November 27, 2017

Mike Molyneux
Kimball Engineering
908 W. Gordon Ave. Suite #3
Layton, UT 84041

Re: Plunkett Kuhr SFD – Plan Review Comments (First Review)
WC^3 Project #: 217-525-188

Mr. Molyneux:

We have reviewed the structural plan review comments listed above and dated October 26, 2017. See below for responses in bold italic font to your comments. The numbering of the responses corresponds to the numbering of the review comments.

Structural Drawings:

S1. Sheet S-000: Please include in the Design Criteria Notes the basic seismic force-resisting systems as required by IBC 1603.1.5.

See revised plans for changes.

S2. Sheet S-101: Footings FC2.5 and FS6.0 do not meet the minimum reinforcement requirements of Section 24.4.3 of ACI 318-14.

See revised plans for changes.

S3. Sheet S-300: The concrete pier size and reinforcing requirements have not been specified in Detail K. Please provide.

See revised plans for changes.

S4. Sheet S-301: Please address the following:

A. Detail F references the plan and schedule for the concrete column reinforcing. The concrete column has not been indicated on the plan. Please clarify.

See revised plans for changes.

B. Detail G references B/S3.1 for wall reinforcing. This detail could not be found. Please clarify.

See revised plans for changes.

S5. Sheet S-400: Details C and D both reference the shear wall schedule, but do not show the correct sheet number. Similar errors were noticed throughout the plans. Please verify that all references are correct.

See revised plans for changes.

S6. The required thicknesses of concrete walls could not be found on the plans. Please clarify.

Foundation note 3 on S-101 directs the contractor to the concrete wall schedule. The thickness of the wall is specified in the schedule. The thickness of the walls is also to scale on the structural plans.

Structural Calculations:

S7. The flat roof snow load is shown to be 192 psf. The exterior concrete deck snow load is shown to be 98 psf. Please explain how the deck snow load can be so much lower that the flat roof snow load.
98 psf was not used in design, 192 psf is the governing snow load for all applicable areas, including the exposed terrace. All beams in the initial calculation set submitted have this value inputted.

S8. Steel column SC1 is shown as HSS5x5x1/2 in the calculations while the plans show HSS4x4x1/2. Please verify.
Column schedule typo, correction made. See revised plans for changes.

S9. The concrete lintel calculations show 16 inches deep by 10 inches wide. Detail S/S-300 shows 12 inches deep and the width could not be found. Please address.
Calculation and detail revisited. See revised plans and calcs for change.

S10. The calculations show W2 Formlok Deck with 5-1/2 inches total slab depth. Detail E/S-301 shows 4 inches total slab depth. Please clarify.
4" is for the structural slab, and the additional 1.5" is for concrete topping if the architect chooses to add it. The design is covered if so. The detail only shows what is required for the deck diaphragm.

S11. Simpson anchorage calculations were done per ACI 318-11. Please verify that ACI 318-14 requirements have been met.
ACI 318-14 have been met. Updated calculation attached for reference. Please note, some holdowns have been eliminated in the locations where a steel column occurs. The typical steel column connection has been checked for max uplift and has been added to the supplemental calculation set for reference.

S12. Snow drift calculations could not be found and drift loads do not appear to be indicated on the plans. Please address.
The rooftop terrace, where drift is applicable, was designed for a live load of 252 psf. Max snow drift is 245 psf. See supplemental calculation for drift calc reference. The balcony that wraps around Part A does not include drift as its surface is covered by the eave.

S13. The proposed structure includes in-plane discontinuity in vertical lateral force-resisting element irregularities as defined by Table 12.3-2 of ASCE 7-10. Please confirm that the requisite forces were increased as required by Sections 12.3.3.3 and 12.3.3.4 of ASCE 7.
These requirements have been met where applicable and the key plans attached show locations of all irregularities considered. Calculations were checked during the initial design phase.

S14. The proposed structure includes nonparallel system irregularities as defined by Table 12.3-1 of ASCE 7-10. Please confirm that the requisite forces were increased as required and that the requirements of Sections 12.5.3 and 12.7.3 of ASCE 7 have been met.
These requirements have been met where applicable and the key plans attached show locations of all irregularities considered. Calculations were checked during the initial design phase.

S15. The lateral design was difficult to follow. Please provide a key plan indicating all lateral resisting elements (i.e. shear walls, moment frames, etc.) along with a horizontal distribution of lateral forces per Section 12.8.4 of ASCE 7-10 for the structure showing which walls/frames were considered in the design and the shear load to each wall/frame.
Key plans and horizontal distribution have been added to the supplemental calculations.

S16. Please provide anchorage calculations for the moment frame per Chapter 17 of ACI 318-14.
Calculations attached.

S17. The calculations appear to indicate that cantilever columns were used as lateral resisting elements, but detailing of these was not clear on the plans. Please clarify and verify that the correct R value was used for design.
Cantilevered columns were not used, see updated plans for verification of lateral resisting elements. The lateral resisting elements for this structure are wood shearwalls, ordinary moment frames, and ordinary concrete shearwalls.

S18. It appears that combinations of framing systems are used in the same direction. Please verify that the requirements of Section 12.2.3 of ASCE 7-10 have been met and that the most stringent applicable structural system limitations contained in Table 12.2-1 have been applied.
Most stringent design used was an R of 3.5 for the ordinary moment frames. Key plans attached will bring clarity to this requirement and what was used. Area B of the structure uses an R of 6.5 for just the shear design above the stringent design, these areas being the high roof and upper floor.

S19. Please provide calculations for the concrete columns and verify that ACI 318-14 detailing requirements have been met.
Calculation added, and detailing requirement noted on plans.

Do not hesitate to contact us with any questions, and thank you for your time reviewing this work.

Respectfully,

C. Fleming

Courtney R. Fleming, E.I.T
Project Engineer

Reviewed By:

LICENSED PROFESSIONAL STRUCTURAL ENGINEER
DANY JP TREMBLAY
STATE OF UTAH

Digitally signed by Dany JP Tremblay
DN: cn=Dany JP Tremblay, o=Canyons Structural Consulting, Inc., ou=President, email=dany@canyonsstructural.com, c=US
Date: 2017.11.27 14:24:14 -07'00'

Dany JP Tremblay, SE PE P.Eng
President | Canyons Structural, Inc

Review Question: S9

Concrete Beam

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ENERCALC, INC. 1983-2016, Build:6.16.7.21, Ver:6.16.7.21

Lic. # : KW-06009078 | **Licensee : Canyons Structural Inc**

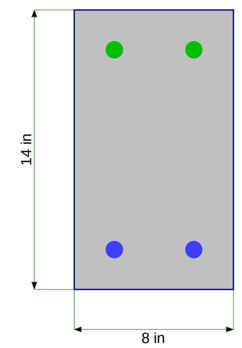
Description : Concrete Lintels around perimter floor at skylights, including lateral load

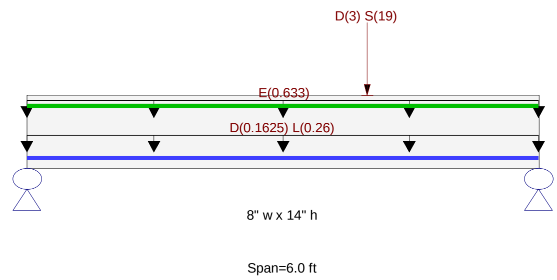
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

f'c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup =		6.0





Cross Section & Reinforcing Details

Rectangular Section, Width = 8.0 in, Height = 14.0 in
Span #1 Reinforcing....
2-#7 at 2.0 in from Bottom, from 0.0 to 6.0 ft in this span
2-#7 at 2.0 in from Top, from 0.0 to 6.0 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.0250, L = 0.040 ksf, Tributary Width = 6.50 ft, (Floor/deck loading)
Point Load : D = 3.0, S = 19.0 k @ 4.0 ft, (Roof Point Load (wehen occurs, w)
Uniform Load : E = 0.6330 k/ft, Tributary Width = 1.0 ft, (Seimic load + 25% increase for i)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.812** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	47.175	k-ft
Mn * Phi : Allowable	58.078	k-ft
Location of maximum on span	4.000	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection				
Max Downward Transient Deflection	0.038 in	Ratio =	1885	>=48
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480
Max Downward Total Deflection	0.049 in	Ratio =	1458	>=24
Max Upward Total Deflection	0.000 in	Ratio =	999	<240

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	8.159	15.493
Overall MINimum	0.780	0.780
+D+H	1.826	2.826
+D+L+H	2.606	3.606
+D+Lr+H	1.826	2.826
+D+S+H	8.159	15.493
+D+0.750Lr+0.750L+H	2.411	3.411
+D+0.750L+0.750S+H	7.161	12.911
+D+0.60W+H	1.826	2.826
+D+0.70E+H	3.155	4.155
+D+0.750Lr+0.750L+0.450W+H	2.411	3.411

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ENERCALC, INC. 1983-2016, Build:6.16.7.21, Ver:6.16.7.21

Concrete Beam

Lic. # : KW-06009078 **Licensee : Canyons Structural Inc**

Description : Concrete Lintels around perimter floor at skylights, including lateral load

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.450W+H	7.161	12.911
+D+0.750L+0.750S+0.5250E+H	8.158	13.908
+0.60D+0.60W+0.60H	1.096	1.695
+0.60D+0.70E+0.60H	2.425	3.025
D Only	1.826	2.826
Lr Only		
L Only	0.780	0.780
S Only	6.333	12.667
W Only		
E Only	1.899	1.899
H Only		

Shear Stirrup Requirements

Entire Beam Span Length : PhiVc < Vu, Req'd Vs = 1.812, use stirrups spaced at 6.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	6.000	47.17	58.08	0.81
+1.40D+1.60H					
Span # 1	1	6.000	7.14	58.08	0.12
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	6.000	7.79	58.08	0.13
+1.20D+1.60L+0.50S+1.60H					
Span # 1	1	6.000	20.45	58.08	0.35
+1.20D+1.60Lr+0.50L+1.60H					
Span # 1	1	6.000	6.64	58.08	0.11
+1.20D+1.60Lr+0.50W+1.60H					
Span # 1	1	6.000	6.12	58.08	0.11
+1.20D+0.50L+1.60S+1.60H					
Span # 1	1	6.000	47.17	58.08	0.81
+1.20D+1.60S+0.50W+1.60H					
Span # 1	1	6.000	46.65	58.08	0.80
+1.20D+0.50Lr+0.50L+W+1.60H					
Span # 1	1	6.000	6.64	58.08	0.11
+1.20D+0.50L+0.50S+W+1.60H					
Span # 1	1	6.000	19.31	58.08	0.33
+1.20D+0.50L+0.70S+E+1.60H					
Span # 1	1	6.000	26.91	58.08	0.46
+1.20D+0.50L+0.70S-E+1.60H					
Span # 1	1	6.000	21.84	58.08	0.38
+0.90D+W+0.90H					
Span # 1	1	6.000	4.59	58.08	0.08
+0.90D+E+0.90H					
Span # 1	1	6.000	7.12	58.08	0.12
+0.90D-E+0.90H					
Span # 1	1	6.000	2.06	58.08	0.04

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.0494	3.295		0.0000	0.000

Company:	Canyons Structural	Date:	11/25/2017
Engineer:	Courtney Fleming	Page:	1/4
Project:	HDU2		
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

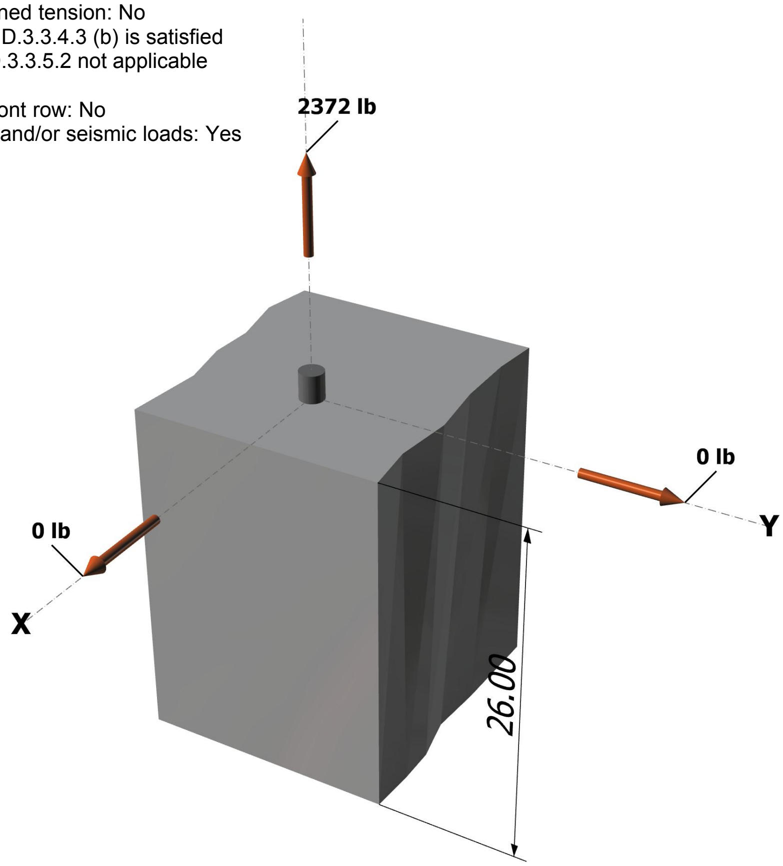
General
Design method:ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 10.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 13.13
c_{ac} (inch): 17.02
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 26.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Periodic
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: D.3.3.4.3 (b) is satisfied
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

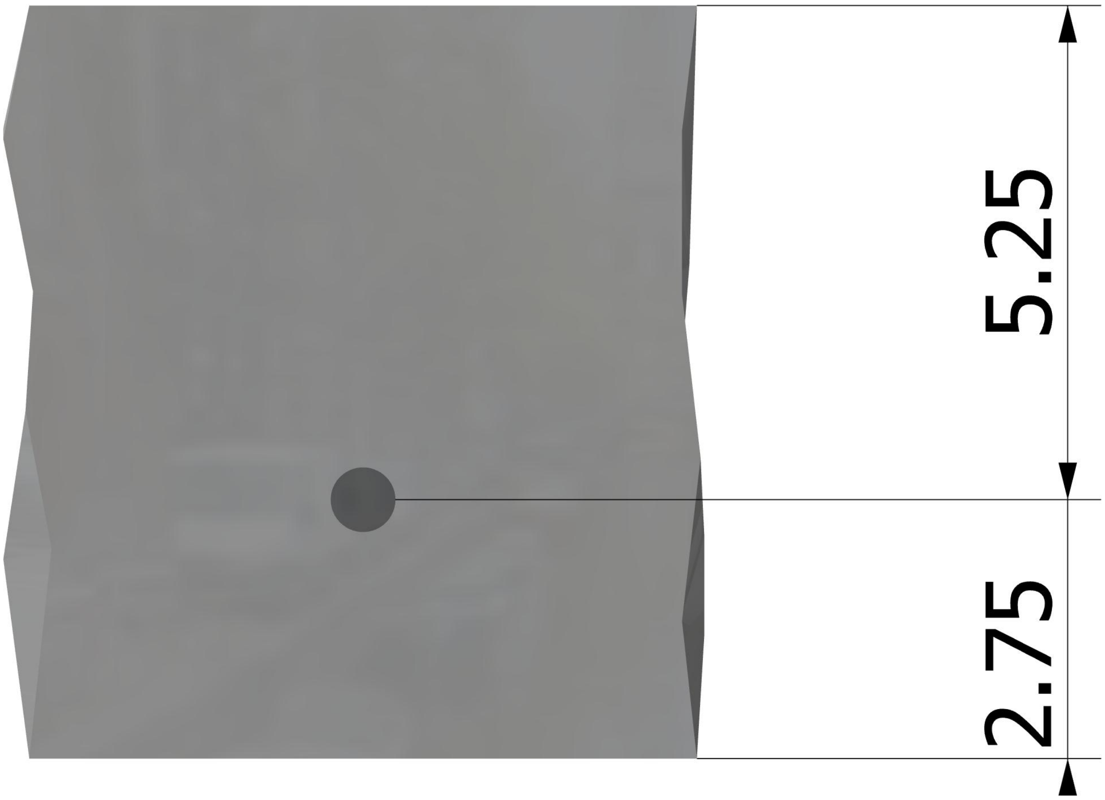
<Figure 1>



SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.5.6464.0

Company:	Canyons Structural	Date:	11/25/2017
Engineer:	Courtney Fleming	Page:	2/4
Project:	HDU2		
Address:			
Phone:			
E-mail:			

<Figure 2>



Recommended Anchor
Anchor Name: SET-XP® - SET-XP w/ 5/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

SIMPSON Strong-Tie®

Anchor Designer™ Software
Version 2.5.6464.0

Company:	Canyons Structural	Date:	11/25/2017
Engineer:	Courtney Fleming	Page:	3/4
Project:	HDU2		
Address:			
Phone:			
E-mail:			

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	2372.0	0.0	0.0	0.0
Sum	2372.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 2372
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

4. Steel Strength of Anchor in Tension (Sec. D.5.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. D-6)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	10.000	26879

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-3)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
240.00	900.00	2.75	0.755	1.00	1.000	26879	0.65	2638

6. Adhesive Strength of Anchor in Tension (Sec. 5.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} \alpha_{N.seis}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N.seis}$	$\tau_{k,cr}$ (psi)
435	1.72	1.00	1.00	748

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. D-22)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	748	0.63	10.000	14691

$0.75\phi N_a = 0.75\phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$ (Sec. D.4.1 & Eq. D-18)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75\phi N_a$ (lb)
128.74	258.98	8.05	2.75	0.803	1.000	14691	0.55	2418

Company:	Canyons Structural	Date:	11/25/2017
Engineer:	Courtney Fleming	Page:	4/4
Project:	HDU2		
Address:			
Phone:			
E-mail:			

11. Results

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

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	2372	9833	0.24	Pass
Concrete breakout	2372	2638	0.90	Pass
Adhesive	**2372**	**2418**	**0.98**	**Pass (Governs)**

SET-XP w/ 5/8"Ø F1554 Gr. 36 with hef = 10.000 inch meets the selected design criteria.

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 D.3.3.5.3 for shear need not be satisfied – designer to verify.

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

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

SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 2.5.6464.0

Company:	Canyons Structural	Date:	11/25/2017
Engineer:	CRF	Page:	1/5
Project:	Ski Lodge		
Address:			
Phone:			
E-mail:			

1.Project information

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

Project description: Steel column tensile load check (worst case, loaction B) where in place of typ. holdown
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): 12.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 15.75
c_{ac} (inch): 28.45
C_{min} (inch): 1.75
S_{min} (inch): 3.00

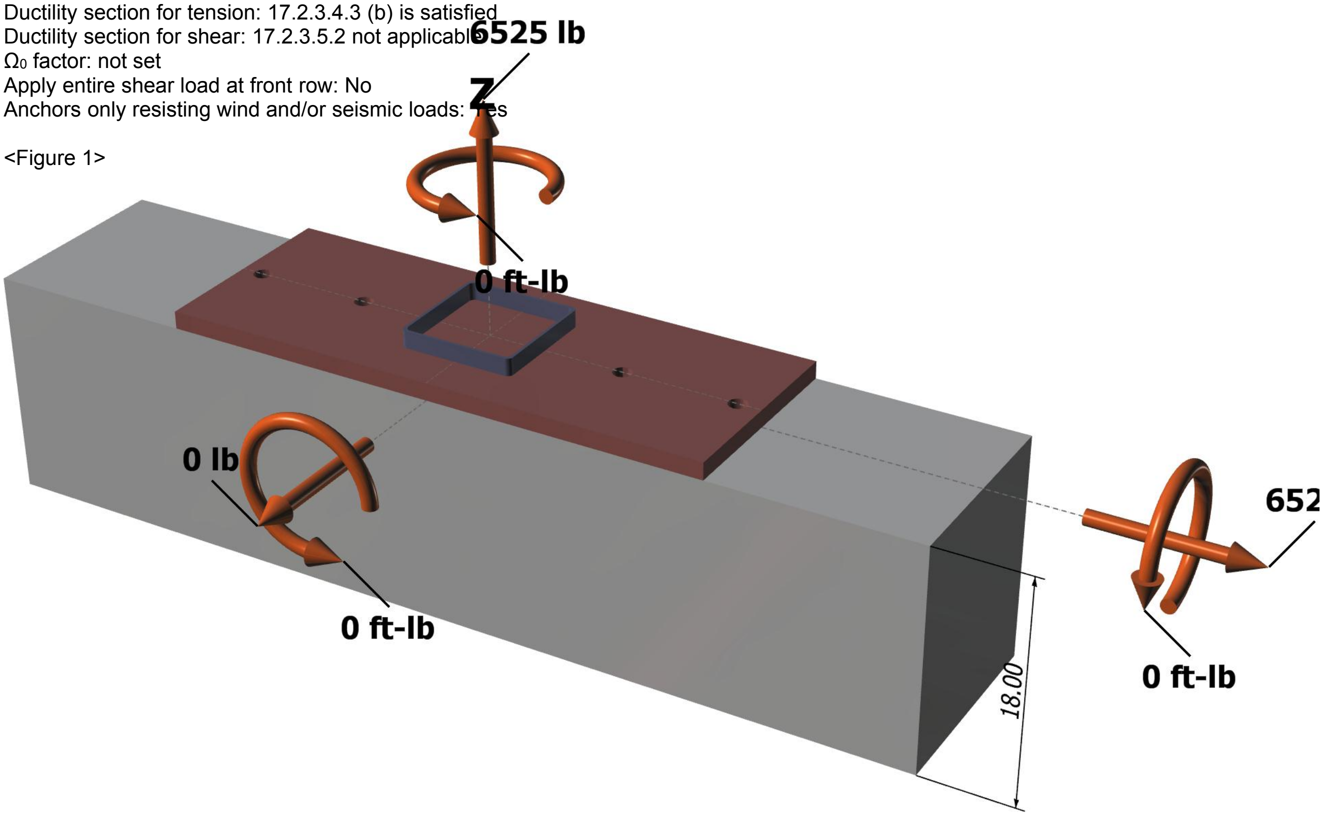
Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Hole condition: Dry concrete
Inspection: Periodic
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.2.3.4.3 (b) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Base Plate
Length x Width x Thickness (inch): 9.00 x 24.00 x 0.75
Yield stress: 34084 psi

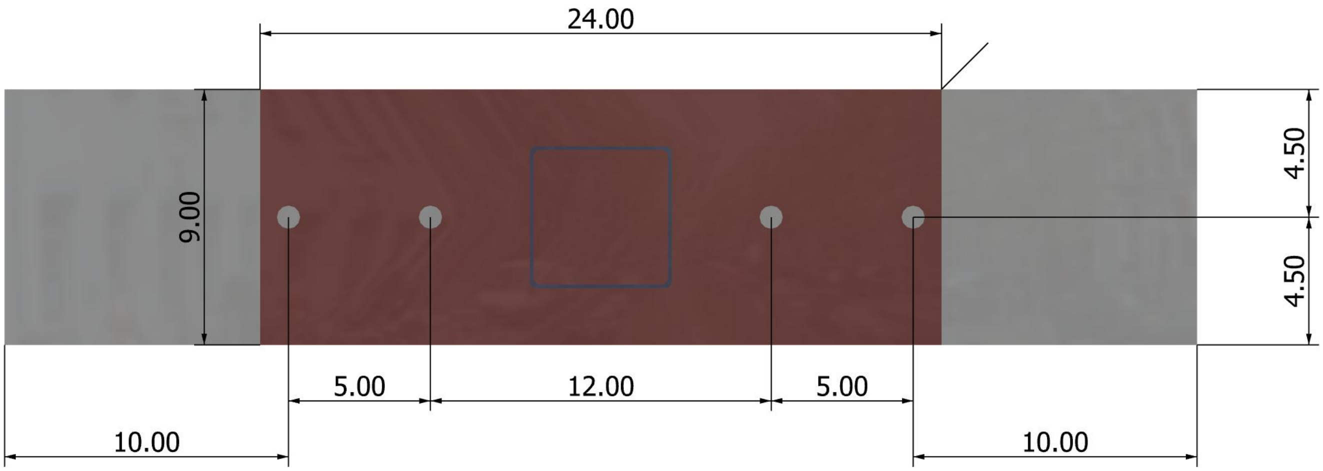
Profile type/size: HSS5X5X1/4

<Figure 1>



Company:	Canyons Structural	Date:	11/25/2017
Engineer:	CRF	Page:	2/5
Project:	Ski Lodge		
Address:			
Phone:			
E-mail:			

<Figure 2>



Recommended Anchor
Anchor Name: SET-XP® - SET-XP w/ 3/4"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

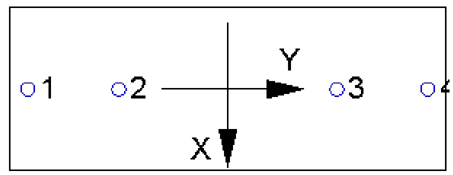
Company:	Canyons Structural	Date:	11/25/2017
Engineer:	CRF	Page:	3/5
Project:	Ski Lodge		
Address:			
Phone:			
E-mail:			

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	1631.3	0.0	163.0	163.0
2	1631.3	0.0	163.0	163.0
3	1631.3	0.0	163.0	163.0
4	1631.3	0.0	163.0	163.0
Sum	6525.0	0.0	652.0	652.0

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>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} \alpha_{N.seis}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N.seis}$	$\tau_{k,cr}$ (psi)
385	1.72	1.00	1.00	662

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	662	0.75	12.000	18723

$0.75\phi N_{ag} = 0.75\phi (A_{Na} / A_{Na0}) \Psi_{ec,Na} \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1b)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ec,Na}$	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba}(lb)	ϕ	$0.75\phi N_{ag}$ (lb)
364.26	341.26	9.24	4.50	1.000	0.846	1.000	18723	0.55	6976

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

V_{sa} (lb)	ϕ_{grout}	ϕ	$\alpha_{V,seis}$	$\phi_{grout}\alpha_{V,seis}\phi V_{sa}$ (lb)
11625	1.0	0.65	0.68	5138

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

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

k_{cp}	A_{Na} (in^2)	A_{Na0} (in^2)	$\Psi_{ed,Na}$	$\Psi_{ec,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	N_a (lb)
2.0	364.26	341.26	0.846	1.000	1.000	18723	16910

A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	N_{cb} (lb)	ϕ
378.00	400.00	1.000	0.835	1.000	1.000	14631	11545	0.70

ϕV_{cpg} (lb)
16163

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	1631	14528	0.11	Pass
Adhesive	**6525**	**6976**	**0.94**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	163	5138	0.03	Pass
Pryout	**652**	**16163**	**0.04**	**Pass (Governs)**

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	0.94	0.00	93.5 %	1.0	Pass

SET-XP w/ 3/4"Ø F1554 Gr. 36 with hef = 12.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.595 inch

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Company:	Canyons Structural	Date:	11/25/2017
Engineer:	CRF	Page:	5/5
Project:	Ski Lodge		
Address:			
Phone:			
E-mail:			

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

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

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

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Engineer:	CRF	Page:	1/5
Project:	Ski Lodge		
Address:			
Phone:			
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1.Project information

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

Project description: Steel column tensile load check (worst case) where in place of typ. holdown
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): 8.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.75
c_{ac} (inch): 13.30
C_{min} (inch): 1.75
S_{min} (inch): 3.00

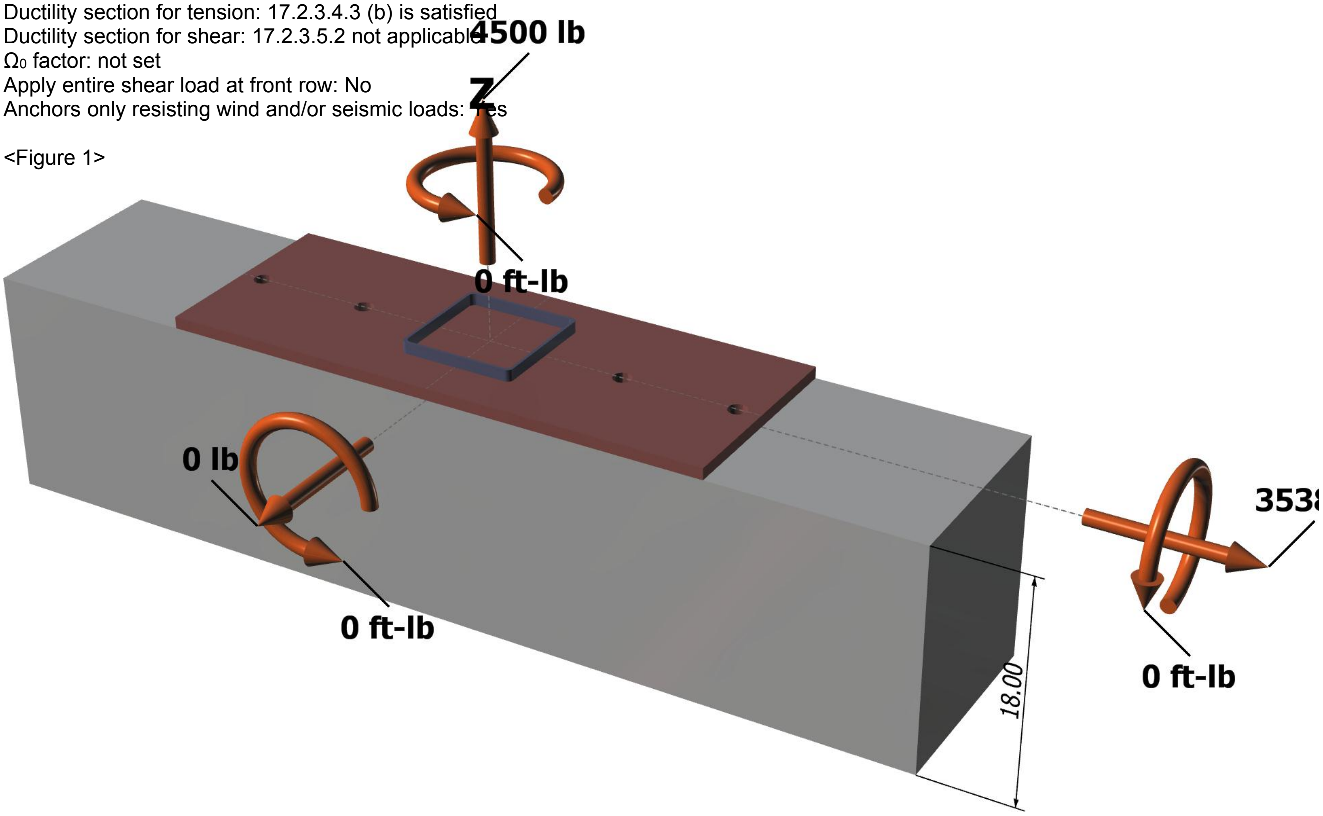
Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Hole condition: Dry concrete
Inspection: Periodic
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.2.3.4.3 (b) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Base Plate
Length x Width x Thickness (inch): 9.00 x 24.00 x 0.50
Yield stress: 34084 psi

Profile type/size: HSS5X5X1/4

<Figure 1>



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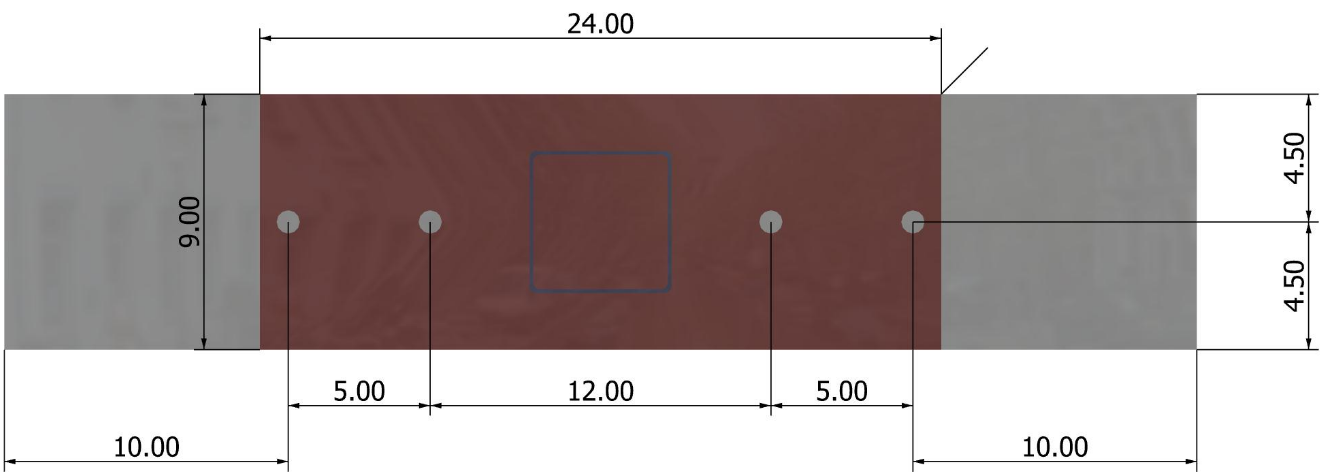
Company:	Canyons Structural	Date:	11/25/2017
Engineer:	CRF	Page:	2/5
Project:	Ski Lodge		
Address:			
Phone:			
E-mail:			

<Figure 2>



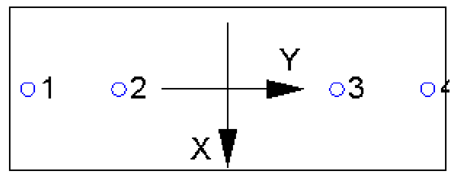
Recommended Anchor
Anchor Name: SET-XP® - SET-XP w/ 3/4"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

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	1125.0	0.0	884.5	884.5
2	1125.0	0.0	884.5	884.5
3	1125.0	0.0	884.5	884.5
4	1125.0	0.0	884.5	884.5
Sum	4500.0	0.0	3538.0	3538.0

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>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} \alpha_{N.seis}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N.seis}$	$\tau_{k,cr}$ (psi)
385	1.72	1.00	1.00	662

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	662	0.75	8.000	12482

$0.75\phi N_{ag} = 0.75\phi\,(A_{Na}/A_{Na0})\Psi_{ec,Na}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1b)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ec,Na}$	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba}(lb)	ϕ	$0.75\phi N_{ag}$ (lb)
364.26	341.26	9.24	4.50	1.000	0.846	1.000	12482	0.55	4650

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Company:	Canyons Structural	Date:	11/25/2017
Engineer:	CRF	Page:	4/5
Project:	Ski Lodge		
Address:			
Phone:			
E-mail:			

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

V_{sa} (lb)	ϕ_{grout}	ϕ	$\alpha_{V,seis}$	$\phi_{grout}\alpha_{V,seis}\phi V_{sa}$ (lb)
11625	1.0	0.65	0.68	5138

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

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

k_{cp}	A_{Na} (in^2)	A_{Na0} (in^2)	$\Psi_{ed,Na}$	$\Psi_{ec,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	N_a (lb)
2.0	364.26	341.26	0.846	1.000	1.000	12482	11274

A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	N_{cb} (lb)	ϕ
378.00	400.00	1.000	0.835	1.000	1.000	14631	11545	0.70

ϕV_{cpg} (lb)
15783

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	1125	14528	0.08	Pass
Adhesive	**4500**	**4650**	**0.97**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	885	5138	0.17	Pass
Pryout	**3538**	**15783**	**0.22**	**Pass (Governs)**

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..3	0.97	0.22	119.2 %	1.2	Pass

SET-XP w/ 3/4"Ø F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.498 inch

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Company:	Canyons Structural	Date:	11/25/2017
Engineer:	CRF	Page:	5/5
Project:	Ski Lodge		
Address:			
Phone:			
E-mail:			

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

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

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

Review Question: S12

DRIFT LOADS

Height up to which Drift is NOT a factor

Difference in height between upper and lower roof or deck - $h_r := 1.5 \cdot ft$

Ground Snow Load - $P_g := 274 \cdot psf$

Roof Snow Load - $P_f := 192 \cdot psf$

Height of balanced snow load on lower roof or deck - $h_b := \frac{P_f}{D}$ $\quad h_b = 5.5\,ft$

$$\frac{(h_r - h_b)}{h_b} = -0.727 \quad (h_{r\_consider} = 6.583\,ft) \quad \text{Drift = "DOES NOT need to be considered"}$$

Drift Area 1

Difference in height between upper and lower roof or deck - $h_r := 5 \cdot ft$

Height of balanced snow load on lower roof or deck - $h_b := \frac{P_f}{D}$ $\quad h_b = 5.5\,ft$

$$\frac{(h_r - h_b)}{h_b} = -0.089 \quad (h_{r\_consider} = 6.583\,ft) \quad \text{Drift = "DOES NOT need to be considered"}$$

Horizontal dimension of upper roof normal to the line of change in roof level,
but not less than 50 ft. or greater than 500 ft. $W_b := 50 \cdot ft$

Maximum height of drift surcharge - $h_d := \left[0.43 \cdot \left(\frac{W_b}{ft}\right)^{.33} \cdot \left(\frac{P_g}{psf} + 10\right)^{.25} - 1.5\right] \cdot ft$ $\quad h_d = 4.9\,ft$

Width of the drift load - $W_d := \min\left[4 \cdot h_d, 4 \cdot (h_r - h_b)\right]$ $\quad W_d = -1.9\,ft$

Maximum intensity of the snow load at the highest point of drift -

$$P_m := \min\left[D \cdot (h_d + h_b), D \cdot h_r\right] \quad P_m = 175\,psf$$

Drift Area 2

Difference in height between upper and lower roof or deck - $h_r := 6 \cdot ft$

Height of balancedsnow load on lower roof or deck - $h_b := \frac{P_f}{D}$ $\quad h_b = 5.5\,ft$

$$\frac{(h_r - h_b)}{h_b} = 0.094 \quad (h_{r\_consider} = 6.583\,ft) \quad \text{Drift = "DOES NOT need to be considered"}$$

Maximum height of drift surcharge - $h_d := \left[0.43 \cdot \left(\frac{W_b}{ft}\right)^{.33} \cdot \left(\frac{P_g}{psf} + 10\right)^{.25} - 1.5\right] \cdot ft$ $\quad h_d = 4.9\,ft$

Width of the drift load - $W_d := \min\left[4 \cdot h_d, 4 \cdot (h_r - h_b)\right]$ $\quad W_d = 2.1\,ft$

Maximum intensity of the snow load at the highest point of drift -

$$P_m := \min\left[D \cdot (h_d + h_b), D \cdot h_r\right] \quad P_m = 210\,psf$$

Maximum Drift

Difference in height between upper and lower roof or deck - $h_r := 7 \cdot ft$

Height of balancedsnow load on lower roof or deck - $h_b := \frac{P_f}{D}$ $h_b = 5.5\,ft$

$\frac{(h_r - h_b)}{h_b} = 0.276$ ($h_{r\_consider} = 6.583\,ft$ Drift = "MUST be considered"

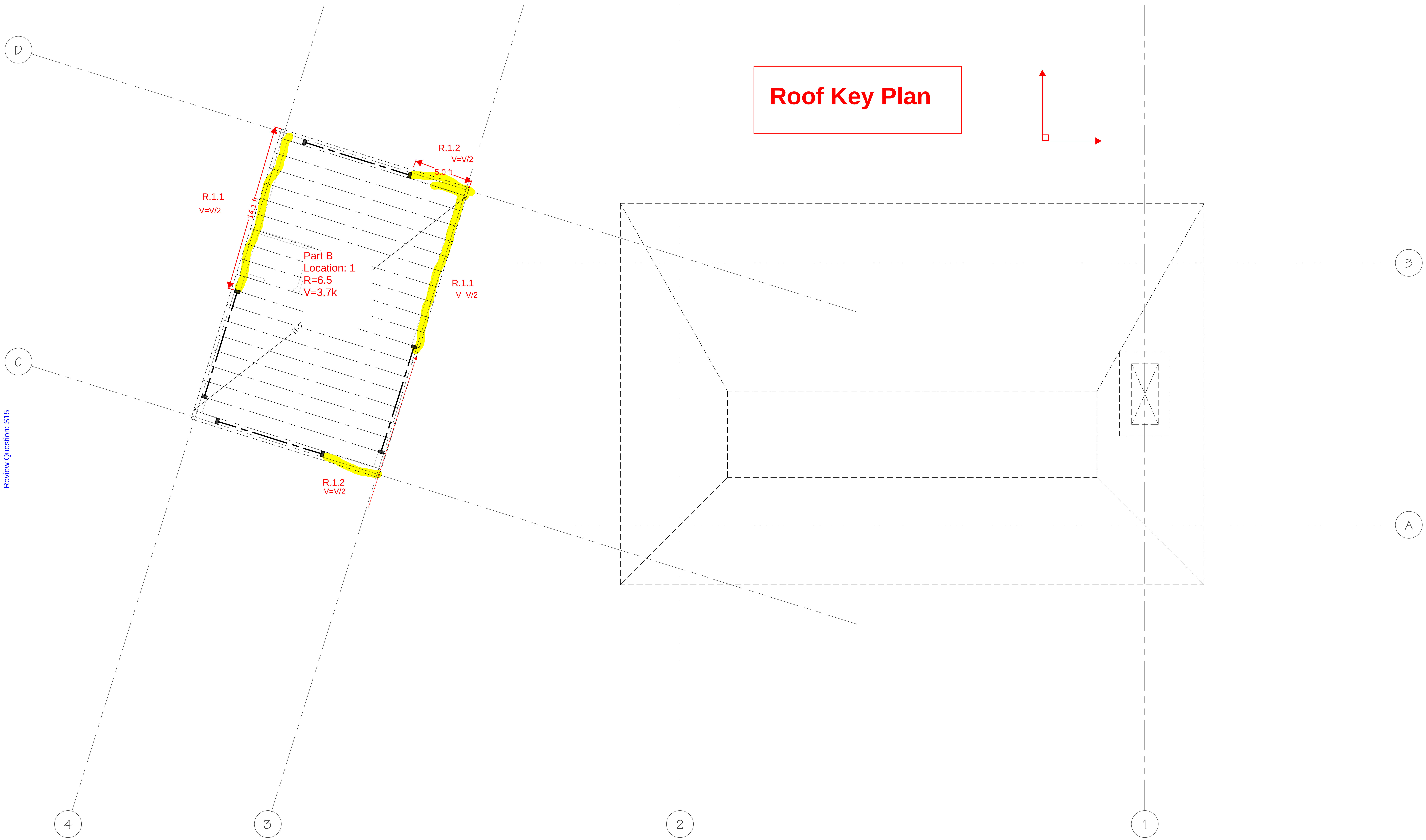
Maximum height of drift surcharge - $h_d := \left[0.43 \cdot \left(\frac{W_b}{ft}\right)^{.33} \cdot \left(\frac{P_g}{psf} + 10\right)^{.25} - 1.5\right] \cdot ft$ $h_d = 4.9\,ft$

Width of the drift load - $W_d := \min\left[4 \cdot h_d, 4 \cdot (h_r - h_b)\right]$ $W_d = 6.1\,ft$

Maximum intensity of the snow load at the highest point of drift -

$P_m := \min\left[D \cdot (h_d + h_b), D \cdot h_r\right]$ $P_m = 245\,psf$

Review Question: S15



1 S-2.21 ROOF FRAMING PLAN (PART B)
SCALE: 1/4" = 1'-0"

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PROJECT NAME
ADDRESS LINE 1
ADDRESS LINE 2
ADDRESS LINE 3

SHEET TITLE
ROOF FRAMING PLAN (PART B)

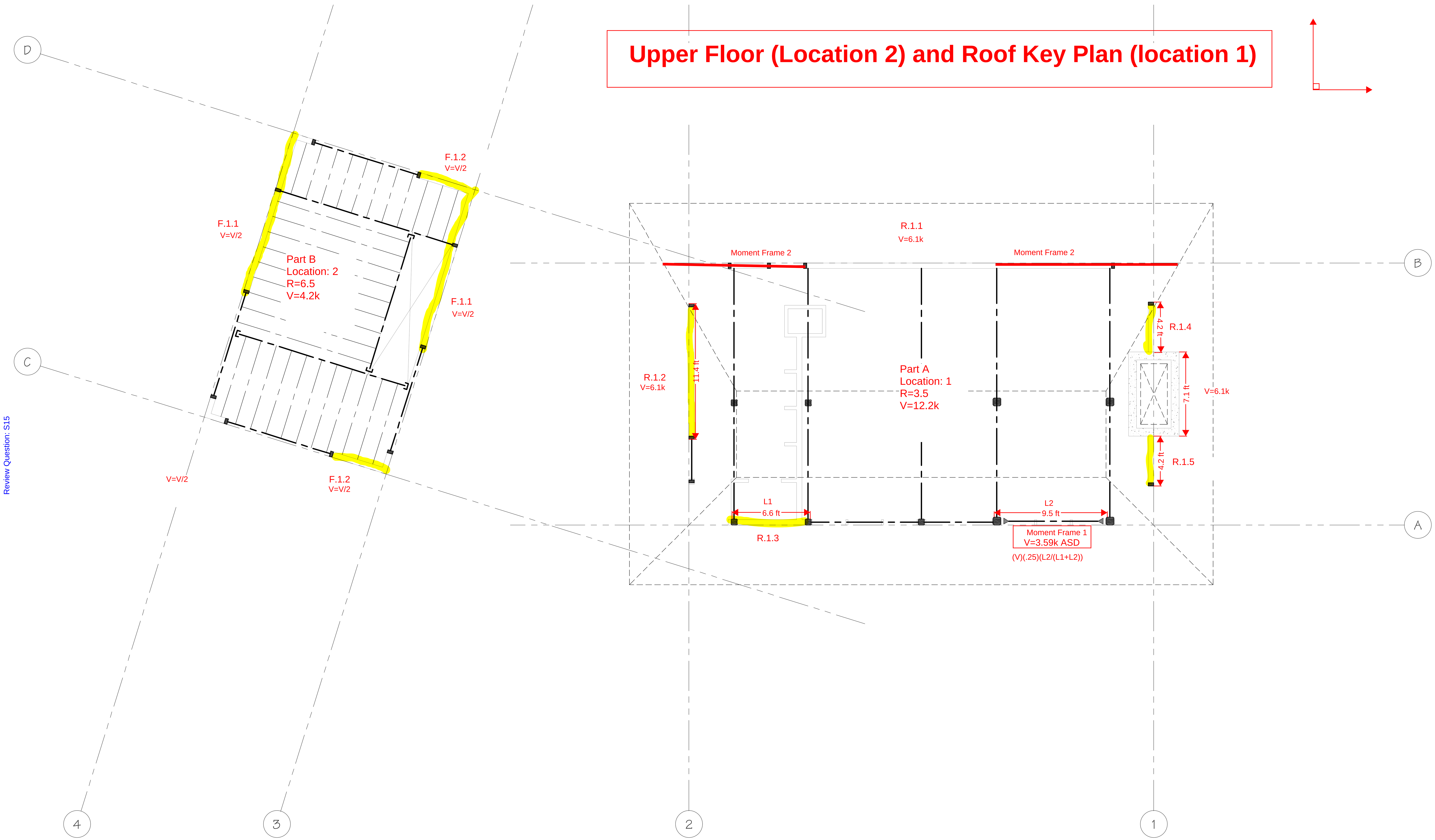
ISSUE DATE

SHEET REVISION DATE

SHEET NUMBER
S-2.21

Review Question: S15

Upper Floor (Location 2) and Roof Key Plan (location 1)



3RD FLOOR FRAMING (PART B) & ROOF FRAMING PLAN (PART A)

1 S-2.11 SCALE: 1/4" = 1'-0"

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SHEET TITLE
3RD FLOOR FRAMING (PART B) & ROOF FRAMING PLAN (PART A)

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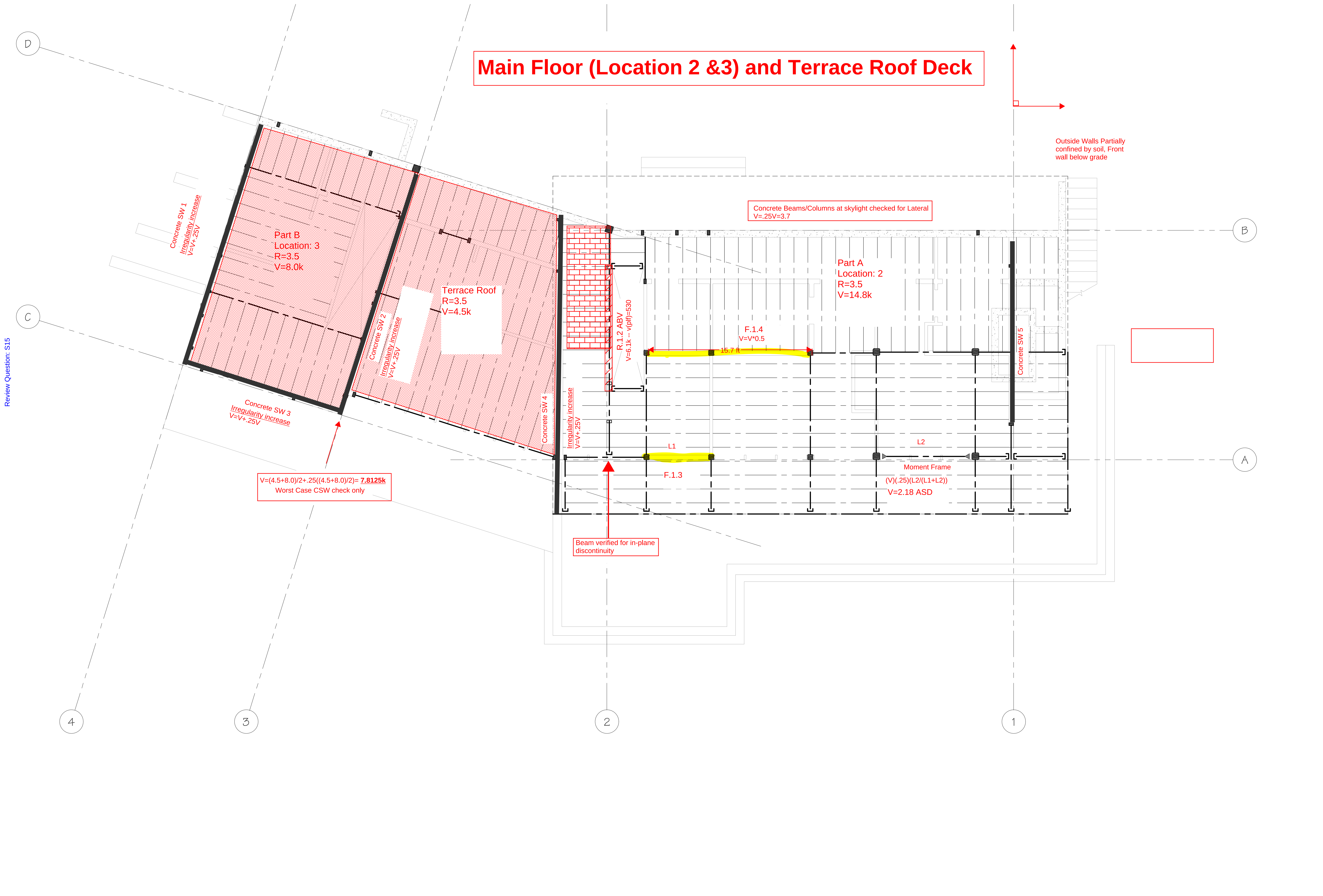
SHEET NUMBER
S-2.11

Review Question: S15



1 S-2.01 2ND FLOOR FRAMING PLAN
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SHEET NUMBER
S-2.01

Browning Ski Lodge (Part B)

Horizontal Seismic Force Distribution

by Courtney R. Fleming

0.085 *W (Shearwall ASD)

11/2017

Location	Area	DL	Seismic SL	Seismic Wt., W_s	Level Force, V_L	# of walls	Wall Length, L	Wall Height, H	Wall DL	Wall Wt., W_w	Wall Force, Vw	Total Force, Vs
1	400 ft^2	15 psf	37 psf	20.8 kips	1.8 kips	2	38.0 ft	9.0 ft	8 psf	2.7 kips	0.2 kips	2.0 kips
2	400 ft^2	10 psf	0 psf	4.0 kips	0.3 kips	2	38.0 ft	9.0 ft	8 psf	2.7 kips	0.2 kips	0.6 kips
3	400 ft^2	78 psf	0 psf	31.2 kips	2.7 kips	2	40.0 ft	9.0 ft	95 psf	34.2 kips	5.4 kips	8.0 kips

V= **0.157** For location 3 Wall Forces Which Occur below Grade

Browning Ski Lodge (Terrace Deck)

Horizontal Seismic Force Distribution

by Courtney R. Fleming

0.157 *W (Shearwall ASD)

Location	Area	DL	Seismic SL	Seismic Wt., W_s	Level Force, V_L	# of walls	Wall Length, L	Wall Height, H	Wall DL	Wall Wt., W_w	Wall Force, Vw	Total Force, Vs
1	375 ft^2	78 psf	37 psf	43.1 kips	3.7 kips	2	36.8 ft	9.0 ft	95 psf	31.5 kips	4.9 kips	8.6 kips

Project:	Browning Ski Lodge (Part B)
Engineer:	Courtney R. Fleming, Project Engineer
Date:	11/23/2017

Equivalent Lateral Force Procedure per latest version of ASCE 7

Seismic Forces Equivalent Lateral Force Procedure

V =	**0.085W**	Base Shear ASCE 7-10 Equation 12.8-1 pg. 89
Cs=	0.085	Seismic Response Coefficient (input from Code Search Spreadsheet: 'EQ'!F61)
T=	0.230	Building Period (input from Code Search Spreadsheet: 'EQ'!K56)

Total Seismic loads:	**Diaphragm**	**Wall**
	64 kips	11 kips
Total Building wt. =	**75 kips**	

Total Base Shear, V:

V, Seismic:	6 kips	**Seismic Controls**

Seismic Controls for all wall designs

Vertical Distribution of Forces: **1**

k= 1.0 ASCE 7-10 Equation 12.8-12, pg. 91

Location		wi	hi	wi*hi^k	wi*hi^k/Σwi*hi^k	Cs	Fx	Vx (kips)	ASD REDUCTION
1	---	30 kips	30.0 ft.	893	0.6	0.085	5.1 kips	**5.1 kips**	3.7 kips
2	---	7 kips	20.0 ft.	134	0.1	0.085	0.8 kips	**5.9 kips**	4.2 kips
3	---	50 kips	10.0 ft.	497	0.3	0.157	5.3 kips	**11.2 kips**	8.0 kips
Slab	---	17 kips	0.0 ft.	0	0.0	0.157	0.0 kips	**0.0 kips**	0.0 kips
	---		0.0 ft.	0	0.0	0.085	0.0 kips	**0.0 kips**	0.0 kips
	---		0.0 ft.	0	0.0	0.085	0.0 kips	**0.0 kips**	0.0 kips
Σ	---	**103.2 kips**	--	**1523**			**11.2 kips**		

Vertical Distribution of Forces: **Terrace**

k= 1.0 ASCE 7-10 Equation 12.8-12, pg. 91

Location		wi	hi	wi*hi^k	wi*hi^k/Σwi*hi^k	Cs	Fx	Vx (kips)	ASD REDUCTION
1	---	59 kips	10.0 ft.	588	0.4	0.157	6.3 kips	**6.3 kips**	4.5 kips
2	---	16 kips	0.0 ft.	0	0.0	0.085	0.0 kips	**0.0 kips**	0.0 kips
3	---	0 kips	0.0 ft.	0	0.0	0.157	0.0 kips	**0.0 kips**	0.0 kips
Slab	---	0 kips	0.0 ft.	0	0.0	0.157	0.0 kips	**0.0 kips**	0.0 kips
	---		0.0 ft.	0	0.0	0.085	0.0 kips	**0.0 kips**	0.0 kips
	---		0.0 ft.	0	0.0	0.085	0.0 kips	**0.0 kips**	0.0 kips
Σ	---	**74.5 kips**	--	**588**			**6.3 kips**		

Review Question: S15

CRF													11/23/2017
				Elliot Group - Browning Ski Lodge (Part B)									
				SUMMARY OF LATERAL FORCES (SEGMENTED DESIGN)									
Level	Line No.	Wall No.	Mark	Force V(k)	Wind/Seismic	Length (ft)	v (plf)	Height (ft)	Reduction	SW Type	Uplift	Uplift LEFT	Uplift RIGHT	Holddowns
R	1	2	R.1.2			5	370	9	1.00	C	3330	3006	3006	CS14
		Totals		1.9	Seismic	5								
					wood/wood									
F	1	2	F.1.2			5	420	9	1.00	C	3780	6525	6525	HDU8
		Totals		2.1	Seismic	5								
					wood/concrete									

CRF													11/23/2017
				Elliot Group - Browning Ski Lodge (Part B)									
				SUMMARY OF LATERAL FORCES (SEGMENTED DESIGN)									
Level	Line No.	Wall No.	Mark	Force V(k)	Wind/Seismic	Length (ft)	v (plf)	Height (ft)	Reduction	SW Type	Uplift	Uplift LEFT	Uplift RIGHT	Holddowns
R	1	1	R.1.1			14	132	9	1.00	A	1189	n/a	n/a	-
		Totals		1.9	Seismic	14								
					wood/wood									
F	1	1	F.1.1			14	150	9	1.00	A	1350	n/a	n/a	-
		Totals		2.1	Seismic	14								
					wood/concrete									

<u>Holdown Note:</u>
Many locations where holdowns should occur, steel columns are in their place for the gravity design of this house. Anchorage in these instances have checked for the worst possible uplift.

Browning Ski Lodge (Part A)

Horizontal Seismic Force Distribution

by Courtney R. Fleming

V= **0.157 *W (Shearwall ASD)**

Location	Area	DL	Seismic SL	Seismic Wt., W_s	Level Force, V_L	# of walls	Eave length	Wall Length, L	Wall Height, H	Wall DL	Wall Wt., W_w	Wall Force, Vw	Total Force, Vs
1	1600 ft^2	15 psf	37 psf	83.2 kips	13.1 kips	2	5.00 ft	70.0 ft	10.0 ft	8 psf	5.6 kips	0.9 kips	13.9 kips
2	1600 ft^2	25 psf	0 psf	40.0 kips	6.3 kips	1	0.00 ft	44.0 ft	10.0 ft	8 psf	3.5 kips	0.6 kips	6.8 kips

Project:	Browning Ski Lodge (Part A)
Engineer:	Courtney R. Fleming, Project Engineer
Date:	11/23/2017

Equivalent Lateral Force Procedure per latest version of ASCE 7

Seismic Forces Equivalent Lateral Force Procedure

V =	**0.157W**	Base Shear ASCE 7-10 Equation 12.8-1 pg. 89
Cs=	0.157	Seismic Response Coefficient (input from Code Search Spreadsheet: 'EQ'!F61)
T=	0.230	Building Period (input from Code Search Spreadsheet: 'EQ'!K56)

Total Seismic loads:	**Diaphragm**	**Wall**
	123 kips	9 kips
Total Building wt. =	**132 kips**	

Total Base Shear, V:

V, Seismic:	21 kips	**Seismic Controls**

Seismic Controls for all wall designs

Vertical Distribution of Forces: 1

k= 1.0 ASCE 7-10 Equation 12.8-12, pg. 91

Location		wi	hi	wi*hi^k	wi*hi^k/Σwi*hi^k	Cs	Fx	Vx (kips)	ASD REDUCTION
1	---	86 kips	24.0 ft.	2064	0.8	0.157	17.1 kips	**17.1 kips**	12.2 kips
2	---	45 kips	10.0 ft.	446	0.2	0.157	3.7 kips	**20.8 kips**	14.8 kips
Grade	---	2 kips	0.0 ft.	0	0.0	0.157	0.0 kips	**0.0 kips**	0.0 kips
	---	0 kips	0.0 ft.	0	0.0	0.157	0.0 kips	**0.0 kips**	0.0 kips
	---		0.0 ft.	0	0.0	0.157	0.0 kips	**0.0 kips**	0.0 kips
	---		0.0 ft.	0	0.0	0.157	0.0 kips	**0.0 kips**	0.0 kips
Σ	---	**132.3 kips**	--	**2510**			**20.8 kips**		

		Elliot Group - Browning Ski Lodge (Part A)											11/23/2017
		SUMMARY OF LATERAL FORCES- SEGMENTED											
Level	Mark	Force V(k)	Wood/Conc.	Wind/Seismic	Length (ft)	ν (plf)	Height (ft)	Reduction	SW Type	Uplift	Uplift LEFT	Uplift RIGHT	Holddowns
R	R.1.2	6.10	wood/concrete	Seismic	11.5	530	10	1.00	**D**	5304	4366	4366	STL COL.
R	R.1.3	2.50	wood/concrete	Seismic	6.6	379	10	1.00	**C**	3788	3249	3249	STL COL.
R	R.1.4	1.14	wood/concrete	Seismic	4.2	271	10	0.84	**B**	2714	2372	2372	HDU2/STL COL.
R	R.1.5	1.14	wood/concrete	Seismic	4.2	271	10	0.84	**B**	2714	2372	2372	HDU2/STL COL.
F	F.1.3	1.52	wood/concrete	Seismic	6.6	230	10	1.00	**A**	2297	4084	4084	STL COL.
F	F.1.4	7.4	wood/concrete	Seismic	15.5	477	10	1.00	**D**	4774	3937	3937	STL COL.

Holdown Note:
Many locations where holdowns should occur, steel columns are in their place for the gravity design of this house. Anchorage in these instances have checked for the worst possible uplift.

SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.5.6464.0

Company:	Canyons Structural	Date:	11/25/2017
Engineer:	CRF	Page:	1/5
Project:	Ski Lodge		
Address:			
Phone:			
E-mail:			

1.Project information

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

Project description: HSS moment Frame Connection
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): 14.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 17.75
c_{ac} (inch): 22.99
C_{min} (inch): 1.75
S_{min} (inch): 3.00

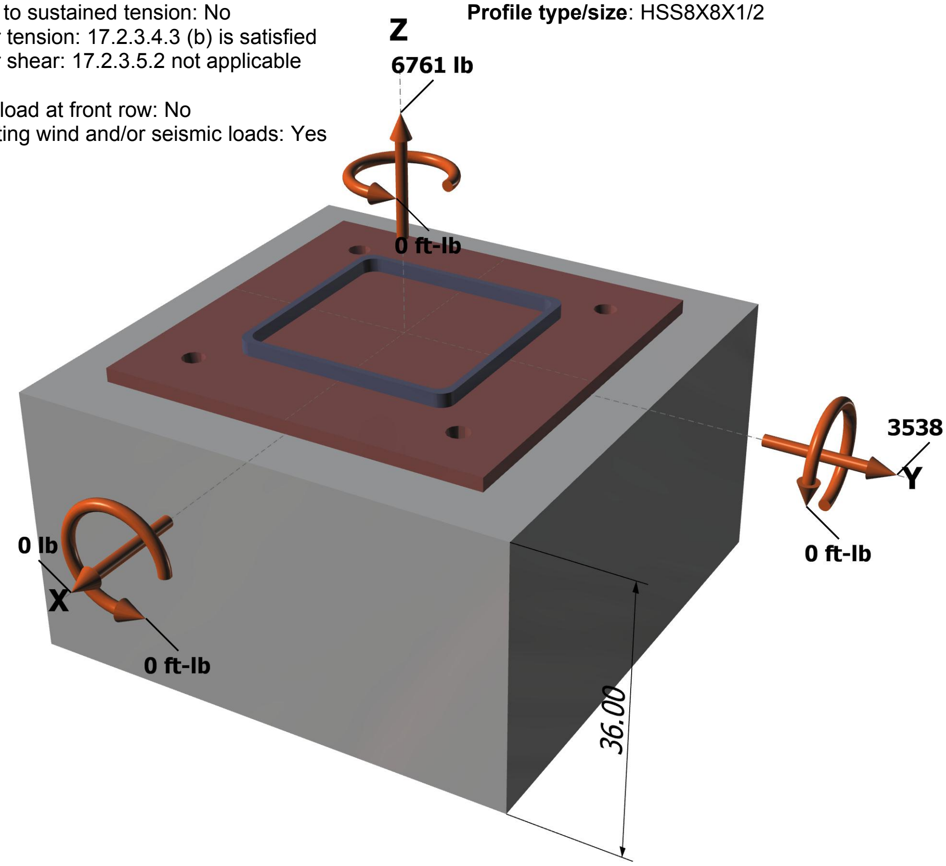
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.2.3.4.3 (b) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 36.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Hole condition: Dry concrete
Inspection: Periodic
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 14.00 x 14.00 x 0.50
Yield stress: 34084 psi

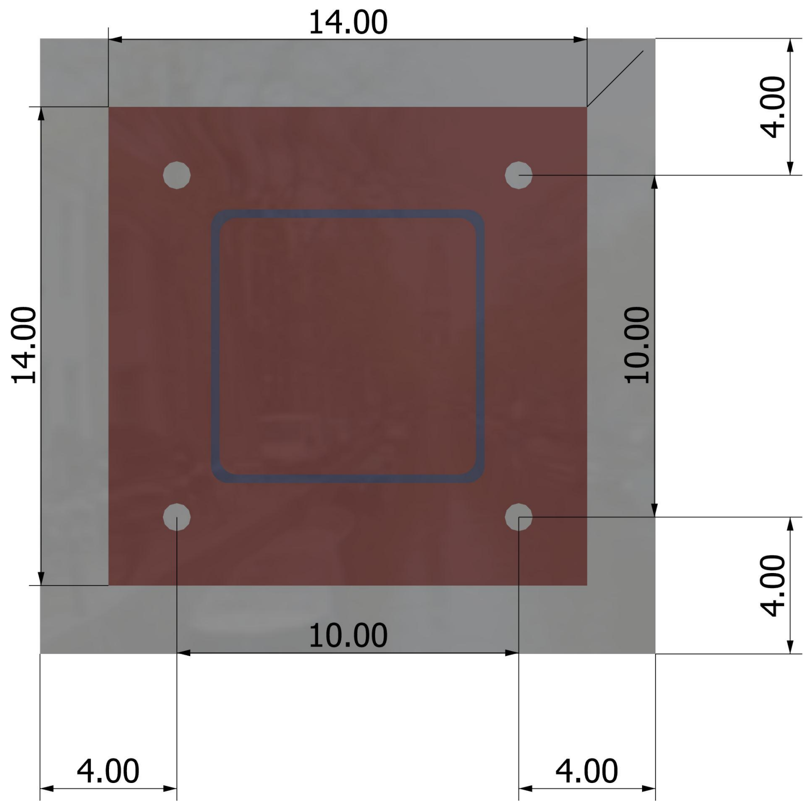
Profile type/size: HSS8X8X1/2

<Figure 1>



Company:	Canyons Structural	Date:	11/25/2017
Engineer:	CRF	Page:	2/5
Project:	Ski Lodge		
Address:			
Phone:			
E-mail:			

<Figure 2>



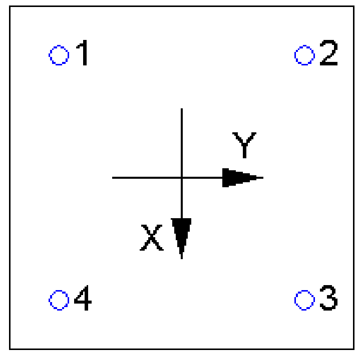
Recommended Anchor
Anchor Name: SET-XP® - SET-XP w/ 3/4"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

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	1690.3	0.0	884.5	884.5
2	1690.3	0.0	884.5	884.5
3	1690.3	0.0	884.5	884.5
4	1690.3	0.0	884.5	884.5
Sum	6761.0	0.0	3538.0	3538.0

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>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short-term} K_{sat} \alpha_{N.seis}$

$\tau_{k,cr}$ (psi)	$f_{short-term}$	K_{sat}	$\alpha_{N.seis}$	$\tau_{k,cr}$ (psi)
385	1.72	1.00	1.00	662

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	662	0.75	14.000	21844

$0.75\phi N_{ag} = 0.75\phi\,(A_{Na} / A_{Na0})\,\Psi_{ec,Na}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1b)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ec,Na}$	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba}(lb)	ϕ	$0.75\phi N_{ag}$ (lb)
324.00	341.26	9.24	4.00	1.000	0.830	1.000	21844	0.55	7100

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

V_{sa} (lb)	ϕ_{grout}	ϕ	$\alpha_{V,seis}$	$\phi_{grout}\alpha_{V,seis}\phi V_{sa}$ (lb)
11625	1.0	0.65	0.68	5138

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

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

k_{cp}	A_{Na} (in^2)	A_{Na0} (in^2)	$\Psi_{ed,Na}$	$\Psi_{ec,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	N_a (lb)
2.0	324.00	341.26	0.830	1.000	1.000	21844	17211

A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	N_{cb} (lb)	ϕ
324.00	100.00	1.000	0.940	1.000	1.000	5173	15755	0.70

ϕV_{cpg} (lb)
22057

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	1690	14528	0.12	Pass
Adhesive	**6761**	**7100**	**0.95**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	**885**	**5138**	**0.17**	**Pass (Governs)**
Pryout	3538	22057	0.16	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	0.95	0.00	95.2 %	1.0	Pass

SET-XP w/ 3/4"Ø F1554 Gr. 36 with hef = 14.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.313 inch

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

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

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

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Concrete Column

Lic. # : KW-06009078 **Licensee : Canyons Structural Inc**

Description : CC1, includes lateral (Worst Case) - Works for all dimensions required in this structure

Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : IBC 2015

General Information

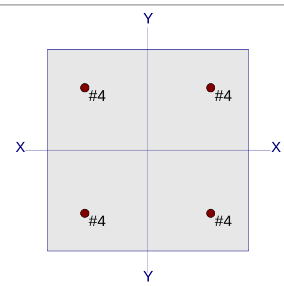
f'c : Concrete 28 day strength	=	2.50 ksi
E =	=	3,122.0 ksi
Density	=	150.0 pcf
β	=	0.850
fy - Main Rebar	=	60.0 ksi
E - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits		*ASTM A615 Bars Used*
Min. Reinf.	=	1.0 %
Max. Reinf.	=	8.0 %

Overall Column Height = 10.0 ft
End Fixity: Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 7.0 ft, K = 0.70
Y-Y (depth) axis :
Unbraced Length for X-X Axis buckling = 7.0 ft, K = 1.0

Column Cross Section

Column Dimensions : 12.0in Square Column, Column Edge to Rebar Edge Cover = 2.0in

Column Reinforcing : 4 - #4 bars @ corners,



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included : 1,500.0 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft above base, D = 8.0, S = 26.0 k
BENDING LOADS . . .
Lat. Point Load at 10.0 ft creating My-y, E = 3.70 k

DESIGN SUMMARY

Load Combination +1.20D+1.60S
Location of max.above base 9.933 ft

Maximum Stress Ratio **0.289 : 1**
Ratio = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5

Pu = 53.0 k   φ * Pn = 183.196 k
Mu-x = 0.0 k-ft   φ * Mn-x = 0.0 k-ft
Mu-y = 0.0 k-ft   φ * Mn-y = 0.0 k-ft
Mu Angle = 0.0 deg
Mu at Angle = 0.0 k-ft   φMn at Angle = 0.0 k-ft

Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . . .
Pnmax : Nominal Max. Compressive Axial Capacity 352.30 k
Pnmin : Nominal Min. Tension Axial Capacity -48.0 k
φ Pn, max : Usable Compressive Axial Capacity 183.196 k
φ Pn, min : Usable Tension Axial Capacity -31.20 k

Maximum SERVICE Load Reactions . .
Top along Y-Y 3.70 k   Bottom along Y-Y 0.0 k
Top along X-X 0.0 k   Bottom along X-X 0.0 k

Maximum SERVICE Load Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination :
Along X-X 0.0 in at 0.0 ft above base
for load combination :

General Section Information . φ = 0.650   β = 0.850   θ = 0.80
ρ : % Reinforcing 0.5556 % Rebar < Min of 1.0 %
Reinforcing Area 0.80 in^2
Concrete Area 144.0 in^2

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	Axial Load k φ * Pn	δx	δx * Mux	δy	δy * Muy	Bending Analysis k-ft Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.40D			9.93	13.30	183.20					0.000			0.073

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Description : CC1, includes lateral (Worst Case) - Works for all dimensions required in this structure

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	Axial Load k φ * Pn	Bending Analysis k-ft δx	δx * Mux	δy	δy * Muy	Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.20D			9.93	11.40	183.20					0.000			0.062
+1.20D+0.50S			9.93	24.40	183.20					0.000			0.133
+1.20D+1.60S			9.93	53.00	183.20					0.000			0.289
+1.20D+0.70S+E			9.93	29.60	183.20					0.000			0.162
+1.20D+0.70S-E			9.93	29.60	183.20					0.000			0.162
+0.90D			9.93	8.55	183.20					0.000			0.047
+0.90D+E			9.93	8.55	183.20					0.000			0.047
+0.90D-E			9.93	8.55	183.20					0.000			0.047

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Reaction along X-X Axis @ Base	Reaction along X-X Axis @ Top	Reaction along Y-Y Axis @ Base	Reaction along Y-Y Axis @ Top	Axial Reaction @ Base
D Only		k		k	9.500 k
+D+S		k		k	35.500 k
+D+0.750S		k		k	29.000 k
+D+0.70E		2.590 k		k	9.500 k
+D+0.750S+0.5250E		1.943 k		k	29.000 k
+0.60D		k		k	5.700 k
+0.60D+0.70E		2.590 k		k	5.700 k
S Only		k		k	26.000 k
E Only		3.700 k		k	k

Maximum Moments

Note: Only non-zero reactions are listed.

Load Combination	Moment About X-X Axis @ Base	Moment About X-X Axis @ Top	Moment About Y-Y Axis @ Base	Moment About Y-Y Axis @ Top
D Only		k-ft		k-ft
+D+S		k-ft		k-ft
+D+0.750S		k-ft		k-ft
+D+0.70E		k-ft		k-ft
+D+0.750S+0.5250E		k-ft		k-ft
+0.60D		k-ft		k-ft
+0.60D+0.70E		k-ft		k-ft
S Only		k-ft		k-ft
E Only		k-ft		k-ft

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750S	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.70E	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750S+0.5250E	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D+0.70E	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
E Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Concrete Column

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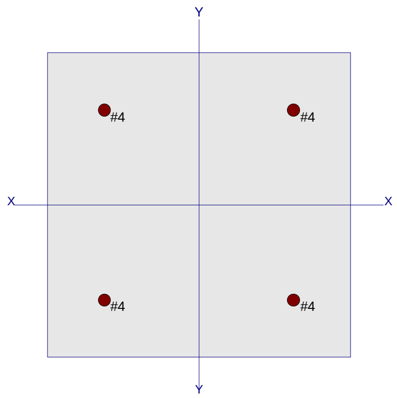
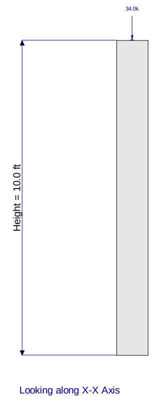
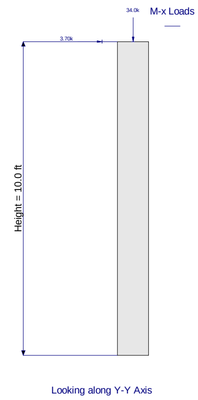
Description : CC1, includes lateral (Worst Case) - Works for all dimensions required in this structure

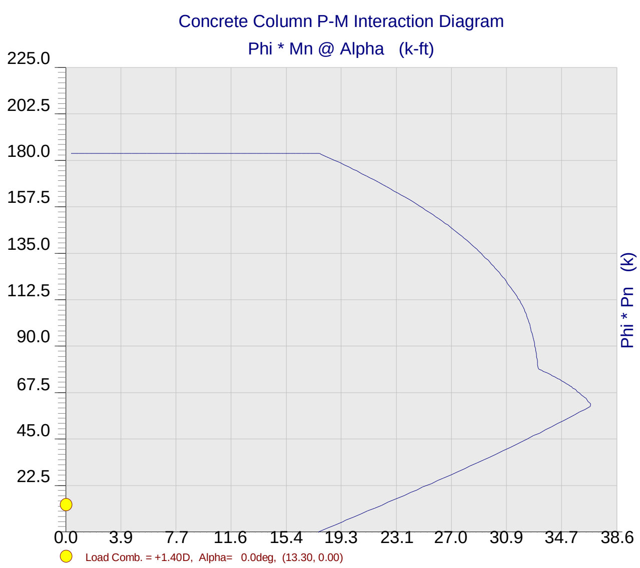
Sketches

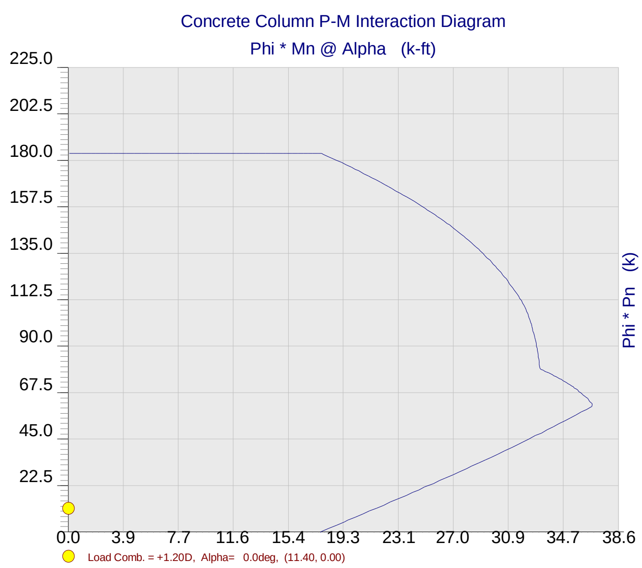






Interaction Diagrams





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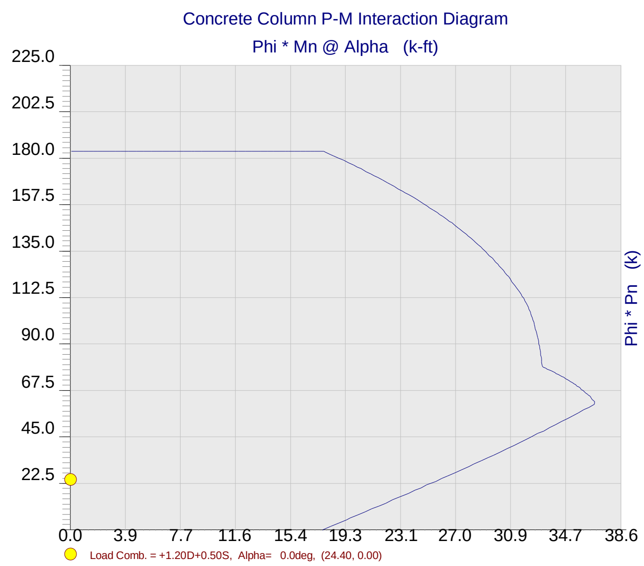
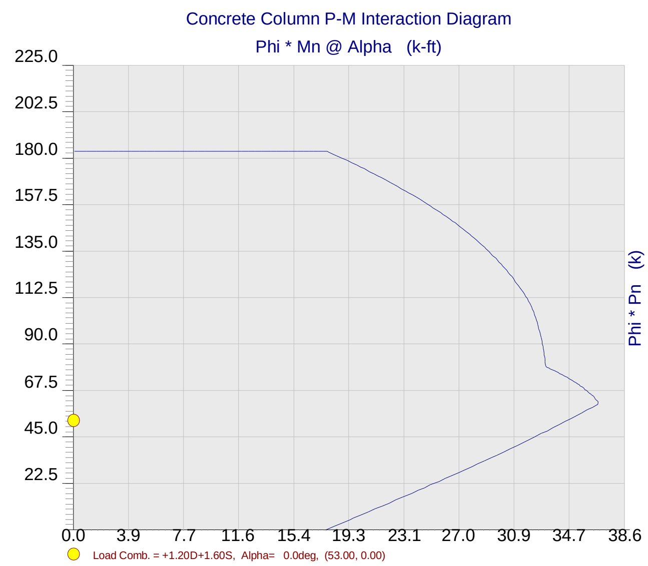
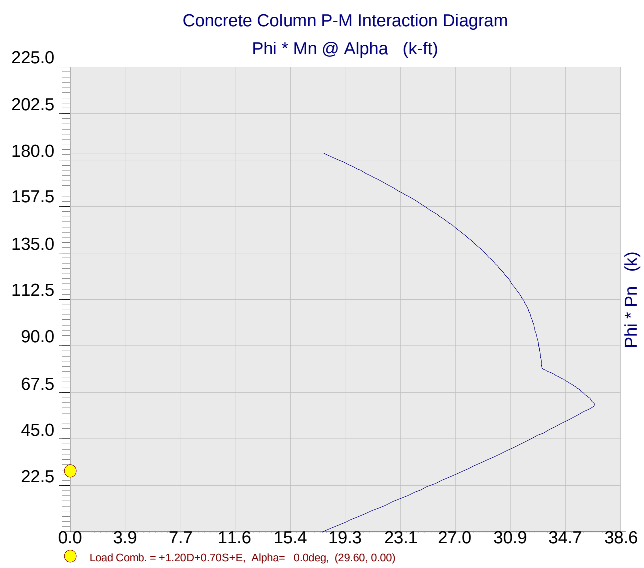
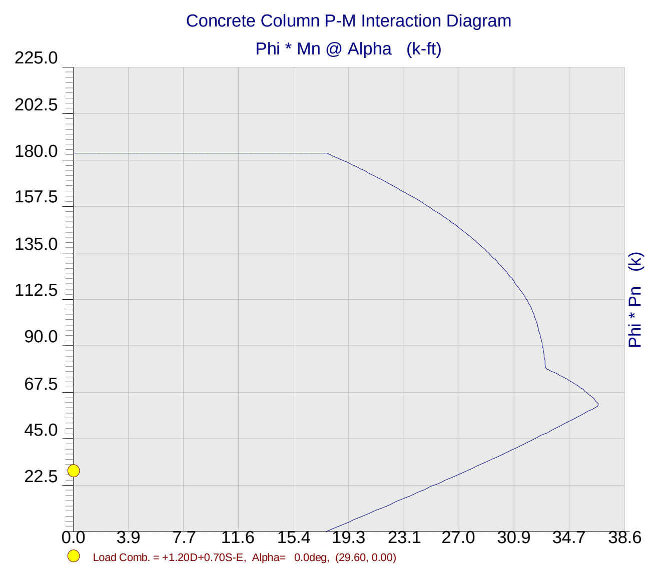
Description : CC1, includes lateral (Worst Case) - Works for all dimensions required in this structure









Concrete Column

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ENERCALC, INC. 1983-2016, Build:6.16.7.21, Ver:6.16.7.21

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Description : CC1, includes lateral (Worst Case) - Works for all dimensions required in this structure