APPENDIX I

Analysis Results

This Appendix contains a summary of the selected analyses results.

- Sensitivity Analysis
- FRP (Fiber Reinforce Polymer) Methodology Hand Calculation & Excel verification
- Arch Rib Analysis
- Vertical Hanger Analysis
- Tie Girder Analysis
- Portal Bracing Analysis
- Substructure Analysis miscellaneous
- Expected Existing Material Properties
- Elevation Shift at Rumsey (NGVD29 to NAVD88)

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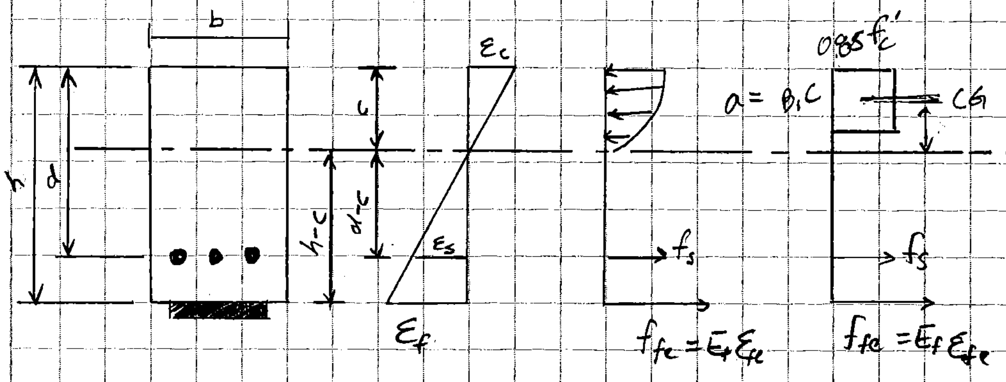
Project FRP
Job No.
By JC Date 8/15/2013

Description

SHEET

$F_c = 0.85 \times f'_c \times a \times b$ [kips]

$\beta = 0.85$ if $f'_c < 4$ksi



$a = \beta_1 c$

$c = \frac{a}{\beta}$

$F_s = A_s \times f_y$

$\sigma = E\,\varepsilon$

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_s}{d-c} = \frac{\varepsilon_{fe}}{h-c}$$

1st Assume steel yields,
$\varepsilon_c = 0.003$

2nd find c

$$\rightarrow \varepsilon_{fe} = \frac{(h-c)(\varepsilon_c)}{c}$$

$h = 36''$

$b = 27''$

$f'_c = 2.5$ ksi

$\varepsilon_c = 0.003$ in/in

$F_y = 40$ ksi

Main bars #9, Tot 6

$A_s = 1 in^2 \times 6EA = 6 in^2$

$d_s = 1.128$ in

Shear bars # 3

$A_r = 0.11 in^2$

$d_v = 0.375$ in

$Clr = 1.5$ in assumed

FRP

$E_{tension} = 1.19 \times 10^4$ ksi

$\varepsilon_{break} = 0.85\%$

$f_u = 121$ ksi

Step 1: Assumes $\varepsilon_s \geq \varepsilon_y$ ie steel yields

$F_s = A_s \times f_y$

$= (6 in^2)(40 ksi)$

$= 240$ kips

$F_{fe} = A_{frp} \times f_{fe}$, $f_{fe} = \varepsilon_{fe} E_f$

f_{fe} effective stress in FRP

Step 2: $C_c = T_s + T_f$, compression of Conc = Tension of steel & FRP

$0.85 f'_c$

$C_c = T_s + T_f$

$0.85 f'_c \times [a \times b] = A_s \times f_y + A_{FRP} \times \underbrace{\varepsilon_{fe} E_f}_{f_{fe}}$

a — NA

$(0.85)(2.5 ksi)(a)(27'') = (6 in^2)(40 ksi) + (0.04 in \times 2EA \times 27'') \times \frac{(h-c)(\varepsilon_c)}{c} \times E_f$

but $a = \beta_1 \cdot c$

$a = 0.85 \times c$

$(0.85)(2.5 ksi)(0.85)(c)(27'') = (6 in^2)(40 ksi) +$

$+ (0.04 in)(2)(27'')\frac{(36''-c)}{c}(0.003)(1.19e^4 ksi)$

$\rightarrow c = 9.4''$ or $\cancel{-6.1''}$

$\rightarrow a = (0.85)(9.4'')$

$a = 8''$

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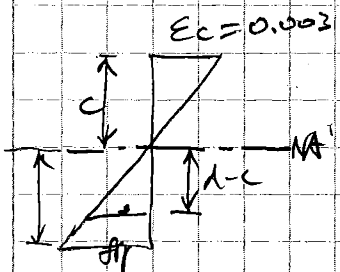
Project______ Description______
Job No.______
By______ Date______

SHEET

step 3: check if $\varepsilon_s \geq \varepsilon_y$, if steel yields



$$\frac{\varepsilon_s}{d-c} = \frac{0.003}{c},$$

but $d = h - clr - \phi_{shear} - \frac{1}{2}\phi_{main}$

$= 36'' - 1\frac{1}{2}'' - 0.375'' - \frac{1}{2}(1.128in'')$

$= 33.56''$

$\rightarrow$

$$\frac{\varepsilon_s}{33.56'' - 9.4''} = \frac{0.003}{9.4''}$$

$\rightarrow \varepsilon_s = 0.00\cancel{77}$

$f_y = E_s \; \varepsilon_y$

$40 ksi = 29{,}000 ksi \times \varepsilon_y$

$\rightarrow \varepsilon_y = 0.0014$

$\varepsilon_s > \varepsilon_y$ $\therefore$ steel does yield !!

step 4: calc ε_{fe} & check limits of FRP

$$\frac{\varepsilon_{fe}}{h-c} = \frac{\varepsilon_c}{c}$$

$$\frac{\varepsilon_{fe}}{36'' - 9.4''} = \frac{0.003}{9.4''}$$

$\rightarrow \varepsilon_{fe} = 0.0085 \leq 0.85\%$ Design Elongation at break ✓

$f_{fe} = \varepsilon_{fe} \; E_f$

$= (0.0085)(1.19 \times 10^4 ksi) = 101 ksi < 121 ksi$

ok

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Project ______ **Description** ______

Job No. ______

By ______ **Date** ______

SHEET /

Calc Moment strength Nominal

Step 5 $M_n = F_c \times Arm_{conc}$

$+ F_s \times Arm_{steel}$

$+ F_f \times Arm_{fiber\ wrap}$

$$M_n = [0.85 \times f_c' \times a \times b]\left[c - \frac{a}{2}\right]$$

$$+ [A_s \times f_y][d - c]$$

$$+ [A_{FRP} \times \varepsilon_{fe} \times E_f][h - c]$$

$$M_n = \underbrace{(0.85)(2.5\,ksi)(8'')(27'')}_{459\,kips}\underbrace{\left(9.4'' - \frac{8''}{2}\right)}_{5.4''}$$

$$+ \underbrace{(6in^2)(40\,ksi)}_{240\,kips}\underbrace{(33.56'' - 9.4'')}_{24.2}$$

$$+ \underbrace{(0.04in)(2\,EA)(27in)\,(\underbrace{0.0085)(1.19 \times 10^4\,ksi}_{101\,ksi})}_{218\,kip}\underbrace{(36'' - 9.4'')}_{26.6''}$$

$M_n = 2{,}478$ kin

$+ 5{,}808$ kin

$+ 5{,}811$ kin

$14{,}097$ kin

$= 1{,}175$ kft

Step 6 ϕM_n

$(0.9)(1{,}175\ kft)$

$= 1{,}057$ kft

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Project:	Rumsey	Description	Fiber Reinforcement Polymer
Job No.			Arch Rib - Strong axis
BY	JC	DATE 8/15/2013	Hand Calc verification

SHEET

Concrete Section input:

Overall, h [in] =	36.00 in	3.00 ft
d [in] =	33.56 in	2.80 ft
Overall, b [in] =	27.00 in	2.25 ft
Concr f'c [ksi] =	2.5 ksi	
Concrete ε_c =	0.003	conc strain limit
PS [yes, no] =	no	
phi_PS =	1.00	
phi_non-ps =	0.90	
ϕ =	0.9	

Steel Reinforcement input:

Fy [ksi] =	40 ksi	
Main Bars	# 9	conservative of 1 1/8" SQ bar
Tot =	6	conservative base on per as-built.
A_s [in^2] =	1	Photos shows 7 bars
d_s [in] =	1.128	Main Reinforcement
As [in^2] =	6 in sq	
Shear Bars	# 3	
A_v [in^2] =	0.11	
d_v [in] =	0.375	Shear Confinement
clr [in] =	1.5 in	assumed

Manufacture FRP input:

Design limits:

Ult Tensile Strength in Primary Direction =	121,000	psi
	4.8	kip/in
Elongation at Break =	0.85%	
Tensile Modulus =	1.19E+07	psi
	1.19E+04	ksi
n, layers [ea] =	2	number of layers of FRP reinforcement
t_f [in] =	0.04 in	FRP composite material thickness per layer
Total thickness	0.08 in	
w_f [in] =	27.00 in	width of FRP reinfocing layers

ϕ = 0.9, if $e_s \geq 0.005$

ϕ = interp if $0.002 \leq e_s < 0.005$

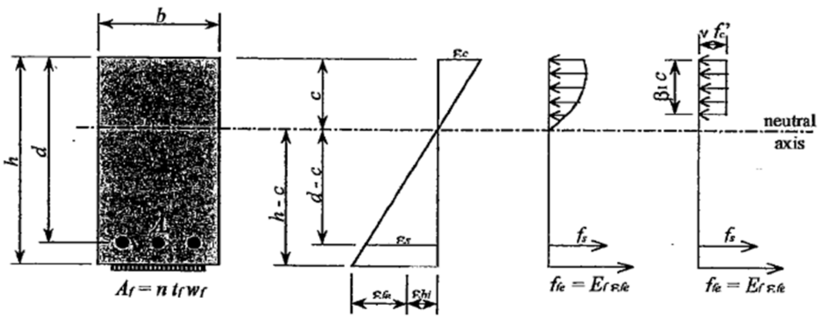
ϕ = 0.65, if otherwise

Assumptions:

Plain section remains plane

Max compressive strain in concrete is 0.003

Stress in steel under service load should be limited to 80% of yield strength



Reinforced Concrete Section | **Strain Distribution** | **Stress Distribution (Non-linear Concrete Stress Distribution)** | **Stress Distribution (Equivalent Fictitious Concrete Stress Distribution)**

Analysis:

Forces:

F_Conc = 0.85*fc*a*b = 458 kip

F_steel = As*fy = 240 kip

$F_{frp} = A_{frp} * f_{fe}$ = 218 kip

$f_{fe} = \varepsilon_{fe} * E_f$ = 1.01E+02 ksi

$A_f = n * t_f * w_f$ = 2.16 in sq

Strains: *(similar triangles)*

ε_c / c = = ε_s / (d - c) = = = ε_fe / (h - c)

Concrete ε_c = 0.003

Steel ε_s = 0.0077

Yeild ε_y = 0.0021 steel yields

RFP ε_{fe} = 0.0085 RFP with in allowable stain

Neutral Axis:

c [in] = 9.40 in calculated distance between N.A. to extreme conc fiber

F_Conc = F_steel + F_frp

0 = - F_Conc + F_steel + F_frp

0 kip

beta = 0.85 (AASHTO 5.7.2.2) function of f'c

a [in] = 8.0 in vertical distance of whitney stress block

Moment Arm:

Conc Arm = (c - a / 2) = 5.40 in

Steel Arm = (d - c) = 24.16 in

Fiber Arm = (h - c) = 26.60 in

Nominal Moment:

Conc Contr = 2,476 kip-in

Steel Contr = 5,799 kip-in

Fiber Contr = 5,807 kip-in

Nominal Moment =
14,083 kip-in
1,174 kip-ft

Factored Moment Strength:

ϕ Mn [k-in] = 12,675 kip-in

ϕ Mn [k-ft] = 1,056 kip-ft

Sensitivity Analysis

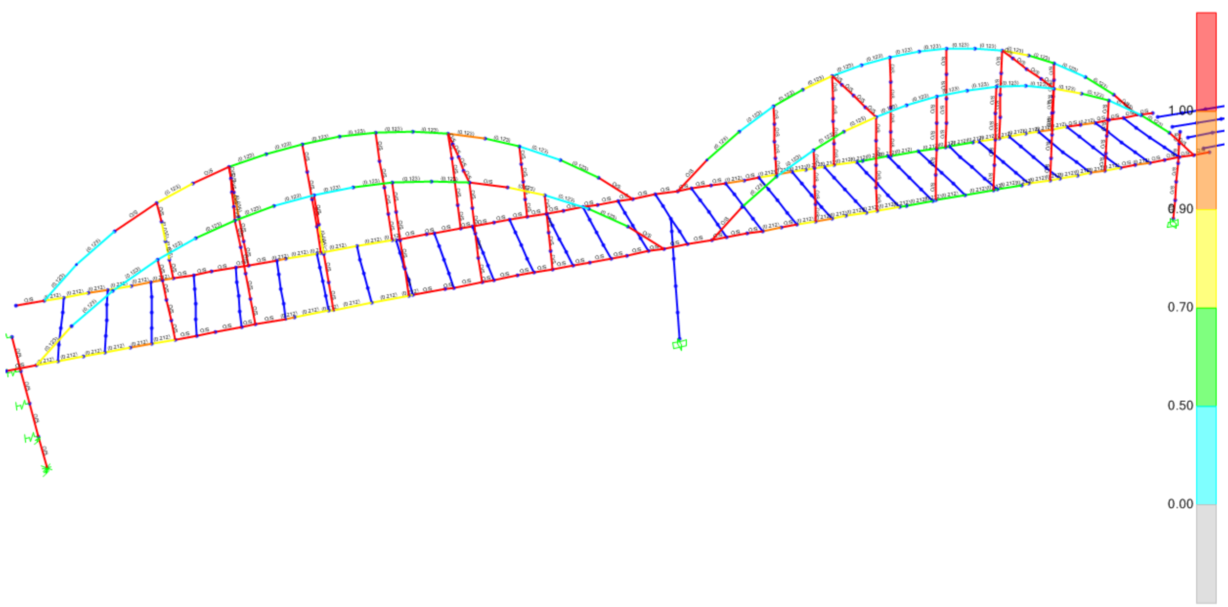
Several sensitivity analyses were performed to assess the necessity of obtaining a concrete strength core to determine a refined concrete strength of the existing Rumsey Bridge. Base on the sensitivity analyses results in the following pages, QEI does not recommend obtaining sampling at this planning phase. For more discussion, please see Section 2's Similar Structure (Stevenson Bridge) study in this Feasibility Study.

Original / Baseline Model:

Existing concrete strength: 2500 psi

Existing bar reinforcing steel strength: 40 ksi

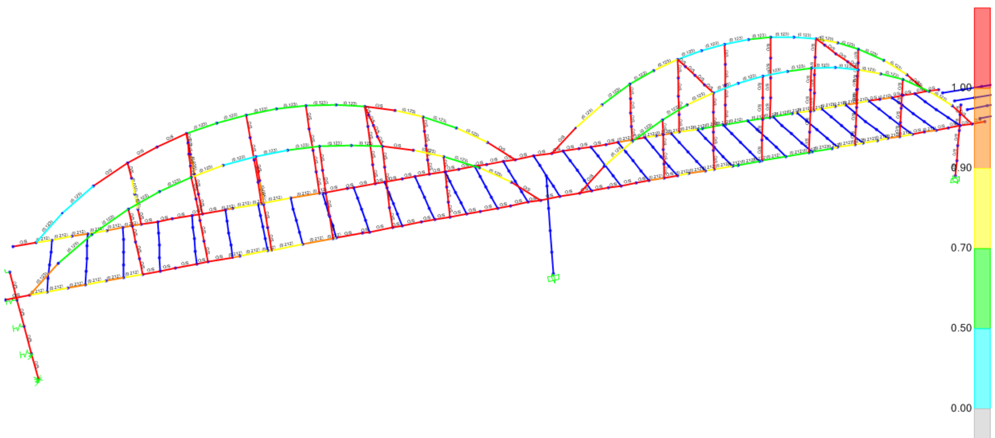
Governing elastic D/C ratios:



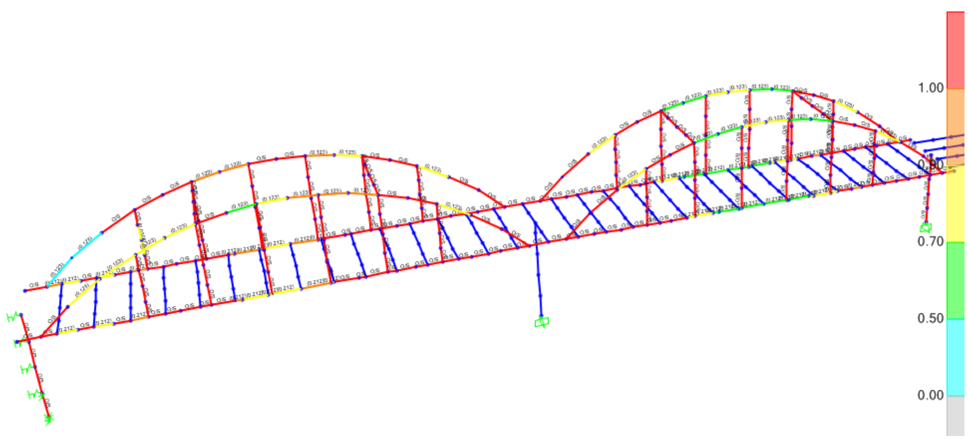
Sensitivity Model:

Sensitivity Modeling Force and Displacement D/C Results are approximately the same—as anticipated. Changing Concrete strength will have more impact on shear strength; however, most D/C ratios are governed by axial and flexural capacities. Therefore, globally, the retrofit strategy is not affected by the concrete strength. During final design the concrete strength will affect eh choice of the number of FRP layers required for a given member.

Concrete strength: 2000 psi Model
Governing elastic D/C ratios:

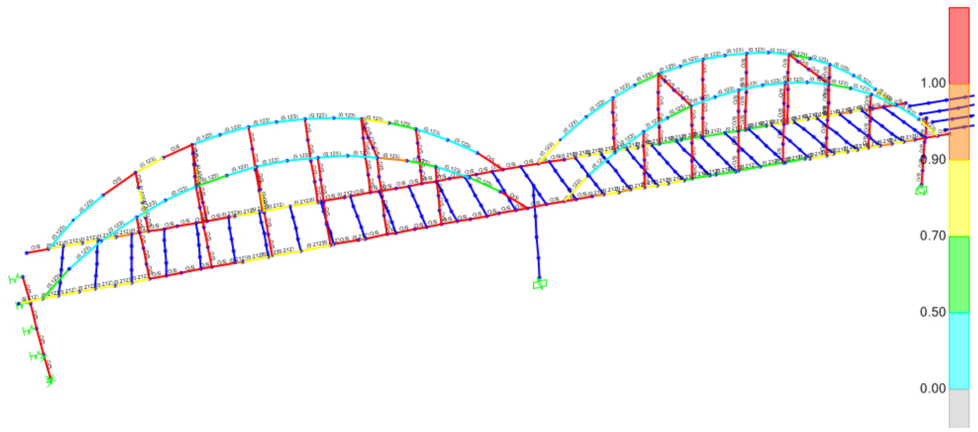


Concrete strength: 1000 psi Model
Governing elastic D/C ratios:



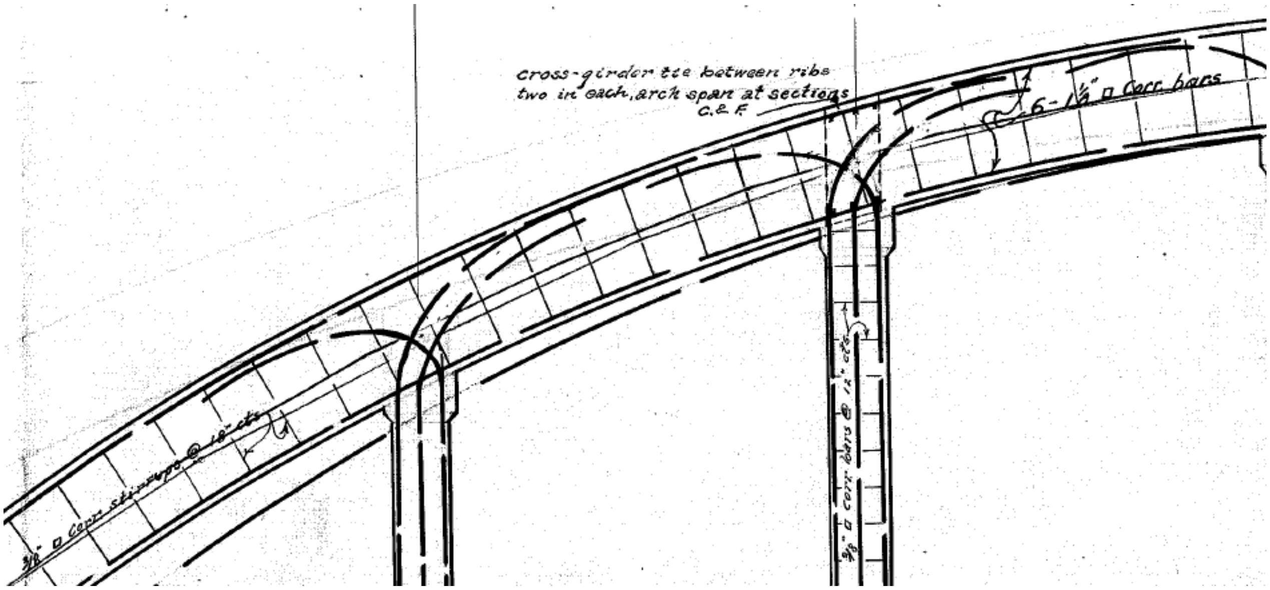
Concrete strength: 4000 psi Model

Governing elastic D/C ratios:

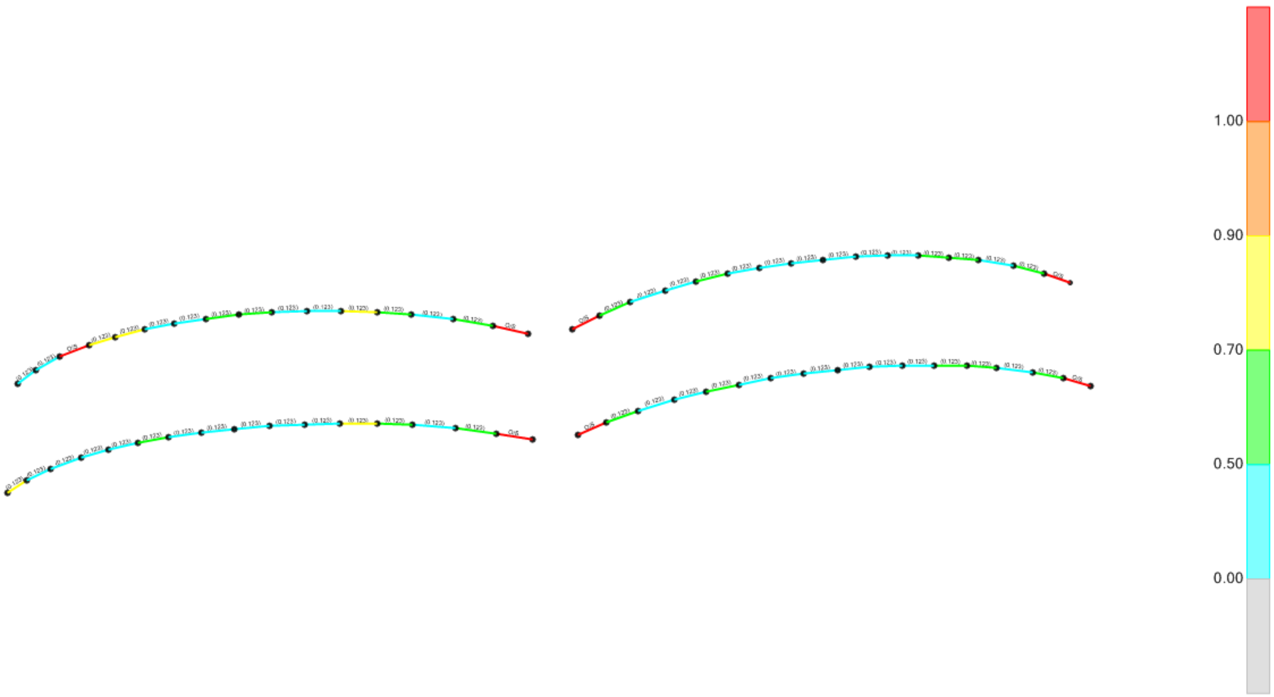


Arch Rib Analysis:

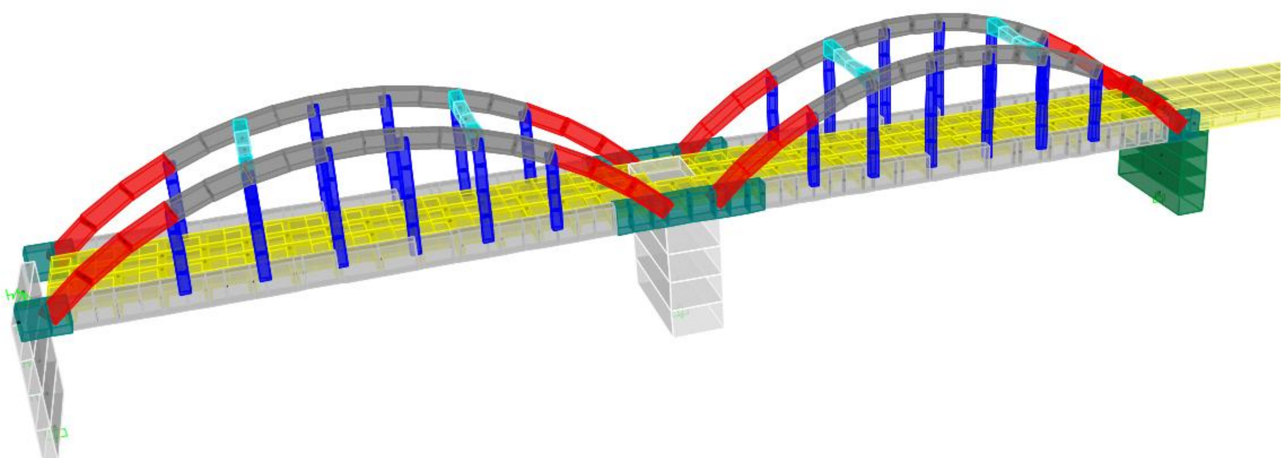
As-built details of Arch sections & reinforcement:



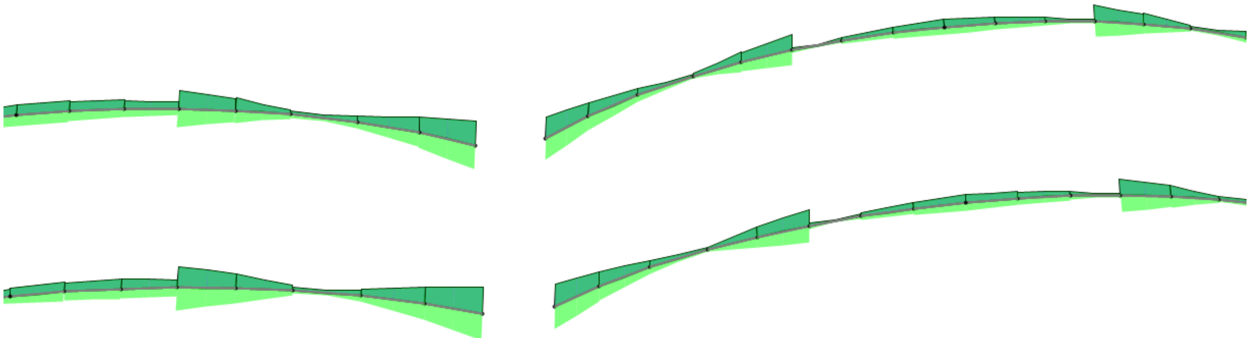
Analysis below shows the limits where the Arch Rib exceed D/C ratio of 1.0 in Red:



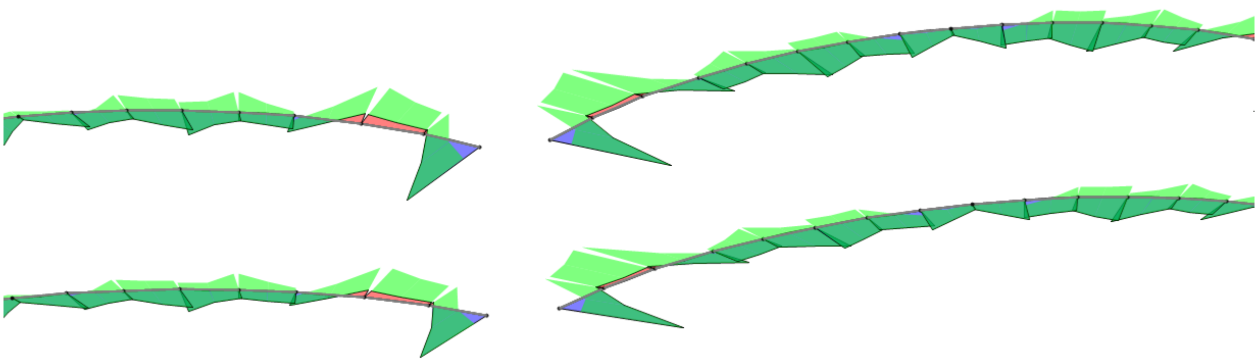
Red block below indicates the approximate location of where the Arch will be retrofitted.



Moment diagram below indicates the weak access moments on the Arch Ribs.



Moment diagram below indicates the Strong access moments on the Arch Ribs.



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Project:	**Rumsey**	Description	**Bending**
Job No.			**Arch - Strong Axis (Existing)**
BY	**JC**	DATE	**5/12/2014**

SHEET

D [in] =	36.00 in	3.00 ft			
d [in] =	33.56 in	2.80 ft	Fy [ksi] =	40 ksi	
b [in] =	27.00 in	2.25 ft			
fc [ksi] =	2.5 ksi		Main Bars	# 9	
			Tot =	6	
PS [yes, no] =	no		A_s [in^2] =	1	
phi_PS =	1.00		d_s [in] =	1.128	Main Reinforcement
phi_non-ps =	1.00		As [in^2] =	6	
ϕ =	1				
			Shear Bars	# 3	
			A_v [in^2] =	0.11	
			d_v [in] =	0.375	Shear Confinement
			clr [in] =	1.5	

Analysis:

F_steel =	As*fy =		F_Conc =	0.85*fc*a*b		
	240 kip			240 kip		
a [in] =	4.18 in					
beta =	0.85	(AASHTO 5.7.2.2)				
x [in] =	a/beta					
	4.92 in					
Check Steel Yield						
e_s =	0.0175	>	e_y =	0.0021		
		steel yields				
Arm [in] =	31.47 in					
ϕ Mn [k-in] =	7,553 kip-in	>	Demand =	5,709	ok	D/C= 0.76
ϕ Mn [k-ft] =	629 kip-ft	>	Demand =	476 kip-ft	ok	

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Project: Rumsey | Description Bending
Job No. | Arch - Weak Axis (Existing) | SHEET
BY JC | DATE 5/12/2014

D [in] =	27.00 in	2.25 ft				
d [in] =	24.56 in	2.05 ft		Fy [ksi] =	40 ksi	
b [in] =	36.00 in	3.00 ft				
fc [ksi] =	2.5 ksi			Main Bars	# 9	
				Tot =	2	
PS [yes, no] =	no			A_s [in^2] =	1	
phi_PS =	1.00			d_s [in] =	1.128	Main Reinforcement
phi_non-ps =	1.00			As [in^2] =	2	
ϕ =	1					
				Shear Bars	# 3	
				A_v [in^2] =	0.11	
				d_v [in] =	0.375	Shear Confinement
				clr [in] =	1.5	

Analysis:

F_steel =	As*fy =		F_Conc =	0.85*fc*a*b		
	80 kip			80 kip		
a [in] =	1.05 in					
beta =	0.85	(AASHTO 5.7.2.2)				
x [in] =	a/beta					
	1.23 in					
Check Steel Yield						
e_s =	0.0569	>	e_y =	0.0021		
		steel yields				
Arm [in] =	24.04 in					
ϕ Mn [k-in] =	1,923 kip-in	<	Demand =	7,606	NG	D/C= 3.96
ϕ Mn [k-ft] =	160 kip-ft	<	Demand =	452 kip-ft	NG	

Compression Member Strength

fc' =	2500	psi
t =	36	in
b =	27	in
d =	33.365	in
d' =	2.635	in
As =	7.62	in2
As' =	7.62	in2
fy =	40,000	psi
beta1 =	0.85	
Pu =	**564 kip**	Demand

Main Bars	# 10	Main Reinforcement
Tot =	6	
A_s [in^2] =	1.27	
d_s [in] =	1.27	
As [in^2] =	7.62	
Shear Bars	# 3	Shear Confinement
Spacing [in] =	18	
A_v [in^2] =	0.11	
d_v [in] =	0.375	
clr [in] =	2	
Comp Bars	# 10	Main Reinforcement
Tot =	6	
A_s' [in^2] =	1.27	
d_s' [in] =	1.27	
As' [in^2] =	7.62	

Reinforcement of Compession Members (LRFD 5.7.4.2)

Max Reinforcement

Ast/Ag < 0.08

Ast/Ag =	(As+As_prime)/(t*b)			
Ast/Ag =	0.018	<	0.08	ok

Min Reinforcement

Ast/Ag > 0.0025 for pier wall, and 0.01 for comp. member

Ast/Ag =	(As+As_prime)/(t*b)			
Ast/Ag =	0.018	>	0.010	ok

Min bar # 5, Check

c-c < 12", Check

Pure Compression (LRFD 5.7.4.4)

Po = phi[0.80*fc' (*b*t - As - As') + (As + As')*fy] (8-30) for ties

ϕ =	0.85				
ϕ Po =	2,112,000	lbs			
ϕ Po =	2,112 kips		>	**564 kip**	ok
				0.27	

Lateral Reinforcement (BDS 8.18.2)

spacing > min(12", t=24"), Check

Greater than #3 if Long. bar <#10, Check

Ties (BDS 8.18.3)

h_wall =	276	in
Ash =	3.5933	in2
h_c =	31.625	in
Ag =	972	in2
Ac =	850	in2

0.30*s_t*h_c*fc' /fy *(Ag/Ac-1) =

1.54 in2

3.59 in2 > Demand = 1.54 in2 ok

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Project:	**Rumsey**	Description **Arch Rib**		
Job No.	**Y01-500**	**Shear yy (LRDF 5.8.3.3)**		SHEET
BY	**JC**	DATE	**5/12/2014**	

LRFD 5.8.3.3

Vn = minium of the following 2 equations
Vn = Vc + Vs + Vp
Vn = 0.25f'c bv dv + Vp

Ecco Results:
ϕ V [kip] = 48 kip < Demand = 79 kip **NG**
1.65

b =	36 in		Fy [ksi] =	40 ksi	
d =	27 in				
Ac =	972 in2		Shear Bars	# 3	
f'c [ksi] =	2,500 psi	2.5 ksi	Spacing	18.00 in	spacing of stirrup
			A_v_bar =	0.11 in2	
bv =	36 in	eff web width	d_v_bar =	0.38 in	
dv =	24 in	eff shear depth	mult by	2	number of bars per plane
			Av	0.01 in	area of shear reinfocement w/in dist s

5.8.2.5 Minimum Transverse Reinforcement

Av > 0.0316 sqrt (f'c) b_v s / fy

Av = 0.01 in2 NG 0.81 in2 = 0.0316 sqrt (f'c) bv s / fy

5.8.3.4.2 General Procedure

es = (Mu/dv + 0.5 Nu + |Vu-Vp| - Apsfpo) / (Es As + Ep Aps)

Mu =	629 k-ft	Moment demand			
Nu =	562 k	Axial demand			
Vu =	78 k	Shear demand			
Vp =	0 k	component in direction of applied shear of the effective prestressing force positive if resisting the applied shear			
Aps =	0.00 in2	Area of PS			
fpo =	175 ksi	0.7 fpu		Flexural steel info:	
Es =	29000 ksi			Fy [ksi] =	40 ksi
Ep =	29000 ksi			Flex Bars	# 9
As =	6.00 in2	flexural steel		A_s_bar =	1.00 in2
				d_s_bar =	1.13 in
es =	0.0022			mult by	6

Sxe = sx * 1.38 / (ag + 0.63)

sx =	min of following	24 in	
	dv =	24 in	
	d__ =	36 in	dist between layers of long crack control reinf
ag =	0.25 in	max agg size	assumed per photos
Sxe =	37.6		

theta = 29 + 3500 es
theta = 37 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met
beta = 4.8 / (1+ 750 εs)
beta = 1.80 factor indicating ability of diagonal cracked concrete to transmit tension & shear
If Min Transverse Reinforcement is NOT met
beta = 4.8 / (1+ 750 εs) x 51 / (39 + Sxe)
beta = 1.22 factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select: beta = 1.2 degrees factor indicating ability of diagonal cracked concrete to transmit tension & shear
theta = 37 degrees angle of inclination of diagonal comp. stress
Select: alpha = 90 degrees angle of inclination of transverse reinforcemen

Vn = Vc + Vs + Vp
Vc = 0.0316*beta*sqrt(f'c) bv dv
Vc = 53 kip
Vs = Av fy sin(alpha) < 0.095 sqrt(f'c) bv dv
Vs = 0.44 kip < 130 kip
Vn = 53 kip Gov

Vn = 0.25f'c bv dv + Vp
Vn = 540 kip

ϕ = 0.90 Seimsic phi for shear

Select: **ϕ V [kip] = 48 kip** < Demand = 79 kip NG
1.65

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Project:	**Rumsey**		Description **Arch Rib**	
Job No.	**Y01-500**		**Shear xx (LRDF 5.8.3.3)**	SHEET
BY	**JC**	DATE **5/12/2014**		

LRFD 5.8.3.3

Vn = minium of the following 2 equations
Vn = Vc + Vs + Vp
Vn = 0.25f'c bv dv + Vp

Ecco Results:
ϕ V [kip] = 42 kip > Demand = 37 kip **ok**
0.87

b =	27 in		Fy [ksi] =	40 ksi	
d =	36 in				
Ac =	972 in2		Shear Bars	# 3	
f'c [ksi] =	2,500 psi	2.5 ksi	Spacing	18.00 in	spacing of stirrup
			A_v_bar =	0.11 in2	
bv =	27 in	eff web width	d_v_bar =	0.38 in	
dv =	33 in	eff shear depth	mult by	2	number of bars per plane
			Av	0.01 in	area of shear reinfocement w/in dist s

5.8.2.5 Minimum Transverse Reinforcement

Av > 0.0316 sqrt (f'c) b_v s / fy

Av = 0.01 in2 NG 0.61 in2 = 0.0316 sqrt (f'c) bv s / fy

5.8.3.4.2 General Procedure

es = (Mu/dv + 0.5 Nu + |Vu-Vp| - Apsfpo) / (Es As + Ep Aps)

Mu =	629 k-ft	Moment demand			
Nu =	562 k	Axial demand			
Vu =	78 k	Shear demand			
Vp =	0 k	component in direction of applied shear of the effective prestressing force positive if resisting the applied shear			
Aps =	0.00 in2	Area of PS			
fpo =	175 ksi	0.7 fpu		Flexural steel info:	
Es =	29000 ksi			Fy [ksi] =	40 ksi
Ep =	29000 ksi			Flex Bars	# 9
As =	6.00 in2	flexural steel		A_s_bar =	1.00 in2
				d_s_bar =	1.13 in
es =	0.0022			mult by	6

Sxe = sx * 1.38 / (ag + 0.63)

sx =	min of following	33 in	
	dv =	33 in	
	d__ =	36 in	dist between layers of long crack control reinf
ag =	0.25 in	max agg size	assumed per photos
Sxe =	51.8		

theta = 29 + 3500 es
theta = 37 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met
beta = 4.8 / (1+ 750 εs)
beta = 1.83 factor indicating ability of diagonal cracked concrete to transmit tension & shear
If Min Transverse Reinforcement is NOT met
beta = 4.8 / (1+ 750 εs) x 51 / (39 + Sxe)
beta = 1.05 factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select:	beta =	1.0 degrees	factor indicating ability of diagonal cracked concrete to transmit tension & shear
	theta =	37 degrees	angle of inclination of diagonal comp. stress
Select:	alpha =	90 degrees	angle of inclination of transverse reinforcemen

Vn = Vc + Vs + Vp
Vc = 0.0316*beta*sqrt(f'c) bv dv
Vc = 47 kip
Vs = Av fy sin(alpha) < 0.095 sqrt(f'c) bv dv
Vs = 0.44 kip < 134 kip
Vn = 47 kip Gov

Vn = 0.25f'c bv dv + Vp
Vn = 557 kip

ϕ = 0.90 Seismic phi for shear

Select: **ϕ V [kip] = 42 kip** > Demand = 37 kip ok
0.87

QUINCY ENGINEERING, INC.

Project:	**Rumsey**	Description	**Fiber Reinforcement Polymer**
Job No.			**Arch Rib**
BY	**JC** DATE **5/12/2014**		**Strong axis (Retofitted)**

SHEET

Concrete Section input:

Overall, h [in] =	36.00 in	3.00 ft
d [in] =	33.56 in	2.80 ft
Overall, b [in] =	27.00 in	2.25 ft
Concr f'c [ksi] =	2.5 ksi	
Concrete ε_c =	0.003	conc strain limit
PS [yes, no] =	no	
phi_PS =	1.00	
phi_non-ps =	0.90	
ϕ =	0.9	

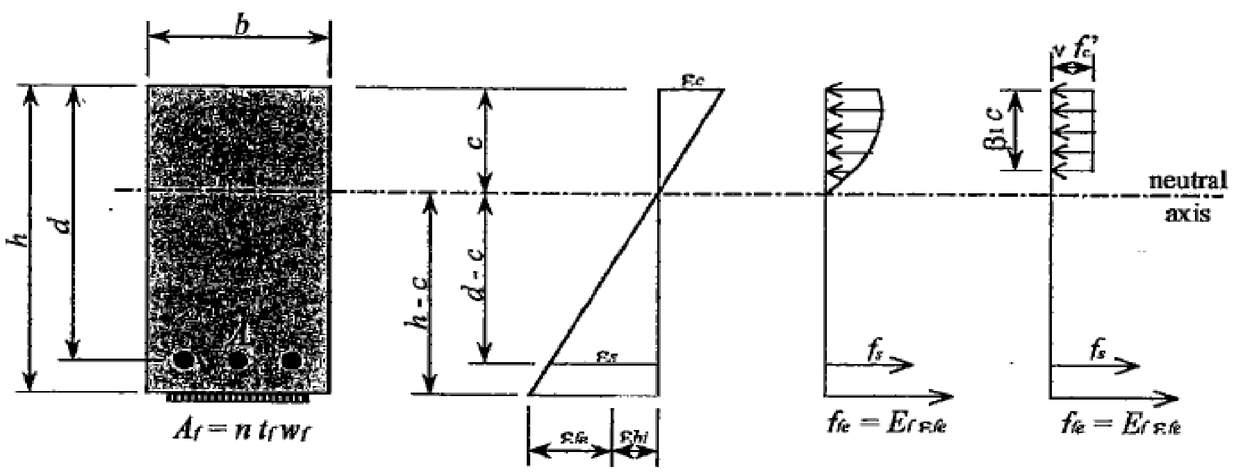
Steel Reinforcement input:

Fy [ksi] =	40 ksi	
Main Bars	# 9	conservative of 1 1/8" SQ bar
Tot =	6	conservative base on per as-built.
A_s [in^2] =	1	Photos shows 7 bars
d_s [in] =	1.128	Main Reinforcement
As [in^2] =	6 in sq	
Shear Bars	# 3	
A_v [in^2] =	0.11	
d_v [in] =	0.375	Shear Confinement
clr [in] =	1.5 in	assumed

Manufacture FRP input:

	Design limits:	
Ult Tensile Strength	121	ksi
in Primary Direction =	121,000	psi
	4.8	kip/in
Elongation at Break =	0.6%	
Tensile Modulus =	1.19E+07	psi
	1.19E+04	ksi
n, layers [ea] =	5	number of layers of FRP reinforcement
t_f [in] =	0.04 in	FRP composite material thickness per layer
Total thickness	0.20 in	
w_f [in] =	27.00 in	width of FRP reinfocing layers

ϕ = 0.9, if $e_s \geq 0.005$
ϕ = interp if $0.002 \leq e_s < 0.005$
ϕ = 0.65, if otherwise



Reinforced Concrete Section | **Strain Distribution** | **Stress Distribution (Non-linear Concrete Stress Distribution)** | **Stress Distribution (Equivalent Fictitious Concrete Stress Distribution)**

Assumptions:
Plain section remains plane
Max compressive strain in concrete is 0.003
Stress in steel under service load should be limited to
80% of yield strength

FRP Analysis:

Forces:

F_Conc = 0.85*fc*a*b
606 kip

F_steel = As*fy =
240 kip

$F_{frp} = A_{frp} * f_{fe}$ = **366 kip**
$f_{fe} = \varepsilon_{fe} * E_f$ = **68 ksi**
$A_f = n * t_f * w_f$ = 5.4 in sq
RFP within allowable stress

Strains:
(similar triangles)
ε_c / c = = ε_s / (d - c) = = = ε_fe / (h - c)

Concrete ε_c = 0.003
Steel ε_s = 0.0051
Yeild ε_y = 0.0021
steel yields
RFP ε_{fe} = 0.0057
RFP within allowable stain

Neutral Axis:
c [in] = 12.42 in calculated distance between N.A. to extreme conc fiber

F_Conc = F_steel + F_frp
0 = - F_Conc + F_steel + F_frp
0 kip

beta = 0.85 (AASHTO 5.7.2.2) function of f'c
a [in] = 10.6 in vertical distance of whitney stress block

Moment Arm:
Conc Arm = (c - a / 2)
7.14 in
Steel Arm = (d - c)
21.14 in
Fiber Arm = (h - c)
23.58 in

Nominal Moment:
Conc Contr = 4,328 kip-in
Steel Contr = 5,073 kip-in
Fiber Contr = 8,626 kip-in

Nominal Moment =
18,027 kip-in
1,502 kip-ft

Factored Moment Strength:

***ϕ* Mn [k-in] =**	16,224 kip-in	**>**	**Demand =**	5,200	**ok**	D/C= 0.32
ϕ Mn [k-ft] =	1,352 kip-ft	>	Demand =	433 kip-ft	ok	

Analysis Existing (without FRP) - Strong Axis:

F_Conc = 0.85*fc*a*b
240 kip
F_steel = As*fy =
240 kip

a [in] = 4.18 in
beta = 0.85 (AASHTO 5.7.2.2)
c [in] = 4.92 in = a/beta

Check Steel Yield
e_s = 0.0175 > e_y = 0.0021
steel yields

Arm [in] = 31.47 in

ϕ Mn [k-in] =	**7,553 kip-in**	>	Demand =	5,200	ok	D/C= 0.69
ϕ Mn [k-ft] =	629 kip-ft	>	Demand =	433 kip-ft	ok	

QUINCY ENGINEERING, INC.

Project:	**Rumsey**	Description	Fiber Reinforcement Polymer
Job No.			Arch Rib
BY	**JC** DATE **5/12/2014**		**Weak axis (Retrofitted)**

SHEET

Concrete Section input:

Overall, h [in] =	27.00 in	2.25 ft
d [in] =	24.56 in	2.05 ft
Overall, b [in] =	36.00 in	3.00 ft
Concr f'c [ksi] =	2.5 ksi	
Concrete ε_c =	0.003	conc strain limit
PS [yes, no] =	no	
phi_PS =	1.00	
phi_non-ps =	0.90	
ϕ =	0.9	

Steel Reinforcement input:

Fy [ksi] =	40 ksi	
Main Bars	# 9	conservative of 1 1/8" SQ bar
Tot =	2	conservative base on per as-built.
A_s [in^2] =	1	Photos shows 7 bars
d_s [in] =	1.128	Main Reinforcement
As [in^2] =	2 in sq	
Shear Bars	# 3	
A_v [in^2] =	0.11	
d_v [in] =	0.375	Shear Confinement
clr [in] =	1.5 in	assumed

Manufacture FRP input:

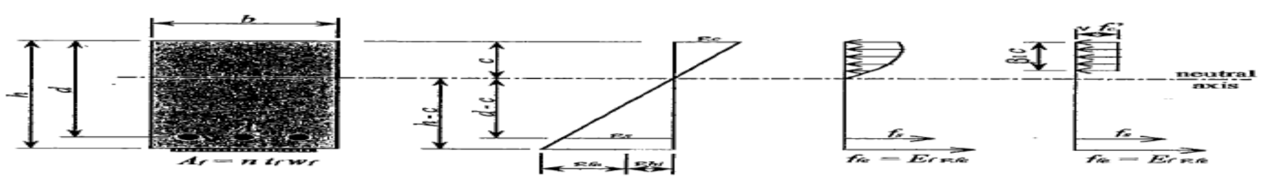
Design limits:

Ult Tensile Strength	121	ksi
in Primary Direction =	121,000	psi
	4.8	kip/in
Elongation at Break =	0.6%	
Tensile Modulus =	1.19E+07	psi
	1.19E+04	ksi
n, layers [ea] =	8	number of layers of FRP reinforcement
t_f [in] =	0.04 in	FRP composite material thickness per layer
Total thickness	0.32 in	
w_f [in] =	36.00 in	width of FRP reinfocing layers

ϕ = 0.9, if $e_s \geq 0.005$

ϕ = interp if $0.002 \leq e_s < 0.005$

ϕ = 0.65, if otherwise



Reinforced Concrete Section | Strain Distribution | Stress Distribution (Non-linear Concrete Stress Distribution) | Stress Distribution (Equivalent Fictitious Concrete Stress Distribution)

Assumptions:
Plain section remains plane
Max compressive strain in concrete is 0.003
Stress in steel under service load should be limited to
80% of yield strength

FRP Analysis:

Forces:

F_Conc = 0.85*fc*a*b **700 kip**

F_steel = As*fy = **40 kip**

$F_{frp} = A_{frp} * f_{fe}$ = **620 kip**

$f_{fe} = \varepsilon_{fe} * E_f$ = **54 ksi**

$A_f = n * t_f * w_f$ = 11.52 in sq

RFP within allowable stress

Strains: *(similar triangles)* ε_c / c = = ε_s / (d - c) = = = ε_fe / (h - c)

Concrete ε_c = 0.003

Steel ε_s = 0.0038

Yeild ε_y = 0.0021

steel yields

RFP ε_{fe} = 0.0045

RFP within allowable stain

Neutral Axis:

c [in] = 10.77 in calculated distance between N.A. to extreme conc fiber

F_Conc = F_steel + F_frp
0 = - F_Conc + F_steel + F_frp
0 kip

beta = 0.85 (AASHTO 5.7.2.2) function of f'c
a [in] = 9.2 in vertical distance of whitney stress block

Moment Arm:

Conc Arm = (c - a / 2) 7.14 in

Steel Arm = (d - c) 21.14 in

Fiber Arm = (h - c) 14.58 in

Nominal Moment:

Conc Contr = 5,001 kip-in

Steel Contr = 846 kip-in

Fiber Contr = 9,039 kip-in

Nominal Moment =
14,886 kip-in
1,240 kip-ft

Factored Moment Strength:

ϕ Mn [k-in] =	13,397 kip-in	**>**	**Demand =**	7,032	**ok**	D/C= 0.52
ϕ Mn [k-ft] =	1,116 kip-ft	>	Demand =	586 kip-ft	ok	

Analysis Existing (without FRP) - Weak Axis:

F_Conc = 0.85*fc*a*b 80 kip

F_steel = As*fy = 80 kip

a [in] = 1.05 in
beta = 0.85 (AASHTO 5.7.2.2)
c [in] = 1.23 in = a/beta

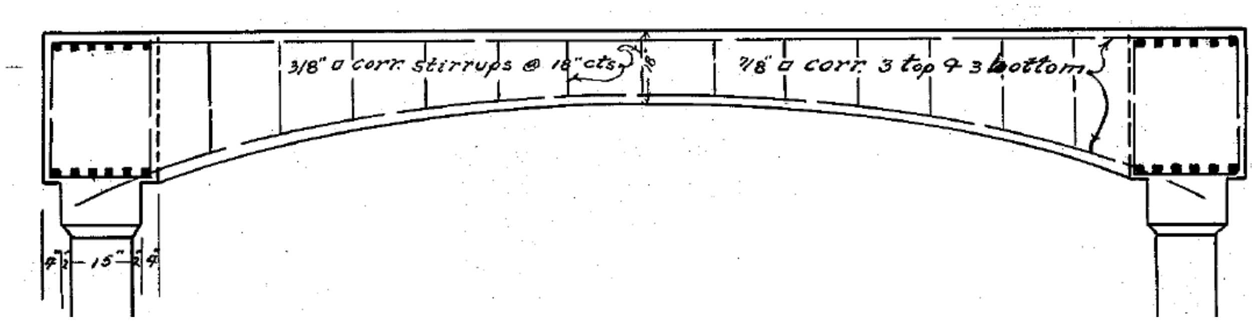
Check Steel Yield
e_s = 0.0788 > e_y = 0.0021
steel yields

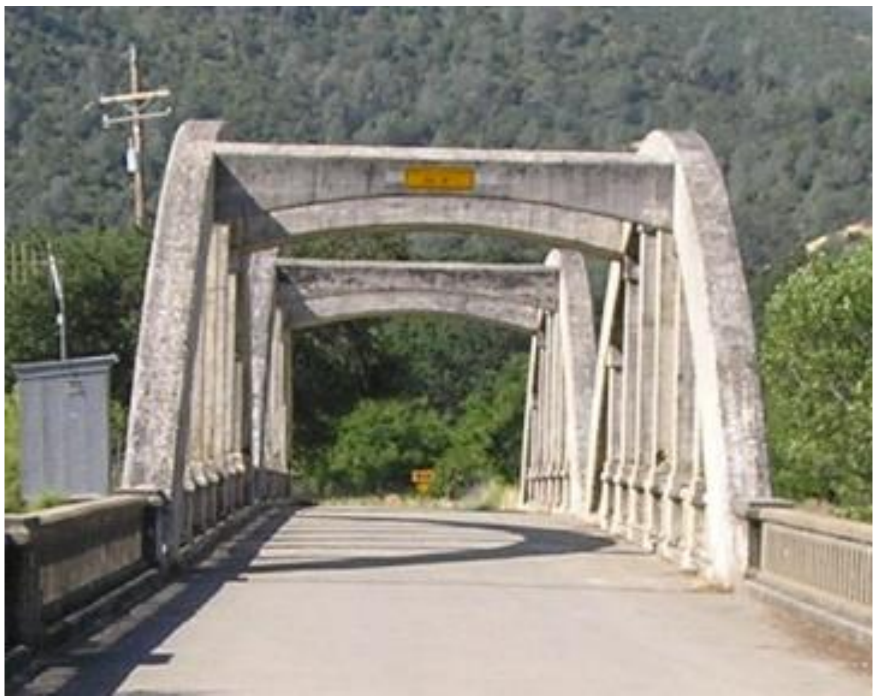
Arm [in] = 24.04 in

ϕ Mn [k-in] =	1,923 kip-in	<	Demand =	7,032	NG	D/C= 3.66
ϕ Mn [k-ft] =	160 kip-ft	<	Demand =	586 kip-ft	NG	

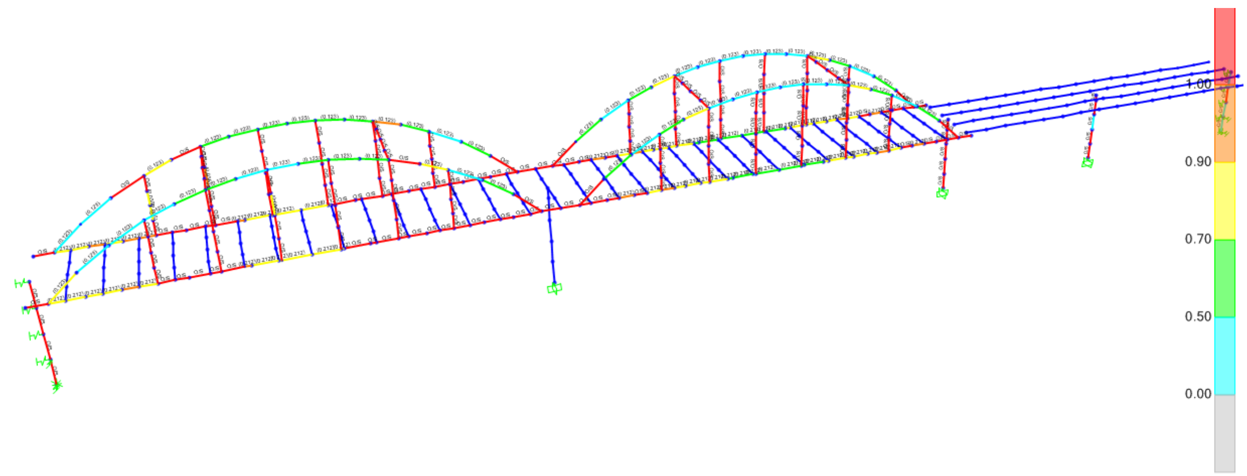
Vertical Hanger Analysis:

As-built details of Portal sections & reinforcement:

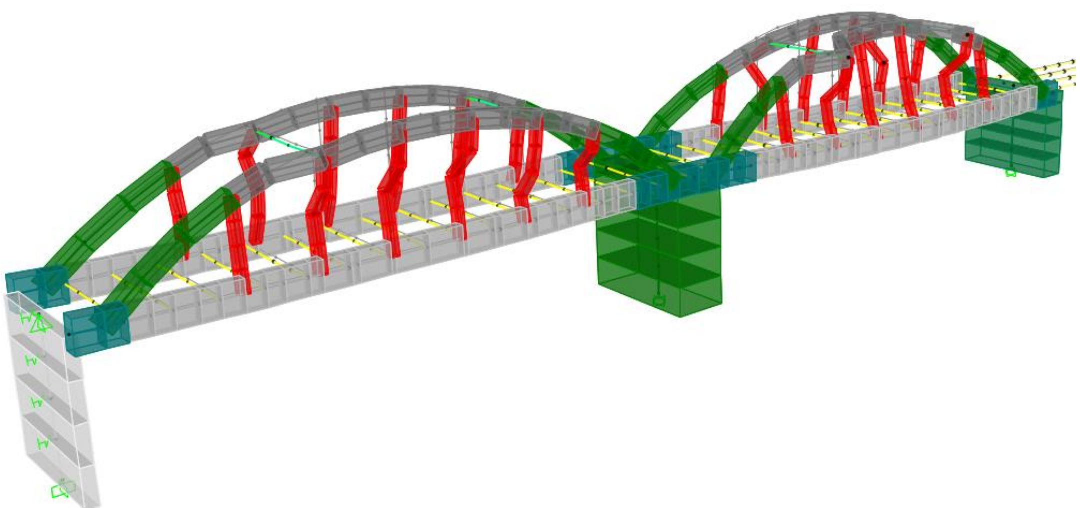




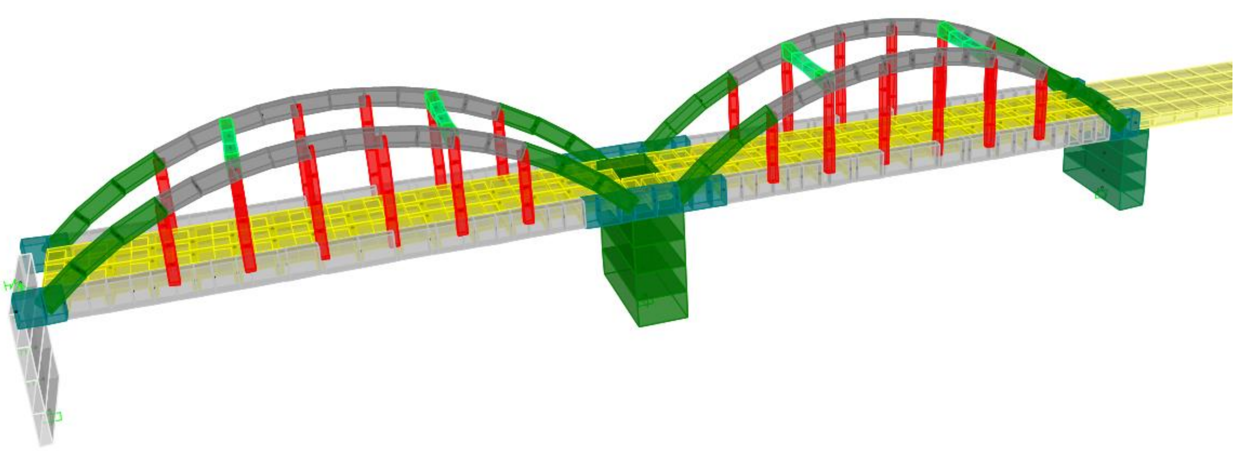
Analysis below shows the limits where the entire Hanger set exceed D/C ratio of 1.0 in Red:



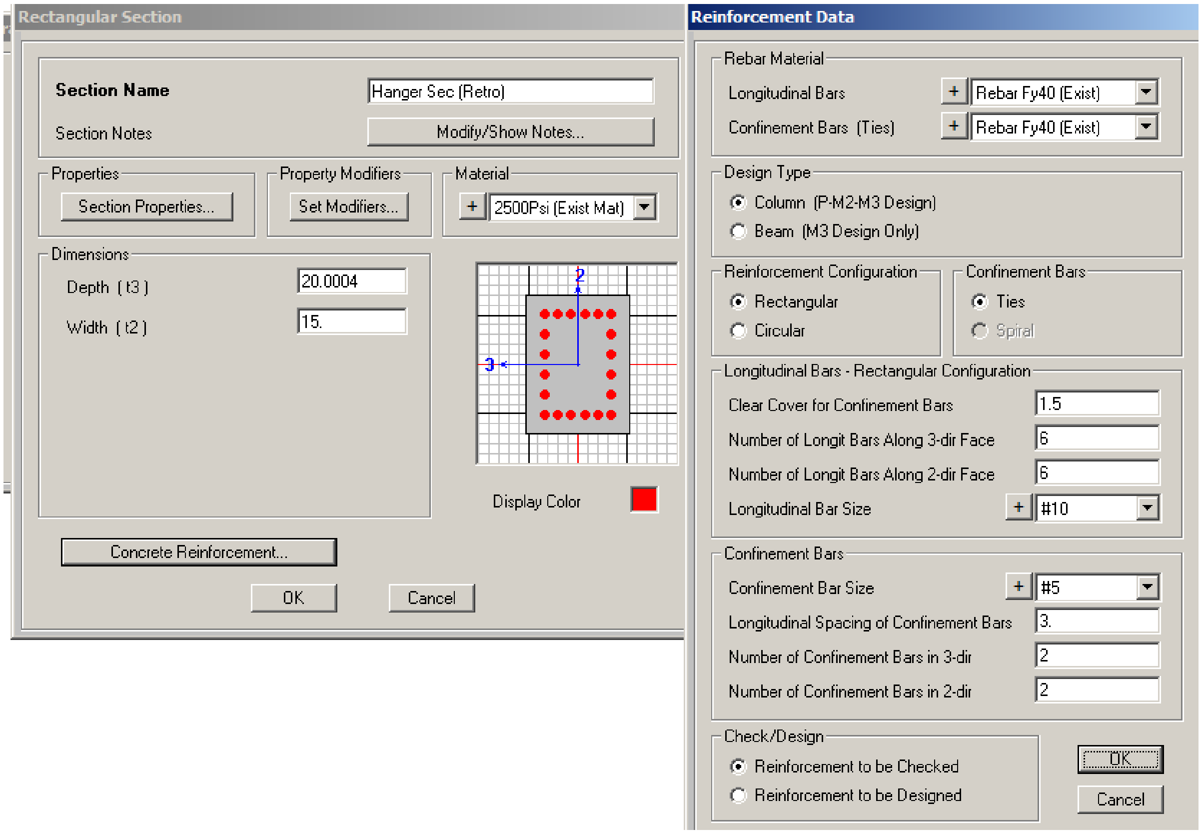
Analysis below shows the deformation from seismic excitation:

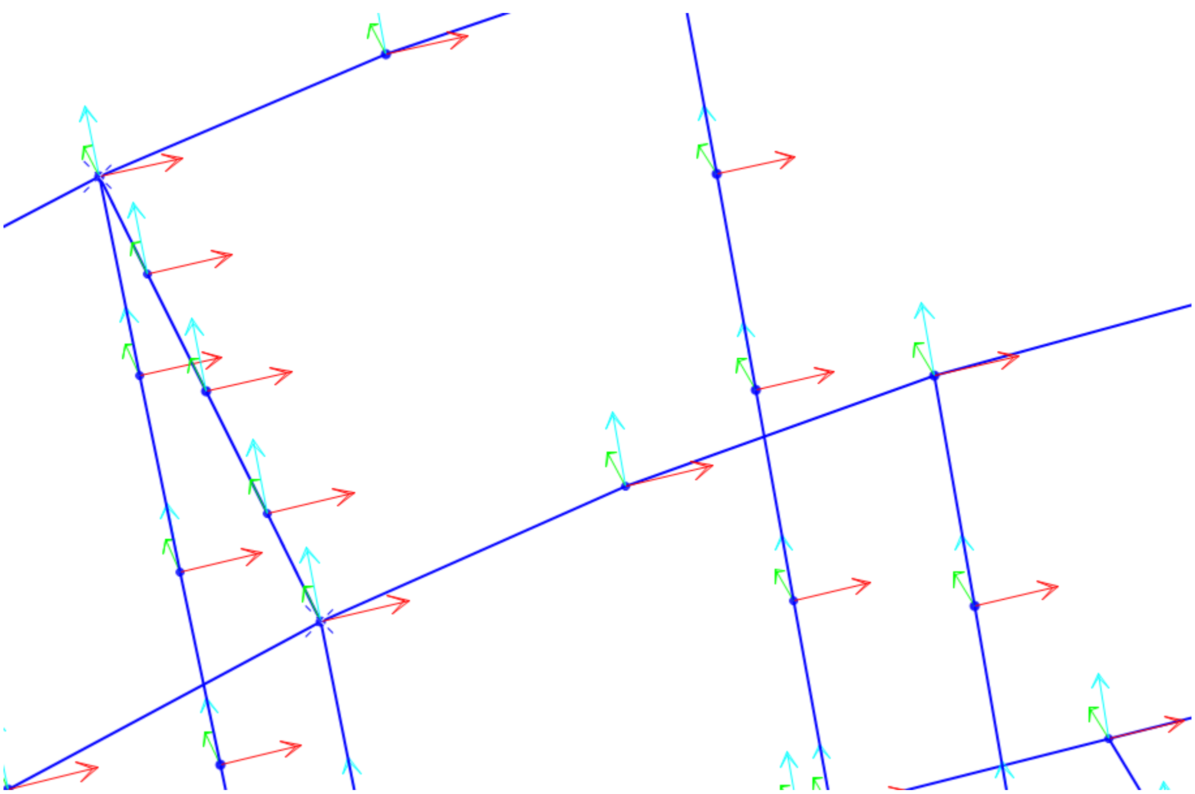


Red block below indicates the approximate location of where the Hanger will be retrofitted.

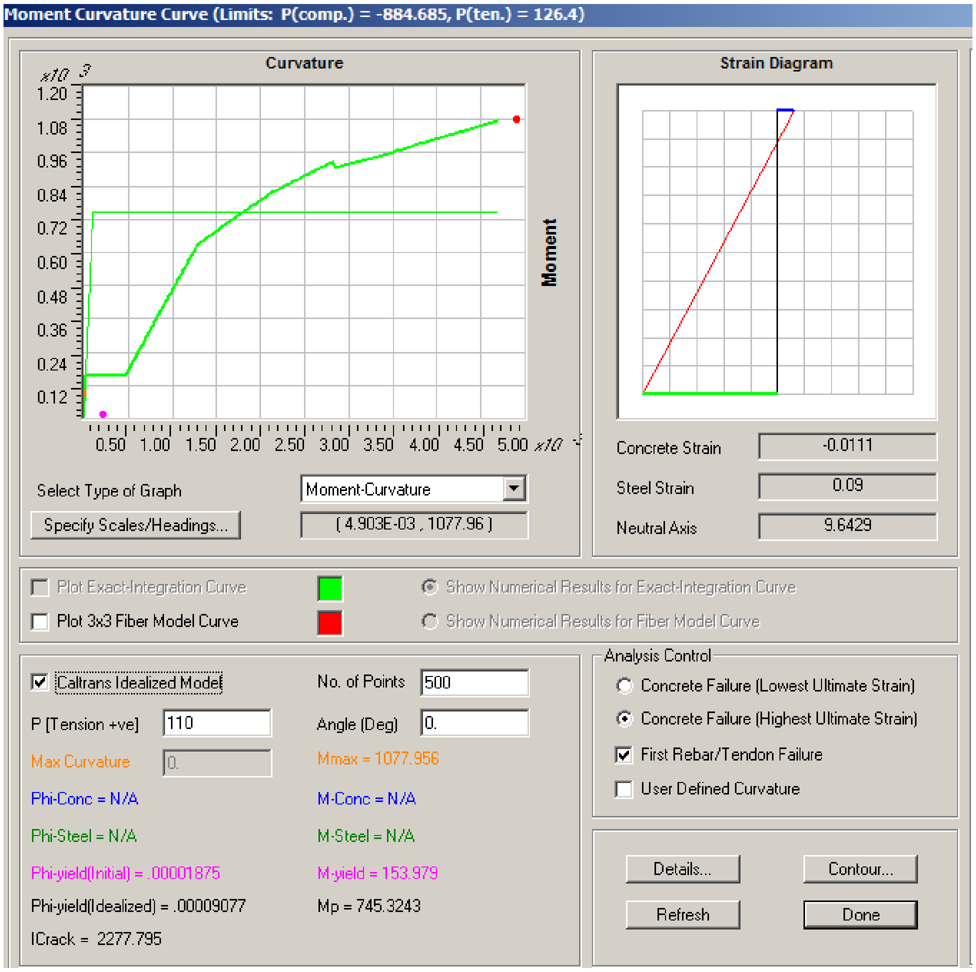


Hanger modeling (iteration only):





Hanger Moment Curvature Strong Axis:



QUINCY ENGINEERING, INC.

Project: Rumsey
Job No.
BY JC DATE 5/12/2014

Description: **Fiber Reinforcement Polymer Hanger Strong axis (Retrofitted)**

SHEET

Concrete Section input:

Overall, h [in] = 20.00 in 1.67 ft
d [in] = 17.49 in 1.46 ft
Overall, b [in] = 15.00 in 1.25 ft
Concr f'c [ksi] = 2.5 ksi
Concrete ε_c = 0.003 conc strain limit

PS [yes, no] = no
phi_PS = 1.00
phi_non-ps = 0.90
ϕ = 0.9

Steel Reinforcement input:

Fy [ksi] = 40 ksi

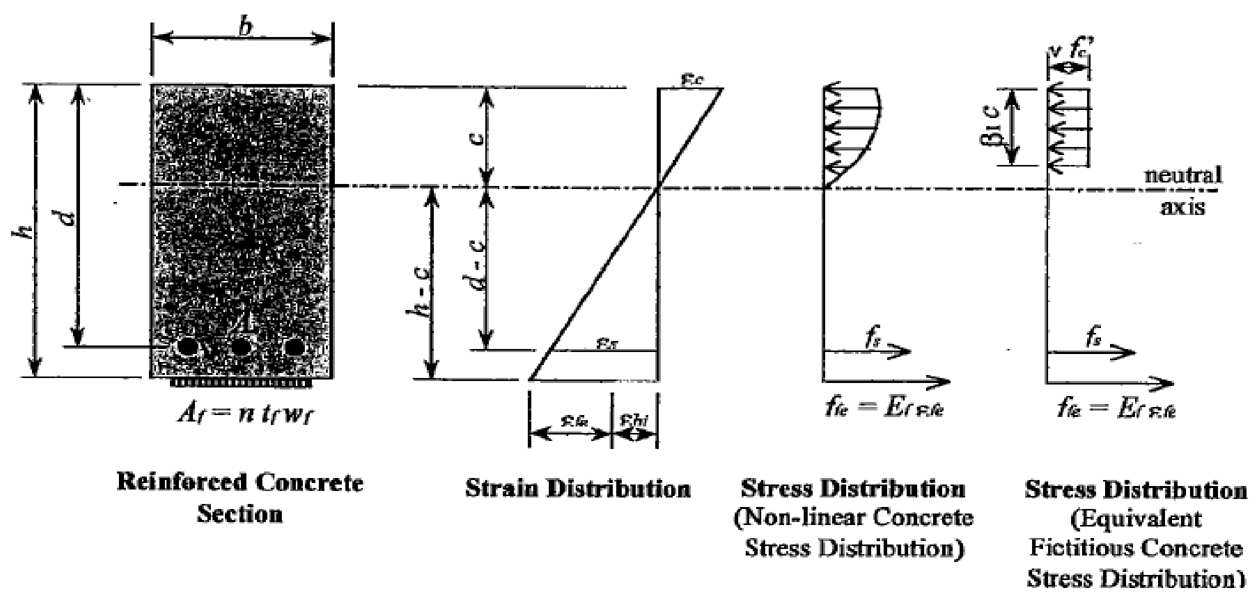
Main Bars # 10
Tot = 2
A_s [in^2] = 1.27 Photos shows 7 bars
d_s [in] = 1.27 Main Reinforcement
As [in^2] = 2.54 in sq

Shear Bars # 3
A_v [in^2] = 0.11
d_v [in] = 0.375 Shear Confinement
clr [in] = 1.5 in assumed

Manufacture FRP input:

Design limits:
Ult Tensile Strength 121 ksi
in Primary Direction = 121,000 psi
4.8 kip/in
Elongation at Break = 0.60%
Tensile Modulus = 1.19E+07 psi
1.19E+04 ksi

n, layers [ea] = 6 number of layers of FRP reinforcement
t_f [in] = 0.04 in FRP composite material thickness per layer
Total thickness 0.24 in
w_f [in] = 15.00 in width of FRP reinfocing layers



ϕ = 0.9, if $e_s \geq 0.005$
ϕ = interp if $0.002 \leq e_s < 0.005$
ϕ = 0.65, if otherwise

Assumptions:
Plain section remains plane
Max compressive strain in concrete is 0.003
Stress in steel under service load should be limited to 80% of yield strength

FRP Analysis:

Forces:

F_Conc = 0.85*fc*a*b
251 kip

F_steel = As*fy =
102 kip

$F_{frp} = A_{frp} * f_{fe}$ = **149 kip**
$f_{fe} = \varepsilon_{fe} * E_f$ = **41 ksi**
$A_f = n * t_f * w_f$ = 3.6 in sq
RFP within allowable stress

Strains: ε_c / c = = ε_s / (d - c) = = = ε_fe / (h - c)
(similar triangles)
Concrete ε_c = 0.003
Steel ε_s = 0.0027
Yeild ε_y = 0.0021
steel yields
RFP ε_{fe} = 0.0035
RFP within allowable stain

Neutral Axis:
c [in] = 9.26 in calculated distance between N.A. to extreme conc fiber

F_Conc = F_steel + F_frp
0 = - F_Conc + F_steel + F_frp
0 kip

beta = 0.85 (AASHTO 5.7.2.2) function of f'c
a [in] = 7.9 in vertical distance of whitney stress block

Moment Arm:
Conc Arm = (c - a / 2)
5.32 in
Steel Arm = (d - c)
8.23 in
Fiber Arm = (h - c)
10.74 in

Nominal Moment:
Conc Contr = 1,335 kip-in
Steel Contr = 837 kip-in
Fiber Contr = 1,603 kip-in

Nominal Moment =
3,774 kip-in
315 kip-ft

Factored Moment Strength:

ϕ Mn [k-in] =	3,397 kip-in	**>**	**Demand =**	3,350 kip-in	**ok**	D/C= 0.99
ϕ Mn [k-ft] =	283 kip-ft	>	Demand =	279 kip-ft	ok	

Analysis Existing (without FRP) - Strong Axis (informational only):

F_Conc = 0.85*fc*a*b
102 kip
F_steel = As*fy =
102 kip

a [in] = 3.19 in
beta = 0.85 (AASHTO 5.7.2.2)
c [in] = 3.75 in = a/beta

Check Steel Yield
e_s = 0.0110 > e_y = 0.0021
steel yields

Arm [in] = 15.90 in

ϕ Mn [k-in] =	**1,454 kip-in**	<	Demand =	3,350	NG	D/C= 2.30
ϕ Mn [k-ft] =	121 kip-ft	<	Demand =	279 kip-ft	NG	

QUINCY ENGINEERING, INC.

Project:	Rumsey		Description	**Fiber Reinforcement Polymer Hanger**	SHEET
Job No.				**Shear & Axial (Retrofitted)**	
BY	JC	DATE 5/12/2014			

Analysis Shear Strength (with FRP) - AlongStrong Axis:

$\phi V_{n,\ T\&B}$ [k] =	$(A_{fv} F_{fe} (\sin(\alpha) + \cos(\alpha))\ d_{fv}) / s_f$					Shear Bars	# 3	Additional Shear RFP added			
						d, depth of strup	17.75 in	n, layers [ea] =	1	number of layers of Shear FRP	
	682 kip		1.28			Spacing	12.00 in	$A_{fv} = 2\ n\ t_f\ w_f$	30 in sq		
Vc = 2*sqrt(fc')*b*d		(LDF 8.16.6.2)				mult by 2 sides	2	F_{fe} , FRP eff stress=	15.2 ksi		
	26 kip		Vs_limit = 8*SQRT(fc')*b*d/1000			Fy [ksi] =	40 ksi	α , shear angle=	20 degrees	= estimated, conservative	
Vs = Av*fy*d/s	*13 kip*	(LDF 8.16.6.3)	*105 kip*			*ϕ (LDF 8.16.1.2.2) =*	0.85	d_{fv} , eff FRP depth=	14 in	= b - 1"	
V_s =	***13 kip***							s_f , FRP spacing=	12.00 in		
V_Exist [kip] =	*39 kip*	**>**	**Demand =**	35 kip	**ok**	D/C= 0.89		Strength Flexural =	15.2 ksi	psi	
V_Retro [kip] =	**721 kip**	**>**	**Demand =**	35 kip	**ok**	D/C= 0.05		Flexural Modulus =	384.2 ksi	psi	

Analysis Axial Tensile Strength:

ϕP_n [k] =	= As n Fy					Main Bars	# 10
	203 kip					Tot =	4
ϕ =	0.9					A_s [in^2] =	1.27
						Fy [ksi] =	40 ksi
ϕP_n [k] =	*183 kip*	**>**	**Demand =**	110 kip	**ok**	D/C= 0.60	

QUINCY ENGINEERING, INC.

Project:	**Rumsey**	Description	Fiber Reinforcement Polymer Hanger	SHEET
Job No.			**Weak axis (Retrofitted)**	
BY	**JC**	DATE **5/12/2014**		

Concrete Section input:

Overall, h [in] =	15.00 in	1.25 ft
d [in] =	12.49 in	1.04 ft
Overall, b [in] =	20.00 in	1.67 ft
Concr f'c [ksi] =	2.5 ksi	
Concrete ε_c =	0.003	conc strain limit
PS [yes, no] =	no	
phi_PS =	1.00	
phi_non-ps =	0.90	
ϕ =	0.9	

Steel Reinforcement input:

Fy [ksi] =	40 ksi	
Main Bars	# 10	
Tot =	2	
A_s [in^2] =	1.27	Photos shows 7 bars
d_s [in] =	1.27	Main Reinforcement
As [in^2] =	2.54 in sq	
Shear Bars	# 3	
A_v [in^2] =	0.11	
d_v [in] =	0.375	Shear Confinement
clr [in] =	1.5 in	assumed

Manufacture FRP input:

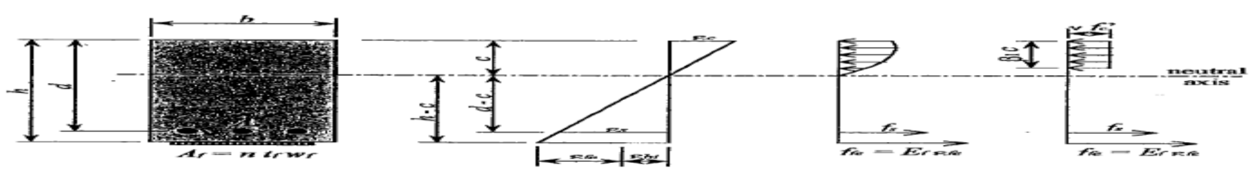
Design limits:

Ult Tensile Strength	121	ksi
in Primary Direction =	121,000	psi
	4.8	kip/in
Elongation at Break =	0.60%	
Tensile Modulus =	1.19E+07	psi
	1.19E+04	ksi
n, layers [ea] =	3	number of layers of FRP reinforcement
t_f [in] =	0.04 in	FRP composite material thickness per layer
Total thickness	0.12 in	
w_f [in] =	20.00 in	width of FRP reinfocing layers

ϕ = 0.9, if $e_s \geq 0.005$

ϕ = interp if $0.002 \leq e_s < 0.005$

ϕ = 0.65, if otherwise



Reinforced Concrete Section — Strain Distribution — Stress Distribution (Non-linear Concrete Stress Distribution) — Stress Distribution (Equivalent Fictitious Concrete Stress Distribution)

Assumptions:

Plain section remains plane

Max compressive strain in concrete is 0.003

Stress in steel under service load should be limited to 80% of yield strength

FRP Analysis:

Forces:

F_Conc = 0.85*fc*a*b — **224 kip**

F_steel = As*fy = **51 kip**

$F_{frp} = A_{frp} * f_{fe}$ = **122 kip**

$f_{fe} = \varepsilon_{fe} * E_f$ = **51 ksi**

$A_f = n * t_f * w_f$ = 2.4 in sq

RFP within allowable stress

Strains: ε_c / c = = ε_s / (d - c) = = = ε_fe / (h - c)

(similar triangles)

Concrete ε_c = 0.003 — Steel ε_s = 0.0031 — RFP ε_{fe} = 0.0043

Yeild ε_y = 0.0021

steel yields — **RFP within allowable stain**

Neutral Axis:

c [in] = 6.19 in — calculated distance between N.A. to extreme conc fiber

F_Conc = F_steel + F_frp

0 = - F_Conc + F_steel + F_frp

0 kip

beta = 0.85 — (AASHTO 5.7.2.2) function of f'c

a [in] = 5.3 in — vertical distance of whitney stress block

Moment Arm:

Conc Arm = (c - a / 2) = 5.32 in — Steel Arm = (d - c) = 8.23 in — Fiber Arm = (h - c) = 5.74 in

Nominal Moment:

Conc Contr = 1,190 kip-in — Steel Contr = 418 kip-in — Fiber Contr = 701 kip-in

Nominal Moment = 2,309 kip-in

192 kip-ft

Factored Moment Strength:

ϕ Mn [k-in] =	2,078 kip-in	**>**	**Demand =**	1,900 kip-in	**ok**	D/C= 0.91
ϕ Mn [k-ft] =	173 kip-ft	>	Demand =	158 kip-ft	ok	

Analysis Existing (without FRP) - Weak Axis (informational only):

F_Conc = 0.85*fc*a*b — 102 kip

F_steel = As*fy = 102 kip

a [in] = 2.39 in

beta = 0.85 (AASHTO 5.7.2.2)

c [in] = 2.81 in = a/beta

Check Steel Yield

e_s = 0.0157 > e_y = 0.0021

steel yields

Arm [in] = 16.29 in

ϕ Mn [k-in] =	1,490 kip-in	<	Demand =	1,900	NG	D/C= 1.28
ϕ Mn [k-ft] =	124 kip-ft	<	Demand =	158 kip-ft	NG	

QUINCY ENGINEERING, INC.

Project:	**Rumsey**	Description	**Vertical Hanger**
Job No.	**Y01-500**		**Rebar Tension Yield**
BY	**JC**	DATE	**5/12/2014**

SHEET

Rebar Steel Properties

Main Bars	1.125	Main Reinforcement
A_s [in^2] =	1.27	
Tot =	5	
A_s [in^2] =	6.33	
Fy	40	ksi

Tension

T_n = Fy * As

T_n	253	kips
ϕT_n	253	kips

Demand

P_u	115	kips

D/C **0.45**

QUINCY ENGINEERING, INC.

Project:	**Rumsey**	Description	**Vertical Hanger**
Job No.	**Y01-500**	**Shear yy (LRDF 5.8.3.3)**	SHEET
BY	**JC**	DATE	**5/12/2014**

LRFD 5.8.3.3

Vn = minium of the following 2 equations
Vn = Vc + Vs + Vp
Vn = 0.25f'c bv dv + Vp

Ecco Results:
Vn = 1 kip < Demand = 15 kip **NG**
15.00

b =	15 in		Fy [ksi] =	40 ksi	
d =	20 in				
Ac =	300 in2		Shear Bars	# 3	
f'c [ksi] =	0,000 psi	0.0 ksi worst case since member in tension	Spacing	12.00 in	spacing of stirrup
			A_v_bar =	0.11 in2	
bv =	15 in	eff web width	d_v_bar =	0.38 in	
dv =	17 in	eff shear depth	mult by	2	number of bars per plane
			Av	0.02 in	area of shear reinfocement w/in dist s

5.8.2.5 Minimum Transverse Reinforcement

Av > 0.0316 sqrt (f'c) b_v s / fy

Av = 0.02 in2 ok 0.00 in2 = 0.0316 sqrt (f'c) bv s / fy

5.8.3.4.2 General Procedure

es = (Mu/dv + 0.5 Nu + |Vu-Vp| - Apsfpo) / (Es As + Ep Aps)

Mu =	32184 k-ft	Moment demand			
Nu =	653 k	Axial demand			
Vu =	441 k	Shear demand			
Vp =	0 k	component in direction of applied shear of the effective prestressing force positive if resisting the applied shear			
Aps =	0.00 in2	Area of PS			
fpo =	175 ksi	0.7 fpu		Flexural steel info:	
Es =	29000 ksi			Fy [ksi] =	40 ksi
Ep =	29000 ksi			Flex Bars	# 10
As =	8.89 in2	flexural steel		A_s_bar =	1.27 in2
				d_s_bar =	1.27 in
es =	0.0103			mult by	7

Sxe = sx * 1.38 / (ag + 0.63)

sx =	min of following	17 in	
	dv =	17 in	
	d__ =	36 in	dist between layers of long crack control reinf
ag =	0.25 in	max agg size	assumed per photos
Sxe =	26.7		

theta = 29 + 3500 es
theta = 65 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met
beta = 4.8 / (1+ 750 εs)
beta = 0.55 factor indicating ability of diagonal cracked concrete to transmit tension & shear
If Min Transverse Reinforcement is NOT met
beta = 4.8 / (1+ 750 εs) x 51 / (39 + Sxe)
beta = 0.43 factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select:	beta =	0.4 degrees	factor indicating ability of diagonal cracked concrete to transmit tension & shear
	theta =	65 degrees	angle of inclination of diagonal comp. stress
Select:	alpha =	90 degrees	angle of inclination of transverse reinforcemen

Vn = Vc + Vs + Vp
Vc = 0.0316*beta*sqrt(f'c) bv dv
Vc = 0 kip
Vs = Av fy sin(alpha) < 0.095 sqrt(f'c) bv dv
Vs = 0.66 kip < 0 kip
Vn = 1 kip **Gov**

Vn = 0.25f'c bv dv + Vp
Vn = 0 kip

ϕ = 0.90 Seimsic phi for shear

Select: **ϕ V [kip] =** **1 kip** < Demand = 15 kip NG
15.00

QUINCY ENGINEERING, INC.

Project: **Rumsey** Description **Vertical Hanger**
Job No. **Y01-500** **Shear xx (LRDF 5.8.3.3)** SHEET
BY **JC** DATE **5/12/2014**

LRFD 5.8.3.3
Vn = minium of the following 2 equations
Vn = Vc + Vs + Vp
Vn = 0.25f'c bv dv + Vp

Ecco Results:
f V [kip] = 1 kip < Demand = 30 kip **NG**
30.00

b =	20 in		Fy [ksi] =	40 ksi	
d =	15 in				
Ac =	300 in2		Shear Bars	# 4	
f'c [ksi] =	0,000 psi	0.0 ksi worst case since member in tension	Spacing	18.00 in	spacing of stirrup
			A_v_bar =	0.20 in2	
bv =	20 in	eff web width	d_v_bar =	0.50 in	
dv =	12 in	eff shear depth	mult by	2	number of bars per plane
			Av	0.02 in	area of shear reinfocement w/in dist s

5.8.2.5 Minimum Transverse Reinforcement
Av > 0.0316 sqrt (f'c) b_v s / fy
Av = 0.02 in2 ok 0.00 in2 = 0.0316 sqrt (f'c) bv s / fy

5.8.3.4.2 General Procedure
es = (Mu/dv + 0.5 Nu + |Vu-Vp| - Apsfpo) / (Es As + Ep Aps)

Mu =	13063 k-ft	Moment demand
Nu =	653 k	Axial demand
Vu =	263 k	Shear demand
Vp =	0 k	component in direction of applied shear of the effective prestressing force positive if resisting the applied shear
Aps =	0.00 in2	Area of PS
fpo =	175 ksi	0.7 fpu
Es =	29000 ksi	
Ep =	29000 ksi	
As =	5.08 in2	flexural steel
es =	0.0114	

Flexural steel info:

Fy [ksi] =	40 ksi	
Flex Bars	# 10	
A_s_bar =	1.27 in2	
d_s_bar =	1.27 in	
mult by	4	approx effective

Sxe = sx * 1.38 / (ag + 0.63)
sx = min of following 12 in
dv = 12 in
d__ = 36 in dist between layers of long crack control reinf
ag = 0.25 in max agg size assumed per photos
Sxe = 18.8

theta = 29 + 3500 es
theta = 69 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met
beta = 4.8 / (1+ 750 εs)
beta = 0.50 factor indicating ability of diagonal cracked concrete to transmit tension & shear
If Min Transverse Reinforcement is NOT met
beta = 4.8 / (1+ 750 εs) x 51 / (39 + Sxe)
beta = 0.45 factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select: beta = 0.5 degrees factor indicating ability of diagonal cracked concrete to transmit tension & shear
theta = 69 degrees angle of inclination of diagonal comp. stress
Select: alpha = 90 degrees angle of inclination of transverse reinforcemen

Vn = Vc + Vs + Vp
Vc = 0.0316*beta*sqrt(f'c) bv dv
Vc = 0 kip
Vs = Av fy sin(alpha) < 0.095 sqrt(f'c) bv dv
Vs = 0.79 kip < 0 kip
Vn = 1 kip Gov

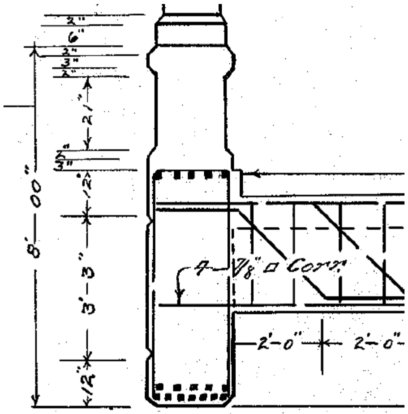
Vn = 0.25f'c bv dv + Vp
Vn = 0 kip

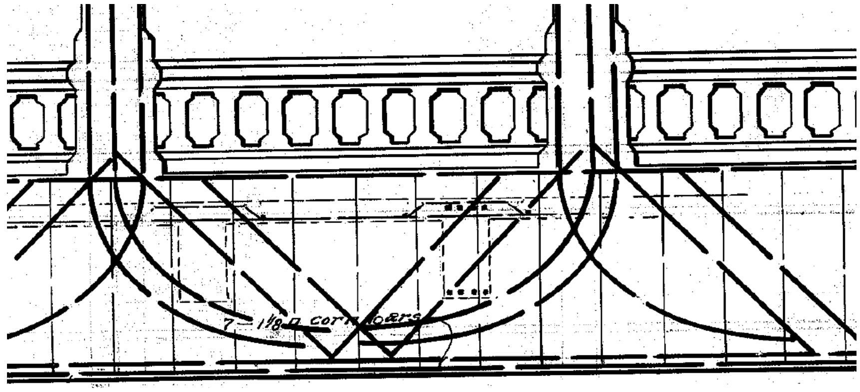
ϕ = 0.90 Seimsic phi for shear

Select: **ϕ V [kip] = 1 kip** < Demand = 30 kip NG
30.00

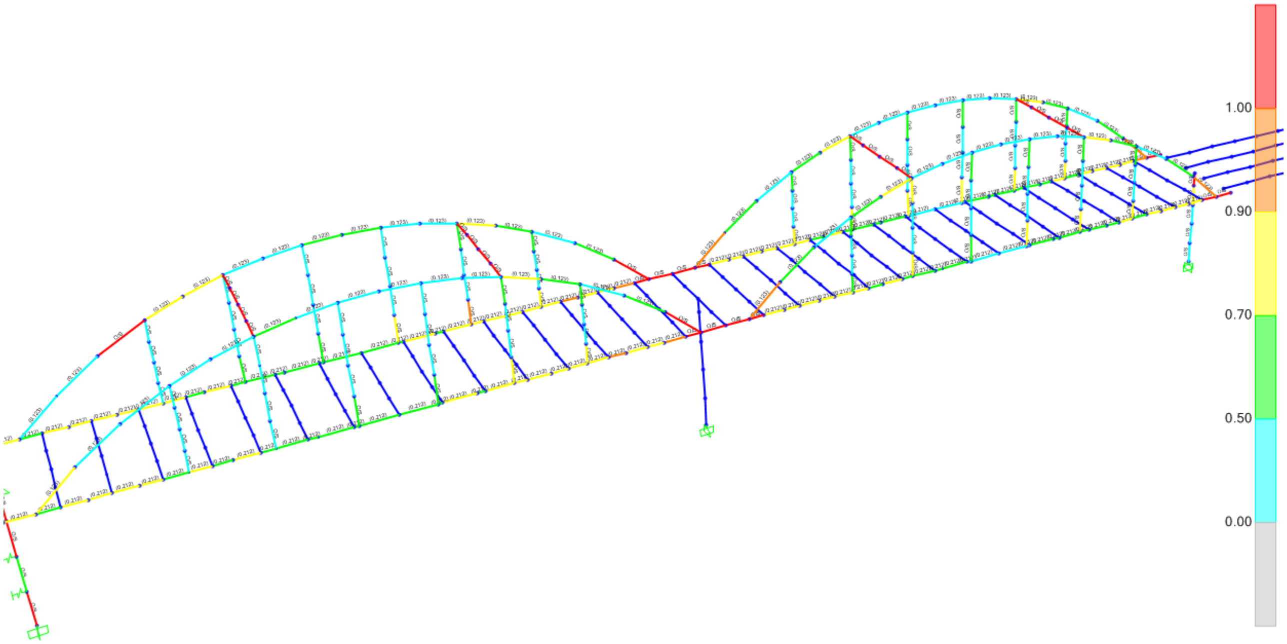
Tie Girder Retrofit:

As-built details of Girder-Tie sections & reinforcement:

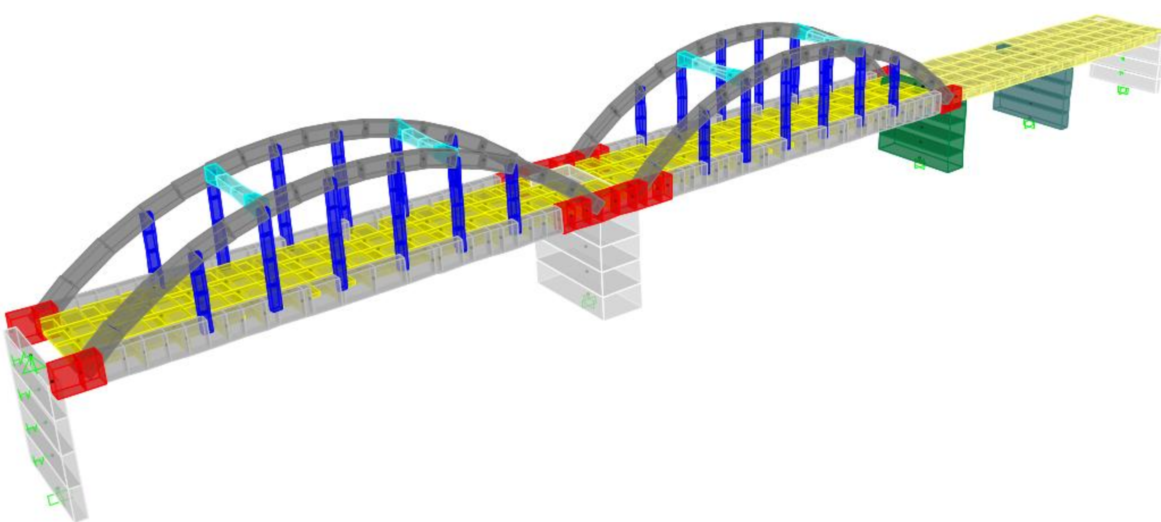




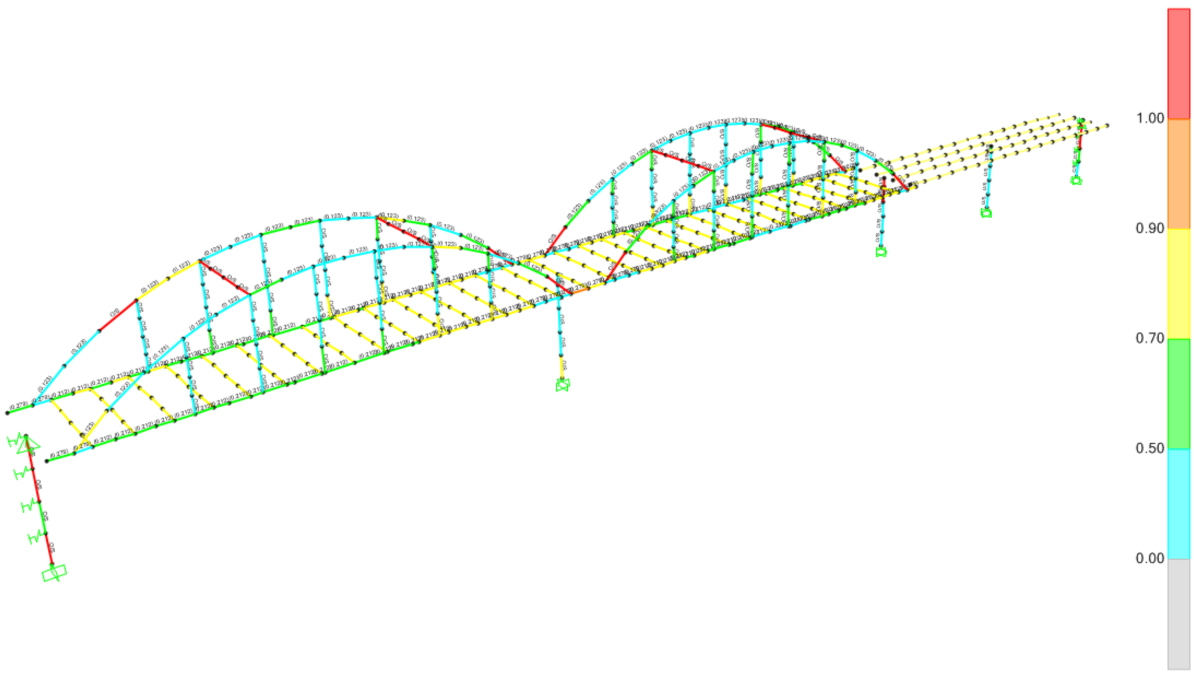
Analysis below shows the limits where the Girder-Ties exceed D/C ratio of 1.0 in Red:



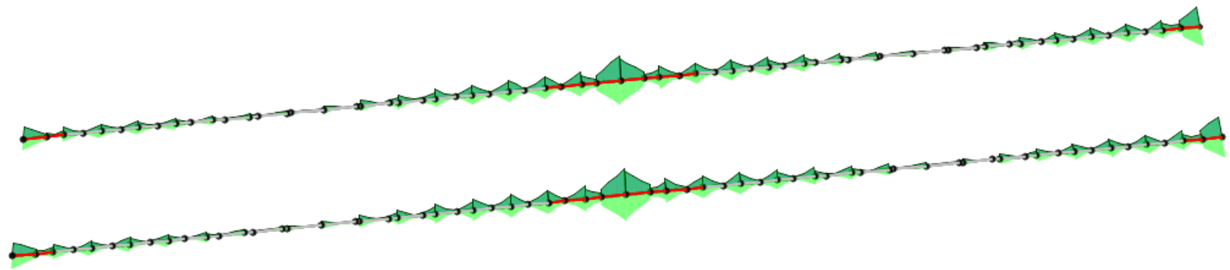
Red block below indicates the approximate location of where the Girder Ties will be retrofitted.



Screenshot below indicates that the retrofitted Girder-Ties all have D/C’s lower than 1.0.



Moment diagram below indicates the weak access moments on the Girder-Ties.



Moment diagram below indicates the Strong access moments on the Girder-Ties.

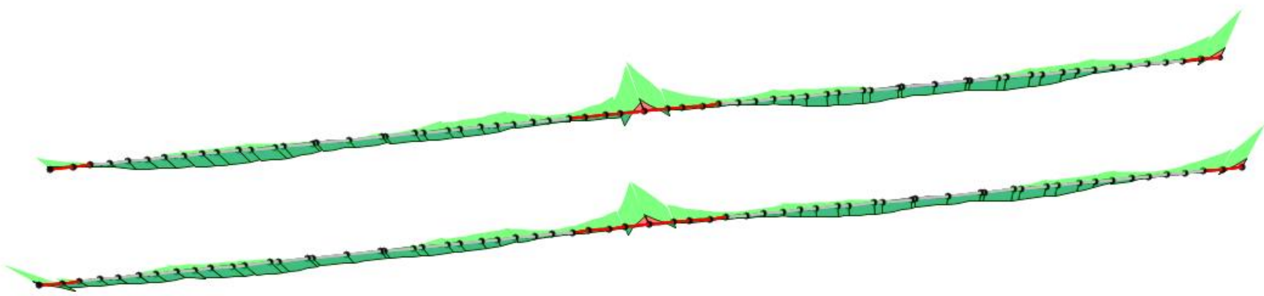


Photo of existing Girder-Ties.

QUINCY ENGINEERING, INC.

Project:	**Rumsey**		Description	**Bending**	
Job No.				**Girder Tie Strong Axis xx**	SHEET
BY	**JC**	DATE	**5/12/2014**		

D [in] =	63.00 in	5.25 ft				
d [in] =	60.44 in	5.04 ft	Fy [ksi] =	40 ksi		
b [in] =	23.00 in	1.92 ft				
fc [ksi] =	2.5 ksi		Main Bars	1 1/8 SQ		35.3
			Tot =	12		25.2
PS [yes, no] =	no		A_s [in^2] =	1.27		
phi_PS =	1.00		d_s [in] =	1.125	Main Reinforcement	
phi_non-ps =	0.90		As [in^2] =	15.1875		
ϕ =	1.00	for seismic				
			Shear Bars	# 4		
			A_v [in^2] =	0.2		
			d_v [in] =	0.5	Shear Confinement	
			clr [in] =	1.5		

Analysis:

F_steel =	As*fy =		F_Conc =	0.85*fc*a*b		
	608 kip			608 kip		
a [in] =	12.43 in					
beta =	0.85	(AASHTO 5.7.2.2)				
x [in] =	a/beta					
	14.62 in					
Check Steel Yield						
e_s =	0.0094	>	e_y =	0.0021		
		steel yields				
Arm [in] =	54.22 in					
ϕ Mn [k-in] =	32,940 kip-in	>	Demand =	31,025 kip-in	ok	D/C= 0.94
ϕ Mn [k-ft] =	2,745 kip-ft	>	Demand =	2,585 kip-ft	ok	

QUINCY ENGINEERING, INC.

Project:	Rumsey	Description	Bending
Job No.			Girder Tie Strong Axis yy
BY	JC	DATE	5/12/2014

SHEET

D [in] =	23.00 in	1.92 ft					
d [in] =	20.44 in	1.70 ft		Fy [ksi] =	40 ksi		
b [in] =	63.00 in	5.25 ft					
fc [ksi] =	2.5 ksi			Main Bars	1 1/8 SQ		11.9
				Tot =	2		8.5
PS [yes, no] =	no			A_s [in^2] =	1.27		
phi_PS =	1.00			d_s [in] =	1.125	Main Reinforcement	
phi_non-ps =	0.90			As [in^2] =	2.53125		
ϕ =	1.00	for seismic					
				Shear Bars	# 4		
				A_v [in^2] =	0.2		
				d_v [in] =	0.5	Shear Confinement	
				clr [in] =	1.5		

Analysis:

F_steel =	As*fy =		F_Conc =	0.85*fc*a*b		
	101 kip			101 kip		
a [in] =	0.76 in					
beta =	0.85	(AASHTO 5.7.2.2)				
x [in] =	a/beta					
	0.89 in					
Check Steel Yield						
e_s =	0.0659	>	e_y =	0.0021		
		steel yields				
Arm [in] =	20.06 in					
ϕ Mn [k-in] =	2,031 kip-in	<	Demand =	3,229 kip-in	NG	D/C= 1.59
ϕ Mn [k-ft] =	169 kip-ft	<	Demand =	269 kip-ft	NG	

QUINCY ENGINEERING, INC.

Project: Rumsey
Job No.
BY JC DATE 5/12/2014

Description: **Fiber Reinforcement Polymer**
Arch Rib
Strong axis (no FRP)

SHEET

Concrete Section input:

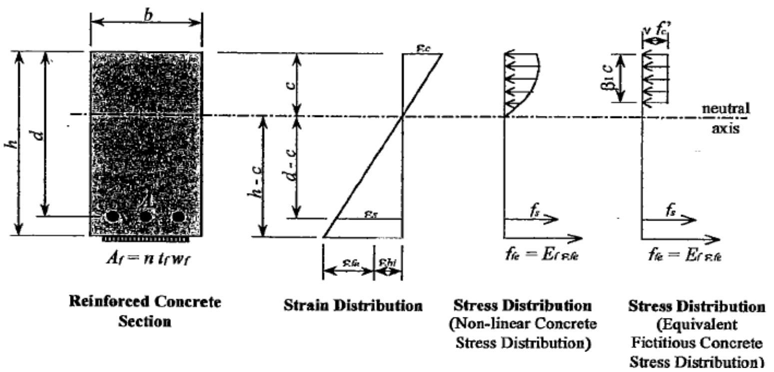
Overall, h [in] =	63.00 in	5.25 ft
d [in] =	60.44 in	5.04 ft
Overall, b [in] =	48.00 in	4.00 ft
Concr f'c [ksi] =	2.5 ksi	
Concrete ε_c =	0.003	conc strain limit
PS [yes, no] =	no	
phi_PS =	1.00	
phi_non-ps =	0.90	
ϕ =	0.9	

Steel Reinforcement input:

Fy [ksi] =	40 ksi	
Main Bars	1 1/8 SQ	conservative of 1 1/8" SQ bar
Tot =	12	conservative base on per as-built.
A_s [in^2] =	1.27	Photos shows 7 bars
d_s [in] =	1.125	Main Reinforcement
As [in^2] =	15.2 in sq	
Shear Bars	# 4	
A_v [in^2] =	0.2	
d_v [in] =	0.5	Shear Confinement
clr [in] =	1.5 in	assumed

Manufacture FRP input:

	Design limits:		
Ult Tensile Strength	121	ksi	
in Primary Direction =	121,000	psi	
	4.8	kip/in	
Elongation at Break =	0.60%		
Tensile Modulus =	1.19E+07	psi	
	1.19E+04	ksi	
n, layers [ea] =	0		number of layers of FRP reinforcement
t_f [in] =	0.04 in		FRP composite material thickness per layer
Total thickness	0.00 in		
w_f [in] =	48.00 in		width of FRP reinfocing layers



$\phi = 0.9$, if $e_s \geq 0.005$
ϕ = interp if $0.002 \leq e_s < 0.005$
$\phi = 0.65$, if otherwise

Assumptions:
Plain section remains plane
Max compressive strain in concrete is 0.003
Stress in steel under service load should be limited to
80% of yield strength

FRP Analysis:

Forces:

F_Conc = 0.85*fc*a*b
608 kip

F_steel = As*fy =
608 kip

$F_{frp} = A_{frp} * f_{fe}$ = **0 kip**
$f_{fe} = \varepsilon_{fe} * E_f$ = **285 ksi**
$A_f = n * t_f * w_f$ = 0 in sq
RFP design tensile stress limit exceeded!!!

Strains: ε_c / c = = ε_s / (d - c) = = = ε_fe / (h - c)
(similar triangles)

Concrete ε_c = 0.003
Steel ε_s = 0.0229
Yeild ε_y = 0.0021
steel yields
RFP ε_{fe} = 0.0240
RFP design longation limit exceeded!!!

Neutral Axis:
c [in] = 7.01 in calculated distance between N.A. to extreme conc fiber

F_Conc = F_steel + F_frp
0 = - F_Conc + F_steel + F_frp
0 kip

beta = 0.85 (AASHTO 5.7.2.2) function of f'c
a [in] = 6.0 in vertical distance of whitney stress block

Moment Arm:
Conc Arm = (c - a / 2)
4.03 in
Steel Arm = (d - c)
53.43 in
Fiber Arm = (h - c)
55.99 in

Nominal Moment:
Conc Contr = 2,448 kip-in
Steel Contr = 32,459 kip-in
Fiber Contr = 0 kip-in

Nominal Moment =
34,907 kip-in
2,909 kip-ft

Factored Moment Strength:

ϕ Mn [k-in] =	31,416 kip-in	**>**	**Demand =**	26,400 kip-in	**ok**	D/C= 0.84
ϕ Mn [k-ft] =	2,618 kip-ft	>	Demand =	2,200 kip-ft	ok	

Analysis Existing (without FRP) - Strong Axis:

F_Conc = 0.85*fc*a*b
608 kip

F_steel = As*fy =
608 kip

a [in] = 5.96 in
beta = 0.85 (AASHTO 5.7.2.2)
c [in] = 7.01 in = a/beta

Check Steel Yield
e_s = 0.0229 > e_y = 0.0021
steel yields

Arm [in] = 57.46 in

ϕ Mn [k-in] =	**31,416 kip-in**	>	Demand =	26,400	ok	D/C= 0.84
ϕ Mn [k-ft] =	2,618 kip-ft	>	Demand =	2,200 kip-ft	ok	

QUINCY ENGINEERING, INC.

Project: **Rumsey**
Job No.
BY **JC** DATE **5/12/2014**

Description: Fiber Reinforcement Polymer
Arch Rib
Weak axis (Retrofitted)

SHEET

Concrete Section input:

Overall, h [in] =	48.00 in	4.00 ft
d [in] =	45.44 in	3.79 ft
Overall, b [in] =	63.00 in	5.25 ft
Concr f'c [ksi] =	2.5 ksi	
Concrete ε_c =	0.003	conc strain limit
PS [yes, no] =	no	
phi_PS =	1.00	
phi_non-ps =	0.90	
ϕ =	0.9	

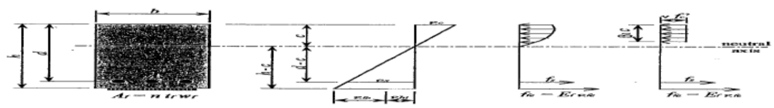
Steel Reinforcement input:

Fy [ksi] =	40 ksi	
Main Bars	1 1/8 SQ	conservative of 1 1/8" SQ bar
Tot =	2	conservative base on per as-built.
A_s [in^2] =	1.27	
d_s [in] =	1.125	
As [in^2] =	2.53 in sq	
Shear Bars	# 4	
A_v [in^2] =	0.2	
d_v [in] =	0.5	Shear Confinement
clr [in] =	1.5 in	assumed

Manufacture FRP input:

	Design limits:	
Ult Tensile Strength	121	ksi
in Primary Direction =	121,000	psi
	4.8	kip/in
Elongation at Break =	0.60%	
Tensile Modulus =	1.19E+07	psi
	1.19E+04	ksi
n, layers [ea] =	2	number of layers of FRP reinforcement
t_f [in] =	0.04 in	FRP composite material thickness per layer
Total thickness	0.08 in	
w_f [in] =	63.00 in	width of FRP reinfocing layers

ϕ = 0.9, if $e_s \geq 0.005$
ϕ = interp if $0.002 \leq e_s < 0.005$
ϕ = 0.65, if otherwise



Reinforced Concrete Section | **Strain Distribution** | **Stress Distribution (Non-linear Concrete Stress Distribution)** | **Stress Distribution (Equivalent Fictitious Concrete Stress Distribution)**

Assumptions:
Plain section remains plane
Max compressive strain in concrete is 0.003
Stress in steel under service load should be limited to 80% of yield strength

FRP Analysis:

Forces:
F_Conc = 0.85*fc*a*b
1,853 kip

F_steel = As*fy =
51 kip

$F_{frp} = A_{frp} * f_{fe}$ = **350 kip**
$f_{fe} = \varepsilon_{fe} * E_f$ = **70 ksi**
$A_f = n * t_f * w_f$ = 5.04 in sq
RFP within allowable stress

Strains:
(similar triangles)
ε_c / c = = ε_s / (d - c) = = = ε_fe / (h - c)

Concrete ε_c = 0.003
Steel ε_s = 0.0054
Yeild ε_y = 0.0021
steel yields
RFP ε_{fe} = 0.0058
RFP within allowable stain

Neutral Axis:
c [in] = 16.29 in calculated distance between N.A. to extreme conc fiber

F_Conc = F_steel + F_frp
0 = - F_Conc + F_steel + F_frp
-1,402 kip

beta = 0.85 (AASHTO 5.7.2.2) function of f'c
a [in] = 13.8 in vertical distance of whitney stress block

Moment Arm:
Conc Arm = (c - a / 2)
4.03 in
Steel Arm = (d - c)
53.43 in
Fiber Arm = (h - c)
40.99 in

Nominal Moment:
Conc Contr = 7,467 kip-in
Steel Contr = 2,705 kip-in
Fiber Contr = 14,364 kip-in

Nominal Moment =
24,535 kip-in
2,045 kip-ft

Factored Moment Strength:

ϕ Mn [k-in] =	22,081 kip-in	**>**	**Demand =**	13,800 kip-in	**ok**	D/C= 0.62
ϕ Mn [k-ft] =	1,840 kip-ft	>	Demand =	1,150 kip-ft	ok	

Analysis Existing (without FRP) - Weak Axis:

F_Conc = 0.85*fc*a*b
101 kip
F_steel = As*fy =
101 kip

a [in] = 0.76 in
beta = 0.85 (AASHTO 5.7.2.2)
c [in] = 0.89 in = a/beta

Check Steel Yield
e_s = 0.2008 > e_y = 0.0021
steel yields

Arm [in] = 60.06 in

ϕ Mn [k-in] =	5,473 kip-in	<	Demand =	13,800	NG	D/C= 2.52
ϕ Mn [k-ft] =	456 kip-ft	<	Demand =	1,150 kip-ft	NG	

QUINCY ENGINEERING, INC.

Project:	Rumsey	Description	Girder Tie
Job No.	Y01-500		Concrete Axial (LRFD)
BY	JC	DATE 5/12/2014	(LRFD 5.7.4)

SHEET

Compression Member Strength

fc' =	2500	psi
t =	63	in
b =	23	in
d =	60.365	in
d' =	2.635	in
As =	7.62	in2
As' =	7.62	in2
fy =	40,000	psi
beta1 =	0.85	
Pu =	635 kip	Demand

Main Bars	# 10	Main Reinforcement
Tot =	6	
A_s [in^2] =	1.27	
d_s [in] =	1.27	
As [in^2] =	7.62	
Shear Bars	# 4	Shear Confinement
Spacing [in] =	18	
A_v [in^2] =	0.2	
d_v [in] =	0.5	
clr [in] =	2	
Comp Bars	# 10	Main Reinforcement
Tot =	6	
A_s' [in^2] =	1.27	
d_s' [in] =	1.27	
As' [in^2] =	7.62	

Reinforcement of Compession Members (LRFD 5.7.4.2)

Max Reinforcement

Ast/Ag < 0.08

Ast/Ag =	(As+As_prime)/(t*b)			
Ast/Ag =	0.012	<	0.08	ok

Min Reinforcement

Ast/Ag > 0.0025 for pier wall, and 0.01 for comp. member

Ast/Ag =	(As+As_prime)/(t*b)			
Ast/Ag =	0.012	>	0.010	ok

Min bar # 5, Check

c-c < 12", Check

Pure Compression (LRFD 5.7.4.4)

Po = phi[0.80*fc' (*b*t - As - As') + (As + As')*fy] (8-30) for ties

ϕ =	1.00				
ϕ Po =	3,477,120	lbs			
ϕ Po =	3,477 kips		>	635 kip	ok
				0.18	

Lateral Reinforcement (BDS 8.18.2)

spacing > min(12", t=24"), Check

Greater than #3 if Long. bar <#10, Check

Ties (BDS 8.18.3)

h_wall =	276	in
Ash =	6.5333	in2
h_c =	58.5	in
Ag =	1449	in2
Ac =	1341	in2

0.30*s_t*h_c*fc' /fy *(Ag/Ac-1) =

1.59 in2

6.53 in2 > Demand = 1.59 in2 ok

QUINCY ENGINEERING, INC.

Project:	**Rumsey**	Description **Girder Tie**	
Job No.	**Y01-500**	**Shear (LRDF 5.8.3.3)**	SHEET
BY	**JC**	DATE **5/12/2014**	

LRFD 5.8.3.3

Vn = minium of the following 2 equations
Vn = Vc + Vs + Vp
Vn = 0.25f'c bv dv + Vp

Ecco Results:
f V [kip] = 29 kip < Demand = 129 kip **NG**
4.45

b =	63 in		Fy [ksi] =	40 ksi	
d =	23 in				
Ac =	1449 in2		Shear Bars	# 4	
f'c [ksi] =	2,500 psi	2.5 ksi	Spacing	18.00 in	spacing of stirrup
			A_v_bar =	0.20 in2	
bv =	63 in	eff web width	d_v_bar =	0.50 in	
dv =	20 in	eff shear depth	mult by	2	number of bars per plane
			Av	0.02 in	area of shear reinfocement w/in dist s

5.8.2.5 Minimum Transverse Reinforcement

Av > 0.0316 sqrt (f'c) b_v s / fy

Av = 0.02 in2 NG 1.42 in2 = 0.0316 sqrt (f'c) bv s / fy

Analysis:

5.8.3.4.2 General Procedure

es = (Mu/dv + 0.5 Nu + |Vu-Vp| - Apsfpo) / (Es As + Ep Aps)

Mu =	32184 k-ft	Moment demand
Nu =	653 k	Axial demand
Vu =	441 k	Shear demand
Vp =	0 k	component in direction of applied shear of the effective prestressing force positive if resisting the applied shear
Aps =	0.00 in2	Area of PS
fpo =	175 ksi	0.7 fpu
Es =	29000 ksi	
Ep =	29000 ksi	
As =	8.89 in2	flexural steel
es =	0.0092	

Flexural steel info:

Fy [ksi] =	40 ksi
Flex Bars	# 10
A_s_bar =	1.27 in2
d_s_bar =	1.27 in
mult by	7

Sxe = sx * 1.38 / (ag + 0.63)

sx =	min of following	20 in	
	dv =	20 in	
	d__ =	36 in	dist between layers of long crack control reinf
ag =	0.25 in	max agg size	assumed per photos
Sxe =	31.4		

theta = 29 + 3500 es
theta = 61 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met
beta = 4.8 / (1+ 750 εs)
beta = 0.61 factor indicating ability of diagonal cracked concrete to transmit tension & shear
If Min Transverse Reinforcement is NOT met
beta = 4.8 / (1+ 750 εs) x 51 / (39 + Sxe)
beta = 0.45 factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select:	beta =	0.4 degrees	factor indicating ability of diagonal cracked concrete to transmit tension & shear
	theta =	61 degrees	angle of inclination of diagonal comp. stress
Select:	alpha =	90 degrees	angle of inclination of transverse reinforcemen

Vn = Vc + Vs + Vp
Vc = 0.0316*beta*sqrt(f'c) bv dv
Vc = 28 kip
Vs = Av fy sin(alpha) < 0.095 sqrt(f'c) bv dv
Vs = 0.79 kip < 189 kip
Vn = 29 kip Gov

Vn = 0.25f'c bv dv + Vp
Vn = 788 kip

ϕ = 1.00 Seimsic phi for shear

Select: **ϕ V [kip] = 29 kip** < Demand = 129 kip NG
4.45

QUINCY ENGINEERING, INC.

Project:	**Rumsey**	Description **Girder Tie**		SHEET
Job No.	**Y01-500**	**Shear (LRDF 5.8.3.3)**		
BY	**JC**	DATE	**5/12/2014**	

LRFD 5.8.3.3

Vn = minium of the following 2 equations
Vn = Vc + Vs + Vp
Vn = 0.25f'c bv dv + Vp

Ecco Results:
f V [kip] = 36 kip < Demand = 69 kip **NG**
1.91

b =	23 in		Fy [ksi] =	40 ksi	
d =	63 in				
Ac =	1449 in2		Shear Bars	# 4	
f'c [ksi] =	2,500 psi	2.5 ksi	Spacing	18.00 in	spacing of stirrup
			A_v_bar =	0.20 in2	
bv =	23 in	eff web width	d_v_bar =	0.50 in	
dv =	60 in	eff shear depth	mult by	2	number of bars per plane
			Av	0.02 in	area of shear reinfocement w/in dist s

5.8.2.5 Minimum Transverse Reinforcement

Av > 0.0316 sqrt (f'c) b_v s / fy

Av = 0.02 in2 NG 0.52 in2 = 0.0316 sqrt (f'c) bv s / fy

<u>Analysis:</u>

5.8.3.4.2 General Procedure

es = (Mu/dv + 0.5 Nu + |Vu-Vp| - Apsfpo) / (Es As + Ep Aps)

Mu =	13063 k-ft	Moment demand				
Nu =	653 k	Axial demand				
Vu =	263 k	Shear demand				
Vp =	0 k	component in direction of applied shear of the effective prestressing force positive if resisting the applied shear				
Aps =	0.00 in2	Area of PS				
fpo =	175 ksi	0.7 fpu		Flexural steel info:		
Es =	29000 ksi			Fy [ksi] =	40 ksi	
Ep =	29000 ksi			Flex Bars	# 10	
As =	5.08 in2	flexural steel		A_s_bar =	1.27 in2	
				d_s_bar =	1.27 in	
es =	0.0055			mult by	4	approx effective

Sxe = sx * 1.38 / (ag + 0.63)

sx =	min of following	36 in	
	dv =	60 in	
	d__ =	36 in	dist between layers of long crack control reinf
ag =	0.25 in	max agg size	assumed per photos
Sxe =	56.5		

theta = 29 + 3500 es
theta = 48 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met
beta = 4.8 / (1+ 750 εs)
beta = 0.94 factor indicating ability of diagonal cracked concrete to transmit tension & shear
If Min Transverse Reinforcement is NOT met
beta = 4.8 / (1+ 750 εs) x 51 / (39 + Sxe)
beta = 0.51 factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select:	beta =	0.5 degrees	factor indicating ability of diagonal cracked concrete to transmit tension & shear
	theta =	48 degrees	angle of inclination of diagonal comp. stress
Select:	alpha =	90 degrees	angle of inclination of transverse reinforcemen

Vn = Vc + Vs + Vp
Vc = 0.0316*beta*sqrt(f'c) bv dv
Vc = 35 kip
Vs = Av fy sin(alpha) < 0.095 sqrt(f'c) bv dv
Vs = 0.79 kip < 207 kip
Vn = 36 kip Gov

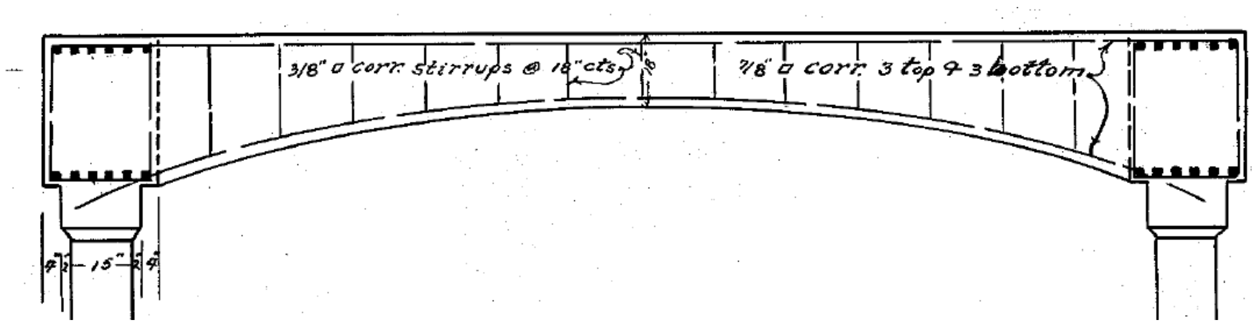
Vn = 0.25f'c bv dv + Vp
Vn = 863 kip

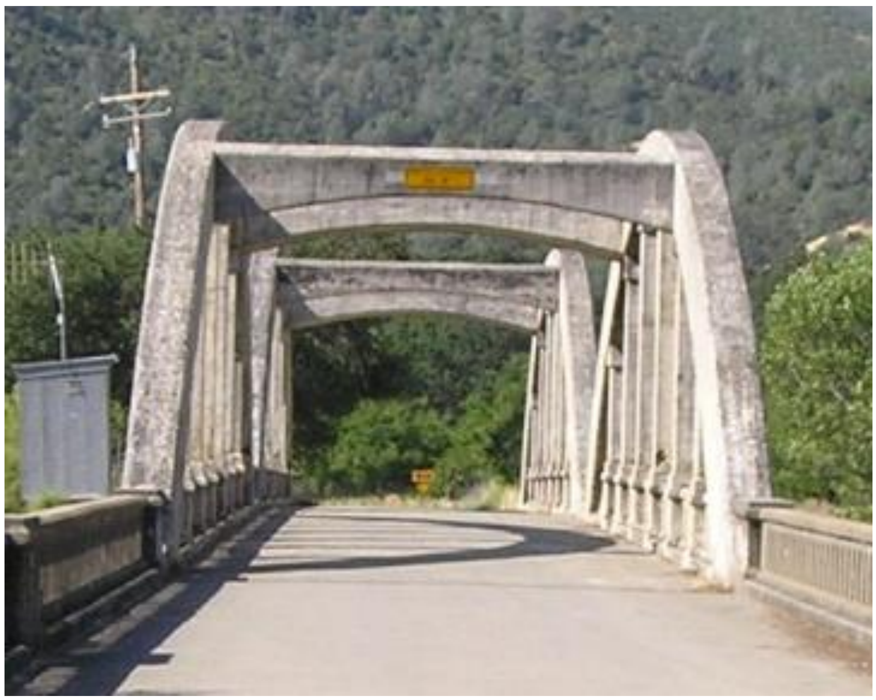
ϕ = 1.00 Seimsic phi for shear

Select: **ϕ V [kip] = 36 kip** < Demand = 69 kip NG
1.91

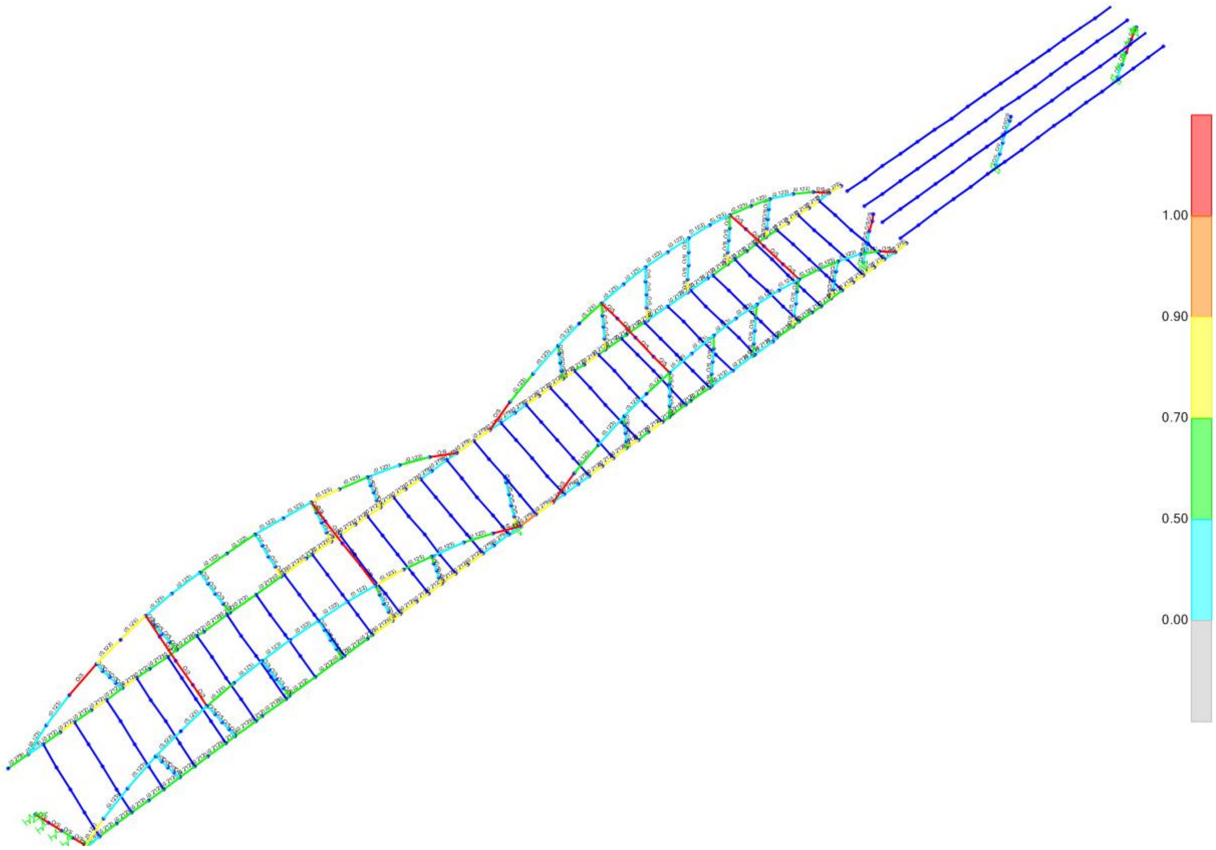
Portal Bracing:

As-built details of Portal sections & reinforcement:

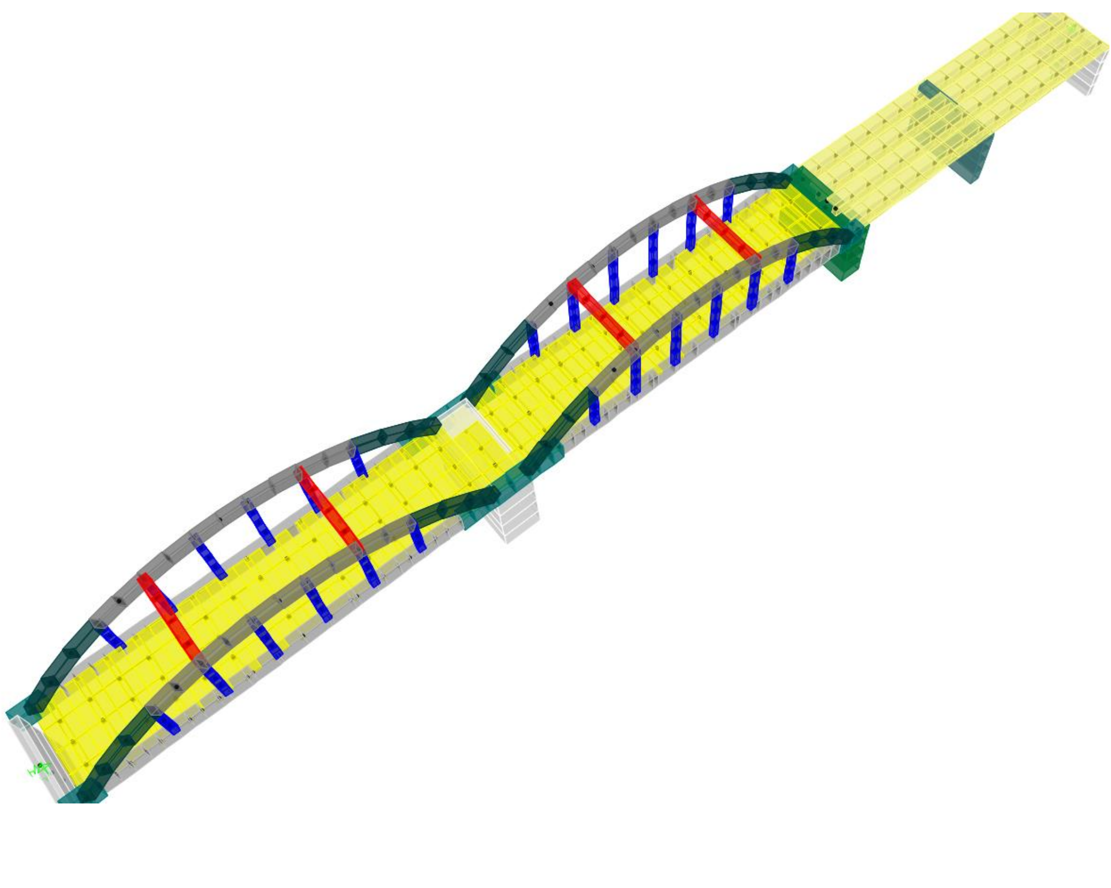




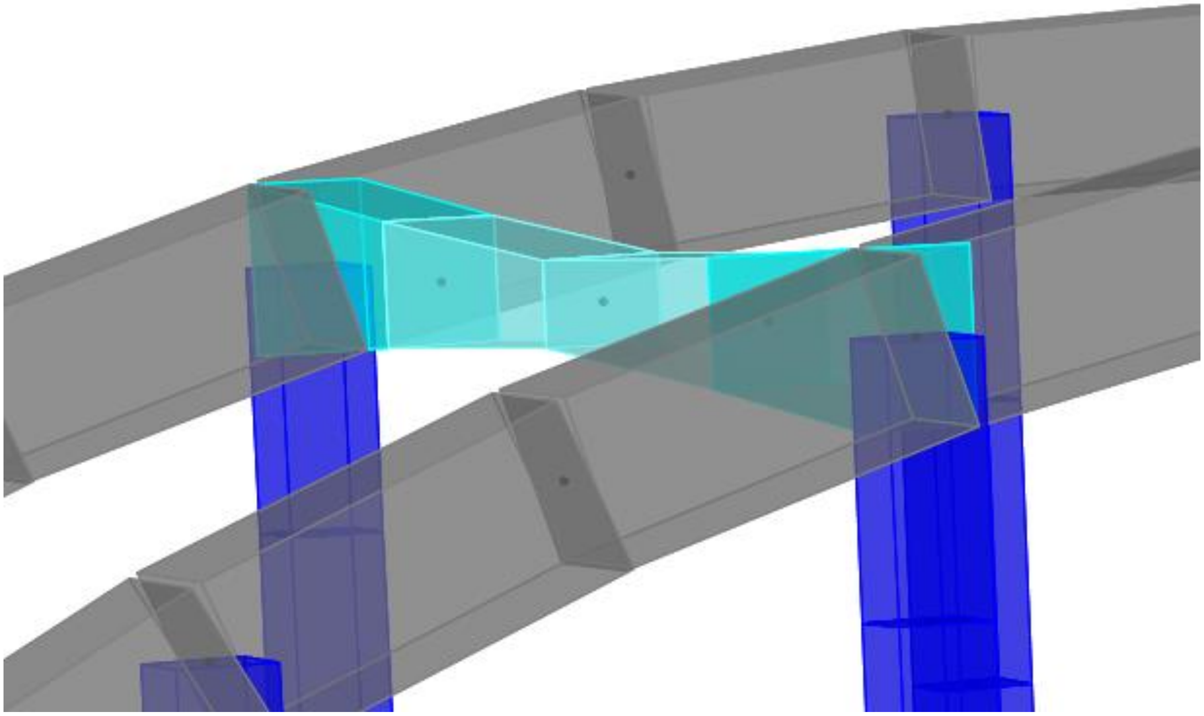
Analysis below shows the limits where the Portal exceed D/C ratio of 1.0 in Red:

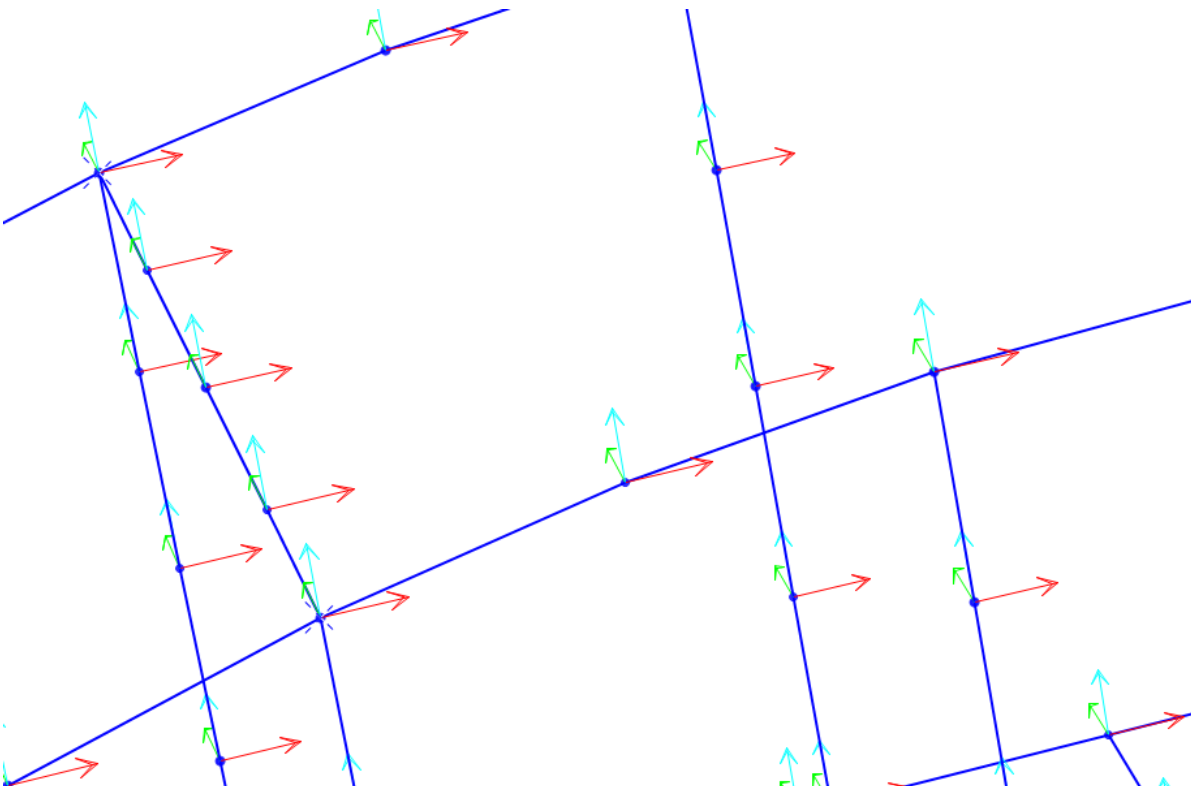


Red block below indicates the approximate location of where the Portal will be retrofitted.

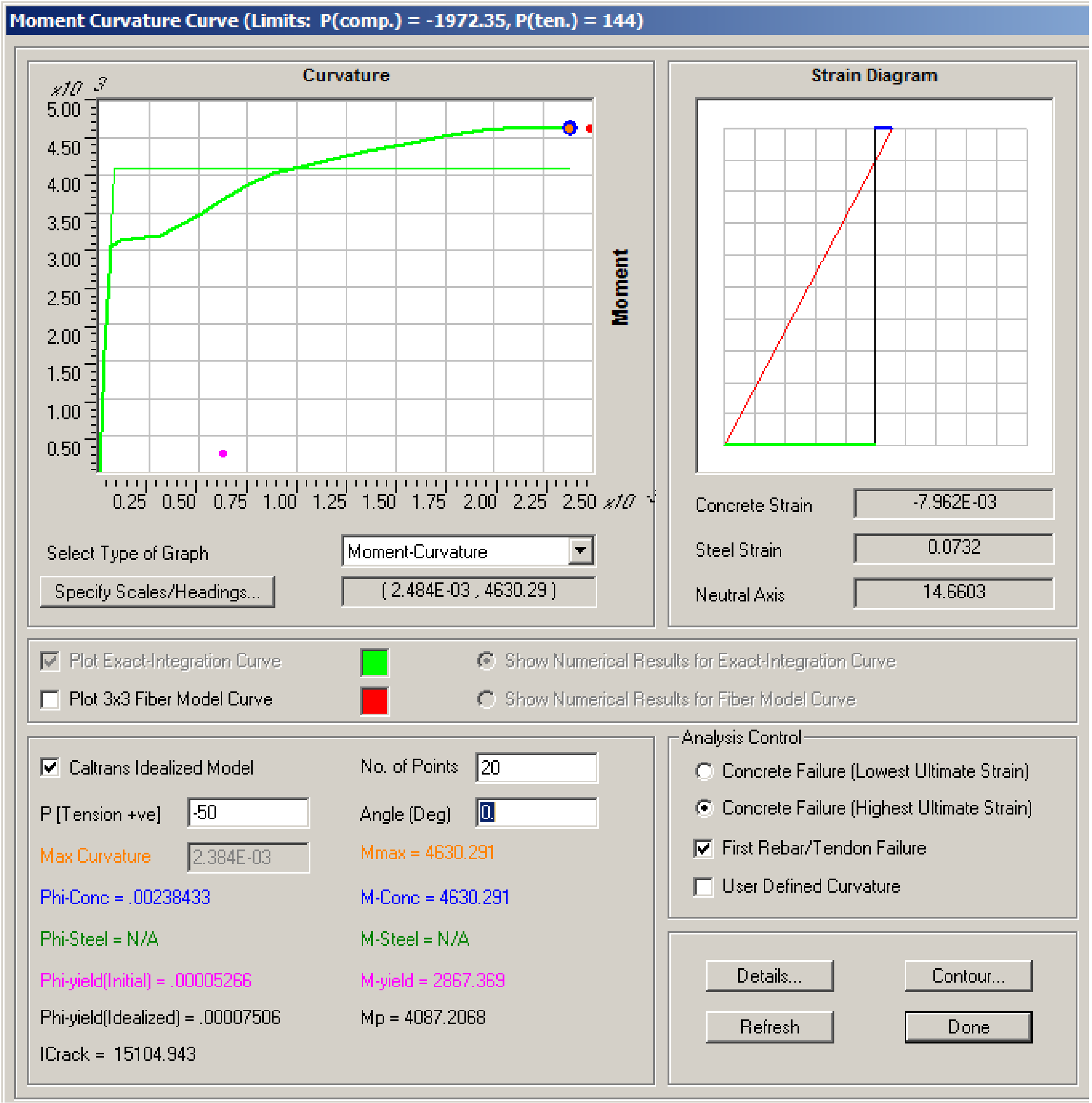


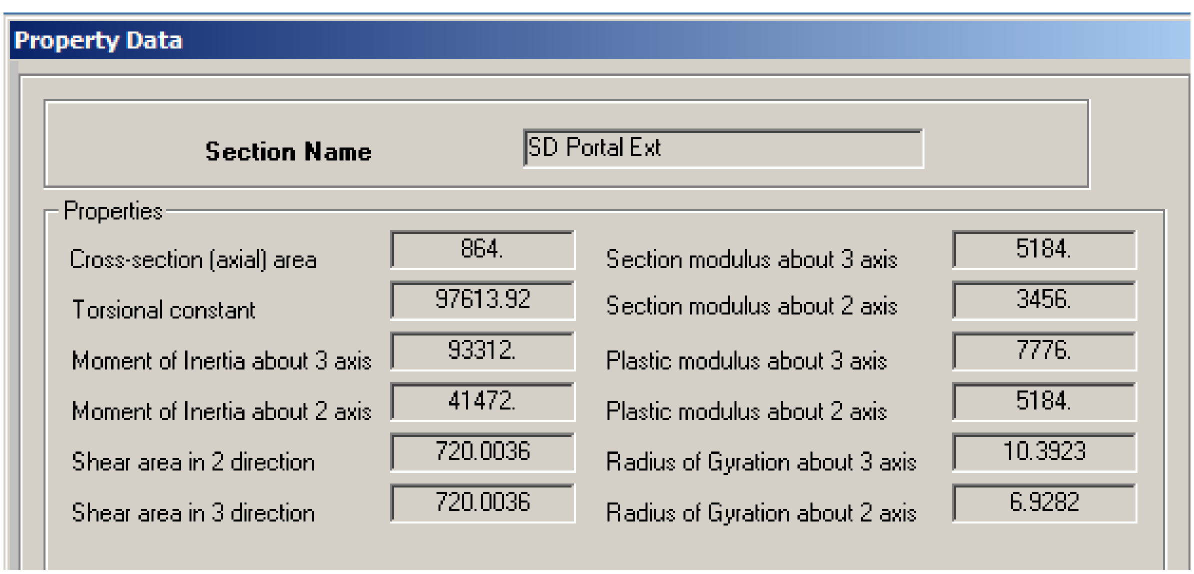
Portal modeling:





Portal Moment Curvature to determine I_crack:





QUINCY ENGINEERING, INC.

Project: **Rumsey**
Job No.
BY **JC** DATE **5/12/2014**

Description **Fiber Reinforcement Polymer**
Portal
Strong axis (Retrofitted)

SHEET

Concrete Section input:

Overall, h [in] =	36.00 in	3.00 ft
d [in] =	33.69 in	2.81 ft
Overall, b [in] =	24.00 in	2.00 ft
Concr f'c [ksi] =	2.5 ksi	
Concrete ε_c =	0.003	conc strain limit
PS [yes, no] =	no	
phi_PS =	1.00	
phi_non-ps =	0.90	
ϕ =	0.9	

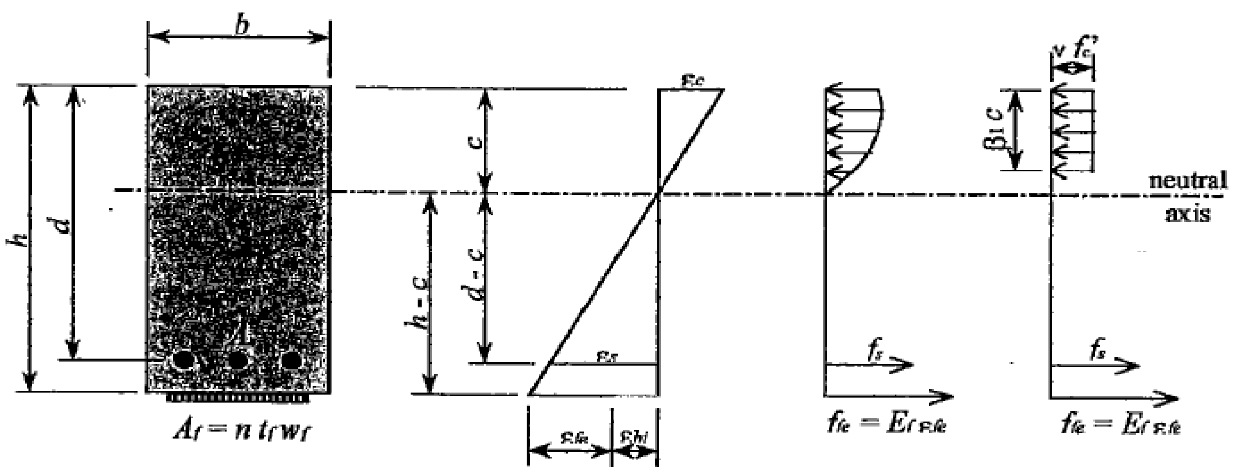
Steel Reinforcement input:

Fy [ksi] =	40 ksi	
Main Bars	# 7	conservative of 1 1/8" SQ bar
Tot =	3	conservative base on per as-built.
A_s [in^2] =	0.6	Photos shows 7 bars
d_s [in] =	0.875	Main Reinforcement
As [in^2] =	1.8 in sq	
Shear Bars	# 3	
A_v [in^2] =	0.11	
d_v [in] =	0.375	Shear Confinement
clr [in] =	1.5 in	assumed

Manufacture FRP input:

	Design limits:	
Ult Tensile Strength	121	ksi
in Primary Direction =	121,000	psi
	4.8	kip/in
Elongation at Break =	0.85%	
Tensile Modulus =	1.19E+07	psi
	1.19E+04	ksi
n, layers [ea] =	4	number of layers of FRP reinforcement
t_f [in] =	0.04 in	FRP composite material thickness per layer
Total thickness	0.16 in	
w_f [in] =	24.00 in	width of FRP reinfocing layers

ϕ = 0.9, if $e_s \geq 0.005$
ϕ = interp if $0.002 \leq e_s < 0.005$
ϕ = 0.65, if otherwise



Reinforced Concrete Section

Strain Distribution

Stress Distribution (Non-linear Concrete Stress Distribution)

Stress Distribution (Equivalent Fictitious Concrete Stress Distribution)

Assumptions:
Plain section remains plane
Max compressive strain in concrete is 0.003
Stress in steel under service load should be limited to 80% of yield strength

FRP Analysis:

Forces:

F_Conc = 0.85*fc*a*b
431 kip

F_steel = As*fy =
72 kip

$F_{frp} = A_{frp} * f_{fe}$ = **359 kip**
$f_{fe} = \varepsilon_{fe} * E_f$ = **94 ksi**
$A_f = n * t_f * w_f$ = 3.84 in sq
RFP within allowable stress

Strains: ε_c / c = = ε_s / (d - c) = = = ε_fe / (h - c)
(similar triangles)
Concrete ε_c = 0.003
Steel ε_s = 0.0072
Yeild ε_y = 0.0021
steel yields
RFP ε_{fe} = 0.0079
RFP within allowable stain

Neutral Axis:
c [in] = 9.95 in calculated distance between N.A. to extreme conc fiber

F_Conc = F_steel + F_frp
0 = - F_Conc + F_steel + F_frp
0 kip

beta = 0.85 (AASHTO 5.7.2.2) function of f'c
a [in] = 8.5 in vertical distance of whitney stress block

Moment Arm:
Conc Arm = (c - a / 2)
5.72 in
Steel Arm = (d - c)
23.74 in
Fiber Arm = (h - c)
26.05 in

Nominal Moment:
Conc Contr = 2,466 kip-in
Steel Contr = 1,709 kip-in
Fiber Contr = 9,357 kip-in

Nominal Moment =
13,532 kip-in
1,128 kip-ft

Factored Moment Strength:

ϕ Mn [k-in] =	12,179 kip-in	**>**	**Demand =**	7,400	**ok**	D/C= 0.61
ϕ Mn [k-ft] =	1,015 kip-ft	>	Demand =	617 kip-ft	ok	

Analysis Existing (without FRP) - Strong Axis:

F_Conc = 0.85*fc*a*b
72 kip

F_steel = As*fy =
72 kip

a [in] = 1.41 in
beta = 0.85 (AASHTO 5.7.2.2)
c [in] = 1.66 in = a/beta

Check Steel Yield
e_s = 0.0578 > e_y = 0.0021
steel yields

Arm [in] = 32.98 in

ϕ Mn [k-in] =	**2,137 kip-in**	<	Demand =	7,400	NG	D/C= 3.46
ϕ Mn [k-ft] =	178 kip-ft	<	Demand =	617 kip-ft	NG	

QUINCY ENGINEERING, INC.

Project:	**Rumsey**	Description	Fiber Reinforcement Polymer	SHEET
Job No.			Portal	
BY	**JC** DATE **5/12/2014**		**Weak axis (Retrofitted)**	

Concrete Section input:

Overall, h [in] =	24.00 in	2.00 ft
d [in] =	21.69 in	1.81 ft
Overall, b [in] =	36.00 in	3.00 ft
Concr f'c [ksi] =	2.5 ksi	
Concrete ε_c =	0.003	conc strain limit
PS [yes, no] =	no	
phi_PS =	1.00	
phi_non-ps =	0.90	
ϕ =	0.9	

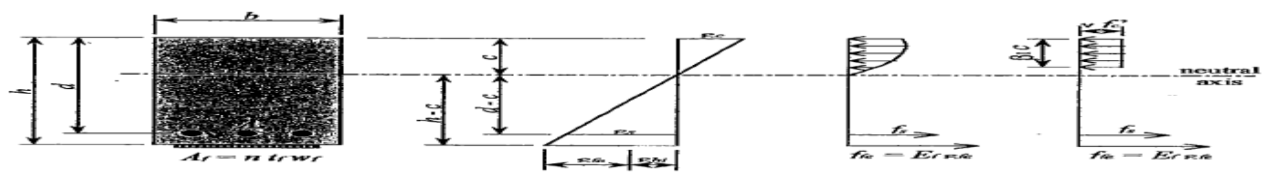
Steel Reinforcement input:

Fy [ksi] =	40 ksi	
Main Bars	# 7	conservative of 1 1/8" SQ bar
Tot =	2	conservative base on per as-built.
A_s [in^2] =	0.6	Photos shows 7 bars
d_s [in] =	0.875	Main Reinforcement
As [in^2] =	1.2 in sq	
Shear Bars	# 3	
A_v [in^2] =	0.11	
d_v [in] =	0.375	Shear Confinement
clr [in] =	1.5 in	assumed

Manufacture FRP input:

	Design limits:	
Ult Tensile Strength	121 ksi	
in Primary Direction =	121,000 psi	
	4.8 kip/in	
Elongation at Break =	0.85%	
Tensile Modulus =	1.19E+07 psi	
	1.19E+04 ksi	
n, layers [ea] =	4	number of layers of FRP reinforcement
t_f [in] =	0.04 in	FRP composite material thickness per layer
Total thickness	0.16 in	
w_f [in] =	36.00 in	width of FRP reinfocing layers

ϕ = 0.9, if $e_s \geq 0.005$
ϕ = interp if $0.002 \leq e_s < 0.005$
ϕ = 0.65, if otherwise



Reinforced Concrete Section   Strain Distribution   Stress Distribution (Non-linear Concrete Stress Distribution)   Stress Distribution (Equivalent Fictitious Concrete Stress Distribution)

Assumptions:
Plain section remains plane
Max compressive strain in concrete is 0.003
Stress in steel under service load should be limited to 80% of yield strength

FRP Analysis:

Forces:

F_Conc = 0.85*fc*a*b = **493 kip**

F_steel = As*fy = **24 kip**

$F_{frp} = A_{frp} * f_{fe}$ = **445 kip**
$f_{fe} = \varepsilon_{fe} * E_f$ = **77 ksi**
$A_f = n * t_f * w_f$ = 5.76 in sq
RFP within allowable stress

Strains: *(similar triangles)* ε_c / c = = ε_s / (d - c) = = = ε_fe / (h - c)

Concrete ε_c = 0.003
Steel ε_s = 0.0056
Yeild ε_y = 0.0021
steel yields
RFP ε_{fe} = 0.0065
RFP within allowable stain

Neutral Axis:
c [in] = 7.58 in calculated distance between N.A. to extreme conc fiber

F_Conc = F_steel + F_frp
0 = - F_Conc + F_steel + F_frp
0 kip

beta = 0.85 (AASHTO 5.7.2.2) function of f'c
a [in] = 6.4 in vertical distance of whitney stress block

Moment Arm:
Conc Arm = (c - a / 2) 5.72 in
Steel Arm = (d - c) 23.74 in
Fiber Arm = (h - c) 14.05 in

Nominal Moment:
Conc Contr = 2,820 kip-in
Steel Contr = 570 kip-in
Fiber Contr = 6,256 kip-in

Nominal Moment =
9,646 kip-in
804 kip-ft

Factored Moment Strength:

ϕ Mn [k-in] =	8,681 kip-in	**>**	**Demand =**	7,000	**ok**	D/C= 0.81
ϕ Mn [k-ft] =	723 kip-ft	>	Demand =	583 kip-ft	ok	

Analysis Existing (without FRP) - Weak Axis:

F_Conc = 0.85*fc*a*b 48 kip
F_steel = As*fy = 48 kip

a [in] = 0.63 in
beta = 0.85 (AASHTO 5.7.2.2)
c [in] = 0.74 in = a/beta

Check Steel Yield
e_s = 0.1339 > e_y = 0.0021
steel yields

Arm [in] = 33.37 in

ϕ Mn [k-in] =	1,442 kip-in	<	Demand =	7,000	NG	D/C= 4.86
ϕ Mn [k-ft] =	120 kip-ft	<	Demand =	583 kip-ft	NG	

QUINCY ENGINEERING, INC.

Project:	**Rumsey**	Description **Portal**	
Job No.	**Y01-500**	**Shear yy (LRDF 5.8.3.3)**	SHEET
BY	**JC**	DATE **5/12/2014**	

LRFD 5.8.3.3

Vn = minium of the following 2 equations
Vn = Vc + Vs + Vp
Vn = 0.25f'c bv dv + Vp

Ecco Results:
Vn = 14 kip < Demand = 62 kip **NG**
4.43

b =	18 in		
d =	15 in		
Ac =	270 in2		
f'c [ksi] =	2,500 psi	2.5 ksi	
bv =	18 in	eff web width	
dv =	12 in	eff shear depth	

Fy [ksi] =	40 ksi	
Shear Bars	# 3	
Spacing	12.00 in	spacing of stirrup
A_v_bar =	0.11 in2	
d_v_bar =	0.38 in	
mult by	2	number of bars per plane
Av	0.02 in	area of shear reinfocement w/in dist s

5.8.2.5 Minimum Transverse Reinforcement

Av > 0.0316 sqrt (f'c) b_v s / fy

Av = 0.02 in2 NG 0.27 in2 = 0.0316 sqrt (f'c) bv s / fy

Analysis:

5.8.3.4.2 General Procedure

es = (Mu/dv + 0.5 Nu + |Vu-Vp| - Apsfpo) / (Es As + Ep Aps)

Mu =	7516 k-ft	Moment demand
Nu =	14 k	Axial demand
Vu =	62 k	Shear demand
Vp =	0 k	component in direction of applied shear of the effective prestressing force positive if resisting the applied shear
Aps =	0.00 in2	Area of PS
fpo =	175 ksi	0.7 fpu
Es =	29000 ksi	
Ep =	29000 ksi	
As =	8.89 in2	flexural steel
es =	0.0027	

Flexural steel info:

Fy [ksi] =	40 ksi
Flex Bars	# 10
A_s_bar =	1.27 in2
d_s_bar =	1.27 in
mult by	7

Sxe = sx * 1.38 / (ag + 0.63)

sx =	min of following	12 in	
	dv =	12 in	
	d__ =	36 in	dist between layers of long crack control reinf
ag =	0.25 in	max agg size	assumed per photos
Sxe =	18.8		

theta = 29 + 3500 es
theta = 38 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met
beta = 4.8 / (1+ 750 εs)
beta = 1.59 factor indicating ability of diagonal cracked concrete to transmit tension & shear
If Min Transverse Reinforcement is NOT met
beta = 4.8 / (1+ 750 εs) x 51 / (39 + Sxe)
beta = 1.43 factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select:	beta =	1.4 degrees	factor indicating ability of diagonal cracked concrete to transmit tension & shear
	theta =	38 degrees	angle of inclination of diagonal comp. stress
Select:	alpha =	90 degrees	angle of inclination of transverse reinforcemen

Vn = Vc + Vs + Vp
Vc = 0.0316*beta*sqrt(f'c) bv dv
Vc = 15 kip
Vs = Av fy sin(alpha) < 0.095 sqrt(f'c) bv dv
Vs = 0.66 kip < 32 kip
Vn = 16 kip Gov

Vn = 0.25f'c bv dv + Vp
Vn = 135 kip

ϕ = 0.90 Seimsic phi for shear

Select: **ϕ V [kip] = 14 kip** < Demand = 62 kip NG
4.43

QUINCY ENGINEERING, INC.

Project: **Rumsey** Description **Portal**
Job No. **Y01-500** **Shear xx (LRDF 5.8.3.3)** SHEET
BY **JC** DATE **5/12/2014**

LRFD 5.8.3.3
Vn = minium of the following 2 equations
Vn = Vc + Vs + Vp
Vn = 0.25f'c bv dv + Vp

Ecco Results:
f V [kip] = 11 kip < Demand = 36 kip **NG**
3.27

b =	15 in			Fy [ksi] =	40 ksi	
d =	18 in					
Ac =	270 in2			Shear Bars	# 4	
f'c [ksi] =	2,500 psi	2.5 ksi		Spacing	18.00 in	spacing of stirrup
				A_v_bar =	0.20 in2	
bv =	15 in	eff web width		d_v_bar =	0.50 in	
dv =	15 in	eff shear depth		mult by	2	number of bars per plane
				Av	0.02 in	area of shear reinfocement w/in dist s

5.8.2.5 Minimum Transverse Reinforcement

Av > 0.0316 sqrt (f'c) b_v s / fy

Av = 0.02 in2 NG 0.34 in2 = 0.0316 sqrt (f'c) bv s / fy

Analysis:

5.8.3.4.2 General Procedure

es = (Mu/dv + 0.5 Nu + |Vu-Vp| - Apsfpo) / (Es As + Ep Aps)

Mu =	7516 k-ft	Moment demand
Nu =	14 k	Axial demand
Vu =	62 k	Shear demand
Vp =	0 k	component in direction of applied shear of the effective prestressing force positive if resisting the applied shear
Aps =	0.00 in2	Area of PS
fpo =	175 ksi	0.7 fpu
Es =	29000 ksi	
Ep =	29000 ksi	
As =	5.08 in2	flexural steel
es =	0.0039	

Flexural steel info:

Fy [ksi] =	40 ksi	
Flex Bars	# 10	
A_s_bar =	1.27 in2	
d_s_bar =	1.27 in	
mult by	4	approx effective

Sxe = sx * 1.38 / (ag + 0.63)

sx =	min of following	15 in	
	dv =	15 in	
	d__ =	36 in	dist between layers of long crack control reinf
ag =	0.25 in	max agg size	assumed per photos
Sxe =	23.5		

theta = 29 + 3500 es
theta = 43 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met
beta = 4.8 / (1+ 750 εs)
beta = 1.23 factor indicating ability of diagonal cracked concrete to transmit tension & shear
If Min Transverse Reinforcement is NOT met
beta = 4.8 / (1+ 750 εs) x 51 / (39 + Sxe)
beta = 1.02 factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select: beta = 1.0 degrees factor indicating ability of diagonal cracked concrete to transmit tension & shear
theta = 43 degrees angle of inclination of diagonal comp. stress
Select: alpha = 90 degrees angle of inclination of transverse reinforcemen

Vn = Vc + Vs + Vp
Vc = 0.0316*beta*sqrt(f'c) bv dv
Vc = 12 kip
Vs = Av fy sin(alpha) < 0.095 sqrt(f'c) bv dv
Vs = 0.79 kip < 34 kip
Vn = 12 kip Gov

Vn = 0.25f'c bv dv + Vp
Vn = 141 kip

ϕ = 0.90 Seimsic phi for shear

Select: **ϕ V [kip] = 11 kip** < Demand = 36 kip NG
3.27

ENGINEERING
DATA REPORT
NUMBER 48

EVALUATION OF REINFORCING BARS IN OLD REINFORCED CONCRETE STRUCTURES

A SERVICE OF THE CONCRETE REINFORCING STEEL INSTITUTE

INTRODUCTION

Most practicing structural engineers sooner or later face the task of evaluating old structures. This task is always an interesting challenge, because it is never a routine application of the current practice in design. Owners commonly require re-evaluation when planning a change in building usage, restoration, additional stories, or lateral additions in any combination. Frequently, the original contract documents, the "as-built" revisions, and so on, cannot be found.

The structural engineering challenge is two-fold. First, the material properties must be determined for the concrete. The concrete can and usually does gain 25 percent or more strength than it had at 28 days, but the concrete can also have deteriorated under fire or chemical exposures. The second challenge concerns the reinforcing bars — determining the yield strength, the bar sizes and their cross-sectional areas, the locations of the bars, effective depths of structural members, the bending and cut-off details of the bars, and development lengths (bond and anchorage).

Where documentation is lacking for the existing structure, the following abbreviated history of reinforcing bars may be a useful starting point.

Reference 1 is an excellent presentation on the history of reinforced concrete. Included in the article are illustrations of a variety of patented reinforcing bars, and an extensive list of references regarding codes, design and construction, and reports on landmark tests.

REINFORCING BARS — SPECIFICATIONS, BAR SIZES AND ALLOWABLE STRESSES

Specifications. Reinforcing bars, as we know them today, came about in 1900. Specifications were first developed by the Association of American Steel Manufacturers in 1910. The American Society for Testing and Materials (ASTM) adopted standard specification A15 for billet-steel concrete reinforcing bars in 1911. Reinforcing bars were plain and deformed in structural, intermediate and hard grades (minimum yield strengths), or deformed, cold-twisted. Structural grade (minimum f_y = 33,000 psi) was normally used, unless otherwise specified. The specified minimum yield strengths of structural, intermediate, and hard grades were 33,000, 40,000, and 50,000 psi, respectively. The minimum yield strength of cold twisted bars was specified at 55,000 psi.

ASTM also issued similar specifications for rail-steel (A16) and axle-steel (A160) reinforcing bars. The minimum yield strength for rail-steel bars was 50,000 psi, and for axle-steel bars the same as for billet steel bars.

Table 1 summarizes the ASTM specifications for reinforcing bars from 1911 to the present.

Bar Sizes. Table 2 shows the standard reinforcing bar sizes recommended by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete in its 1924 Report (Reference 2).

Allowable Stresses. Some early authorities stated that allowable stresses in tension in the reinforcement higher than 12,000 psi show "very little to be gained in economy" and recommended a maximum of 14,000 psi (Reference 3). Recommended allowable stresses in tension in the 1924 Joint Committee Report (Reference 2) were:

- 16,000 psi for structural grade and rail-steel bars
- 18,000 psi for intermediate and hard grade bars and twisted bars.

In its 1940 Report, the Joint Committee increased its recommended allowable stresses to:

Tension

- 18,000 psi for structural grade bars
- 20,000 psi for intermediate and hard grades or rail-steel bars
- 16,000 psi for all web reinforcement

Compression

- 16,000 psi for intermediate grade bars
- 20,000 psi for hard grade or rail-steel bars

Table 1—Reinforcing Bars 1911 to Present; ASTM Specifications; Minimum Yield and Tensile Strengths in psi

ASTM Spec	Years		Steel Type	Grade 33 (Structural)		Grade 40 (Intermediate)		Grade 50 (Hard)		Grade 60		Grade 75	
	Start	End		Min. Yield	Min. Tensile	Min. Yield	Min. Tensile	Min. Yield	Min. Tensile	Min. Yield	Min. Tensile	Min. Yield	Min. Tensile
A15	1911	1966	Billet	33,000	55,000	40,000	70,000	50,000	80,000				
A408	1957	1966	Billet	33,000	55,000	40,000	70,000	50,000	80,000				
A432	1959	1966	Billet							60,000	90,000		
A431	1959	1966	Billet									75,000	100,000
A615	1968	1972	Billet			40,000	70,000			60,000	90,000	75,000	100,000
A615	1974	1986	Billet			40,000	70,000			60,000	90,000		
A615	1987	Present	Billet			40,000	70,000			60,000	90,000	75,000	100,000
A16	1913	1966	Rail					50,000	80,000				
A61	1963	1966	Rail							60,000	90,000		
A616	1968	1999	Rail					50,000	80,000	60,000	90,000		
A160	1936	1964	Axle	33,000	55,000	40,000	70,000	50,000	80,000				
A160	1965	1966	Axle	33,000	55,000	40,000	70,000	50,000	80,000	60,000	90,000		
A617	1968	1999	Axle			40,000	70,000			60,000	90,000		
A996	2000	Present	Rail, Axle			40,000	70,000	50,000	80,000	60,000	90,000		
A706	1974	Present	Low-Alloy							60,000	80,000		
A955M	1996	Present	Stainless			40,000	70,000			60,000	90,000	75,000	100,000

BOND AND ANCHORAGE

After establishing the yield strength of the reinforcing bars, the next important property required for evaluation of old structures concerns bond and anchorage. Steel mills in the USA completed conversion of their production to "high-bond" deformations about 1947, which continue virtually unchanged to the present day. In 1947, ASTM issued a specification, designated as A305, which prescribed requirements for deformations on reinforcing bars. The A305 specification existed from 1947 to 1968. In 1968, the requirements for deformations were merged into the specifications for reinforcing bars—A615 (billet-steel), A616 (rail-steel), and A617 (axle-steel).

For older structures, it is prudent to consider all varieties of reinforcing bars—plain round, old-style deformed, twisted square, and so on—conservatively and simply as 50 percent as effective in bond and anchorage as current bars. In other words, the tension development lengths, ℓ_d, for the old bars would be twice (double) the ℓ_d required for modern reinforcing bars. Since most strength design reviews for flexure will be based on a yield strength, $f_y = 33,000$ psi instead of today's 60,000 psi, the tension development lengths for the old bars can be determined by adding 10 percent to any current table of tension development lengths, ℓ_d, for modern reinforcing bars. The main deficiencies encountered in old structures will be in tension lap splice lengths provided for bars larger than #6, and typical details with top bars larger than #6 cut off at 0.25 times clear span.

Standard end hooks, 90° or usually 180°, on old-style bars in earlier codes were considered to develop half the allowable tension stress. Under today's strength design method, this value would approximate $\phi f_y/2 = (0.90)(33,000 \text{ psi})/2 \approx 15,000$ psi.

DETAILS OF REINFORCING BARS

Flexural Members. For structures built during the period 1900 to 1940, design standards and accompanying typical details of reinforcing bars evolved gradually, beginning with a bewildering variety of patented systems. Where design drawings or project specifications are not available, and no clue remains to the system used, caution is particularly prudent. Many of the older patented systems would be considered much less effective today—some were theoretically sound and went out of style because of high costs, but others were based upon theory not acceptable today. In two-way slabs, do ***not*** assume that there was only two-way reinforcement. Especially, if the topmost layer is disappointingly light, it may be part of a *four-way* system, with four layers instead of two. Look for diagonal bands of bars.

Where original design drawings are not available, typical details for reinforcing bars as shown in ACI Detailing Manuals (Reference 4) were commonly used since 1947. These typical details can be assumed and used for initial calculations if original service loads are known. In any case, these calculations should be confirmed or modified as soon as data on bar sizes, bar spacings, and effective depths of structural members can be checked in the field.

Particularly for flexural members, load tests are especially convincing when used to check calculated capacity based upon material tests and reconstituted placing drawings. In particular, even non-destructive load tests can thus be used to validate calculated deflections before and after cracking. (Reference 5).

Columns. Non-destructive surface tests should be employed at numerous locations to evaluate the concrete. If it is necessary, column concrete cover can be removed to observe vertical bar sizes, splice details, ties or spirals, etc., and replaced with little or no impairment of the structural capacity. Load tests on columns are generally not feasible, and so evaluation of column strength must be analytical. Even cutting out sample test cores to determine concrete strength is not generally advisable, since vertical reinforcing bars may be damaged and replacing removed concrete is not likely to be effective.

Under present codes, the contribution of spiral reinforcement to column capacity is considerably less than under old codes. In a present day evaluation, therefore, spiral columns, especially square or rectangular, are more likely to limit the total capacity than tied columns.

Locating Reinforcing Bars. Instruments now available permit the user to locate and follow individual reinforcing bars inside concrete slabs or beams. Some give accurate indications for the depth of concrete cover and even relative size of bar. Again, it is desirable to calibrate such readings by exposing the bars at some non-critical locations. These readings are particularly valuable in re-constructing the design details—bend points, cut-off points, and bar spacings—at least for the outside layers of bars.

CONCRETE PROPERTIES

The present day concrete properties in place should be determined by tests. Even if original project specifications are available, the specified concrete compressive strength, f_c', is not a reliable value years later. Evaluation of present in-place concrete strength may be demonstrated by several more or less non-destructive methods. The ASTM standard test methods are:

(a) Test of cast-in-place cylinders, ASTM C873 (limited to use in slabs)

(b) Pulse velocity testing, ASTM C597

(c) Rebound number, ASTM C805

(d) Penetration resistance, ASTM C803

(e) Pullout strength, ASTM C900

It should be noted that all these methods require correlation with strength tests on drilled cores. The measurements of these various properties of concrete are ***related*** to compressive strength, tensile strength, or modulus of elasticity which can be converted to compressive strength of standard cylinders for design strength. Even instruments purporting to read "psi" or with "conversions provided" must be calibrated with the tests on cores from the actual concrete in question.

Table 2—Standard* Reinforcing Bar Sizes (1924)

Size, in.	Area, in.[2]	
	Round	Square
	**	†
3/8	0.11	–
1/2	0.20	0.25
5/8	0.31	–
3/4	0.44	–
7/8	0.60	–
1	0.79	1.00
1-1/8	–	1.27
1-1/4	–	1.56

* Recommended by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete in its 1924 Report.
** Most suppliers offered a ¼-inch round bar, as well as the recommended standard sizes.
† The ¼-inch square bar was used, but to a lesser extent. Square bars were usually deformed, or if plain in structural grade, twisted to enhance bond and yield strength properties.
1. Round bars were plain or deformed.
2. A number of producers offered additional sizes, in 1/16-inch increments, prior to adoption of this reduced list of standard sizes.

SELECTED REFERENCES

1. "Reinforced Concrete at the Turn of the Century", by Robert E. Loov, *ACI Concrete International*, December 1991, pp. 67-73.

2. "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete" by Joint Committee on Standard Specifications for Concrete and Reinforced Concrete; the committee was composed of representatives of ACI, AIA, AREA, ASCE, ASTM and PCA. Reports were published in 1916, 1924 and 1940.

3. *Principles of Reinforced Concrete Construction*, by F. E. Turneaure and E. R. Maurer, John Wiley & Sons, New York, 1908.

4. ACI Detailing Manual for Buildings, 1947, ACI Committee 315, and Detailing Manual, 1957, 1965, 1974, 1980 . . .

5. "Full-Scale Load Testing of Structures", STP-702, ASTM, 1980 (Symposium Collection).

OTHER RESOURCES

ACI Building Codes, 1928, 1936, 1941, 1947, 1951, 1956, 1963, 1971, 1977 . . .

"Strength Evaluation of Existing Structures", Chapter 20, ACI 318-77, ACI 318-83, ACI 318-89 . . .

"Application of ACI 318 Load Test Requirements", by R. C. Elstner, D. P. Gustafson, J. M. Hanson and P. F. Rice, CRSI *Professional Members' Bulletin*, No. 16, 1987, CRSI, 11 pp.

"Strength Evaluation of Existing Concrete Buildings (ACI 437R-91)", by ACI Committee 437, 24 pp.

This report No. 48 replaces EDR No. 11.

SOFT METRIC REINFORCING BARS

While the focus of this report is on the past, it is important for readers of this document to be aware of current industry practice regarding soft metric reinforcing bars. The term "soft metric" is used in the context of bar sizes and bar size designations. "Soft metric conversion" means describing the nominal dimensions of inch-pound reinforcing bars in terms of metric units, but not physically changing the bar sizes. In 1997, producers of reinforcing bars (the steel mills) began to phase in the production of soft metric bars. Within a few years, the shift to exclusive production of soft metric reinforcing bars was essentially achieved. Virtually all reinforcing bars currently produced in the USA are soft metric. The steel mills' initiative of soft metric conversion enables the industry to furnish the same reinforcing bars to inch-pound construction projects as well as to metric construction projects, and eliminates the need for the steel mills and fabricators to maintain a dual inventory. Thus, USA-produced reinforcing bars furnished to any construction project most likely will be soft metric.

Designations of Bar Sizes. The sizes of soft metric reinforcing bars are physically the same as the corresponding sizes of inch-pound bars. Soft metric bar sizes, which are designated #10, #13, #16, and so on, correspond to inch-pound bar sizes #3, #4, #5, and so on. The metric bar designations are simply a re-labeling of the inch-pound bar designations. The following table shows the one-to-one correspondence of the soft metric bar sizes to the inch-pound bar sizes.

Soft Metric Bar Sizes vs. Inch-Pound Bar Sizes

Soft Metric Bar Size Designation	Inch-Pound Bar Size Designation
#10 #13 #16	#3 #4 #5
#19 #22 #25	#6 #7 #8
#29 #32 #36	#9 #10 #11
#43 #57	#14 #18

Minimum Yield Strengths or Grades. Virtually all steel mills in the USA are currently producing reinforcing bars to meet the metric requirements for tensile properties in the ASTM specifications. Minimum yield strengths in metric units are 300, 350, 420 and 520 MPa (megapascals), which are equivalent to 40,000, 50,000, 60,000 and 75,000 psi, respectively. Metric Grade 420 is the counterpart of standard Grade 60.

Bar Marking. Soft metric reinforcing bars are required to be identified with the Producer's mill designation, bar size, type of steel, and minimum yield strength or grade. For example, consider the marking requirements for a #25, Grade 420 metric bar, which is the counterpart of an inch-pound #8, Grade 60 bar. Regarding the bar size and grade, the ASTM specifications require the number "25" to be rolled onto the surface of the metric bar to indicate its size. For identifying or designating the yield strength or grade, the ASTM specifications provide an option. A mill can choose to roll a "4" (the first digit in the grade number) onto the bar, or roll an additional longitudinal rib or grade line to indicate Grade 420.

The 27th Edition of the CRSI *Manual of Standard Practice* was published in March 2001. Chapter 1 in the Manual includes a detailed presentation of the inch-pound and metric requirements in the ASTM specifications for reinforcing bars. Appendix A in the Manual shows the bar marks used by USA producers to identify Grade 420 soft metric bars.

More information about soft metric reinforcing bars is also provided in Engineering Data Report No. 42, "Using Soft Metric Reinforcing Bars in Non-Metric Construction Projects". EDR No. 42 can be found on CRSI's Website at **www.crsi.org**.

Readers of this report are also encouraged to visit the CRSI Website for:

- Descriptions of CRSI publications and software, and ordering information
- Institute documents available for downloading
- Technical information on epoxy-coated reinforcing bars
- Technical information on continuously reinforced concrete pavement
- Membership in CRSI and member web links
- General information on the CRSI Foundation
- Information on the CRSI Design Awards competition

CRSI

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H N H 0301/19M Printed in U.S.A.

QUINCY ENGINEERING, INC.

Project:	**Rumsey**	Description	**Column Shear**
Job No.			**Seismic Shear Capacity (SDC 3.6)**
BY	**JC**	DATE	**8/28/2013**

SHEET

P_c	2,300 kip	Column axial force
ϕ	0.90	Shear Strength reudction factor SDC 3.2.1
R^{col}	84 in	Column radius
clr	3 in	Clear cover
bar_s	# 5	Shear confinement reinforcement
s	9 in	Shear confinement reinforcement spacing
ρ_s	0.01	Ratio of volume of spiral or hoop reinforcement to the core column confined by the spiral or hoop reinforcement (measured out-to-out)
f_{yh}	60 ksi	Nominal yield stress of transerse column reinforcement
μ_d	1.00	Local displacement ductility demand, limit set by SDC 2.2.4 on page 2-9
f'c	3.6 ksi	Assumed 1 to not yeild
bar_l	# 11	Main longitudinal steel reinforcement
A_b	1.56 in2	Area of individual reinforcing steel bar
A_g	22,167 in2	Gross section area of column
A_e	17,734 in2	$A_e = 0.8 * A_g$ (SDC Eqn 3.17)
D'	81 in	Cross sectional dimension of confined concrete core measured between the centerline of the peripheral hoop or spiral

$F1 = \rho_s * f_{yh} / 0.150 + 3.67 - \mu_d$		(SDC Eqn 3.20)	$F2 = 1 + P_c / (2000 * A_g)$		(SDC Eqn 3.21)
$F1_{cal}$	8.39	, 0.3 ≤ F1 ≤ 3	$F2_{cal}$	1.05	, Fw ≤ 1.5
F1	3.00		F2	1.05	

Inside Plastic Hinge Zone			Inside Plastic Hinge Zone		
v_c = F1 * F2 * sqrt(f'c) ≤ 4*sqrt(f'		(SDC Eqn 3.18)	v_c = 3 * F2 * sqrt(f'c) ≤ 4*sqrt(f'c)		(SDC Eqn 3.19)
$v_{c\ cal}$	5.99		$v_{c\ cal}$	5.99	
4*sqrt(f'c)	7.59		4*sqrt(f'c)	7.59	
v_c	5.99		v_c	5.99	

$Vc = v_c * A_e$ (SDC Eqn 3.16)

Vc 106 kip — For Inside Plastic Hinge Zone

$A_v = n * (pi /2)*A_b$

A_v 2.45 in2

$Vs = A_v * f_{yh} * D' / s$ (SDC Eqn 3.16) — For confined circular columns

Vs 1,318 kip

Vn = Vc + Vs (SDC Eqn 3.15)

Vn 1,424 kip

ϕVn 1,282 kip

ϕVn ≥ Vo (SDC Eqn 3.14)

Vocol 1,010 kip Obtained by $(Mo^{col} + Mo^{col}) / L^{col}$

Assumed L = 10', conservative

D/C 0.79 ok

Questions concerning the VERTCON process may be mailed to NGS

Latitude: 38.89022308

Longitude: 122.2384572

NGVD 29 height:

Datum shift(NAVD 88 minus NGVD 29): 0.831 meter

Patch Spalled or Delaminated Concrete:

Details below indicate the work required to repair spalled surface area. Photos following indicate several locations of concrete surface repair are required.