Report on Techniques for Bridge Strengthening

Design Example – Concrete Cap Strengthening

June 2018

U.S. Department of Transportation

Federal Highway Administration

FHWA-HIF-18-045

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Foreword

This design example is targeted at bridge owners and bridge engineers who have been tasked with strengthening an existing bridge. It is intended to be an aid in designing appropriate bridge strengthening retrofits. Each example, in the set of examples, covers a different situation for which strengthening is commonly needed.

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This design example, concrete pier cap strengthening, involves the addition of external post-tensioning bars to a concrete pier cap. The bridge is to be strengthened to carry HL-93 design live load. The existing bridge was designed for H-15 live load, and the previous widening was designed for HS-15 live load. This example is based on AASHTO LRFD Bridge Design Specifications, 7th Edition.</td></tr>
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

Design Procedure

The following American Association of State Highway and Transportation Officials (AASHTO) documents were used for this example.

Publication Title	Publication Year	Publication Number	Available for Download
AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 with 2016 Interims	2014	---	No
Manual for Bridge Evaluation, 2nd Edition, 2011	2011	—	No

Summary of Design/Analysis Procedure:

Define the bridge data, material properties, section properties and existing dead load member forces. Identify the standard or specification to be used for the analysis/design along with the required design live loading and applicable load combinations and design factors.

The solution of the example follows the general steps below:

Step 1. Calculate nominal resistance of members.

Step 2. Calculate existing bridge member load rating factors.

Step 3. Design member strengthening,

Step 4. Calculate strengthened member load rating factors.

A summary will be given at the end of the example, listing the dimensions and location of the strengthening system and the final capacity provided.

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

A	=	gross area of concrete cross-section (in.^{2})
A_b	=	area of an individual bar (in.^{2})
A_{bf}	=	area of bottom flange cover plate (in.^{2})
A_e	=	effective area of weld (in.^{2})
A_i	=	area of ith component (in.^{2})
A_{pn}	=	area of projecting element of stiffener outside of flange-to-web welds but not beyond the edge of the flange (in.^{2})
A_{ps}	=	area of post-tensioning steel (in.^{2})
A_{PT}	=	area of an individual PT bar (in.^{2})
A_{req}	=	area of post-tensioning steel required (in.^{2})
A_s	=	area of flexural reinforcing steel (in.^{2})
A_{sb}	=	area of bearing stiffener (in.^{2})
A_{st}	=	Area of steel beams (in.^{2})
A_v	=	area of interface reinforcement crossing the shear plane (in.^{2})
A_w	=	area of rolled steel web (in.^{2})
$A_{w,provd}$	=	provided web area of rolled steel section (in.^{2})
A_{wreqd}	=	required web area of rolled steel section (in.^{2})
A_1	=	loaded area (in.^{2})
$A_{1,reqd}$	=	required loaded area in bearing (in.^{2})
$A_{1,provd}$	=	provided loaded area in bearing (in.^{2})
a	=	depth of equivalent rectangular stress block (in.)
b	=	width of compression face of member (in.)
b_f	=	full width of the flange (in.)
b_t	=	bearing stiffener width (in.)
b_w	=	width of the compression face of the member (in.)
C	=	factored capacity corresponding to rating factor being calculated; ratio of shear-buckling resistance to the shear specified minimum yield strength
c	=	distance from extreme compression fiber to the neutral axis (in.)
d	=	depth of steel section (in.)
d_b	=	nominal diameter of reinforcing bar (in.)
d_p	=	distance from extreme compression fiber to the centroid of the post-tensioning bars (in.)
d_s	=	depth to centroid of flexural tension steel from extreme compression fiber (in.)
E_s	=	modulus of elasticity of steel (ksi)
e	=	distance between centroid of gross cross-section and centroid of post-tensioning (in.)
F_{exx}	=	classification strength of weld metal (ksi)

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

F_{urn} = specified minimum ultimate strength of new rolled steel (ksi)
F_{yrn} = specified minimum yield strength of new rolled steel (ksi)
F_{ys} = specified minimum yield strength of bearing stiffener (ksi)
f_b = bending stress on tension face due to Service I moment (ksi)
f_{bot} = stress on bottom of section under combined effects of post-tensioning and Service I moment (ksi)
f'_{ce} = specified minimum compressive strength of concrete in existing structure (ksi)
f'_{cw} = specified minimum compressive strength of concrete in widened structure (ksi)
f_{pe} = effective stress in post-tensioning bar after losses (ksi)
f_{ps} = average stress in prestressing steel at the time for which the nominal resistance of the member is required (ksi)
f_{pun} = specified minimum tensile strength of post-tensioning rod (ksi)
f_{py} = yield strength of post-tensioning rod (ksi)
f_{sw} = stress developed in reinforcement crossing the shear plane (ksi)
f_t = allowable tensile stress in concrete (ksi)
f_{ye} = specified minimum yield stress of reinforcement in existing structure (ksi)
f_{yw} = specified minimum yield stress of reinforcement in widened structure (ksi)
h = overall depth of member (in.)
I = moment of inertia of built-up steel section (in.^{4})
I_o = moment of inertia about own centroid (in.^{4})
I_s = moment of inertia of bearing stiffener about stiffener-web interface (in.^{4})
I_x = moment of inertia of steel section about strong axis (in.^{4})
I_y = moment of inertia of steel section about weak axis (in.^{4})
K = effective length factor in plane of buckling
k = plate buckling coefficient
l = unbraced length in plane of buckling (in.)
l_d = development length (in.)
l_{db} = basic development length for straight reinforcement to which modification factors are applied to determine the development length (in.)
l_{dh} = development length of standard hook in tension as measured from critical section to outside end of hook (in.)
l_e = effective tendon length (in.); effective weld length (in.)
l_{hb} = basic development length of standard hook in tension (in.)
l_i = length of tendon between anchorages (in.)
M_{DC} = moment due to component dead load (kip-in.)
M_{DW} = moment due to wearing surface and utilities (kip-in.)
M_{LL+IM} = moment due to live load plus dynamic load allowance (kip-in.)
M_n = nominal flexural resistance of a section (kip-in.)
M_r = factored flexural resistance of a section in bending (kip-in.)
M_{SerI} = factored Service I limit state moment (kip-in.)

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

M_u	=	moment due to factored loads (kip-in.)
m	=	modification factor
N_{PT}	=	number of post-tensioning bars required
N_s	=	number of support hinges crossed by the tendon between anchorages or discretely bonded points
P_c	=	permanent net compressive force normal to shear plane (kip)
P_e	=	elastic critical buckling resistance (kip)
P_n	=	nominal axial resistance (kip)
P_o	=	equivalent nominal axial resistance (kip)
P_r	=	factor bearing resistance (kip); factored axial resistance (kip)
P_{req}	=	required post-tensioning force required to keep f_b less than f_t (kip)
P_u	=	factored axial load (kip)
Q	=	slender element reduction factor
Q_v	=	first moment of area (in.^{3})
R_{DC}	=	girder reaction due to DC dead loads (kip)
R_{DW}	=	girder reaction due to DW dead loads (kip)
R_{LL}	=	girder reaction due to live load plus dynamic load allowance (kip)
R_r	=	factored weld resistance (kip/in.)
$(R_{sb})_n$	=	nominal bearing stiffener resistance (kip)
$(R_{sb})_r$	=	factored bearing stiffener resistance (kip)
$(R_{sb})_u$	=	factored load in bearing stiffener (kip)
R_u	=	factored load applied to weld (kip/in.)
RF	=	live load rating factor
r	=	radius of gyration of bearing stiffener (in.)
S	=	section modulus of concrete pier cap (in.^{3})
S_x	=	section modulus about strong axis for rolled steel section (in.^{3})
$S_{x,provd}$	=	provided section modulus about strong axis for rolled steel section (in.^{3})
S_{xreqd}	=	required section modulus about strong axis of rolled steel section (in.^{3})
S_y	=	section modulus about weak axis for rolled steel section (in.^{3})
t_{bf}	=	thickness of bottom flange cover plate (in.)
t_e	=	effective throat of weld (in.)
t_f	=	flange thickness (in.)
t_p	=	thickness of bearing stiffener (in.)
t_w	=	thickness of web (in.)
V_{ni}	=	nominal interface shear resistance (kip)
V_p	=	plastic shear force (kip)
V_r	=	factored shear resistance (kip)

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

V_{ri}	=	factored interface shear resistance (kip)
V_u	=	factored shear force at section (kip)
y_{bar}	=	distance to centroid of built-up steel section from bottom of section (in.)
y_i	=	distance from bottom of section to centroid of i^{th} component (in.)
α_f	=	angle of reinforcement with respect to shear plane (deg.)
β_1	=	ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone.
ε_{cl}	=	compression-controlled strain limit in the extreme tension steel (in./in.)
ε_t	=	net tensile strain in extreme tension steel at nominal resistance (in./in.)
ε_{tl}	=	tension-controlled strain limit in the extreme tension steel (in./in.)
ϕ_{brg}	=	resistance factor for bearing on concrete
ϕ_b	=	resistance factor for bearing on milled surfaces
ϕ_c	=	resistance factor for axial compression
ϕ_{e2}	=	resistance factor for shear in throat of fillet weld metal
ϕ_f	=	resistance factor for flexure
ϕ_v	=	resistance factor for shear
γ_{DC}	=	load factor for component dead loads, non-composite and composite
γ_{DW}	=	load factor for future wearing surface and utility loads
γ_{LL+IM}	=	load factor for live load and dynamic load allowance
η_D	=	load modifier for ductility
η_R	=	load modifier for redundancy
η_I	=	load modifier for operational classification
η_i	=	load modifier relating to ductility, redundancy, and operational classification
μ	=	coefficient of friction

Worked Design Example

Introduction: This example involves the addition of external post-tensioning bars to a concrete pier cap. The bridge is to be strengthened to carry HL-93 design live load. The existing bridge was designed for H-15 live load, and the previous widening was designed for HS-15 live load. This example is based on AASHTO LRFD Bridge Design Specifications, 7th Edition.

Bridge Data:

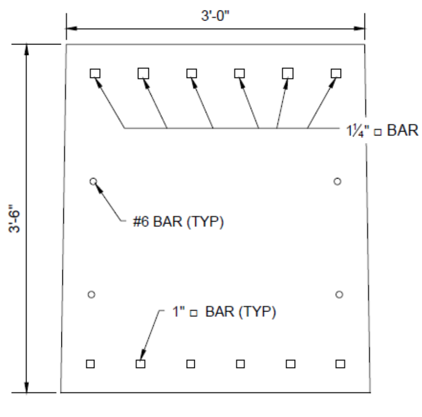
Bridge Type:	Steel Deck Truss
Span length:	Two, two-span continuous trusses and two simple span steel girder approach spans at each end equal 1783’ -1⅞” between end bearings.
Year Built:	1949, Widened in 1977
Location:	State of Tennessee

Material Properties:

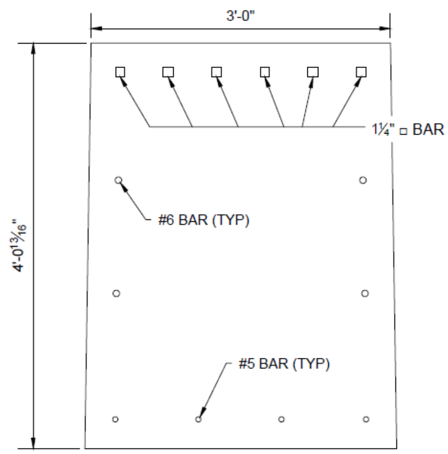
Existing Rebar Yield Strength:	f_{ye} = 33 ksi
Existing Concrete Compressive Strength:	f'_{ce} = 3 ksi
Widening Rebar Yield Strength:	f_{yw} = 60 ksi (ASTM A615)
Widening Concrete Compressive Strength:	f'_{cw} = 3 ksi
New Post-Tensioning Rod Tensile Strength:	f_{pun} = 150 ksi
New Post-Tensioning Rod Yield Strength:	$f_{py} = 0.80 f_{pu}$ = 120.0 ksi
New Rolled Steel Yield Strength:	F_{yrn} = 50 ksi (ASTM A709 Grade 50)
New Rolled Steel Ultimate Strength:	F_{urn} = 65 ksi
Modulus of Elasticity of Rolled Steel:	E_s = 29,000 ksi

Existing Member Properties:

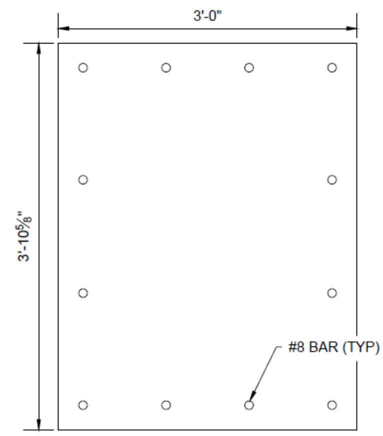
	1949 Pier Cap	**1977 Pier Cap**
Member:	3’-0” Wide RC Beam Depth varies from 3’-6” to 4’-9”	3’-0”x3’-10½” RC Beam
Reinforcement:		
Shear:	#5s @ 1’-3” o.c.	#4s @ 1’-0” o.c.
Top Flexural:	6-1¼” □ bars &	4 - #8 Bars
Bottom Flexural:	6-1” □ bars btwn columns 4 - #5 bars in overhang	4 - #8 Bars
Skin Steel:	2 - #6s on each face	2 - #8s on each face



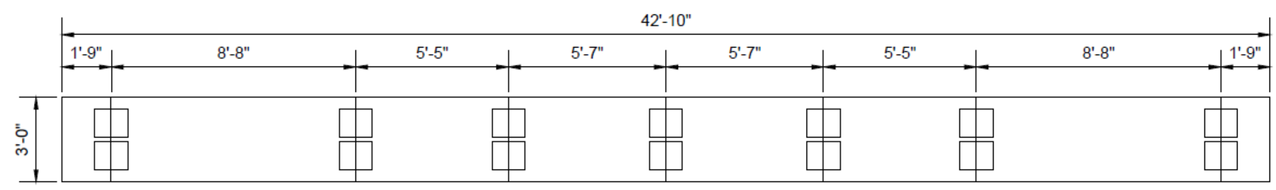
SECTION A–A



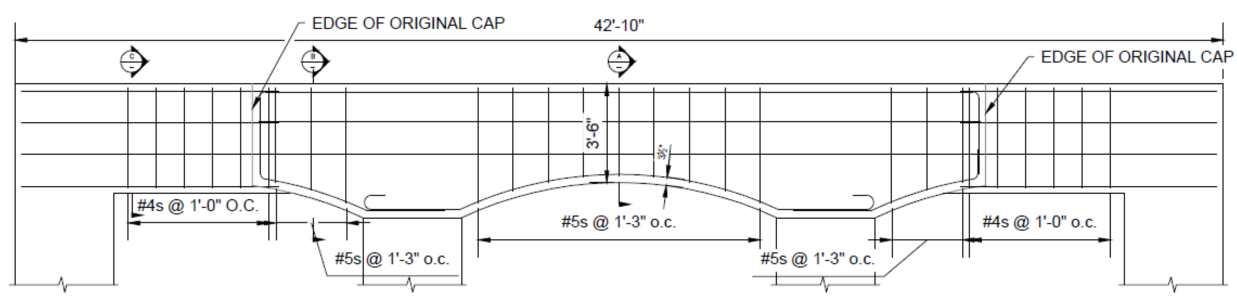
SECTION B–B



SECTION C–C



PIER CAP – PLAN VIEW



PIER CAP – ELEVATION VIEW

LRFD Factors:

For this example use: $\eta_D = 1.0$ $\eta_R = 1.0$ $\eta_I = 1.0$ therefore: $\eta_i = 1.0$

Resistance Factors	
Type of Resistance	**Factor, ϕ**
Flexure in Concrete	$\phi_f = 0.75\text{-}0.90$
Shear in Concrete	$\phi_v = 0.90$
Bearing on Concrete	$\phi_{brg} = 0.70$
Flexure in Steel	$\phi_f = 1.00$
Shear in Steel	$\phi_v = 1.00$
Bearing on Steel	$\phi_b = 1.00$
Compression on Steel	$\phi_c = 1.00$
Shear in Weld Metal	$\phi_{e2} = 1.00$

Load Combinations and Load Factors				
Limit State	**Load Factors**			
	DC	**DW**	**LL**	**IM**
Strength I	0.9/1.25	0.65/1.50	1.75	1.75
Service I	1.00	1.00	1.0	1.0

Girder Reactions:

The following table shows the girder bearing reactions for the existing condition. Note that the dead (DC and DW) and HL-93 live loads (LL+IM) are the same for the strengthened condition as for the existing condition. All reactions act in a downward direction.

Unfactored Girder Reactions				
Girder Line	**Girder Reactions**			
	DC	**DW**	**LL+IM**	
	R_{DC}	**R_{DW}**	**R_{LL} (2 Lanes)**	**R_{LL} (1 Lane)**
	(kip)	(kip)	(kip)	(kip)
E1/E7	61.3	6.5	20.3	0
E2/E6	61.4	6.5	88.1	0
E3/E5	53.6	6.5	32.5	37.9
E4	55.1	6.5	0.0	93.4

Member Forces:

The following table shows the unfactored member forces for the existing and strengthened conditions. These values were determined by applying the girder reactions to a model of the pier.

Unfactored Member Forces				
Section	**Member Forces**			
	DC (kip-in.)		**DW (kip-in.)**	**Live Load (kip-in.)**
	Cap Self-Weight	Superstructure Self-Weight		
B	-5	186	8	1681

Calculate factored moment at Section B:

$$\begin{aligned} M_u &= \eta_i\left(\gamma_{DC}M_{DC} + \gamma_{DW}M_{DW} + \gamma_{LL+IM}M_{LL+IM}\right) \\ &= 1.0\left(0.9(-5\ \text{k}\cdot\text{in.}) + 1.25(186\ \text{k}\cdot\text{in.}) + 1.5(8\ \text{k}\cdot\text{in.}) + 1.75(1680.7\text{k}\cdot\text{in})\right) \\ &= 3181.4\ \text{k}\cdot\text{in.} \end{aligned}$$

Calculate Flexural Resistance of Existing Pier Cap at Section B:

Location B (X=126.75 in from outside face of pier):

Flexural Resistance: LRFD 5.7.3

A_s = 3.00 in.^{2} f_{ye} = 33 ksi d_s = 32.15 in. ϕ_f = calculated

f'_{ce} = 3 ksi β_1 = 0.85 b_w = 36 in.

Calculate basic development length of straight #5 bar. LRFD 5.11.2.1.1

$$l_{db} = \max\left(\frac{1.25A_b f_{ye}}{\sqrt{f'_{ce}}}, 0.4d_b f_{ye}\right) = \max\left(\frac{1.25(0.31\ \text{in.}^2)33\ \text{ksi}}{\sqrt{3\ \text{ksi}}}, 0.4\times 0.625\ \text{in.}\times 33\ \text{ksi}\right)$$

= max(7.39 in., 8.25 in.) = 8.25 in.

Modification factors: LRFD 5.11.2.1.2/3

- No modification factors are applicable

Calculate development length of straight #5 bar.

l_d = 1.0 × 8.25in. = 8.25 in.

Extension past face of critical section >> 8.25 in. → Reinforcement will reach f_{ye}.

Calculate Flexural Resistance of Existing Pier Cap at Section B: (continued)

Calculate basic development length of hooked #5 bar. LRFD 5.11.2.1.1

$$l_{hb} = \frac{38.0 d_b}{\sqrt{f'_{ce}}} = \frac{38.0 \times 0.625\ \text{in.}}{\sqrt{3\ \text{ksi}}} = 13.7\ \text{in.}$$

Modification factors: LRFD 5.11.2.1.2/3

0.7 – side cover is not less than 2.5 in. and cover on bar extension of 90° hook is not less than 2.0 in. for #11 bars and smaller

Calculate development length of hooked #11 bar.

(M-) l_{dh} = max(0.7 × 13.7 in., 6 in., 8 × 0.625 in.) = 9.6 in.

Extension past critical section = 22.25 in. > l_d = 9.6 in. → Reinforcement will reach f_{ye}.

Calculate depth of compression block (neglect 1¼” □ bars):LRFD 5.7.3.1

$$c = \frac{A_s f_{ye}}{0.85 f'_{ce} \beta_1 b_w} = \frac{3.00\ \text{in.}^2 \times 33\ \text{ksi}}{0.85 \times 3\ \text{ksi} \times 0.85 \times 36\ \text{in.}} = 1.27\ \text{in.}$$

(1 ¼” □ bars are on tension side of neutral axis; ignore contribution)

$a = \beta_1 c$ = 0.85 × 1.27 in. = 1.08 in.

Determine strain in tension steel using similar triangles. LRFD 5.7.2.1

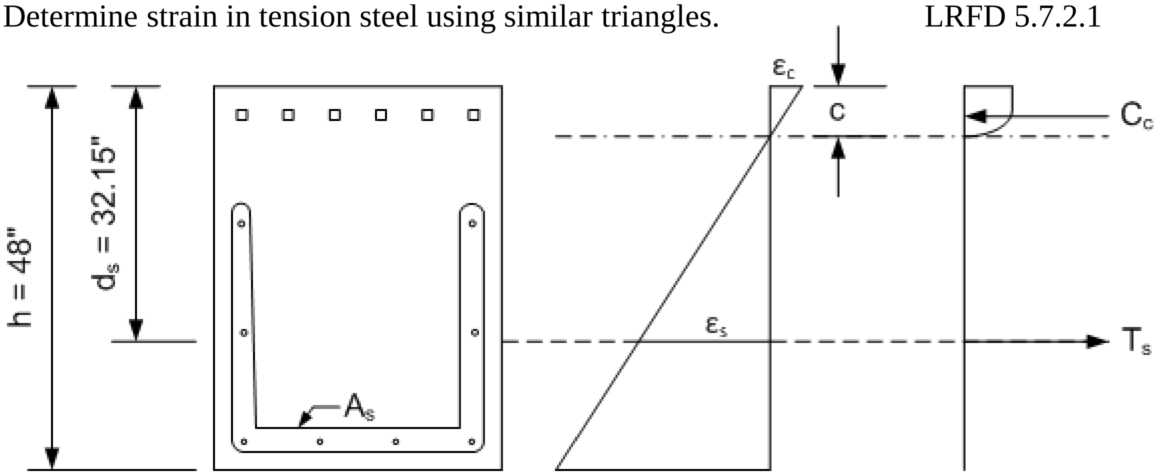


Figure 2 – Strain Compatibility

$$\varepsilon_t = \frac{0.003(d_s - c)}{c} = \frac{0.003(32.15\ \text{in.} - 1.27\ \text{in.})}{1.27\ \text{in.}} = 0.073$$

Determine ϕ_f based on strain in tensile steel. LRFD 5.5.4.2.1

$$0.75 \le \phi_f = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} = 0.75 + \frac{0.15(0.073 - 0.002)}{(0.005 - 0.002)} = 4.30 \le 0.9 \Rightarrow \phi_f = 0.9$$

where: ε_{cl} = 0.002

ε_{tl} = 0.005

Calculate Moment Capacity (neglecting compression steel) LRFD 5.7.3.2.2

$$M_r = \phi_f A_s F_{ye}\left(d_s - \frac{a}{2}\right) = 0.9\left[3.00\text{ in.}^2 \times 33\text{ ksi} \times \left(32.15\text{ in.} - \frac{1.08\text{ in.}}{2}\right)\right]$$

$= 2816$ k-in.

Summary of Existing Member Factored Resistance and Unfactored Forces:

Existing Member Factored Resistances and Unfactored Forces					
Location	Flexural Resistance	Unfactored Moments			
		DC (Self-Weight)	DC (Superstructure Reactions)	DW	LL+IM
	(k-in.)	(k-in.)	(k-in.)	(k-in.)	(k-in.)
B	2816	-5	186	8	1681

Calculate Demand to Capacity (D/C) Ratios

Location B:

$$\frac{M_u}{M_r} = \frac{1.0(0.9(-5\text{ k}\cdot\text{in.}) + 1.25(186\text{ k}\cdot\text{in.}) + 1.5(8\text{ k}\cdot\text{in.}) + 1.75(1681\text{ k}\cdot\text{in.}))}{2816\text{ k}\cdot\text{in.}} = 1.13 \geq 1.0 \quad \text{No Good}$$

Calculate Rating Factor for Existing Member: LRFD 5.5.4.2.1

Rating factors are very typical values used to evaluate existing bridge members. The rating factor, RF, is the ratio of the design live load vehicle's effect that the member can safely carry for the investigated limit state to the actual design live load effects. The AASHTO Manual for Bridge Evaluation (LRFR), 2nd Edition with 2016 Interims, June 2015, is used for the load rating equations.

The general load rating equation is as follows (simplified LRFR Eqn. 6A.4.2.1-1):

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_{LL})(LL + IM)}$$

where:

γ_{DC}, γ_{DW}, and γ_{LL} are the LRFD load factors

DC, DW, and LL+I are the force effects

C is the factored member capacity

$$RF = \frac{2816\text{ k}\cdot\text{in.} - 0.9(-5\text{ k}\cdot\text{in.}) - 1.25(186\text{ k}\cdot\text{in.}) - 1.5(8\text{ k}\cdot\text{in.})}{1.75(1681\text{ k}\cdot\text{in.})} = 0.88$$

A Rating Factor less than 1.0 indicates the member has insufficient capacity to resist the full factored design live load effect.

The pier cap will require strengthening to obtain a Rating Factor greater than 1.0 for the HL-93 design live loading for the Strength-I Limit State.

Design Member Strengthening:

Assume strengthening consists of external post-tensioning rods with steel anchorage assemblies on the exterior faces of the outside columns.

Factors to consider:

- Increasing the amount of flexural steel in the bottom of the cap is very difficult without demolishing a significant portion of the existing cap. Developing full capacity of new rebar would require significant embedment lengths.
- Using internal post-tensioning would require coring through the existing bent cap.
- Structure was to remain open while repairs were being completed.

Pier Cap Strengthening:

Use post-tensioning rods with steel anchorage assemblies to bracket pier and increase flexural capacity.

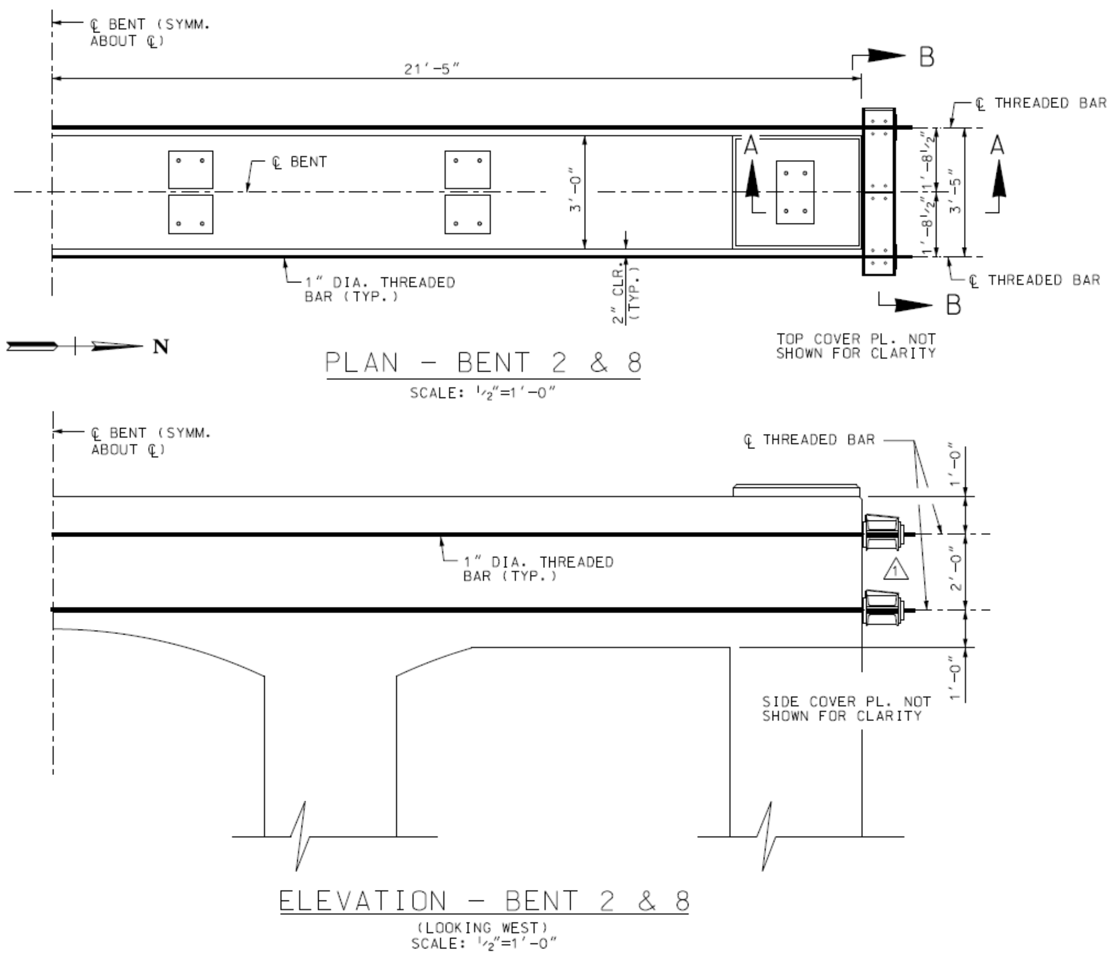


Figure 3 – Strengthening Details

Pier Cap Strengthening: (continued)

Determine Required Post-Tensioning Force to Limit Stresses at Service I Limit State:

Calculate Service I Moment:

$$M_{SerI} = (\gamma_{DC}M_{DC} + \gamma_{DW}M_{DW} + \gamma_{LL+IM}M_{LL+IM})$$
$$= 1.0(-5\ \text{k}\cdot\text{in.}) + 1.0(186\ \text{k}\cdot\text{in.}) + 1.0(8\ \text{k}\cdot\text{in.}) + 1.0(1681\ \text{k}\cdot\text{in.})$$
$$= 1870\ \text{k-in.}$$

Calculate section modulus of pier cap: $S = \dfrac{bh^2}{6} = \dfrac{36\ \text{in.}(48\ \text{in.})^2}{6} = 13824\ \text{in.}^3$

Allowable tensile stress in concrete: $f_t = 0$ ksi LRFD Table 5.9.4.1.2

Actual tensile stress in concrete: $f_b = \dfrac{M_{SerI}}{S} = \dfrac{1870\ \text{k}\cdot\text{in.}}{13824\ \text{in.}^3} = 0.135\ \text{ksi}$

Calculate amount of post-tensioning required to prevent tension in bottom of pier cap:

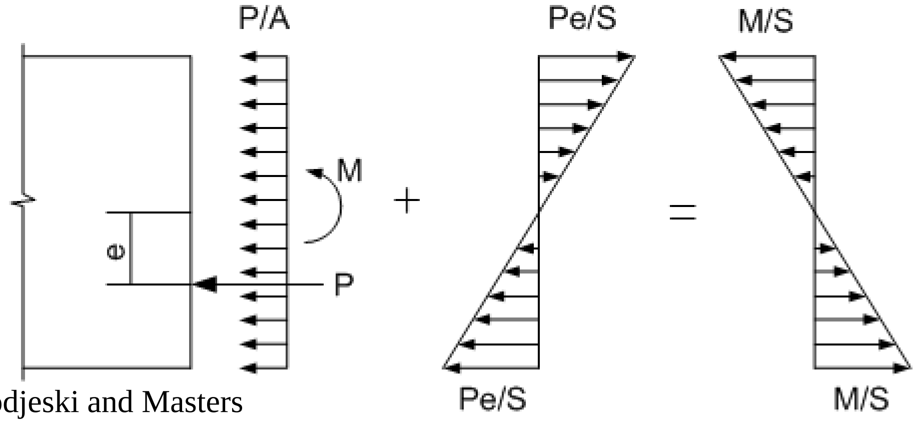


Figure 4 – Stresses on Section due to Applied Loads and Post-Tensioning

To have zero tension on the bottom of the cap, P/A + Pe/S = M/S. The post-tensioning is assumed to be applied at the centroid of the cap in this example; therefore, e = 0 in. If the post-tensioning is applied eccentrically, e would be non-zero and the Pe/S term would remain.

$$\frac{P_{req}}{A} + \frac{P_{req}e}{S} = f_b - f_t$$

Solving the above equation for P_{req}:

$$P_{req} = \frac{f_b - f_t}{\dfrac{1}{A} + \dfrac{e}{S}} = \frac{0.135\ \text{ksi} - 0\ \text{ksi}}{\dfrac{1}{36\ \text{in.}(48\ \text{in.})} + \dfrac{0\ \text{in.}}{13824\ \text{in.}^3}} = 233.3\ \text{kips}$$

Pier Cap Strengthening: Determine Amount of Post-Tensioning Required (continued)

P_{req} = 233.3 kips

Determine required area of post-tensioning:

$$A_{req} = \frac{P_{req}}{0.80 f_{py}} = \frac{233 \text{ kips}}{0.80(0.80 \times 150 \text{ ksi})} = 2.43 \text{ in.}^2$$

Area of 1" diameter PT bar, A_{PT} = 0.85 in.^{2}

Number of PT bars required, $N_{PT} = A_{req}/A_{PT}$ = 2.43 in.^{2}/0.85 in.^{2} = 2.86 → need a minimum of 3 – 1"ϕ PT bars, use 2 sets of 2-1" ϕ PT bars stressed to 0.7f_{pun} to bracket pier cap.

Check stress on bottom of section with 4-1" ϕ PT bars with effective stress of 96 ksi.

$$f_{bot} = \frac{-P}{A} + \frac{M}{S} = \frac{-4(0.85 \text{ in.}^2)(96 \text{ ksi})}{(48 \text{ in.})(36 \text{ in.})} + 0.135 \text{ ksi} = -0.054 \text{ ksi} \qquad \text{(54 psi in compression)}$$

Section is fully compressed at the Service I Limit State.

Calculate Demand to Capacity Ratio:

$$\frac{D}{C} = \frac{M_{SerI}/S}{P/A} = \frac{1870 \text{ k}\cdot\text{in.}/13824 \text{ in.}^3}{4(0.85 \text{ in.}^2)(96 \text{ ksi})/(36 \text{ in.})(48 \text{ in.})} = 0.72 \leq 1.00 \quad \text{OK}$$

Calculate Rating Factor for Strengthened Member:

$$RF = \frac{0 \text{ ksi} - \frac{-5 \text{ k}\cdot\text{in}}{13824 \text{ in.}^3} - \frac{186 \text{ k}\cdot\text{in}}{13824 \text{ in.}^3} - \frac{8 \text{ k}\cdot\text{in}}{13824 \text{ in.}^3} + \frac{326 \text{ kips}}{1728 \text{ in.}^2}}{\frac{1681 \text{ k}\cdot\text{in}}{13824 \text{ in.}^3}} = 1.44 > 1.00 \quad \text{OK}$$

Check Ultimate Moment Capacity:

Flexural Resistance: LRFD 5.7.3

A_s = 3.00 in.^{2}	f_{ye} = 33 ksi	d_s = 32.15 in.	ϕ_f = calculated
A_{ps} = 3.40 in.^{2}	f_{pun} = 150 ksi	f_{pe} = 96 ksi	d_p = 24 in.
f'_{ce} = 3 ksi	β_1 = 0.85	b_w = 36 in.	

Determine development length of existing reinforcement – from existing condition calculations, we know that the reinforcement is able to fully develop prior to the point at which it is needed, therefore $f_s = f_y$ in ultimate moment capacity calculations.

Determine depth of compression block, c. LRFD 5.7.3.1.2

$$c = \frac{A_{ps} f_{ps} + A_s f_y}{0.85 f'_{ce} \beta_1 b_w} \qquad \text{LRFD Eq. 5.7.3.1.2-4}$$

$$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{l_e}\right) \leq f_{py} \qquad \text{LRFD Eq. 5.7.3.1.2-1}$$

Pier Cap Strengthening: Ultimate Moment Check (continued)

$$l_e = \frac{2l_i}{2+N_s} = \frac{2(540.25\text{ in.})}{2+0} = 540.25\text{ in.}$$ LRFD Eq. 5.7.3.1.2-2

Where: l_i = length of tendon between anchorages (in.) = 540.25 in.

l_e = effective tendon length (in.)

N_s = number of support hinges crossed by the tendon between anchorages or discretely bonded points = 0

f_{py} = yield strength of prestressing steel (ksi)

f_{pe} = effective stress in prestressing steel after all losses (ksi)

The depth of the compression block, c, and the stress in the PT bars at ultimate, f_{ps}, are related, therefore the equations must be solved simultaneously or through an iterative procedure. For the first iteration, assume that $f_{ps} = f_{pe}$ + 15 ksi as recommended in LRFD C5.7.3.1.2.

f_{ps} = 96 ksi + 15 ksi = 111 ksi

$$c = \frac{A_{ps}f_{ps} + A_s f_y}{0.85 f'_c \beta_1 b_w} = \frac{3.40\text{ in.}^2(111\text{ ksi}) + 3.00\text{ in.}^2(33\text{ ksi})}{0.85(3\text{ ksi})(0.85)(36\text{ in.})} = 6.1\text{ in.}$$

Substituting c back into the equation for f_{ps}:

$$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{l_e}\right) = 96\text{ ksi} + 900\left(\frac{24\text{ in.} - 6.1\text{ in.}}{540.25\text{ in.}}\right) = 125.8\text{ ksi} \le f_{py} = 120\text{ ksi}$$

For the second iteration, use f_{ps} = 120 ksi.

$$c = \frac{A_{ps}f_{ps} + A_s f_y}{0.85 f'_c \beta_1 b_w} = \frac{3.40\text{ in.}^2(120\text{ ksi}) + 3.00\text{ in.}^2(33\text{ ksi})}{0.85(3\text{ ksi})(0.85)(36\text{ in.})} = 6.5\text{ in.}$$

Substituting c back into the equation for f_{ps}:

$$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{l_e}\right) = 96\text{ ksi} + 900\left(\frac{24\text{ in.} - 6.5\text{ in.}}{540.25\text{ in.}}\right) = 125.2\text{ ksi} \le f_{py} = 120\text{ ksi}$$

The solution has converged with f_{ps} = 120 ksi and c = 6.5 in.

$a = \beta_1 c$ = 0.85(6.5 in.) = 5.53 in.

Pier Cap Strengthening: Ultimate Moment Check (continued)

Calculate strain in extreme tensile steel.

$$\varepsilon_t = \frac{0.003(d_s - c)}{c} = \frac{0.003(32.15\text{ in.} - 6.5\text{ in.})}{6.5\text{ in.}} = 0.012$$

Determine ϕ_f based on strain in tensile steel. LRFD 5.5.4.2.1

$$0.75 \le \phi_f = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 0.9$$

where: $\varepsilon_{cl} = 0.002$

$\varepsilon_{tl} = 0.005$

$$\phi_f = 0.75 + \frac{0.15(0.012 - 0.002)}{(0.005 - 0.002)} = 1.25 \le 0.9 \Rightarrow \phi_f = 0.9$$

Calculate Moment Capacity (neglecting compression steel) LRFD 5.7.3.2.2

$$M_r = \phi_f\left(A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_{ye}\left(d_s - \frac{a}{2}\right)\right)$$

$$= 0.9\left[3.40\text{ in.}^2(120\text{ ksi})\left(24\text{ in.} - \frac{5.53\text{ in.}}{2}\right) + 3.00\text{ in.}^2(33\text{ ksi})\left(32.15\text{ in.} - \frac{5.53\text{ in.}}{2}\right)\right]$$

$= 10416$ k-in.

Calculate Demand to Capacity (D/C) Ratio:

$$\frac{M_u}{M_r} = \frac{0.9(-5\text{ k}\cdot\text{in}) + 1.25(186\text{ k}\cdot\text{in}) + 1.5(8\text{ k}\cdot\text{in}) + 1.75(1681\text{ k}\cdot\text{in})}{10416\text{ k}\cdot\text{in}} = 0.31 \le 1.0\ \ OK$$

Calculate Rating Factor for Strengthened Member:

$$RF = \frac{10416\text{ k}\cdot\text{in.} - 0.9(-5\text{ k}\cdot\text{in.}) - 1.25(186\text{ k}\cdot\text{in.}) - 1.5(8\text{ k}\cdot\text{in.})}{1.75(1681\text{ k}\cdot\text{in.})}$$

= 3.46 (HL-93 at Strength-I Limit State)

Therefore, strengthening of pier cap is sufficient.

The next step is to design the anchorage device for the post-tensioning bars and check the pier cap to ensure that the corner won’t spall when the post-tensioning bars are stressed.

Pier Cap Strengthening: Evaluate Shear Friction Capacity of Pier Cap Corner: LRFD 5.8.4

Consider failure plane to start at bottom edge of bearing plate at top anchor location and propagate at 45° angle:

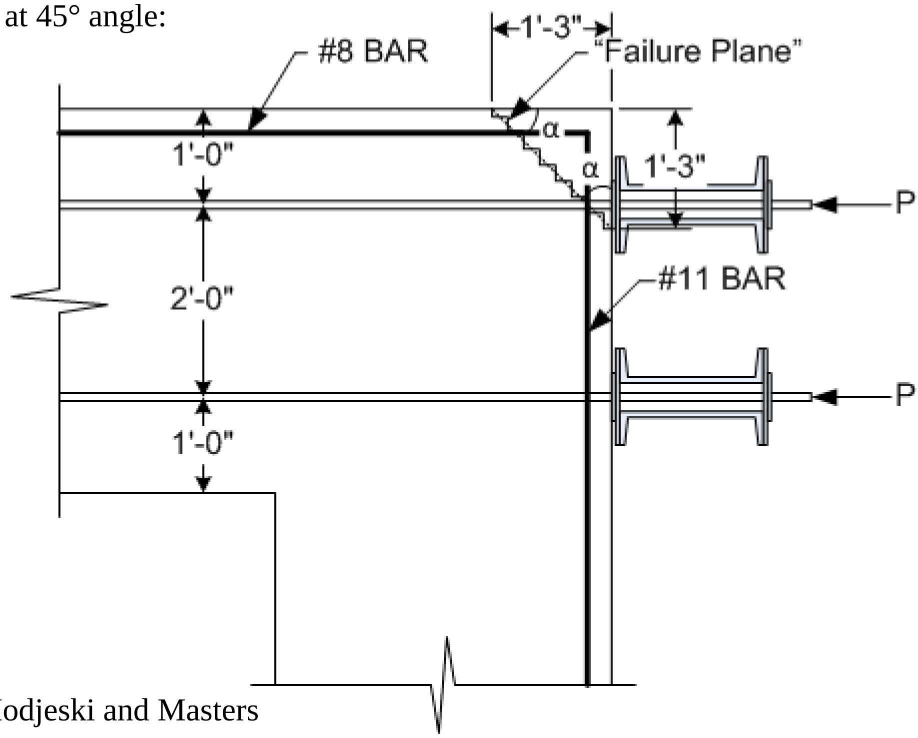


Figure 5 – Shear Friction Failure Mode

Calculate basic development length of straight #8 bars:

$$l_{db} = \max\left(\frac{1.25A_b f_{yw}}{\sqrt{f'_{cw}}}, 0.4d_b f_{yw}\right) = \max\left(\frac{1.25(0.79\text{ in.}^2)60\text{ ksi}}{\sqrt{3\text{ ksi}}}, 0.4\times 1.0\text{ in.}\times 60\text{ ksi}\right)$$

= max(34.2 in., 24 in.) = 34.2 in.

Modification factors: LRFD 5.11.2.1.2/3

- 1.4 – for top horizontal steel with more than 12" of fresh concrete cast below the reinforcement

Calculate development length of straight #8 bar.

l_d = 1.4 × 34.2in. = 47.9 in.

Extension past failure plane = 9 in. < l_d = 47.9 in. → Reinforcement will not reach f_{yw}.

% developed = 9 in./47.9 in. = 0.19

Pier Cap Strengthening : Evaluate Shear Friction Capacity (continued)

Calculate basic development length of straight #11 bars:

$$l_{db} = \max\left(\frac{1.25A_b f_{yw}}{\sqrt{f'_{cw}}}, 0.4d_b f_{yw}\right) = \max\left(\frac{1.25(1.56\text{ in.}^2)60\text{ ksi}}{\sqrt{3\text{ ksi}}}, 0.4\times1.41\text{ in.}\times60\text{ ksi}\right)$$

= max(67.6 in., 33.8 in.) = 67.6 in.

Modification factors: LRFD 5.11.2.1.2/3

- No modification factors are applicable.

Calculate development length of straight #11 bar.

l_d = 1.0 × 67.6 in. = 67.6 in.

Extension past failure plane = 9 in. < l_d = 67.6 in. → Reinforcement will not reach f_{yw}.

% developed = 9 in./67.6 in. = 0.13

Calculate interface shear resistance (neglecting concrete component):

$$V_{ni} = \sum \mu A_v f_{sw} + \mu P_c$$ LRFD Eq. 5.8.4.1-3

where:
- A_{v1} = area of interface reinforcement crossing shear plane (in.^{2})
  = 4(0.79 in.^{2}) = 3.16 in.^{2} (4-#8 Bars)
- f_{sw1} = developed stress in reinforcement across shear plane (ksi)
  = 0.19(60 ksi) = 11.3 ksi
- A_{v2} = area of interface reinforcement crossing shear plane (in.2)
  = 4(1.56 in.^{2}) = 6.24 in.^{2} (4-#11 Bars)
- f_{sw2} = developed stress in reinforcement across shear plane (ksi)
  = 0.13(60 ksi) = 7.8 ksi
- α_f = angle of reinforcement with respect to shear plane (deg.)
  = 45 deg.
- μ = friction factor specified in LRFD 5.8.4.3 for normal-weight monolithic concrete placement
  = 1.4
- P_c = permanent net compressive force normal to shear plane (kips)
  = (178.6 kips) sin(45 deg.) = 126.3 kips

$$V_{ni} = 1.4\left(3.16\text{ in.}^2(11.3\text{ ksi}) + 6.24\text{ in.}^2(7.8\text{ ksi}) + 126.3\text{ kip}\right) = 295\text{ kip}$$

Pier Cap Strengthening: Evaluate Shear Friction Capacity (continued):

Determine factored resistance:

$\phi_v \quad = \quad 0.9$

$$V_{ri} = \phi_v V_{ni} = 0.9(295\text{ kip}) = 265\text{ kip} \;\; > 126.3\text{ kip}$$

Calculate Demand to Capacity (D/C) Ratio:

$$\frac{V_u}{V_{ri}} = \frac{1.00(178.6\text{ kip} \times \cos(45\text{deg.}))}{265\text{ kip}} = 0.48 \le 1.00 \;\; \text{OK}$$

Calculate Rating Factor for Strengthened Member:

A rating factor cannot be calculated for this check as there is no live load applied. The demand to capacity ratio is less than 1.0; therefore, the member is sufficient to carry the applied loads.

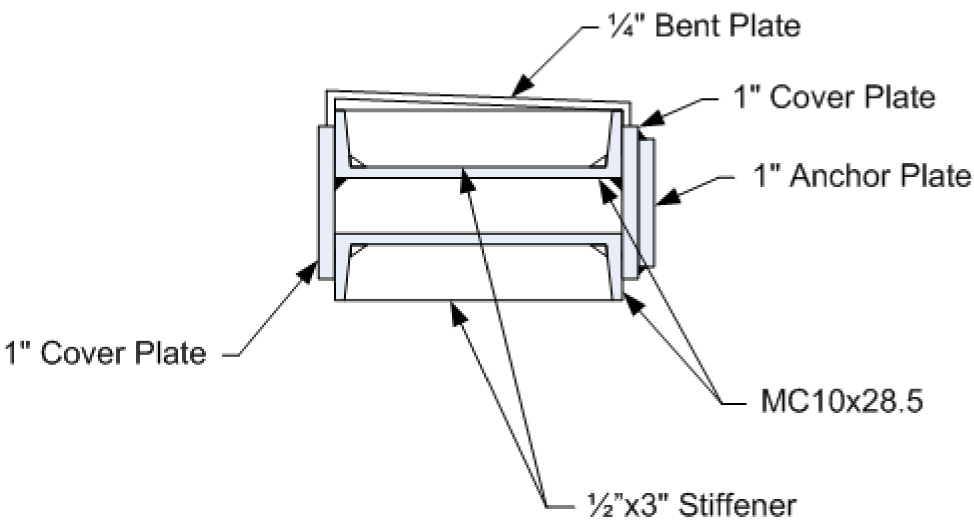


Figure 6 – PT Anchorage Beam

Determine Maximum Shear and Moment in PT Anchorage:

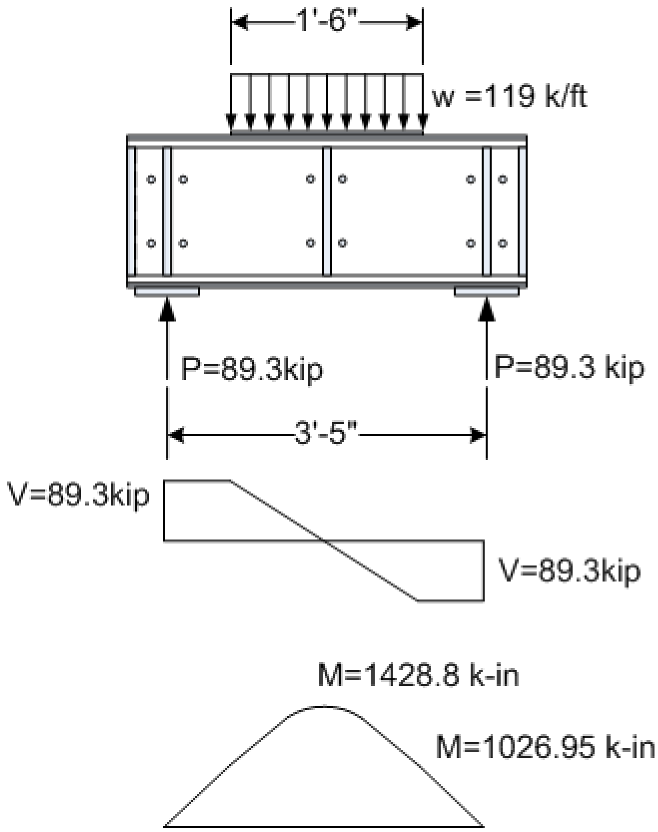


Figure 7 - Applied Loads, Shear and Moment Diagrams

Determine Minimum Section Properties to resist design loads:

Flexure:

$M_u \leq \phi_f M_n = \phi_f S_x F_{yrn}$

Solving for S_{xreqd},

$$S_{xreqd} \geq \frac{M_u}{\phi_f F_{yrn}} = \frac{1.25(1428.8\ \text{k} \cdot \text{in.})}{1.0(50\ \text{ksi})} = 35.7\ \text{in.}^3$$

Shear:

$V_u \leq \phi_v V_n = \phi_v C V_p = \phi_v C(0.58 F_{yrn} A_w)$

Solving for A_{wreqd},

$$A_{wreqd} \geq \frac{V_u}{\phi_v C(0.58 F_{yrn})} = \frac{1.25(89.3\ \text{kip})}{1.0(1.0)(0.58)(50\ \text{ksi})} = 3.85\ \text{in.}^2$$

Pier Cap Strengthening: Design Steel Beam for PT Anchorage (continued):

Try 2-MC10x28.5 channels.

MC10x28.5 Section Properties:

A_{st} = 8.37 in.^{2} d = 10.0 in. t_w = 0.425 in. b_f = 3.95 in. t_f = 0.575 in.

I_x = 126 in.^{4} S_x = 25.3 in.^{3} I_y = 11.3 in.^{4} S_y = 3.99 in.^{3}

Provided Section Properties:

$S_{x,provd}$ = 2(25.3 in.^{3}) = 50.6 in.^{3} > S_{xreqd} = 35.7 in.^{3} → OK

$A_{w,provd}$ = 2(10.0 in.)(0.425 in.) = 8.50 in.^{2} > 3.85 in.^{2} → OK

Calculate nominal capacities for flexure and shear:

$M_r = \phi_f S_{x,provd} F_{yrn}$ = (1.00)(50.6 in.^{3})(50 ksi) = 2530 k-in.

$V_r = \phi_v (0.58) A_{w,provd} C F_{yrn}$ = (1.00)(0.58)(8.50 in.^{2})(1.00)(50 ksi) = 246.5 kips

Calculate Demand to Capacity Ratios:

$$\frac{M_u}{M_r} = \frac{1.25(1428.8\ \mathrm{k \cdot in})}{2530\ \mathrm{k \cdot in}} = 0.71 \le 1.00\ \ \mathrm{OK}$$

$$\frac{V_u}{V_r} = \frac{1.25(89.3\ \mathrm{kip})}{246.5\ \mathrm{kip}} = 0.46 \le 1.00\ \ \mathrm{OK}$$

Determine Required Bearing Area to Resist PT Force:

$P_u \le \phi_{brg} P_n = \phi_{brg}(0.85 f'_{cw} A_1 m)$

Solving for A_1, conservatively assuming that m = 1.0:

$$A_{1reqd} \ge \frac{P_u}{\phi_{brg} m (0.85 f'_{cw})} = \frac{1.25(178.6\ \mathrm{kip})}{0.7(1.0)(0.85)(3\ \mathrm{ksi})} = 125.1\ \mathrm{in.}^2$$

Provided Bearing Area:

$A_{1,provd}$ = 18 in. × 8 in. = 144 in.^{2}

Calculate nominal bearing capacity:

$P_r = \phi_{brg}(0.85 f'_{cw} A_{1,provd} m)$ = (0.70)(0.85)(3 ksi)(144 in.^{2})(1.0) = 257 kip

Calculate Demand to Capacity Ratios:

$$\frac{P_u}{P_r} = \frac{1.25(178.6\ \mathrm{kip})}{257\ \mathrm{kip}} = 0.87 \le 1.00\ \ \mathrm{OK}$$

Pier Cap Strengthening: Design Steel Beam for PT Anchorage (continued):

Design Bearing Stiffeners at PT Rod Locations (neglect channel web when determining effective section):

Bearing Stiffener Properties (note that I_s is calculated with respect to the face of the web):
b_t = 3 in. t_p = 0.5625 in. I_s = 5.07 in.^{4} A_{sb} = 1.69 in.^{2} r_s = 1.73 in.

Check Projecting Width of Bearing Stiffener: LRFD 6.10.11.2.2-1

$$b_t = 3.00\text{ in.} \le 0.48 t_p \sqrt{\frac{E_s}{F_{ys}}} = 0.48(0.5625\text{ in.})\sqrt{\frac{29000\text{ ksi}}{50\text{ ksi}}} = 6.50\text{ in.}$$ Projecting Width is OK.

Check Bearing Resistance of Stiffener on Flange: LRFD Eq. 6.10.11.2.3-2

$(R_{sb})_r = \phi_b(R_{sb})_n = \phi_b(1.4A_{pn}F_{ys})$ = (1.0)(1.4)(0.84 in.^{2})(50 ksi) = 59.1 kip > $(R_{sb})_u$ = 55.8 kip

where: ϕ_b = resistance factor for bearing = 1.0

A_{pn} = area of projecting element of stiffener outside of flange-to-web welds but not beyond the edge of the flange (in.^{2}) = 0.84 in.^{2}

F_{ys} = yield strength of stiffener (ksi) = 50 ksi

$(R_{sb})_u$= 1.25(89.3 kip/2) = 55.8 kip

Calculate Demand to Capacity Ratios:

$$\frac{(R_{sb})_u}{(R_{sb})_r} = \frac{1.25(44.7\text{ kip})}{59.1\, kip} = 0.94 \le 1.00 \text{ OK}$$

Calculate Axial Resistance of Bearing Stiffener

$P_r = \phi_c P_n$ LRFD Eq. 6.9.2.1-1

If $\frac{P_e}{P_o} \ge 0.44$, then: $P_n = \left[0.658^{\left(\frac{P_e}{P_o}\right)}\right]P_o$ LRFD Eq. 6.9.4.1.1-1

If $\frac{P_e}{P_o} < 0.44$, then: $P_n = 0.877P_e$ LRFD Eq. 6.9.4.1.1-2

Where: P_o = equivalent nominal resistance (kips) = QF_yA_{sb} = 1.0(50 ksi)(1.69 in.^{2}) = 84.5 kips

Q = slender element reduction factor = 1.0 for bearing stiffeners

P_e = elastic critical buckling resistance (kips)

Pier Cap Strengthening: Design Steel Beam for PT Anchorage (continued):

$$P_e = \frac{\pi^2 E_s}{\left(\frac{Kl}{r_s}\right)^2} A_{sb} = \frac{\pi^2 (29000 \text{ ksi})}{\left(\frac{0.75(10.0 \text{ in.})}{1.73 \text{ in.}}\right)^2}\left(1.69 \text{ in.}^2\right) = 25737 \text{ kips}$$ LRFD Eq. 6.9.4.1.2-1

Where: K = effective length factor in plane of buckling = 0.75 (per LRFD 6.10.11.2.4a)

l = unbraced length in plane of buckling (in.) = 10.0 in.

r_s = radius of gyration about the axis normal to the plane of buckling (in.)

$$\frac{P_e}{P_o} = \frac{25737 \; kips}{84.5 \; kips} = 305 >> 0.44$$ Use LRFD Eq. 6.9.4.1.1-1 to calculate P_n.

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o = \left[0.658^{\left(\frac{84.5 \text{ kips}}{25737 \text{ kips}}\right)}\right](84.5 \text{ kips}) = 84.3 \text{ kips}$$

$P_r = \phi_c P_n = 0.9(84.3 \text{ kips}) = 75.9 \text{ kips}$ (per bearing stiffener)

Check slenderness of bearing stiffener: LRFD Eq. 6.9.4.2-1

$$\frac{b_t}{t_p} \le k\sqrt{\frac{E_s}{F_{y_s}}} \Rightarrow b_t \le kt_p\sqrt{\frac{E_s}{F_{ys}}} = 0.45(0.5625 \text{ in.})\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 6.1 \text{ in.} > b = 3.0 \text{ in.} \rightarrow \text{OK}$$

Calculate Demand to Capacity Ratio:

$$\frac{P_u}{P_r} = \frac{1.25(89.3 \text{ kips})}{2(75.9 \text{ kips})} = 0.74 < 1.00 \quad \text{OK}$$

Pier Cap Strengthening: Design Steel Beam for PT Anchorage (continued):

Calculate Section Properties of Built-Up Section:

Section Properties of Built-Up Section						
Component	Area (in.^{2})	y_i (in.)	$A_i y_i$ (in.^{3})	A $(y_{bar}-y_i)^2$ (in.^{4})	I_o (in.^{4})	$I = I_o + A(y_{bar}-y_i)^2$ (in.^{4})
1”x8” Cover Plate	8.00	11.5	92.0	242.00	0.67	242.67
2-MC10x28.5	16.74	6	100.44	0.00	252.00	252.00
1”x8” Cover Plate	8.00	0.5	4.00	242.00	0.67	242.67
Total =	32.74		196.44			737.34

$$y_{bar} = \frac{\sum A_i y_i}{\sum A_i} = \frac{196.44 \text{ in.}^3}{32.74 \text{ in.}^2} = 6 \text{ in.} \qquad Q_v = A_{bf}\left(y_{bar} - \frac{t_{bf}}{2}\right) = 8.00 \text{ in.}^2\left(6.00 \text{ in.} - \frac{1.00 \text{ in.}}{2}\right) = 44.0 \text{ in.}^3$$

Calculate required weld strength:

$$R_u = \frac{V_u Q_v}{I} = \frac{1.25(89.3 \text{ kips})(44.0 \text{ in.}^3)}{737.3 \text{ in.}^4} = 6.7 \text{ k/in.}$$

Factored Strength of Fillet Weld: LRFD 6.13.3.2.4b

$R_r = 0.6\phi_{e2}F_{exx}A_e$ LRFD Eq. 6.13.3.2.4b-1

ϕ_{e2} = resistance factor for weld metal = 0.80 LRFD 6.5.4.2

F_{exx} = classification strength of weld metal (ksi) = 70 ksi

A_e = effective area of weld (in.^{2}) = $t_e l_e$

t_e = effective throat (in.)

l_e = effective weld length (in.)

Minimum weld size = 5/16” LRFD Table 6.13.3.4-1

t_e = 0.3125 in. × cos(45 deg.) = 0.22 in.

A_e = 0.22 in. × 1.00 in. = 0.22 in.^{2}/in.

R_r = 0.6(0.8)(70 ksi)(0.22 in.^{2}/in.) = 7.39 kips/in./weld

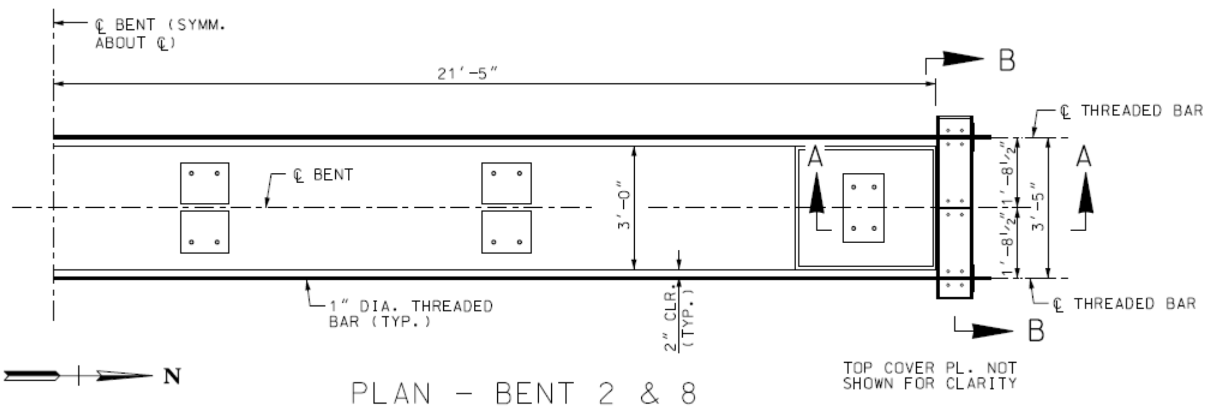
= 7.3 kips/in./weld × 2 welds = 14.8 kips/in.

Pier Cap Strengthening: Design Steel Beam for PT Anchorage (continued):

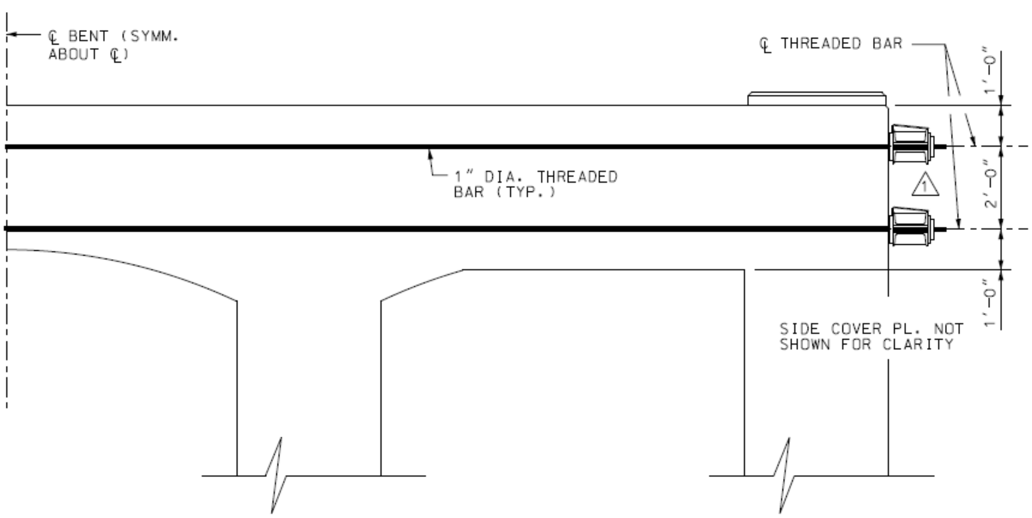
Calculate Demand to Capacity Ratio:

$$\frac{R_u}{R_r} = \frac{6.7 \text{ kip/in.}}{14.8 \text{ kip/in.}} = 0.45 < 1.00 \quad \text{OK}$$

Strengthening Design Sketch:

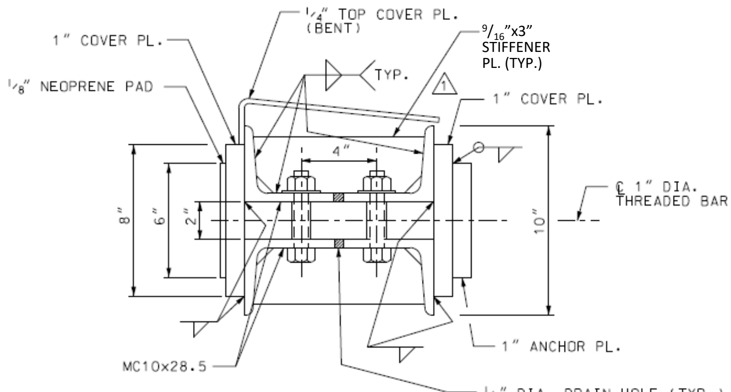


PLAN – BENT 2 & 8
SCALE: ½"=1'-0"

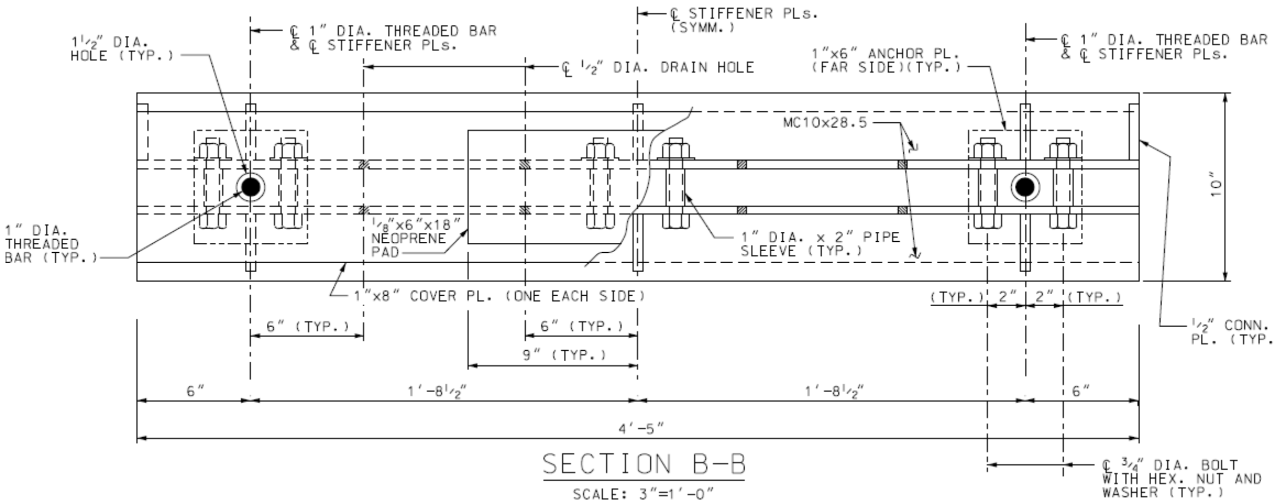


ELEVATION – BENT 2 & 8
(LOOKING WEST)
SCALE: ½"=1'-0"

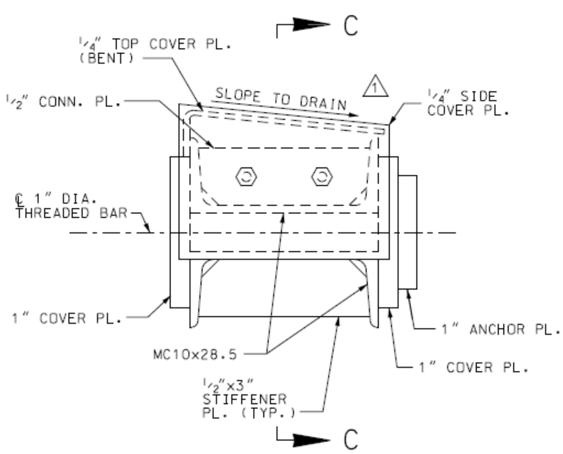
Strengthening Design Sketch: (continued)



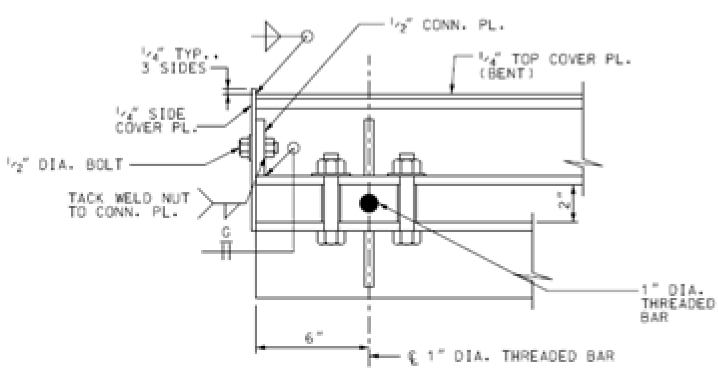
SECTION A–A
SCALE: 3"=1'-0"



SECTION B–B
SCALE: 3"=1'-0"



COVER PLATE DETAILS
SCALE: 3"=1'-0"



SECTION C–C
SCALE: 3"=1'-0"

Summary

There are several factors that must be considered when designing the concrete pier cap strengthening.

The capacity of the existing cap is limited by the amount of reinforcement as well as the reinforcement strength. The amount of reinforcement and it's strength cannot be increased without demolishing a portion of the existing cap and rebuilding it or at a minimum, drilling through the length of the cap and embedding new reinforcement. Drilling through the length of the cap may be feasible (depending on the length) but placing the reinforcement at the location where it will be most effective is dependent upon the profile of the cap. Additionally, when drilling through the cap the existing reinforcement needs to be avoided.

Another consideration is whether or not the bridge will remain open to traffic while the strengthening is being completed. If the bridge is to remain open while the strengthening work is performed, demolishing some or all of the pier cap may not be a feasible solution without providing an alternate support system for the girders.

For these and other possible reasons, an external post-tensioning system is an appropriate method to strengthen an existing concrete pier cap. This type of strengthening can be added while the structure remains open and without modification to the existing structure.

When designing the pier cap strengthening, both the existing pier cap and the new strengthening material must be checked for all applicable limit states. The existing components are typically loaded in compression and will have sufficient capacity as long as sufficient bearing area is provided and the applied PT loads are not applied too close to the edge. The amount of post-tensioning required is a function of the allowable tension stresses and the applied moments. The new strengthening material should be sized for the amount of post-tensioning required to limit the tension stresses to the allowable.

References Page

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