# 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)

Project_	FRP		Description	,
Job No.		·		SHEET
Job No By	JC	Date 8/15/2013		
			$F = 0.85 \times f' \times a \times b$	Tkips7
			Fz = 0.85 x fi x a x b  085 fi  085 fi	f' < 4ks
<b>7</b>		ε <sub>c</sub>	08512	
			a= 8,C - C6 c- a	N. Marian
r 9		7 8		
		1 8 82	$f_s$ $f_s$ $f_s$ $f_s = A_s \times f_y$	
		$\mathcal{E}_{\mathcal{E}}$	ffe = Efe	
		- T	$f_{fe} = \overline{\xi_i} \xi_{fe} \qquad f_{fe} = \overline{\xi_f} \xi_{fe}$ $\nabla = E \mathcal{E}$	
Sept. 1				
	ες	E. E.	15t Assume Steel yields, Ec = 0.003	
	c	&s _ & fe d-c h-c	2nd find c	
			- Tha Fina C	
		Efe = (n-c)(	ες \	
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	10 10			

Project	Description + K		CILERE
Job No	Arch Rib	Strong Aris	SHEET
ByDate			
	The second secon		
h= 36"	Fy=40ksi	L       F	RP
b ± 27"	Main bars #9,		
	A5 = 1 = x	60A = 412	E tonsin   Fsi
f'c = 2.5 ksi	10 = 1.11		
	ds= 1.1281	4	1 - 0 000
Ec = 0,003			brant = 0.85%
	ster bars # 3		<del>                                     </del>
	$Ar = 0.11 \text{ in}^2$	L L L	$a = 121 \pm si$
	dv = a 27511		
	CW = 1.5 in a	assmed	
420 1 1 1 1 5			
Step 1: Assumes Es >	ey & steel y	ieda fge, ffe = Efe E	
Fs = As x fy	Fx = Asx	fe fe = Efe E	4
= (6/n²) (40 ksi)		tellective	stics in FRP
= 240 kips			
			1
step 2 ! Cc = Ts + Tf,	Compression	of conc = Tension	
0.85f		L ste	rel & FRP
	Ce =	15+74	440
	0.85 fe * [a * ]	57 = Ax A. + Am	x E Es
		1   1   1   1   1   1   1   1   1   1	72
	185)(2,5ksi)(a)(27"	1) = (Kin 2 / Vun + ci) + (C	204m × 254×254
	10 J( 7/3 /3/ / ( 4 / 4 / 4 / 4 / 4 / 4 / 4 / 4 / 4		
┠┈┤╴┼┈┼┈╂┈┼┈╎┈┤┈┼┈┼┈	but a = B, C	x (h-c)	(Ec) Ex
			•
	a=0.85 C		
(0.85)(2.5 ksi)(0.85)	$(c)(27'')=/(2^2)($	40kg) +	
├─ <del>├</del> ┈ <del>├</del> ┈ <del>├</del> ┈┼─┼─┼─┼─┼─┼─			
	+ (0.04,,	MLLX 27/1(36" -C	10000 1104
		)(2)(27")(36"-C)	· ( ) ( ) ( ) ( )
-> C= 9.4" or +00	H		
$\rightarrow a = (6.85)(9.4'')$			

	Project_				.,					·		Des	scrip	otior	١		·								<sub>1</sub> ,				7
Step 3: check if $E_3 \ge E_y$ if steel yields $E_1 = 0.002$ $E_2 = 0.003$ $E_3 = 0.003$ $E_4 = 0.003$ $E_4 = 0.003$ $E_5 = 0.0007$ $E_5 = 0.$									_		_				_			,	<del></del>	<del></del> .					-	S	HEE /	T	
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$f_y = E_s  E_y$ $f_{0ksi} = 29,000 ksi  E_y$ $E_y = 0.0014$ $E_s > E_y  \therefore  \text{stal dow yield } U$ $\text{Stap 4: }  \text{calc}  \text{Efe}  \text{if }  \text{check limits of FRP}$ $\text{Efe}  =  E_c$ $h-c  C$ $E_{fe}  =  0.003$ $E_{fe}  =  9.47$ $36" - 9.4" = 9.47$ $\Rightarrow E_{fe} = 0.0085 = 0.85\%$ Design Florgation at preak																	1		İ										-
$f_{y} = E_{s} E_{y}$ $Hoks = 29,000 ksi \times E_{y}$ $E_{y} = 0.0014$ $E_{s} > E_{y} \cdot \text{ stal dow yield !!}$ $E_{t} = E_{t}$ $h-c$ $E_{t} = 0.003$ $E_{t} = 0.0085 = 0.85\%$ $E_{t} = 0.0085$ $E_{t} = 0.0085$ $E_{t} = 0.0085$								>	٤	ء ی		0.	0	<del>-</del>	77	7	†	1							1	<del> </del> -	-	+	
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$\frac{\varepsilon_{fe}}{3b'' - 9.4''} = \frac{9.4''}{9.4''}$ $\frac{1}{3b'' - 9.4''} = \frac{9.4''}{9.4''}$						- 1	+	╁╧	<u> </u>	C	-					-	<b> </b>	ļ							<u> </u>	-	ļ	<u> </u>	
$3b'' - 9.4''$ $\Rightarrow 2fe = 0.0085 = 0.85\%$ Elongation (				- -	-   -		-			<u> </u>	F	<del> </del>		<u> </u>	ļ		┼	-	<del> </del>			<u> </u>			<del> </del>	<u> </u>			-
36" - 9.4" $9.4"$ $9.45$ Design Elongation  for a Ex. Ex.							<del> </del>	-			0	DO	3	<del> </del>	ļ					<del> </del>					ļ	<del> </del>	<del> </del>	ļ	
$3b'' + 9.97$ $-> 2fe = 0.0085 \leq 0.85\%$ Elongation (				_	Z f	<u>-</u>	<u> </u>	111	<b> -</b>	ļ <b>:</b>				-			<u> </u>	<u> </u>		ļ	ļ	ļ			<u> </u>	ļ	<del> </del>		-
$f_{i} = \varepsilon_{i}  \varepsilon_{f}$				3	6"	+_'	9,4	\\	-		7.	7	ļ	ļ	ļ	ļ		<u> </u>			ļ	ļ	<b>1</b>	-	on o	<u></u>	ļ	<u> </u>	
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$f_{fe} = \mathcal{E}_{fe}  \mathcal{E}_{f}$ $= (0.0085)(1.19\times10^{4} \text{ksi}) = 101 \text{ksi} < 121 \text{k}$																					ļ			~~					
= (0.0085)(1.19x104 ksi) = 10/ksi < 12/k		-				1			ļ	8	2.	1	£,	4		İ													
= (0.0085)(1.19x10/ksi)=10/ksi = 14/ksi			-			1-1	re			- J	-			-		<del> </del>		4	, 1		,		<b> </b>			,	71	L.	
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Project	Description	
Job No	-	SHEET
By Date	<del>-</del>	
Calc More	ant strength Nonhal	
Step 5 Mn = Fe	EX Arm conc	
	5 × Armsted	
	f X Am Fiber Map	
Ma = [0.85x4	: ( xaxb] [ C - a]	
	] L d+4]	
+ [AFRP]	× Ex × Ex ][h-c]	
4594:45	5,4"	
Mn = (0.85)(2.5ksi)(8	B")(27")(9.4" - B")	
240 K/Y	24.20	
+ (6in²)(40 Ksi)(	(33.56" - 9.4")	26.6"
+ (0.04i)(2 EA)(271	in) (0.0085) (119×104 Así) (36	"- 9,4")
	ICI ksi	
14701		
/V(n) =  2/T/D R/N		
+ 5,80B KIN		
$M_{n} = 2478 k_{n}$ + 5,808 kin + 5,811 kin		
14,097 kin		
= 1,175 kfr		
Step 6 AMn (0.9)(1,175 k		
(a9)(1,175 k	: <del>(</del> f)	
± 1,057 kff		

 Project:
 Rumsey

 Job No.
 BY

 JC
 DATE
 8/15/2013

Description

Fiber Reinforcement Polymer Arch Rib - Strong axis

### Hand Calc verification

SHEET

Concrete Section	input:		Steel Reinfor	cement i	nput:		Manufacture FR		
Overall, h [in] = d [in] = Overall, b [in] =	36.00 in 33.56 in 27.00 in	3.00 ft 2.80 ft 2.25 ft	Fy [ksi] =  Main Bars	40 ksi # 9	conservative	e of 1 1/8" SQ bar	Ult Tensile Strengt in Primary Direction	= 121,000	
Concr f'c [ksi] = Concrete $\varepsilon_c$ =	2.5 ksi 0.003	conc strain limit	Tot = A_s [in^2] = d_s [in] =	6 1 1.128	conservative Photos show Main Reinfo		Elongation at Break Tensile Modulus		7 psi
PS [yes, no] = phi_PS = phi_non-ps = φ =	no 1.00 0.90 0.9		As [in^2] =  Shear Bars  A_v [in^2] =  d_v [in] =  clr [in] =	# 3 0.11 0.375 1.5 in	Shear Confi	nement	n, layers [ea] = t <sub>i</sub> [in] = Total thickness w <sub>i</sub> [in] =	2 0.04 in 0.08 in 27.00 in	number of layers of FRP reinforcement FRP composite material thickness per layer width of FRP reinfocing layers
h h		anticut	Es Sh	f	$\frac{f_{i}}{\tilde{\epsilon}} > \frac{f_{i}}{\tilde{\epsilon}} > \frac{f_{i}}{\tilde{\epsilon}}$	neutral axis $f_{k} = E_{fRR}$	φ = inter φ = 0.65  Assumptions: Plain section rem Max compressive	e strain in co	e <sub>s</sub> < 0.005
Analysis:		ed Concrete Str ction	ain Distribution	(Non-line	istribution ear Concrete istribution)	Stress Distribution (Equivalent Fictitious Concrete Stress Distribution)			
Forces: F_Conc =	0.85*fc*a*b 458 kip		F_steel =	As*fy = 240 kip			$F_{frp} = A_{frp} * f_{fe}$ $f_{fe} = \varepsilon_{fe} * E_{f}$ $A_{f} = n * t_{f} * w_{f}$	= = =	218 kip 1.01E+02 ksi 2.16 in sq
Strains: (similar triangles) Concrete $\varepsilon_c$ =	ε_c/c 0.003	= =	$\epsilon_s / (d - c)$ Steel $\epsilon_s =$ Yeild $\epsilon_y =$	= 0.0077 0.0021	= steel yields	=	$\epsilon_{fe}$ / (h - c)	0.0085	RFP with in allowable stain
	9.40 in F_Conc = F_9 0 = - F_Conc 0 kip	calculated distance but steel + F_frp + F_steel + F_frp	etween N.A. to extr	eme cond	: fiber				
beta = a [in] =	0.85 8.0 in	(AASHTO 5.7.2.2) fur vertical distance of wh							
Moment Arm: Conc Arm = (o	c - a / 2) i.40 in		Steel Arm =	d - c) 24.16 in			Fiber Arm =	(h - c) 26.60 in	
Nominal Moment: Conc Contr = Nominal Moment	2,476 = 14,083 kip-ir 1,174 kip-ft	kip-in	Steel Contr =	5,799	kip-in		Fiber Contr =	= 5,807	kip-in
Factored Moment φ Mn [k-in] = φ Mn [k-ft] =	Strength: 12,675 kip-ir 1,056 kip-ft								

#### Sensitivity Analysis

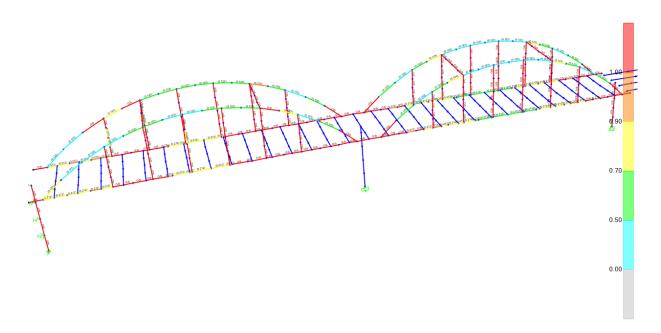
Several sensitivity analyses were performed to assess the necessity of obtaining a concrete strength core to determine a refined concrete strength of the existing Rumsey Bridge. Base on the sensitivity analyses results in the following pages, QEI does not recommend obtaining sampling at this planning 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

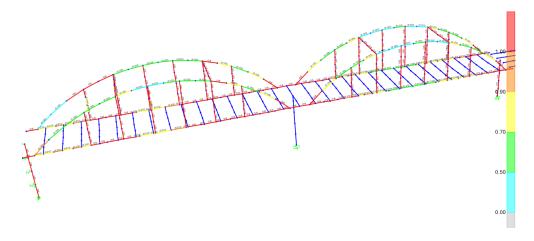
Governing elastic D/C ratios:



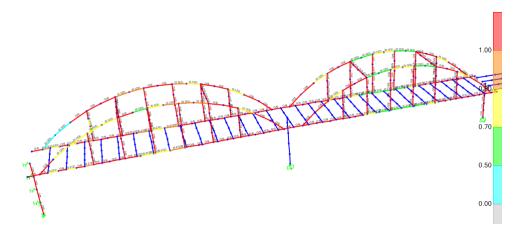
### Sensitivity Model:

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.

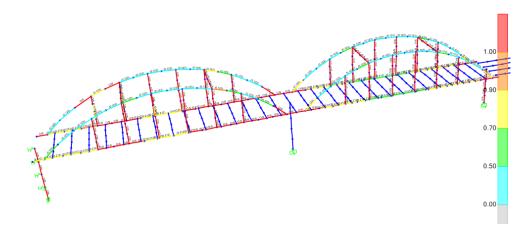
Concrete strength: 2000 psi Model Governing elastic D/C ratios:



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

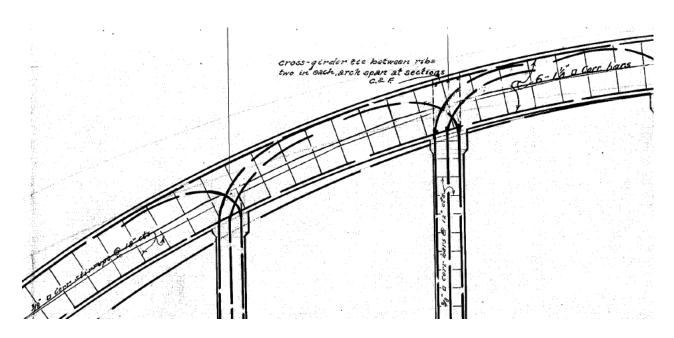


Concrete strength: 4000 psi Model Governing elastic D/C ratios:



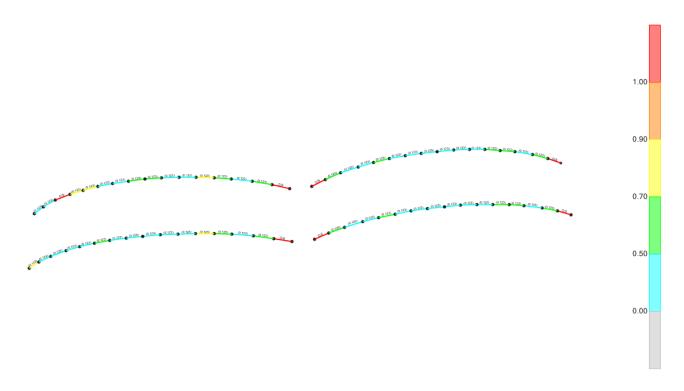
### **Arch Rib Analysis:**

As-built details of Arch sections & reinforcement:

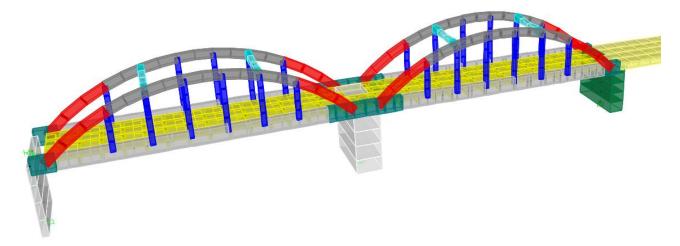




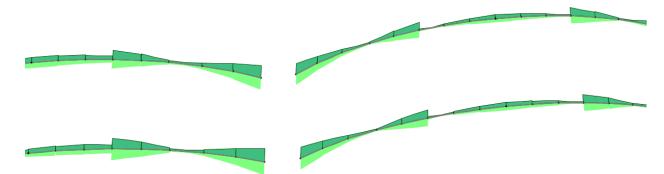
Analysis below shows the limits where the Arch Rib exceed D/C ratio of 1.0 in Red:



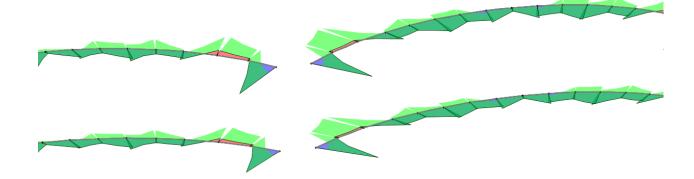
Red block below indicates the approximate location of where the Arch will be retrofitted.



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



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



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

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

F_steel =	As*fy = 240 kip		85*fc*a*b 240 kip			
a [in] = beta = x [in] =	4.18 in 0.85 a/beta 4.92 in	(AASHTO 5.7.2.2)				
Check Steel Yield e_s =	0.0175	> e_y = steel yields	0.0021			
Arm [in] =	31.47 in					
$\phi$ Mn [k-in] = $\phi$ Mn [k-ft] =	7,553 kip-in 629 kip-ft	> Demand = > Demand = 4	5,709 76 kip-ft	ok ok	D/C= 0.76	

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

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

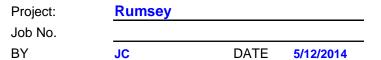
### Analysis:

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

				<b>QUINCY ENGIN</b>	IEERING, INC	-			
Project:	Rumsey			Description Arch	·				
Job No.	Y01-500			Concrete Axial (L	RFD)				SHEET
3Y	JC	DATE	5/12/2014		,				011122
				=					
ompression	Member Strer	ngth							
fc' =	2500	psi			Main Bars	# 10 N	/lain Reinfo	cement	
t = b =	36 27	in in			Tot =	6			
<b>5</b> –	21				A_s [in^2] =	1.27			
d =	33.365	in			d_s [in] =	1.27			
d' =	2.635	in			As [in^2] =	7.62			
As =	7.62	in2			7 to [iii 2] -	7.02			
As' =	7.62	in2			Shear Bars	#3 8	Shear Confir	nement	
713 -	7.02	1112			Spacing [in] =	18	onear comm	icinicin	
fy =	40,000	psi			A_v [in^2] =	0.11			
beta1 =	0.85	psi			d_v [in] =	0.375			
Dela I =	0.00				u_v [iii] =	0.070			
Pu =	564 kip	Demand			clr [in] =	2			
					Comp Bars	# 10 N	/lain Reinfo	cement	
					Tot =	6			
					A_s' [in^2] =	1.27			
					d_s' [in] =	1.27			
					$As' [in^2] =$	7.62			
					Reinforcement Max Reinforcer Ast/Ag < 0.08 Ast/Ag = Ast/Ag =	nent	rime)/(t*b)	bers (LRF	<b>D 5.7.4</b> .: ok
					-			0.00	•
					Min Reinforcem				
					Ast/Ag > 0.0025			for comp.	member
					Ast/Ag =		rime)/(t*b)		
					Ast/Ag =	0.018	>	0.010	ok
					Min bar # 5, Ch	eck			
ure Compr	ession (LRFD	5.7.4.4)			c-c < 12", Chec	k			
0 = phi[0.80]	)*fc' (*b*t - As -	As') + (As + As')	*fy]	(8-30) for ties					
φ =	0.85				Lateral Reinfo				
φ Po =	2,112,000	lbs			spacing > min(				
φ Po =	2,112 kips	>	<b>564 kip</b> 0.27	ok	Greater than #3	3 if Long. ba	r <#10, Che	eck	
					Ties (BDS 8.18	3.3)			
					h_wall =	276 ir	ı		
					Ash =	3.5933 ir			
						04 :			
					h_c =	31.625 ir			
					Ag =	972 ir			
					Ac =	850 ir			
					0.30*s_t*h_c*f				
						1.54 ir	n2		
					3.59 in2	> [	Demand =	1.54 in2	

				QUINCY E	NGINEER	ING, INC.				
Project:	Rumsey			Description		ŕ				
Job No.	Y01-500			Shear yy (LR						SHEET
	JC	DATE	5/12/2014							
RFD 5.8.3.3										
n = minium of	the following	g 2 equations		Ecco Results:						
n = Vc + Vs +				$\phi$ V [kip] =	48 kip	<	Demand =	79 kip	NG	
/n = 0.25f'c bv	dv + Vp							1.65		
) =	36 in			Fy [ksi] =	40 ksi					
<b>=</b>	27 in			Oh D	".0					
\c = c [ksi] =	972 in2 2,500 psi	2.5 ksi		Shear Bars Spacing	# 3 18.00 in	spacing of s	stirrun			
0 [101] –	2,000 poi	2.0 101		A_v_bar =	0.11 in2	opaoing or c	житар			
ov =	36 in	eff web width		d_v_bar =	0.38 in					
lv =	24 in	eff shear deptl	า	mult by	2		oars per plane			
				Av	0.01 in	area of she	ar reinfocemen	t w/in dist s		
	5.8.2.5 Min	imum Transver								
			qrt (f'c) b <sub>v</sub> s/fy		0.04: -	0.00:-	(8) - 3.1			
		Av =	0.01 in2	NG	0.81 in2	= 0.0316  sc	rt (f'c) bvs/fy	/		
.8.3.4.2 Gene	ral Procedui	re								
20110		/ + 0.5 Nu +  Vu	-Vp  - Apsfpo )	/ (Es As + Ep Ar	os)					
		Mu =	629 k-ft	Moment demai	nd					
		Nu =	562 k	Axial demand						
		Vu = Vp =	78 k 0 k	Shear demand component in o		plied chear of	the effective n	roctroccina	force	
		νp =	UK	positive if resis			the ellective p	restressing	ioice	
		Aps =	0.00 in2	Area of PS	ang are appar	ou 01.0u.				
		fpo =	175 ksi	0.7 fpu		Flexural ste	el info:			
		Es =	29000 ksi			Fy [ksi] =	40 ksi			
		Ep =	29000 ksi	flammal at a al		Flex Bars	# 9			
		As =	6.00 in2	flexural steel		A_s_bar = d_s_bar =	1.00 in2 1.13 in			
		es =	0.0022			mult by	6			
	Sxe = sx * 1	1.38 / (ag + 0.63	,							
		sx = r	nin of following							
			dv = d =	24 in 36 in	dist hetween	lavers of long	crack control re	einf		
		ag =	0.25 in	max agg size			Crack control is	CII II		
		Sxe =	37.6		·					
	theta = 29 +	+ 3500 es								
	theta =	37	degrees	angle of inclina	tion of diagor	nal comp. stre	SS			
		F								
		sverse Reinforce .8 / (1+ 750 εs)	ement is met							
	beta = 4.		actor indicating	ability of diagor	nal cracked co	oncrete to tran	smit tension &	shear		
		sverse Reinforce		, , ,						
	beta = 4.	.8 / (1+ 750 εs)	x 51 / (39 + 8	Sxe)						
	beta =	1.22 f	actor indicating	ability of diagor	nal cracked co	oncrete to tran	smit tension &	shear		
Select:	heta –	1.2 degrees f	actor indication	ability of diagor	nal cracked o	ncrete to tran	nsmit tension &	shear		
Jeieut.	theta =			ion of diagonal o		onoroto to trai	ionnii tonoion α	oricai		
Select:	alpha =			ion of transvers		en				
	., ., .									
	Vn = Vc + V		s) by dy							
	Vc = 0.03 Vc =	316*beta*sqrt(f'o 53 kip	η ων αν							
		ี 53 หม fy sin(alpha) < 0	.095 sart(f'c) by	v dv						
	Vs =	0.44 kip	<	130 kip						
	Vn =		Gov	·						
	\/m 0.05"	a lave also is N								
	Vn = 0.25f'd Vn =	bv dv + Vp 540 kip								
	VII =	эно кір								
	$\phi =$	0.90	Seimsic phi for	shear						
Select:	φ = <b>φ</b> V [kip] =		Seimsic phi for	shear Demand =	79 kip	NG				

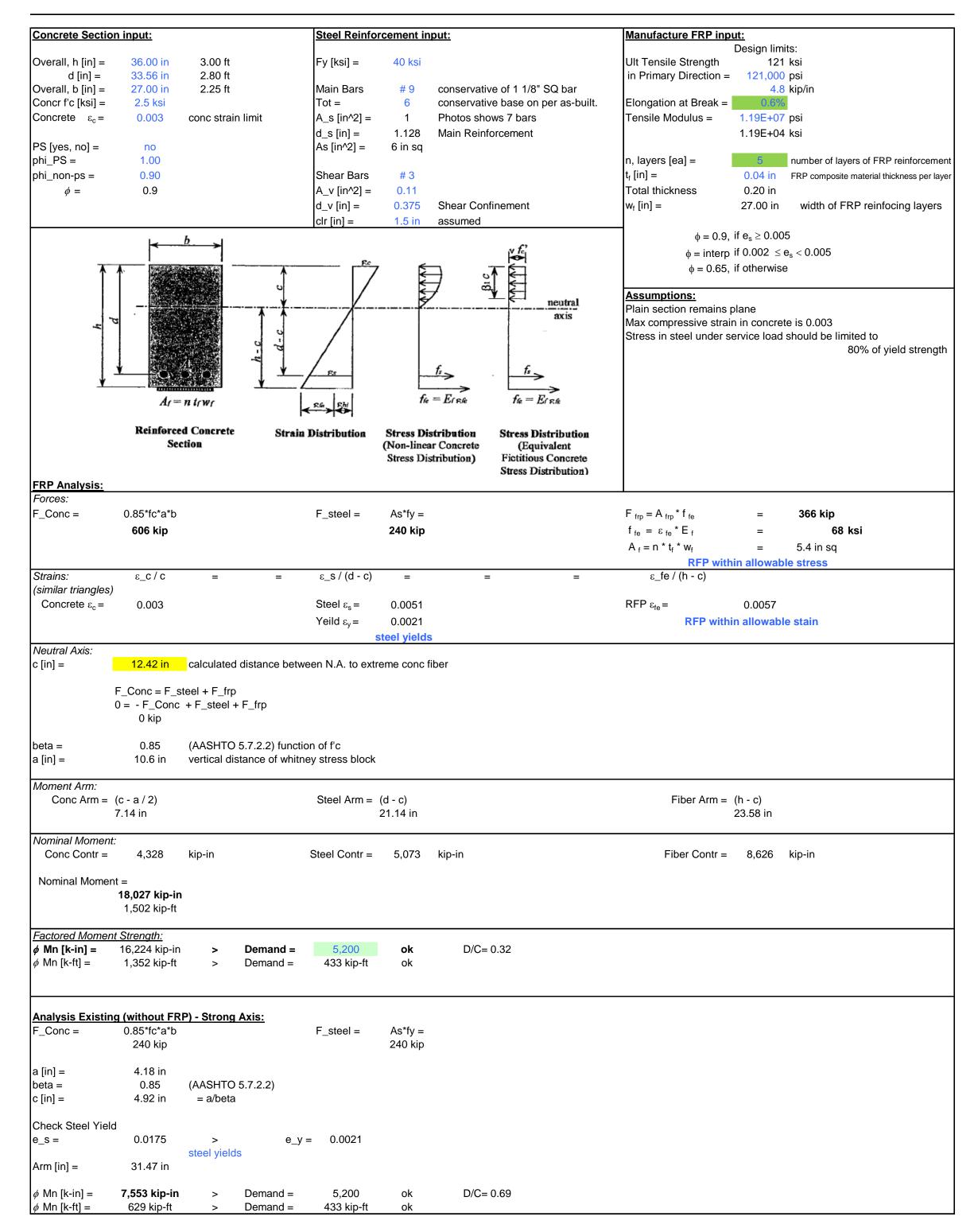
				QUINCY E	NGINEER	ING, INC.				
Project:	Rumsey			Description	Arch Rib					
ob No	Y01-500			Shear xx (LR	DF 5.8.3.3)				_	SHEET
BY	JC	DATE	5/12/2014	•					_	
RFD 5.8.3.3	of the following	n 2 equations		Ecco Results:						
n = Vc + Vs	•	g z oquationo		φ V [kip] =	42 kip	>	Demand =	37 kip	ok	
n = 0.25 f/c b				,				0.87		
=	27 in 36 in			Fy [ksi] =	40 ksi					
_ c =	972 in2			Shear Bars	#3					
c [ksi] =	2,500 psi	2.5 ksi		Spacing	18.00 in	spacing of s	tirrup			
		_		A_v_bar =	0.11 in2					
v =	27 in	eff web width		d_v_bar =	0.38 in					
v =	33 in	eff shear dep	th	mult by	2		ars per plane			
				Av	0.01 in	area of shea	ır reinfocemer	nt w/in dist s		
	5.8.2.5 Min	imum Transve	erse Reinforcen	nent						
		Av > 0.0316 s	sqrt (f'c) b <sub>v</sub> s/fy	,						
		Av =	0.01 in2	NG	0.61 in2	= 0.0316 sq	rt (f'c) bvs/f	y		
						_				
.8.3.4.2 Gen	eral Procedu									
	es = (Mu/d		u-Vp  - Apsfpo )							
		Mu =	629 k-ft	Moment dema	nd					
		Nu = Vu =	562 k	Axial demand						
		Vu = Vp =	78 k 0 k	Shear demand component in o		onlied chear of	the effective r	reetreesing	force	
		vp =	UK	positive if resis			tile ellective p	nestressing	ioice	
		Aps =	0.00 in2	Area of PS	ang the applic	ou oriour				
		fpo =	175 ksi	0.7 fpu		Flexural stee	el 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 + 0.6								
		sx =	min of following							
			dv =	33 in	P 4 I 4					
			d =			layers of long	crack control i	eint		
		ag = Sxe =	0.25 in 51.8	max agg size	assumed per	r priotos				
	theta = 29 -	+ 3500 es								
	theta =	37	degrees	angle of inclina	tion of diagon	nal comp. stres	S			
		sverse Reinford								
		.8 / (1+ 750 εs)		obility of dia	anl aracles d	anarata ta terr	amit tonaias o	oboor		
	beta =		factor indicating		iai cracked co	Discrete to trans	SITHE LENSION &	snear		
			cement is NOT n x 51 / (39 + 5							
	beta = 4		factor indicating		nal cracked co	oncrete to trans	smit tension &	shear		
		1.0 degrees	factor indicating	ability of diagor	nal cracked co	oncrete to trans	smit tension &	shear		
Select:	beta =	-		ion of diagonal						
Select	beta = theta =	37 degrees	angle of momia		e reinforceme	n				
Select:	theta =		angle of inclinat	ion of transvers		311				
	theta =	90 degrees		ion of transvers		711				
	theta = alpha = Vn = Vc + \	90 degrees	angle of inclinat	ion of transvers		51 I				
	theta = alpha = Vn = Vc + \	90 degrees /s + Vp	angle of inclinat	ion of transvers		511				
	theta = alpha = Vn = Vc + Vc = 0.0 Vc =	90 degrees  /s + Vp 316*beta*sqrt(f' 47 kip	angle of inclinat			511				
	theta = alpha = Vn = Vc + Vc = 0.0 Vc = Vs = Av Vs =	90 degrees  /s + Vp 316*beta*sqrt(f 47 kip fy sin(alpha) < 0 0.44 kip	angle of inclinat  c) bv dv  0.095 sqrt(f'c) bv							
	theta = alpha = Vn = Vc + V	90 degrees  /s + Vp 316*beta*sqrt(f 47 kip fy sin(alpha) < 0 0.44 kip	angle of inclinat c) bv dv 0.095 sqrt(f'c) bv	/ dv		11				
	theta = alpha =  Vn = Vc + \	90 degrees /s + Vp 316*beta*sqrt(f' 47 kip fy sin(alpha) < 0 0.44 kip 47 kip	angle of inclinat  c) bv dv  0.095 sqrt(f'c) bv	/ dv						
	theta = alpha = Vn = Vc + V	90 degrees  /s + Vp 316*beta*sqrt(f 47 kip fy sin(alpha) < 0 0.44 kip 47 kip c bv dv + Vp	angle of inclinat  c) bv dv  0.095 sqrt(f'c) bv	/ dv						
	theta = alpha =  Vn = Vc + \	90 degrees /s + Vp 316*beta*sqrt(f' 47 kip fy sin(alpha) < 0 0.44 kip 47 kip	angle of inclinat  c) bv dv  0.095 sqrt(f'c) bv	/ dv						
	theta = alpha = Vn = Vc + Vc = 0.0 Vc = Vs = AV Vs = Vn = Vn = 0.25f6	90 degrees  /s + Vp 316*beta*sqrt(f 47 kip fy sin(alpha) < 0 0.44 kip 47 kip c bv dv + Vp 557 kip	angle of inclinat  c) bv dv  0.095 sqrt(f'c) bv  Cov	/ dv 134 kip						
	theta = alpha = $Vn = Vc + V$ $Vc = 0.0$ $Vc = Vs = AV$ $Vs = Vn = Vn = Vn = 0.25fo$ $Vn = \phi = 0.25fo$	90 degrees  /s + Vp 316*beta*sqrt(f 47 kip fy sin(alpha) < 0 .44 kip 47 kip c bv dv + Vp 557 kip 0.90	angle of inclinat  c) bv dv  0.095 sqrt(f'c) bv	/ dv 134 kip	37 kip	ok				



Description

Fiber Reinforcement Polymer Arch Rib Strong axis (Retrofitted)

SHEET



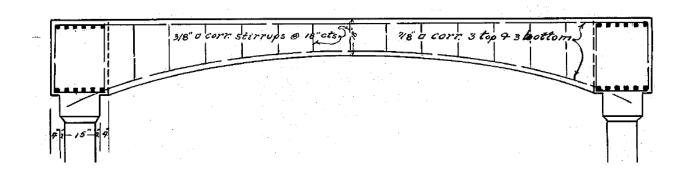


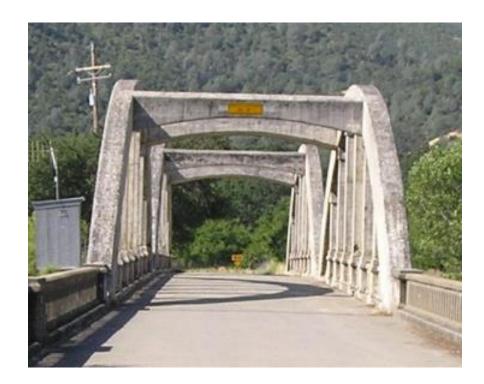
SHEET

Concrete Section	ı innut:		Steel Paints	reament in	nut·	Manufacture EDD innut:
Concrete Section	<del>-</del>		Steel Reinfo		<u>put.</u>	Manufacture FRP input:  Design limits:
Overall, h [in] = d [in] = Overall, b [in] =	27.00 in 24.56 in 36.00 in	2.25 ft 2.05 ft 3.00 ft	Fy [ksi] = Main Bars	40 ksi # 9	conservative of 1 1/8" SQ bar	Ult Tensile Strength 121 ksi in Primary Direction = 121,000 psi 4.8 kip/in
Concr f'c [ksi] = Concrete $\varepsilon_c$ =	2.5 ksi 0.003	conc strain limit	Tot = A_s [in^2] = d_s [in] =	1 1.128	conservative base on per as-built. Photos shows 7 bars Main Reinforcement	Elongation at Break = 0.6%  Tensile Modulus = 1.19E+07 psi 1.19E+04 ksi
PS [yes, no] = phi_PS = phi_non-ps =	no 1.00 0.90		As [in^2] = Shear Bars	2 in sq # 3		n, layers [ea] = $t_f$ number of layers of FRP reinforcement $t_f$ [in] = $0.04$ in FRP composite material thickness per layer
$\phi =$	0.9		A_v [in^2] = d_v [in] = clr [in] =	0.11 0.375 1.5 in	Shear Confinement assumed	Total thickness 0.32 in  w <sub>f</sub> [in] = 36.00 in width of FRP reinfocing layers
			len find			$\phi = 0.9, \text{ if } e_s \ge 0.005$ $\phi = \text{interp if } 0.002 \le e_s < 0.005$ $\phi = 0.65, \text{ if otherwise}$
	Ar - nrewe		2	fa = Eina	neutral exis	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 Concre Section	te Strain Distribu	(Non-lin	Distribution near Concret Distribution)	e (Equivalent	
FRP Analysis: Forces:						
F_Conc =	0.85*fc*a*b <b>700 kip</b>		F_steel =	As*fy = <b>40 kip</b>		$F_{frp} = A_{frp} * f_{fe} = 620 \text{ kip}$ $f_{fe} = \epsilon_{fe} * E_{f} = 54 \text{ ksi}$ $A_{f} = n * t_{f} * w_{f} = 11.52 \text{ in sq}$ RFP within allowable stress
Strains:	ε_c / c	= =	ε_s / (d - c)	=	= =	ε_fe / (h - c)
(similar triangles) Concrete $\varepsilon_c$ =	0.003		Steel $\varepsilon_s$ = Yeild $\varepsilon_y$ =	0.0038 0.0021 steel yields		RFP $\varepsilon_{\text{fe}}$ = 0.0045  RFP within allowable stain
Neutral Axis:	40.771					
c [in] =	10.77 in  F_Conc = F_steel + 0 = - F_Conc + F_ 0 kip	•	een N.A. to ext	reme conc i	iber	
beta = a [in] =	0.85 9.2 in	(AASHTO 5.7.2.2) funct vertical distance of white				
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	t = 14,886 kip-in 1,240 kip-ft					
<u>Factored Moment</u> <b>φ Mn [k-in] =</b> φ Mn [k-ft] =	Strength: 13,397 kip-in 1,116 kip-ft	> Demand = > Demand =	7,032 586 kip-ft	<b>ok</b> ok	D/C= 0.52	
Analysis Existing F_Conc =	<b>g (without FRP) - W</b> 0.85*fc*a*b 80 kip	leak Axis:	F_steel =	As*fy = 80 kip		
a [in] = beta = c [in] =	1.05 in 0.85 1.23 in	(AASHTO 5.7.2.2) = a/beta				
Check Steel Yield e_s =	0.0788	> e_y steel yields	= 0.0021			
Arm [in] =	24.04 in			.,-	D/O . C . C . C	
$ \phi \text{ Mn [k-in]} =  \phi \text{ Mn [k-ft]} =  $	1,923 kip-in 160 kip-ft	< Demand = < Demand =	7,032 586 kip-ft	NG NG	D/C= 3.66	

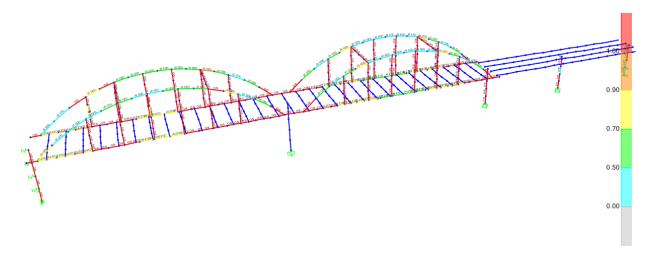
### **Vertical Hanger Analysis:**

As-built details of Portal sections & reinforcement:

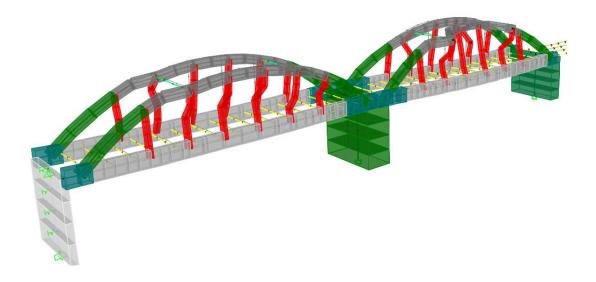




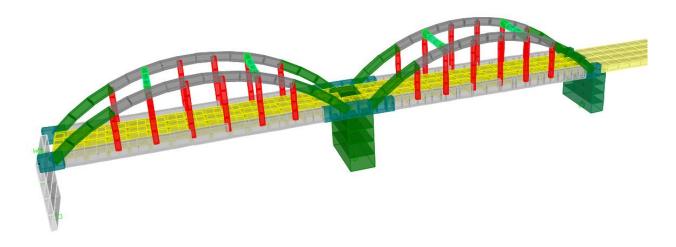
Analysis below shows the limits where the entire Hanger set exceed D/C ratio of 1.0 in Red:



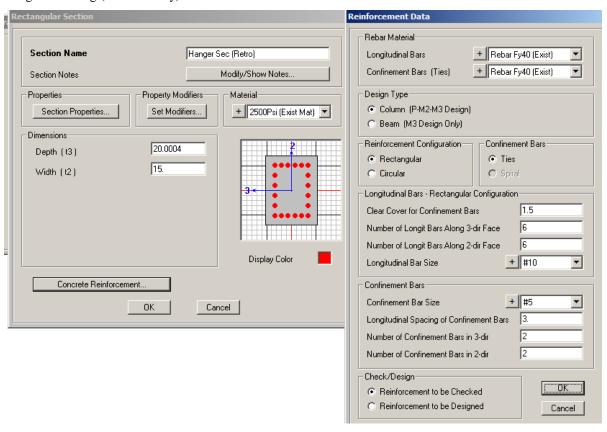
Analysis below shows the deformation from seismic excitation:

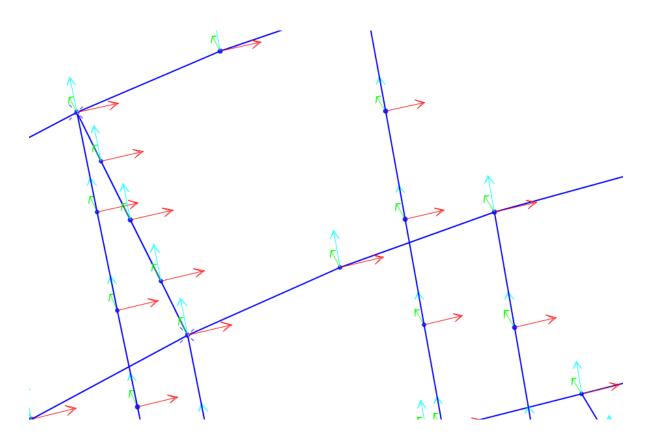


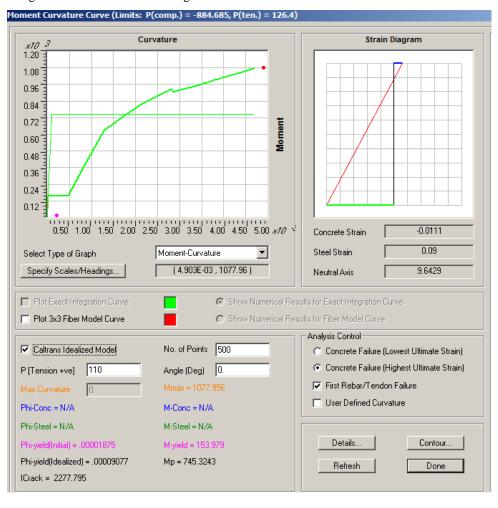
Red block below indicates the approximate location of where the Hanger will be retrofitted.



#### Hanger modeling (iteration only):







Project: Rumsey

Job No.

BY

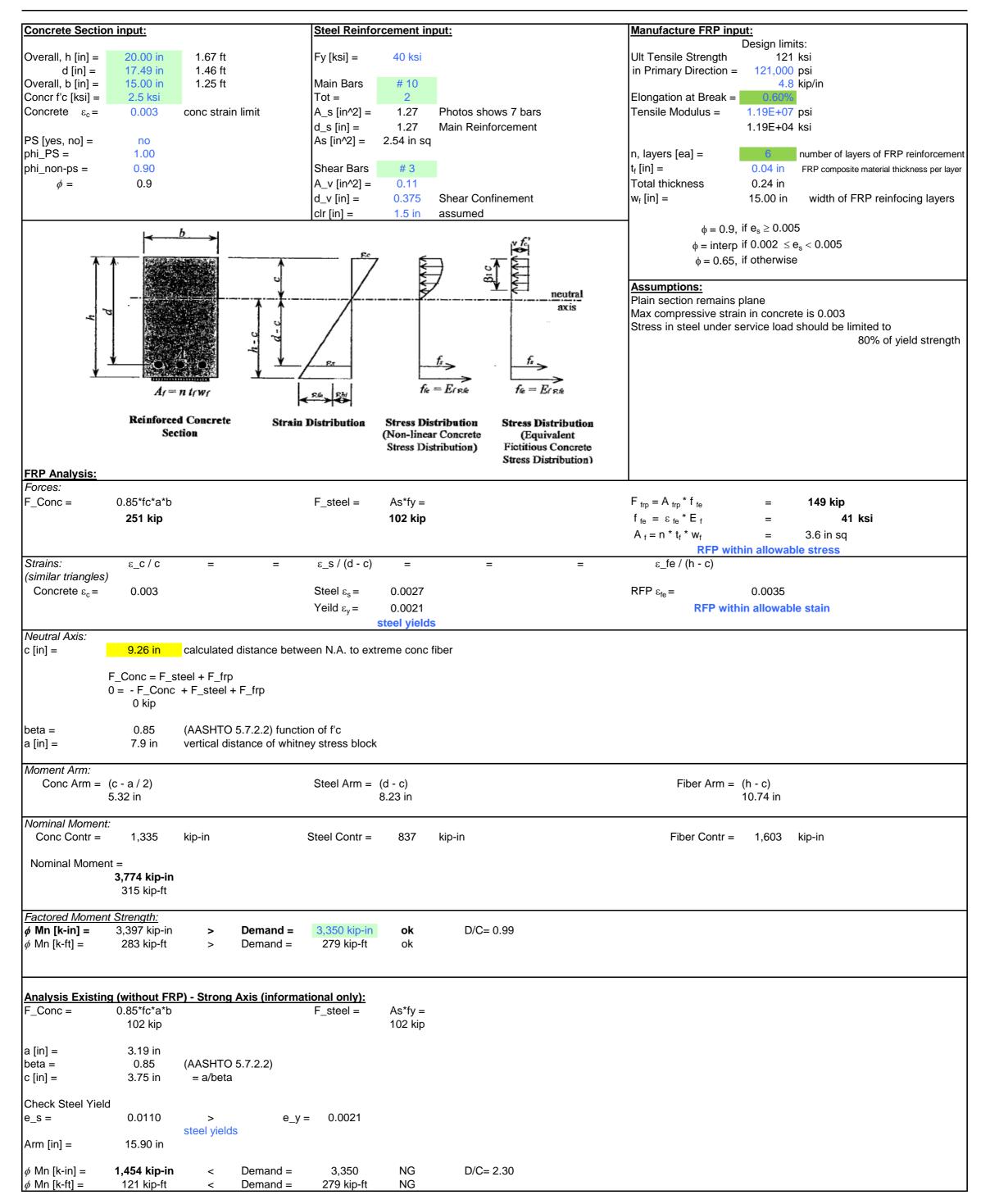
Description

Fiber Reinforcement Polymer

Hanger

Strong axis (Retrofitted)

SHEET



 Project:
 Rumsey
 Description
 Fiber Reinforcement Polymer

 Job No.
 Hanger
 SHEET

 BY
 JC
 DATE 5/12/2014
 Shear & Axial (Retrofitted)

=		-	ongStrong Axis:	<u>:</u>		Shear Bars	# 3	Additional Shear RFP	added	
$\phi V_n$ , <sub>T&amp;B</sub> [k] =	$(A_{fv} F_{fe} (sin(a)))$	$\alpha$ ) + cos( $\alpha$	) ) d <sub>fv</sub> ) / s <sub>f</sub>			d, depth of strup	17.75 in	n, layers [ea] =	1	number of layers of Shear FRF
	682 kip		1.28			Spacing	12.00 in	$A_{fv} = 2 n t_f w_f$	30 in sq	
Vc = 2*sqrt(fc' )*	o*d	(LDF 8.10	6.6.2)			mult by 2 sides	2	F <sub>fe</sub> , FRP eff stress=	15.2 ksi	
	26 kip		Vs_limit = 8*\$	SQRT(fc')*b*c	1/1000	Fy [ksi] =	40 ksi	$\boldsymbol{\alpha}$ , shear angle=	20 degrees	= estimated, conservative
Vs = Av*fy*d/s	13 kip	(LDF 8.10	6.6.3)	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 <sub>f</sub> , FRP spacing=	12.00 in	
V_Exist [kip] = V_Retro [kip] =	39 <i>kip</i> <b>721 kip</b>	>	Demand = Demand =	35 kip 35 kip	ok ok	D/C= 0.89 D/C= 0.05		Strength Flexural = Flexural Modulus =	15.2 ksi 384.2 ksi	psi psi
Analysis Axial T		th:								
$\phi P_n[k] =$	= As n Fy					Main Bars	# 10			
	203 kip					Tot =	4			
φ =	0.9					A_s [in^2] = Fy [ksi] =	1.27 40 ksi			
φ P <sub>n</sub> [k] =	183 kip	>	Demand =	110 kip	ok	D/C= 0.60				

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

SHEET

Concrete Section in	nput:		Steel Reinforcem	ent input:	<u>:</u>	Manufacture FRP input:	$\neg$
Concrete Section in  Overall, h [in] = d [in] = Overall, b [in] = Concr f'c [ksi] = Concrete $\varepsilon_c$ =  PS [yes, no] = phi_PS = phi_non-ps = $\phi$ =	15.00 in 12.49 in 20.00 in 2.5 ksi 0.003 no 1.00 0.90 0.9	1.25 ft 1.04 ft 1.67 ft  conc strain limit	Fy [ksi] = 40  Main Bars  Tot =  A_s [in^2] = 1  d_s [in] = 1  As [in^2] = 2.56  Shear Bars  A_v [in^2] = 0  d_v [in] = 0	0 ksi # 10 2 1.27 Ph 1.27 Ma 4 in sq # 3 0.11	notos shows 7 bars ain Reinforcement near Confinement ssumed	$\begin{array}{c ccccccccccccccccccccccccccccccccccc$	layer
R	einforced Concre Section	ete Strain Distribut	-1	bution concrete	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 stren	gth
FRP Analysis: Forces: F_Conc =	0.85*fc*a*b <b>224 kip</b>			s*fy = 1 kip		$F_{frp} = A_{frp} * f_{fe} = 122 \text{ kip}$ $f_{fe} = \varepsilon_{fe} * E_{f} = 51 \text{ ksi}$ $A_{f} = n * t_{f} * w_{f} = 2.4 \text{ in sq}$	
Strains: (similar triangles) Concrete $\varepsilon_c$ =	ε_c / c 0.003	= =	Yeild $\varepsilon_y = 0$ .	= 0031 0021 yields	= =	RFP within allowable stress $\epsilon_{-} \text{fe / (h - c)}$ RFP $\epsilon_{\text{fe}}$ = 0.0043 RFP within allowable stain	
	6.19 in _Conc = F_steel + = - F_Conc + F_ 0 kip				r		
beta = a [in] =	0.85 5.3 in	(AASHTO 5.7.2.2) function vertical distance of white					
Moment Arm: Conc Arm = (c 5.	- a / 2) 32 in		Steel Arm = (d - 8.23			Fiber Arm = (h - c) 5.74 in	
Nominal Moment: Conc Contr =  Nominal Moment =	1,190 : 2,309 kip-in 192 kip-ft	kip-in	Steel Contr =	418 kip	o-in	Fiber Contr = 701 kip-in	
<u>Factored Moment Si</u> <b>φ Mn [k-in] =</b> φ Mn [k-ft] =	<u>trength:</u> 2,078 kip-in 173 kip-ft	> Demand = > Demand =		<b>ok</b> Dok	D/C= 0.91		
Analysis Existing (v F_Conc =	without FRP) - W 0.85*fc*a*b 102 kip	Veak Axis (informational	F_steel = As	s*fy = )2 kip			
a [in] = beta = c [in] =	2.39 in 0.85 2.81 in	(AASHTO 5.7.2.2) = a/beta					
Check Steel Yield e_s = Arm [in] =	0.0157 16.29 in	> e_y steel yields	= 0.0021				
$\phi$ Mn [k-in] = $\phi$ Mn [k-ft] =	1,490 kip-in 124 kip-ft	< Demand = Compand = Compa		NG D NG	D/C= 1.28		

			(	QUINCY ENGINEERING, INC.	
Project:	Rumsey			Description Vertical Hanger	
Job No.	Y01-500			Rebar Tension Yield	SHEET
BY	JC	DATE	5/12/2014		
ĺ		_			

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

$$\begin{array}{ll} T_n &= Fy * As \\ T_n & 253 & kips \\ \phi T_n & 253 & kips \end{array}$$

### Demand

$$P_u$$
 115 kips

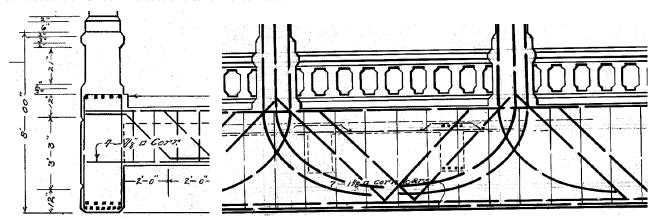
D/C 0.45

				QUINCY E	NGINEER	ING, INC.				
Project:	Rumsey				Vertical Ha	inger				
Job No	Y01-500			Shear yy (LF	RDF 5.8.3.3)				_	SHEET
3Y	JC	DATE	5/12/2014	<u>.</u>					_	
RFD 5.8.3	3									
	n of the following	2 equations		Ecco Results:						
/n = Vc + \	,	, . ,		Vn =	1 kip	<	Demand =	15 kip	NG	
/n = 0.25f'o	bv dv + Vp							15.00		
) =	15 in			Fy [ksi] =	40 ksi					
=	20 in			01 5	" 0					
C =	300 in2 0,000 psi	0.0 ksi	worst case	Shear Bars Spacing	# 3 12.00 in	enacing of c	etirrun			
c [ksi] =	0,000 psi		mber in tension		0.11 in2	spacing of s	surrup			
ov =	15 in	eff web width	ilibei ili terision	d_v_bar =	0.11 iii2 0.38 in					
Iv =	17 in	eff shear dept	h	mult by	2	number of b	ars per plane			
-		J. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1.		Av	0.02 in		ar reinfocemer	nt w/in dist s		
	5 0 0 5 Mi	·	D-i							
	5.8.2.5 Min	imum Transve								
			qrt (f'c) b <sub>v</sub> s / fy		0.00: 0	0.0040	mt (f) = \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \			
		Av =	0.02 in2	ok	0.00 in2	= 0.0316  sq	rt (f'c) bv s / f	У		
93430	eneral Procedu	re								
.o.s.4.2 G		<b>re</b> / + 0.5 Nu +  Vu	-Vnl - Anefna V	/ (Fe Ae + En ^	ne)					
	es = (iviu/u	7 + 0.5 Nu +  VU Mu =	32184 k-ft	Moment dema						
		Nu =	653 k	Axial demand						
		Vu =	441 k	Shear demand	1					
		Vp =	0 k	component in		plied shear of	the effective r	orestressina	force	
		٠,٢	0.1	positive if resi			0000	,, oo oo o		
		Aps =	0.00 in2	Area of PS	9					
		fpo =	175 ksi	0.7 fpu		Flexural stee	el info:			
		Es =	29000 ksi	i i		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							
			dv =	17 in	lear a l					
			d =	36 in			crack control r	eint		
		ag =	0.25 in	max agg size	assumed per	pnotos				
		Sxe =	26.7							
	theta = 29 -	L 3500 AC								
	theta = 29	65 65	degrees	angle of incline	ation of diagon	al comp. stres	ss			
		55	2091000	3g.0 01 11 10 11 16	o or diagon	Jonny. 30163				
	If Min Trans	sverse Reinford	ement is met							
		.8 / (1+ 750 εs)								
	beta =		factor indicating	ability of diago	nal cracked co	ncrete to tran	smit tension &	shear		
		sverse Reinford								
		.8 / (1+ 750 εs)								
	beta =			ability of diago	nal cracked co	ncrete to tran	smit tension &	shear		
				2						
Sele	ect: beta =			ability of diago		ncrete to tran	smit tension &	shear		
	theta =			ion of diagonal						
Sele	ect: alpha =	90 degrees	angle of inclinat	ion of transvers	e reinforceme	n				
	Vn = Vc + \	/s + Vn								
		75 + vp 316*beta*sgrt(f'	c) by dy							
	Vc = 0.0 Vc =	0 kip	.,							
		fy sin(alpha) < (	).095 sart(f'c) h	v dv						
	Vs =	0.66 kip	<	0 kip						
	Vn =		Gov							
				-						
	Vn = 0.25f'	bv dv + Vp								
	Vn =	0 kip								
		•								
	$\phi =$	0.90	Seimsic phi for	shear						
						_				
Sele	ect:	1 kip	<	Demand =	15 kip	NG				
					15.00					

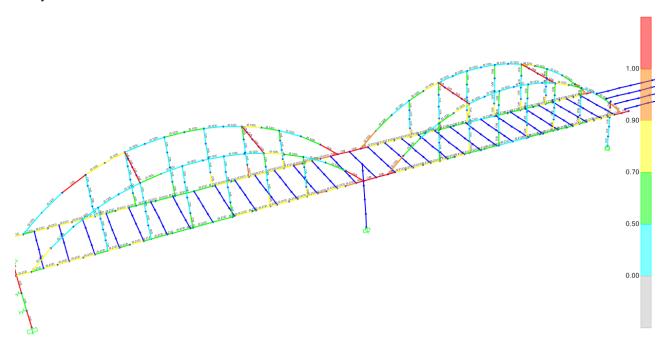
				QUINCY E	NGINEER	ING, INC.				
Project:	Rumsey			Description	Vertical Ha	anger				
Job No	Y01-500			Shear xx (LF	RDF 5.8.3.3)				_	SHEET
BY	JC	DATE	5/12/2014						_	
.RFD 5.8.3.3 /n – minium	of the followin	a 2 equations		Ecco Results:						
/n = Vc + Vs		g z equations		f V [kip] =	1 kip	<	Demand =	30 kip	NG	
/n = 0.25f'c				[[.				30.00		
) = I =	20 in 15 in			Fy [ksi] =	40 ksi					
Ac =	300 in2			Shear Bars	# 4					
c [ksi] =	0,000 psi	0.0 ksi	worst case	Spacing	18.00 in	spacing of	stirrup			
			mber in tension		0.20 in2					
V =	20 in	eff web width		d_v_bar =	0.50 in					
v =	12 in	eff shear dept	n	mult by Av	2 0.02 in		bars per plane ar reinfocemer	nt w/in dist s		
	5.8.2.5 Mir	nimum Transve								
			qrt (f'c) b <sub>v</sub> s/fy		0.001.5	0.6515				
		Av =	0.02 in2	ok	0.00 in2	= 0.0316 sc	qrt (f'c) bvs/f	У		
.8.3.4.2 Ge	neral Procedu									
	es = (Mu/d	lv + 0.5 Nu +  Vu								
		Mu = Nu =	13063 k-ft	Moment dema Axial demand	ind					
		Vu =	653 k 263 k	Shear demand	1					
		Vu = Vp =	0 k	component in		oplied shear o	f the effective	orestressina	force	
		·		positive if resis			,			
		Aps =	0.00 in2	Area of PS						
		fpo =	175 ksi	0.7 fpu		Flexural ste				
		Es =	29000 ksi			Fy [ksi] =	40 ksi			
		Ep = As =	29000 ksi 5.08 in2	flexural steel		Flex Bars	# 10 1.27 in2			
		A5 =	5.06 1112	ilexurai steei		A_s_bar = d_s_bar =	1.27 in			
		es =	0.0114			mult by	4	approx effect	ctive	
	Svo - cv *	1.38 / (ag + 0.63	1)							
	0x0 = 3x		nin 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	r photos				
		Sxe =	18.8							
	theta = 29 theta =	+ 3500 es 69	degrees	angle of inclina	ation of diagor	nal comp. stre	ess			
	If Min Tron	sverse Reinforce	· ·	· ·	J					
		isverse Reimorα 1.8 / (1+ 750 εs)	ement is met							
	beta =	,	actor indicating	ability of diago	nal cracked co	oncrete to trai	nsmit tension 8	shear		
		sverse Reinforce								
		1.8 / (1+ 750 εs)	•	,						
	beta =	0.45	factor indicating	ability of diago	nal cracked co	oncrete to trai	nsmit tension 8	shear		
Saloo	t: beta =	0.5 degrees	factor indication	ability of diago	nal cracked or	norete to tra	nemit teneion 9	. shear		
Selec	theta =			ion of diagonal		SHOLETE TO HAI	STILL LEUSION C	SIICAI		
Selec	t: alpha =			ion of transvers		en				
	Vn = Vc +	Vs + Vp								
		)316*beta*sqrt(f'	c) bv dv							
	Vc =	0 kip								
		fy sin(alpha) < 0								
	Vs = <b>Vn =</b>	0.79 kip <b>1 kip</b>	< Gov	0 kip						
				•						
	Vn = 0.25f Vn =	'c bv dv + Vp 0 kip								
	$\phi =$	0.90	Seimsic phi for	shear						
Selec	: øV[kip] =	1 kip	<	Demand =	30 kip	NG				
Jeiec	7 - [mp] -	TRIP	`	Joinana –	30.00	140				

### **Tie Girder Retrofit:**

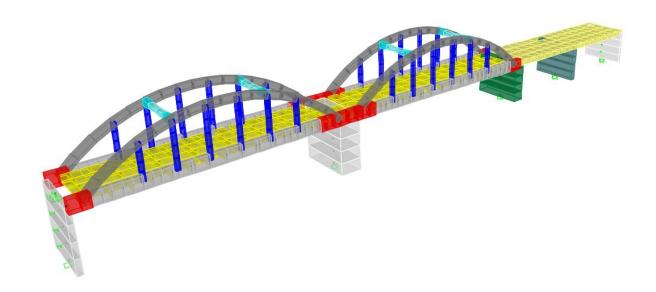
As-built details of Girder-Tie sections & reinforcement:

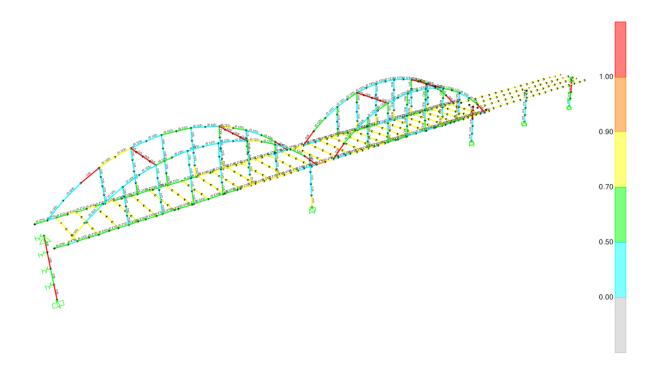


Analysis below shows the limits where the Girder-Ties exceed D/C ratio of 1.0 in Red:

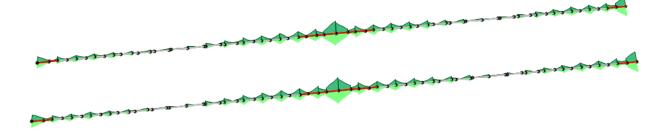


Red block below indicates the approximate location of where the Girder Ties will be retrofitted.





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



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

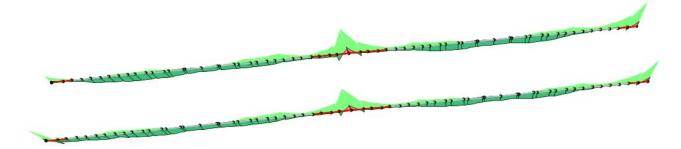


Photo of existing Girder-Ties.



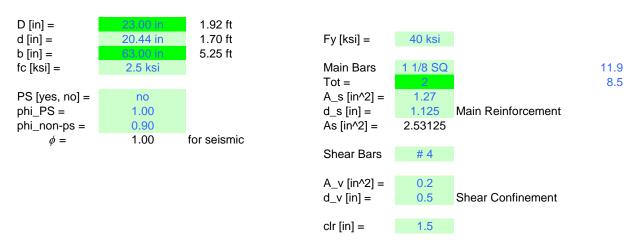
Project:	Rumsey			Description <b>Bending</b>	
Job No.				<b>Girder Tie Strong Axis xx</b>	SHEET
BY	JC	DATE	5/12/2014		

D [in] = d [in] = b [in] =	63.00 in 60.44 in 23.00 in	5.25 ft 5.04 ft 1.92 ft	Fy [ksi] =	40 ksi		
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		
$\phi =$	1.00	for seismic				
			Shear Bars	# 4		
			A 51 401	0.0		
			A_v [in^2] =	0.2	0, 0, ",	
			d_v [in] =	0.5	Shear Confinement	
			clr [in] =	1.5		

### Analysis:

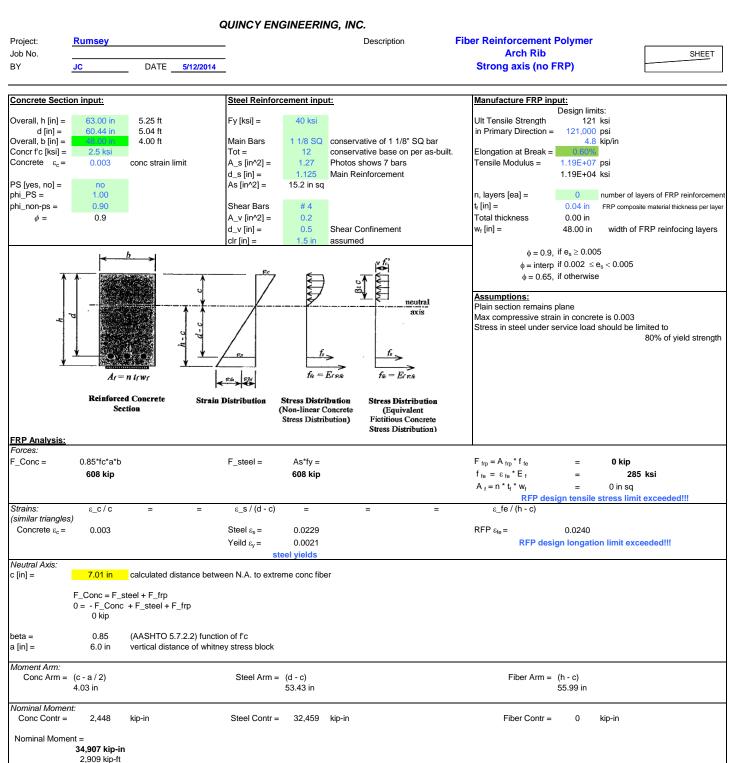
F_steel =	As*fy = 608 kip	F_Conc =	0.85*fc*a*b 608 kip		
a [in] = beta = x [in] =	12.43 in 0.85 a/beta 14.62 in	(AASHTO 5.7.2.2)			
Check Steel Yield e_s =	d 0.0094	> e_y = steel yields	0.0021		
Arm [in] =	54.22 in				
$\phi$ Mn [k-in] = $\phi$ Mn [k-ft] =	32,940 kip-in 2,745 kip-ft	> Demand = > Demand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

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



#### Analysis:

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

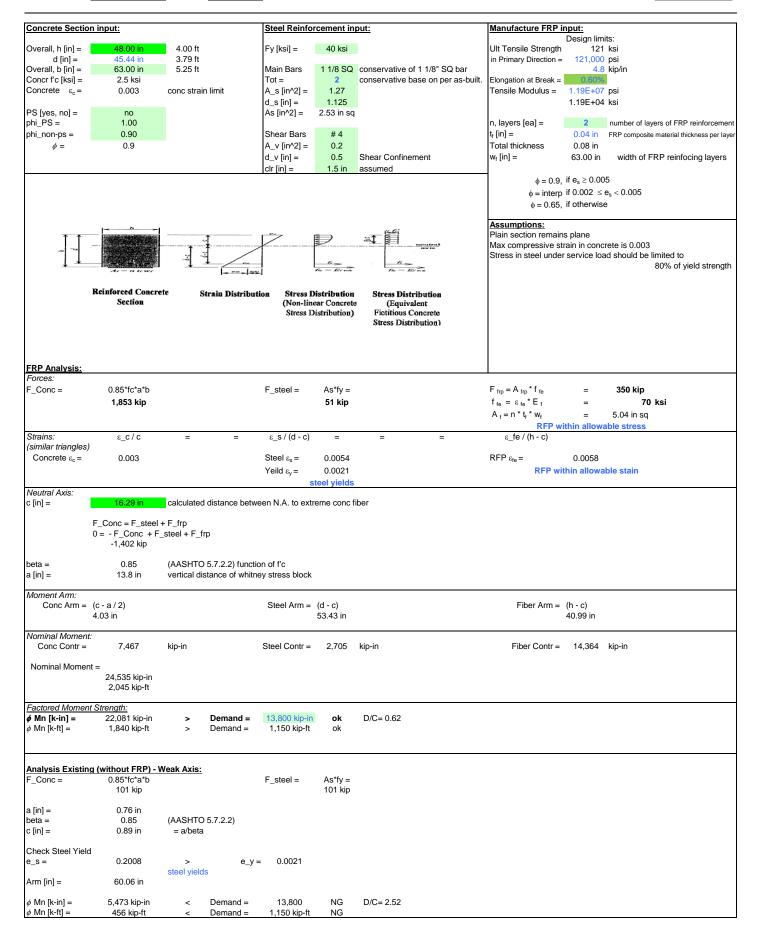


c [in] =	7.01 in	calculated	distance between	een N.A. to extren	ne conc fib	er		
	F_Conc = F_s 0 = - F_Conc 0 kip							
beta =	0.85		5.7.2.2) function					
a [in] =	6.0 in	vertical dis	stance of whitne	ey stress block				
Moment Arm: Conc Arm =	(c - a / 2) 4.03 in			Steel Arm =	(d - c) 53.43 in		Fiber Arm = (h - c) 55.99 in	
Nominal Momer Conc Contr =	nt: 2,448	kip-in		Steel Contr =	32,459	kip-in	Fiber Contr = 0 I	kip-in
Nominal Mome	nt = <b>34,907 kip-in</b> 2,909 kip-ft							
Factored Mome	nt Strength: 31,416 kip-in		Demand =	26,400 kip-in	ok	D/C= 0.84		
φ Mn [k-ft] = φ Mn [k-ft] =	2,618 kip-ft	>	Demand =	2,200 kip-ft	ok	D/C= 0.64		
Analysis Existi	na (without FR	P) - Strong	ı Axis:					
F_Conc =	0.85*fc*a*b 608 kip	,		F_steel =	As*fy = 608 kip			
a [in] =	5.96 in							
beta =	0.85 7.01 in	(AASHTO = a/beta						
c [in] =	7.01 III							
c [in] = Check Steel Yie		> stool yield	e_y :	= 0.0021				
c [in] =	ld	> steel yield		= 0.0021				

Description

Fiber Reinforcement Polymer Arch Rib Weak axis (Retrofitted)

SHEET



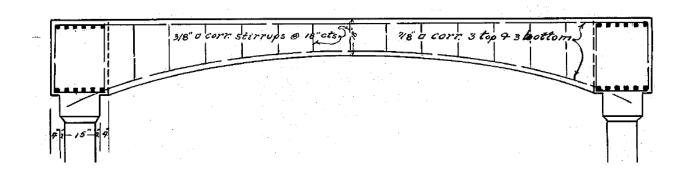
					QUINCY ENG	INEERING, INC.	•			
Project:	Rumsey				Description Gird	ler Tie				
lob No.	Y01-500				Concrete Axial					SHEET
BY	JC	_	DATE	5/12/2014	(LRFD 5.7.4)	,				
ompressio	n Member Strer	ngth								
fc' = t =	2500 63	psi in				Main Bars	# 10	Main Reinfo	rcement	
b =	23	in				Tot = A_s [in^2] =	6 1.27			
d =	60.365	in				d_s [in] =	1.27			
d = d' =	2.635	in				u_s [iii] = As [in^2] =	7.62			
As =	7.62	in2				A3 [III 2] =	7.02			
						Choor Poro	# 1	Shoor Confi	aamant	
As' =	7.62	in2				Shear Bars	# 4	Shear Confi	nement	
,	40.000					Spacing [in] =	18			
fy =	40,000	psi				A_v [in^2] =	0.2			
beta1 =	0.85					d_v [in] =	0.5			
Pu =	635 kip	Demand				clr [in] =	2			
						Comp Bars	# 10	Main Reinfo	rcement	
						Tot =	6			
						A_s' [in^2] =	1.27			
						d_s' [in] =	1.27			
						As' [in^2] =	7.62			
						Reinforcement Max Reinforcen Ast/Ag < 0.08 Ast/Ag = Ast/Ag =	nent	prime)/(t*b)	0.08	ok
						Min Reinforcem	ent			
						Ast/Ag > 0.0025		vall and 0.01	for comp r	nemher
						Ast/Ag =		_prime)/(t*b)	ioi comp. i	Hellibei
						Ast/Ag =	0.012	-pιιιιο <i>)</i> /(ι υ)	0.010	ok
									0.010	ÜK
						Min bar # 5, Ch	eck			
	ression (LRFD 0*fc' (*b*t - As -	,	⊥ Λc' \*f	v1	(8-30) for ties	c-c < 12", Chec	k			
φ =	1.00	A3) 1 (A3	173 ) 1	у]	(0-30) 101 1103	Lateral Reinfo	rcoment (	BDS 8 18 2)		
φ – φ Po =	3,477,120	lhe				spacing > min(1				
φ Po =	3,477,120 3,477 kips	>		635 kip	ok	Greater than #3			eck	
				0.18		Ties (BDS 8.18	3.3)			
						h_wall =	276			
						Ash =	6.5333	in2		
						h_c =	58.5			
						Ag =	1449	in2		
						Ac =	1341			
						0.30*s_t*h_c*f	c' /fy *(Ag/	Ac-1) =		
							1 50	in?		
							1.59	IIIZ		
						6.53 in2	7.59	Demand =	1.59 in2	ok

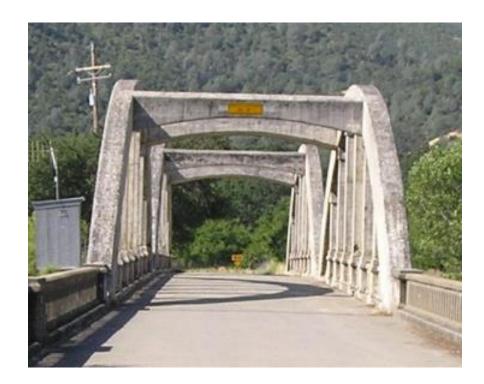
				<b>QUINCY E</b>	NGINEER	ING, INC.				
Project:	Rumsey			Description	Girder Tie	,				
Job No.	Y01-500			Shear (LRDF						SHEET
BY	JC	DATE	5/12/2014						<u> </u>	01122
				_					,	
.RFD 5.8.3.3	3									
	of the following	2 equations		Ecco Results:						
/n = Vc + Vs				f V [kip] =	29 kip	<	Demand =	129 kip	NG	
/n = 0.25f'c b	ov dv + Vp							4.45		
=	63 in			Fy [ksi] =	40 ksi					
=	23 in									
C =	1449 in2	0.51-3		Shear Bars	# 4		•			
c [ksi] =	2,500 psi	2.5 ksi		Spacing A_v_bar =	18.00 in 0.20 in2	spacing of s	штир			
v =	63 in	eff web width		d_v_bar =	0.50 in					
v =	20 in	eff shear dep		mult by	2	number of ba	ars per plane			
				Av	0.02 in	area of shea	r reinfocemen	t w/in dist s		
	5.8.2.5 Min		rse Reinforcem							
			sqrt (f'c) b <sub>v</sub> s/fy							
		Av =	0.02 in2	NG	1.42 in2	= 0.0316 sq	t (f'c) bvs/fy	,		
nalysis:	neral Procedur	•								
.o.o.4.∠ Ger			ı-Vp  - Apsfpo ) /	(Es As + Fn An	os)					
	00 - (	Mu =	32184 k-ft	Moment dema						
		Nu =	653 k	Axial demand						
		Vu =	441 k	Shear demand	I					
		Vp =	0 k	component in			the effective p	restressing f	orce	
		۸	0.001.0	positive if resis	sting the applie	ed shear				
		Aps = fpo =	0.00 in2 175 ksi	Area of PS		Florural at-	al info:			
		1po = Es =	29000 ksi	0.7 fpu		Flexural stee 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 =	0.0092			mult by	7			
	Sxe = sx * 1	.38 / (ag + 0.63	3)							
		sx =	min of following							
			dv =	20 in						
			d =	36 in		layers of long	crack control r	einf		
		ag = Sxe =	0.25 in 31.4	max agg size	assumed per	photos				
	4									
	theta = 29 + theta =	- 3500 es 61	degrees	angle of inclina	ation of diagon	nal comp stres	s			
			· ·	angle of monte	on diagor	oomp. onco	~			
		verse Reinford								
	beta = 4. beta =	8 / (1+ 750 εs) 0.61	factor indicating	ability of diago	nal cracked or	norata to trans	mit tancion 9	shear		
			ement is NOT m		nai ciackeu CC	morete to trans	mini renoiun &	oi icdi		
			x 51/(39+S							
	beta =	, ,	factor indicating	,	nal cracked co	oncrete to trans	mit tension &	shear		
Selec	t: beta =	0.4 degrees	factor indicating	ability of diago	nal cracked co	oncrete to trans	mit tension &	shear		
23.30	theta =		angle of inclinat							
Selec	t: alpha =		angle of inclinat	•		n				
	Vn = Vc + V	/s + Vp								
		316*beta*sqrt(f	c) bv dv							
	Vc =	28 kip	•							
		, , ,	0.095 sqrt(f'c) by							
	Vs = <b>Vn =</b>	0.79 kip <b>29 kip</b>	< Gov	189 kip						
	V 11 =	∠a vih	JUV							
	Vn = 0.25f'c									
	Vn = 0.25f'c Vn =	bv dv + Vp 788 kip								
			Seimsic phi for	shear						
Selec	Vn =	788 kip	Seimsic phi for	shear Demand =	129 kip	NG				

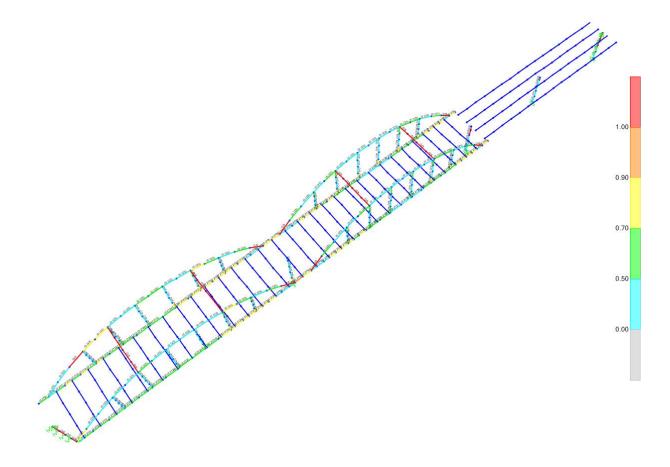
				QUINCY E	NGINEER	ING, INC.			
roject:	Rumsey			Description	Girder Tie				
ob No.	Y01-500			Shear (LRDF	5.8.3.3)				SHE <u>E</u> T
3Y	JC	DATE	5/12/2014						
RFD 5.8.3.									
/n = minium /n = Vc + Vs	of the following	g 2 equations		Ecco Results: f V [kip] =	36 kip	<	Demand =	69 kip <b>I</b>	NG
/n = 0.25f'c				i v [kip] –	30 KIP	`	Demand =	1.91	
=	23 in			Fy [ksi] =	40 ksi				
=	63 in			0. 5					
.c = c [ksi] =	1449 in2 2,500 psi	2.5 ksi		Shear Bars Spacing	# 4 18.00 in	specing of s	tirrun		
c [ksi] =	2,300 psi	2.5 K31		A_v_bar =	0.20 in2	spacing of s	штир		
v =	23 in	eff web width		d_v_bar =	0.50 in				
v =	60 in	eff shear dept	h	mult by	2		ars per plane		
				Av	0.02 in	area of shea	ar reinfocemer	nt w/in dist s	
	5.8.2.5 Min		rse Reinforcem qrt (f'c ) b <sub>v</sub> s / fy						
		Av =	0.02 in2	NG	0.52 in2	= 0.0316 sq	rt (f'c) bvs/f	y	
Analysis:									
	neral Procedu	re							
	es = (Mu/dv		-Vp  - Apsfpo ) /						
		Mu =	13063 k-ft	Moment demai	nd				
		Nu = Vu =	653 k 263 k	Axial demand Shear demand					
		Vu = Vp =	263 K	component in o		nlied shear of	the effective r	arestressing for	rca
		٧p –	U K	positive if resis			o oncouve	101	
		Aps =	0.00 in2	Area of PS	3				
		fpo =	175 ksi	0.7 fpu		Flexural stee			
		Es =	29000 ksi			Fy [ksi] =	40 ksi		
		Ep =	29000 ksi	flexural stool		Flex Bars	# 10 1 27 in2		
		As =	5.08 in2	flexural steel		A_s_bar = d_s_bar =	1.27 in2 1.27 in		
		es =	0.0055			u_s_bar = mult by	4	approx effective	ve
						•			
	Sxe = sx *	1.38 / (ag + 0.63	,						
		sx =	min of following						
			dv = d =	60 in 36 in	dist hatwaan	layers of long	crack control	reinf	
		ag =	0.25 in	max agg size			CIACK COILLOI	TOTAL	
		Sxe =	56.5			,			
	46-4- 00	. 2500							
	theta = 29 - theta =	+ 3500 es 48	degrees	angle of inclina	ation of diagon	al completros	e e		
	uleta =	40	degrees	angle of inclina	mon or diagor	iai comp. stres	00		
		sverse Reinforce	ement is met						
	beta = 4	.8 / (1+ 750 εs)							
	beta =		factor indicating		nal cracked co	oncrete to trans	smit tension 8	shear	
			ement is NOT m						
	beta = 4 beta =		x 51 / (39 + S factor indicating		nal cracked co	oncrete to trans	smit tension &	shear	
	2014 -	3.01		Si diagoi	0.00100 00				
Selec	t: beta =		factor indicating			oncrete to trans	smit tension 8	shear	
<u> </u>	theta =		angle of inclinat	•					
Selec	t: alpha =	90 degrees	angle of inclinat	ion of transverse	e reinforceme	n			
	Vn = Vc + V								
		316*beta*sqrt(f	c) bv dv						
	Vc = ^\/	35 kip	) 005 cart(fla) L.	, dv					
	VS = AV VS =	ty sin(aipna) < 0 0.79 kip	0.095 sqrt(f'c) bv <	av 207 kip					
	Vn =		Gov	_0. Np					
	Vn = 0.25f/	c bv dv + Vp							
	Vn = 0.2310	863 kip							
	φ =	1.00	Seimsic phi for	shear					
0.1	,				00.11	NO			
Selec	t: ø V [kip] =	36 kip	<	Demand =	69 kip 1.91	NG			

## **Portal Bracing:**

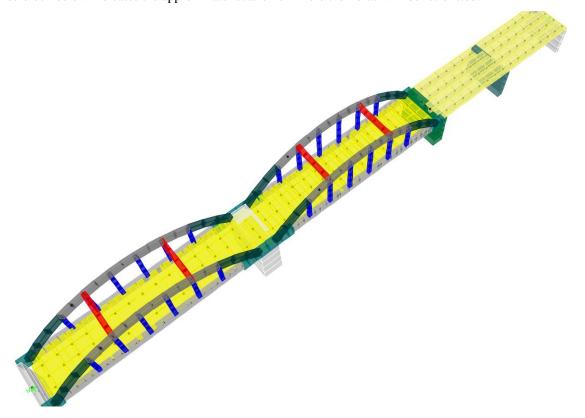
As-built details of Portal sections & reinforcement:

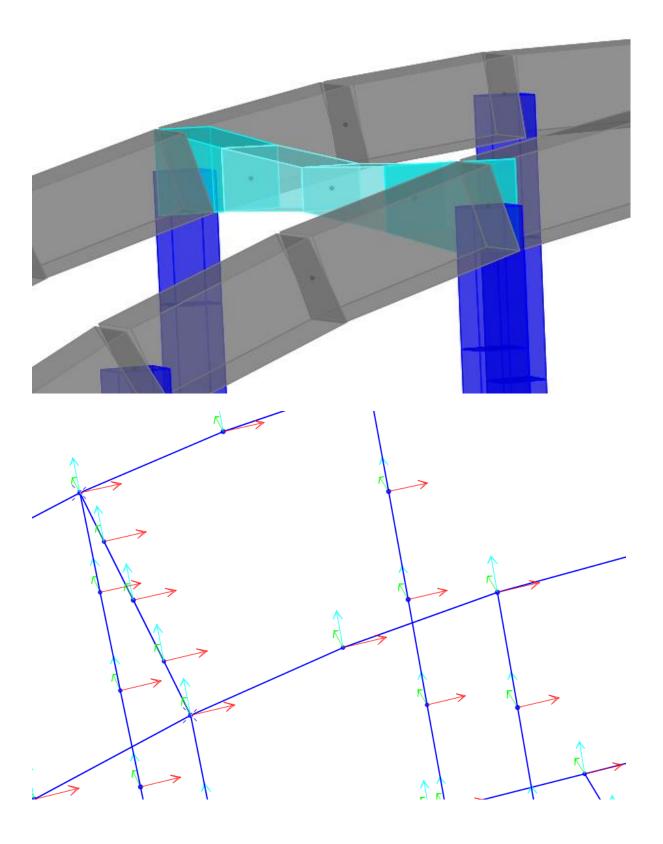


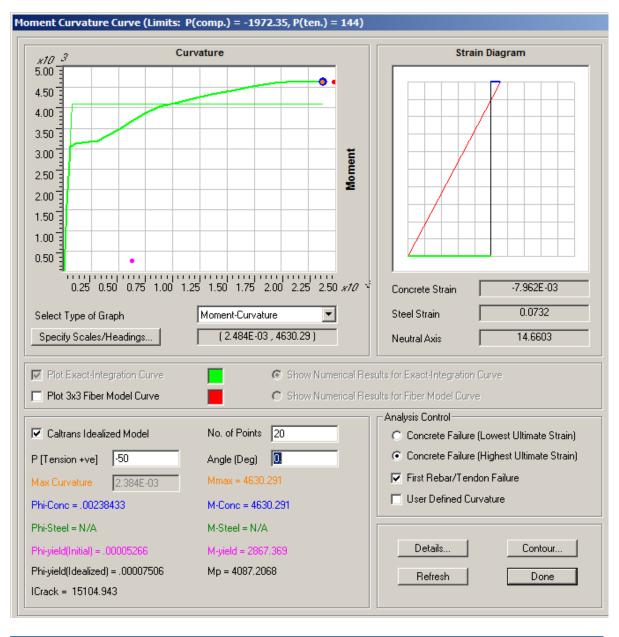


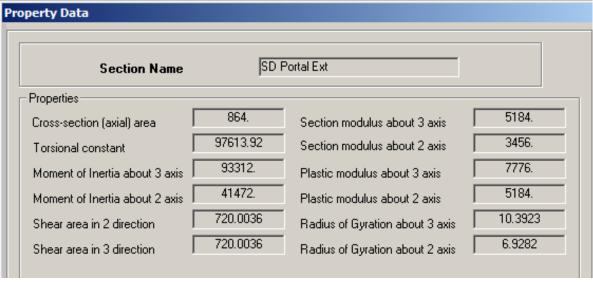


Red block below indicates the approximate location of where the Portal will be retrofitted.







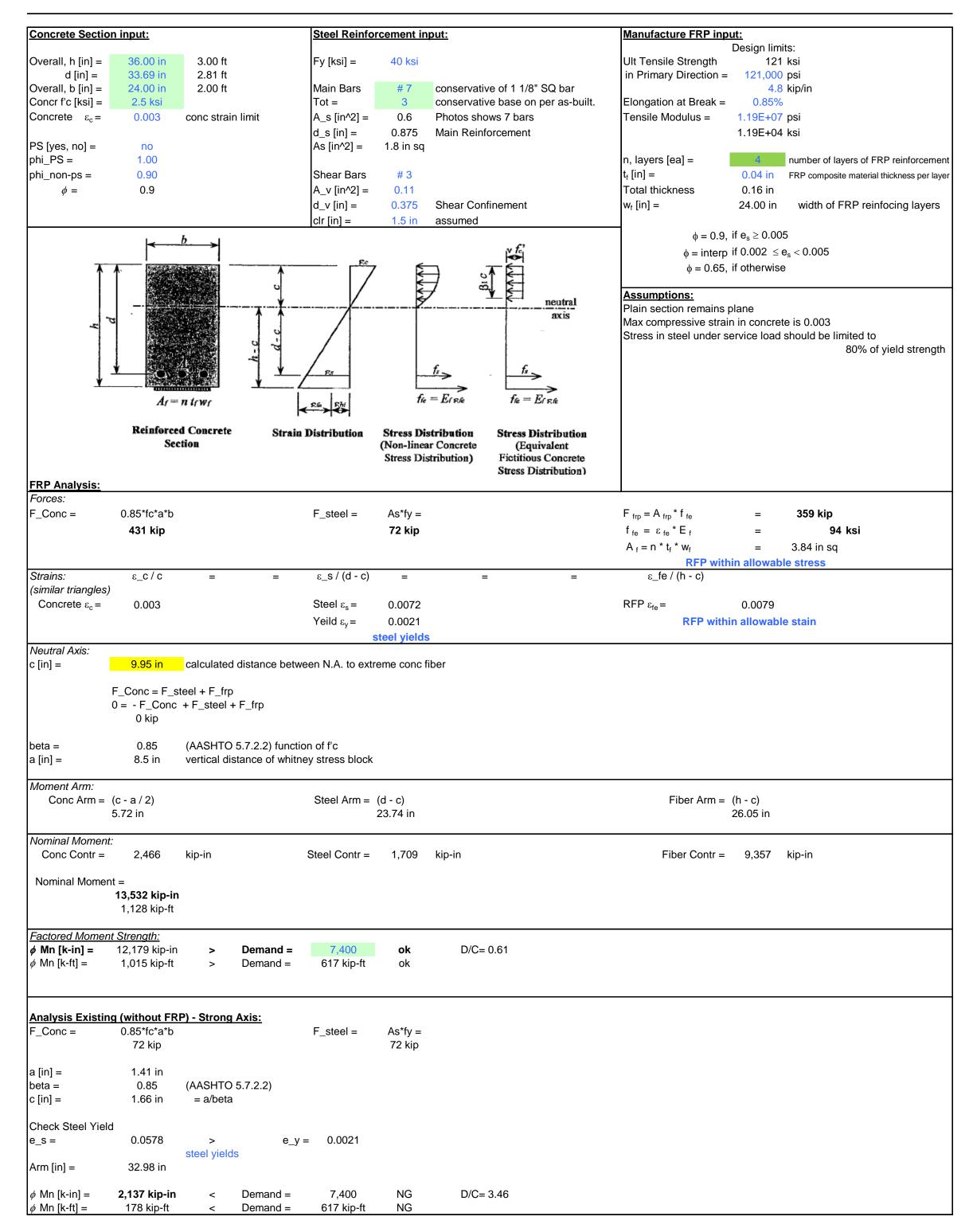




Description

Fiber Reinforcement Polymer
Portal
Strong axis (Retrofitted)

SHEET



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

SHEET

Concrete Section	innut:		Steel Reinfo	rcement in	nut·	Manufacture FRP input:
	<del>-</del>	0.00 %			<u>μαι.</u>	Design limits:
Overall, h [in] = d [in] =	24.00 in 21.69 in	2.00 ft 1.81 ft	Fy [ksi] =	40 ksi		Ult Tensile Strength 121 ksi in Primary Direction = 121,000 psi
Overall, b [in] = Concr f'c [ksi] =	36.00 in 2.5 ksi	3.00 ft	Main Bars Tot =	# 7 2	conservative of 1 1/8" SQ bar conservative base on per as-built.	4.8 kip/in Elongation at Break = 0.85%
Concrete $\varepsilon_c =$	0.003	conc strain limit	A_s [in^2] =	0.6	Photos shows 7 bars	Tensile Modulus = 1.19E+07 psi
PS [yes, no] =	no		d_s [in] = As [in^2] =	0.875 1.2 in sq	Main Reinforcement	1.19E+04 ksi
phi_PS =	1.00					n, layers [ea] =
$phi_non-ps = $ $\phi =$	0.90 0.9		Shear Bars A_v [in^2] =	# 3 0.11		$t_f$ [in] = 0.04 in FRP composite material thickness per layer Total thickness 0.16 in
Ψ	0.0		d_v [in] =	0.375	Shear Confinement	$w_f$ [in] = 36.00 in width of FRP reinfocing layers
			clr [in] =	1.5 in	assumed	$\phi = 0.9$ , if $e_s \ge 0.005$
						$\phi = \text{interp if } 0.002 \le e_s < 0.005$
						$\phi$ = 0.65, if otherwise
	· · · · ·				r.Ei	Assumptions:
		<b>→</b>			neutral	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
	Ar = n trwr	S. C. S.	4	fa = Erna	fa - Erna	
	Reinforced Concre Section	ete Strain Distribut		Distribution near Concret		
	Jeanou		•	Distribution)	Fictitious Concrete	
					Stress Distribution)	
FRP Analysis:						
Forces: F_Conc =	0.85*fc*a*b		F_steel =	As*fy =		$F_{frp} = A_{frp} * f_{fe} = 445 \text{ kip}$
	493 kip			24 kip		$f_{fe} = \varepsilon_{fe} * E_f$ = 77 ksi
						$A_f = n * t_f * w_f = 5.76 \text{ in sq}$ RFP within allowable stress
Strains:	ε_c / c	= =	ε_s / (d - c)	=	= =	ε_fe / (h - c)
(similar triangles) Concrete $\varepsilon_c$ =	0.003		Steel $\varepsilon_{\rm s}$ =	0.0056		RFP $\varepsilon_{\text{fe}}$ = 0.0065
			Yeild $\epsilon_y$ =	0.0021		RFP within allowable stain
Neutral Axis:		_		steel yields		
c [in] =	7.58 in	calculated distance betwe	een N.A. to ext	reme conc f	iber	
	F_Conc = F_steel = 0 = - F_Conc + F_					
	0 kip	_0.00p				
beta =	0.85	(AASHTO 5.7.2.2) function				
a [in] =	6.4 in	vertical distance of whitne	ey stress block			
Moment Arm: Conc Arm =	(0, 2/2)		Steel Arm =	(d c)		Fiber Arm = (h - c)
	5.72 in		Steel Allii =	23.74 in		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:					
<b>φ Mn [k-in] =</b> φ Mn [k-ft] =	8,681 kip-in 723 kip-ft	> <b>Demand =</b> > Demand =	7,000 583 kip-ft	<b>ok</b> ok	D/C= 0.81	
<i>φ</i> ινιιι [ιν⁻ιι.] <b>–</b>	120 NIP-IL	> Demanu =	JOJ KIP-II	UK		
Analysis Existing F_Conc =	<b>y (without FRP) - V</b> 0.85*fc*a*b	<u>Veak Axis:</u>	F_steel =	As*fy =		
	48 kip		-	48 kip		
a [in] =	0.63 in	/AAQ::=== :				
beta = c [in] =	0.85 0.74 in	(AASHTO 5.7.2.2) = a/beta				
Check Steel Yield						
e_s =	0.1339	> e_y =	= 0.0021			
Arm [in] =	33.37 in	steel yields				
φ Mn [k-in] =	1,442 kip-in	< Demand =	7,000	NG	D/C= 4.86	
$\phi$ Mn [k-ft] =	120 kip-ft	< Demand =	583 kip-ft	NG		

Project   Rumsey					QUINCY E	NGINEER	ING, INC.				
	roject:	Rumsey			Description	Portal					
### Table 1.3.3  **In a minimum of the following 2 equations	ob No	•									SHEET
The ministration of the following 2 equations   Econ Results:   Vn =   14 kip   <   Demand =   62 kip   No	SY	JC	DATE	5/12/2014	_						
The minimum of the following 2 equations   Econ Results:   Value   14 kip   Command   62 kip   No   No   Value   Val	DED 5 0 0 0										
The Vu Vu Vu Vu Vu Vu Vu Vu Vu Vu Vu Vu Vu		the following	2 equations		Ecco Results:						
18			2 0444410110			14 kip	<	Demand =	62 kip	NG	
	'n = 0.25f'c bv	dv + Vp							4.43		
C   E   S   E   S   S   S   S   S   S   S					Fy [ksi] =	40 ksi					
C [ksi] =   2,500 psi   2,5 kel					Shear Bare	# 3					
N			2.5 ksi				spacing of s	stirrun			
12 in	o [o.]	2,000 pc.	210 1101				opacing or c	ж ар			
Salada   S	v =	18 in	eff web width			0.38 in					
5.8.2.5 Minimum Transverse Reinforcement	v =	12 in	eff shear depth	1	mult by	2					
Av > 0.0316 sqrt (fc ) b, s / fy					Av	0.02 in	area of she	ar reinfocemer	nt w/in dist s		
Av > 0.0316 sqrt (fc ) b, s / fy		E O O E Mini	imum Transvar	aa Bainfaraam	.ant						
Ave		3.6.2.3 WIIIII									
Analysis:  18.3.4.2 General Procedure  es = (Mu/dv + 0.5 Nu +  vu-Vp  - Apstpo ) / (Es As + Ep Aps)  Mu = 7516 kit Nu = 14 k Vu = 62 k Vu = 62 k Vu = 62 k Vu = 0.00 in2 Aps = 0.00 in2 Aps = 0.00 in2 Aps = 0.00 in2 Aps = 1.75 ksi Double in the sisting the applied shear of the effective prestressing force positive if resisting the applied shear  Aps = 0.00 in2 Aps = 0.00 in2 Aps = 1.75 ksi Double in the sisting the applied shear of the effective prestressing force positive if resisting the applied shear  Aps = 0.00 in2 Aps = 0.00 in2 Aps = 0.00 in2 Aps = 1.75 ksi Aps = 29000 ksi Aps =						0.27 in2	= 0.0316 sc	ırt (f'c) bvs/f	/		
### Action   Action				0.022		0.272	0.00.00	,() 21 07	,		
### Action   Action	nalvsis:										
Mu = 14 k Nu = 1		ral Procedur	е								
Nu = Vu = 62 k Shear demand Vu = 62 k Shear demand Component in direction of applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear in direction of applied shear in the construction of the effective prestressing force positive if resisting the applied shear in direction of the effective prestressing force positive if resisting the applied shear in the resurressing to the same in the positive of the effective prestressing force positive if resisting the applied shear in the effective prestressing force positive if resisting the applied shear in the effective prestressing force positive if resisting the applied shear in the effective prestressing force positive in the effective prestressing force positive in the effective prestressing force positive in the effective prestressing force positive in the effective prestressing force positive in distribution of the effective prestressing force positive prestressing force positive prestressing force positive prestressing force prestressing force positive prestressing force prestressing force prestressing force prestressing force positive pr		es = (Mu/dv									
Vu = 0 k component in direction of applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of PS for positive if resisting the applied shear of PS for positive if resisting the applied shear of PS for positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of the effective prestressing force positive if resisting the applied shear of PS for PS f						ind					
Vp =											
Aps =   0.00 in2   175 ksi   0.7 fpu   Flexural steel info: Fy [ksi] =   40 ksi   Flex Bars   40 ksi   40 ksi   Flex Bars   40 ksi							-1:1 -14	4h#		£	
Aps			vp =	UK				the effective p	restressing	iorce	
$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$			Ans -	0.00 in2		surig trie applie	u SHEdi				
Es = 29000 ksi As = 8.89 in 2 flexural steel As = 1.27 in 2 flexural steel As = 1.27 in 2 flexural steel As = 1.27 in 2 flexural steel As = 1.27 in 2 flexural steel As = 8.89 in 2 flexural steel As = 8.89 in 2 flexural steel As = 8.89 in 2 flexural steel As = 1.27 in 2 flexural steel As = 1.27 in 2 flexural steel As = 1.27 in 2 flexural steel As = 1.27 in 2 flexural steel As = 1.27 in 2 flexural steel As = 1.27 in 2 flexural steel As = 1.27 in 2 flexural steel As = 1.27 in 2 flexur							Flexural ste	el info:			
Ep = 29000 ksi As = 8.89 in2 flexural steel A_s_bar = 1.27 in2 d_s_bar = 1.27 in d_s_bar = 1.27 in d_s_bar = 1.27 in mult by  Sxe = sx * 1.38 / (ag + 0.63)  Sx = min of following dv = 12 in dv = 12 in dv = 12 in dv = 12 in dx = 12											
As = 8.89 in2 flexural steel  es = 0.0027  Sxe = sx * 1.38 / (ag + 0.63)  sx = min of following dv = 12 in dv											
Size = sx * 1.38 / (ag + 0.63)  sx = min of following dv = 12 in d_ = 36 in dist between layers of long crack control reinf ag = 0.25 in max agg size assumed per photos  sx = 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 cs) 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 cs) 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 cracked concrete to transmit tension & shear  Select: alpha = 90 degrees angle of inclination of transverse reinforcemen  Vn = Vc + Vs + Vp Vc = 0.0316*beta*sqrt(fc) bv dv Vc = 15 kip Vs = Av fy sin(alpha) < 0.095 sqrt(fc) bv dv Vs = 0.66 kip Vn = 16 kip Gov  Vn = 0.25fc bv dv + Vp Vn = 135 kip  Ø = 0.90 Seimsic phi for shear					flexural steel						
Sx = sx * 1.38 / (ag + 0.63) $sx = min of following dv = 12 in dv = 36 in dist between layers of long crack control reinf ag = 0.25 in max agg size assumed per photos  Sx = 18.8 theta = 29 + 3500  es theta = 38  degrees  angle of inclination of diagonal comp. stress  If Min Transverse Reinforcement is met beta = 4.8 / (1+750  cs) beta = 1.59 factor indicating ability of diagonal cracked concrete to transmit tension & shear of the second of the secon$			es =	0.0027							
sx = min of following dv = 12 in dv = 36 in dist between layers of long crack control reinf ag = 36 in dist between layers of long crack control reinf ag = 36 in dist between layers of long crack control reinf ag = 36 in dist between layers of long crack control reinf ag = 36 in dist between layers of long crack control reinf ag = 36 in dist between layers of long crack control reinf ag = 36 in dist between layers of long crack control reinf ag = 36 in dist between layers of long crack control reinf assumed per photos  the ag = 29 + 3500 es theta = 38 degrees angle of inclination of diagonal cracked concrete to transmit tension & shear life in transverse Reinforcement is NOT met beta = 4.8 / (1+750 ss) x 51 / (39 + Sxe) beta = 1.43 factor indicating ability of diagonal cracked concrete to transmit tension & shear late = 38 degrees factor indicating ability of diagonal cracked concrete to transmit tension & shear late = 38 degrees angle of inclination of diagonal comp. stress  Select: beta = 1.4 degrees factor indicating ability of diagonal cracked concrete to transmit tension & shear late = 38 degrees angle of inclination of transverse reinforcemen  Vn = Vc + Vs + Vp											
dv = d_ = 36 in dist between layers of long crack control reinf ag = 0.25 in max agg size assumed per photos  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 cs) 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 cs) 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 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 cracked concrete to transmit tension & shear  theta = 38 degrees angle of inclination of transverse reinforcemen  Vn = Vc + Vs + Vp Vc = 0.0316*beta*sqrt(fc) bv dv Vc = 15 kip Vs = Av fy sin(alpha) < 0.095 sqrt(fc) bv dv Vs = 0.66 kip		Sxe = sx * 1			401						
d_ = 36 in dist between layers of long crack control reinf ag = 0.25 in max agg size assumed per photos  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 ss) beta = 1.59 factor indicating ability of diagonal cracked concrete to transmit tension & shear leat = 4.8 / (1+ 750 ss) x 51 / (39 + Sxe) beta = 1.43 factor indicating ability of diagonal cracked concrete to transmit tension & shear shear leat = 1.44 degrees factor indicating ability of diagonal cracked concrete to transmit tension & shear shear leat = 38 degrees angle of inclination of diagonal cracked concrete to transmit tension & shear shear leat = 38 degrees angle of inclination of diagonal cracked concrete to transmit tension & shear shear leat = 38 degrees angle of inclination of transverse reinforcemen  Vn = Vc + Vs + Vp Vc = 0.0316*beta*sqrt(fc) bv dv Vc = 15 kip Vs = Av fy sin(alpha) < 0.095 sqrt(fc) bv dv Vs = 0.66 kip			sx = r								
ag =						diat batusan I	avers of long	araak aantral i	oinf		
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 s) 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 s) x 51 / (39 + 5xe) 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 cracked concrete to transmit tension & shear  theta = 38 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			an =					CIACK COILLOI	CIIII		
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 shear = 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(fc) bv dv Vc = 15 kip Vs = Av fy sin(alpha) < 0.095 sqrt(fc) bv dv Vs = 0.66 kip < 32 kip  Vn = 16 kip Gov  Vn = 0.25fc bv dv + Vp Vn = 135 kip  φ = 0.90 Seimsic phi for shear					max agg oizo	accamoa por	priotoo				
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 shear = 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(fc) bv dv Vc = 15 kip Vs = Av fy sin(alpha) < 0.095 sqrt(fc) bv dv Vs = 0.66 kip < 32 kip  Vn = 16 kip Gov  Vn = 0.25fc bv dv + Vp Vn = 135 kip  φ = 0.90 Seimsic phi for shear		theta = 29 +	· 3500 es								
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# 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 "asbuilt" 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 — SPECIFICA-TIONS, 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  $f_y=33{,}000~{\rm psi}$ ) 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 hars
- 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	Ye	ears	Steel		le 33 ctural)		de 40 nediate)		le 50 ard)	Grac	le 60	Grad	le 75
Spec	Start	End	Туре	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,  $\ell_d$ , for the old bars would be twice (double) the  $\ell_d$  required for modern reinforcing bars. Since most strength design reviews for flexure will be based on a yield strength,  $f_{\rm V}=33{,}000~{\rm psi}$ 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,  $\ell_d$ , 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 oldstyle bars in earlier codes were considered to develop half the allowable tension stress. Under today's strength design method, this value would approximate  $\phi f_{\rm V}/2 = (0.90)(33,000~{\rm psi})/2 \approx 15,000~{\rm psi}$ .

#### **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 fourway 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,  $f_{\mathcal{C}}'$ , 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	a, in. <sup>2</sup>
312e, 111.	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

<sup>\*</sup> Recommended by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete in its 1924 Report.

- 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.

<sup>\*\*</sup> Most suppliers offered a  $\frac{1}{4}$ -inch round bar, as well as the recommended standard sizes.

 $<sup>\</sup>dagger$  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.

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



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Project:	Rumsey			Description	Column S	hear		
Job No.				-	ear Capacity		)	SHE
BY	JC	DATE	8/28/2013	_				
P <sub>c</sub>	2,300 kip		Column axial force	e				
φ	0.90		Shear Strength re	udction factor SD	C 3.2.1			
R col	84 in		Column radius					
clr	3 in		Clear cover					
bar_s	# 5		Shear confinemen	nt reinforcement				
S	9 in		Shear confinemen					
p <sub>s</sub>	0.01		(measured out-to-	• •	inforcement to the	core column c	onfined by the spiral	or hoop reinforcer
$f_{yh}$	60 ksi		Nominal yield stre	ss of transerse co	olumn reinforceme	nt		
$\mu_{\sf d}$	1.00		Local displacement	nt ductility demand	d, 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 reinforcemer	nt			
$A_b$	1.56 in2		Area of individual	reinforcing steel b	par			
$A_g$	22,167 in2		Gross section are	a of column				
$A_{e}$	17,734 in2		$A_e = 0.8 * A_g$	(SDC Eqn 3.17)				
D'	81 in		Cross sectional di	mension of confine	ed concrete core me	easured betwee	n the centerline of the	peripheral hoop or
$F1 = \rho_s * f_{yh}$	/ 0.150 + 3.67	7 - μ <sub>d</sub>	(SDC Eqn 3.20)	F2 = 1+ P <sub>c</sub> / (	2000*A <sub>a</sub> )		(SDC Eqn 3.21)	
					y,			
F1 <sub>cal</sub>	8.39	, 0.3 ≤ F	1 ≤ 3	F2 <sub>cal</sub>	1.05	, Fw ≤ 1.5		
F1 <sub>cal</sub> F1	8.39 3.00	, 0.3 ≤ F	1 ≤ 3		ū	, Fw ≤ 1.5		
F1	3.00		1 ≤ 3	F2 <sub>cal</sub> F2	1.05 1.05	, Fw ≤ 1.5		
F1 Inside Plas	3.00 stic Hinge Zon	ne		F2 cal F2 Inside Plastic	1.05 1.05 Hinge Zone		5	
F1 Inside Plas v <sub>c</sub> = F1 * F	3.00 stic Hinge Zon 2 * sqrt(f'c) ≤	ne		F2 cal F2 Inside Plastic $v_c = 3 * F2 * s$	1.05 1.05 c Hinge Zone sqrt(f'c) ≤ 4*s			
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$	3.00 stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99	ne		F2 cal F2  Inside Plastic $v_c = 3 * F2 * s$ $v_{c cal}$	1.05 1.05 : Hinge Zone sqrt(f'c) ≤ 4*s 5.99		5	
F1 Inside Plas v <sub>c</sub> = F1 * F v <sub>c cal</sub> 4*sqrt(f'c)	3.00  stic Hinge Zon  2 * sqrt(f'c) ≤  5.99  7.59	ne		F2 cal F2  Inside Plastic v <sub>c</sub> = 3 * F2 * s v <sub>c cal</sub> 4*sqrt(f'c)	1.05 1.05 c Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59		5	
F1 Inside Plas	3.00 stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99	ne		F2 cal F2  Inside Plastic $v_c = 3 * F2 * s$ $v_{c cal}$	1.05 1.05 : Hinge Zone sqrt(f'c) ≤ 4*s 5.99		5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99	ne		F2 cal F2  Inside Plastic v <sub>c</sub> = 3 * F2 * s v <sub>c cal</sub> 4*sqrt(f'c)	1.05 1.05 c Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59		5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $V_c = V_c * A$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99	ne	(SDC Eqn 3.18)	F2 cal F2  Inside Plastic v <sub>c</sub> = 3 * F2 * s v <sub>c cal</sub> 4*sqrt(f'c) v <sub>c</sub>	1.05 1.05 c Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $V_c = V_c * A$ $V_c$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip	ne	(SDC Eqn 3.18)	F2 cal F2  Inside Plastic v <sub>c</sub> = 3 * F2 * s v <sub>c cal</sub> 4*sqrt(f'c) v <sub>c</sub>	1.05 1.05 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $V_c = V_c * A$ $V_c$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip	ne	(SDC Eqn 3.18)	F2 cal F2  Inside Plastic v <sub>c</sub> = 3 * F2 * s v <sub>c cal</sub> 4*sqrt(f'c) v <sub>c</sub>	1.05 1.05 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99	qrt(f'c)	5	
F1 Inside Plas $v_c = F1 * F$ $v_{c cal}$ $4*sqrt(f'c)$ $v_c$ $V_c = v_c * A$ $V_c$ $A_v = n * (pi)$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip	ne	(SDC Eqn 3.18)	F2 cal F2  Inside Plastic v <sub>c</sub> = 3 * F2 * s v <sub>c cal</sub> 4*sqrt(f'c) v <sub>c</sub>	1.05 1.05 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $V_c = V_c * A$ $V_c$ $A_v = n * (pi$ $A_v$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip i /2)*A <sub>b</sub> 2.45 in2	ne	(SDC Eqn 3.18)	F2 cal F2  Inside Plastic V <sub>c</sub> = 3 * F2 * s V <sub>c cal</sub> 4*sqrt(f'c) V <sub>c</sub> For Inside Pla	1.05 1.05 2 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99 astic Hinge Zo	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $VC = V_c * A$ $VC$ $A_v = n * (pi$ $A_v$ $VS = A_v * f_v$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip i /2)*A <sub>b</sub> 2.45 in2	ne	(SDC Eqn 3.18)	F2 cal F2  Inside Plastic V <sub>c</sub> = 3 * F2 * s V <sub>c cal</sub> 4*sqrt(f'c) V <sub>c</sub> For Inside Pla	1.05 1.05 2 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99 astic Hinge Zo	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $VC = V_c * A$ $VC$ $A_v = n * (pi$ $A_v$ $VS = A_v * f_v$ $VS = A_v * f_v$	3.00  stic Hinge Zon 7.2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip  i /2)*A <sub>b</sub> 2.45 in2 yh * D' / s 1,318 kip	ne	(SDC Eqn 3.18)	F2 cal F2  Inside Plastic V <sub>c</sub> = 3 * F2 * s V <sub>c cal</sub> 4*sqrt(f'c) V <sub>c</sub> For Inside Pla	1.05 1.05 2 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99 astic Hinge Zo	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $V_c = V_c * A$ $V_c$ $A_v = n * (pi$ $A_v$ $V_s = A_v * f_s$ $V_s = V_c * A$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip  i /2)*A <sub>b</sub> 2.45 in2 yh * D' / s 1,318 kip  Vs	ne	(SDC Eqn 3.18)	F2 cal F2  Inside Plastic V <sub>c</sub> = 3 * F2 * s V <sub>c cal</sub> 4*sqrt(f'c) V <sub>c</sub> For Inside Pla	1.05 1.05 2 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99 astic Hinge Zo	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $V_c = V_c * A$ $V_c$ $A_v = n * (pi$ $A_v$ $V_s = A_v * f_v$ $V_s = V_c * A_v$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip  i /2)*A <sub>b</sub> 2.45 in2 yh * D' / s 1,318 kip  Vs 1,424 kip	ne	(SDC Eqn 3.18)	F2 cal F2  Inside Plastic V <sub>c</sub> = 3 * F2 * s V <sub>c cal</sub> 4*sqrt(f'c) V <sub>c</sub> For Inside Pla	1.05 1.05 2 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99 astic Hinge Zo	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $V_c = V_c * A$ $V_c$ $A_v = n * (pi$ $A_v$ $V_s = A_v * f_v$ $V_s = V_c * A_v$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip  i /2)*A <sub>b</sub> 2.45 in2 yh * D' / s 1,318 kip  Vs	ne	(SDC Eqn 3.18)	F2 cal F2  Inside Plastic V <sub>c</sub> = 3 * F2 * s V <sub>c cal</sub> 4*sqrt(f'c) V <sub>c</sub> For Inside Pla	1.05 1.05 2 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99 astic Hinge Zo	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $V_c = V_c * A$ $V_c$ $A_v = n * (pi)$ $A_v$ $V_s = A_v * f_s$ $V_s$ $V_s = V_c * A_s$ $V_s = V_s * A_s$ $V_$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip  i /2)*A <sub>b</sub> 2.45 in2 yh * D' / s 1,318 kip  Vs 1,424 kip	ne	(SDC Eqn 3.16) (SDC Eqn 3.16) (SDC Eqn 3.15)	F2 cal F2  Inside Plastic V <sub>c</sub> = 3 * F2 * s V <sub>c cal</sub> 4*sqrt(f'c) V <sub>c</sub> For Inside Pla	1.05 1.05 2 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99 astic Hinge Zo	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $V_$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip  i /2)*A <sub>b</sub> 2.45 in2 yh * D' / s 1,318 kip  Vs 1,424 kip 1,282 kip	ne	(SDC Eqn 3.16) (SDC Eqn 3.16) (SDC Eqn 3.15)	F2 cal F2  Inside Plastic V <sub>c</sub> = 3 * F2 * s V <sub>c cal</sub> 4*sqrt(f'c) V <sub>c</sub> For Inside Plastic	1.05 1.05 2 Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99 astic Hinge Zo	qrt(f'c)	5	
F1 Inside Plas $V_c = F1 * F$ $V_{c cal}$ $4*sqrt(f'c)$ $V_c$ $V_c = V_c * A$ $V_c$ $A_v = n * (pi)$ $A_v$ $V_s = A_v * f_s$ $V_s$ $V_s = V_c * A_s$ $V_s = V_s * A_s$ $V_$	3.00  stic Hinge Zon 2 * sqrt(f'c) ≤ 5.99 7.59 5.99  106 kip  i /2)*A <sub>b</sub> 2.45 in2 yh * D' / s 1,318 kip  Vs 1,424 kip	ne	(SDC Eqn 3.16) (SDC Eqn 3.16) (SDC Eqn 3.15)	F2 cal F2  Inside Plastic V <sub>c</sub> = 3 * F2 * s V <sub>c cal</sub> 4*sqrt(f'c) V <sub>c</sub> For Inside Plastic  (Mo <sup>col</sup> + Mo <sup>col</sup>	1.05 1.05 1.05 E Hinge Zone sqrt(f'c) ≤ 4*s 5.99 7.59 5.99  astic Hinge Zo circular colum	qrt(f'c)	5	

Questions concerning the VERTCON process may be mailed to  $\underline{\mbox{NGS}}$ 

Latitude: 38.89022308

Longitude: 122.2384572

NGVD 29 height:

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

1 of 1 11/6/2013 2:01 PM

#### **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.

