



STRUCTURAL CALCULATIONS

Project Name: Schuster's – New Wall Opening

Patera LLC Project Number: 21-341

Date: 7/29/2021



Prepared for: Alex Kostner

Schuster PROJECT NAME: ATERA DATE: Excellence in Engineering PROJECT NUMBER: 21-148 PHONE: 262-786-6776 FAX: 262-786-7036 SHEET NUMBER: Edge of Contrate Col Beyond Multi-wythe Masonry Existing Concrete on Metal Deck infill .(1) Anchor Through angle to Concrete Masoney Anchor Liss @ 32 O.C. W18×50 w/ 3×12" cont. PL 2-12 Stagger 1/4 Connect to Channel w/ 14×4×3 w/ 4" to Beam Q (4) 3 " Bolts to Wib 6 Shap weld and te to Channel [(1) Anchor through angle] 0 - O.F. Channel C12 x 20.7x4-9" Bolted to Existing to Align w/ O.F. 8"Typ Concrete Column w/ Concrete Col. (6) 3 4 F1554 GR. 36 Threaded Rod w/ Simpson Set 3G Adhesive w/ 6" Embedment

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PROJECT NAME: Schusters DATE: Excellence in Engineering PROJECT NUMBER: 21-149 PHONE: 262-786-6776 FAX: 262-786-7036 SHEET NUMBER: Connection Design Cal 7 18 un Exist Plans 28" Square w (8) # 1 Verdical 17.25" = 243" T. T.E DL = 16.34 =4 = To I.F Rer-F 4.8 K LL = Find Required # of Bolts Simpson Titen HD Screw Anchor (A)Assume 2 a ao psi Concrete 2 4 Edge : 6 5 pacing = 8" Vally : 2830 # 6 Embed Botts Regd 21100 # 2830 Tholf : 7.5 => 8 Bolts 2830 Tholf Regd Min 5-0 - Section of C - Channel -(8) 1/2 "a T: Han Hd - 6" Embed B) Set 36 (6) 34" \$ w/ 6" Embed @ 8" OC

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Schnsters PROJECT NAME: DATE: Excellence in Engineering PROJECT NUMBER: 21-148 PHONE: 262-786-6776 FAX: 262-786-7036 SHEET NUMBER: Beam For Masonry opening Giren 1 Floor to Floor 44. 20' 4= 21-4= New opening Hd. = 9' Full opening s Beam Loading - 2nd Floor Supported by Concrete Beam @ 2nd Floor Level - New beam to Carry Masonry + Floor infill @ old Elevator Shakt Masonty wall Load (120psf) (10ft) = 1200 plf Floor Load DL = 62.5psf) (4.5) > 282plf concrete on metal Deck LL = (100 psf) (4.5) = 450plf Steel Lintel Options A 4/500 W16 x 50 Flange Width Best Value W18×50 7.5" Most Shallow W12×79 12.1ª RXN Each End DL = 16.3 K LL: 4.8 K Conclusion: Use W18×50 w/ R 3×12 Cont.

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	Project		Job Ref.					
Excellence in Engineering 2001 S. Summy Slope Rd. • New Burlin, WI 33151	Schusters				Schusters 21-341			341
Patara	Section			Sheet no./rev.				
Patera 2601 South Sunnyslope Road	New wall opening				1			
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	jjl	7/7/2021						

Steel beam analysis & design (AISC360)
Steel beam analysis & design (AISC360-10)2
Steel column design (AISC360)
Steel column design
Column and loading details
Section classification
Slenderness
Second order effects
Shear strength7
Reduction factor for slender elements7
Compressive strength7
Flexural strength about the minor axis8
Combined forces



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2601 S. Sunny Slope Rd. • New Berlin, WI 53151	0	301	usiers			21-341	
Patera	Section	Newwoll opening			Sneet no./rev	Sheet no./rev.	
2601 South Sunnyslope Road	Opto hu			Dete	J Appld by Data		
	Calc. by	7/7/2021	Спка бу	Dale	Арр а ру	Date	
	JJI	111/2021					
				Roof live	× 1.00		
				Snow \times 1	.00		
		Span 1		Dead \times 1.	00		
		•		Live × 1.0	0		
				Roof live	× 1.00		
				Snow \times 1	.00		
		Support B		Dead \times 1.	.00		
				Live × 1.0	0		
				Roof live	× 1.00		
				Snow × 1	.00		
A							
Analysis results		M - 113 9 k	ing ft:	M 0 k	vino ft		
Maximum shear		$V_{max} = 21.1 \text{kir}$	ips_it,	V _{min} = -21	1 kins		
Deflection:		$\delta_{max} = 0.5 \text{ in}$		⊽min = - 2 1 δ _{min} = 0 in	ппрэ		
Maximum reaction at support A		R₄ max = 21.1 k	kins:	R₄ min = 2	1.1 kips		
Unfactored dead load reaction a	, at support A:	RA Dead = 16.3	kips		in apo		
Unfactored live load reaction at support A;		$R_{A \text{ Live}} = 4.8 \text{ kips}$					
Maximum reaction at support B;		R _{B_max} = 21.1 kips; R _{B_}		R _{B_min} = 2	1.1 kips		
Unfactored dead load reaction a	at support B;	R _{B_Dead} = 16.3	kips				
Unfactored live load reaction at	support B;	R _{B_Live} = 4.8 ki	ps				
Section details							
Section type;		W 16x50 (AIS	C 14th Edn (\	/14.1))			
ASTM steel designation;		A992					
Steel yield stress;		F _y = 50 ksi					
Steel tensile stress;		F _u = 65 ksi					
Modulus of elasticity;		E = 29000 ksi					
		-0.63"					
		Ť					
		- 16.3	← 0.38″				
		3.					
		▲ ★					
		₹7.0	7"				
Safety factors							
Safety factor for tensile yielding	l;	Ω _{ty} = 1.67					

	Project				Job Ref.				
Excellence in Engineering		Sch	usters		21-341				
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Palera 2601 South Suppysione Road		New wa	all opening			4			
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	•			•					
Safety factor for tensile rupture;		$Ω_{\rm tr}$ = 2.00							
Safety factor for compression;		Ω _c = 1.67							
Safety factor for flexure;		Ω _b = 1.67							
Safety factor for shear;		Ω _v = 1.50							
Lateral bracing									
		Span 1 has lat	eral bracing at s	upports only					
Classification of sections for loca	al buckling - Se	ection B4.1							
Classification of flanges in flowur	o Tablo B4 1h	(0200 10)							
Width to thicknoss ratio:		$b_{\rm c}/(2 \times t_{\rm c}) = 5$	6 1						
Limiting ratio for compact costion:		$D_{\rm f} / (2 \times t_{\rm f}) = 3.$	C/C1-045						
Limiting ratio for per section,		$\lambda_{\text{pff}} = 0.36 \times \sqrt{1}$	$[-7, r_y] = 9.15$	Commont					
Limiting ratio for non-compact section	on;	$\lambda_{\rm rff} = 1.0 \times V[E]$	/ ⊢ _y] = 24.08 ;	Compact					
Classification of web in flexure - Table B4.1b (case 15)									
Width to thickness ratio;		$(d - 2 \times k) / t_w =$	= 37.47						
Limiting ratio for compact section;	Limiting ratio for compact section;			$\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]} = 90.55$					
Limiting ratio for non-compact secti	on;	λ _{rwf} = 5.70 × √[E / F _y] = 137.27 ;	Compact					
				Sec	ction is comp	act in flexure			
Design of members for shear - C	hapter G								
Required shear strength;		V _r = max(abs()	/ _{max}), abs(V _{min}))	= 21.148 kips					
Web area;		$A_w = d \times t_w = 6$.194 in ²						
Web plate buckling coefficient;		k _v = 5							
Web shear coefficient - eq G2-2;		C _v = 1.000							
Nominal shear strength - eq G2-1;		$V_n = 0.6 \times F_y \times$	$A_w \times C_v$ = 185.8	20 kips					
Allowable shear strength;		$V_c = V_n / \Omega_v = V_n$	123.880 kips	23.880 kips					
		PASS - A	llowable shear	strength exceed	ls required sl	near strength			
Design of members for flexure in	the major axis	- Chapter F							
Required flexural strength;	-	M _r = max(abs(M _{s1 max}), abs(M _s	1 min)) = 112.825	kips ft				
Vielding - Section F2 1			- // (-					
Nominal flexural strength for vieldin	a - ea E2-1:	$M_{madel} = M_m = F_m$	~ 7. = 383 333	kine ft					
	g = cq i 2=i,	ivingia – ivip – i y	~ 2x - 000.000	hp5_h					
Lateral-torsional buckling - Secti	on F2.2		a a :						
	50 5	$L_b = L_{s1} = 256.$	08 in	6.4. ¹					
Limiting unbraced length for yielding	g - eq F2-5;	$L_p = 1.76 \times r_y >$	< ∿[E / F _y] = 67.3	94 in					
Distance between flange centroids;		$n_o = d - t_f = 15.$	67 IN						
		C = 1) (0 1 - 4 004 in						
Limiting unbroad longth for include		$f_{ts} = V[V(Iy \times C_w)]$	/) / Sx] = 1.894 In	1					
				$\mathbf{N} = \mathbf{N}^2 + \mathbf{C} = \mathbf{T} \mathbf{C}$		2)1 - 200 FZ in			
$L_r = 1.95 \times I_{ts} \times I_{ts}$	$E / (0.7 \times F_y) \times$	$V[(J \times c / (S_x \times n))]$	l₀)) + ∿((J × C / (3	$5_{X} \times (1_{0}))^{2} + 0.70 \times$	$(0.7 \times Fy/E)^{\circ}$	-)] = 206.57 in			
Moment at quarter point of accord	มาเอเอเ, +·		ns ft						
Moment at center line of sogment	ι,	Mp = 112 925	ps_n kine ft						
Moment at three quarter point of co	ament.	$M_{\rm C} = 84.610 \text{k}$	ns ft						
Maximum moment in segment.	gineni,	$M_{abs} = 112.82$	kins ft						
Maximum moment in beginent,			hipo_h						

Excellence in Engineering	Project	Job Re Schusters				b Ref. 21-341	
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Lateral torsional buckling modification factor - eq F1-1; $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C] =$ Critical flexural stress - eq F2-4; $F_{cr} = C_b \times \pi^2 \times E / (L_b / r_{ts})^2 \times \sqrt{[1 + 0.078 \times J \times c / (S_x \times h_o) \times (L_b / r_t)]}$					Mc] = 1.136 <(L _b / r _{ts}) ²] =		
Nominal flexural strength for late	ral torsional buck	29.278 ksi kling - eq F2-2; Ma = min(Mayle	$M_{nltb} = F_{cr} \times M_{nltb} = 197.$	S _x = 197.626 k . 626 kips_ft	ips_ft		
Normal nextral strength; $M_c = M_n / \Omega_b = 118.339$ kips_ftAllowable flexural strength; $PASS - Allowable flexural strength exceeds required flexural strength exceeds require$			exural strength				
Design of members for vertica	I deflection						
Consider deflection due to dead,	live, roof live and	d snow loads					
Limiting deflection; $\delta_{\text{lim}} = L_{s1} / 500 = 0.512$ in							
Maximum deflection span 1; $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 0.484$ in							

PASS - Maximum deflection does not exceed deflection limit

STEEL COLUMN DESIGN (AISC360)

STEEL COLUMN DESIGN

In accordance with AISC360-10 and the LRFD method

Tedds calculation version 1.0.09



Solution and loading details

Column details	
Column section;	C 10x15.3
Design loading	
Required axial strength;	Pr = 22 kips; (Compression)
Maximum moment about x axis;	M _x = 0.0 kips_ft
Moment about y axis at end 1;	M _{y1} = 2.0 kips_ft
Moment about y axis at end 2;	M _{y2} = 2.0 kips_ft
	Single curvature bending about y axis

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2001 South Sunnysiope Road	Calc. by	Date	Chk'd by	Date	App'd by	Date	
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Maximum moment about y axis;		M _y = max(abs(M _{y1}), abs(M _{y2})) =	= 2.0 kips_ft			
Maximum shear force parallel to y a	kis;	V _{ry} = 0.0 kips					
Maximum shear force parallel to x a	kis;	V _{rx} = 0.2 kips					
Material details							
Steel grade;		A992					
Yield strength;		F _y = 50 ksi					
Ultimate strength;		F _u = 65 ksi					
Modulus of elasticity;		E = 29000 ksi					
Shear modulus of elasticity;		G = 11200 ksi					
Unbraced lengths							
For buckling about x axis;		L _x = 84 in					
For buckling about y axis;		L _v = 84 in					
For torsional buckling;		L _z = 84 in					
Effective length factors							
For buckling about x axis:		K. = 1 00					
For buckling about v axis:		$K_{\rm x} = 1.00$					
For torsional buckling:		Ky = 1.00 K ₋ = 1.00					
i or torsional buoking,		102 - 1100					
Section classification							
Section classification for local bu	ckling (cl. B4)						
Critical flange width;		b = b _f = 2.600 i	n				
Width to thickness ratio of flange;		$\lambda_{\rm f} = b / t_{\rm f} = 5.96$	63				
Depth between root radii;		$h = d - 2 \times k = 3$	8.000 in				
Width to thickness ratio of web;		λ _w = h / t _w = 33	.333				
Compression							
Limit for nonslender flange:		$\lambda_{\rm rfo} = 0.56 \times \sqrt{10}$	(E / E ₂) = 13 487				
Elimit for honoicriaer hange,		Mi_c = 0.00 × W	(E / T y) = 10.407	The flange is no	onslender in (compression	
l imit for nonslender web:		$\lambda = 1.49 \times 10$	(E / E.) = 35 884	1		,ompression	
		/dw_c = 1.43 × 0	(∟ / Ty) = 33.00 -	The web is n	onslandar in i	romnression	
			7	The web is in The section is n	nslender in (compression	
			,	ne section is no		201110110331011	
Flexure			·- · - ·				
Limit for compact flange;		$\lambda_{pf_f} = 0.38 \times \sqrt{6}$	(E / F _y) = 9.152				
Limit for noncompact flange;		$\lambda_{rf_f} = 1.0 \times \sqrt{E}$	E / F _y) = 24.083				
				The fla	inge is compa	act in flexure	
Limit for compact web;		$\lambda_{pw_f} = 3.76 \times v_{f}$	(E / F _y) = 90.55 3	3			
Limit for noncompact web;		$\lambda_{rw_f} = 5.70 \times \sqrt{100}$	(E / F _y) = 137.27	'4			
				The	web is compa	act in flexure	
				The sec	tion is compa	act in flexure	
<u>Slenderness</u>							
Member slenderness							
Slanderness ratio about y avia:		SP K v I /	r. – 21 E				
Sienuemess fatto about x axis,		$SR_X = R_X \times L_X /$	1x = 21.0				
Sienderness ratio about y axis;		$SR_y = R_y \times L_y /$	ry = 118.1				

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Second order effects							
Second order effects for bendin	g about x axis	(cl. App 8.1)					
Coefficient C _m ;		C _{mx} = 1.0					
Coefficient α;		α = 1.0					
Elastic critical buckling stress;	$P_{e1x} = \pi^2 \times E >$	$\langle I_x / (K_{1x} \times L_x)^2 \rangle$	² = 2729.9 kips				
P-δ amplifier;	$B_{1x} = max(1.0, C_{mx} / (1 - \alpha \times P_r / P_{e1x})) = 1.008$						
Required flexural strength;		$M_{rx} = B_{1x} \times M_x = 0.0 \text{ kips_ft}$					
Second order effects for bendin	g about y axis	(cl. App 8.1)					
Coefficient C _m ;		$C_{my} = 0.6 + 0.6$	$4 imes M_{y1} / M_{y2}$ =	= ;1.000			
Coefficient α ;	α = 1.0						
Elastic critical buckling stress;	$P_{e1y} = \pi^2 \times E \times I_y / (K_{1y} \times L_y)^2 = 92.1 \text{ kips}$						
P-δ amplifier;	B _{1y} = max(1.0, C _{my} / (1 - α × P _r / P _{e1y})) = 1.314						
Required flexural strength; $M_{ry} = B_{1y} \times M_y = 2.6 \text{ kips_ft}$							
Shear strength							
Shear parallel to the major axis	(cl. G2 1)						
Shear area	(01. 02.1)	$A_{\rm W} = 2 \times b_{\rm f} \times t_{\rm f}$	e = 2 267 in ²				
Web plate buckling coefficient:		k _v = 1 2	- 2.20 7 m				
Web shear coefficient:		$C_{v} = :1.000$					
Nominal shear strength;		$V_{nx} = 0.6 \times F_v$	$\times A_w \times C_v = 68$	3.0 kips			
Design shear strength (cl G1 & (32 1(a))	,		•			
Resistance factor for shear	32. ((u))	φ _ν = 0 90					
Design shear strength:		$\psi_V = 0.00$	= 61 2 kins				
Deelgn enear erengri,		PASS - The design shear strength exceeds the required shear strength					
Deduction footon for clouder clo			j	J. J	· · · · · · · · · · · · · · · · · · ·	J. J. J. J. J.	
Reduction factor for stender ele	ments						
Reduction factor for slender ele	ments (E7)						
I he section does not contain any s	siender elemen	ts therefore:-					
Siender element reduction factor,		Q = 1.0					
Compressive strength							
Flexural buckling about x axis (cl. E3)						
Elastic critical buckling stress;		$F_{ex} = (\pi^2 \times E)$	/ (SR _x) ² = 610	. 7 ksi			
;		;					
Flexural buckling stress about x ax	kis;	$F_{crx} = Q_x \times (0.0)$	658 ^{Qx×Fy/Fex}) ×	F _y = 48.3 ksi			
Nominal flexural buckling strength;		$P_{nx} = F_{crx} \times A_g$	= 216.5 kips				
Flexural buckling about y axis (cl. E3)						
Elastic critical buckling stress;		$F_{ey} = (\pi^2 \times E)$	/ (SR _y) ² = 20.5	i ksi			
Flexural buckling stress about y a	kis;	F_{cry} = 0.877 $ imes$	F _{ey} = 18.0 ksi				
Nominal flexural buckling strength		$P_{ny} = F_{cry} \times A_g$	= 80.6 kips				

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Torsional and flexural-torsion	al buckling (cl. F	4)					
Elastic torsional buckling stress;		$F_{ez} = [\pi^2 \times E \times$	C _w / (K _z × L _z)	P^2 + G × J] / (A _g	× r₀²) = 53.2 ksi		
Torsional/flexural-torsional elasti	c buckling stress;	F _{et} = (F _{ex} + F _{ez}) / (2 × H) × [1 - √(1 - 4 × Fe	$x \times F_{ez} \times H / (F_{ex} +$	- F _{ez}) ²)]	
	•	F _{et} = 52.7 ksi	, , , <u>-</u>	,		, ,-	
;		;					
Torsional/flexural-torsional buckl	$F_{crt} = Q_z \times (0.658^{Qz \times Fy/Fet}) \times F_y = 33.6 \text{ ksi}$						
Nom. torsional/flex-torsional buc	$P_{nt} = F_{crt} \times A_g = 150.5 \text{ kips}$						
Design compressive strength	(cl.E1)						
Resistance factor for compression	on;	$\phi_c = 0.90$					
Design compressive strength;		$P_c = \phi_c \times min(F)$	Pnx, Pny, Pnt) =	; 72.5 ; kips			
	PASS - The c	lesign compress	ive strength	n exceeds the	required compre	essive stren	
Flexural strength about the mi	nor axis						
Yielding (cl. F6.1)							
Nominal flexural strength;		$M_{ny_y} = M_{py} = min(F_y \times Z_y, 1.6 \times F_y \times S_y) = 7.7 kips_ft$					
Design flexural strength about	the minor axis (cl. F1)					
Resistance factor for flexure;	$\phi_{\rm b}=0.90$						
Design flexural strength;	$M_{cy} = \phi_b \times M_{ny_yld} = $;6.9; kips_ft						
PASS	The design flex	ural strength ab	out the mind	or axis exceed	s the required fl	exural stren	
Combined forces							
Member utilization (cl. H1.1)							
Equation H1-1a;		UR = abs(P _r) /	Pc + 8 / 9 × (Mrx / Mcx + Mry	/ M _{cy}) = 0.642		
		-	PASS - The	member is ad	equate for the co	ombined for	

;

SIMPSON

Strong-I

Anchor Designer™ Software Version 2.9.7376.12

Company:	Date:	7/7/2021
Engineer:	Page:	1/5
Project:		
Address:		
Phone:		
E-mail:		

1.Project information

Customer company: Customer contact name: Customer e-mail: Comment:

2. Input Data & Anchor Parameters

General Design method:ACI 318-14 Units: Imperial units

Anchor Information:

Anchor type: Bonded anchor Material: F1554 Grade 36 Diameter (inch): 0.750 Effective Embedment depth, h_{ef} (inch): 3.500 Code report: ICC-ES ESR-4057 Anchor category: -Anchor ductility: Yes h_{min} (inch): 5.25 c_{ac} (inch): 5.25 c_{ac} (inch): 4.59 C_{min} (inch): 1.75 S_{min} (inch): 3.00

Recommended Anchor

Anchor Name: SET-3G - SET-3G w/ 3/4"Ø F1554 Gr. 36 Code Report: ICC-ES ESR-4057 $\,$



Project description: Location: Fastening description:

Base Material

Concrete: Normal-weight Concrete thickness, h (inch): 26.00 State: Uncracked Compressive strength, f_c (psi): 2500 $\Psi_{c,V}$: 1.4 Reinforcement condition: B tension, B shear Supplemental reinforcement: No Reinforcement provided at corners: No Ignore concrete breakout in tension: No Ignore concrete breakout in shear: No Hole condition: Dry concrete Inspection: Continuous Temperature range, Short/Long: 110/75°F Ignore 6do requirement: Not applicable Build-up grout pad: No

Base Plate

Length x Width x Thickness (inch): 8.00 x 48.00 x 0.25

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Strong-Tie

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Load and Geometry Load factor source: ACI 318 Section 5.3 Load combination: not set Seismic design: No Anchors subjected to sustained tension: No Apply entire shear load at front row: No Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 0 V_{uax} [lb]: 0 V_{uay} [lb]: 22000 M_{ux} [ft-lb]: 0 M_{uy} [ft-lb]: 0 Muz [ft-lb]: 2500

<Figure 1>





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Company:	Date:	7/7/2021
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Project:		-
Address:		
Phone:		
E-mail:		

<Figure 2>



Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility. Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.strongtie.com

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Software
Version 2.9.7376.12

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3. Resulting Anchor Forces

SIMPSON

Strong-Tie

Anchor	Tension load, N _{ua} (lb)	Shear load x, V _{uax} (lb)	Shear load y, V _{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2 + (V_{uay})^2}$ (lb)
1	0.0	1125.0	5500.0	5613.9
2	0.0	375.0	5500.0	5512.8
3	0.0	-375.0	5500.0	5512.8
4	0.0	-1125.0	5500.0	5613.9
Sum	0.0	0.0	22000.0	22253.3

Maximum concrete compression strain (‰): 0.00

Maximum concrete compression stress (psi): 0

Resultant tension force (lb): 0

Resultant compression force (lb): 0

Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00 Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

Eccentricity of resultant shear forces in x-axis, e'_{vx} (inch): 0.00

Eccentricity of resultant shear forces in y-axis, e'vy (inch): 0.00

<Figure 3>



8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V _{sa} (lb)	$\phi_{ ext{grout}}$	ϕ	$\phi_{grout} \phi V_{sa}$ (lb)	
11625	1.0	0.65	7556	

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min[7(I_e$	$/d_a)^{0.2}\sqrt{d_a\lambda_a}\sqrt{f'}$	c C a1 ^{1.5} ; 9λa√ f ′c C	a1 ^{1.5} (Eq. 17.5.2	.2a & Eq. 17.5.2	2.2b)			
le (in)	da (in)	λa	f′c (psi)	<i>c</i> a1 (in)	V _{bx} (lb)			
3.50	0.750	1.00	2500	6.00	6062			
$\phi V_{cbgx} = \phi \left(A_V \right)$	′c / A∨co) Ψec,∨ Ψe	$_{d,V} \Psi_{c,V} \Psi_{h,V} V_{bx}$	(Sec. 17.3.1 & E	q. 17.5.2.1b)				
A_{Vc} (in ²)	Avco (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V _{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
234.00	162.00	0.818	1.000	1.400	1.000	6062	0.70	7021
$\frac{\phi V_{cbgx} = \phi \left(A_V \right)}{A_{Vc} \left(in^2 \right)}$ 234.00	νc / Ανco) Ψec,ν Ψe Ανco (in²) 162.00	^{2d, V} Ψc, V Ψh, VVbx Ψec, V 0.818	(Sec. 17.3.1 & E <u>Ψ_{ed,V}</u> 1.000	q. 17.5.2.1b) <u>Ψ_{c,V}</u> 1.400	Ψ _{h,V} 1.000	<i>V_{bx}</i> (lb) 6062	φ 0.70	<i>φV_{cbgx}</i> (lb) 7021

Shear parallel to edge in y-direction:

$V_{bx} = \min[7($	$I_e / d_a)^{0.2} √ d_a λ_a √ f$	"c C a1 ^{1.5} ; 9λa√ f "co	Ca1 ^{1.5} (Eq. 17.5.2	.2a & Eq. 17.5.2	2.2b)			
I _e (in)	da (in)	λa	ťc (psi)	<i>c</i> a1 (in)	V _{bx} (lb)			
3.50	0.750	1.00	2500	6.00	6062			
$\phi V_{cbgy} = \phi$ (2)	2)(Avc/Avco) Vec,	$_{V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V}$	/bx (Sec. 17.3.1,	17.5.2.1(c) & Ed	q. 17.5.2.1b)			
Avc (in ²)	Avco (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	Ψc, v	$\Psi_{h,V}$	V _{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
378.00	162.00	1.000	1.000	1.400	1.000	6062	0.70	27724

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility. Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.strongtie.com

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10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

 $\phi V_{cp} = \phi \min|k_{cp} N_a; k_{cp} N_{cb}| = \phi \min|k_{cp} (A_{Na} / A_{Na0}) \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}; k_{cp} (A_{Nc} / A_{Nc0}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,Nb} N_b| (Sec. 17.3.1 \& Eq. 17.5.3.1a)$

<i>k</i> _{cp}	A_{Na} (in ²)	A _{Na0} (in ²)	$\Psi_{ed,Na}$	$arPsi_{cp,Na}$	N _{ba} (lb)	Na (lb)			
2.0	130.19	422.18	0.875	1.000	17021	4594			
A_{Nc} (in ²)	A_{Nco} (in ²)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N _b (lb)	N _{cb} (lb)	ϕ	ϕV_{cp} (lb)	
84.00	110.25	1.000	1.000	1.000	7857	5987	0.70	6431	

<u>11. Results</u>

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Shear	Factored Load, Vua (Ib)	Design Strength, øVn (lb)	Ratio	Status
Steel	5614	7556	0.74	Pass
T Concrete breakout x+	1500	7021	0.21	Pass
Concrete breakout x+	22000	27724	0.79	Pass
Pryout	5513	6431	0.86	Pass (Governs)

SET-3G w/ 3/4"Ø F1554 Gr. 36 with hef = 3.500 inch meets the selected design criteria.

12. Warnings

- This temperature range is currently outside the scope of ACI 318-14/-11 and ACI 355.4. Designer must exercise judgement to determine if this design is suitable.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.