PATERA LLC
2601 S. Sunny Slope Rd. • New Berlin, WI 53151
Office: 262.786.6776 Fax: 262.786.7036

STRUCTURAL CALCULATIONS

Project Name:
Schuster's – New Wall Opening

Patera LLC Project Number:
21-341

Date:
7/29/2021

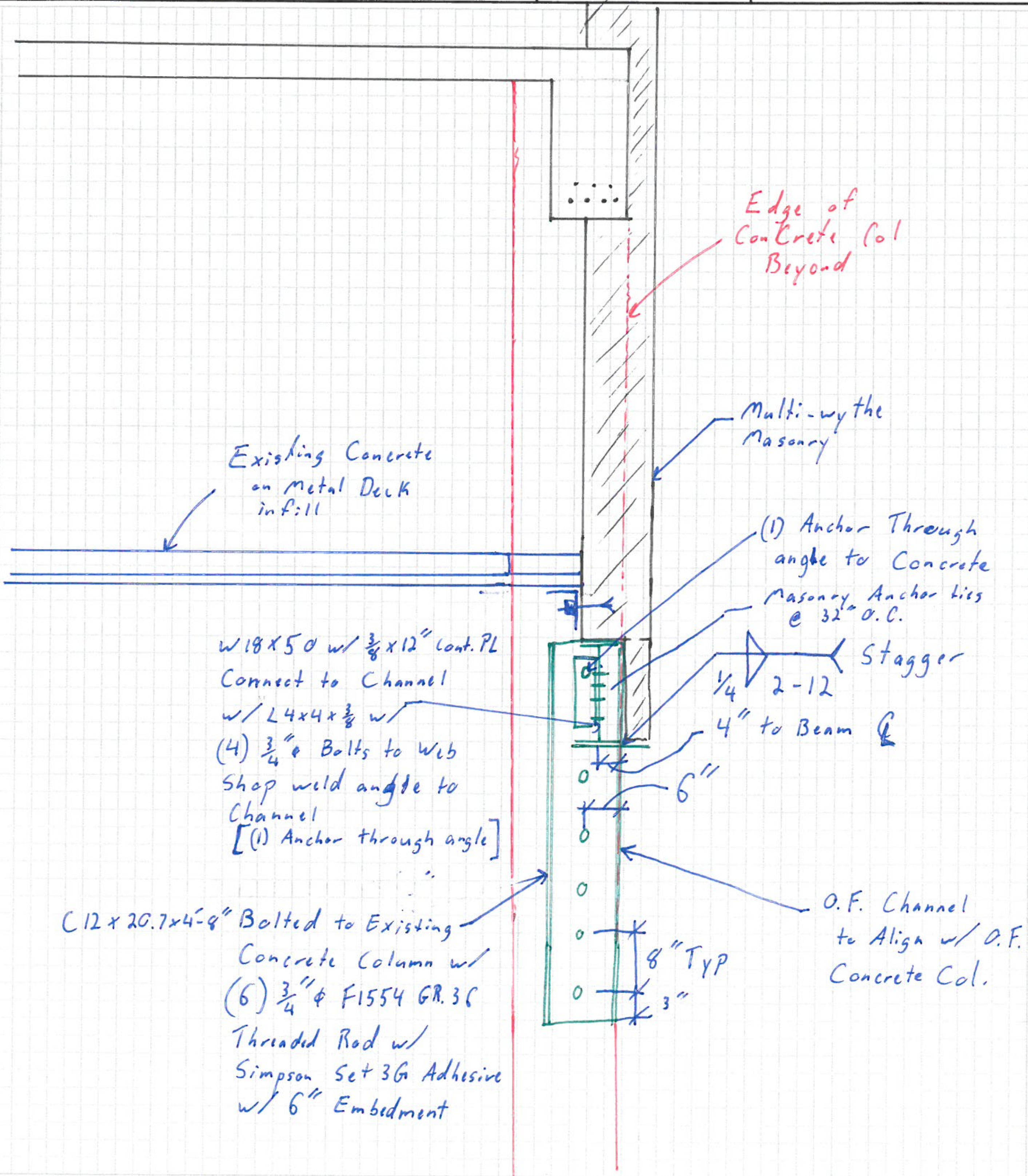
WISCONSIN
DEVON S.
ELLIS
E-34633
MILWAUKEE,
WI
PROFESSIONAL ENGINEER

8/01/2021

Prepared for:
Alex Kostner

PATERA LLC
Excellence in Engineering
PHONE: 262-786-6776 FAX: 262-786-7036

PROJECT NAME:	Schuster
DATE:	
PROJECT NUMBER:	21-148
SHEET NUMBER:	



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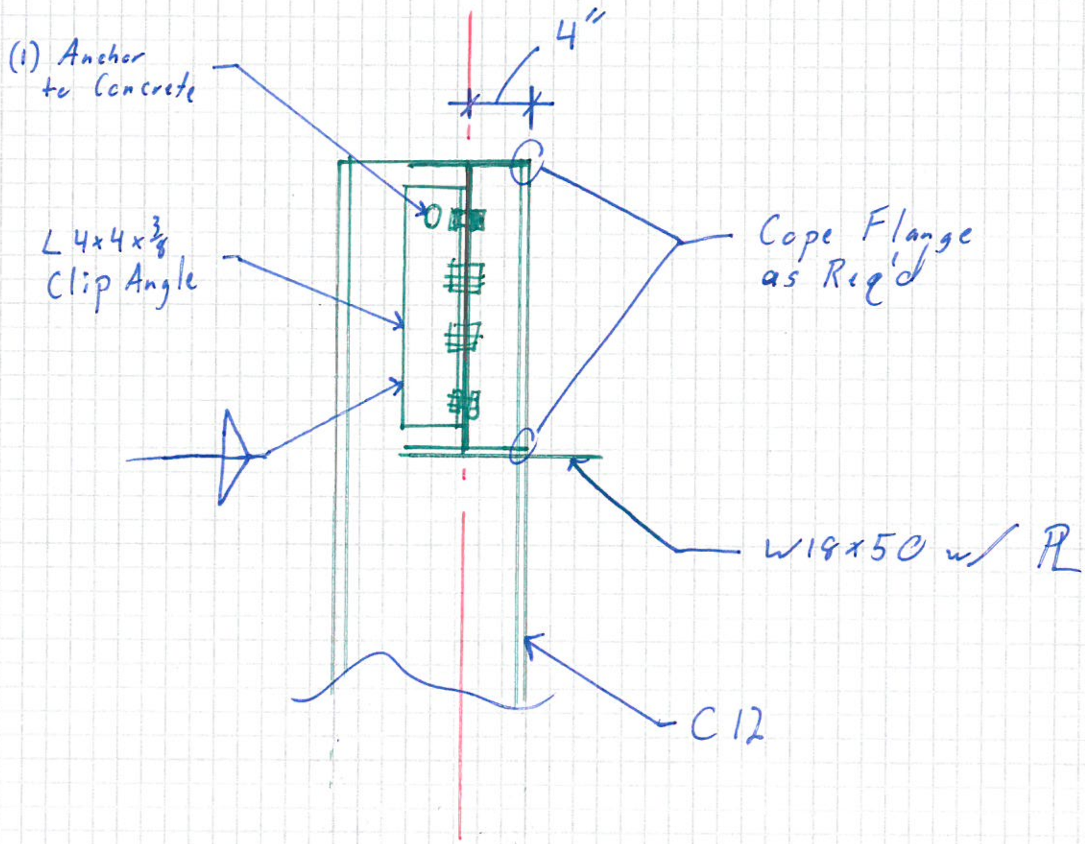

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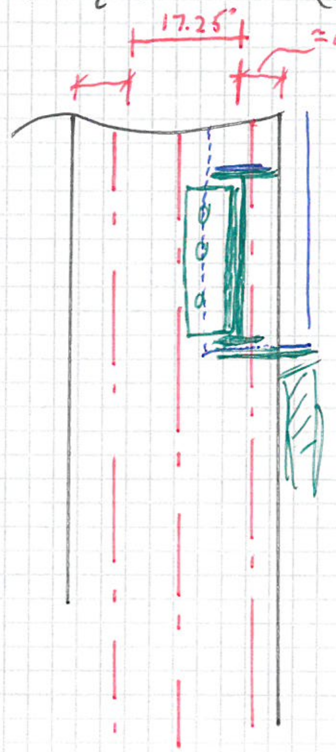
Connection Design

DL = 16.3 K

LL = 4.8 K

Col 7 + 8 on Exist Plans

28" Square w/ (8) #1 Vertical



Find Required # of Bolts

(A) Simpson Titen HD Screw Anchor

Assume 2000 psi Concrete

1/2" ϕ Edge: 6"

Spacing: 8"

$V_{all}/bolt$: 2830 #

6" Embed

Bolts Req'd

$21100\# / 2830\#/bolt$: 7.5 ⇒ 8 Bolts Req'd

Min 6'-0" Section of

C-Channel w/

(8) 1/2" ϕ Titen Hd

w/ 6" Embed

(B) Set 3G

(6) 3/4" ϕ w/ 6" Embed @ 8" OC

PATERA LLC Excellence in Engineering		PROJECT NAME:	Schusters
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Beam For Masonry opening

Given: Floor to Floor Ht. 20' L = 21'-4"
New opening Ht. ≈ 9' Full opening's

Beam Loading
- 2nd Floor Supported by Concrete Beam @ 2nd Floor Level
- New beam to Carry Masonry + Floor infill @ old Elevator Shaft

Masonry wall Load $^{DL}(120psf)(10ft) = 1200 plf$

Floor Load (concrete on metal Deck) $DL = 62.5psf)(4.5') = 282 plf$

$LL = (100\ psf)(4.5) = 450 plf$

Steel Lintel Options Δ L/500 Flange width
W16x50 7"

Best Value W18x50 7.5"

Most Shallow W12x79 12.1"

RxN Each End
DL = 16.3 K
LL = 4.8 K

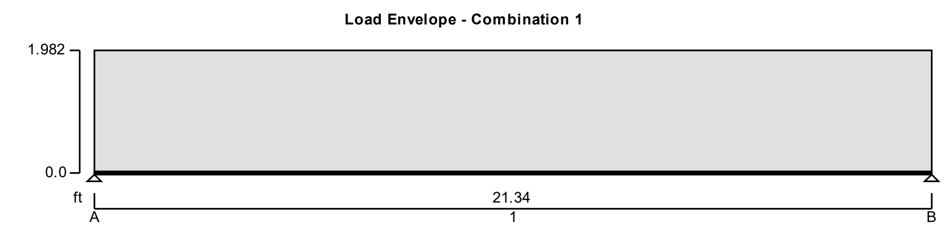
Conclusion:
Use W18x50 w/ PL 3/8 x 12 Cont.

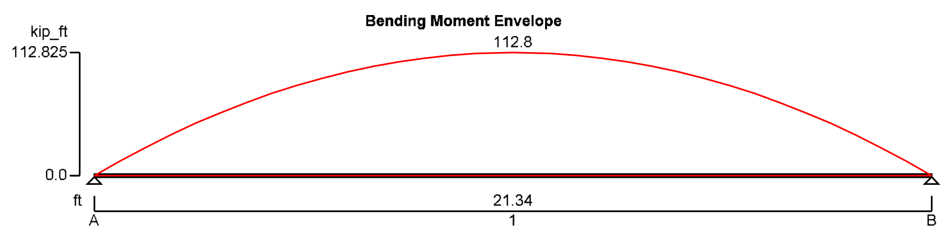
STEEL BEAM ANALYSIS & DESIGN (AISC360)

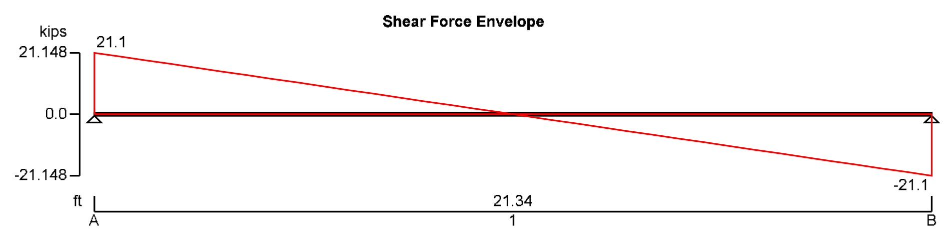
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 3.0.12







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	masonry - Dead full UDL 1.2 kips/ft
	floor infill - Dead full UDL 0.282 kips/ft
	floor infill - Live full UDL 0.45 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00

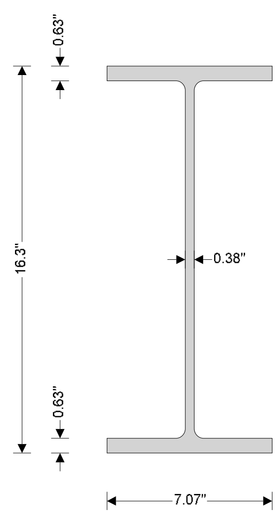
		Roof live × 1.00
		Snow × 1.00
	Span 1	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **112.8** kips_ft;	M_{min} = **0** kips_ft
Maximum shear;	V_{max} = **21.1** kips;	V_{min} = **-21.1** kips
Deflection;	δ_{max} = **0.5** in;	δ_{min} = **0** in
Maximum reaction at support A;	R_{A_max} = **21.1** kips;	R_{A_min} = **21.1** kips
Unfactored dead load reaction at support A;	R_{A_Dead} = **16.3** kips	
Unfactored live load reaction at support A;	R_{A_Live} = **4.8** kips	
Maximum reaction at support B;	R_{B_max} = **21.1** kips;	R_{B_min} = **21.1** kips
Unfactored dead load reaction at support B;	R_{B_Dead} = **16.3** kips	
Unfactored live load reaction at support B;	R_{B_Live} = **4.8** kips	

Section details

Section type;	**W 16x50 (AISC 14th Edn (v14.1))**
ASTM steel designation;	**A992**
Steel yield stress;	F_y = **50** ksi
Steel tensile stress;	F_u = **65** ksi
Modulus of elasticity;	E = **29000** ksi



Safety factors

Safety factor for tensile yielding;	Ω_{ty} = **1.67**

Safety factor for tensile rupture; Ω_{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 lateral bracing at supports only

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio; $b_f / (2 \times t_f)$ = **5.61**

Limiting ratio for compact section; $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section; $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **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; $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section; $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **137.27**; Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength; $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **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; $V_n = 0.6 \times F_y \times A_w \times C_v$ = **185.820** kips

Allowable shear strength; $V_c = V_n / \Omega_v$ = **123.880** kips

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure in the major axis - Chapter F

Required flexural strength; $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **112.825** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1; $M_{nyld} = M_p = F_y \times Z_x$ = **383.333** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length; $L_b = L_{s1}$ = **256.08** in

Limiting unbraced length for yielding - eq F2-5; $L_p = 1.76 \times r_y \times \sqrt{[E / F_y]}$ = **67.394** in

Distance between flange centroids; $h_o = d - t_f$ = **15.67** in

c = **1**

$r_{ts} = \sqrt{[\sqrt{(I_y \times C_w)} / S_x]}$ = **1.894** in

Limiting unbraced length for inelastic LTB - eq F2-6

$$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)}]} = \mathbf{206.57} \text{ in}$$

Cross-section mono-symmetry parameter; R_m = **1.000**

Moment at quarter point of segment; M_A = **84.619** kips_ft

Moment at center-line of segment; M_B = **112.825** kips_ft

Moment at three quarter point of segment; M_C = **84.619** kips_ft

Maximum moment in segment; M_{abs} = **112.825** kips_ft

Lateral torsional buckling modification factor - eq F1-1; $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C]$ = **1.136**

Critical flexural stress - eq F2-4; $F_{cr} = C_b \times \pi^2 \times E / (L_b / r_{ts})^2 \times \sqrt{[1 + 0.078 \times J \times c / (S_x \times h_o) \times (L_b / r_{ts})^2]}$ = **29.278** ksi

Nominal flexural strength for lateral torsional buckling - eq F2-2; $M_{nltb} = F_{cr} \times S_x$ = **197.626** kips_ft

Nominal flexural strength; $M_n = \min(M_{nyld}, M_{nltb})$ = **197.626** kips_ft

Allowable flexural strength; $M_c = M_n / \Omega_b$ = **118.339** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection; $\delta_{lim} = L_{s1} / 500$ = **0.512** in

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.484** in

PASS - Maximum deflection does not exceed deflection limit

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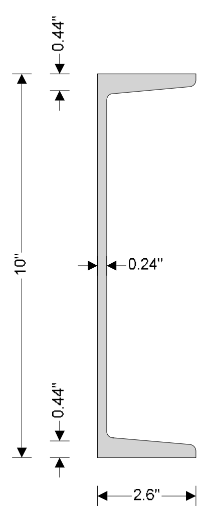
STEEL COLUMN DESIGN (AISC360)

STEEL COLUMN DESIGN

In accordance with AISC360-10 and the LRFD method

Tedds calculation version 1.0.09



Column and loading details

Column details

Column section; **C 10x15.3**

Design loading

Required axial strength; P_r = **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

Maximum moment about y axis; $M_y = \max(\text{abs}(M_{y1}), \text{abs}(M_{y2}))$ = **2.0** kips_ft
Maximum shear force parallel to y axis; V_{ry} = **0.0** kips
Maximum shear force parallel to x axis; 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_y = **84** in
For torsional buckling; L_z = **84** in

Effective length factors

For buckling about x axis; K_x = **1.00**
For buckling about y axis; K_y = **1.00**
For torsional buckling; K_z = **1.00**

<u>Section classification</u>

Section classification for local buckling (cl. B4)

Critical flange width; $b = b_f$ = **2.600** in
Width to thickness ratio of flange; $\lambda_f = b / t_f$ = **5.963**
Depth between root radii; $h = d - 2 \times k$ = **8.000** in
Width to thickness ratio of web; $\lambda_w = h / t_w$ = **33.333**

Compression

Limit for nonslender flange; $\lambda_{rf\_c} = 0.56 \times \sqrt{(E / F_y)}$ = **13.487**

The flange is nonslender in compression

Limit for nonslender web; $\lambda_{rw\_c} = 1.49 \times \sqrt{(E / F_y)}$ = **35.884**

The web is nonslender in compression

The section is nonslender in compression

Flexure

Limit for compact flange; $\lambda_{pf\_f} = 0.38 \times \sqrt{(E / F_y)}$ = **9.152**
Limit for noncompact flange; $\lambda_{rf\_f} = 1.0 \times \sqrt{(E / F_y)}$ = **24.083**

The flange is compact in flexure

Limit for compact web; $\lambda_{pw\_f} = 3.76 \times \sqrt{(E / F_y)}$ = **90.553**
Limit for noncompact web; $\lambda_{rw\_f} = 5.70 \times \sqrt{(E / F_y)}$ = **137.274**

The web is compact in flexure

The section is compact in flexure

<u>Slenderness</u>

Member slenderness

Slenderness ratio about x axis; $SR_x = K_x \times L_x / r_x$ = **21.6**
Slenderness ratio about y axis; $SR_y = K_y \times L_y / r_y$ = **118.1**

Second order effects

Second order effects for bending about x axis (cl. App 8.1)

Coefficient C_m; C_{mx} = **1.0**

Coefficient α; α = **1.0**

Elastic critical buckling stress; $P_{e1x} = \pi^2 \times E \times I_x / (K_{1x} \times L_x)^2$ = **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** kips_ft

Second order effects for bending about y axis (cl. App 8.1)

Coefficient C_m; $C_{my} = 0.6 + 0.4 \times M_{y1} / M_{y2}$ = ;**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** kips

P-δ amplifier; $B_{1y} = \max(1.0, C_{my} / (1 - \alpha \times P_r / P_{e1y}))$ = **1.314**

Required flexural strength; $M_{ry} = B_{1y} \times M_y$ = **2.6** kips_ft

Shear strength

Shear parallel to the major axis (cl. G2.1)

Shear area; $A_w = 2 \times b_f \times t_f$ = **2.267** in^2

Web plate buckling coefficient; k_v = **1.2**

Web shear coefficient; C_v = ;**1.000**

Nominal shear strength; $V_{nx} = 0.6 \times F_y \times A_w \times C_v$ = **68.0** kips

Design shear strength (cl.G1 & G2.1(a))

Resistance factor for shear; ϕ_v = **0.90**

Design shear strength; $V_{cx} = \phi_v \times V_{nx}$ = **61.2** kips

PASS - The design shear strength exceeds the required shear strength

Reduction factor for slender elements

Reduction factor for slender elements (E7)

The section does not contain any slender elements therefore:-

Slender 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 axis; $F_{crx} = Q_x \times (0.658^{Q_x \times F_y / F_{ex}}) \times F_y$ = **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** ksi

Flexural buckling stress about y axis; $F_{cry} = 0.877 \times F_{ey}$ = **18.0** ksi

Nominal flexural buckling strength; $P_{ny} = F_{cry} \times A_g$ = **80.6** kips

Torsional and flexural-torsional buckling (cl. E4)

Elastic torsional buckling stress; $F_{ez} = [\pi^2 \times E \times C_w / (K_z \times L_z)^2 + G \times J] / (A_g \times r_o^2) = \mathbf{53.2}$ ksi

Torsional/flexural-torsional elastic buckling stress; $F_{et} = (F_{ex} + F_{ez}) / (2 \times H) \times [1 - \sqrt{(1 - 4 \times F_{ex} \times F_{ez} \times H / (F_{ex} + F_{ez})^2)}]$

$F_{et} = \mathbf{52.7}$ ksi

; ;

Torsional/flexural-torsional buckling stress; $F_{crt} = Q_z \times (0.658^{Qz \times Fy/Fet}) \times F_y = \mathbf{33.6}$ ksi

Nom. torsional/flex-torsional buckling strength; $P_{nt} = F_{crt} \times A_g = \mathbf{150.5}$ kips

Design compressive strength (cl.E1)

Resistance factor for compression; $\phi_c = \mathbf{0.90}$

Design compressive strength; $P_c = \phi_c \times \min(P_{nx}, P_{ny}, P_{nt})$ = ;**72.5**; kips

PASS - The design compressive strength exceeds the required compressive strength

<u>Flexural strength about the minor axis</u>

Yielding (cl. F6.1)

Nominal flexural strength; $M_{ny\_yld} = M_{py} = \min(F_y \times Z_y, 1.6 \times F_y \times S_y) = \mathbf{7.7}$ kips_ft

Design flexural strength about the minor axis (cl. F1)

Resistance factor for flexure; $\phi_b = \mathbf{0.90}$

Design flexural strength; $M_{cy} = \phi_b \times M_{ny\_yld}$ = ;**6.9**; kips_ft

PASS - The design flexural strength about the minor axis exceeds the required flexural strength

<u>Combined forces</u>

Member utilization (cl. H1.1)

Equation H1-1a; $UR = abs(P_r) / P_c + 8 / 9 \times (M_{rx} / M_{cx} + M_{ry} / M_{cy}) = \mathbf{0.642}$

PASS - The member is adequate for the combined forces

;

SIMPSON Strong-Tie®

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:

Project description:
Location:
Fastening description:

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): 4.59
C_{min} (inch): 1.75
S_{min} (inch): 3.00

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

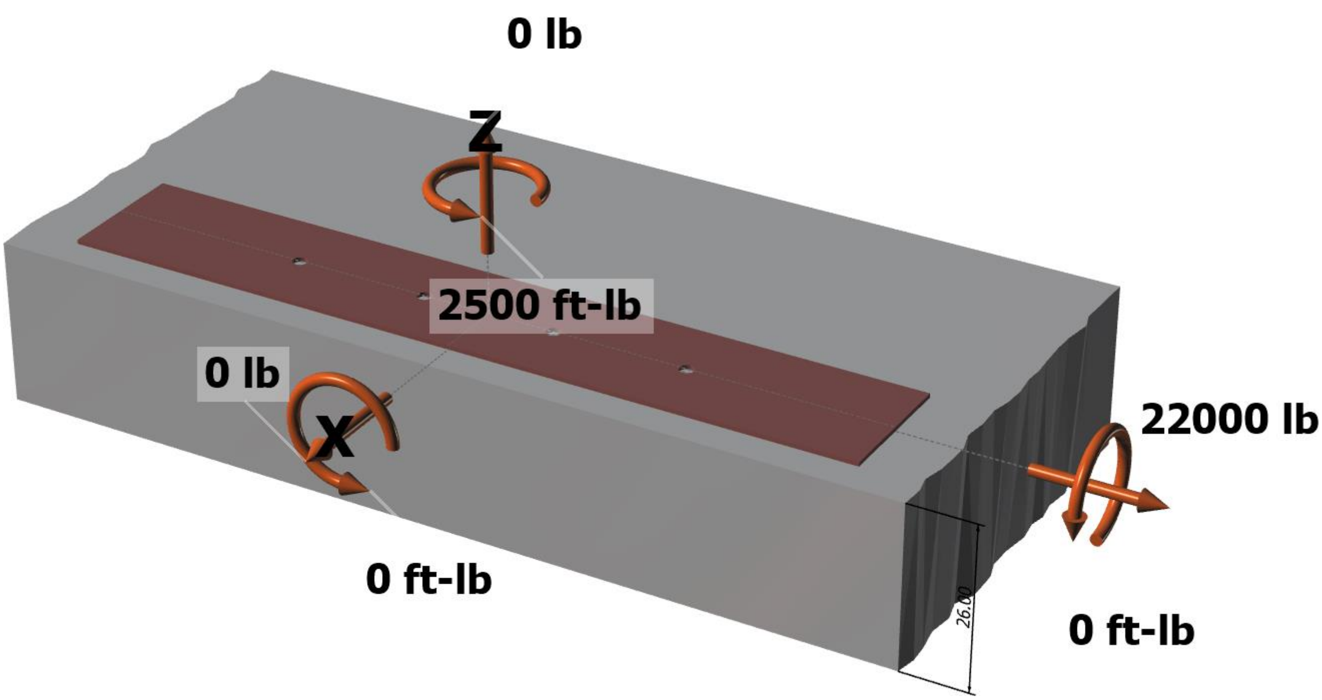
Recommended Anchor
Anchor Name: SET-3G - SET-3G w/ 3/4"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-4057

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

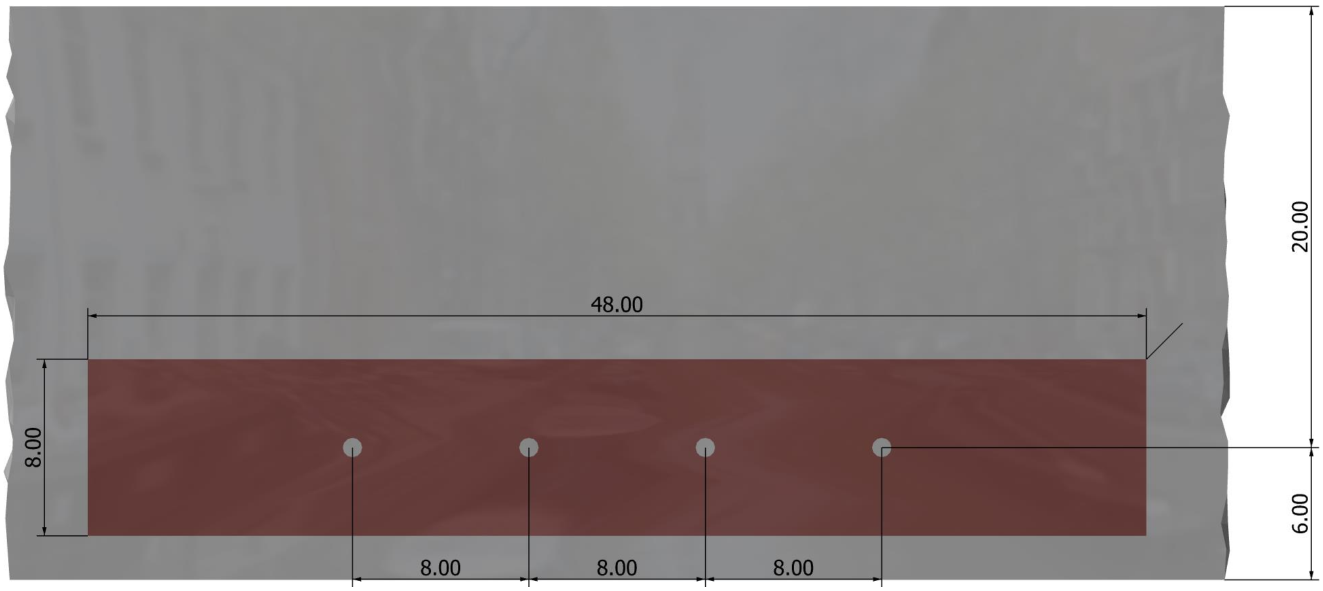
Strength level loads:

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

<Figure 1>



<Figure 2>

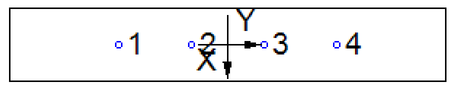


3. Resulting Anchor Forces

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)	ϕ_{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:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
3.50	0.750	1.00	2500	6.00	6062

$\phi V_{cbgx} = \phi\,(A_{Vc}/A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
234.00	162.00	0.818	1.000	1.400	1.000	6062	0.70	7021

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
3.50	0.750	1.00	2500	6.00	6062

$\phi V_{cbgy} = \phi\,(2)(A_{Vc}/A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
378.00	162.00	1.000	1.000	1.400	1.000	6062	0.70	27724

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

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

k_{cp}	A_{Na} (in^2)	A_{Na0} (in^2)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	N_a (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

11. Results

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

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (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.