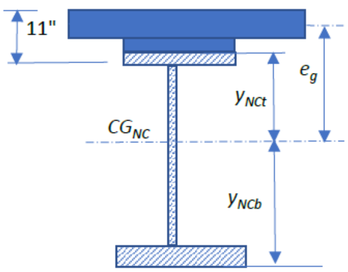
To calculate the live load distribution factors, we need to calculate the longitudinal stiffness parameter, K_g, as follows.

16.1.6.5.1 Existing Section Properties at Midspan Section

The longitudinal stiffness parameter, K_g is estimated per AASHTO Equation 4.6.2.2.1-1, as shown in Table 16.1.6-3.

Table 16.1.6-3 Existing Section Properties at Midspan Section

Component	A_i (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	$y_i - y_{NCb}$ (in.)	$A_i (y_i - y_{NCb})^2$ (in.^{4})	I_o (in.^{4})
Top Flange (1 1/4x10)	12.50	44.25	553.13	28.33	10,029	1.63
Web (5/16x42)	13.125	22.63	296.95	6.70	589	1,929
Bottom Flange (1 5/8x18)	29.25	0.81	23.77	-15.11	6679.68	6.44
Total (Σ)	54.88		873.84		17,298	1,937



y_i = Component CG to the bottom of bottom flange

I_o = Moment of inertia of component about its CG

$y_{NCb} = \Sigma A_i y_i / \Sigma A_i$ = 15.92 in.

y_{NCt} = (1.25+42+1.625)-15.92 = 28.95 in.

$I_{NC} = \Sigma I_0 + \Sigma A_i (y_i - y_{NCb})^2$ = 19,236 in.^{4}

e_g = 28.95+11-1.25-8.25/2 = 34.58 in.

n = Modular Ratio = 8

$K_g = n(I_{NC} + Ae_g^2)$ = 678,703 in.^{4}

(AASHTO 4.6.2.2.1-1)

16.1.6.5.2 Calculate Live Load Distribution Factors

From AASHTO Table 4.6.2.2.1-1, the cross-section of this example is Type "*a*" structure, and the number of girders N_b = 4.

Strength Limit States - Live Load Shear Distribution Factors (AASHTO Table 4.6.2.2.3a-1)

One design lane loaded: $DF_v = 0.36 + \frac{S}{25} = 0.84$

Two or more design lanes loaded:

$$DF_v = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 = 1.08 \qquad \text{Control}$$

where:

L = span length for moment is being calculated = 70 ft

S = girder spacing = 12.00 ft

t_s = concrete slab thickness = 8.25 in.

K_g = stiffness parameter = 678,703 in.^{4}

Note: the above equations have multiple presence factors, *m*, included in them (Article C3.6.1.1.2).

Fatigue Limit States - Live Load Shear Distribution Factors

For the fatigue limit states, the live load is one *HL-93* or one *P9* truck as specified in CA 3.6.1.4.1; the multiple presence factor of 1.2 should be divided from the above one lane factors (Article 3.6.1.1.2).

Fatigue Limit States - Live Load Shear Distribution Factor

$$DF_v = \frac{0.84}{1.2} = 0.70$$

Live load distribution factors are summarized in Table 16.1.6-4.

Table 16.1.6-4 Summary of Live Load Distribution Factors

Limit States	DF_v
Strength Limit States	1.08
Service Limit States	1.08
Fatigue Limit States	0.7

16.1.6.6 Determine Load and Resistance Factors and Load Combinations

16.1.6.6.1 Design Equation

$$\Sigma \eta_i \gamma_i Q_i \leq \phi R_n = R_r \qquad \text{(AASHTO 1.3.2.1-1)}$$

where:

η_i = load modifier factor = 1.0
γ_i = load factor
Q_i = force effect
ϕ = resistance factor
R_n = nominal resistance
R_r = factored resistance

16.1.6.6.2 Determine Applicable Resistance Factors for Strength Limit State

According to Article 6.5.4.2, the following resistance factors are used for the strength limit states in this example.

For flexure ϕ_f = 1.00
For shear ϕ_v = 1.00
For ASTM F3125 bolts in shear ϕ_s = 0.80

For bolt bearing on material ϕ_{bb} = 0.80

For weld metal in fillet weld

– shear in throat of weld metal ϕ_{e2} = 0.80

16.1.6.6.3 Determine Applicable Load Factors and Load Combinations

According to CA Table 3.4.1-1, the following five load combination groups are considered for this example:

Strength I: $1.25DC+1.5DW+1.75(DF)(LL+IM)_{HL\text{-}93}$

Strength II: $1.25DC+1.5DW+1.35(DF)(LL+IM)_{P15}$

Service II: $1.0DC+1.0DW+1.3(DF)(LL+IM)_{HL\text{-}93}$

Fatigue I: $1.75(DF)(LL+IM)_{HL\text{-}93}$

Fatigue II: $1.0(DF)(LL+IM)_{\mathrm{P9}}$

16.1.6.7 Calculate Factored Shears – Strength Limit States

Using load combinations as discussed in Section 16.1.6.6.3, factored shears for strength limit states I and II are calculated and listed in Table 16.1.6-5.

16.1.6-5 Factored Shear Envelopes for Interior Girder

Point x/L	Dead Load			Live Load		Load Combination	
	DC1	*DC2*	*DW*	*HL93*	*P15*	Strength I	Strength II
	$1.25V_{DC1}$ (kip)	$1.25V_{DC2}$ (kip)	$1.5V_{DW}$ (kip)	$1.75DFV_{HL93}$ (kip)	$1.35DFV_{P15}$ (kip)	V_u (kip)	V_u (kip)
0.0	66.6	9.2	18.2	194.1	232.5	288.0	326.5
0.1	53.6	7.4	14.6	174.1	202.9	249.6	278.4
0.2	40.6	5.5	10.9	147.5	163.5	204.5	220.5
0.3	27.0	3.7	7.3	120.8	126.8	158.8	164.8
0.4	13.5	1.8	3.6	86.1	87.4	105.1	106.4
0.5	0.0	0.0	0.0	-23.2	-49.3	-23.2	-49.3
0.6	-13.5	-1.8	-3.6	-86.1	-87.4	-105.1	-106.4
0.7	-27.0	-3.7	-7.3	-120.8	-126.8	-158.8	-164.8
0.8	-40.6	-5.5	-10.9	-147.5	-163.5	-204.5	-220.5
0.9	-53.6	-7.4	-14.6	-174.1	-202.9	-249.6	-278.4
1.0	-66.6	-9.2	-18.2	-194.1	-232.5	-288.0	-326.5

16.1.6.8 Calculate Factored Shears - Fatigue Limit States

Using load combinations as discussed in Section 16.1.6.6.3, factored shears for fatigue limit states are calculated and listed in Table 16.1.6-6.

Table 16.1.6-6 Fatigue Limit State -Factored Shear by Live Load only

Point x/L	Live Load Shear $(DF)(LL+IM)$		Fatigue Shear	
	HL93	*P9*	Fatigue I	Fatigue II
	DFV_{FHL93} (kip)	DFV_{FP9} (kip)	$1.75DFV_{FHL93}$ (kip)	DFV_{FP9} (kip)
0.0	41.4	91.1	72.5	91.1
0.1	37.1	79.5	64.9	79.5
0.2	31.3	64.0	54.7	64.0
0.3	24.4	49.7	42.7	49.7
0.4	18.6	34.2	32.5	34.2
0.5	-12.8	-19.3	-22.4	-19.3
0.6	-18.6	-34.2	-32.5	-34.2
0.7	-24.4	-49.7	-42.7	-49.7
0.8	-31.3	-64.0	-54.7	-64.0
0.9	-37.1	-79.5	-64.9	-79.5
1.0	-41.4	-91.1	-72.5	-91.1

V_u, shear due to the unfactored dead load plus the factored fatigue load (Fatigue I) is also calculated in Table 16.1.6-7 for checking the special fatigue requirement for webs as required by Article 6.10.5.3.

$$V_u = V_{dc1} + V_{dc2} + V_{dw} + (1.75)(DF_v)(LL + IM)_{HL-93}$$

Table 16.1.6-7 Special Fatigue Shear Requirement for Web Check

Point x/L	Dead Load Shear			Fatigue I	Special
	DC1	*DC2*	*DW*	Shear	Shear
	V_{DC1} (kip)	V_{DC1} (kip)	V_{DW} (kip)	V_u (kip)	V_u (kip)
0.0	53.2	7.4	12.1	72.5	145.2
0.1	42.8	5.9	9.7	64.9	123.3
0.2	32.4	4.4	7.3	54.7	98.9
0.3	21.6	2.9	4.9	42.7	72.1
0.4	10.8	1.5	2.4	32.5	47.2
0.5	0.0	0.0	0.0	-22.4	-22.4
0.6	-10.8	-1.5	-2.4	-32.5	-47.2
0.7	-21.6	-2.9	-4.9	-42.7	-72.1
0.8	-32.4	-4.4	-7.3	-54.7	-98.9
0.9	-42.8	-5.9	-9.7	-64.9	-123.3
1.0	-53.2	-7.4	-12.1	-72.5	-145.2

16.1.6.9 Calculate Factored Shears - Service Limit State II

Factored shears at service limit state II are calculated in Figure 16.1.6-8.

Table 16.1.6-8 Factored Shear Envelopes for Interior Girder

Point x/L	Dead Load			Live Load	Load Combination
	DC1	*DC2*	*DW*	*HL93*	Service II
	V_{DC1} (kip)	V_{DC1} (kip)	V_{DW} (kip)	$1.3\,DF\,V_{HL93}$ (kip)	V_u (kip)
0.0	53.2	7.4	12.1	144.2	216.9
0.1	42.8	5.9	9.7	129.3	187.8
0.2	32.4	4.4	7.3	109.5	153.7
0.3	21.6	2.9	4.9	89.8	119.2
0.4	10.8	1.5	2.4	64.0	78.7
0.5	0.0	0.0	0.0	-17.3	-17.3
0.6	-10.8	-1.5	-2.4	-64.0	-78.7
0.7	-21.6	-2.9	-4.9	-89.8	-119.2
0.8	-32.4	-4.4	-7.3	-109.5	-153.7
0.9	-42.8	-5.9	-9.7	-129.3	-187.8
1.0	-53.2	-7.4	-12.1	-144.2	-216.9

16.1.6.10 Check Shear Resistances of Existing Steel Web Panels

16.1.6.10.1 End Panel

From Tables 16.1.6-5 and 16.1.6-8, the maximum shears at the end panel are as follows:

Strength limit state: $V_u = 326.5$ kip

Service II limit state: $V_u = 216.9$ kip

According to STP 16.6.7.1, the nominal resistance of a stiffened web end panel is calculated as:

$$V_n = V_p\left[C + \alpha\frac{0.87(1-C)}{\sqrt{1+\left(d_o/D\right)^2}}\right] \quad \text{(STP 16.6.7.1.1)}$$

$$\alpha = \frac{2.8\left(\sqrt{M_{pf}+M_{pm}}+\sqrt{M_{pst}+M_{pm}}\right)}{D\sqrt{t_w F_{yw}(1-C)}} \le 1.0 \quad \text{(STP 16.6.7.1.2)}$$

where

V_p = plastic shear force (kip) as specified in Article 6.10.9.3

$$V_p = 0.58 F_{yw} D t_w \quad \text{(AASHTO 6.10.9.3.2-3)}$$

D = depth of web (in.) = 42 in.

t_w = thickness of web (in.) = 0.3125 in.

F_{yw} = specified minimum (existing) yield strength of the web (ksi) = 36 ksi

$$V_p = 0.58F_{yw}Dt_w = (0.58)(36)(42)(0.3125) = 274.1\text{ kip} \quad \text{(AASHTO 6.10.9.3.2-3)}$$

d_o = transverse stiffener spacing (in.) = 36 in.

C = ratio of the shear-buckling resistance to the shear yield strength determined by Article 6.10.9.3

$$k = 5 + \frac{5}{(d_o / D)} = 5 + \frac{5}{(36/42)^2} = 11.81 \qquad \text{(AASHTO 6.10.9.3.2-7)}$$

$$1.12\sqrt{\frac{E_s k}{F_{yw}}} = 1.12\sqrt{\frac{(29{,}000)(11.81)}{36}} = 109.2$$

$$1.4\sqrt{\frac{E_s k}{F_{yw}}} = 1.4\sqrt{\frac{(29{,}000)(11.81)}{36}} = 136.5$$

$$\because 1.12\sqrt{\frac{E_s k}{F_{yw}}} = 109.2 < \frac{D}{t_w} = \frac{42}{0.3125} = 134.4 < 1.4\sqrt{\frac{E_s k}{F_{yw}}} = 136.5$$

$$\therefore\ C = \frac{1.12}{\frac{D}{t_w}}\sqrt{\frac{E_s k}{F_{yw}}} = \frac{1.12}{\frac{42}{0.3125}}\sqrt{\frac{29{,}000(11.81)}{36}} = 0.813 \qquad \text{(AASHTO 6.10.9.3.2-5)}$$

α = parameter to consider partial tension-field action

M_{pf} = plastic moment of the section composed of the top flange and the segment of the web with the depth, d_e, about the plastic neutral axis as shown in STP Figure 16.6.7.1.1 (kip-in.)

M_{pm} = smaller of M_{pst} and M_{pf} (kip-in.)

M_{pst} = plastic moment of the section composed of the bearing stiffener and the segment of the web with depth, d_e, plus the distance from the inside face of the stiffener to the end of the girder, e, about the plastic neutral axis as shown in STP Figure 16.6.7.1.1 (kip-in.)

e = distance from the inside face of the bearing stiffener to the end of the girder, for calculation purposes, shall not exceed $0.84t_w\sqrt{\frac{E_s}{F_{yw}}}$ (in.)

$$0.84 t_w \sqrt{\frac{E_s}{F_{yw}}} = (0.84)(0.3125)\sqrt{\frac{(29{,}000)}{36}} = 7.45 \text{ in.}$$

Since C > 0.8,

d_e = effective web depth = 0.0 in. (STP 16.6.7.1.4)

If C ≤ 0.8

$$d_e = 35 t_w (0.8 - C)^2 \quad \text{(STP 16.6.7.1.3)}$$

Calculation of M_{pst}, M_{pf} and M_{pm}

For the existing bridge, the bearing stiffener is PL 1" x 4" x 42, and the first intermediate transverse stiffener is at a spacing of 3'-0" from the bearing support. The effective web area and plastic neutral axes for M_{pf} and M_{pst} are shown in Figure 16.1.6-6.

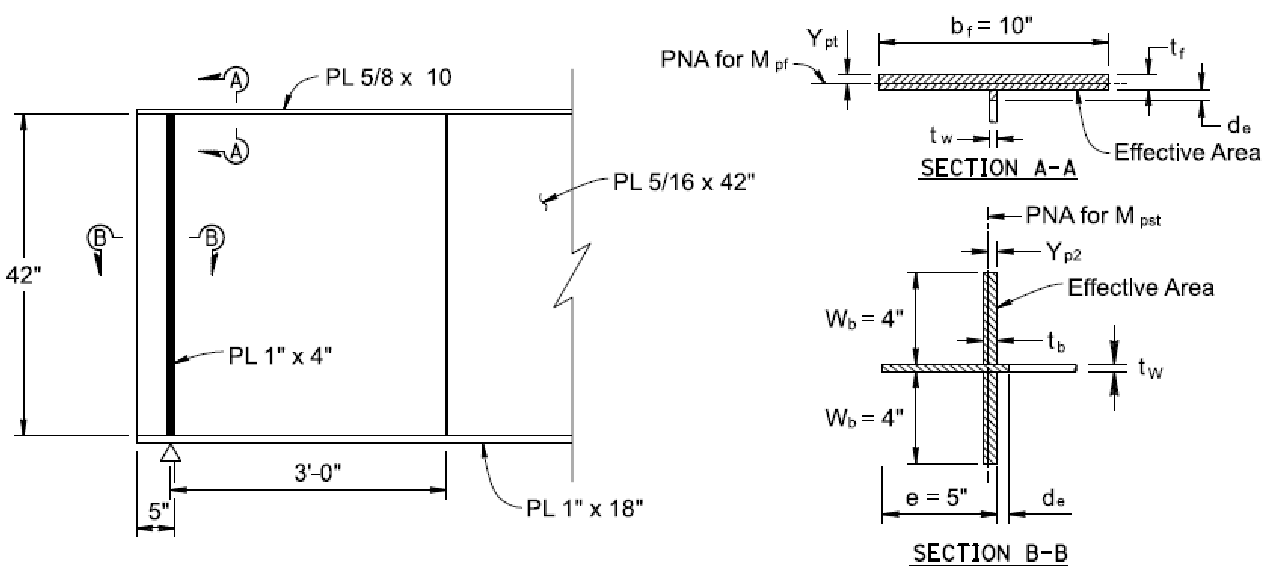


Figure 16.1.6-6 Effective web Area and Plastic Neutral Axis

Compute M_{pf} (Section A-A)

b_f = 10 in.; t_f = 0.625 in.; d_e = 0.0 in.

$$y_{p1} = \frac{b_f t_f + d_e t_w}{2 b_f} = \frac{(10)(0.625) + (0.0)(0.3125)}{(2)(10)} = 0.31 \text{ in.}$$

The plastic section modulus is calculated as follows:

$$Z_f = \frac{b_f y_{p1}^2}{2} + \frac{b_f \left(t_f - y_{p1}\right)^2}{2} + \frac{d_e t_w \left(t_f - y_{p1} + d_e\right)}{2} = 0.49 + 0.49 + 0.0 = 0.98 \ \text{in.}^3$$

$$M_{pf} = F_{yw} Z_f = (36)(0.98) = 35.2 \text{ kip-in.} \qquad \text{Control}$$

Compute M_{pst} (Section B-B)

W_b = 4 in.; t_w = = 0.3125 in.; t_b = 1.0 in.

$$e = 5.0 \text{ in.} < 0.84 t_w \sqrt{\frac{E_s}{F_{yw}}} = (0.84)(0.3125)\sqrt{\frac{(29{,}000)}{36}} = 7.45 \text{ in.} \quad \text{OK}$$

$$y_{p2} = \frac{(2W_b + t_w)t_b + (e - d_e - t_b)t_w}{2(2W_b + t_w)}$$

$$= \frac{\left[(2)(4) + 0.3125\right](1.0) + (5 - 0 - 1)(0.3125)}{(2)\left[(2)(4) + 0.3125\right]} = 0.575 \text{ in.}$$

The plastic section modulus is calculated as follows:

$$Z_b = \frac{2W_b y_{p2}^2}{2} + \frac{2W_b \left(t_b - y_{p2}\right)^2}{2} + \frac{\left(e - y_{p2}\right)^2 t_w}{2} + \frac{\left(d_e + y_{p2}\right)^2 t_w}{2}$$

$$= 1.323 + 0.722 + 3.059 + 0.05 = 5.16 \ \text{in.}^3$$

$$M_{pst} = F_{yw} Z_b = (36)(5.16) = 186 \text{ kip-in.}$$

$$M_{pm} = \min\left(M_{pf}, M_{pst}\right) = \min(35.2, 186) = 35.2 \text{ kip-in.}$$

Compute α,

$$\alpha = \frac{2.8\left(\sqrt{M_{pf} + M_{pm}} + \sqrt{M_{pst} + M_{pm}}\right)}{D\sqrt{t_w F_{yw}(1 - C)}}$$

$$= \frac{2.8\left(\sqrt{35.2 + 35.2} + \sqrt{186 + 35.2}\right)}{(42)\sqrt{(0.3125)(36)(1 - 0.813)}} = 1.07 > 1.0$$

Use α = 1.0.

The nominal shear resistance of the existing girder end panel is obtained as:

$$V_n = V_p\left[C + \alpha\frac{0.87(1-C)}{\sqrt{1+\left(d_o/D\right)^2}}\right]$$

$$=\left(274.1\right)\left[0.813+\left(1.0\right)\frac{\left(0.87\right)\left(1-0.813\right)}{\sqrt{1+\left(\dfrac{36}{42}\right)^2}}\right] = 256.7 \text{ kip}$$

$$\phi V_n = \left(1.0\right)\left(256.7\right) = 256.7 \text{ kip} < V_u = 326.5 \text{ kip} \qquad \text{NG}$$

Shear strengthening is needed at the end panel.

Since factored plastic shear ϕV_p = (1.0)(274.1) = 274.1 kip < V_u = 326.5 kip, the end panel shall be strengthened by adding new web plates.

16.1.6.10.2 Interior Panels

Factored shear forces at strength limit states for the beginning of interior panels are calculated from Table 16.1.6-5 by interpolation as summarized in Table 16.1.6-9.

For the first interior panel, the distance between the bearing and the first interior stiffener is 3 ft, and factored shear force is obtained as:

$$V_u = 326.5 - \frac{\left(3\right)(326.5-278.4)}{7} = 305.9 \text{ kip}$$

The factored shear resistance for a typical interior panel with d_o = 72 in. is calculated below and shown in Table 16.1.6-9.

D = 42 in; t_w = 0.3125 in.; F_{yw} =36 ksi; d_o = 72 in.

$$V_p = 0.58F_{yw}Dt_w = \left(0.58\right)\left(36\right)\left(42\right)\left(0.3125\right) = 274.1 \text{ kip} \quad \text{(AASHTO 6.10.9.3.2-3)}$$

$$k = 5 + \frac{5}{\left(d_o / D\right)^2} = 5 + \frac{5}{\left(72/42\right)^2} = 6.70 \qquad \text{(AASHTO 6.10.9.3.2-7)}$$

$$1.12\sqrt{\frac{E_s k}{F_{yws}}} = 1.12\sqrt{\frac{\left(29{,}000\right)\left(6.7\right)}{36}} = 82.3$$

$$1.4\sqrt{\frac{E_s k}{F_{yws}}} = 1.4\sqrt{\frac{(29{,}000)(6.7)}{36}} = 102.9$$

$$\because \; \frac{D}{t_w} = \frac{42}{0.3125} = 134.4 > 1.4\sqrt{\frac{E_s k}{F_{yw}}} = 102.9$$

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{E_s k}{F_{yw}}\right) = \frac{1.57}{(134.4)^2}\left(\frac{29{,}000(6.7)}{36}\right) = 0.469 \qquad \text{(AASHTO 6.10.9.3.2-6)}$$

$$V_n = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right]$$

(AASHTO 6.10.9.3.2-2)

$$= 274.1\left[0.469 + \frac{0.87(1-0.469)}{\sqrt{1+\left(\frac{72}{42}\right)^2}}\right] = 192.4 \text{ kip}$$

The factored shear resistance for a typical interior panel with d_o = 84 in. is obtained as 178.5 kip and is shown in Table 16.1.6-9.

Table 16.1.6-9 Factored Shear Forces and Resistances for Interior Panels at Strength Limit State

Interior Panel No	Position x (ft)	V_u (kip)	V_p (kip)	V_n (kip)	ϕV_n (kip)	$\phi V_n \geq V_u$	$\phi V_p \geq V_u$	Strenthening method
1	3.0 (67.0)	305.9	274.1	192.4	192.4	NG	NG	Add web plate
2	9.0 (61.0)	261.9	274.1	192.4	192.4	NG	OK	Add stiffener
3	15.0 (55.0)	212.5	274.1	192.4	192.4	NG	OK	Add stiffener
4	21.0 (49.0)	164.8	274.1	178.5	178.5	OK	OK	Not required

From Table 16.1.6-9, it is seen that

- For the first interior panel, since the factored plastic shear ϕV_p = (1.0)(274.1) = 274.1 kip < V_u = 305.9 kip, the first interior panel shall be strengthened by adding new web plates.
- For the 2nd and 3rd interior panels, since the factored plastic shear is larger than the factored shear force, the panels are strengthened by adding new stiffeners.
- There is no need to strengthen the rest of the interior panels.

16.1.6.11 Design for Shear Strengthening for End Panel

16.1.6.11.1 Design for Shear Resistance of Strengthened Web

Try to add ASTM A709 Grade 36, 1/4" web plate as shown in Figures 16.1.6-7 and 16.1-6-8.

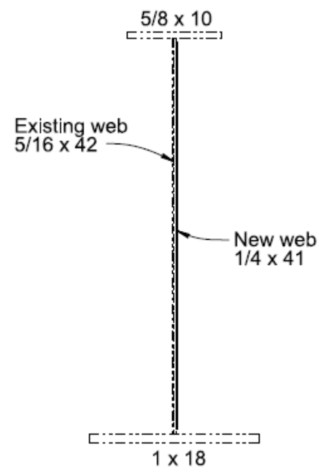


Figure 16.1.6 -7 Shear Strengthening by Adding Web Plate

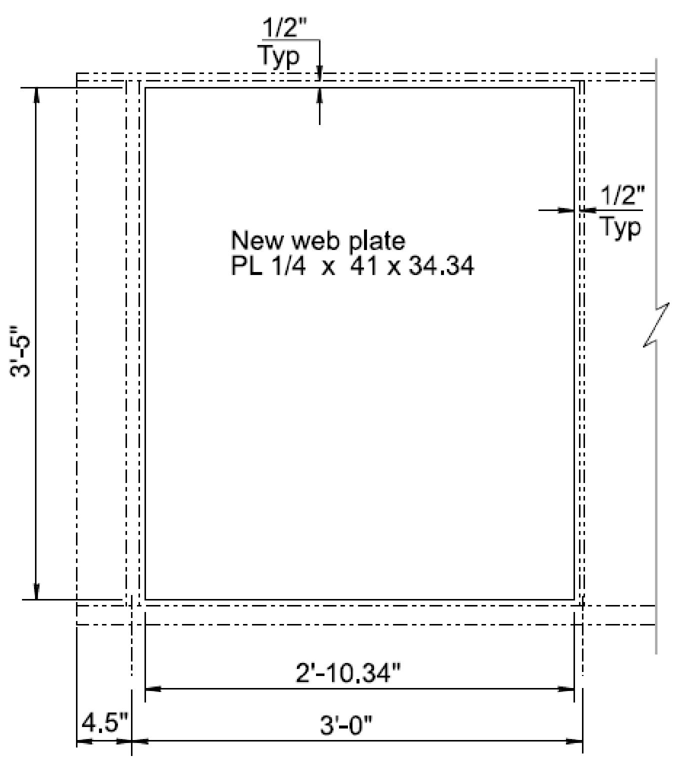


Figure 16.1.6-8 New Web Plate

Per STP 16.6.7.3.2, the combined thickness and the equivalent yield strength shall be used in determining the ratio of the shear-buckling resistance to the shear yield strength, *C*.

For this example,

t_{we}	=	thickness of the existing web (in.) = 0.3125 in.
t_{wn}	=	thickness of the new web (in.) = 0.25 in.
t_{ws}	=	thickness of the strengthened web (in.) = t_{we} + t_{wn} = 0.5625 in.
F_{yaw}	=	actual yield strength of the existing web (ksi) = 36 ksi
F_{yw}	=	specified minimum yield strength of the new web (ksi) = 36 ksi
F_{yws}	=	equivalent yield strength of the strengthened web (ksi)

$$F_{yws} = \frac{t_{we}F_{yaw} + t_{wn}F_{yw}}{t_{we} + t_{wn}} = 36 \text{ ksi} \qquad \text{(STP 16.6.7.3.1)}$$

Assume the distance between the new web plate and stiffeners or flanges to be 1/2 inch as shown in Figure 16.1.6–8. Use the average depth of the new web and the existing web as the strengthened web depth D_s = 41.5 in.

$$k = 5 + \frac{5}{\left(d_o / D_s\right)^2} = 5 + \frac{5}{\left(36/41.5\right)^2} = 11.64 \qquad \text{(AASHTO 6.10.9.3.2-7)}$$

$$\because \frac{D_s}{t_w} = \frac{41.5}{0.5625} = 73.8 < 1.12\sqrt{\frac{Ek}{F_{yw}}} = 108.5$$

$$\therefore C = 1.0 \qquad \text{(AASHTO 6.10.9.3.2-4)}$$

Since C = 1.0 and α = 1.0, the nominal shear resistance of the strengthened end panel is obtained as:

$$V_n = V_p\left[C + \alpha\frac{0.87(1-C)}{\sqrt{1+\left(d_o/D_s\right)^2}}\right]$$

$$= V_p = 0.58F_{yws}D_st_{ws} = (0.58)(36)(41.5)(0.5625) = 487.4 \text{ kip}$$

$$\phi V_n = (1.0)(487.4) = 487.4 \text{ kip} > V_u = 326.5 \text{ kip} \qquad \text{OK}$$

Added 1/4" web plate is sufficient at the end panel.

16.1.6.11.2 Design Web Plate Connection

Per STP 16.6.7.3.3, a new web plate shall be connected to the existing web by high strength bolts, and a new web plate connection shall be designed as slip-critical for the combined shear and moment.

At the strength limit state, a web plate connection shall be designed for a shear force taken as $0.58F_{yw}t_{wn}D$, combined with shear induced moments.

- Select Connection Bolt Layout

  Try F3125 Grade A325 HS 3/4" bolts with threads excluded from the shear plane for the new web plate connection to the existing steel web. Web plate connections shall satisfy the strength limit state, and the service II limit state and all requirements as specified in Article 6.13.2 as applicable.

  Article 6.13.2.6.3 specifies that the maximum pitch of fasteners in mechanically fastened builtup members shall not exceed the lesser of the requirements for sealing or stitch. The maximum sealing spacing is 7.0 in. (Article 6.13.2.6.2). The maximum pitch and gauge for stitch bolts in tension members is 24 times the thickness of the thinner outside plate = 24(0.25) = 6.0 in. (Article 6.13.2.6.3).

  Article 6.13.2.6.1 specifies that the minimum spacing between centers of bolts in standard holes shall be no less than three times the diameter of the bolt.

  Try the bolt layout – 7 bolts spaced at 5.22 in. along the horizontal direction and 8 bolts spaced at 5.43 in. along the vertical direction, as shown in Figure 16.1.6-9. This layout satisfies the maximum and minimum spacing requirements specified in Article 6.13.2.

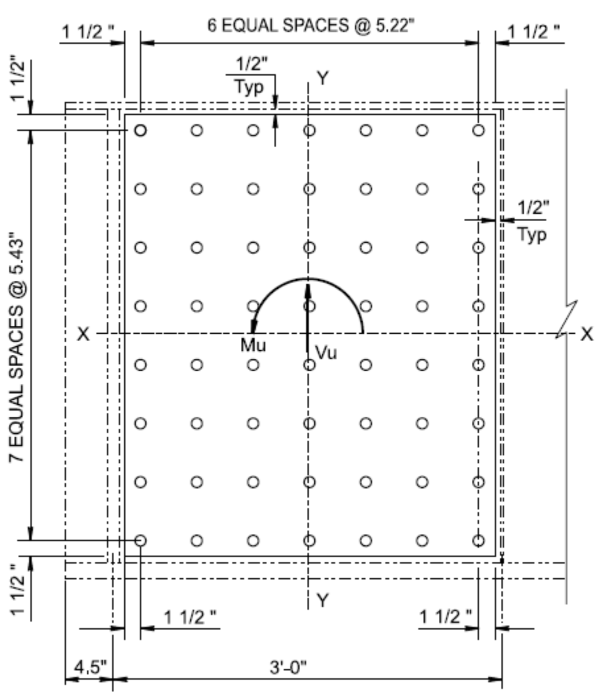


Figure 16.1.6-9 Web Connection Bolt Layout

- Calculate Shear and Shear Induced Moment about the Center of Gravity of the Bolt Group at Strength Limit State (STP 16.6.7.3.3)

$$V_u = 0.58F_{yw}t_{wn}D_n = (0.58)(36)(0.25)(41) = 214.0 \text{ kip}$$

$$M_u = V_u e_{vu} = 214.0\left(\frac{0.3125}{2} + 0.5 + 1.5 + 3(5.22)\right) = 214.0(17.82) = 3{,}813.5 \text{ kip-in.}$$

- Calculate Shear Resistance Per Bolt

For Grade A325 3/4” bolt,

A_b = cross-sectional area = 0.442 in.^{2}

F_{ub} = tensile strength of bolt = 120 ksi (AASHTO 6.4.3.1)

N_s = number of slip planes in connection = 1

The nominal shear resistance of a bolt is obtained as:

$$R_{n1} = 0.56A_bF_{ub}N_s. = (0.56)(0.442)(120)(1) = 29.7 \text{ kip} \qquad \text{(AASHTO 6.13.2.7-1)}$$

The nominal bearing resistance at bolt holes is obtained as:

For a Grade A325 3/4” bolt, the nominal diameter of the bolt, d = 0.75 in.; the bolt hole diameter is 0.813 in. (AASHTO Table 6.13.2.4.2-1); the edge distance is 1.5 in (AASHTO Table 6.13.2.6.6-1).

$$\because\ L_c = \text{the clear edge distance} = 1.5 - (0.813/2) = 1.09 \text{ in.} < 2d = 1.5 \text{ in.}$$

$$R_{n2} = 1.2L_ct_{wn}F_u = (1.2)(1.09)(0.25)(58) = 19.0 \text{ kip} \qquad \text{(AASHTO 6.13.2.9-2)}$$

It is seen that bearing resistance at bolt holes controls, and the nominal shear resistance per bolt is obtained as: .

$$R_n = \min(R_{n1,}R_{n2}) = 19.0 \text{ kip}$$

- Calculate Polar Moment of Inertia I_p of Bolts with Respect to Neutral Axis of New Web

From Figure 16.1.6-9, it can be seen that the upper and lower corner bolts are the most highly stressed and will be investigated. The “Vector” method is used to calculate the shear force R on the top right bolt.

$$I_p = \sum x^2 + \sum y^2$$
$$= (2)(8)\left(5.22^2 + 10.44^2 + 15.66^2\right) + (2)(7)\left(2.715^2 + 8.145^2 + 13.575^2 + 19.005^2\right)$$
$$= 14{,}772 \text{ in.}^2$$

- Check Shear Resistance of Lower Right Corner Bolt

Shear forces applied on the low right corner bolt are:

$$R_x = \frac{M_u y}{I_p} = \frac{3{,}813.5(19.005)}{14{,}772} = 4.91 \text{ kip} \quad (\rightarrow)$$

$$R_y = \frac{M_u x}{I_p} = \frac{3{,}813.5(15.66)}{14{,}772} = 4.04 \text{ kip} \quad (\uparrow)$$

$$R_v = \frac{V_u}{(7)(8)} = \frac{214.0}{56} = 3.82 \text{ kip} \quad (\uparrow)$$

$$R_h = 0$$

$$R_{bolt} = \sqrt{(R_h + R_x)^2 + (R_v + R_y)^2}$$
$$= \sqrt{(0 + 4.91)^2 + (3.82 + 4.04)^2}$$
$$= 9.27 \text{ kip} < \phi_{bb} R_n = (0.8)(19.0) = 15.2 \text{ kip} \qquad \text{OK}$$

- Calculate Shear and Shear Induced Moment at Service Limit State II (STP 16.6.7.3.3)

From Table 16.1.6-8, the factored shear at the end panel is equal to 216.9 kip. Factored shear resisted by the new web is obtained by proportioning the total web area.

$$V_u = \frac{t_{wn}}{t_{we} + t_{wn}}(216.9) = \frac{0.25}{0.25 + 0.3125}(216.9) = 96.4 \text{ kip}$$

$$M_u = V_u e_{vu} = 96.4(17.82) = 1{,}717.8 \text{ kip-in.}$$

- Calculate Slip Resistance Per Bolt

The nominal slip resistance per bolt is:

$$R_n = K_h K_s N_s P_t \qquad \text{(AASHTO 6.13.2.8-1)}$$

where K_h is the hole size factor and is equal to 1.0 for the standard hole (AASHTO

Table 6.13.2.8-2); K_s is the surface condition factor and is taken 0.5 for Class B surface condition (AASHTO Table 6.13.2.8-3); N_s is the number of slip planes and is equal to 1.0; P_t is the minimum required bolt tension and is equal to 28 kips (AASHTO Table 6.13.2.8-1).

$$R_n = K_h K_s N_s P_t = (1.0)(0.5)(1)(28) = 14.0 \text{ kip} \qquad \text{(AASHTO 6.13.2.8-1)}$$

- Check Slip Resistance of Lower Right Corner Bolt

Shear forces applied on the lower right corner bolt are:

$$R_x = \frac{M_u y}{I_p} = \frac{1{,}717.8(19.005)}{14{,}772} = 2.21 \text{ kip} \quad (\rightarrow)$$

$$R_y = \frac{M_u x}{I_p} = \frac{1{,}717.8(15.66)}{14{,}772} = 1.82 \text{ kip} \quad (\uparrow)$$

$$R_v = \frac{V_u}{(7)(8)} = \frac{96.4}{56} = 1.72 \text{ kip} \quad (\uparrow)$$

$$R_h = 0$$

$$R_{bolt} = \sqrt{(R_h + R_x)^2 + (R_v + R_y)^2}$$

$$= \sqrt{(0 + 2.21)^2 + (1.72 + 1.82)^2}$$

$$= 4.17 \text{ kip} < \phi R_n = (1.0)(14.0) = 14.0 \text{ kip} \qquad \text{OK}$$

16.1.6.11.3 Design Web-to-Flange Connection

- Check Existing Web-to-Flange Weld

Assume two fillet welds 5/16”, F_{exx} = 70 ksi.

$$R_r = 0.6\phi_{e2} F_{exx} = (0.6)(0.8)(70) = 33.6 \text{ ksi} \qquad \text{(AASHTO 6.13.3.2.4-1)}$$

The shear flow resistance is:

$$s_r = 2(0.707)(0.3125)R_r = (2)(0.707)(0.3125)(33.6) = 14.85 \text{ kip/in.}$$

The required horizontal shear flow at the strength limit state is:

$$s_u = 0.58(t_{we} + t_{wn})F_{yws} = (0.58)(0.3125 + 0.25)(36)$$

$$= 11.75 \text{ kip/in.} < s_r = 14.85 \text{ kip/in.} \qquad \text{OK}$$

Since the shear flow resistance of existing web-to-flange welds is larger than the required shear flow of the strengthened web, there is no need to design a new web-to-flange connection.

16.1.6.12 Design for Shear Strengthening for First Interior Panel

Try to add ASTM A709 Grade 36, 1/4" web plate as shown in Figure 16.1.6-10.

All design calculations are not illustrated herein. A similar design procedure is shown in Section 16.1.6.11.

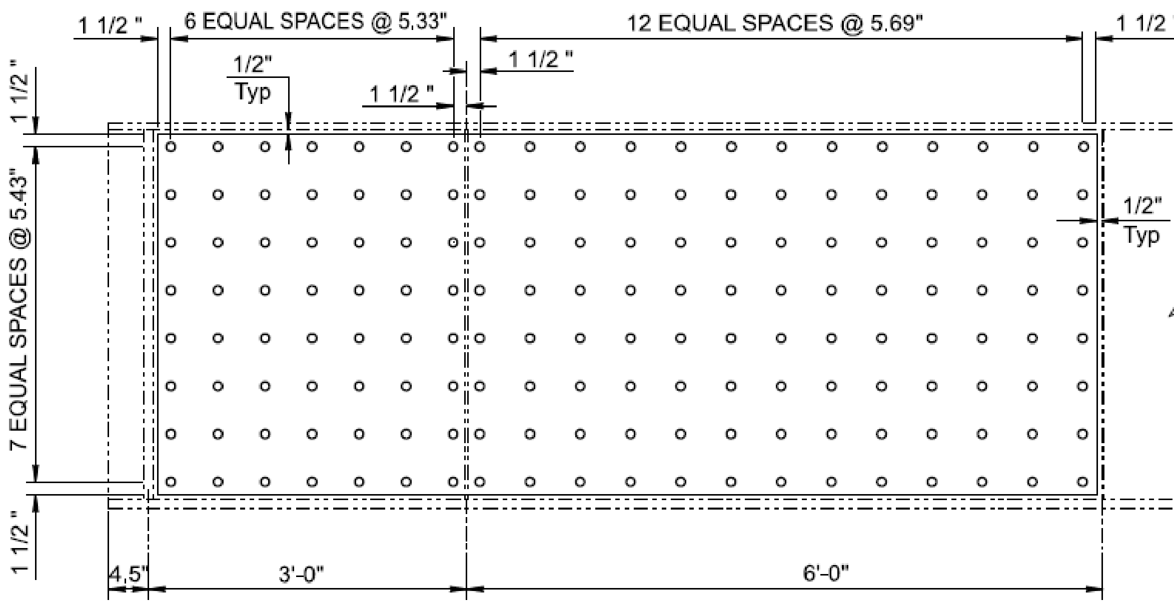


Figure 16.1.6-10 Connection Design Layout for End and 1st Interior Panels

16.1.6.13 Design for Shear Strengthening for Other Interior Panels

For the 2nd and 3rd interior panels, since the factored plastic shear is larger than the factored shear force, the panels are strengthened by adding new stiffeners.

16.1.6.13.1 Add New Stiffener within Second Interior Panel

Try to add a new stiffener Grade A36 L4x4x3/8 at d_o = 30 in. for the second interior panel as shown in Figure 16.1.6-11.

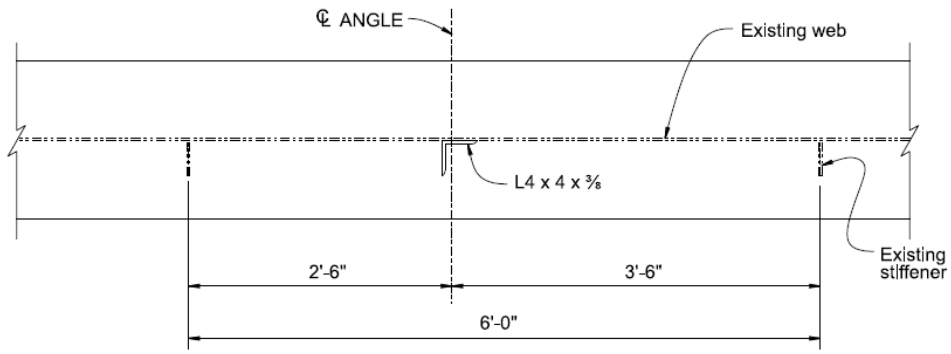


Figure 16.1.6-11 New Stiffener Layout for 2nd Interior Panel

The factored shear at the second interior panel (x = 9 ft) and the new stiffener location (x = 11.5 ft) are calculated from Table 16.1.6-5 by interpolation:

$$V_{u-x=9} = 261.9 \text{ kip} \qquad \text{(Table 16.1.6-9)}$$

$$V_{u-x=11.5} = 278.4 - \frac{(278.4 - 220.5)(4.5)}{7} = 241.2 \text{ kip} \qquad \text{(Table 16.1.6-5)}$$

For d_o = 30 in.,

$$k = 5 + \frac{5}{(d_o / D)^2} = 5 + \frac{5}{(30/42)^2} = 14.8 \qquad \text{(AASHTO 6.10.9.3.2-7)}$$

$$1.12\sqrt{\frac{E_s k}{F_{yws}}} = 1.12\sqrt{\frac{(29{,}000)(14.8)}{36}} = 122.3$$

$$1.4\sqrt{\frac{E_s k}{F_{yws}}} = 1.4\sqrt{\frac{(29{,}000)(14.8)}{36}} = 152.9$$

$$\because\ 1.12\sqrt{\frac{E_s k}{F_{yw}}} = 122.3 < \frac{D}{t_w} = \frac{42}{0.3125} = 134.4 < 1.4\sqrt{\frac{E_s k}{F_{yw}}} = 152.9$$

$$C = \frac{1.12}{\frac{D}{t_w}}\sqrt{\frac{E_s k}{F_{yw}}} = \frac{1.12}{134.4}\sqrt{\frac{29{,}000(14.8)}{36}} = 0.910 \qquad \text{(AASHTO 6.10.9.3.2-5)}$$

$$V_n = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(d_o/D\right)^2}}\right]$$

(AASHTO 6.10.9.3.2-2)

$$= 274.1\left[0.91 + \frac{0.87\left(1-0.91\right)}{\sqrt{1+\left(\frac{30}{42}\right)^2}}\right] = 266.8 \text{ kip}$$

$$\phi_v V_n = \left(1.0\right)\left(266.8\right) = 266.8 \text{ kip} > V_{u-x=9} = 261.9 \text{ kip} \qquad \text{OK}$$

For d_o = 42 in. at the new stiffener location, perform the similar calculation above, we obtain:

$$k = 5 + \frac{5}{\left(d_o / D\right)^2} = 5 + \frac{5}{\left(42/42\right)^2} = 10.0$$ (AASHTO 6.10.9.3.2-7)

$$C = \frac{1.12}{\frac{D}{t_w}}\sqrt{\frac{E_s k}{F_{yw}}} = \frac{1.12}{134.4}\sqrt{\frac{29{,}000\left(10.0\right)}{36}} = 0.7$$ (AASHTO 6.10.9.3.2-5)

$$V_n = 274.1\left[0.7 + \frac{0.87\left(1-0.7\right)}{\sqrt{1+\left(\frac{42}{42}\right)^2}}\right] = 242.5 \text{ kip}$$ (AASHTO 6.10.9.3.2-2)

$$\phi V_n = \left(1.0\right)\left(242.5\right) = 242.5 \text{ kip} > V_{u-x=11.5} = 241.2 \text{ kip} \qquad \text{OK}$$

16.1.6.13.2 Add New Stiffener within Third Interior Panel

Try to add a new stiffener, Grade A36 L4x4x3/8 at d_o = 36 in. for the third interior panel as shown in Figure 16.1.6-12.

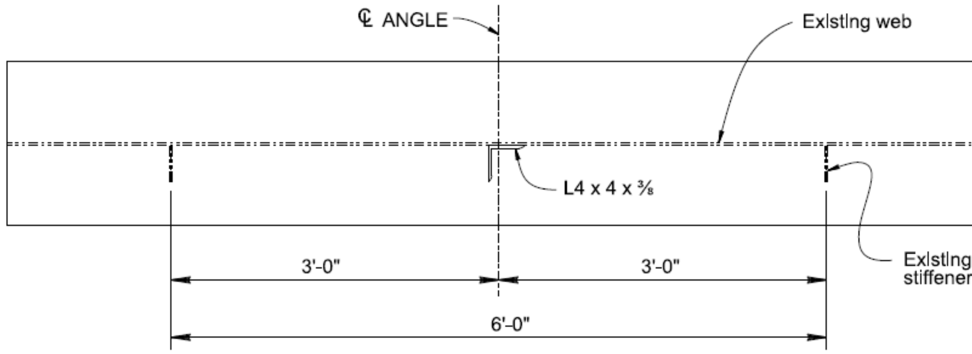


Figure 16.1.6-12 New Stiffener Layout for 3rd Interior Panel

The factored shear at the third interior panel (x = 15 ft) and the new stiffener location (x = 18 ft) are calculated from Table 16.1.6-5 by interpolation:

$$V_{u-x=15} = 212.5 \text{ kip} \qquad \text{(Table 16.1.6-9)}$$

$$V_{u-x=18} = 220.5 - \frac{(220.5-164.8)(18-14)}{7} = 188.7 \text{ kip} \qquad \text{(Table 16.1.6-5)}$$

From Section 16.1.6.10.1, for d_o = 36 in. we have:

$$V_n = 256.7 \text{ kip}$$

$$\phi V_n = (1.0)(256.7) = 256.7 \text{ kip} > V_{u-x=15} = 212.5 \text{ kip} \qquad \text{OK}$$

16.1.6.13.3 Design New Stiffener

The transverse stiffeners attached to the plate girder web panel are to provide nodal lines or simple support conditions during local buckling due to shear, thereby increasing the shear strength. The transverse stiffeners consist of plates welded or bolted to either one or both sides of the web and are required to satisfy the following requirements as specified in Article 6.10.11.1.

Check Transverse Stiffener L4x4x3/8

The following calculation is for the 2nd Interior Panel.

- Projecting width

$$b_t = 4.0 \text{ in.} > 2.0 + \frac{D}{30} = 2.0 + \frac{42}{30} = 3.4 \text{ in.} \qquad \text{OK} \qquad \text{(AASHTO 6.10.11.1.2-1)}$$

$16t_p = (16)(3/8) = 6.0 \text{ in.} > b_t = 4.0 \text{ in.} > b_f / 4 = 10/4 = 2.5 \text{ in. OK}$

(AASHTO 6.10.11.1.2-2)

- Moment of inertia

  For transverse stiffeners adjacent to web panels subject to the post-buckling tension-field action, the moment of inertia, I_t, of the transverse stiffener shall satisfy requirements specified in Article 6.10.11.1.3:

$$I_{t1} = b\, t_w^3\, J \qquad \text{(AASHTO 6.10.11.1.3-3)}$$

$$I_{t2} = \frac{D^4 \rho_t^{1.3}}{40}\left(\frac{F_{yw}}{E_s}\right)^{1.5} \qquad \text{(AASHTO 6.10.11.1.3-4)}$$

where I_t is the moment of inertia for the transverse stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs; b is the smaller of d_o and D; d_o is the smaller of the adjacent web panel widths.

$$J = \frac{2.5}{(d_o / D)^2} - 2.0 \geq 0.5 \qquad \text{(AASHTO 6.10.11.1.3-5)}$$

ρ_t is the larger of F_{yw}/F_{crs} and 1.0

$$F_{crs} = \frac{0.31 E_s}{(b_t / t_p)^2} \leq F_{ys} \qquad \text{(AASHTO 6.10.11.1.3-6)}$$

F_{ys} is the specified minimum yield strength of the stiffener = 36 ksi

$$\because J = \frac{2.5}{(30/42)^2} - 2.0 = 2.9 > 0.5 \qquad \text{OK}$$

b = smaller (d_o = 30 in. and D = 42 in.) = 30 in.

$$\because F_{crs} = \frac{0.31(29{,}000)}{(4/0.375)^2} = 79.01 \text{ ksi} > F_{ys} = 36 \text{ ksi}$$

Use F_{crs} = 36 ksi

ρ_t = larger (F_{yw}/F_{crs} = 36/36 = 1.0; 1.0) = 1.0

$$I_{t1} = b t_w^3 J = (30)(0.3125)^3 (2.9) = 2.66 \text{ in.}^4$$

$$I_{t2} = \frac{D^4 \rho_t^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5} = \frac{(42)^4 (1.0)^{1.3}}{40}\left(\frac{36}{29{,}000}\right)^{1.5} = 3.40 \text{ in.}^4$$

For L4x4x3/8, I_L = 4.32 in.^{4}; A_L = 2.86 in.^{2}

e = center of gravity from the back of angle = 1.13 in.

$$I_t = 4.32 + 2.86(1.13)^2 = 7.97 \text{ in.}^4$$

$$\because \; I_{t2} = 3.40 \text{ in.}^4 > I_{t1} = 2.66 \text{ in.}^4$$

$$I_t \geq I_{t1} + (I_{t2} - I_{t1})\rho_w \qquad \text{(AASHTO 6.10.11.1.3-7)}$$

$$\rho_w = \left(\frac{V_u - \phi_v V_{cr}}{\phi_v V_n - \phi_v V_{cr}}\right)$$

For the 2nd interior Panel, 1st segment 2'-6",

$$V_{cr} = CV_p = (0.91)(274.1) = 249.4 \text{ kip}$$

$$V_u = 261.9 \text{ kip}$$

$$\phi_v V_n = (1.0)(266.8) = 266.8 \text{ kip}$$

$$\rho_w = \frac{V_u - \phi_v V_{cr}}{\phi_v V_n - \phi_v V_{cr}} = \frac{261.9 - (1)(249.4)}{(1)(266.8) - (1)(249.4)} = 0.715$$

$$I_t = 7.97 \text{ in.}^4 > I_{t1} + (I_{t2} - I_{t1})\rho_w = 2.66 + (3.4 - 2.66)(0.715) = 3.19 \text{ in.}^4$$

OK (AASHTO 6.10.11.1.3-9)

For the 2nd interior Panel, 2nd segment 3'-6", from Section 16.1.6.13.1, we have C = 0.7.

$$V_{cr} = CV_p = (0.7)(274.1) = 191.9 \text{ kip}$$

$$V_u = 241.2 \text{ kip}$$

$$\phi_v V_n = (1.0)(242.5) = 242.5 \text{ kip}$$

$$\rho_w = \frac{V_u - \phi_v V_{cr}}{\phi_v V_n - \phi_v V_{cr}} = \frac{241.2 - (1)(191.9)}{(1)(242.5) - (1)(191.9)} = 0.974$$

$$I_t = 7.97 \text{ in.}^4 > I_{t1} + (I_{t2} - I_{t1})\rho_w = 2.66 + (3.4 - 2.66)(0.974) = 3.38 \text{ in.}^4$$

OK (AASHTO 6.10.11.1.3-9)

The new stiffener L4x4x3/8 is sufficient for the 2nd interior panel at the 2nd segment 3'-6".

The 3rd interior panel check is similar, since forces are smaller at the 3rd interior panel, no further check is needed.

Design Stiffener Connection to Web

STP 16.6.7.2 specifies that transverse stiffeners should be bolted to the existing web. However, both AASHTO-CA BDS-08 and STP 16.6 do not provide specific provisions on how to design the transverse stiffener connection to the web. Since transverse stiffeners are mainly expected to resist the axial forces developed by anchoring the diagonal tension field action, the stiffener-to-web connection is, therefore, designed for the axial force induced by the diagonal tension field action.

As stated in Article C6.10.9.3.2, the nominal shear resistance, AASHTO Equation 6.10.9.3.2-2, is contributed by the first term in the bracket related to either the shear yield or shear-buckling force and the second term related to the postbuckling tension-field force. Therefore, the compression force developed in the transverse stiffener is:

$$P_{st} = V_p \left[\frac{0.87(1-C)}{\sqrt{1+\left(d_o/D\right)^2}} \right] \tag{16.1.6.13.3-1}$$

- Calculate Compression Force Developed in Transverse Stiffener

  The compression force developed in the transverse stiffener in the second interior panel is shown in Figure 16.1.6-13.

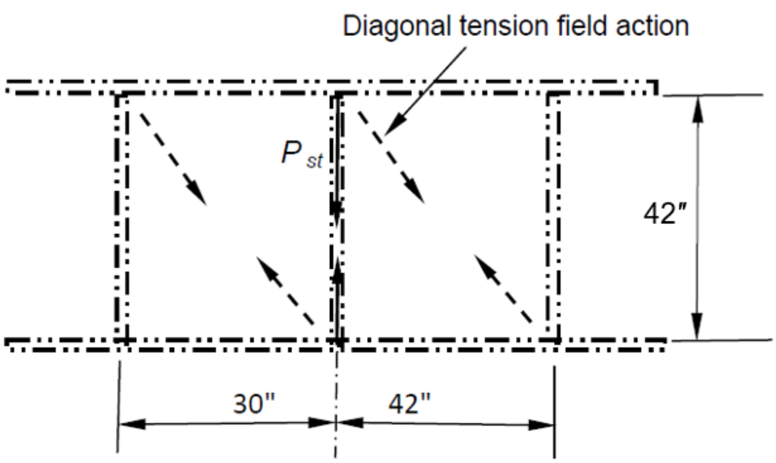


Figure 16.1.6-13 Compression Force Developed by Tension Field Action

The compression force developed in the transverse stiffener for the 2nd interior Panel, 1st segment 2'-6", is obtained as:

$$P_{st} = 274.1\left[\frac{0.87(1-0.91)}{\sqrt{1+\left(\frac{30}{42}\right)^2}}\right] = 30.7 \text{ kip}$$

- Design Bolted Stiffener to Web Connection

Try Grade A325 HS 3/4" bolts with threads excluded from the shear plane.

As calculated in Section 16.1.6.11.2, the nominal shear resistance of a bolt, $R_{n1} = 29.7$ kip

Try the edge distance of 1.5 in. for L4x4x3/8, nominal bearing resistance at bolt holes

$$\because\ L_c = \text{the clear edge distance} = 1.5 - (0.813/2) = 1.09 \text{ in.} < 2d = 1.5 \text{ in.}$$

$$R_{n2} = 1.2 L_c t_l F_u = (1.2)(1.09)(3/8)(58) = 28.5 \text{ kip} \qquad \text{(AASHTO 6.13.2.9-2)}$$

It is seen that the bearing resistance at bolt holes controls and the nominal shear resistance per bolt is obtained as:

$$R_n = \min\left(R_{n1}, R_{n2}\right) = 28.5 \text{ kip}$$

The maximum bolt spacing for stitching of the compression member (Article 6.13.2.6.3) is:

$$s_{\max} = 12t = (12)(3/8) = 4.5 \text{ in.}$$

The minimum bolt spacing (Article 6.13.2.6.1) is:

$$s_{\min} = 3d_b = (3)(0.75) = 2.25 \text{ in.}$$

Try N = 10 bolts with 9 equal spaces at 4.33″ as shown in Figure 16.1.6-14.

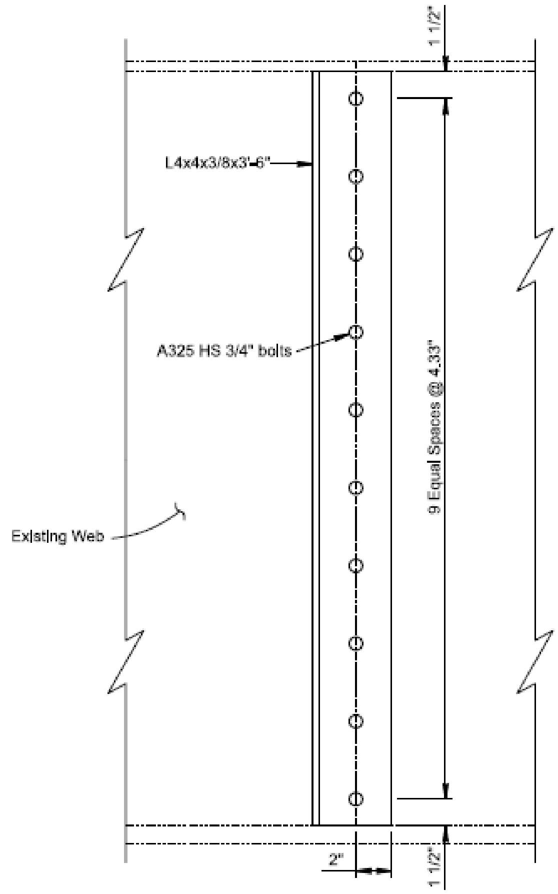


Figure 16.1.6-14 Stiffener Connection to Web

$$s_{\min} = 2.25 \text{ in.} < s = 3.9 \text{ in.} < s_{\max} = 4.5 \text{ in.} \qquad \text{OK}$$

The required number of bolts

$$N_{reqd} = \frac{P_{st}}{R_n} = \frac{30.7}{28.5} = 1.08 < N = 10 \qquad \text{OK}$$

It is obvious that the bolt layout shown in Figure 16.1.6-14 is sufficient for both the 2nd and 3rd interior panels since the 3rd panel shear force at the added stiffener is smaller than the 2nd panel.

NOTATION

A_b = cross-sectional area of a bolt (in.^{2})

A_c = cross-sectional area of concrete deck (in.^{2})

A_{fc} = area of the existing compression flange; area of the existing compression flange (in.^{2})

A_{ft} = area of the tension flange; area of the existing tension flange (in.^{2})

A_{gcp} = gross area of the tension cover plate (in.^{2})

A_{ge} = gross area of the steel girder at end span (in.^{2})

A_{gf} = gross area of the existing tension flange (in.^{2})

A_{gm} = gross area of the steel girder at midspan (in.^{2})

A_i = area of the component *i* (in.^{2})

A_{LTs} = long-term composite section area of the strengthened steel girder (in.^{2})

A_{ncp} = net area of the tension cover plate (in.^{2})

A_{nf} = net area of the existing tension flange (in.^{2})

A_{ps} = area of the prestressing steel (in.^{2})

A_{ST} = short-term composite section area of the strengthened steel girder (in.^{2})

A_{sc} = cross-sectional area of a stud shear connector (in.^{2})

A_w = area of the web; area of the existing web (in.^{2})

a = length of the web plate in an anchorage bracket (in)

a_{wc} = ratio of two times the web area in compression to the area of the compression

$ADTT$ = average daily truck traffic over the design life

$ADTT_{SL}$ = single-lane $ADTT$

b = width of the web plate in an anchorage bracket (in); the smaller of d_o and D;

b_{brpn} = net width of the bearing plate in an anchorage bracket (in)

b_{cp} = width of the cove plate (in.)

b_{eff} = effective flange width (in.)

b_f = width of the flange (in.)

b_{fc} = width of the compression flange (in.)

b_{ft} = width of the tension flange (in.)

b_s = effective width of concrete deck (in.)

b_t = width of the stiffener (in.)

C = ratio of the shear-buckling resistance to the shear yield strength determined by Article 6.10.9.3

C_b = moment gradient modifier

CG = centroid of gravity

c = distance from the extreme compression fiber to the neutral axis (in.)

D = depth of the web (in.)

D_c = depth of the web in compression in the elastic range (in.)

D_{cp} = depth of the web in compression at the plastic moment state (in.)

D_p = depth from the top of the concrete deck to the PNA (in.)

D_s = average depth of the new web and the existing web

D_t = total depth of the composite section (in.)

$DC1$ = structural dead load, acting on the noncomposite section

$DC2$ = nonstructural dead load, acting on the long term composite section

DF_m = live load moment distribution factor

DF_v = live load shear distribution factor

DW = future wearing surface load

d = nominal diameter of the bolt (in.)

d_e = effective web depth (in.)

d_o = transverse stiffener spacing (in.); smaller of the adjacent web panel widths (in.)

d_p = distance from extreme compression fiber (deck) to the centroid of the prestressing tendons (in.)

d_{ps} = distance from plastic neutral axis to the centroid of the prestressing tendons (in.)

d_s = distance from plastic neutral axis to the midthickness of the concrete deck used to compute the plastic moment (in.)

d_t = distance from plastic neutral axis to the midthickness of the tension flange used to compute the plastic moment (in.)

d_w = distance from plastic neutral axis to the middepth of the web used to compute the plastic moment (in.)

E_s = modulus of elasticity of steel (ksi)

E_c = modulus of elasticity of concrete (ksi)

E_{ca} = actual modulus of elasticity of concrete (ksi)

E_p = modulus of elasticity of prestressing steel (ksi)

e = distance from the inside face of the bearing stiffener to the end of the girder, for calculation purposes, must not be exceed $0.84t_w\sqrt{E/F_{yw}}$ (in.); distance from the edge of the web plate to prestressing force in an anchorage bracket (in.)

e_g = distance between CG of the concrete deck slab and CG of noncomposite section (in.)

e_s = distance from prestressing force to CG of the bearing plate in an anchorage bracket (in.)

e_{LTs} = eccentricity of prestressing force, distance from CG of prestressing steel to the neutral axis of the long-term composite section of the strengthened girder (in.)

e_{ST} = eccentricity of prestressing force, distance from CG of prestressing steel to the neutral axis of the short-term composite elastic section of the existing steel girder (in.)

e_{STs} = eccentricity of prestressing force, distance from CG of prestressing steel to the neutral axis of the short-term composite elastic section of the strengthened girder (in.)

e_{vu} = eccentricity of the shear force about center of gravity of bolt group (in.)

F_{crs} = local buckling stress for the stiffener (ksi)

F_{exx} = specified minimum tensile strength of the weld metal (ksi)

F_u = specified minimum tensile strength of steel (ksi)

F_{ua} = actual tensile strength of steel (ksi)

F_{uaf} = actual tensile strength of the existing tension flange (ksi)

F_{ub} = tensile strength of bolt (ksi)

F_{ucp} = specified minimum tensile strength of the cover plate (ksi)

F_y = specified minimum yield strength of steel (ksi)

F_{ya} = actual yield strength of steel (ksi)

F_{yaf} = actual yield strength of the existing steel tension flange (ksi); actual yield strength of the existing steel tension flange without holes (ksi)

F_{yf} = specified yield strength of the steel flange (ksi)

F_y^* = yielding stress for the existing flange, or the cover plate (ksi)

F_{yc} = specified minimum yield strength of the cover plate; specified minimum yield strength of the compression flange (ksi)

F_{yr} = compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects but not including compression-flange lateral bending (ksi)

F_{yt} = specified minimum yield strength of the tension flange (ksi)

F_{yaw} = actual yield strength of the existing web (ksi)

F_{ycp} = specified minimum yield strength of the cover plate (ksi)

F_{yw} = specified minimum yield strength of the web (ksi)

F_{yws} = nominal yield strength of the strengthened web (ksi)

f_c	=	allowable compression stress for flexure for concrete (ksi)
f_{cgp}	=	sum of steel stresses at the center of gravity of prestressing tendons due to the prestressing force after jacking and the self-weight of the member at the section of the maximum moment (ksi)
f'_c	=	concrete compressive strength used in design (ksi)
f'_{ca}	=	actual concrete compressive strength (ksi)
f_f	=	stress in flanges and cover plates at the service II limit state as specified in Article 6.10.4.2.2 (ksi)
f_{fps}	=	stress in the steel flange due to the effective prestressing force P_{pe} (ksi)
f_j	=	jacking stress in prestressing steel (ksi)
f_{pe}	=	effective prestress stress in prestressing steel after losses (ksi)
f_{ps}	=	stress in the unbonded prestressing steel at the strength limit state (ksi)
f_{pu}	=	specified tensile strength of prestressing steel (ksi)
f_{py}	=	yield strength of prestressing steel (ksi)
f_{ra}	=	actual modulus of rupture of concrete (ksi)
f_s	=	allowable stress for steel reinforcement (ksi)
f_{sps}	=	stress in prestressing steel (ksi)
$HL93$	=	design Vehicular live load
h	=	depth between the centerline of flange (in.)
I_o	=	moment of inertia of the component about its CG (in.^{4})
I_{NC}	=	moment inertia of the existing steel girder section alone (in.^{4})
I_p	=	polar moment of Inertia of bolts with respect to neutral axis(in.^{2})
I_{ST}	=	moment inertia of the short-term composite section of the existing steel girder (in.^{4})
I_{STs}	=	moment inertia of the short-term composite section of the strengthened steel girder (in.^{4})
I_t	=	moment of inertia for the transverse stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs (in.^{4})
I_x	=	moment of inertia of the section about x-x axis (in.^{4})
I_{yc}	=	moment of inertia of the compression flange about its principal axis within the plane of the web of the section (in.^{4})
I_{yt}	=	moment of inertia of the tension flange about its principal axis within the plane of the web of the section (in.^{4})
IM	=	dynamic load allowance percent

J = St. Venant Torsional constant (in.^{4}); stiffener bending rigidity parameter

K_g = the longitudinal stiffness parameter (in.^{4})

K_h = hole size factor

K_s = surface condition factor

k = shear-buckling coefficient for webs

L = length of the girder span under consideration (in.)

L_b = unbraced length (in.)

L_{brp} = length of the bearing plate in an anchorage bracket (in.)

L_c = clear edge distance (in.)

L_{cp} = length of the cover plate (in.)

L_p = limiting unbraced length to achieve the nominal flexural resistance of $R_bR_hF_{yc}$ under the uniform bending (in.)

L_{ps} = length of prestressing steel (in.)

L_r = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression-flange residual stress effects (in.)

L_{st} = theoretically required strengthening length (in.)

l_e = effective tendon length (in.)

l_i = length of a tendon between anchorages (in.)

M_A = absolute values of moments at quarter point in the unbraced segment (kip-in.)

M_{AD} = additional moment that must be applied to the short-term composite section of the strengthened steel girder to cause yielding in either the flanges, or the cover plates (kip-in.)

M_B = absolute values of moments at centerline in the unbraced segment (kip-in.)

M_C = absolute values of moments at the three-quarter point in the unbraced segment (kip-in.)

M_{cp} = moment due to unfactored permanent load of cover plate (kip-in.)

M_{DC1} = moment due to unfactored permanent load *DC1* (kip-in.)

M_{DC2} = moment due to unfactored permanent load *DC2* (kip-in.)

M_{DW} = moment due to unfactored permanent load *DW* (kip-in.)

M_{D1} = moment due to the factored permanent load at the strength limit state applied to the steel section of the existing girder before the concrete deck has hardened or is made composite (kip-in.)

M_{D2p} = moment due to the factored permanent loads after he post-tensioning at the strength limit state applied to the long-term composite section of the strengthened steel girder (kip-in.)

M_{D2} = moment due to the factored permanent load at the strength limit state applied to the long-term composite section of the existing steel girder after the concrete deck has hardened or is made composite (kip-in.)

M_{FHL93} = moment due to the unfactored fatigue truck *HL93* and dynamic load allowance (kip-in.)

M_{FP9} = moment due to the unfactored fatigue permit truck *P9* and dynamic load allowance (kip-in.)

M_{HL93} = moment due to the unfactored fatigue truck *HL93* and dynamic load allowance (kip-in.)

M_{P15} = moment due to the unfactored fatigue permit truck *P15* and dynamic load allowance (kip-in.)

M_{max} = absolute value of the maximum moment in the unbraced segment (kip-in.)

M_n = nominal flexural resistance (kip-in.)

$M_{nc(FLB)}$= nominal flexural resistance based on compression flange local buckling (kip-in.)

$M_{nc(LTB)}$= nominal flexural resistance based on compression flange lateral torsional buckling (kip-in.)

M_{nt} = nominal flexural resistance based on tension flange yielding (kip-in.)

M_p = plastic moment (kip-in.)

M_{pf} = plastic moment of the vertical section composed of the top flange and the segment of the web with the depth, d_e, about the plastic neutral axis perpendicular to the web (kip-in.)

M_{pm} = smaller of M_{pst} and M_{pf} (kip-in.)

M_{pst} = plastic moment of the horizontal section composed of the bearing stiffener and the segment of the web with depth, d_e, plus the distance from the inside face of the stiffener to the end of the girder, *e*, about the plastic neutral axis perpendicular to the web (kip-in.)

M_{SD1} = moment due to the factored permanent load at the service limit state applied to the steel section of the existing girder before the concrete deck has hardened or is made composite (kip-in.)

M_{SD2} = moment due to the factored permanent load at the service limit State applied to the long-term composite section of the existing steel girder after the concrete deck has hardened or is made composite (kip-in.)

M_{SDp} = moment due to the factored permanent loads after the post-tensioning at the service II limit state applied to the long-term composite section of the strengthened steel girder (kip-in.)

M_{SL} = moment due to the factored transient load at the service II limit state applied to the short-term composite section of the strengthened steel girder (kip-in.)

M_y = yield moment of the existing girder corresponds to the first yielding of either steel flange (kip-in.)

M_{yc} = yield moment of the strengthened girder corresponds to the yielding of the compression flange (kip-in.)

M_{yt} = yield moment of the strengthened girder corresponds to the yielding of the tension flange (kip-in.)

M_{ys} = yield moment of the strengthened girder corresponds to the first yielding of either steel flange, or cover plate (kip-in.)

M_u = factored moment (kip-in.)

N = number of fatigue cycles

N_b = number of girders

N_s = number of the slip plane in connection; number of plastic hinges at supports in an assumed failure mechanism crossed by the tendon between anchorages or discretely bonded points as assumed as = 0 for simple span

N_{reqd} = total number of bolts required

n = number of braced points with the span; modular ratio; number of shear studs

n_{prod} = number of stud shear connectors provided

n_{ps} = modular ratio for prestressing steel

P = total nominal shear force in the concrete deck for the design of the shear connectors at the strength limit state (kip)

P_c = plastic force in the compression flange used to compute the plastic moment (kip)

P_{c1} = plastic compression force in the top portion of the compression flange used to compute the plastic moment (kip)

P_{c2} = plastic tension force in the bottom portion of the compression flange used to compute the plastic moment (kip)

P_j = prestressing force at jacking (kip)

PNA = plastic neutral axis

P_{pe} = effective prestress force after all losses (kip)

P_{rb} = plastic force in the bottom layer of longitudinal deck reinforcement in the compression flange used to compute the plastic moment (kip)

P_{rt} = plastic force in the top layer of longitudinal deck reinforcement in the compression flange used to compute the plastic moment (kip)

P_s = plastic compressive force in the concrete deck used to compute the plastic moment (kip)

P_{st} = compression force developed in the transverse stiffener (kip)

P_t = plastic force in the tension flange used to compute the plastic moment; minimum required bolt tension (kip)

P_u = factored axial force (kip)

P_w = plastic force in the web used to compute the plastic moment (kip)

P15 = design permit truck

P9 = fatigue design permit truck

p = fraction of truck traffic in a single lane; pitch of shear connectors (in.);

p_{reqd} = required pitch of shear connectors (in.)

Q = first moment of transformed short-term area of the concrete deck about the neutral axis of the short-term composite section (in.^{3})

Q_i = Force effect

Q_n = nominal shear resistance of one shear connector (kip)

Q_r = factored shear resistance of one shear connector (kip)

R_{bot} = resultant force applied on a bolt (kip)

R_h = Hybrid factor determined as specified in Article 6.10.1.10.1; shear force applied on a bolt in horizontal direction due to shear (kip)

R_n = nominal resistance; nominal resistance of the bolt, connection, or connection material, in a nearing type connection, as specified in Articles 6.13.2.7 and 6.13.2.9; nominal resistance of the bolt in a slip-critical connection as specified in Article 6.13.2.8

R_{pc} = web plastification factor for the compression flange

R_{pt} = web plastification factor for the tension flange

R_r = factored resistance

R_v = shear force applied on a bolt in vertical direction due to shear (kip)

R_x = shear force applied on a bolt in *x-x* direction due to moment (kip)

R_y = shear force applied on a bolt in y-y direction due to moment (kip)

r_t = effective radius of gyration for lateral torsional buckling (in.)

S = girder spacing (in.);

S_{bp} = section modulus of the base plate in an anchorage bracket (in.)

S_{brp} = section modulus of the bearing plate in an anchorage bracket (in.)

S_{LT} = long-term composite elastic section modulus of the existing steel girder (in.^{3})

S_{LTb} = long-term composite elastic section modulus for the bottom flange of the existing steel girder (in.^{3})

S_{LTd} = long-term composite elastic section modulus for the concrete deck on the existing steel girder (in.^{3})

S_{LTt} = long-term composite elastic section modulus for the top flange of the existing steel girder (in.^{3})

S_{LTs} = long-term composite elastic section modulus of the strengthened steel girder (in.^{3})

S_{LTsb} = long-term composite elastic section modulus for the bottom flange of the strengthened steel girder (in.^{3})

S_{LTst} = long-term composite elastic section modulus for the top flange of the strengthened steel girder (in.^{3})

S_{NC} = noncomposite elastic section (steel section alone) modulus of the existing steel girder (in.^{3})

S_{reqd} = required bolt spacing (in.)

S_{ST} = short-term composite elastic section modulus of the existing steel girder (in.^{3})

S_{STb} = short-term composite elastic section modulus for the bottom flange of the existing steel girder (in.^{3})

S_{STd} = short-term composite elastic section modulus for the concrete deck on the existing steel girder (in.^{3})

S_{STt} = short-term composite elastic section modulus for the top flange of the existing steel girder (in.^{3})

S_{STs} = short-term composite elastic section modulus of the strengthened steel girder (in.^{3})

S_{STsb} = short-term composite elastic section modulus for the bottom flange of the strengthened steel girder (in.^{3})

S_{STsb2} = short-term composite elastic section modulus for the bottom cover plate of the strengthened steel girder (in.^{3})

S_{STst} = short-term composite elastic section modulus for the top flange of the strengthened steel girder (in.^{3})

S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3})

S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})

s_{max} = maximum bolt spacing (in.)

s_{min} = minimum bolt spacing (in.)

s_r = shear flow resistance of the weld (kip/in.)

s_u = resultant force on the weld (kip/in.)

s_{uvx} = shear component on the weld (kip/in.)

s_{utx} = tension or compression component on the weld (kip/in.)

T_{cp} = longitudinal force developed in the cover plate (kip)

T_n = nominal tensile resistance of a bolt subjected to combined shear and axial tensile (kip)

T_u = tension force in a bolt (kip)

t_{brp} = thickness of the bearing plate in an anchorage bracket (in.)

t_{cp} = thickness of a cover plate (in.)

t_{fc} = thickness of the compression flange (in.)

t_{ft} = thickness of the tension flange (in.)

t_p = thickness of a projecting stiffener element (in.)

t_t = thickness of the stiffener (in.)

t_s = thickness of the concrete deck (in.)

t_w = thickness of the web (in.)

t_{we} = thickness of the existing web (in.)

t_{weld} = fillet weld size (in.)

t_{wn} = thickness of the new web (in.)

t_{wp} = thickness of the web plate in an anchorage bracket (in.)

t_{ws} = thickness of the strengthened web (in.)

V_{cp} = shear due to unfactored permanent load of cover plate (kip)

V_{cr} = shear-yielding or shear-buckling resistance (kip)

V_{DC1} = shear due to unfactored permanent load *DC1* (kip)

V_{DC2} = shear due to unfactored permanent load *DC2* (kip)

V_{DW} = shear due to unfactored permanent load *DW* (kip)

V_f = factored fatigue vertical shear force range (kip)

V_{FHL93} = shear due to the unfactored fatigue truck *HL93* and dynamic load allowance (kip)

V_{FP9} = shear due to the unfactored fatigue permit truck *P9* and dynamic load allowance (kip)

V_{HL93} = shear due to the unfactored design live load *HL93* and dynamic load allowance (kip)

V_n = nominal shear resistance (kip)

V_{P15} = shear due to the unfactored design permit truck *P15* and dynamic load allowance (kip)

V_p = plastic shear force as specified in Article 6.10.9.2 (kip)

V_{sr} = range of horizontal shear flow (kip)

V_{st} = stiffener volume (in.^{3})

V_u = shear due to factored loads at the strength limit state (kip); shear due to the factored transient loads at the service II limit state (kip)

$W_{barrier}$ = weight of barriers (lb/ft)

W_{bp} = width of the base plate in an anchorage bracket (in.)

W_{brp} = width of the bearing plate in an anchorage bracket (in.)

W_{cp} = weight of the cover plate (k/ft)

W_{br} = weight of bracings (lb/ft)

W_{ge} = weight of steel girder at end span (lb/ft)

W_{gm} = weight of steel girder at midspan (lb/ft)

W_{misc} = miscellaneous dead loads (lb/ft)

$W_{railing}$ = weight of railings miscellaneous dead loads (lb/ft)

W_s = weight of deck slab (kip)

W_{st} = weight of stiffeners (kip/ft)

w_c = unit weight of concrete (kcf)

w_s = unit weight of steel (kcf)

y_i = distance between component CG and the bottom of the bottom flange (in.)

y_{LT} = distance between the component and CG of the long-term existing composite girder section (in.)

y_{LTb} = distance between the bottom of the bottom flange and CG of the long-term existing composite girder section (in.)

y_{LTd} = distance between the top of the concrete deck and CG of the long-term existing composite girder section (in.)

y_{LTt} = distance between the top of the top flange and CG of the long-term existing composite girder section (in.)

y_{NAb} = distance between the bottom flange and CG of the combined prestressing steel and existing steel girder section alone (in.)

y_{NAt} = distance between the top flange and CG of the combined prestressing steel and existing steel girder section alone (in.)

y_{NCb} = distance between the bottom flange and CG of noncomposite steel section (in.)

y_{NCt} = distance between the top flange and CG of noncomposite steel section (in.)

y_{ST} = distance between the component and CG of the short-term existing composite girder section (in.)

y_{STb} = distance between the bottom of the bottom flange and CG of the short-term existing composite girder section (in.)

y_{STd} = distance between the top of the concrete deck and CG of the short-term existing composite girder section (in.)

y_{STs} = distance between the component and CG of the short-term strengthened composite girder section (in.)

y_{STt} = distance between the top of the top flange and CG of the short-term existing composite girder section (in.)

y_{STsb} = distance between the bottom of the bottom flange and CG of the short-term strengthened composite girder section (in.)

y_{STsb2} = distance between the bottom of the cover plate and CG of the short-term strengthened composite girder section (in.)

y_{STst} = distance between the top of the top flange and CG of the short-term strengthened composite girder section (in.)

$\bar{Y}$ = distance from the plastic neutral axis to the top of the element where the plastic neutral axis is located (in.)

Z_b = plastic section modulus of bearing stiffener (in.^{3})

Z_f = plastic section modulus of flange (in.^{3})

Z_r = shear fatigue resistance of an individual shear connector (kip)

$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold (ksi)

Δf = fatigue stress range (ksi)

Δf_{pA} = loss due to anchorage set (ksi)

Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)

Δf_{pLT} = long term loss (ksi)

Δf_{pT} = total loss (ksi)

ϕ = resistance factor

ϕ_{bb} = resistance factor for bolt bearing on material

ϕ_{e2} = resistance factor for the weld metal in fillet weld - shear in throat of weld metal

ϕ_f = resistance factor for flexure

ϕ_s = resistance factor for ASTM F3125 bolts in shear

ϕ_{sc} = resistance factor for shear connector

ϕ_u = resistance factor for tension, fracture in net section

ϕ_v = resistance factor for shear

ϕ_y = resistance factor for tension, yielding in gross section

α = parameter to consider partial tension-field action; factor defining the sloping straight line representing the finite-life portion of the fatigue shear resistance of an individual stud shear connector

λ_f = slenderness ratio for compression flange

λ_{pf} = limiting slenderness ratio for a compact flange

λ_{rw} = limiting slenderness ratio for a noncompact web

$\lambda_{rw(Dc)}$ = limiting slenderness ratio for a compact web corresponding to $2D_c/t_w$

$\lambda_{pw(Dcp)}$ = limiting slenderness ratio for a compact web corresponding to $2D_{cp}/t_w$

λ_w = slenderness ratio for the web based on the elastic moment

ρ_t = the larger of F_{yw}/F_{crs} and 1.0

ρ_w = shear force factor

η_i = load modifier factor = 1.0

γ_i = load factor

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