ReidMiddleton

CIVIL ENGINEERING
STRUCTURAL ENGINEERING
SURVEYING
PLANNING

Alaska Court System Snowden Admin Building Mechanical Upgrades

Anchorage, Alaska

RM Project #402021.105.000

Structural Calculations
June 24, 2022

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Alaska Court System
Snow Admin Building
Mechanical Upgrades
RM Project 402021.105.000
June 24, 2022

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Hazards by Location

DESIGN LOADS

Search Information

Address: 820 W 4th Ave, Anchorage, AK 99501, USA

Coordinates: 61.21826910000001, -149.8983215

Elevation: 94 ft

Timestamp: 2022-05-05T20:49:18.082Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default

Basic Parameters

Name	Value	Description
S_S	1.5	MCE_R ground motion (period=0.2s)
S_1	0.676	MCE_R ground motion (period=1.0s)
S_{MS}	1.8	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.2	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	1.114	Coefficient of risk (0.2s)
CR_1	1.038	Coefficient of risk (1.0s)
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.6	Site modified peak ground acceleration

T_L	16	Long-period transition period (s)
SsRT	1.897	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.703	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.831	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.8	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.676	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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Client	AMC	Sheet	of
Project	Snowden Mech	Design by	EH
		Date	03/29/22
		Checked by	
Project No.	40-21-205	Date	

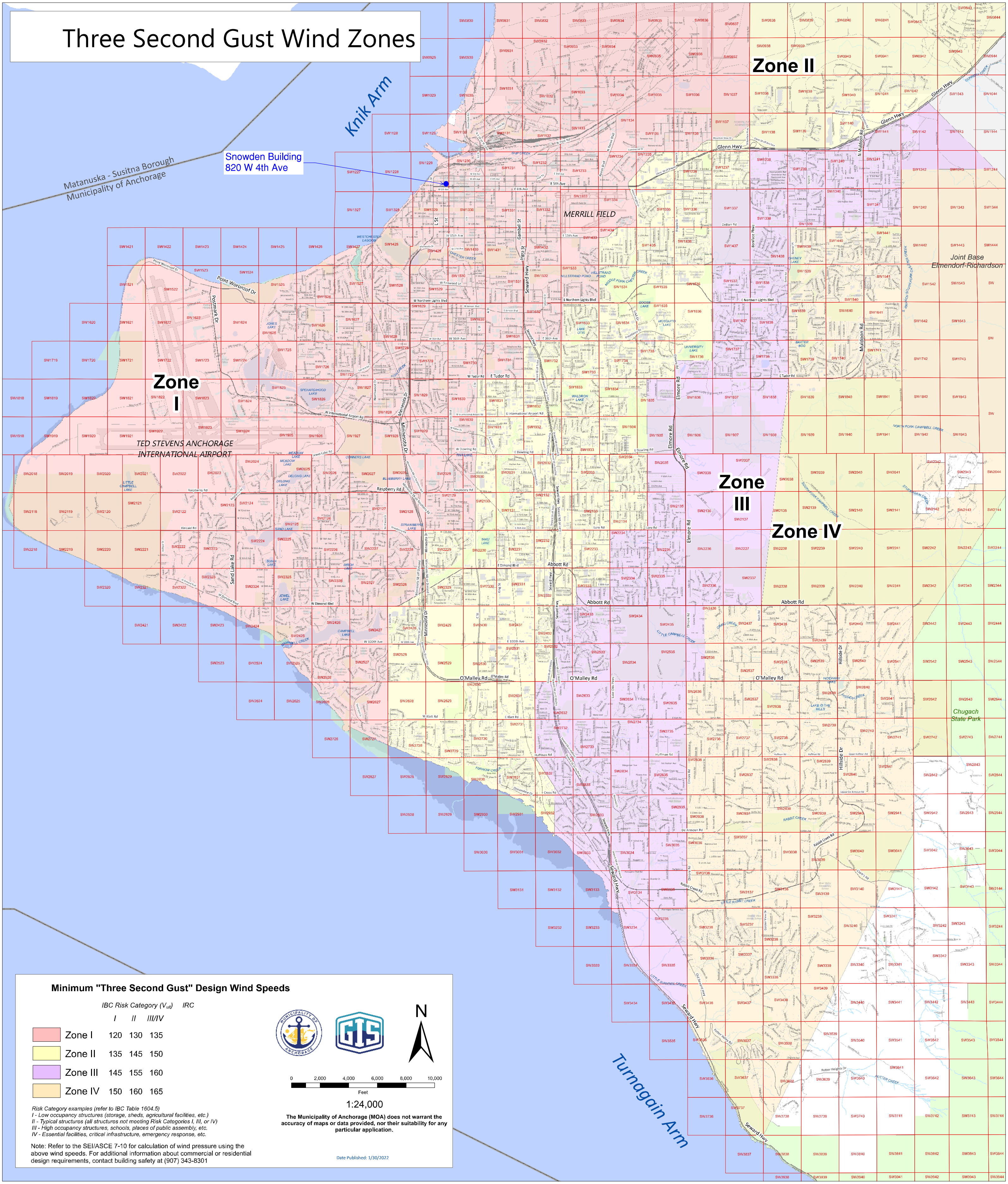
ASCE 7-16 EARTHQUAKE FORCES PER IBC 2018- Equivalent Lateral Force (ELF) Procedure

I.D.: Penthouse Mech Space

INPUT:				**Ref:**
Mapped short period accel @ T=0.2 s:	S_s =	1.500	g	11.4.2
Mapped acceleration @ T=1 s:	S_1 =	0.676	g	11.4.2
Mapped Long Term Transition Period:	T_L=	16	sec	11.4.6
Site class:		D Default	Site Class E, 11.4.8 Exception 1? no	11.4.3
Basic seismic-force-resisting system:		Bearing wall system		12.2.1
Lateral system:		Light-frame (wood/cold-formed steel) walls with wood shear panels (bearing wall)		12.2.1
Occupancy Category:		I or II (Nonsubstantial Hazard to Human Life/Nonessential)		1.5.1
Building height:	h_n =	36	ft	12.8.2.1
Calculated fundamental period:	T_c =		s	12.8.2
Site-specific Ground Motion Analysis Required?		No, Cs per 12.8-2		11.4.8
Cs Requirement based on 11.4.8:		Default Cs - eq. 12.8-2 - 11.4.8 exception 2		11.4.8

BASE SHEAR CALCULATION:			**Ref:**
Calculated acceleration site coefficient:	F_a =	1.20	11.4.4
Calculated velocity site coefficient:	F_v =	1.70	11.4.4
Adjusted short period acceleration:	$S_{MS} = F_a{*}S_s$ =	1.800 g	eq 11.4-1
Adjusted 1 sec. period acceleration:	$S_{M1} = F_v{*}S_1$ =	1.149 g	eq 11.4-2
Design short period acceleration:	$S_{DS} = 2/3{*}S_{MS}$ =	1.200 g	eq 11.4-3
Design 1 sec. period accel.:	$S_{D1} = 2/3{*}S_{M1}$ =	0.766 g	eq 11.4-4
Control period 1:	$T_O = 0.2{*}S_{D1}/S_{DS}$ =	0.128 s	11.4.6
Control period 2:	$T_S = S_{D1}/S_{DS}$ =	0.638 s	11.4.6
Period threshold for 11.4.8 Exception 2:	1.5*Ts =	0.958 s	11.4.8
Response modification factor:	R =	6.5	12.2.1
Overstrength Factor	Ω_0 =	3.0	12.2.1
Deflection Amplification Factor	C_d =	4	12.2.1
Seismic importance factor:	I_E =	1.00	11.5.1
Seismic design category:	SDC =	D	11.6
Period coefficient:	C_t =	0.020	12.8.2.1
Period exponent:	x =	0.75	12.8.2.1
Approximate fundamental period:	$T_a = C_t{*}h_n^x$ =	0.294 s	eq 12.8-8
Upper bound period coefficient:	C_u =	1.4	12.8.2
Upper bound period:	$T_u = C_u{*}T_a$ =	0.412 s	12.8.2
Fundamental period:	T =	0.294 s	12.8.2
Default seismic response coefficient:	$C_s = S_{DS}/(R/I_E)$ =	0.185	eq 12.8-2
Upper bound response coefficient:	C_s = SD1/(R/IE*T)	0.401	eq 12.8-3/4
Lower bound response coefficient:	$C_s = 0.044{*}S_{DS}{*}I_E$ =	0.053	eq 12.8-5
S_1 > 0.6g lower bound coefficient	$C_s = 0.5{*}S_1/(R/I_E)$ =	0.052	eq 12.8-6
Controlling lower bound coefficient:	C_s =	0.053	12.8.1.1
Controlling response coefficient:	C_s =	0.185	12.8.1.1
Final Response Coefficient:	**C_s =**	**0.185**	
Controlling response coefficient:	$C_{s(ASD)} = C_s/1.4$ =	0.132	12.8.1.1

**Seismically isolated & structures with damping on sites with S1 ≥ 0.6 require Site Specific Ground Motion Analysis*



Three Second Gust Wind Zones
Zone I
Zone II
Zone III
Zone IV
Knik Arm
Turnagain Arm
Matanuska - Susitna Borough
Municipality of Anchorage
Snowden Building
820 W 4th Ave
MERRILL FIELD
TED STEVENS ANCHORAGE INTERNATIONAL AIRPORT
Joint Base Elmendorf-Richardson
Chugach State Park
Glenn Hwy
Seward Hwy
Muldoon Rd
Elmore Rd
Hillside Dr
Abbott Rd
O'Malley Rd
Point Woronzof Dr
Minimum "Three Second Gust" Design Wind Speeds
IBC Risk Category (V_{ult}) IRC
I II III/IV
Zone I 120 130 135
Zone II 135 145 150
Zone III 145 155 160
Zone IV 150 160 165
Risk Category examples (refer to IBC Table 1604.5)
I - Low occupancy structures (storage, sheds, agricultural facilities, etc.)
II - Typical structures (all structures not meeting Risk Categories I, III, or IV)
III - High occupancy structures, schools, places of public assembly, etc.
IV - Essential facilities, critical infrastructure, emergency response, etc.
Note: Refer to the SEI/ASCE 7-10 for calculation of wind pressure using the above wind speeds. For additional information about commercial or residential design requirements, contact building safety at (907) 343-8301
N
0 2,000 4,000 6,000 8,000 10,000
Feet
1:24,000
The Municipality of Anchorage (MOA) does not warrant the accuracy of maps or data provided, nor their suitability for any particular application.
Date Published: 1/30/2022

ASCE 7-16 Wind Analysis for Enclosed or Partially Open/Enclosed Buildings

Wind speed for the applicable wind category, V= 130 mph (basic wind speed/3-sec gust speed at 33 ft)
Wind Directionality Factor, K_d= 0.85 (Table 26.6-1)
Elevation (asl) of site, z_g= 107 ft (Art. 26.9; elevation above sea level)
Ground Elevation Factor, K_e= 0.996 (Table 26.9-1)
Exposure: B (Art. 26.7.3)
Mean Roof Height, h= 37 ft (Spreadsheet maxes out at 110 ft)
Top of Parapet Height (from ground), h_p= 37 ft
Enclosure: Partially Enclosed (Art. 26.2)
Roof Slope, θ= 0.0°

Slope Converter: 12
0.0° = 0
Roof Slope

125 ft = W_1
BUILDING PLAN
N
180 ft = W_2

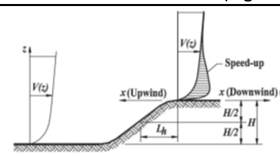
Topographic Effects: (Fig 26.8-1)

Ridge, Escarpment, or Hill? None
Height of Feature, H= 20 ft
Horiz. distance from halfway up the hill to the top, L_h= 100 ft
Distance upwind(-) or down(+) from crest to building site, x= 50 ft
Height of Building Site, z= 50 ft

K_1=
K_2=
γ =
K_3=

Note b in Fig. 26.8-1
H/L_h= 0.20
H/L_h for K_1= 0.20
L_h for K_2 & K_3= 100
μ= 0

(Art 26.8.2)
K_{zt}= $(1+K_1K_2K_3)^2$
K_{zt}= 1.00



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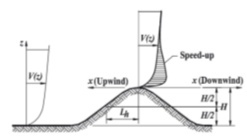


2-D RIDGE OR 3-D AXISYMMETRICAL HILL

Components and Cladding - Part 1 (Art. 30.3) & Part 3 (Art. 30.5):

GC_p Values from **Figure 30.3-2A** Roof Type: Flat or Gable

Zone	10 SF		20 SF		50 SF		100 SF		Exact Solution	
1	0.30	-1.70	0.27	-1.58	0.23	-1.41	0.20	-1.29	0.25	-1.49
1-OH	N/A	-1.70	N/A	-1.67	N/A	-1.63	N/A	-1.60	N/A	-1.65
1'	0.30	-0.90	0.27	-0.90	0.23	-0.90	0.20	-0.90	0.25	-0.90
1'-OH	N/A	-1.70	N/A	-1.67	N/A	-1.63	N/A	-1.60	N/A	-1.65
2	0.30	-2.30	0.27	-2.14	0.23	-1.93	0.20	-1.77	0.25	-2.03
2-OH	N/A	-2.30	N/A	-2.09	N/A	-1.81	N/A	-1.59	N/A	-1.94
3	0.30	-3.20	0.27	-2.88	0.23	-2.46	0.20	-2.14	0.25	-2.66
3-OH	N/A	-3.20	N/A	-2.83	N/A	-2.34	N/A	-1.96	N/A	-2.58
4	0.90	-0.99	0.85	-0.94	0.79	-0.88	0.74	-0.83	0.82	-0.91
5	0.90	-1.26	0.85	-1.16	0.79	-1.04	0.74	-0.94	0.82	-1.10

GC_{pi}= ± 0.55 q_h= 27.1 psf

a = 12.5 ft (H<60', FIG 30.3)

$GC_{pi\text{-}OVHNG}$= ± 0

for buildings with:

h ≤ 60' : $p = q_h[(GC_p)-(GC_{pi})]$ (Eq. 30.3-1)

h > 60' : $p = q(GC_p)-q_i(GC_{pi})$ (Eq. 30.5-1)

Notes: $q_i = q_h$ for all cases herein
(+) pressures are always toward surface

Components and Cladding Wind Load Chart

Zone			10 SQUARE FEET		20 SQUARE FEET		50 SQUARE FEET		100 SQUARE FEET	
ROOF	Main Int	**1**	23 psf	-61 psf	22 psf	-58 psf	21 psf	-53 psf	20 psf	-50 psf
ROOF	Main Int OH	**1-OH**	N/A	-46 psf	N/A	-45 psf	N/A	-44 psf	N/A	-43 psf
ROOF	Central Int	**1'**	23 psf	-39 psf	22 psf	-39 psf	21 psf	-39 psf	20 psf	-39 psf
ROOF	Central Int OH	**1'-OH**	N/A	-46 psf	N/A	-45 psf	N/A	-44 psf	N/A	-43 psf
ROOF	Edge	**2**	23 psf	-77 psf	22 psf	-73 psf	21 psf	-67 psf	20 psf	-63 psf
ROOF	Edge OH	**2-OH**	N/A	-62 psf	N/A	-57 psf	N/A	-49 psf	N/A	-43 psf
ROOF	Corner	**3**	23 psf	-102 psf	22 psf	-93 psf	21 psf	-82 psf	20 psf	-73 psf
ROOF	Corner OH	**3-OH**	N/A	-87 psf	N/A	-77 psf	N/A	-63 psf	N/A	-53 psf
WALL	Int. Wall	**4**	39 psf	-42 psf	38 psf	-41 psf	36 psf	-39 psf	35 psf	-37 psf
WALL	Edge Wall	**5**	39 psf	-49 psf	38 psf	-47 psf	36 psf	-43 psf	35 psf	-41 psf

Snow Load

ASCE 7-16

Section 7.3, p. 51-54

Project:	Snowden
Job Number:	402021.105
Date:	2/7/2022
Engineer:	DV

Location ("Anchorage" or other):	**Anchorage**	*elevation addition*	*0.0 psf*
Anchorage, Eagle River, or Homer?	**Yes**		
Ground Snow Load, p_g =	50 psf	**Final p_g:**	**50.0 psf**
Elevation =	107 ft		
Importance Factor, I_s =	Risk Category II		
	1.00	Table 1.5-2, p. 5	
Exposure Factor, C_e =	1.00	Table 7-3.1, p. 58	
Thermal Factor, C_t =	All other structures		
	1.00	Table 7-3.2, p. 58	
Roof Slope Factor, C_s =	All Other Surfaces		
	1.000	Figure 7-4.1, p. 59	
Roof Slope Calculator:	0.00	:12 =	0.00°
Roof Pitch, θ =	0.00	deg	
Flat Roof Snow Load (per calc), $p_f = 0.7{*}I{*}C_e{*}C_t{*}p_g$ =	35 psf	Eq 7.3-1	
Local Amendment Flat Roof Snow Load (if any), $p_{f\text{-}local}$ =	40 psf		
Req'd Sloped Roof Snow Load ps = $p_{f\text{-}max}{*}C_s$ =	**40 psf**	Eq 7.4-1	

Partial Snow Load:
Section 7.5, p. 54-57
Figure 7.5-1

Slope < 2.4deg or >30deg, Partial loads need not be considered per 7.5.1, p. 57

Unbalanced Snow Load:
Figure 7-6.2, p. 57-61

Slope < 2.4deg or >30deg, Unbalanced loads need not be considered per 7.6.1, p. 57

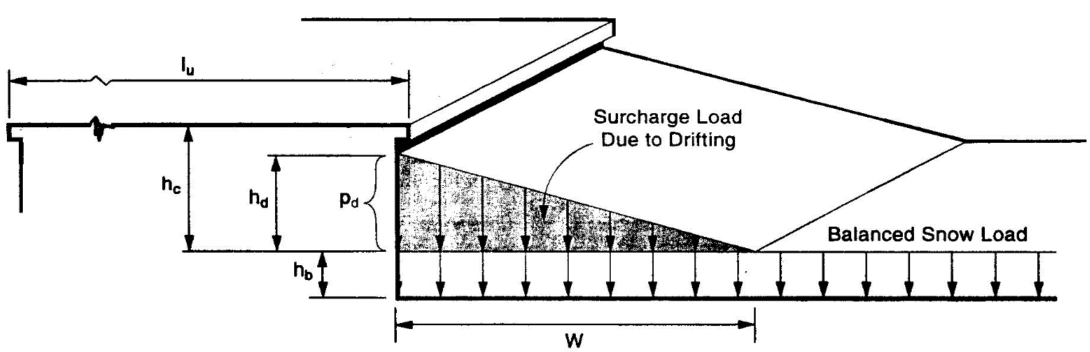
Drifting per ASCE 7-16 Sect 7.7:

Job Name:	Snowden
RM Job Number:	402021.105
Engineer:	DV

Ground Snow Load, p_g = 50 psf
Basic Roof Snow Load, p_f = 40 psf
Importance Factor, I_s = 1.0

	Location:	1	2	3
Width of upper roof, 20' min	l_u	52.6 ft	67.6 ft	65.8 ft
Width of Lower Roof, 20' min in equation	l_l	76.5 ft	34.0 ft	63.3 ft
Difference in roof heights	h_r	12.0 ft	11.0 ft	11.0 ft
Is this a parapet or roof projection? (if no = no change, if yes = 3/4 of drift per Sect 7.8)		no	no	no
$h_c = h_r - h_b$	h_c	10.0 ft	9.0 ft	9.0 ft
Height of drift = $I_s^{1/2}$ * [(0.43 * $l_x^{1/3}$ * $(p_g+10)^{1/4}$) - 1.5], (*limited to 60% of length of lower roof*)	h_d	3.0 ft	3.4 ft	3.3 ft
Governing direction (leeward or windward)		leeward	leeward	leeward
Width of drift = 4*h_d, 4h_d^2/h_c, 8h_c max	W	**11.9 ft**	**13.5 ft**	**13.3 ft**
Density of snow = 0.13p_g+14, 30pcf max	γ	20.5 pcf	20.5 pcf	20.5 pcf
Depth of balance snow load = p_f/γ	h_b	2.0 ft	2.0 ft	2.0 ft
Consider drift? Is h_c/h_b > 0.2?		*YES*	*YES*	*YES*
Added Snow Drift at high point	P_{max}=	**61 psf**	**69 psf**	**68 psf**
Added Snow Drift at low point	P_{min}=	**0 psf**	**0 psf**	**0 psf**

	Location:	4	5
Width of upper roof, 20' min	l_u	36.0 ft	62.8 ft
Width of Lower Roof, 20' min in equation	l_l	14.6 ft	66.3 ft
difference in roof heights	h_r	11.0 ft	12.0 ft
Is this a parapet or roof projection? (if no = no change, if yes = 3/4 of drift per Sect 7.8)		no	no
$h_c = h_r - h_b$	h_c	9.0 ft	10.0 ft
Height of drift = sqrt(I_s) *[(0.43 * $l^{1/3}$ * $(p_g+10)^{1/4}$) - 1.5], (*limited to 60% of length of lower roof*)	h_d	2.5 ft	3.3 ft
governing direction (leeward or windward)		leeward	leeward
Width of drift = 4*h_d, 4h_d^2/h_c, 8h_c max	W	**9.8 ft**	**13.0 ft**
Density of snow = .13p_g+14, 30 max	γ	20.5 pcf	20.5 pcf
Depth of balance snow load = p_f/γ	h_b	2.0 ft	2.0 ft
Consider drift?		*YES*	*YES*
Added Snow Drift at high point	P_{max}=	**50 psf**	**67 psf**
Added Snow Drift at low point	P_{min}=	**0 psf**	**0 psf**



Drifting per ASCE 7-16 Sect 7.7:

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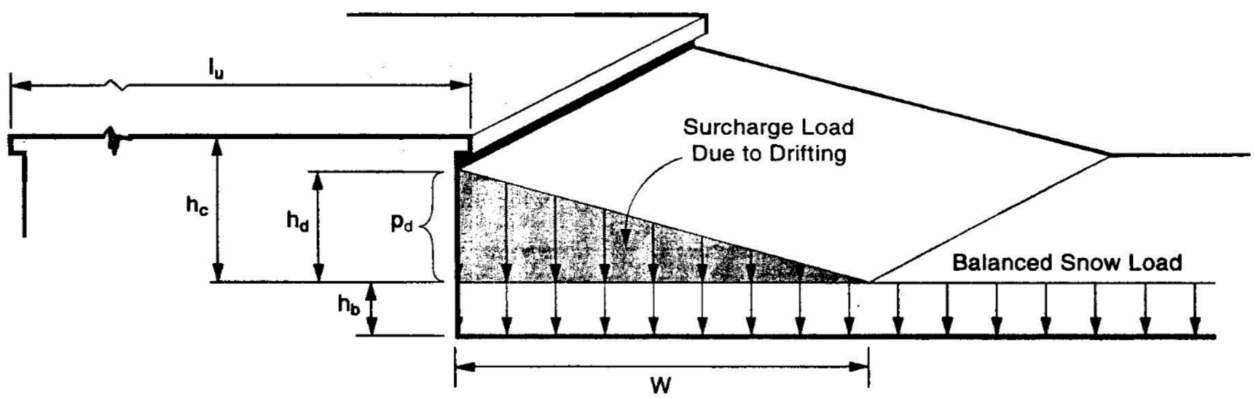
Job Name:	Snowden
RM Job Number:	402021.105
Engineer:	DV

Ground Snow Load, p_g = 50 psf
Basic Roof Snow Load, p_f = 40 psf
Importance Factor, I_s = 1.0

	Location:	0	6
Width of upper roof, 20' min	l_u	20.8 ft	19.1 ft
Width of Lower Roof, 20' min in equation	l_l	13.8 ft	14.6 ft
Difference in roof heights	h_r	12.0 ft	12.0 ft
Is this a parapet or roof projection? (if no = no change, if yes = 3/4 of drift per Sect 7.8)		no	no
$h_c = h_r - h_b$	h_c	10.0 ft	10.0 ft
Height of drift = $I_s^{1/2}$ * [(0.43 * $l_x^{1/3}$ * $(p_g+10)^{1/4}$) - 1.5], (*limited to 60% of length of lower roof*)	h_d	1.8 ft	1.7 ft
Governing direction (leeward or windward)		leeward	leeward
Width of drift = $4 h_d$, $4h_d^2/h_c$, $8h_c$ max	W	**7.2 ft**	**7.0 ft**
Density of snow = $0.13p_g+14$, 30pcf max	γ	20.5 pcf	20.5 pcf
Depth of balance snow load = p_f/γ	h_b	2.0 ft	2.0 ft
Consider drift? Is h_c/h_b > 0.2?		***YES***	***YES***
Added Snow Drift at high point	P_{max}=	**37 psf**	**36 psf**
Added Snow Drift at low point	P_{min}=	**0 psf**	**0 psf**



Snow Load
ASCE 7-16

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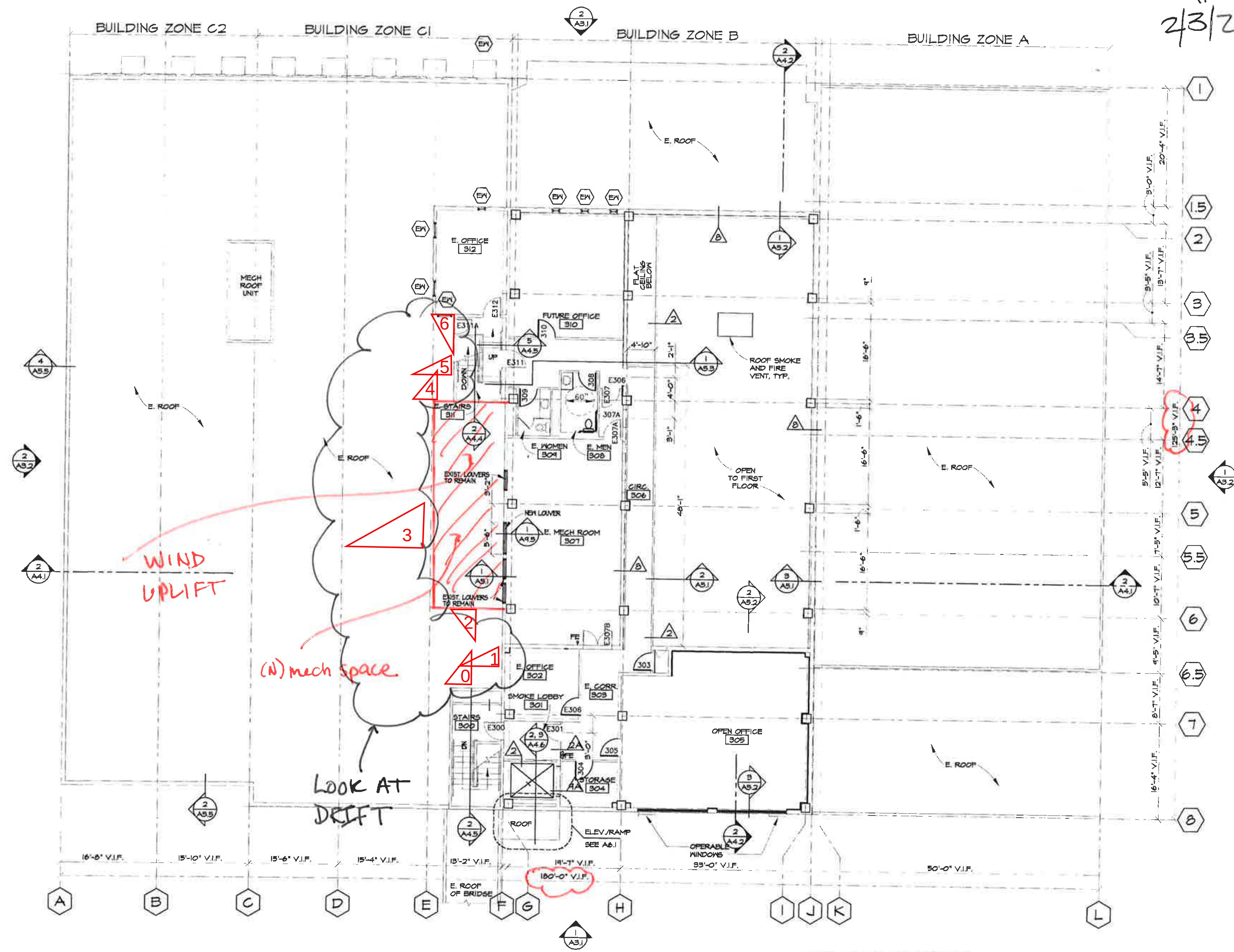
Project: Snowden
Job Number: 402021.1
Date: 2/9/2022
Engineer: DV

Drift Summary

Label	h_d (ft)	p_d (psf)	w (ft)
0	1.8	36.7	7.2
1	3.0	61.2	11.9
2	3.4	69.2	13.5
3	3.3	68.3	13.3
4	2.5	50.3	9.8
5	3.3	66.8	13.0
6	1.7	35.8	7.0

EH 2/3/22



3RD FLOOR PLAN - RENOVATION

SCALE: 1/8"=1'-0"

PROJECT RECORD DRAWINGS
McCOOL CARLSON GREEN-DATE:02-02-98

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LEGEND

- FLOOR ELEVATION VERIFY IN FIELD
- NEW CMU WALL
- NEW METAL STUD WALL
- NEW CONC. IN-FILL

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BY

DATE

Date Stamped:

By | Revision | Date

USKH

Architecture • Engineering
Land Surveying • Planning

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800 West Evergreen Street
Palmer, Alaska 99645
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Project:
ALASKA COURT SYSTEM
ADMINISTRATIVE OFFICES
RENOVATION AT THE ANCHORAGE TIMES BUILDING
820 W. 4TH AVE.
ANCHORAGE, ALASKA

Project Mgr.	JAH	
Drawn	JLL	PRM
Drawn	SDK	
Checked	JAH	
Date	6-10-96	

Sheet Contents:
RENOVATION
3RD FLOOR PLAN
4TH AVE. BLDG.

Sheet No.:
A2.3

USKH W.O. 4426.5

13

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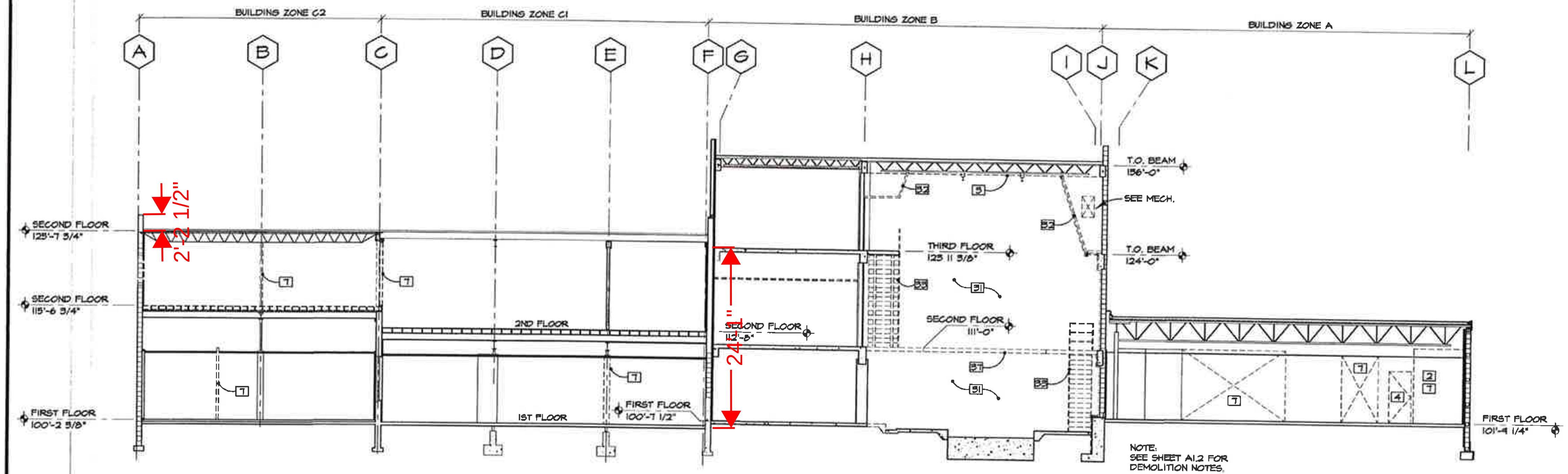
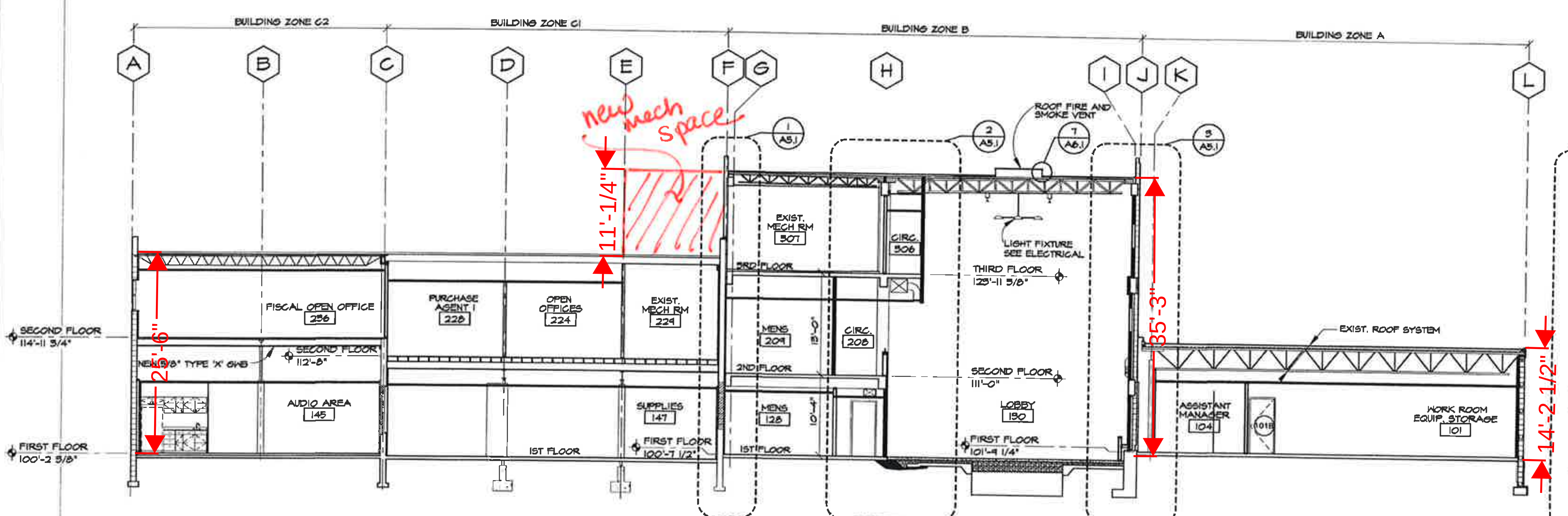
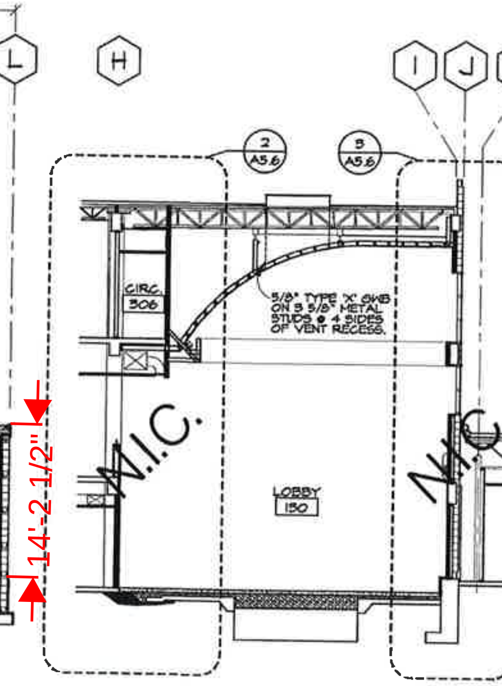
EH 2/3/22



1 A4.1 BUILDING SECTION-EXISTING
SCALE: 1/8" = 1'-0"



2 A4.1 BUILDING SECTION - RENOVATION
SCALE: 1/8" = 1'-0"
BASE BID



3 A4.1 PARTIAL BUILDING SECTION
SCALE: 1/8" = 1'-0"
ALTERNATE BID #5

FILE NAME: I:\44265\A\DWGS\44265A41.DWG
PLOT SCALE: 1=1

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BY

DATE 1-6-97

Date Stamped:

By | Revision | Date

USKH

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Project:
ALASKA COURT SYSTEM
ADMINISTRATIVE OFFICES
RENOVATION AT THE ANCHORAGE TIMES BUILDING
820 W. 4TH AVE.
ANCHORAGE, ALASKA

Project Mgr.	JAH	
Drawn	PRM	
Drawn	JLL	
Checked	JAH	JH
Date	6-10-96	

Sheet Contents:
BUILDING SECTIONS

Sheet No.:
A4.1

USKH W.O. 4426.5

Compare New Lateral Loads with Existing

Existing Roof Weight

Consider flexible diaphragm, spanning from Grid C to F

INPUTS:

existing DL =	20	psf	PH roof DL =	10
0.2 S =	8	psf	PH wall DL =	5
wood wall DL =	10	psf		
conc wall DL =	100	psf (8")		

Level 2 to 3, h =	13	ft
PH h =	11	ft
parapet h =	2.83	ft

EXISTING SEISMIC WEIGHT

			Weight	
Main area, L =	21.5	ft	74.6	k, roof (LxWx (DL + 0.2S))
W =	124	ft	155.9	k, concrete walls (2xL + W) * (0.5 x h + parapet h) * conc wall DL
north PH, L =	32	ft	8.3	k, roof (L x W) x (DL)
W =	13	ft	6.38	k, walls (2xW + L) x (PH h) x wood wall DL
south PH, L =	22	ft	4.0	k, roof (L x W) x (DL)
W =	9	ft	4.4	k, walls (2xW + L) x (PH h) x wood wall DL
	Existing seismic W =		253.6	k

ADDITIONAL SEISMIC WEIGHT

			Weight	
new PH, L =	34	ft	4.8	k, roof (LxW) x (PH DL)
W =	14	ft	5.28	k, walls (2xL + 2xW) x PH h x PH wall DL
hkp pad, t =	3.5	in	5.25	k, hkp (L x W x t/12 x 150 pcf)
L =	15		15.3	k
W =	8			

Additional seismic weight/existing seismic weight (at Grid Line F): 6.0% < 10%

Therefore, the exception of IEBC Section 806.3 is met and existing lateral system shall be permitted to remain unaltered.

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Project Title: REVIEW OF EXISTING ROOF MEMBERS FOR DRIFT
Engineer:
Project ID:
Project Descr:

Printed: 5 APR 2022, 8:21AM

Wood Beam

Project File: snowden.ec6

DESCRIPTION: (E) 2X10 JOIST BTWN D&E W/ DRIFT 3 LOAD BTWN GRIDS 4-6

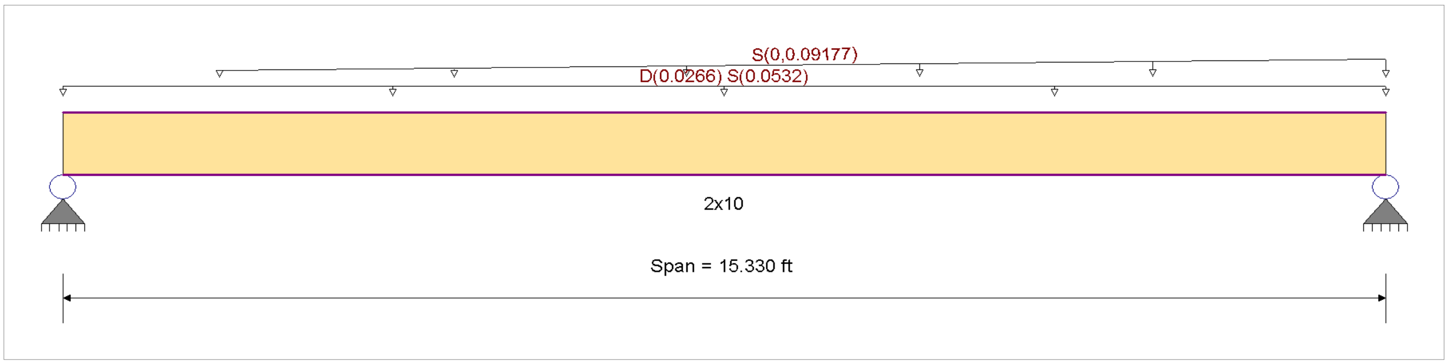
note to reviewer: material properties adjusted to historical values.

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,600.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1,600.0 psi	Ebend- xx	1,500.0 ksi
		Fc - Prll	1,050.0 psi	Eminbend - xx	470.0 ksi
Wood Species :	Hem-Fir	Fc - Perp	245.0 psi		
Wood Grade :	No.1	Fv	146.250 psi		
		Ft	825.0 psi	Density	28.720 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, S = 0.040 ksf, Tributary Width = 1.330 ft, (TYPICAL ROOF LOAD)
Varying Uniform Load : S= 0.0->0.0690 ksf, Extent = 1.830 -->> 15.330 ft, Trib Width = 1.330 ft, (DRIFT 3 LOAD)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	**1.001**: 1	Maximum Shear Stress Ratio	=	**0.608** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
fb: Actual	=	2,026.22 psi	fv: Actual	=	102.21 psi
Fb: Allowable	=	2,024.00 psi	Fv: Allowable	=	168.19 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.224 ft	Location of maximum on span	=	14.603 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.784 in	Ratio =	234 <240	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <240	n/a
Max Downward Total Deflection	1.031 in	Ratio =	178 <180	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.306	0.167	0.90	1.100	1.00	1.00	1.00	1.00	1.00	0.86	483.97	1584.00	0.20	22.03	131.63
+D+S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	1.001	0.608	1.15	1.100	1.00	1.00	1.00	1.00	1.00	3.61	2,026.22	2024.00	0.95	102.21	168.19
+D+0.750S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.810	0.489	1.15	1.100	1.00	1.00	1.00	1.00	1.00	2.92	1,640.13	2024.00	0.76	82.17	168.19
+0.60D					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.103	0.056	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.52	290.38	2816.00	0.12	13.22	234.00

Wood Beam

DESCRIPTION: (E) 2X10 JOIST BTWN D&E W/ DRIFT 3 LOAD BTWN GRIDS 4-6

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	1.0309	7.833		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.815	1.070
Overall MINimum	0.590	0.845
D Only	0.225	0.225
+D+S	0.815	1.070
+D+0.750S	0.667	0.859
+0.60D	0.135	0.135
S Only	0.590	0.845

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Engineer:
Project ID:
Project Descr:

Printed: 5 APR 2022, 8:19AM

Wood Beam

Project File: snowden.ec6

DESCRIPTION: (E) 2X10 JOIST BTWN D&E W/ DRIFT 3 LOAD BTWN GRIDS 4 & 6 - AFTER SISTER

note to reviewer: material properties adjusted to historical values.

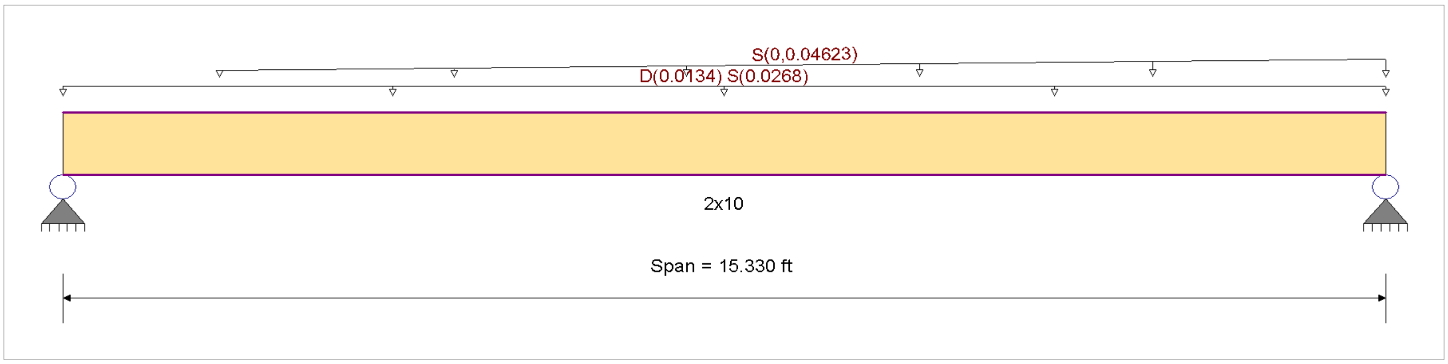
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,600.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1,600.0 psi	Ebend- xx	1,500.0 ksi
		Fc - Prll	1,050.0 psi	Eminbend - xx	470.0 ksi
Wood Species :	Hem-Fir	Fc - Perp	245.0 psi		
Wood Grade :	No.1	Fv	146.250 psi		
		Ft	825.0 psi	Density	28.720 pcf

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, S = 0.040 ksf, Tributary Width = 0.670 ft, (TYPICAL ROOF LOAD)
Varying Uniform Load : S= 0.0->0.0690 ksf, Extent = 1.830 -->> 15.330 ft, Trib Width = 0.670 ft, (DRIFT 3 LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.515** : 1	Maximum Shear Stress Ratio	=	**0.312** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
fb: Actual	=	1,043.26 psi	fv: Actual	=	52.52 psi
Fb: Allowable	=	2,024.00 psi	Fv: Allowable	=	168.19 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.169 ft	Location of maximum on span	=	14.603 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.395 in	Ratio =	466 >=240	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <240	n/a
Max Downward Total Deflection	0.531 in	Ratio =	346 >=180	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.168	0.092	0.90	1.100	1.00	1.00	1.00	1.00	1.00	0.47	266.43	1584.00	0.11	12.13	131.63
+D+S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.515	0.312	1.15	1.100	1.00	1.00	1.00	1.00	1.00	1.86	1,043.26	2024.00	0.49	52.52	168.19
+D+0.750S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.419	0.252	1.15	1.100	1.00	1.00	1.00	1.00	1.00	1.51	848.76	2024.00	0.39	42.42	168.19
+0.60D					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.057	0.031	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.28	159.86	2816.00	0.07	7.28	234.00

Wood Beam

DESCRIPTION: (E) 2X10 JOIST BTWN D&E W/ DRIFT 3 LOAD BTWN GRIDS 4 & 6 - AFTER SISTER

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.5309	7.833		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.421	0.550
Overall MINimum	0.297	0.426
D Only	0.124	0.124
+D+S	0.421	0.550
+D+0.750S	0.347	0.443
+0.60D	0.074	0.074
S Only	0.297	0.426

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Project Title:
Engineer:
Project ID:
Project Descr:

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Wood Beam

Project File: snowden.ec6

LIC# : KW-06017698, Build:20.22.3.16 REID MIDDLETON, INC.

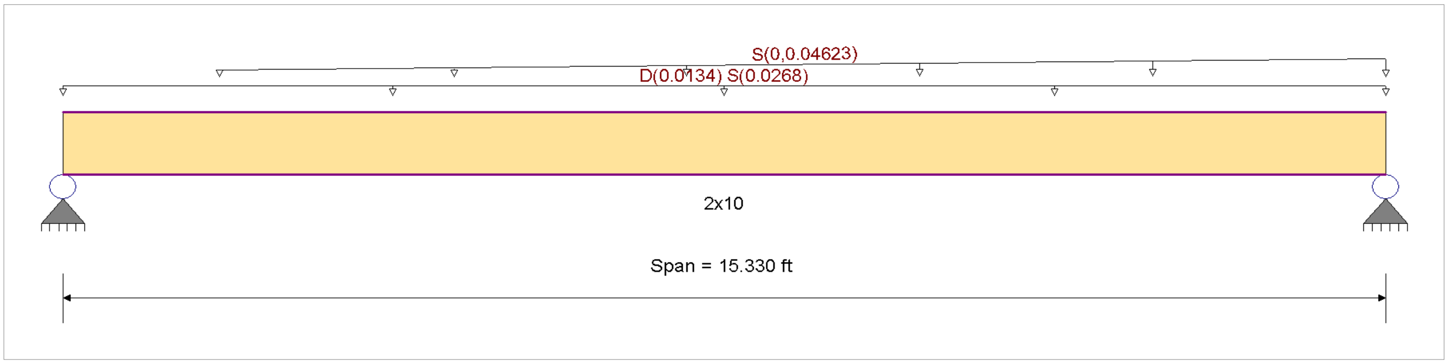
DESCRIPTION: (N) 2X10 SISTERED JOIST BTWN D&E

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*		
Load Combination :	IBC 2018	Fb -	850.0 psi		Ebend- xx	1,300.0 ksi
		Fc - Prll	1,300.0 psi		Eminbend - xx	470.0 ksi
Wood Species :	Hem-Fir	Fc - Perp	405.0 psi			
Wood Grade :	No.2	Fv	150.0 psi			
		Ft	525.0 psi		Density	26.840 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, S = 0.040 ksf, Tributary Width = 0.670 ft, (TYPICAL ROOF LOAD)
Varying Uniform Load : S= 0.0->0.0690 ksf, Extent = 1.830 -->> 15.330 ft, Trib Width = 0.670 ft, (DRIFT 3 LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.841**: 1	Maximum Shear Stress Ratio	=	**0.304** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
fb: Actual	=	1,040.28 psi	fv: Actual	=	52.39 psi
Fb: Allowable	=	1,236.54 psi	Fv: Allowable	=	172.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.169 ft	Location of maximum on span	=	14.603 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.455 in	Ratio =	403	>=240	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<240	n/a
Max Downward Total Deflection	0.611 in	Ratio =	301	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.272	0.089	0.90	1.100	1.00	1.15	1.00	1.00	1.00	0.47	263.45	967.73	0.11	11.99	135.00
+D+S					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.841	0.304	1.15	1.100	1.00	1.15	1.00	1.00	1.00	1.85	1,040.28	1236.54	0.48	52.39	172.50
+D+0.750S					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.684	0.245	1.15	1.100	1.00	1.15	1.00	1.00	1.00	1.51	845.79	1236.54	0.39	42.29	172.50
+0.60D					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.330 ft	**1**	0.092	0.030	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.28	158.07	1720.40	0.07	7.19	240.00

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Wood Beam

DESCRIPTION: (N) 2X10 SISTERED JOIST BTWN D&E

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.6108	7.833		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.420	0.548
Overall MINimum	0.297	0.426
D Only	0.123	0.123
+D+S	0.420	0.548
+D+0.750S	0.345	0.442
+0.60D	0.074	0.074
S Only	0.297	0.426

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Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 5 APR 2022, 8:21AM

Wood Beam

Project File: snowden.ec6

DESCRIPTION: (E) 2X10 JOIST BTWN E&F W/ DRIFT 5 LOAD (BETWEEN GRIDS 3.5 & 4)

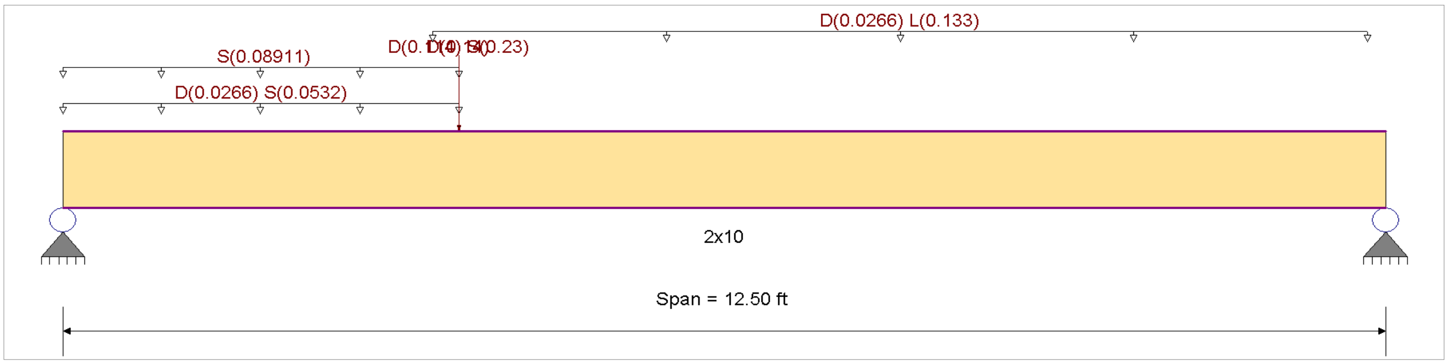
note to reviewer: material properties adjusted to historical values.

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,600.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1,600.0 psi	Ebend- xx	1,500.0 ksi
		Fc - Prll	1,050.0 psi	Eminbend - xx	470.0 ksi
Wood Species :	Hem-Fir	Fc - Perp	245.0 psi		
Wood Grade :	No.1	Fv	146.250 psi		
		Ft	825.0 psi	Density	28.720 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
- Uniform Load : D = 0.020, S = 0.040 ksf, Extent = 0.0 -->> 3.750 ft, Tributary Width = 1.330 ft, (TYPICAL ROOF LOAD)
- Uniform Load : S = 0.0670 ksf, Extent = 0.0 -->> 3.750 ft, Tributary Width = 1.330 ft, (DRIFT LOAD)
- Point Load : D = 0.140 k @ 3.750 ft, (WALL WEIGHT)
- Point Load : D = 0.1140, S = 0.230 k @ 3.750 ft, (LOAD FROM ROOF ABOVE)
- Uniform Load : D = 0.020, L = 0.10 ksf, Extent = 3.50 -->> 12.330 ft, Tributary Width = 1.330 ft, (TYPICAL FLOOR LOAD)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	1.036 : 1	Maximum Shear Stress Ratio	=	0.675 : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
fb: Actual	=	1,824.12 psi	fv: Actual	=	113.59 psi
Fb: Allowable	=	1,760.00 psi	Fv: Allowable	=	168.19 psi
Load Combination		+D+L	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	6.159 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.407 in	Ratio =	368 >=240	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <240	n/a
Max Downward Total Deflection	0.659 in	Ratio =	227 >=180	Span: 1 : +D+0.750L+0.750S
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.413	0.283	0.90	1.100	1.00	1.00	1.00	1.00	1.00	1.17	653.63	1584.00	0.34	37.25	131.63
+D+L					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	1.036	0.672	1.00	1.100	1.00	1.00	1.00	1.00	1.00	3.25	1,824.12	1760.00	0.91	98.32	146.25
+D+S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.684	0.550	1.15	1.100	1.00	1.00	1.00	1.00	1.00	2.47	1,383.68	2024.00	0.86	92.47	168.19
+D+0.750L					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.690	0.439	1.25	1.100	1.00	1.00	1.00	1.00	1.00	2.71	1,518.65	2200.00	0.74	80.23	182.81

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Wood Beam

DESCRIPTION: (E) 2X10 JOIST BTWN E&F W/ DRIFT 5 LOAD (BETWEEN GRIDS 3.5 & 4)

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+0.750L+0.750S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	**1**	0.964	0.675	1.15	1.100	1.00	1.00	1.00	1.00	1.00	3.48	1,952.02	2024.00	1.05	113.59	168.19
+0.60D					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	**1**	0.139	0.096	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.70	392.18	2816.00	0.21	22.35	234.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S	1	0.6592	6.113		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.150	1.001
Overall MINimum	0.615	0.149
D Only	0.366	0.257
+D+L	0.797	1.001
+D+S	0.981	0.406
+D+0.750L	0.689	0.815
+D+0.750L+0.750S	1.150	0.927
+0.60D	0.220	0.154
L Only	0.431	0.744
S Only	0.615	0.149

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Wood Beam

Project File: Snowden.ec6

LIC# : KW-06017698, Build:20.22.3.16 REID MIDDLETON, INC.

DESCRIPTION: (E) 2X10 JOIST BTWN E&F W/ DRIFT 5 LOAD - AFTER SISTER

CODE REFERENCES

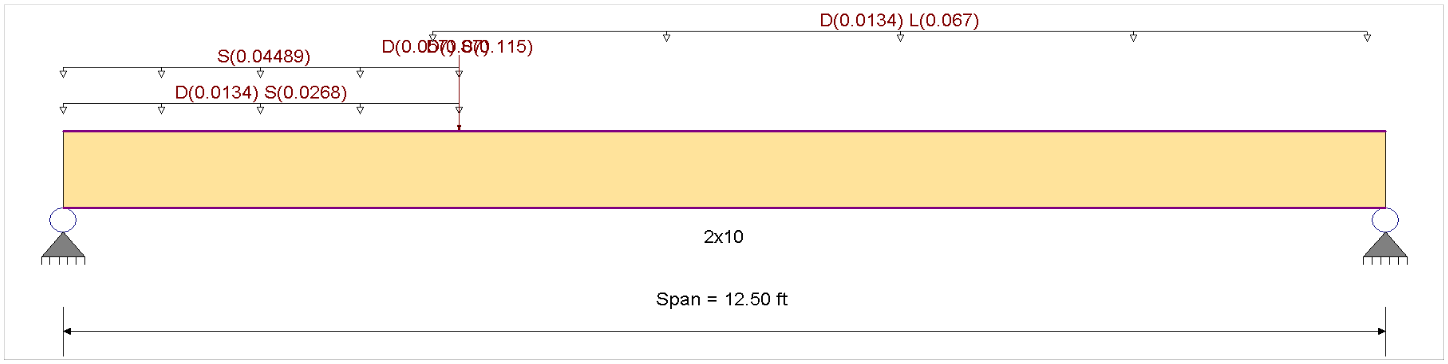
note to reviewer: material properties adjusted to historical values.

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,600.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1,600.0 psi	Ebend- xx	1,500.0 ksi
		Fc - Prll	1,050.0 psi	Eminbend - xx	470.0 ksi
Wood Species :	Hem-Fir	Fc - Perp	245.0 psi		
Wood Grade :	No.1	Fv	146.250 psi		
		Ft	825.0 psi	Density	28.720 pcf

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
- Uniform Load : D = 0.020, S = 0.040 ksf, Extent = 0.0 -->> 3.750 ft, Tributary Width = 0.670 ft, (TYPICAL ROOF LOAD)
- Uniform Load : S = 0.0670 ksf, Extent = 0.0 -->> 3.750 ft, Tributary Width = 0.670 ft, (DRIFT LOAD)
- Point Load : D = 0.070 k @ 3.750 ft, (WALL WEIGHT)
- Point Load : D = 0.0570, S = 0.1150 k @ 3.750 ft, (LOAD FROM ROOF ABOVE)
- Uniform Load : D = 0.020, L = 0.10 ksf, Extent = 3.50 -->> 12.330 ft, Tributary Width = 0.670 ft, (TYPICAL FLOOR LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.530**: 1	Maximum Shear Stress Ratio	=	**0.344** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
fb: Actual	=	932.94 psi	fv: Actual	=	57.92 psi
Fb: Allowable	=	1,760.00 psi	Fv: Allowable	=	168.19 psi
Load Combination		+D+L	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	6.159 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.205 in	Ratio =	731	>=240	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<240	n/a
Max Downward Total Deflection	0.337 in	Ratio =	445	>=180	Span: 1 : +D+0.750L+0.750S
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.215	0.148	0.90	1.100	1.00	1.00	1.00	1.00	1.00	0.61	340.59	1584.00	0.18	19.51	131.63
+D+L					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.530	0.344	1.00	1.100	1.00	1.00	1.00	1.00	1.00	1.66	932.94	1760.00	0.47	50.32	146.25
+D+S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.349	0.281	1.15	1.100	1.00	1.00	1.00	1.00	1.00	1.26	706.99	2024.00	0.44	47.26	168.19
+D+0.750L					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.354	0.225	1.25	1.100	1.00	1.00	1.00	1.00	1.00	1.39	779.02	2200.00	0.38	41.21	182.81

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Wood Beam

DESCRIPTION: (E) 2X10 JOIST BTWN E&F W/ DRIFT 5 LOAD - AFTER SISTER

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+0.750L+0.750S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	**1**	0.492	0.344	1.15	1.100	1.00	1.00	1.00	1.00	1.00	1.78	995.97	2024.00	0.54	57.92	168.19
+0.60D					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	**1**	0.073	0.050	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.36	204.35	2816.00	0.11	11.71	234.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S	1	0.3366	6.113		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.587	0.512
Overall MINimum	0.309	0.075
D Only	0.192	0.138
+D+L	0.409	0.512
+D+S	0.501	0.213
+D+0.750L	0.355	0.419
+D+0.750L+0.750S	0.587	0.475
+0.60D	0.115	0.083
L Only	0.217	0.375
S Only	0.309	0.075

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Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: snowden.ec6

LIC# : KW-06017698, Build:20.22.3.16 REID MIDDLETON, INC.

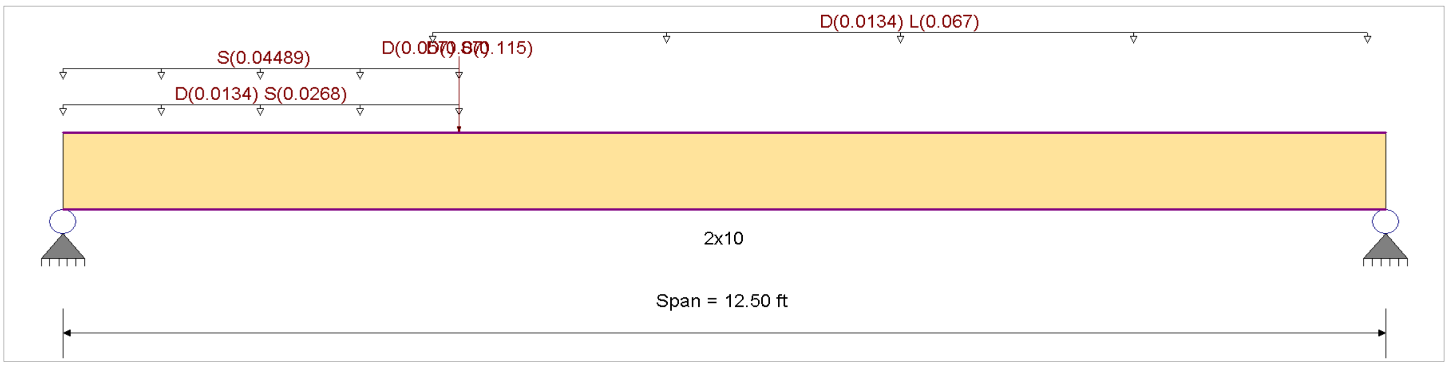
DESCRIPTION: (N) SISTERED 2X10 JOIST BTWN E&F W/ DRIFT 5 LOAD

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*		
Load Combination :	IBC 2018	Fb -	850.0 psi	Ebend- xx	1,300.0 ksi	
		Fc - Prll	1,300.0 psi	Eminbend - xx	470.0 ksi	
Wood Species :	Hem-Fir	Fc - Perp	405.0 psi			
Wood Grade :	No.2	Fv	150.0 psi			
		Ft	525.0 psi	Density	26.840 pcf	
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase		



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.020, S = 0.040 ksf, Extent = 0.0 -->> 3.750 ft, Tributary Width = 0.670 ft, (TYPICAL ROOF LOAD)
Uniform Load : S = 0.0670 ksf, Extent = 0.0 -->> 3.750 ft, Tributary Width = 0.670 ft, (DRIFT LOAD)
Point Load : D = 0.070 k @ 3.750 ft, (WALL WEIGHT)
Point Load : D = 0.0570, S = 0.1150 k @ 3.750 ft, (LOAD FROM ROOF ABOVE)
Uniform Load : D = 0.020, L = 0.10 ksf, Extent = 3.50 -->> 12.330 ft, Tributary Width = 0.670 ft, (TYPICAL FLOOR LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.866**: 1	Maximum Shear Stress Ratio	=	**0.335** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
fb: Actual	=	930.96 psi	fv: Actual	=	57.81 psi
Fb: Allowable	=	1,075.25 psi	Fv: Allowable	=	172.50 psi
Load Combination		+D+L	Load Combination		+D+0.750L+0.750S
Location of maximum on span	=	6.159 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.236 in	Ratio =	634 >=240	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <240	n/a
Max Downward Total Deflection	0.388 in	Ratio =	387 >=180	Span: 1 : +D+0.750L+0.750S
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.350	0.144	0.90	1.100	1.00	1.15	1.00	1.00	1.00	0.60	338.90	967.73	0.18	19.41	135.00
+D+L					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.866	0.335	1.00	1.100	1.00	1.15	1.00	1.00	1.00	1.66	930.96	1075.25	0.46	50.21	150.00
+D+S					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.570	0.273	1.15	1.100	1.00	1.15	1.00	1.00	1.00	1.26	705.32	1236.54	0.44	47.16	172.50
+D+0.750L					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.578	0.219	1.25	1.100	1.00	1.15	1.00	1.00	1.00	1.39	777.04	1344.06	0.38	41.10	187.50

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Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 5 APR 2022, 8:18AM

Wood Beam

Project File: snowden.ec6

LIC# : KW-06017698, Build:20.22.3.16 REID MIDDLETON, INC. (c) ENERCALC INC 1983-2022

DESCRIPTION: (N) SISTERED 2X10 JOIST BTWN E&F W/ DRIFT 5 LOAD

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+0.750L+0.750S					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	**1**	0.804	0.335	1.15	1.100	1.00	1.15	1.00	1.00	1.00	1.77	994.04	1236.54	0.53	57.81	172.50
+0.60D					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	**1**	0.118	0.049	1.60	1.100	1.00	1.15	1.00	1.00	1.00	0.36	203.34	1720.40	0.11	11.64	240.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S	1	0.3876	6.113		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.586	0.511
Overall MINimum	0.309	0.075
D Only	0.191	0.137
+D+L	0.408	0.511
+D+S	0.500	0.212
+D+0.750L	0.354	0.418
+D+0.750L+0.750S	0.586	0.474
+0.60D	0.115	0.082
L Only	0.217	0.375
S Only	0.309	0.075

max reaction to be transferred via screws

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Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

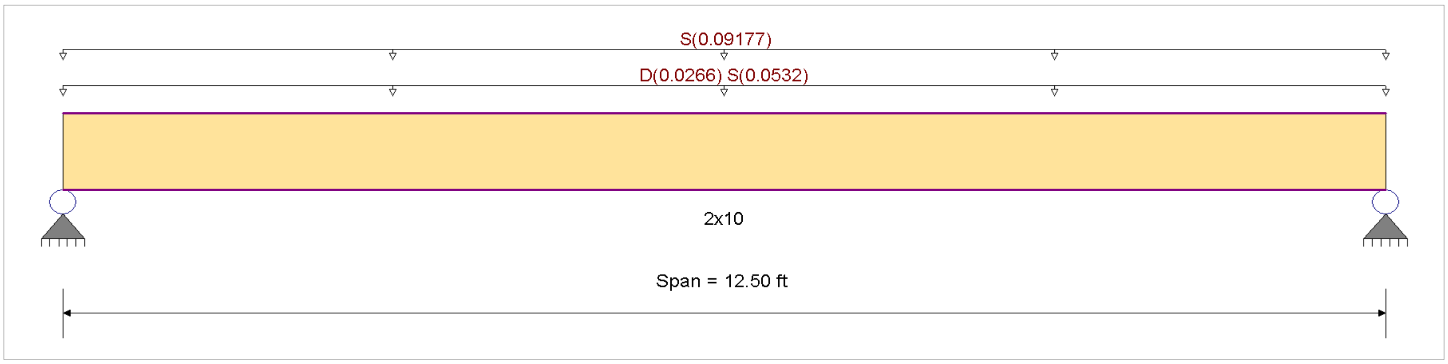
DESCRIPTION: (E) 2X10 JOIST BTWN E&F W/ DRIFT 2 LOAD (BETWEEN GRIDS 6 & 7)

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,600.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1,600.0 psi	Ebend- xx	1,500.0 ksi
		Fc - Prll	1,050.0 psi	Eminbend - xx	470.0 ksi
Wood Species :	Hem-Fir	Fc - Perp	245.0 psi		
Wood Grade :	No.1	Fv	146.250 psi		
		Ft	825.0 psi	Density	28.720 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, S = 0.040 ksf, Tributary Width = 1.330 ft, (TYPICAL ROOF LOAD)
Uniform Load : S = 0.0690 ksf, Tributary Width = 1.330 ft, (DRIFT LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.944** : 1	Maximum Shear Stress Ratio	=	**0.619** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
fb: Actual	=	1,910.20 psi	fv: Actual	=	104.04 psi
Fb: Allowable	=	2,024.00 psi	Fv: Allowable	=	168.19 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.250 ft	Location of maximum on span	=	11.770 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.540 in	Ratio =	277	>=240	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<240	n/a
Max Downward Total Deflection	0.649 in	Ratio =	231	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.203	0.133	0.90	1.100	1.00	1.00	1.00	1.00	1.00	0.57	321.77	1584.00	0.16	17.53	131.63
+D+S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.944	0.619	1.15	1.100	1.00	1.00	1.00	1.00	1.00	3.41	1,910.20	2024.00	0.96	104.04	168.19
+D+0.750S					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.748	0.490	1.15	1.100	1.00	1.00	1.00	1.00	1.00	2.70	1,513.09	2024.00	0.76	82.41	168.19
+0.60D					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.069	0.045	1.60	1.100	1.00	1.00	1.00	1.00	1.00	0.34	193.06	2816.00	0.10	10.52	234.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.6491	6.296		0.0000	0.000

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Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: Snowden.ec6

DESCRIPTION: (E) 2X10 JOIST BTWN E&F W/ DRIFT 2 LOAD (BETWEEN GRIDS 6 & 7)

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.090	1.090
Overall MINimum	0.906	0.906
D Only	0.184	0.184
+D+S	1.090	1.090
+D+0.750S	0.863	0.863
+0.60D	0.110	0.110
S Only	0.906	0.906

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Client	AMC	Sheet	of
Project	Snowden Mechanical Upgrades	Design by	GSB
		Date	3/29/2022
		Checked	
Project No.	40-21-105	Date	

worst case new joist reaction, see enercalc reaction between grids E&F

~ COMBINED SHEAR AND TENSION CONNECTION PER NDS 2018, CHAPTER 12 ~

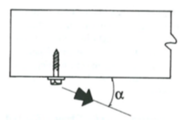
INPUT: **Sistered Joist Connection, TYP**

Method: **ASD**

Forces:

Tension/Withdrawal:	R_t =	0.0 lbs
Shear:	R_s =	586.0 lbs
Resultant Load:	R_R =	586.0 lbs
Direction:	α =	0.0°

0 ° is Shear Only
90° is Tension Only



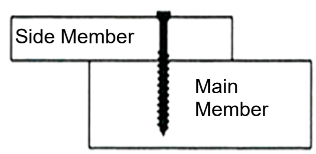
		NDS Ref.:
ASD		
Tension Load Duration:	2 Months/Snow Load	11.3.2
Shear Load Duration:	2 Months/Snow Load	11.3.2
ASD & LRFD		
Moisture Content[A]:	≤19% @ fab. & ≤19% in service	11.3.3
Temperature:	In Service Dry & Temp.≤100°F	11.3.4
Group Action:	Input C_g below	11.3.6
Geometry:	Input C_Δ below	12.5.1
End Grain:	NO End Grain	12.5.2
Diaphragm:	NOT a Diaphragm	12.5.3
Toe-nail:	NO Toe-Nailing	12.5.4
LRFD		
Format Conversion:	2.16/Φ	11.3.7
Φ Resistance:	0.65	11.3.8
Time Effect:	λ approximated below, see Appendix N.3.3	11.3.9

Fastener: Wood Screw

# of Fasteners, *n* =	4
Size:	#12
Length, *L* =	3.00 in
Nominal Diameter[B], *D* =	0.216 in
Head Diameter, D_H =	0.414 in

Wood:

Side Member Material[C]:	Wood
Main Member Material[C]:	Wood
Side Member Specific Gravity, G_s=	0.43
Main Member Specific Gravity, G_m=	0.43
Side Member Thickness, t_{ns}=	1.50 in
Main Member Thickness, t_m=	1.50 in
*Only used for toe-nailing:	2.00 in



Main Member Pen: 1.50 in *(6.9 D)*

Shear:

Main Member Shear Action Angle, θ_m=	90°	0° = // to grain
Side Member Shear Action Angle, θ_s=	90°	90° = ⊥ to grain

Connection OK 99%

Adjustment Factors: NDS Pg # **NDS Ref.:**

	Tension/Withdrawal	Shear		NDS Pg #	NDS Ref.
Load Duration Factor:	C_D = 1.15	C_D = 1.15	ASD only	pg. 11	11.3.2
Wet Service Factor:	C_M = 1.00	$C_M{}^1$ = 1.00	ASD & LRFD	pg. 61	10.3.3 & 11.5.4
Temperature Factor:	C_t = 1.00	C_t = 1.00	ASD & LRFD	pg. 66	11.3.4
Group Action Factor:	C_g = 1.00	C_g = 1.00	ASD & LRFD	pg. 68-72	11.3.6
Geometry Factor:	C_Δ = 1.00	C_Δ = 1.00	ASD & LRFD	pg. 89-91	12.5.1
End Grain Factor:	C_{eg} = 1.00	C_{eg} = 1.00	ASD & LRFD	pg. 91	12.5.2
Diaphragm Factor:	C_{di} = 1.00	C_{di} = 1.00	ASD & LRFD	pg. 91	12.5.3
Toe-nail Factor:	C_{tn} = 1.00	C_{tn} = 1.00	ASD & LRFD	pg. 91	12.5.4
Format Conversion Facto	K_F = 3.32	K_F = 3.32	LRFD only	pg. 187	11.3.7
Resistance Factor:	Φ_z = 0.65	Φ_z = 0.65	LRFD only	pg. 187	11.3.8
Time Effect Factor:	λ = 0.80	λ = 0.80	LRFD only	pg. 187	11.3.9

(4) #12x3" screws ok, #12x3" wood screws @ 6" oc ok by inspection. ensure (4) screws within first foot at either end of new joists.

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Client	AMC	Sheet	of
Project	Snowden Mechanical Upgrades	Design by	GSB
		Date	3/29/2022
		Checked	
Project No.	40-21-105	Date	

~ COMBINED SHEAR AND TENSION CONNECTION PER NDS 2018, CHAPTER 12 ~

INPUT: **Sistered Joist Connection, TYP**

Withdrawal Capacity: (NDS Ref.:)

Withdrawal Values, W		
Lag Screw	Wood Screw	Nail
N/A	114 lbs/in	N/A

NDS Ref.: Eq 12.2-1, 12.2-2, & 12.2-3

Length of Fastener Engaged/Thread Length, T			
Lag Screw		Wood Screw	Nail
T	E	T	T
N/A	N/A	2.00 in	N/A

NDS Ref.: Appendix L — 12.2

W= 114 lbs/in/fastener

W' = Cd*Cm*Ct*Cg*CΔ*Ceg*Ctn*W*n= 524 lbs/in

Unthreaded Shank Length: S = L-T= 1.00 in

Max Penetration allowed by Fastener: p_{max} = IF(Lag Screw, use T-E; else T)= 2.00 in

Penetration into Main Member: p = MIN(t_m, IF(t_{ns}<S, MIN(t_{sm}+t_m-S, p_{max}), S+p_{max}-t_{sm})= 1.50 in — 12.1.5.6

W' = W * p = 785 lbs

Pull-Through Values, W_H		
Lag Screw	Wood Screw	Nail
NA	172 lbs	N/A

12.2.5

W_H = 172 lbs/fastener

WH' = WH*Cd*Cm*Ct*n = 790 lbs

Controlling W' = 785 lbs

Shear Capacity: (NDS Ref.:)

p_m= 1.5 in — ℓ_s= 1.5 in

ℓ_m=	Lag Screw*	Wood Screw*	Nail
	N/A	1.158 in	N/A

*screw tapered tip not included when P<10D — 12.3.2

D_r = 0.171 in — F_{yb}= 80,000 psi

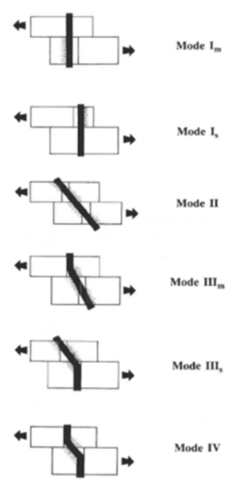
$F_{em\perp}$= 3,513 psi — $F_{em||}$= 3,513 psi — Tbl 12.3.2

$F_{es\perp}$= 3,513 psi — $F_{es||}$= 3,513 psi

R_e= 1.000 — R_t= 0.772

$F_{em\theta}$= 3,513 psi — $F_{es\theta}$= 3,513 psi

Yield Mode — Tbl 12.3.1A



I_m $$Z=\frac{D\ell_m F_{em}}{R_d}$$ R_d= 2.210 — Z= 315 lbs/fastener

I_s $$Z=\frac{D\ell_s F_{es}}{R_d}$$ R_d= 2.210 — Z= 408 lbs/fastener

II $$Z=\frac{k_1 D\ell_s F_{es}}{R_d}$$ k_1= 0.372, R_d= 2.210 — Z= 152 lbs/fastener

III_m $$Z=\frac{k_2 D\ell_m F_{em}}{(1+2R_e)R_d}$$ k_2= 1.235, R_d= 2.210 — Z= 130 lbs/fastener

III_s $$Z=\frac{k_3 D\ell_s F_{em}}{(2+R_e)R_d}$$ k_3= 1.143, R_d= 2.210 — Z= 155 lbs/fastener

IV $$Z=\frac{D^2}{R_d}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$$ R_d= 2.210 — Z= 128 lbs/fastener

Controlling Z= 128 lbs/fastener — 12.3

Z' = Cd*Cm*Ct*Cg*CΔ*Ceg*Ctn*Z*n= 589 lbs

Combined Capacity: (NDS Ref.:)

$Z_\alpha'_{Screws}$: $$Z_\alpha' = \frac{(W'p)Z'}{(W'p)\cos^2\alpha + Z'\sin^2\alpha}$$ **589 lbs** — 12.4-1

$Z_\alpha'_{Nails}$: $$Z_\alpha' = \frac{(W'p)Z'}{(W'p)\cos\alpha + Z'\sin\alpha}$$ N/A — 12.4-2

A. If Moisture Content is >19% at time of fabrication and ≤19% in service, 0.4 is a conservative factor. For a more precise factor see NDS 11.3.3.

B. Diameter of lag screws are for "reduced body diameter" lag screws. (Similar to tables 11J & 11k in NDS)

C. The adujsted values for $F_{e\perp}$ and $F_{e||}$ are not supported by NDS for diameters of fasteners >1/4" in plywood or OSB & therefore not recommended. F_e is assumed for all fastener sizes herein.

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Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

File: snowden.ec6

Lic. # : KW-06001667 REID MIDDLETON, INC.

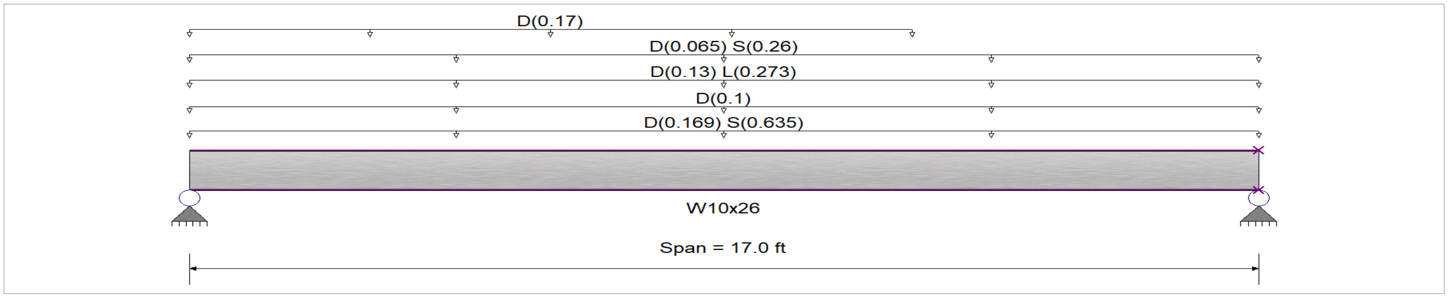
DESCRIPTION: Grid E Girder - W/ drift load (LONG SPAN)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	36.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.1690, S = 0.6350 k/ft, Tributary Width = 1.0 ft, (load from joists with drift)

Uniform Load : D = 0.10 k/ft, Tributary Width = 1.0 ft, (wall weight)

Uniform Load : D = 0.020, L = 0.0420 ksf, Tributary Width = 6.50 ft, (PH Floor)

Uniform Load : D = 0.010, S = 0.040 ksf, Tributary Width = 6.50 ft, (PH Roof)

Uniform Load : D = 0.170 k/ft, Extent = 0.0 -->> 11.50 ft, Tributary Width = 1.0 ft, (HKP load (3.5" thick, 1/2 load to WF))

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.976** : 1	Maximum Shear Stress Ratio =	**0.339** : 1
Section used for this span	**W10x26**	Section used for this span	**W10x26**
Ma : Applied	54.893 k-ft	Va : Applied	13.065 k
Mn / Omega : Allowable	56.228 k-ft	Vn/Omega : Allowable	38.563 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	8.403 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.404 in	Ratio =	504 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.685 in	Ratio =	298 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 17.00 ft	1	0.401	0.142	22.57		22.57	93.90	56.23	1.00	1.00	5.46	57.84	38.56
+D+L													
Dsgn. L = 17.00 ft	1	0.577	0.202	32.43		32.43	93.90	56.23	1.00	1.00	7.78	57.84	38.56
+D+S													
Dsgn. L = 17.00 ft	1	0.976	0.339	54.89		54.89	93.90	56.23	1.00	1.00	13.07	57.84	38.56
+D+0.750L													
Dsgn. L = 17.00 ft	1	0.533	0.187	29.96		29.96	93.90	56.23	1.00	1.00	7.20	57.84	38.56
+D+0.750L+0.750S													
Dsgn. L = 17.00 ft	1	0.964	0.335	54.21		54.21	93.90	56.23	1.00	1.00	12.90	57.84	38.56
+0.60D													
Dsgn. L = 17.00 ft	1	0.241	0.085	13.54		13.54	93.90	56.23	1.00	1.00	3.27	57.84	38.56

Steel Beam

DESCRIPTION: Grid E Girder - W/ drift load (LONG SPAN)

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.6850	8.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	13.065	12.433
Overall MINimum	2.321	2.321
D Only	5.458	4.825
+D+L	7.778	7.146
+D+S	13.065	12.433
+D+0.750L	7.198	6.566
+D+0.750L+0.750S	12.904	12.271
+0.60D	3.275	2.895
L Only	2.321	2.321
S Only	7.608	7.608

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Project Title: REVIEW OF EXISTING MEMBERS FOR MECH LOADS
Engineer:
Project ID:
Project Descr:

Wood Beam

File: snowden.ec6
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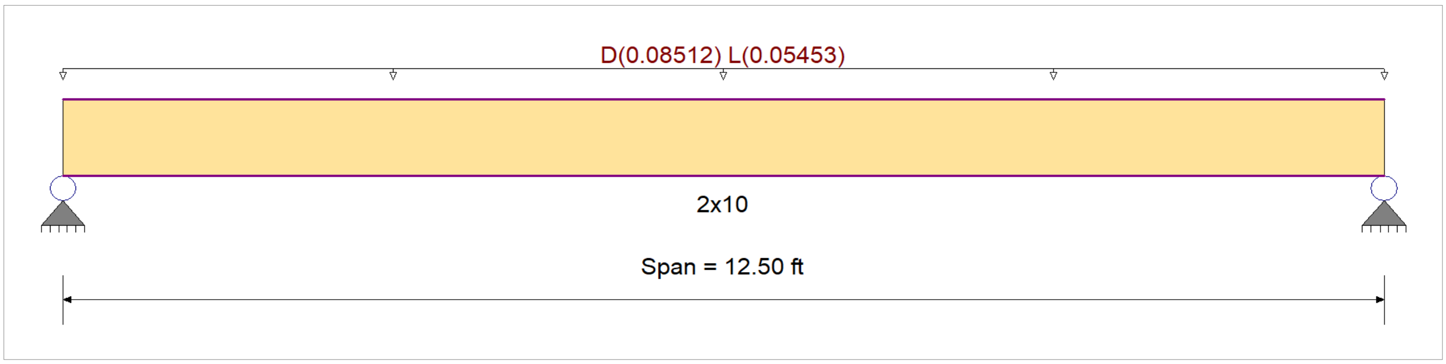
DESCRIPTION: (E) 2X10 JOIST BTWN E&F AT NEW AHU 2 (w/3.5" hkp)

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,600.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1,600.0 psi	Ebend- xx	1,500.0 ksi
		Fc - Prll	1,050.0 psi	Eminbend - xx	470.0 ksi
Wood Species	: Hem-Fir	Fc - Perp	245.0 psi		
Wood Grade	: No.1	Fv	146.250 psi		
		Ft	825.0 psi	Density	28.720 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0640, L = 0.0410 ksf, Tributary Width = 1.330 ft, (AHU load and DL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.771**: 1	Maximum Shear Stress Ratio	=	**0.581** : 1
Section used for this span		**2x10**	Section used for this span		**2x10**
fb: Actual	=	1,560.45 psi	fv: Actual	=	84.99 psi
Fb: Allowable	=	2,024.00 psi	Fv: Allowable	=	146.25 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	6.250 ft	Location of maximum on span	=	11.770 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.203 in	Ratio =	738 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.530 in	Ratio =	282 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.529	0.398	0.90	1.100	1.00	1.15	1.00	1.00	1.00	1.72	962.97	1821.60	0.49	52.45	131.63
+D+L					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.771	0.581	1.00	1.100	1.00	1.15	1.00	1.00	1.00	2.78	1,560.45	2024.00	0.79	84.99	146.25
+D+0.750L					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.558	0.420	1.25	1.100	1.00	1.15	1.00	1.00	1.00	2.52	1,411.08	2530.00	0.71	76.85	182.81
+0.60D					1.100	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.178	0.134	1.60	1.100	1.00	1.15	1.00	1.00	1.00	1.03	577.78	3238.40	0.29	31.47	234.00

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.890	0.890
Overall MINimum	0.341	0.341

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Wood Beam

DESCRIPTION: (E) 2X10 JOIST BTWN E&F AT NEW AHU 2 (w/3.5" hkp)

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
D Only	0.549	0.549
+D+L	0.890	0.890
+D+0.750L	0.805	0.805
+0.60D	0.330	0.330
L Only	0.341	0.341

Wood Beam

DESCRIPTION: (E) 2X12 JOIST Level 2 at NEW AHU 1 (w/3.5" hkp)

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

Wood Species : Hem-Fir
Wood Grade : No.1

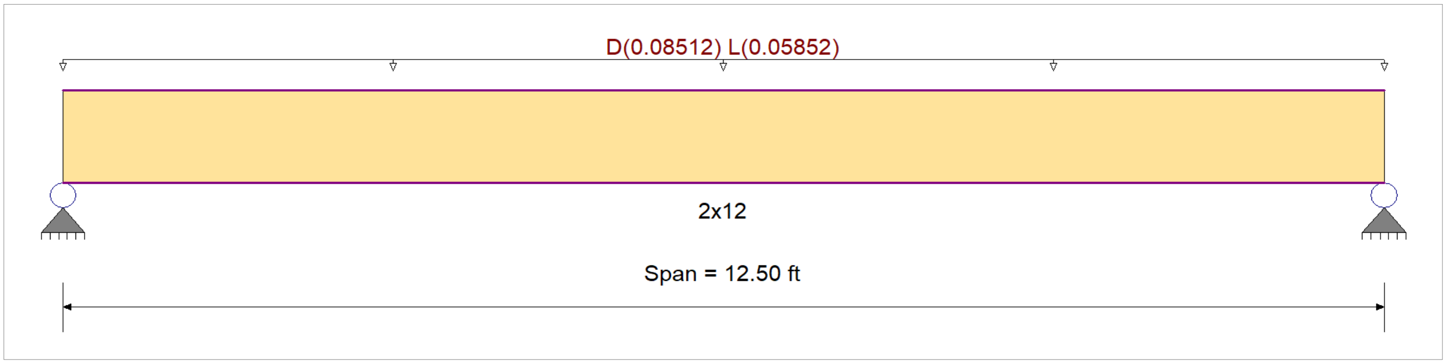
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1,600.0	psi
Fb -	1,600.0	psi
Fc - Prll	1,050.0	psi
Fc - Perp	245.0	psi
Fv	146.250	psi
Ft	825.0	psi

E : Modulus of Elasticity

Ebend- xx	1,500.0	ksi
Eminbend - xx	470.0	ksi
Density	28.720	pcf

Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0640, L = 0.0440 ksf, Tributary Width = 1.330 ft, (AHU load and DL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.592** : 1	Maximum Shear Stress Ratio	=	**0.477** : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
fb: Actual	=	1,088.93 psi	fv: Actual	=	69.75 psi
Fb: Allowable	=	1,840.00 psi	Fv: Allowable	=	146.25 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	6.250 ft	Location of maximum on span	=	11.588 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.121 in	Ratio =	1238 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.304 in	Ratio =	493 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.396	0.319	0.90	1.000	1.00	1.15	1.00	1.00	1.00	1.73	655.45	1656.00	0.47	41.98	131.63
+D+L					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.592	0.477	1.00	1.000	1.00	1.15	1.00	1.00	1.00	2.87	1,088.93	1840.00	0.78	69.75	146.25
+D+0.750L					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.426	0.344	1.25	1.000	1.00	1.15	1.00	1.00	1.00	2.59	980.56	2300.00	0.71	62.81	182.81
+0.60D					1.000	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.50 ft	1	0.134	0.108	1.60	1.000	1.00	1.15	1.00	1.00	1.00	1.04	393.27	2944.00	0.28	25.19	234.00

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.919	0.919
Overall MINimum	0.366	0.366

Wood Beam

DESCRIPTION: (E) 2X12 JOIST Level 2 at NEW AHU 1 (w/3.5" hkp)

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
D Only	0.553	0.553
+D+L	0.919	0.919
+D+0.750L	0.827	0.827
+0.60D	0.332	0.332
L Only	0.366	0.366

Concrete Beam

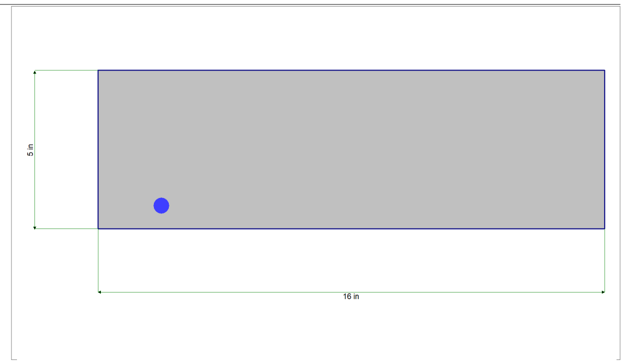
DESCRIPTION: existing slab in mech room, level 3 w/AHU & 3.5" hkp

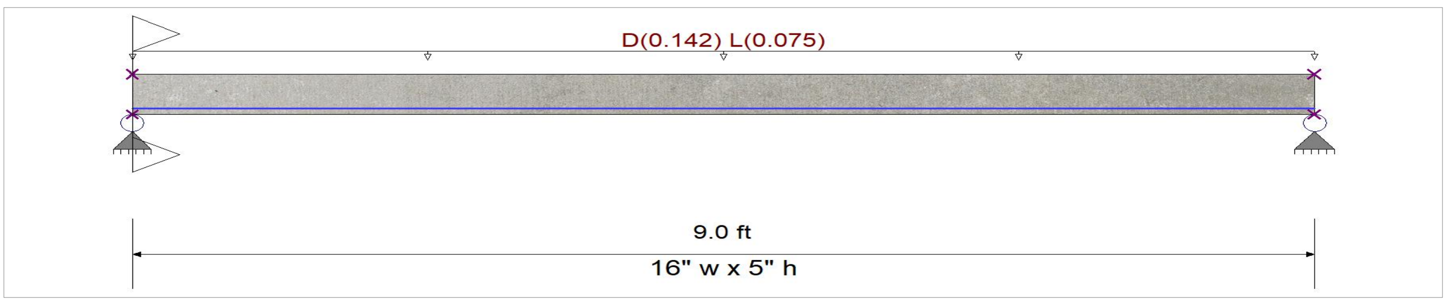
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

f'c	=	3.750 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	459.279 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 =		1.0





Cross Section & Reinforcing Details

Rectangular Section, Width = 16.0 in, Height = 5.0 in
Span #1 Reinforcing....
1-#4 at 0.750 in from Bottom, from 0.0 to 9.0 ft in this span

Load for Span Number 1
Uniform Load : D = 0.1420, L = 0.0750 ksf, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.791** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	2.940	k-ft
Mn * Phi : Allowable	3.719	k-ft
Location of maximum on span	4.508	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection				
Max Downward Transient Deflection	0.021 in	Ratio =	5077	>=360.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0
Max Downward Total Deflection	0.062 in	Ratio =	1754	>=180.
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	0.977	0.977
Overall MINimum	0.338	0.338
+D+H	0.639	0.639
+D+L+H	0.977	0.977
+D+Lr+H	0.639	0.639
+D+S+H	0.639	0.639
+D+0.750Lr+0.750L+H	0.892	0.892
+D+0.750L+0.750S+H	0.892	0.892
+D+0.60W+H	0.639	0.639
+D+0.750Lr+0.750L+0.450W+H	0.892	0.892
+D+0.750L+0.750S+0.450W+H	0.892	0.892
+0.60D+0.60W+0.60H	0.383	0.383
+D+0.70E+0.60H	0.639	0.639
+D+0.750L+0.750S+0.5250E+H	0.892	0.892

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Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

DESCRIPTION: existing slab in mech room, level 3 w/AHU & 3.5" hkp

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+0.60D+0.70E+H	0.383	0.383
D Only	0.639	0.639
L Only	0.338	0.338
H Only		

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
+1.20D+1.60L+0.50S+1.60H	1	0.00	4.25	1.31	1.31	0.00	1.00	6.31	Vu < PhiVc/2	lot Reqd 9.6.	6.3	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.10	4.25	1.28	1.28	0.13	1.00	6.31	Vu < PhiVc/2	lot Reqd 9.6.	6.3	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.20	4.25	1.25	1.25	0.25	1.00	6.31	Vu < PhiVc/2	lot Reqd 9.6.	6.3	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.30	4.25	1.22	1.22	0.37	1.00	6.31	Vu < PhiVc/2	lot Reqd 9.6.	6.3	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.39	4.25	1.19	1.19	0.49	0.86	6.26	Vu < PhiVc/2	lot Reqd 9.6.	6.3	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.49	4.25	1.16	1.16	0.61	0.68	6.19	Vu < PhiVc/2	lot Reqd 9.6.	6.2	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.59	4.25	1.14	1.14	0.72	0.56	6.14	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.69	4.25	1.11	1.11	0.83	0.47	6.11	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.79	4.25	1.08	1.08	0.94	0.41	6.09	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.89	4.25	1.05	1.05	1.04	0.36	6.07	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.98	4.25	1.02	1.02	1.14	0.32	6.05	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.08	4.25	0.99	0.99	1.24	0.28	6.04	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.18	4.25	0.96	0.96	1.34	0.25	6.03	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.28	4.25	0.94	0.94	1.43	0.23	6.02	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.38	4.25	0.91	0.91	1.52	0.21	6.01	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.48	4.25	0.88	0.88	1.61	0.19	6.01	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.57	4.25	0.85	0.85	1.70	0.18	6.00	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.67	4.25	0.82	0.82	1.78	0.16	6.00	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.77	4.25	0.79	0.79	1.86	0.15	5.99	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.87	4.25	0.76	0.76	1.94	0.14	5.99	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.97	4.25	0.74	0.74	2.01	0.13	5.98	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.07	4.25	0.71	0.71	2.08	0.12	5.98	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.16	4.25	0.68	0.68	2.15	0.11	5.98	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.26	4.25	0.65	0.65	2.21	0.10	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.36	4.25	0.62	0.62	2.28	0.10	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.46	4.25	0.59	0.59	2.34	0.09	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.56	4.25	0.56	0.56	2.39	0.08	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.66	4.25	0.54	0.54	2.45	0.08	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.75	4.25	0.51	0.51	2.50	0.07	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.85	4.25	0.48	0.48	2.55	0.07	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.95	4.25	0.45	0.45	2.59	0.06	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.05	4.25	0.42	0.42	2.63	0.06	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.15	4.25	0.39	0.39	2.67	0.05	5.95	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.25	4.25	0.36	0.36	2.71	0.05	5.95	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.34	4.25	0.34	0.34	2.75	0.04	5.95	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.44	4.25	0.31	0.31	2.78	0.04	5.95	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.54	4.25	0.28	0.28	2.81	0.04	5.95	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.64	4.25	0.25	0.25	2.83	0.03	5.95	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.74	4.25	0.22	0.22	2.86	0.03	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.84	4.25	0.19	0.19	2.88	0.02	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.93	4.25	0.16	0.16	2.89	0.02	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.03	4.25	0.14	0.14	2.91	0.02	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.13	4.25	0.11	0.11	2.92	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.23	4.25	0.08	0.08	2.93	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.33	4.25	0.05	0.05	2.94	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.43	4.25	0.02	0.02	2.94	0.00	5.93	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.52	4.25	-0.01	0.01	2.94	0.00	5.93	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.62	4.25	-0.04	0.04	2.94	0.00	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0

Concrete Beam

DESCRIPTION: existing slab in mech room, level 3 w/AHU & 3.5" hkp

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu Actual	(k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Suggest
+1.20D+1.60L+0.50S+1.60H	1	4.72	4.25	-0.06	0.06	2.93	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.82	4.25	-0.09	0.09	2.93	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.92	4.25	-0.12	0.12	2.91	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.02	4.25	-0.15	0.15	2.90	0.02	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.11	4.25	-0.18	0.18	2.89	0.02	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.21	4.25	-0.21	0.21	2.87	0.03	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.31	4.25	-0.24	0.24	2.84	0.03	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.41	4.25	-0.26	0.26	2.82	0.03	5.95	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.51	4.25	-0.29	0.29	2.79	0.04	5.95	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.61	4.25	-0.32	0.32	2.76	0.04	5.95	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.70	4.25	-0.35	0.35	2.73	0.05	5.95	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.80	4.25	-0.38	0.38	2.69	0.05	5.95	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.90	4.25	-0.41	0.41	2.66	0.05	5.95	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.00	4.25	-0.44	0.44	2.61	0.06	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.10	4.25	-0.46	0.46	2.57	0.06	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.20	4.25	-0.49	0.49	2.52	0.07	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.30	4.25	-0.52	0.52	2.47	0.07	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.39	4.25	-0.55	0.55	2.42	0.08	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.49	4.25	-0.58	0.58	2.36	0.09	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.59	4.25	-0.61	0.61	2.31	0.09	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.69	4.25	-0.64	0.64	2.24	0.10	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.79	4.25	-0.66	0.66	2.18	0.11	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.89	4.25	-0.69	0.69	2.11	0.12	5.98	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.98	4.25	-0.72	0.72	2.04	0.12	5.98	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.08	4.25	-0.75	0.75	1.97	0.13	5.98	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.18	4.25	-0.78	0.78	1.90	0.15	5.99	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.28	4.25	-0.81	0.81	1.82	0.16	5.99	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.38	4.25	-0.84	0.84	1.74	0.17	6.00	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.48	4.25	-0.86	0.86	1.65	0.18	6.00	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.57	4.25	-0.89	0.89	1.57	0.20	6.01	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.67	4.25	-0.92	0.92	1.48	0.22	6.02	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.77	4.25	-0.95	0.95	1.39	0.24	6.02	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.87	4.25	-0.98	0.98	1.29	0.27	6.03	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.97	4.25	-1.01	1.01	1.19	0.30	6.05	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.07	4.25	-1.04	1.04	1.09	0.34	6.06	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.16	4.25	-1.06	1.06	0.99	0.38	6.08	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.26	4.25	-1.09	1.09	0.89	0.44	6.10	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.36	4.25	-1.12	1.12	0.78	0.51	6.13	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.46	4.25	-1.15	1.15	0.66	0.61	6.16	Vu < PhiVc/2	lot Reqd 9.6.	6.2	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.56	4.25	-1.18	1.18	0.55	0.76	6.22	Vu < PhiVc/2	lot Reqd 9.6.	6.2	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.66	4.25	-1.21	1.21	0.43	0.99	6.30	Vu < PhiVc/2	lot Reqd 9.6.	6.3	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.75	4.25	-1.24	1.24	0.31	1.00	6.31	Vu < PhiVc/2	lot Reqd 9.6.	6.3	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.85	4.25	-1.26	1.26	0.19	1.00	6.31	Vu < PhiVc/2	lot Reqd 9.6.	6.3	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.95	4.25	-1.29	1.29	0.06	1.00	6.31	Vu < PhiVc/2	lot Reqd 9.6.	6.3	0.0	0.0

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	9.000	2.94	3.72	0.79
+1.40D+1.60H					
Span # 1	1	9.000	2.01	3.72	0.54
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	9.000	2.94	3.72	0.79
+1.20D+1.60L+0.50S+1.60H					
Span # 1	1	9.000	2.94	3.72	0.79

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Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

File: snowden.ec6

Lic. # : KW-06001667
REID MIDDLETON, INC.

DESCRIPTION: existing slab in mech room, level 3 w/AHU & 3.5" hkp

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
+1.20D+1.60Lr+L+1.60H					
Span # 1	1	9.000	2.48	3.72	0.67
+1.20D+1.60Lr+0.50W+1.60H					
Span # 1	1	9.000	1.73	3.72	0.46
+1.20D+L+1.60S+1.60H					
Span # 1	1	9.000	2.48	3.72	0.67
+1.20D+1.60S+0.50W+1.60H					
Span # 1	1	9.000	1.73	3.72	0.46
+1.20D+0.50Lr+L+W+1.60H					
Span # 1	1	9.000	2.48	3.72	0.67
+1.20D+L+0.50S+W+1.60H					
Span # 1	1	9.000	2.48	3.72	0.67
+0.90D+W+1.60H					
Span # 1	1	9.000	1.29	3.72	0.35
+1.20D+L+0.20S+E+1.60H					
Span # 1	1	9.000	2.48	3.72	0.67
+0.90D+E+0.90H					
Span # 1	1	9.000	1.29	3.72	0.35

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L+H	1	0.0615	4.500		0.0000	0.000

ReidMiddleton
4300 B STREET
SUITE 302
ANCHORAGE, AK 99503
PH: (907)562-3439
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Concrete Beam

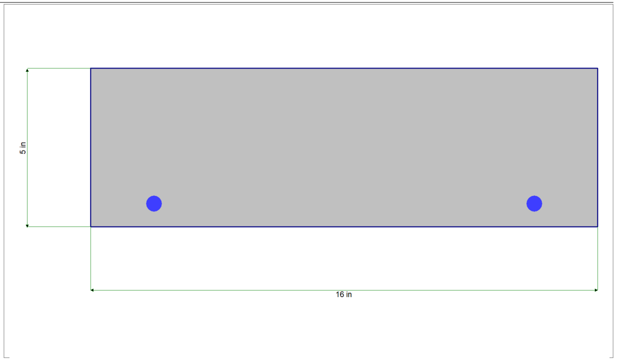
DESCRIPTION: existing slab, level 3 w/Boiler 1

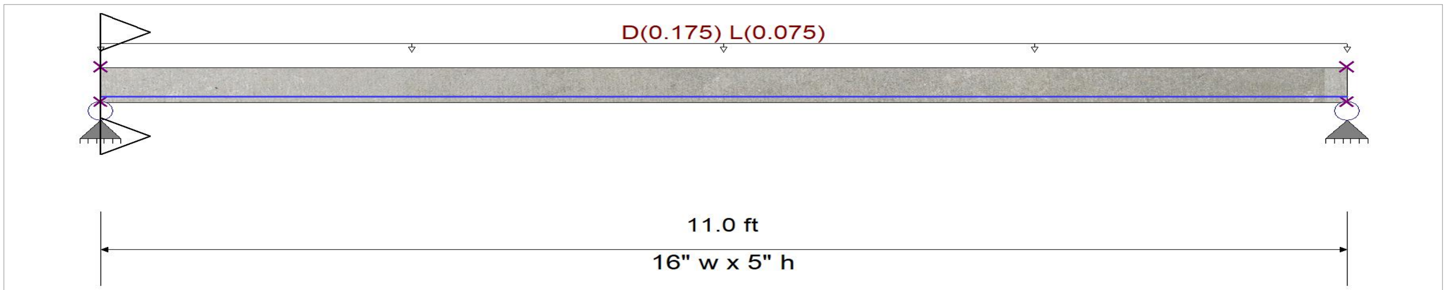
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

f'c	=	3.750 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	459.279 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 =		1.0





Cross Section & Reinforcing Details

Rectangular Section, Width = 16.0 in, Height = 5.0 in
Span #1 Reinforcing....
2-#4 at 0.750 in from Bottom, from 0.0 to 11.0 ft in this span

Load for Span Number 1
Uniform Load : D = 0.1750, L = 0.0750 ksf, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.691** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	4.991	k-ft
Mn * Phi : Allowable	7.226	k-ft
Location of maximum on span	5.510	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection				
Max Downward Transient Deflection	0.047 in	Ratio =	2780	>=360.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0
Max Downward Total Deflection	0.312 in	Ratio =	422	>=180.
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	1.375	1.375
Overall MINimum	0.412	0.413
+D+H	0.962	0.963
+D+L+H	1.375	1.375
+D+Lr+H	0.962	0.963
+D+S+H	0.962	0.963
+D+0.750Lr+0.750L+H	1.272	1.272
+D+0.750L+0.750S+H	1.272	1.272
+D+0.60W+H	0.962	0.963
+D+0.750Lr+0.750L+0.450W+H	1.272	1.272
+D+0.750L+0.750S+0.450W+H	1.272	1.272
+0.60D+0.60W+0.60H	0.577	0.578
+D+0.70E+0.60H	0.962	0.963
+D+0.750L+0.750S+0.5250E+H	1.272	1.272

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Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

File: snowden.ec6

Lic. # : KW-06001667
REID MIDDLETON, INC.

DESCRIPTION: existing slab, level 3 w/Boiler 1

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+0.60D+0.70E+H	0.577	0.578
D Only	0.962	0.963
L Only	0.412	0.413
H Only		

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment		Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
+1.20D+1.60L+0.50S+1.60H	1	0.00	4.25	1.81	1.81	0.00	1.00	6.68	Vu < PhiVc/2	lot Reqd 9.6.		6.7	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.12	4.25	1.78	1.78	0.22	1.00	6.68	Vu < PhiVc/2	lot Reqd 9.6.		6.7	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.24	4.25	1.74	1.74	0.43	1.00	6.68	Vu < PhiVc/2	lot Reqd 9.6.		6.7	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.36	4.25	1.70	1.70	0.63	0.95	6.65	Vu < PhiVc/2	lot Reqd 9.6.		6.6	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.48	4.25	1.66	1.66	0.83	0.70	6.46	Vu < PhiVc/2	lot Reqd 9.6.		6.5	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.60	4.25	1.62	1.62	1.03	0.56	6.35	Vu < PhiVc/2	lot Reqd 9.6.		6.4	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.72	4.25	1.58	1.58	1.22	0.46	6.28	Vu < PhiVc/2	lot Reqd 9.6.		6.3	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.84	4.25	1.54	1.54	1.41	0.39	6.22	Vu < PhiVc/2	lot Reqd 9.6.		6.2	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	0.96	4.25	1.50	1.50	1.59	0.33	6.18	Vu < PhiVc/2	lot Reqd 9.6.		6.2	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.08	4.25	1.46	1.46	1.77	0.29	6.15	Vu < PhiVc/2	lot Reqd 9.6.		6.2	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.20	4.25	1.42	1.42	1.94	0.26	6.13	Vu < PhiVc/2	lot Reqd 9.6.		6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.32	4.25	1.38	1.38	2.11	0.23	6.11	Vu < PhiVc/2	lot Reqd 9.6.		6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.44	4.25	1.34	1.34	2.27	0.21	6.09	Vu < PhiVc/2	lot Reqd 9.6.		6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.56	4.25	1.30	1.30	2.43	0.19	6.08	Vu < PhiVc/2	lot Reqd 9.6.		6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.68	4.25	1.26	1.26	2.59	0.17	6.06	Vu < PhiVc/2	lot Reqd 9.6.		6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.80	4.25	1.22	1.22	2.74	0.16	6.05	Vu < PhiVc/2	lot Reqd 9.6.		6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	1.92	4.25	1.18	1.18	2.88	0.15	6.04	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.04	4.25	1.14	1.14	3.02	0.13	6.03	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.16	4.25	1.10	1.10	3.15	0.12	6.03	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.28	4.25	1.06	1.06	3.28	0.11	6.02	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.40	4.25	1.02	1.02	3.41	0.11	6.01	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.52	4.25	0.98	0.98	3.53	0.10	6.01	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.64	4.25	0.94	0.94	3.65	0.09	6.00	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.77	4.25	0.90	0.90	3.76	0.09	6.00	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	2.89	4.25	0.86	0.86	3.86	0.08	5.99	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.01	4.25	0.82	0.82	3.96	0.07	5.99	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.13	4.25	0.78	0.78	4.06	0.07	5.99	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.25	4.25	0.74	0.74	4.15	0.06	5.98	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.37	4.25	0.70	0.70	4.24	0.06	5.98	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.49	4.25	0.66	0.66	4.32	0.05	5.97	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.61	4.25	0.62	0.62	4.40	0.05	5.97	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.73	4.25	0.59	0.59	4.47	0.05	5.97	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.85	4.25	0.55	0.55	4.54	0.04	5.97	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	3.97	4.25	0.51	0.51	4.60	0.04	5.96	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.09	4.25	0.47	0.47	4.66	0.04	5.96	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.21	4.25	0.43	0.43	4.72	0.03	5.96	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.33	4.25	0.39	0.39	4.76	0.03	5.96	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.45	4.25	0.35	0.35	4.81	0.03	5.95	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.57	4.25	0.31	0.31	4.85	0.02	5.95	Vu < PhiVc/2	lot Reqd 9.6.		6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.69	4.25	0.27	0.27	4.88	0.02	5.95	Vu < PhiVc/2	lot Reqd 9.6.		5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.81	4.25	0.23	0.23	4.91	0.02	5.95	Vu < PhiVc/2	lot Reqd 9.6.		5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	4.93	4.25	0.19	0.19	4.94	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.		5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.05	4.25	0.15	0.15	4.96	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.		5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.17	4.25	0.11	0.11	4.97	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.		5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.29	4.25	0.07	0.07	4.98	0.00	5.94	Vu < PhiVc/2	lot Reqd 9.6.		5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.41	4.25	0.03	0.03	4.99	0.00	5.94	Vu < PhiVc/2	lot Reqd 9.6.		5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.53	4.25	-0.01	0.01	4.99	0.00	5.93	Vu < PhiVc/2	lot Reqd 9.6.		5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.65	4.25	-0.05	0.05	4.99	0.00	5.94	Vu < PhiVc/2	lot Reqd 9.6.		5.9	0.0	0.0

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Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

File: snowden.ec6

Lic. # : KW-06001667
REID MIDDLETON, INC.

DESCRIPTION: existing slab, level 3 w/Boiler 1

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Spacing (in) Suggest
+1.20D+1.60L+0.50S+1.60H	1	5.77	4.25	-0.09	0.09	4.98	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	5.89	4.25	-0.13	0.13	4.97	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.01	4.25	-0.17	0.17	4.95	0.01	5.94	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.13	4.25	-0.21	0.21	4.93	0.01	5.95	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.25	4.25	-0.25	0.25	4.90	0.02	5.95	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.37	4.25	-0.29	0.29	4.87	0.02	5.95	Vu < PhiVc/2	lot Reqd 9.6.	5.9	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.49	4.25	-0.33	0.33	4.83	0.02	5.95	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.61	4.25	-0.37	0.37	4.79	0.03	5.95	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.73	4.25	-0.41	0.41	4.74	0.03	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.85	4.25	-0.45	0.45	4.69	0.03	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	6.97	4.25	-0.49	0.49	4.63	0.04	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.09	4.25	-0.53	0.53	4.57	0.04	5.96	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.21	4.25	-0.57	0.57	4.51	0.04	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.33	4.25	-0.61	0.61	4.44	0.05	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.45	4.25	-0.64	0.64	4.36	0.05	5.97	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.57	4.25	-0.68	0.68	4.28	0.06	5.98	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.69	4.25	-0.72	0.72	4.20	0.06	5.98	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.81	4.25	-0.76	0.76	4.11	0.07	5.98	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	7.93	4.25	-0.80	0.80	4.01	0.07	5.99	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.05	4.25	-0.84	0.84	3.91	0.08	5.99	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.17	4.25	-0.88	0.88	3.81	0.08	6.00	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.30	4.25	-0.92	0.92	3.70	0.09	6.00	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.42	4.25	-0.96	0.96	3.59	0.09	6.01	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.54	4.25	-1.00	1.00	3.47	0.10	6.01	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.66	4.25	-1.04	1.04	3.35	0.11	6.02	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.78	4.25	-1.08	1.08	3.22	0.12	6.02	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	8.90	4.25	-1.12	1.12	3.09	0.13	6.03	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	9.02	4.25	-1.16	1.16	2.95	0.14	6.04	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	9.14	4.25	-1.20	1.20	2.81	0.15	6.05	Vu < PhiVc/2	lot Reqd 9.6.	6.0	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	9.26	4.25	-1.24	1.24	2.66	0.16	6.06	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	9.38	4.25	-1.28	1.28	2.51	0.18	6.07	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	9.50	4.25	-1.32	1.32	2.35	0.20	6.08	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	9.62	4.25	-1.36	1.36	2.19	0.22	6.10	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	9.74	4.25	-1.40	1.40	2.03	0.24	6.12	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	9.86	4.25	-1.44	1.44	1.86	0.27	6.14	Vu < PhiVc/2	lot Reqd 9.6.	6.1	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	9.98	4.25	-1.48	1.48	1.68	0.31	6.17	Vu < PhiVc/2	lot Reqd 9.6.	6.2	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	10.10	4.25	-1.52	1.52	1.50	0.36	6.20	Vu < PhiVc/2	lot Reqd 9.6.	6.2	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	10.22	4.25	-1.56	1.56	1.32	0.42	6.25	Vu < PhiVc/2	lot Reqd 9.6.	6.2	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	10.34	4.25	-1.60	1.60	1.13	0.50	6.31	Vu < PhiVc/2	lot Reqd 9.6.	6.3	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	10.46	4.25	-1.64	1.64	0.93	0.62	6.40	Vu < PhiVc/2	lot Reqd 9.6.	6.4	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	10.58	4.25	-1.68	1.68	0.73	0.81	6.54	Vu < PhiVc/2	lot Reqd 9.6.	6.5	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	10.70	4.25	-1.72	1.72	0.53	1.00	6.68	Vu < PhiVc/2	lot Reqd 9.6.	6.7	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	10.82	4.25	-1.76	1.76	0.32	1.00	6.68	Vu < PhiVc/2	lot Reqd 9.6.	6.7	0.0	0.0
+1.20D+1.60L+0.50S+1.60H	1	10.94	4.25	-1.80	1.80	0.11	1.00	6.68	Vu < PhiVc/2	lot Reqd 9.6.	6.7	0.0	0.0

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	11.000	4.99	7.23	0.69
+1.40D+1.60H					
Span # 1	1	11.000	3.71	7.23	0.51
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	11.000	4.99	7.23	0.69
+1.20D+1.60L+0.50S+1.60H					
Span # 1	1	11.000	4.99	7.23	0.69

Concrete Beam

DESCRIPTION: existing slab, level 3 w/Boiler 1

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
+1.20D+1.60Lr+L+1.60H					
Span # 1	1	11.000	4.31	7.23	0.60
+1.20D+1.60Lr+0.50W+1.60H					
Span # 1	1	11.000	3.18	7.23	0.44
+1.20D+L+1.60S+1.60H					
Span # 1	1	11.000	4.31	7.23	0.60
+1.20D+1.60S+0.50W+1.60H					
Span # 1	1	11.000	3.18	7.23	0.44
+1.20D+0.50Lr+L+W+1.60H					
Span # 1	1	11.000	4.31	7.23	0.60
+1.20D+L+0.50S+W+1.60H					
Span # 1	1	11.000	4.31	7.23	0.60
+0.90D+W+1.60H					
Span # 1	1	11.000	2.38	7.23	0.33
+1.20D+L+0.20S+E+1.60H					
Span # 1	1	11.000	4.31	7.23	0.60
+0.90D+E+0.90H					
Span # 1	1	11.000	2.38	7.23	0.33

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L+H	1	0.3122	5.500		0.0000	0.000

MECHANICAL ANCHORAGE

ReidMiddleton
4300 B Street, Suite 302
Anchorage, Alaska 99503
Phone: 907 562-3439
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Client	AMC	Page	1 of
Project	Snowden	Design by	EH
Project No.	40-21-105	Date	4/12/2022

Seismic Anchors for Mechanical & Electrical Equipment

	ID	Equipment Description	Loc	Elev (z)	Roof Ht (h)	Mount Type	Length (in)	Width (in)	Height (in)	CG HT	CG W	Weight (lbs)	Notes:
1	EF-1	exhaust fan 1	fanroom L3	24.00	36.00	Suspended	26	29	26	13	13	140	
2	RF-2	Relief fan 1	fanroom L3	24.00	36.00	Suspended	26	26	30	15	13	200	
3	SCF-1	small cabinet fan	fanroom L3	24.00	36.00	Suspended	68	50	18	9	25	500	(6) anchors provided ea side

Notes

1. Equipment listed limited to (per exceptions of ASCE7-16 Section 13.1.4 and MOA requirements):
   a. Equipment mounted at less than 4' above finished floor and weighing more than 400 lbs in service
   b. Equipment mounted at higher than 4' above finished floor, exposed to view, weighing more than 100 lbs in service
   c. Equipment mounted at higher than 4' above finished floor, NOT exposed to view, weighing more than 50 lbs in service
   d. Equipment and Distribution systems not covered by standard SMACNA guidelines and typical details

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Client	AMC	Page	2 of
Project	Snowden	Design by	EH
Project No.	40-21-105	Date	4/12/2022

S_{DS} = 1.20

ID	Wp (lb)	Mount Type	a_p	R_p	I_p	z	Roof Height (ft)	Fp_{MAX}	Fp_{MIN}	Fp_{CALC}	**$Fp_{control}$**
EF-1	140	Suspended	2.5	6.0	1.0	24.00	36.00	269	50	65	**65**
RF-2	200	Suspended	2.5	6.0	1.0	24.00	36.00	384	72	93	**93**
SCF-1	500	Suspended	2.5	6	1.0	24.00	36.00	960	180	233	**233**

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Client	AMC	Page	1 of
Project	Snowden	Design by	EH
Project No.	40-21-105	Date	4/12/2022

Suspended Equipment

Assumed Brace Angle: 45 0.79 radians
No. Braces per Direction: 2

ID	Fp ASD (lb)	Weight (lb)	# Vertical Conn to Structure	T @ Vertical Connection (lb)	Axial Force @ Brace (lb)	T @ Brace Conn (lb)	V @ Brace Conn (lb)
EF-1	46	140	4	35	32	23	23
RF-2	65	200	4	50	46	33	33
SCF-1	163	500	12	42	115	82	82

Max Loads for Design			
T @ Vertical Connection (lb)	**Axial Force @ Brace (lb)**	**T @ Brace Conn (lb)**	**V @ Brace Conn (lb)**
50	115	82	82

Check Axial Force Per Brace

3/16" aircraft cable, breaking strength = 3700 lbs
FOS = 5
capacity = 740 lbs
DCR = 0.16

Vertical rods okay by inspection

Check connection of Brace to Structure (SCF governs)

Tension (EQ) = 82 lb total T (ASD) = 123.3 lb
Tension (DL) = 42 lb
Shear (EQ) = 82 lb

AISI table IV-10a, nominal pullout
574 lb, nominal
191.3 lb, allowable

AISI table IV-9a, shear
1090 lb, nominal
363.3 lb, allowable

For (1) 1/4" screw: T allow = 191.3 lb
V allow = 363.3 lb

DCR = (T/Tallow)+(V/Vallow) = ***0.87***

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Client AMC
Project Snowden Bldg Mech Upgrades
Project No. 40-21-105

Design by EH
Date 4/12/2022

Seismic Anchors

	ID	Equipment Description	Loc	Elev (z)	Roof Ht (h)	Mount Type	Depth (in)	Width (in)	Height (in)	CG HT	CG W	Weight (lbs)	Notes:
1	BLR-1	Boiler 1	Boiler Rm L3	24.0	36.0	Floor	33.3	60.8	83.0	41.5	16.7	1900	Wet wgt, 5.5" hkp on (E) conc slab
2	WH-1	Water Heater 1	Boiler Rm L3	24.0	36.0	Floor	23.5	23.5	45.2	22.6	11.8	450	Tank, diam = 23.5", on (E) conc slab
3	ET-1	Exp Tank 1	Boiler Rm L3	24.0	36.0	Floor	24.0	24.0	47.0	23.5	12.0	800	Tank, diam = 24", 3.5" hkp on (E) conc slab
4	AHU-1	AHU-1	Fan Rm L2	11.0	36.0	Floor	69.0	168.0	51.0	25.5	34.5	3500	3.5" hkp on (E) wood
5	AHU-2	AHU-2	New Fan Rm L3	24.0	36.0	Floor	81.0	174.0	51.0	25.5	40.5	4000	3.5" hkp on (E) wood
6	AHU-3	AHU-3	Existing Fan Rm L3	24.0	36.0	Floor	54.0	261.0	72.0	36.0	27.0	5500	3.5" hkp on (E) conc slab
7	AHU-4	AHU-4	Existing Fan Rm L3	24.0	36.0	Floor	54.0	220.0	72.0	36.0	27.0	4500	3.5" hkp on (E) conc slab

S_{DS} = 1.20 *Snowden Building, Anchorage*

	ID	Wp (lb)	Mount Type	a_p	R_p	I_p	z	Roof Height (ft)	Fp_{MAX}	Fp_{MIN}	Fp_{CALC}	$Fp_{control}$
1	BLR-1	1,900	Floor	1.0	2.5	1.00	24.00	36.00	3,648	684	851	851
2	WH-1	450	Floor	1.0	2.5	1.00	24.00	36.00	864	162	202	202
3	ET-1	800	Floor	1.0	2.5	1.00	24.00	36.00	1,536	288	358	358
4	AHU-1	3,500	Floor	2.5	6.0	1.00	11.00	36.00	6,720	1,260	1,128	1,260
5	AHU-2	4,000	Floor	2.5	6.0	1.00	24.00	36.00	7,680	1,440	1,867	1,867
6	AHU-3	5,500	Floor	2.5	6.0	1.00	24.00	36.00	10,560	1,980	2,567	2,567
7	AHU-4	4,500	Floor	2.5	6.0	1.00	24.00	36.00	8,640	1,620	2,100	2,100

	ID	# Anchors resisting tension	# Anchors resisting shear	OM (lb-in)	RM (lb-in)	M_{NET} (lb-in)	T (lb)	T/Anchor (lb)	V (lb)	V/Anchor (lb)	ΩT (lb/anchor)	ΩV (lb/anchor)
1	BLR-1	2	4	35,325	20,879	14,446	868	434	851	213	1494.6	426
2	WH-1	1	3	4,556	3,490	1,066	91	91	202	67	478.5	134
3	ET-1	1	3	8,422	6,336	2,086	174	174	358	119	875.7	239
4	AHU-1	2	8	32,130	79,695	0	0	0	1,260	158	0.0	315
5	AHU-2	2	8	47,600	106,920	0	0	0	1,867	233	0.0	467
6	AHU-3	2	8	92,400	98,010	0	0	0	2,567	321	1607.2	642
7	AHU-4	2	8	75,600	80,190	0	0	0	2,100	263	1315.0	525

$$OM = Fp * CG_{ht}$$

$$RM = (0.9 - 0.2 * S_{DS}) * W_p * CG_w$$

$$T = \frac{OM - RM}{Width} \qquad V = F_p$$

** Assumed qty of anchors for AHUs

Note, use overstrength = 2 for concrete anchorage calcs

Max V with overstrength =	642	lb				

Check 18 gage steel at 3/4" edge distance for shear load

$R_n = 1.5l_c tF_u < 3.0dtF_u$			let Fu =	45	ksi
R_n =	3.23	k	l_c =	0.5	in
R_n/Ω =	1613	lb	t =	0.0478	in
DCR =	0.20		d =	0.5	in

Max T with overstrength =	1,607	lb

Design anchorage of AHU-3 and use for BLR 1 and AHU-4.
Desing for ET-1 and use for WH-1

AHU 1 and AHU 2 - use thru bolts, shear capacity adequate by inspection.

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	ET-1 anchorage	Date:	5/25/2022
Fastening point:			

Specifier's comments:

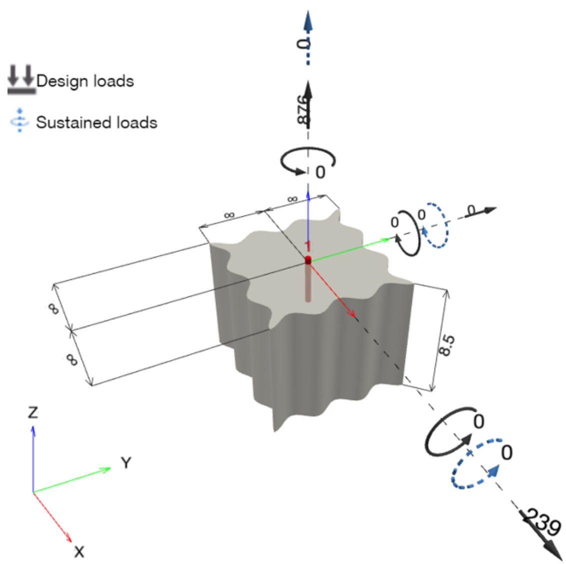
1 Input data

Anchor type and diameter:	**HIT-HY 200 + HAS-E 3/8**
Item number:	385418 HAS 5.8 3/8"x4-3/8" (element) / 2022791 HIT-HY 200-A (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 3.000 in. ($h_{ef,limit}$ = - in.)
Material:	5.8
Evaluation Service Report:	ESR-3187
Issued I Valid:	5/1/2021 | 3/1/2022
Proof:	Design Method ACI 318-11 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, , f_c' = 4,500 psi; h = 8.500 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (D.3.3.4.3 (d))
	Shear load: yes (D.3.3.5.3 (c))

Note: the HIT-HY 200 + HAS-E anchor is in the process of phase-out. As a result, there is limited/no inventory available.

Application also possible with HIT-HY 200 V3 + HAS-E under the selected boundary conditions.

Geometry [in.] & Loading [lb, in.lb]



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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	ET-1 anchorage	Date:	5/25/2022
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 876$; $V_x = 239$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$; $N_{sus} = 0$; $M_{x,sus} = 0$; $M_{y,sus} = 0$;	yes	53

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	876	239	239	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	876	3,653	24	OK
Bond Strength**	876	1,680	53	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	876	2,889	31	OK

* highest loaded anchor **anchor group (anchors in tension)

Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	ET-1 anchorage	Date:	5/25/2022
Fastening point:			

3.1 Steel Strength

N_{sa} = ESR value　　refer to ICC-ES ESR-3187
$\phi\ N_{sa} \geq N_{ua}$　　ACI 318-11 Table D.4.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.08	72,500

Calculations

N_{sa} [lb]
5,620

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
5,620	0.650	1.000	3,653	876

3.2 Bond Strength

$N_a = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$ ACI 318-11 Eq. (D-18)

$\phi\ N_a \geq N_{ua}$ ACI 318-11 Table D.4.1.1

A_{Na} see ACI 318-11, Part D.5.5.1, Fig. RD.5.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-11 Eq. (D-20)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-11 Eq. (D-21)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-11 Eq. (D-25)

$\psi_{cp,Na} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-11 Eq. (D-27)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \alpha_{N,seis} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-11 Eq. (D-22)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,354	0.375	3.000	∞	1.000	1,108

c_{ac} [in.]	λ_a	$\alpha_{N,seis}$
5.575	1.000	0.880

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
5.461	119.31	119.31	1.000

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	3,447

Results

N_a [lb]	ϕ_{bond}	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_a$ [lb]	N_{ua} [lb]
3,447	0.650	0.750	1.000	1,680	876

3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$	ACI 318-11 Eq. (D-3)
$\phi\ N_{cb} \geq N_{ua}$	ACI 318-11 Table D.4.1.1
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-11 Eq. (D-5)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-11 Eq. (D-10)
$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-11 Eq. (D-12)
$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$	ACI 318-11 Eq. (D-6)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
3.000	∞	1.000	5.575	17	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
81.00	81.00	1.000	1.000	5,926

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
5,926	0.650	0.750	1.000	2,889	876

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	239	1,415	17	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)**	239	4,826	5	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-3187

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.08	72,500	0.700

Calculations

$V_{sa,eq}$ [lb]
2,359

Results

$V_{sa,eq}$ [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ V_{sa,eq}$ [lb]	V_{ua} [lb]
2,359	0.600	1.000	1,415	239

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4.2 Pryout Strength (Bond Strength controls)

$V_{cp} = k_{cp}\left[\left(\frac{A_{Na}}{A_{Na0}}\right)\psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}\right]$	ACI 318-11 Eq. (D-40)
$\phi\ V_{cp} \geq V_{ua}$	ACI 318-11 Table (D.4.1.1)
A_{Na} see ACI 318-11, Part D.5.5.1, Fig. RD.5.5.1(b)	
$A_{Na0} = (2\ c_{Na})^2$	ACI 318-11 Eq. (D-20)
$c_{Na} = 10\ d_a\sqrt{\frac{\tau_{uncr}}{1100}}$	ACI 318-11 Eq. (D-21)
$\psi_{ed,Na} = 0.7 + 0.3\left(\frac{c_{a,min}}{c_{na}}\right) \leq 1.0$	ACI 318-11 Eq. (D-25)
$\psi_{cp,Na} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$	ACI 318-11 Eq. (D-27)
$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \alpha_{N,seis} \cdot \pi \cdot d_a \cdot h_{ef}$	ACI 318-11 Eq. (D-22)

Variables

k_{cp}	$\alpha_{overhead}$	$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{k,c}$ [psi]
2	1.000	2,354	0.375	3.000	∞	1,108

c_{ac} [in.]	λ_a	$\alpha_{N,seis}$
5.575	1.000	0.880

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
5.461	119.31	119.31	1.000

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	3,447

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
6,894	0.700	1.000	1.000	4,826	239

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.521	0.169	5/3	39	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-11 Appendix D, Part D.3.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Part D.3.3.4.3 (b), Part D.3.3.4.3 (c), or Part D.3.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Part D.3.3.5.3 (a), Part D.3.3.5.3 (b), or Part D.3.3.5.3 (c).
- Part D.3.3.4.3 (b) / part D.3.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Part D.3.3.4.3 (c) / part D.3.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Part D.3.3.4.3 (d) / part D.3.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-11, Part D.9.1

Fastening meets the design criteria!

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7 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 + HAS-E 3/8

Item number: 385418 HAS 5.8 3/8"x4-3/8" (element) / 2022791 HIT-HY 200-A (adhesive)

Maximum installation torque: 180 in.lb

Hole diameter in the base material: 0.438 in.

Hole depth in the base material: 3.000 in.

Minimum thickness of the base material: 4.250 in.

3/8 Hilti HAS Carbon steel threaded rod with Hilti HIT-HY 200 Safe Set System

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	-	-	-	-

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8 Remarks; Your Cooperation Duties

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Specifier's comments:

1 Input data

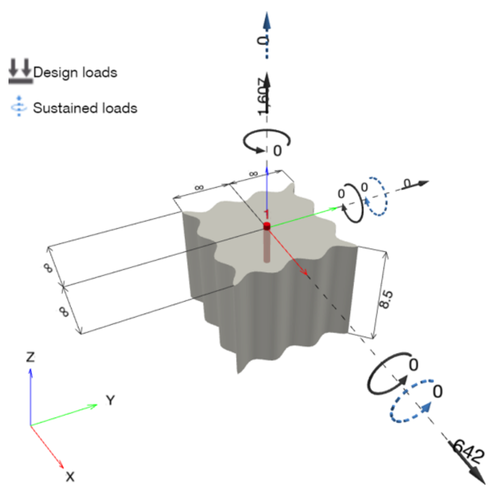
Anchor type and diameter:	**HIT-HY 200 + HAS-E 1/2**
Item number:	385423 HAS 5.8 1/2"x4-1/2" (element) / 2022791 HIT-HY 200-A (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 3.000 in. ($h_{ef,limit}$ = - in.)
Material:	5.8
Evaluation Service Report:	ESR-3187
Issued I Valid:	5/1/2021 | 3/1/2022
Proof:	Design Method ACI 318-11 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, , f_c' = 4,500 psi; h = 8.500 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (D.3.3.4.3 (d)) Shear load: yes (D.3.3.5.3 (c))

Note: the HIT-HY 200 + HAS-E anchor is in the process of phase-out. As a result, there is limited/no inventory available.

Application also possible with HIT-HY 200 V3 + HAS-E under the selected boundary conditions.

Application also possible with HVU2 + HAS 1/2 under the selected boundary conditions.
More information in section Alternative fastening data of this report.

Geometry [in.] & Loading [lb, in.lb]



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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 1,607; V_x = 642; V_y = 0; M_x = 0; M_y = 0; M_z = 0; N_{sus} = 0; $M_{x,sus}$ = 0; $M_{y,sus}$ = 0;	yes	59

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,607	642	642	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	1,607	6,688	25	OK
Bond Strength**	1,607	2,738	59	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	1,607	2,889	56	OK

* highest loaded anchor **anchor group (anchors in tension)

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3.1 Steel Strength

N_{sa} = ESR value   refer to ICC-ES ESR-3187
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.14	72,500

Calculations

N_{sa} [lb]
10,290

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
10,290	0.650	1.000	6,688	1,607

3.2 Bond Strength

$N_a = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$	ACI 318-11 Eq. (D-18)
$\phi\ N_a \geq N_{ua}$	ACI 318-11 Table D.4.1.1
A_{Na} see ACI 318-11, Part D.5.5.1, Fig. RD.5.5.1(b)	
$A_{Na0} = (2\ c_{Na})^2$	ACI 318-11 Eq. (D-20)
$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$	ACI 318-11 Eq. (D-21)
$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$	ACI 318-11 Eq. (D-25)
$\psi_{cp,Na} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$	ACI 318-11 Eq. (D-27)
$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \alpha_{N,seis} \cdot \pi \cdot d_a \cdot h_{ef}$	ACI 318-11 Eq. (D-22)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,354	0.500	3.000	∞	1.000	1,204

c_{ac} [in.]	λ_a	$\alpha_{N,seis}$
4.979	1.000	0.990

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
7.282	212.11	212.11	1.000

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	5,616

Results

N_a [lb]	ϕ_{bond}	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_a$ [lb]	N_{ua} [lb]
5,616	0.650	0.750	1.000	2,738	1,607

3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$	ACI 318-11 Eq. (D-3)
$\phi\ N_{cb} \geq N_{ua}$	ACI 318-11 Table D.4.1.1
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-11 Eq. (D-5)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-11 Eq. (D-10)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-11 Eq. (D-12)
$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$	ACI 318-11 Eq. (D-6)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
3.000	∞	1.000	4.979	17	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
81.00	81.00	1.000	1.000	5,926

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
5,926	0.650	0.750	1.000	2,889	1,607

4 Shear load

	Load V_{ua} [lb]	Capacity $\phi\ V_n$ [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	642	2,593	25	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)**	642	7,862	9	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-3187

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.14	72,500	0.700

Calculations

$V_{sa,eq}$ [lb]
4,321

Results

$V_{sa,eq}$ [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ V_{sa,eq}$ [lb]	V_{ua} [lb]
4,321	0.600	1.000	2,593	642

4.2 Pryout Strength (Bond Strength controls)

$V_{cp} = k_{cp}\left[\left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}\right]$ ACI 318-11 Eq. (D-40)

$\phi\ V_{cp} \geq V_{ua}$ ACI 318-11 Table (D.4.1.1)

A_{Na} see ACI 318-11, Part D.5.5.1, Fig. RD.5.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-11 Eq. (D-20)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-11 Eq. (D-21)

$\psi_{ed,Na} = 0.7 + 0.3\left(\frac{c_{a,min}}{c_{na}}\right) \leq 1.0$ ACI 318-11 Eq. (D-25)

$\psi_{cp,Na} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-11 Eq. (D-27)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \alpha_{N,seis} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-11 Eq. (D-22)

Variables

k_{cp}	$\alpha_{overhead}$	$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{k,c}$ [psi]
2	1.000	2,354	0.500	3.000	∞	1,204

c_{ac} [in.]	λ_a	$\alpha_{N,seis}$
4.979	1.000	0.990

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
7.282	212.11	212.11	1.000

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	5,616

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
11,231	0.700	1.000	1.000	7,862	642

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.587	0.248	5/3	51	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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Design:	Snowden - AHU-3	Date:	5/25/2022
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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-11 Appendix D, Part D.3.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Part D.3.3.4.3 (b), Part D.3.3.4.3 (c), or Part D.3.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Part D.3.3.5.3 (a), Part D.3.3.5.3 (b), or Part D.3.3.5.3 (c).
- Part D.3.3.4.3 (b) / part D.3.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Part D.3.3.4.3 (c) / part D.3.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Part D.3.3.4.3 (d) / part D.3.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-11, Part D.9.1

Fastening meets the design criteria!

7 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 + HAS-E 1/2

Item number: 385423 HAS 5.8 1/2"x4-1/2" (element) / 2022791 HIT-HY 200-A (adhesive)

Maximum installation torque: 360 in.lb

Hole diameter in the base material: 0.562 in.

Hole depth in the base material: 3.000 in.

Minimum thickness of the base material: 4.250 in.

1/2 Hilti HAS Carbon steel threaded rod with Hilti HIT-HY 200 Safe Set System

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	-	-	-	-

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8 Alternative fastening

8.1 Alternative fastening data

Anchor type and diameter:	**HVU2 + HAS 1/2**
Item number:	333190 HAS-E 1/2"x6 1/2" (element) / not available (capsule)
Effective embedment depth:	$h_{ef,act}$ = 4.250 in., h_{nom} = 4.250 in.
Material:	5.8
Evaluation Service Report:	ESR-4372
Issued I Valid:	9/1/2021 | 6/1/2022
Proof:	Design Method ACI 318-11 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, , f_c' = 4,500 psi; h = 8.500 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry, Installation direction: vertical downward**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (D.3.3.4.3 (d))
	Shear load: yes (D.3.3.5.3 (c))

Max. Utilization with HVU2 + HAS 1/2: 44 %
Fastening meets the design criteria!

8.2 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Hammer drilled
Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HVU2 + HAS 1/2
Item number: 333190 HAS-E 1/2"x6 1/2" (element) / not available (capsule)
Maximum installation torque: 360 in.lb
Hole diameter in the base material: 0.562 in.
Hole depth in the base material: 4.250 in.
Minimum thickness of the base material: 5.500 in.

1/2 Hilti HAS Carbon steel threaded rod with Hilti HVU2

8.2.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• HVA square drive shafts • Torque wrench

9 Remarks; Your Cooperation Duties

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Snowden Mech Upgrades EH
40.2021.105 5/5/2022

Design Frame for ACC Units

See Enercalc for gravity calculations, including existing framing.

Per ASCE 7-16, Section 15.3.1, use Chapter 13 for design of ACC support frame.

Rp = 1.25 per Table 15.4-2, conservatively
ap = 1 per Table 13.5-1, footnote a

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\frac{R_p}{I_p}}\left(1 + 2\frac{z}{h}\right)$$ Eqn 13.3-2

$Fp < 1.6\ S_{DS}\ Ip\ Wp$ = 3613 lb

$Fp > 0.3\ S_{DS}\ Ip\ Wp$ = 678 lb

S_{DS} = 1.2
Ip = 1

Wp =>	Wacc =	1122	lb	
	Wsteel:	L =	22	ft
		w/ft =	20	plf
		Wsteel =	440	lb
	Wsnow (psf) =	8	psf (0.2 x 40 psf)	
	Asnow:			
		L =	8	ft
		w =	5	ft
		Asnow =	40	sf
		Wsnow =	320	lb
		Wp =	1882	lb

z/h = 1

Fp calc = 2168.1 lb
Fp = 2168.1 lb

Check cantilever pipe:

# pipes =	4	
Fp to ea pipe =	542.0	lb
M = Fp x h:		
h =	5.875	ft (base of pipe to midheight of ACC unit)
M =	3184.3	lbft (LRFD)

6" STD pipe

D/t =	25.4	
E =	29000	ksi
Fy =	35	ksi
0.45 E/Fy =	372.9	> D/t
Z =	10.6	in^3
Mn = Mp = FyZ =	30916.7	lbft

Check if compact:

0.07 * E/Fy =	58	
	compact	
ϕ =	0.9	
ϕ Mn =	27825	lbft
DCR =	0.11	

Design bolts

M = T x d

d =	9	in
T =	4245.8	lb
# bolts =	2	in tension
ϕ r_n =	29800	lb (3/4" diameter bolt, Group A, tension)
DCR =	0.07	tension
shear, ϕ r_n =	17900	lb (3/4" diameter bolt, Group A, shear)
DCR =	0.008	

consideration of combined shear and tension is not required b/c DCR < 30%

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Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Lic. # : KW-06001667 **REID MIDDLETON, INC.**

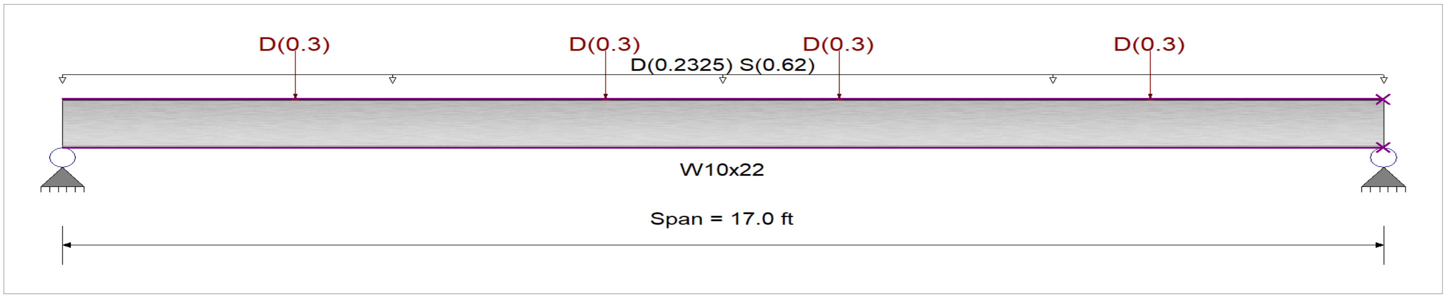
DESCRIPTION: (E) Steel beam with ACC load

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	36.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.0150, S = 0.040 ksf, Tributary Width = 15.50 ft

Point Load : D = 0.30 k @ 3.0 ft

Point Load : D = 0.30 k @ 7.0 ft

Point Load : D = 0.30 k @ 10.0 ft

Point Load : D = 0.30 k @ 14.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.724 : 1	Maximum Shear Stress Ratio =	0.223 : 1
Section used for this span	**W10x22**	Section used for this span	**W10x22**
Ma : Applied	33.797 k-ft	Va : Applied	7.846 k
Mn / Omega : Allowable	46.707 k-ft	Vn/Omega : Allowable	35.251 k
Load Combination	+D+S+H	Load Combination	+D+S+H
Location of maximum on span	8.500 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.342 in	Ratio =	596 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.516 in	Ratio =	395 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 17.00 ft	1	0.244	0.073	11.40		11.40	78.00	46.71	1.00	1.00	2.58	52.88	35.25
+D+L+H													
Dsgn. L = 17.00 ft	1	0.244	0.073	11.40		11.40	78.00	46.71	1.00	1.00	2.58	52.88	35.25
+D+Lr+H													
Dsgn. L = 17.00 ft	1	0.244	0.073	11.40		11.40	78.00	46.71	1.00	1.00	2.58	52.88	35.25
+D+S+H													
Dsgn. L = 17.00 ft	1	0.724	0.223	33.80		33.80	78.00	46.71	1.00	1.00	7.85	52.88	35.25
+D+0.750Lr+0.750L+H													
Dsgn. L = 17.00 ft	1	0.244	0.073	11.40		11.40	78.00	46.71	1.00	1.00	2.58	52.88	35.25
+D+0.750L+0.750S+H													
Dsgn. L = 17.00 ft	1	0.604	0.185	28.20		28.20	78.00	46.71	1.00	1.00	6.53	52.88	35.25
+D+0.60W+H													

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Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

File: snowden.ec6

Lic. # : KW-06001667 **REID MIDDLETON, INC.**

DESCRIPTION: (E) Steel beam with ACC load

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
Dsgn. L = 17.00 ft	1	0.244	0.073	11.40		11.40	78.00	46.71	1.00	1.00	2.58	52.88	35.25
+D+0.750Lr+0.750L+0.450W+H													
Dsgn. L = 17.00 ft	1	0.244	0.073	11.40		11.40	78.00	46.71	1.00	1.00	2.58	52.88	35.25
+D+0.750L+0.750S+0.450W+H													
Dsgn. L = 17.00 ft	1	0.604	0.185	28.20		28.20	78.00	46.71	1.00	1.00	6.53	52.88	35.25
+0.60D+0.60W+0.60H													
Dsgn. L = 17.00 ft	1	0.146	0.044	6.84		6.84	78.00	46.71	1.00	1.00	1.55	52.88	35.25
+D+0.70E+0.60H													
Dsgn. L = 17.00 ft	1	0.244	0.073	11.40		11.40	78.00	46.71	1.00	1.00	2.58	52.88	35.25
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 17.00 ft	1	0.604	0.185	28.20		28.20	78.00	46.71	1.00	1.00	6.53	52.88	35.25
+0.60D+0.70E+H													
Dsgn. L = 17.00 ft	1	0.146	0.044	6.84		6.84	78.00	46.71	1.00	1.00	1.55	52.88	35.25

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.5159	8.549		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	7.846	7.846
Overall MINimum	1.546	1.546
+D+H	2.576	2.576
+D+L+H	2.576	2.576
+D+Lr+H	2.576	2.576
+D+S+H	7.846	7.846
+D+0.750Lr+0.750L+H	2.576	2.576
+D+0.750L+0.750S+H	6.529	6.529
+D+0.60W+H	2.576	2.576
+D+0.750Lr+0.750L+0.450W+H	2.576	2.576
+D+0.750L+0.750S+0.450W+H	6.529	6.529
+0.60D+0.60W+0.60H	1.546	1.546
+D+0.70E+0.60H	2.576	2.576
+D+0.750L+0.750S+0.5250E+H	6.529	6.529
+0.60D+0.70E+H	1.546	1.546
D Only	2.576	2.576
S Only	5.270	5.270
H Only		

Steel Beam

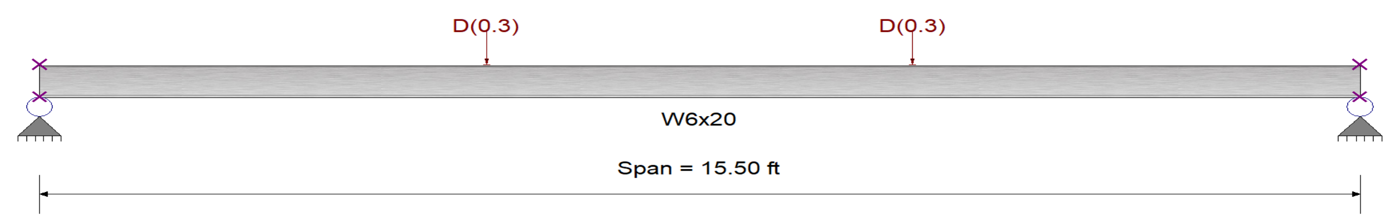
DESCRIPTION: (N) beam for ACC support

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Load(s) for Span Number 1
Point Load : D = 0.30 k @ 5.250 ft

Point Load : D = 0.30 k @ 10.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.050** : 1	Maximum Shear Stress Ratio =	**0.009** : 1
Section used for this span	**W6x20**	Section used for this span	**W6x20**
Ma : Applied	1.575 k-ft	Va : Applied	0.30 k
Mn / Omega : Allowable	31.395 k-ft	Vn/Omega : Allowable	32.240 k
Load Combination	+D+H	Load Combination	+D+H
Location of maximum on span	9.123 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.058 in	Ratio =	3211 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+D+L+H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+D+Lr+H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+D+S+H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+D+0.750Lr+0.750L+H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+D+0.750L+0.750S+H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+D+0.60W+H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24

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Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Lic. # : KW-06001667 REID MIDDLETON, INC.

DESCRIPTION: (N) beam for ACC support

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+D+0.750Lr+0.750L+0.450W+H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+D+0.750L+0.750S+0.450W+H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+0.60D+0.60W+0.60H													
Dsgn. L = 15.46 ft	1	0.030	0.006	0.95		0.95	52.43	31.39	1.14	1.00	0.18	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.006	0.01		0.01	62.08	37.18	1.00	1.00	0.18	48.36	32.24
+D+0.70E+0.60H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 15.46 ft	1	0.050	0.009	1.58		1.58	52.43	31.39	1.14	1.00	0.30	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.009	0.01		0.01	62.08	37.18	1.00	1.00	0.30	48.36	32.24
+0.60D+0.70E+H													
Dsgn. L = 15.46 ft	1	0.030	0.006	0.95		0.95	52.43	31.39	1.14	1.00	0.18	48.36	32.24
Dsgn. L = 0.04 ft	1	0.000	0.006	0.01		0.01	62.08	37.18	1.00	1.00	0.18	48.36	32.24

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.0579	7.794		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

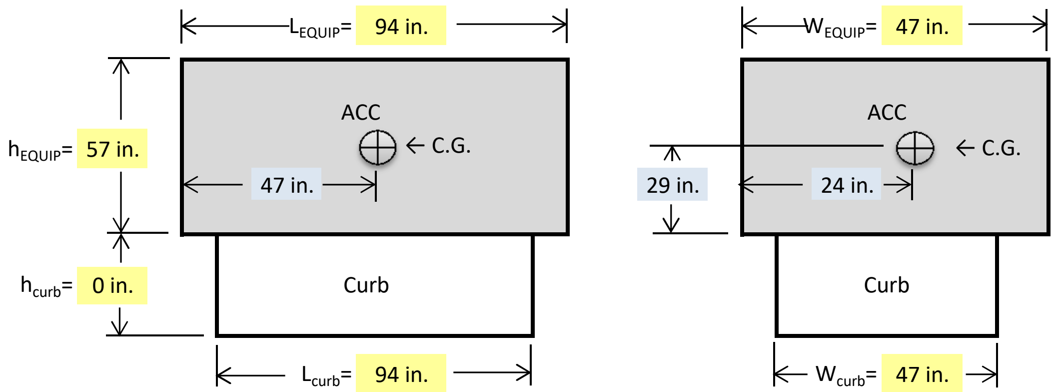
Load Combination	Support 1	Support 2
Overall MAXimum	0.300	0.300
Overall MINimum	0.180	0.180
+D+H	0.300	0.300
+D+L+H	0.300	0.300
+D+Lr+H	0.300	0.300
+D+S+H	0.300	0.300
+D+0.750Lr+0.750L+H	0.300	0.300
+D+0.750L+0.750S+H	0.300	0.300
+D+0.60W+H	0.300	0.300
+D+0.750Lr+0.750L+0.450W+H	0.300	0.300
+D+0.750L+0.750S+0.450W+H	0.300	0.300
+0.60D+0.60W+0.60H	0.180	0.180
+D+0.70E+0.60H	0.300	0.300
+D+0.750L+0.750S+0.5250E+H	0.300	0.300
+0.60D+0.70E+H	0.180	0.180
D Only	0.300	0.300
H Only		

ReidMiddleton

Project: Snowden ACC
Date: 5/1/2022
Engineer: EH
Project #: 40.2021.105

Equipment Tag: ACC rooftop

ACC rooftop Attachment Forces - (ASCE 7-16, ASD)



rooftop weight= 1122 lbs Curb Weight= 0 lbs

Plan Dims of Building: B= 243 ft L= 125 ft h= 37 ft

Seismic a_p= 2.5 R_p= 6.0 S_{DS}= 1.2 I_p= 1.0 Z= 37 ft h= 37 ft

(ASCE 7-16) W_p= 1122 lbs

F_p= $(0.4 x a_p x S_{DS} x W_p/(R_p/I_p)) x (1+2(z/h))$= **673 lbs** (eqn 13.3-1)

$F_{p\ max}$ = $1.6 x S_{ds} x I_p x W_p$= **2154 lbs** (eqn 13.3-2)

$F_{p\ min}$ = $0.3 x S_{ds} x I_p x W_p$= **404 lbs** (eqn 13.3-3)

$0.7{*}F_p$= 471 lbs ASD

ASD Net Overturning Moments (M_{ovt} - M_{rest}) $(0.6-0.14S_{DS})D+0.7E$ Sec. 12.14.3.2.3

Equipment to Curb: 2040 lbs-in

Curb to Structure: 2040 lbs-in

Wind V= 130 mph Exp= B K_{zt}= 1.0 Risk Category: II

(ASCE 7-16) K_d= 0.85 α= 7 z_g= 1200

K_z= 0.744 A_f= 37.21 sf $A_{f\ EQUIP\ only}$= 37.21 sf

A_r= 30.68 sf

K_e = 1.00 Ground elevation factor, see Table 26.9-1

$GC_{r\ LATERAL}$ = 1.9 (eqn 29.4.2) $GC_{r\ VERTICAL}$ = 1.5 (eqn 29.4.3)

q_z= $0.00256 x K_z x K_{zt} x K_d x K_e {*} V^2$= 27.35 psf (eqn 26.10-1)

$F=q_z x A_f x (GC_r)$= 1934 lbs (eqn 29.4.2) **$0.6{*}F$= 1160 lbs** ASD

$F_{EQUIP\ only}$= 1934 lbs **$0.6{*}F_{EQUIP\ only}$= 1160 lbs** ASD

$F_v=q_z x A_r x (GC_{r\text{-}vert})$= 1259 lbs (eqn 29.4.3) **$0.6{*}F_v$= 755 lbs** ASD

Net Overturning Moments (M_{ovt} - M_{rest}) $0.6D+0.6W_{lateral}+0.6W_{uplift}$

Equipment to Curb: 34999 lbs-in Wind Controls

Curb to Structure: 34999 lbs-in Wind Controls

ReidMiddleton

Project:	Snowden ACC	Date:	5/1/2022
Engineer:	EH	Project #:	40.2021.105

Equipment Tag: ACC rooftop

ACC rooftop Attachment Forces - (ASCE 7-16, ASD)

Equipment to Curb ASD			NOT USED	**Curb to Structure ASD**			
M/d=	**T= 745 lbs**			M/d=	**T= 745 lbs**		
	V= 1160 lbs				**V= 1160 lbs**		
# of Screws Used				# of Screws Used			
In T=	30	Screws		In T=	4	Screws	
In V=	60	Screws		In V=	8	Screws	
Force Per Screw				Force Per Screw			
Ts=	25	lbs/screw		Ts=	186	lbs/screw	
Vs=	19	lbs/screw		Vs=	145	lbs/screw	
#8 Screw (20GA to 20GA)				1/4 Screw (20GA to W16)			
Pnt =	217	lbs/screw		Pnt =	1290	lbs/screw	
Pnv =	493	lbs/screw		Pnv =	1460	lbs/screw	
Screw Combined Shear and Tension				Screw Combined Shear and Tension			
AISI D100-17E, J4.5.2				AISI D100-17E, J4.5.2			
Ω =	2.55			Ω =	2.55		
((Ts/Pnt)+(Vs/Pnv))*Ω <= 1.15				((Ts/Pnt)+(Vs/Pnv))*Ω <= 1.15			
Combo DCR =	0.39	< 1.15, OK		Combo DCR =	0.62	< 1.15, OK	

Isolator Type	Spring Color	Rated Capacity		Rated Deflection		Spring O.D.		L		W		D		d		H	
		lbs.	kg	in.	mm	in.	mm	in.	mm	in.	mm	in.	mm	in.	mm	in.	mm
FLSS-1-250	Blue	250	113	1.79	45	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-450	Green	450	204	1.54	39	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-625	Black	625	283	1.44	37	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-800	Gray	800	363	1.31	33	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-1000	Red	1000	454	1.15	29	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-1250	Brown	1250	567	1.09	28	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-1700	Orange	1700	771	0.95	24	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-2200	Org/Gray	2200	998	1.00	26	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-2465	Blue	2465	1118	1.00	25	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-2865	Blu/Gray	2865	1300	1.00	25	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-3500	Blue/Brn	3500	1588	1.00	25	3.00	76	9.00	227	4.00	102	0.69	17	0.56	14	6.25	159
FLSS-1-2500	Brown	2500	1134	1.09	28	3.00	76	15.25	387	5.00	127	0.69	17	0.56	14	6.88	175
FLSS-1-3400	Orange	3400	1542	0.95	24	3.00	76	15.25	387	5.00	127	0.69	17	0.56	14	6.88	175
FLSS-1-4400	Org/Gray	4400	1996	1.00	26	3.00	76	15.25	387	5.00	127	0.69	17	0.56	14	6.88	175
FLSS-1-4930	Blue	4930	2236	1.00	25	3.00	76	15.25	387	5.00	127	0.69	17	0.56	14	6.88	175
FLSS-1-5730	Blu/Gray	5730	2599	1.00	25	3.00	76	15.25	387	5.00	127	0.69	17	0.56	14	6.88	175
FLSS-1-7000	Blu/Brn	7000	3175	1.00	25	3.00	76	15.25	387	5.00	127	0.69	17	0.56	14	6.88	175
FLSS-1-5000	Brown	5000	2270	1.09	28	3.00	76	15.75	400	8.00	203	0.81	21	0.69	17	7.50	191
FLSS-1-6800	Orange	6800	3085	0.95	24	3.00	76	15.75	400	8.00	203	0.81	21	0.69	17	7.50	191
FLSS-1-8800	Org/Gray	8800	3992	1.00	26	3.00	76	15.75	400	8.00	203	0.81	21	0.69	17	7.50	191
FLSS-1-9860	Blue	9860	4472	1.00	25	3.00	76	15.75	400	8.00	203	0.81	21	0.69	17	7.50	191
FLSS-1-11460	Blu/Gray	11460	5198	1.00	25	3.00	76	15.75	400	8.00	203	0.81	21	0.69	17	7.50	191
FLSS-1-14000	Blu/Brn	14000	6350	1.00	25	3.00	76	15.75	400	8.00	203	0.81	21	0.69	17	7.50	191
FLSS-2-100	Gray	100	45	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-135	Beige	135	61	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-185	Brown	185	84	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-225	Gray/Brn	225	102	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-250	Blue	250	113	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-375	Blue/Brn	375	170	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-500	Green	500	227	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-625	Grn/Brn	625	283	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-750	Black	750	340	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-875	Blk/Brn	875	397	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-995	Orange	995	451	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-1120	Org/Brn	1120	208	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-1400	Org/Grn	1400	635	2.01	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-1600	Red	1600	726	2.00	51	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-1975	Red/Grn	1975	896	1.98	50	3.50	86	10.00	254	5.00	127	0.69	17	0.56	14	8.00	203
FLSS-2-2000	Orange	2000	907	2.00	51	5.00	127	11.00	280	5.00	127	0.69	17	0.56	14	10.00	254
FLSS-2-2500	Blue	2500	1134	2.00	51	5.00	127	11.00	280	5.00	127	0.69	17	0.56	14	10.00	254
FLSS-2-2750	Blu/Blu	2750	1247	2.00	51	5.00	127	11.00	280	5.00	127	0.69	17	0.56	14	10.00	254
FLSS-2-3025	Blu/Grn	3025	1372	2.02	51	5.00	127	11.00	280	5.00	127	0.69	17	0.56	14	10.00	254
FLSS-2-3250	Blu/Blk	3250	1474	2.00	51	5.00	127	11.00	280	5.00	127	0.69	17	0.56	14	10.00	254
FLSS-2-3500	Blu/Org	3500	1588	2.00	51	5.00	127	11.00	280	5.00	127	0.69	17	0.56	14	10.00	254
FLSS-2-3900	Blu/Org/Grn	3900	1769	2.00	51	5.00	127	11.00	280	5.00	127	0.69	17	0.56	14	10.00	254
FLSS-2-4100	Blu/Red	4100	1860	2.00	51	5.00	127	11.00	280	5.00	127	0.69	17	0.56	14	10.00	254
FLSS-2-4500	Blu/Red/Grn	4500	2041	2.00	51	5.00	127	11.00	280	5.00	127	0.69	17	0.56	14	10.00	254
FLSS-2-4000	Orange	4000	1814	2.00	51	5.00	127	17.50	444	6.00	152	0.69	17	0.56	14	10.13	257
FLSS-2-5000	Blu	5000	2268	2.00	51	5.00	127	17.50	444	6.00	152	0.69	17	0.56	14	10.13	257
FLSS-2-5500	Blu/Blu	5500	2495	2.00	51	5.00	127	17.50	444	6.00	152	0.69	17	0.56	14	10.13	257
FLSS-2-6050	Blu/Grn	6050	2744	2.02	51	5.00	127	17.50	444	6.00	152	0.69	17	0.56	14	10.13	257
FLSS-2-6500	Blu/Blk	6500	2948	2.00	51	5.00	127	17.50	444	6.00	152	0.69	17	0.56	14	10.13	257
FLSS-2-7000	Blu/Org	7000	3175	2.00	51	5.00	127	17.50	444	6.00	152	0.69	17	0.56	14	10.13	257
FLSS-2-7800	Blu/Org/Grn	7800	3538	2.00	51	5.00	127	17.50	444	6.00	152	0.69	17	0.56	14	10.13	257
FLSS-2-8200	Blu/Red	8200	3719	2.00	51	5.00	127	17.50	444	6.00	152	0.69	17	0.56	14	10.13	257
FLSS-2-9000	Blu/Red/Grn	9000	4082	2.00	51	5.00	127	17.50	444	6.00	152	0.69	17	0.56	14	10.13	257
FLSS-2-8000	Orange	8000	3629	2.00	51	5.00	127	21.50	546	11.00	279	0.81	21	0.69	17	11.00	279
FLSS-2-10000	Blue	10000	4536	2.00	51	5.00	127	21.50	546	11.00	279	0.81	21	0.69	17	11.00	279
FLSS-2-11000	Blu/Blu	11000	4990	2.00	51	5.00	127	21.50	546	11.00	279	0.81	21	0.69	17	11.00	279
FLSS-2-12100	Blu/Grn	12100	5488	2.02	51	5.00	127	21.50	546	11.00	279	0.81	21	0.69	17	11.00	279
FLSS-2-13000	Blu/Blk	13000	5897	2.00	51	5.00	127	21.50	546	11.00	279	0.81	21	0.69	17	11.00	279
FLSS-2-14000	Blu/Org	14000	6350	2.00	51	5.00	127	21.50	546	11.00	279	0.81	21	0.69	17	11.00	279
FLSS-2-15600	Blu/Org/Grn	15600	7076	2.00	51	5.00	127	21.50	546	11.00	279	0.81	21	0.69	17	11.00	279
FLSS-2-16400	Blu/Red	16400	7439	2.00	51	5.00	127	21.50	546	11.00	279	0.81	21	0.69	17	11.00	279
FLSS-2-18000	Blu/Red/Grn	18000	8165	2.00	51	5.00	127	21.50	546	11.00	279	0.81	21	0.69	17	11.00	279

DESIGN OF NEW MECH PENTHOUSE

Roof Deck

1.5" x 20GA metal roof deck
Snow Load, S = 40 psf
Verco span table, allowable load = 300 psf > snow load

wind uplift = 77 psf (wind uplift map)
wind uplift (ASD) = 46.2 psf = 0.6 LRFD
Verco span table, allowable load = 300 psf > wind uplift

Per Verco Table 1: Allowable Tension load (lbs/connection) for arc spot welds
allowable = 602 lb
area = joist spacing x weld spacing
roof joist spacing = 2 ft
weld spacing = 1 ft
load = 92.4 lb (ASD)
DCR = 0.15

Penthouse Stud Walls

h = 11 ft

Wind = 49 psf (LRFD, Zone 5)
Wind = 29.4 psf (ASD) per stud = 39.2 plf

roof DL = 10 psf
S = 40 psf
DL + S = 50 psf
joist span = 13 ft
point load = 433.3 lb per stud

stud parameters:
depth = 6 in
spacing = 16 in

Use CFS for design (following)

Penthouse roof joists

Use CFS for design

Determine seismic weight of cupolas

ASCE 7-16
weight = DL + 20% of flat roof snow

roof, DL =	10	psf
0.2*snow =	8	psf
	18	psf

wall DL =	5	psf

PH width =	14	ft	wall A =	528	ft^2
PH length =	34	ft	plan A =	476	ft^2

total DL = 11.2 k

See excel printouts for seismic reference

ASCE 7-16 forces
Seismic vertical load distribution

Wind calculations

Reference Main Wind Force Resisting System excel printout
Combine 1 + 2:

A, 3.6 + 24.4 =	28	psf (LRFD)		
B, 33.5 - 5.4 =	28.1	psf (LRFD)	16.86	psf (ASD)

determine plf:
psf x wind h
wind h = h/2 + 2 ft parapet (est.)
plf = 210.8 plf (LRFD)

total force = plf x length
total force = 7166 lbs (LRFD)
total force = 4299 lbs (ASD)

Shear wall design

Seismic, LRFD =	2.1	k, base shear
Seismic, ASD =	1.47	k, base shear (LRFD x 0.7)
	divide by wall width, divide by 2 walls	
shear wall load =	52.5	plf (ASD)

Reference AISI Table C2.1-3
Nominal shear strength for seismic (4" spacing at panel edge)
0.027" steel sheet one side
thickness of stud = 43 mils, min

Rn =	1000	plf
Ω =	2.5	
Rn/Ω =	400	
DCR =	0.13	

Wind, LRFD =	7166	k, base shear
Wind, ASD =	4299.3	k, base shear (LRFD x 0.6)
	divide by wall width, divide by 2 walls	
shear wall load =	153.5	plf (ASD)

Nominal shear strength for wind (4" spacing at panel edge)
0.027" steel sheet one side
thickness of stud = 43 mils, min

Rn =	1000	plf
Ω =	2	
Rn/Ω =	500	
DCR =	0.31	

Check for wall with opening:

opening width =	6	ft
wall width - opening w =	8	ft
plf accounting for opening =	268.7	plf
DCR =	0.54	

bottom track to blocking connection — Wall perp to existing
see spreadsheet
(1) 1/4" screws @ 4" OC (therefore (3) per foot)

bottom track to plywood — Wall parallel to existing
#8 sheet metal screw

ult lat load =	465	lb (APA 830)
x 1.6 =	744	lb (multiply by 1.6 for short duration)
/ 5 =	149	lb allowable
spacing =	4	in
allowable per ft =	446	lb/ft
DCR =	0.60	

Diaphragm Design

Seismic load

LRFD w_p = 5.65 psf (seismic diaphragm load distribution)

ASD w_p = 4.0 psf (LRFD x 0.7)

$R = w_p * L/2$

R = 67.2 plf (ASD)

Wind load

total load = 4299.3 lb (ASD)

divide by area = 9.03 psf (ASD)

$R = w * L/2$

R = 153.5 plf (ASD)

Use 1.5" steel deck

36/4 pattern w/button punch @ 24" OC

allowable capacity = 403 plf

DCR = 0.38

Diaphragm boundary nailing:

#12 @ 12" OC deck to track

reference AISI cold-formed steel design manual volume 1

roof deck t =	0.0345	inches	
nominal shear tilting =	855	lbs	#12, Table IV-9c
allowable =	285	lbs (FOS = 3)	

use spacing of 12" OC

DCR = 0.54

Holdowns

Reference ASCE 7-16, and AISI S213-07 C5.1.2

** Use overstrength load combinations when doing holdowns

Seismic overturning:

ASD

$(0.6 - 0.14S_{DS})D + 0.7\ \Omega_o Q_E$

$(0.6 - 0.14*1)D + 0.7(3)Q_E$

$0.46D + 3\ (0.7Q_E)$

Holdown required at base of PH:
Mo = 3 * base shear * height

w roof =	4.36	psf (seismic load distribution)
load from roof =	2075	lb, LRFD
	1453	lb, ASD
h =	11	ft
Mo =	47.9	kft

Minimal resisting moment

Seismic Tension force = (Mo - M_R) / width
T = 3.42 kips TOTAL

Wind overturning:
0.6D + 0.6W

Holdown required at base of PH:

load at roof level =	7.2	k (LRFD)
	4.3	k (ASD)
Mo =	47.3	kft

Wind Tension force = (Mo - M_R) / width
T = 3.38 kips TOTAL

Max T =	3.42	k
per wall =	1.71	k

stud to bottom track
(6) 1/4" screws (occurs in 32")

screw nominal =	905	lb (AISI table IV-9a)
screw capacity =	301.7	lb
x 6 screws =	1810.0	lb
DCR =	0.95	

track to existing 2x blocking
See excel calc following

design shear load =	268.7	plf
space screws @	4	in
length required =	12	in
shear in length =	268.7	lb
# screws in length =	3	

Use Holdown from wall stud to existing WF

load =	1712	lb
S/HDU4 capacity =	3825	lb
DCR =	0.45	

DCR < 0.70 therefore works as corner holdown

Wind Uplift

Design connection for components and cladding wind uplift

Track to deck connection

loading =	77	psf	
wind uplift taken by long wall			
trib area = length x 1/2 width			
trib area =	238	ft^2	
load to each long wall =	18326	lb	
per ft =	539	plf (LRFD)	
per ft =	323.4	plf (ASD)	

from shear wall calcs:

(1) 1/4" screws @ 4" OC (therefore (3) per foot)

see excel spreadsheet for combined shear/tension calc

Sheathing to bottom track using #8 screws

Per AISI Table IV-9c (shear)

nominal =	564	lbs	0.027" sheathing to 0.0451" bottom track
allowable =	188.0	lbs	
(1) screw @ 4" OC			
in 1 ft =	564	plf	
DCR =	0.57		

At anchorage to existing CMU

ASD load =	223.9	lb (shear)	
	705.0	lb (tension)	holdown loading (overturning M / length)

5/8" diameter x 5" embed anchor

allowable shear =	930	lb	spacing =	10	in
allowable tension =	1045	lb			

DCR for tension =	0.67	for (1) screw
DCR for shear =	0.24	for (1) screw
sum DCR =	0.92	for (1) screw
DCR for tension =	0.34	for (2) screws
DCR for shear =	0.24	for (2) screws
sum DCR =	0.58	for (2) screws

< 0.70 therefore use as corner holdown

(2) screws occur in 10"

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Client: AMC
Project: Snowden Mech
Project No. 40.2021.105

Sheet ____ of ____
Design by: EH
Date: 04/27/22
Checked by: ____
Date: ____

SEISMIC VERTICAL LOAD DISTRIBUTION

Per ASCE7-16

ASCE7-16 (eq 12.8-12) $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$

ASCE7-16 (eq 12.8-11) $F_x = C_{vx} V$ "hx" in feet

ASCE7-16 (eq 12.8-13) $V = \sum_{i=1}^{n} F_i$

all forces Fx and Vx in kips, uon

T = 0.294
k = 1.000

R= 6.5
I_E= 1

S_{DS}= 1.2
S_1= 0.676

Exterior Wall Weight = 5.0 psf

Seismic Weight: Per ASCE 7-16, 12.7.2
1.0 * self weight (dead load) +
10 psf interior partition allowance +
20% flat roof snow load +
25% storage load > 125 psf
1.0 * permanent equipment weight

Level	Seismic Weight	Ext Wall Area
PH Roof	18 psf	528 sf

Level	Area	h_x	Equipment	w_x	$w_x{}^*h_x{}^k$	%V	Seis Coef	F_x	V_x	seis psf
PH Roof	476	11.00	0 lbs	11.21	123	100.0%	0.1850	2.1	2.1	4.36
				11	123	100%	**static base shear**		**2.07 k**	**(LRFD)**

Drift Min Base Shear: **0.6 k** (ASCE 7-16 12.9.1.4.2)

SEISMIC DIAPHRAGM LOAD DISTRIBUTION

ASCE7-16 (eq 12.10-1)

$$F_{Px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{Px}$$

Diaphragm force limits

$0.4S_{DS}I_E w_{px}$ =	0.480 $^*w_{px}$	max	(eq 12.10-3)
$0.2S_{DS}I_E w_{px}$=	0.240 $^*w_{px}$	min	(eq 12.10-2)

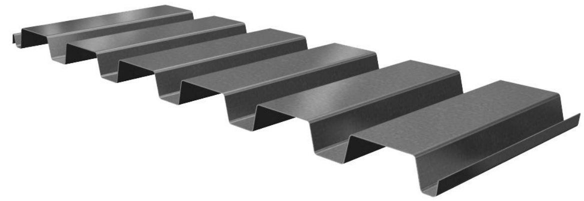
Level	F_i	ΣF_i	$\omega_i = \omega_{px}$	$\Sigma \omega_i$	F_{px}	F_{px} (LRFD) Min	Max	Controlling	ω_p (psf)
PH Roof	2.1	2.1	11	11	2.69	2.69	5.38	min	5.65

EFFECTS OF REDUNDANCY/OVERSTRENGTH

ρ = 1.0
Ωo = 3.00

Level	Vert Dist F_x * ρ	Diaph Dist ω_p * ρ	Diaph Dist ω_p * Ω_o
PH Roof	2.1 kips	5.7 psf	17.0 psf

Type PLB™-36 or HSB®-36



Allowable Uniform Loads (psf)

SPAN	DECK GAGE	CRITERIA	2'-0"	3'-0"	4'-0"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	11'-0"	12'-0"
SINGLE	22	Stress	300	300	220	141	116	98	83	72	63	55	49	43	39	35	29	24
		L/360	♦♦♦	287	121	62	47	36	28	23	18	15	13	11	9	8	6	4
		L/240	♦♦♦	♦♦♦	182	93	70	54	42	34	28	23	19	16	14	12	9	7
		L/180	♦♦♦	♦♦♦	♦♦♦	124	93	72	56	45	37	30	25	21	18	15	12	9
	20	Stress	300	300	288	184	152	128	109	94	82	72	64	57	51	46	38	32
		L/360	♦♦♦	♦♦♦	150	77	58	44	35	28	23	19	16	13	11	10	7	6
		L/240	♦♦♦	♦♦♦	225	115	86	67	52	42	34	28	23	20	17	14	11	8
		L/180	♦♦♦	♦♦♦	♦♦♦	153	115	89	70	56	45	37	31	26	22	19	14	11
	18	Stress	300	300	300	251	208	174	149	128	112	98	87	78	70	63	52	44
		L/360	♦♦♦	♦♦♦	207	106	79	61	48	39	31	26	22	18	15	13	10	8
		L/240	♦♦♦	♦♦♦	♦♦♦	159	119	92	72	58	47	39	32	27	23	20	15	11
		L/180	♦♦♦	♦♦♦	♦♦♦	212	159	122	96	77	63	52	43	36	31	26	20	15
	16	Stress	300	300	300	300	264	222	189	163	142	125	110	99	88	80	66	55
		L/360	♦♦♦	♦♦♦	261	133	100	77	61	49	40	33	27	23	19	17	13	10
		L/240	♦♦♦	♦♦♦	♦♦♦	200	150	116	91	73	59	49	41	34	29	25	19	14
		L/180	♦♦♦	♦♦♦	♦♦♦	267	200	154	121	97	79	65	54	46	39	33	25	19
DOUBLE	22	Stress	300	300	235	150	124	104	89	77	67	59	52	46	42	38	31	26
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	122	94	74	59	48	40	33	28	24	20	15	12
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	49	42	35	30	23	18
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	30	23
	20	Stress	300	300	296	190	157	132	112	97	84	74	66	59	53	47	39	33
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	146	113	89	71	58	48	40	33	28	24	18	14
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	71	59	50	43	37	27	21
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	37	28
	18	Stress	300	300	300	265	219	184	157	135	118	103	92	82	73	66	55	46
		L/360	♦♦♦	♦♦♦	♦♦♦	258	194	149	117	94	76	63	53	44	38	32	24	19
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	115	94	79	66	56	48	36	28
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	64	48	37
	16	Stress	300	300	300	300	271	228	194	167	146	128	113	101	91	82	68	57
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	241	186	146	117	95	78	65	55	47	40	30	23
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	143	118	98	83	70	60	45	35
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	80	60	46
TRIPLE	22	Stress	300	300	294	188	155	131	111	96	84	73	65	58	52	47	39	33
		L/360	♦♦♦	♦♦♦	247	127	95	73	58	46	38	31	26	22	18	16	12	9
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	143	110	86	69	56	46	39	33	28	24	18	14
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	92	75	62	52	43	37	32	24	18
	20	Stress	300	300	300	237	196	165	140	121	105	93	82	73	66	59	49	41
		L/360	♦♦♦	♦♦♦	298	152	115	88	69	56	45	37	31	26	22	19	14	11
		L/240	♦♦♦	♦♦♦	♦♦♦	229	172	132	104	83	68	56	47	39	33	29	21	17
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	139	111	90	74	62	52	44	38	29	22
	18	Stress	300	300	300	300	274	230	196	169	147	129	115	102	92	83	68	57
		L/360	♦♦♦	♦♦♦	♦♦♦	202	152	117	92	74	60	49	41	35	29	25	19	15
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	228	175	138	110	90	74	62	52	44	38	28	22
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	184	147	120	99	82	69	59	50	38	29
	16	Stress	300	300	300	300	300	285	243	209	182	160	142	127	114	103	85	71
		L/360	♦♦♦	♦♦♦	♦♦♦	251	189	145	114	92	74	61	51	43	37	31	24	18
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	283	218	172	137	112	92	77	65	55	47	35	27
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	229	183	149	123	102	86	73	63	47	36

See footnotes on page 27.

Table 1: Allowable Tension Loads (lbs/connection) for Arc Spot Welds, Hilti Fasteners and Pneutek Fasteners Subject to Wind Uplift Loads for Verco B and N Steel Deck Panels

Gage	Profile	BMT	Arc Spot Weld	Hilti X-EDNK22 or X-HSN 24	Hilti X-ENP-19	Pneutek SDK61, SKD63, K64 or K66
		(in.)	(lbs)	(lbs)	(lbs)	(lbs)
22	B & N	0.0299	505	493	525	297
20	B & N	0.0359	602	592	631	429
18	B & N	0.0478	790	788	840	760
16	B & N	0.0598	975	985	1050	1190

Table 2: Allowable Tension Loads (lbs/connection) for #12 Screws Subject to Wind Uplift Loads for Verco Deck Panels

Gage	Profile	BMT	Support Thickness (in.) and Strength, F_y / F_u (ksi)											
			33 mil (0.0346 in.)		43 mil (0.0451 in.)		54 mil (0.0566 in.)		68 mil (0.0713 in.)		97 mil (0.1017 in.)		1/8 in.	≥3/16 in.
		(in.)	33/45	50/65	33/45	50/65	33/45	50/65	33/45	50/65	33/45	50/65	36/58	36/58
26	SV	0.0179	95	138	124	173	156	173	173	173	173	173	173	173
	DV	0.0195	95	138	124	179	156	189	189	189	189	189	189	189
24	SV	0.0239	95	138	124	179	156	225	196	232	232	232	232	232
	DV	0.0254	95	138	124	179	156	225	196	246	246	246	246	246
22	SV	0.0299	95	138	124	179	156	225	196	284	280	290	290	290
	B & N	0.0299	95	138	124	179	156	225	196	284	280	290	290	290
	DV	0.0314	95	138	124	179	156	225	196	284	280	304	304	304
20	B & N	0.0359	95	138	124	179	156	225	196	284	280	348	348	348
	DV	0.0374	95	138	124	179	156	225	196	284	280	362	362	362
18	B & N	0.0478	95	138	124	179	156	225	196	284	280	405	444	463
16	B & N	0.0598	95	138	124	179	156	225	196	284	280	405	444	579

Notes for Tables 1 and 2:

1. The profile designations used in this table apply to the profile families as summarized below:
   "SV" - Shallow VERCOR
   "DV" - Deep VERCOR
   "B" – PLB & HSB roof deck (including web perforated acoustical deck)
   "N" – PLN3, HSN3, HSN3-NS, PLN24 & N24 roof deck (including web perforated acoustical deck)
2. Base metal thickness (BMT) = specified minimum uncoated base metal thickness used in design. Deck subject to thickness tolerances as described in Section A2.4 of AISI S100.
3. The minimum arc spot weld effective fusion diameter, d_e, is 1/2 inch. The values for arc spot welds may be applied to arc seam weld with minimum effective fusion width, d_e, of 3/8 inch and minimum length is 1 inch excluding circular ends.
4. Details, workmanship, technique and qualification of welds must comply with AWS D1.3.
5. The Hilti fasteners are applicable to the following substrate thicknesses:
   X-EDNK22: 1/8 in. ≤ substrate thickness ≤ 1/4 in.
   X-HSN 24: 1/8 in. ≤ substrate thickness ≤ 3/8 in.
   X-ENP-19: substrate thickness ≥ 1/4 in.
6. The Pneutek fasteners are applicable to the following substrate thicknesses:
   SDK61 series: 0.113 in. ≤ substrate thickness ≤ 0.155 in.
   SDK63 series: 0.155 in. ≤ substrate thickness ≤ 0.250 in.
   K64 series: 0.187 in. ≤ substrate thickness ≤ 0.312 in.
   K66 series: substrate thickness ≥ 0.281 in.
7. The #12 screws are self-drilling self-tapping screws with a minimum washer diameter of 5/16-in. and a minimum washer thickness of 0.05 in. The screws must be compliant with ASTM C1513.
8. The allowable tensile strength of the individual screws, as published by their manufacturer, must meet or exceed the allowable screw connection tensile strengths listed above.
9. The strength is the ASD allowable connection tensile strength, where Ω is 2.5 for welds and 3.0 for Screws, Hilti and Pneutek fasteners. Convert ASD tensile strengths to LRFD based on Ø = 0.60 for welds and Ø = 0.50 for Screws, Hilti or Pneutek fasteners.

Table C2.1-3
United States and Mexico
Nominal Shear Strength (R_n) for Seismic and Other In-Plane Loads for Shear Walls [1,4,7,8]
(Pounds Per Foot)

Assembly Description	Max. Aspect Ratio (h/w)	Fastener Spacing at Panel Edges[2] (inches)				Designation Thickness[5,6] of Stud, Track and Blocking (mils)	Required Sheathing Screw Size
		6	4	3	2		
15/32" Structural 1 sheathing (4-ply), one side	2:1[3]	780	990	-	-	33 or 43	8
	2:1	890	1330	1775	2190	43 or 54	8
						68	10
7/16" OSB, one side	2:1[3]	700	915	-	-	33	8
	2:1[3]	825	1235	1545	2060	43 or 54	8
	2:1	940	1410	1760	2350	54	8
	2:1	1232	1848	2310	3080	68	10
0.018" steel sheet, one side	2:1	390	-	-	-	33 (min.)	8
0.027" steel sheet, one side	4:1	-	1000	1085	1170	43 (min.)	8
	2:1[3]	647	710	778	845	33 (min.)	8

1. *Nominal strength* shall be multiplied by the *resistance factor* (ϕ) to determine *design strength* or divided by the *safety factor* (Ω) to determine *allowable strength* as set forth in Section C2.1.
2. Screws in the field of the panel shall be installed 12 inches (305 mm) o.c. unless otherwise shown.
3. *Shear wall* height to width aspect ratios (h/w) greater than 2:1, but not exceeding 4:1, shall be permitted provided the *nominal strength* values are multiplied by 2w/h. See Section C2.1.
4. See Section C2.1 for requirements for sheathing applied to both sides of wall.
5. Unless noted as (min.), substitution of a *stud* or *track* of a different *designation thickness* is not permitted.
6. Wall *studs* and *track* shall be of ASTM A1003 Structural Grade 33 (Grade 230) Type H steel for members with a *designation thickness* of 33 and 43 mils, and A1003 Structural Grade 50 (Grade 340) Type H steel for members with a *designation thickness* equal to or greater than 54 mils.
7. For wood structural panel sheathed shear walls, tabulated R_n values applicable for short-term load duration (seismic loads). For other in-plane lateral loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above for wood structural panel sheathed shear walls shall be multiplied by 0.63 (normal) or 0.56 (permanent).
8. For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.

1. *Fiberboard* panels shall comply with AHA A194.1 or ASTM C 208.
2. *Nominal* shear strengths shall be given in Table C2.1-2.
3. *Fiberboard* shall be applied perpendicular to framing with strap *blocking* behind the horizontal joint and with solid *blocking* between the first two end *studs*, at each end of the wall, or applied vertically with all edges attached to framing members.
4. Screws used to attach *fiberboard* shall be a minimum No. 8 in accordance with ASTM C1513. Head style shall be selected to provide a flat bearing surface in contact with the sheathing with a head diameter not less than 0.43 inches (10.9 mm). Screws shall be driven so that their flat bearing surface is flush with the surface of the sheathing.

Table C2.1-1
United States and Mexico
Nominal Shear Strength (R_n) for Wind and Other In-Plane Loads for Shear Walls [1,4,6,7,8]
(Pounds Per Foot)

Assembly Description	Maximum Aspect Ratio (h/w)	Fastener Spacing at Panel Edges[2] (inches)			
		6	4	3	2
15/32" structural 1 sheathing (4-ply), one side	2:1	1065 [3]	-	-	-
7/16" rated sheathing (OSB), one side	2:1	910 [3]	1410	1735	1910
7/16" rated sheathing (OSB), one side oriented perpendicular to framing	2:1	1020	-	-	-
7/16" rated sheathing (OSB), one side	2:1 [5]	-	1025	1425	1825
0.018" steel sheet, one side	2:1	485	-	-	-
0.027" steel sheet, one side	4:1	-	1,000 [9]	1085 [9]	1170 [9]
	2:1 [5]	647	710	778	845

1. *Nominal strengths* shall be multiplied by the *resistance factor* (ϕ) to determine *design strength* or divided by the *safety factor* (Ω) to determine *allowable strengths* as set forth in Section C2.1.
2. Screws in the field of the panel shall be installed 12 inches (305 mm) o.c. unless otherwise shown.
3. Where fully blocked gypsum board is applied to the opposite side of this assembly, per Table C2.1-2 with screw spacing at 7 inches (178 mm) o.c. edge and 7 inches (178 mm) o.c. field, these *nominal strengths* are permitted to be increased by 30%.
4. See Section C2.1 for requirements for sheathing applied to both sides of wall.
5. *Shear wall* height to width aspect ratio's (h/w) greater than 2:1, but not exceeding 4:1, shall be permitted provided the *nominal strength* is multiplied by 2w/h. See Section C2.1.
6. Shear values are permitted for use in seismic design where the seismic response modification factor, R, is taken equal to or less than 3, subject to the limitations in Section C1.1.
7. For wood structural panel sheathed shear walls, tabulated R_n values shall be applicable for short-term load duration (wind loads). For other in-plane lateral loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above for wood structural panel sheathed shear walls shall be multiplied by 0.63 (normal) or 0.56 (permanent).
8. For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.
9. For these assemblies, the *designation thickness* of *stud*, *track* and *blocking* shall be a minimum of 43 mils.

Type HSB®-36

- **36/4 Weld Pattern at Supports**
- **Sidelaps connected with Button Punch or 1½" Top Seam Weld**

Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F ((in./lb)x10^6)

DECK GAGE	SIDELAP ATTACHMENT		SPAN (ft-in.)								
			4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
22	BP @ 24"	q	282	234	190	169	144	135	121		
		F	-1.3+267R	4.2+212R	9.1+174R	12.6+148R	16.3+127R	18.9+112R	22.1+98R		
	BP @ 12"	q	318	262	226	199	180	167	157		
		F	-2.3+267R	3.1+212R	7.2+175R	10.5+149R	13.3+129R	15.7+114R	17.9+101R		
	TSW @ 24"	q	628	649	562	588	526	552	505		
		F	-9.4+271R	-6.3+217R	-3.5+181R	-2.2+155R	-0.5+135R	0.1+120R	1.2+108R		
	TSW @ 18"	q	763	756	663	673	681	622	633		
		F	-10.2+271R	-6.9+217R	-4.2+181R	-2.8+155R	-1.7+136R	-0.4+121R	0.2+108R		
	TSW @ 12"	q	871	846	828	815	805	798	791		
		F	-10.7+272R	-7.3+217R	-5.1+181R	-3.5+155R	-2.3+136R	-1.3+121R	-0.6+109R		
	TSW @ 6"	q	1117	1107	1101	1096	1092	1089	1001		
		F	-11.6+272R	-8.2+217R	-6+181R	-4.4+155R	-3.2+136R	-2.3+121R	-1.5+109R		
20	BP @ 24"	q	403	336	275	246	211	195	175	169	155
		F	3.1+167R	7.2+132R	11.1+108R	13.8+91R	16.9+78R	19+68R	21.7+59R	23.4+53R	25.8+47R
	BP @ 12"	q	454	378	326	290	262	241	227	216	206
		F	2.2+168R	6.2+133R	9.3+109R	11.9+93R	14.2+80R	16.1+70R	17.8+62R	19.4+55R	20.8+50R
	TSW @ 24"	q	824	846	733	764	685	715	654	683	634
		F	-4.2+171R	-2.3+137R	-0.4+114R	0.3+98R	1.5+86R	1.9+76R	2.7+68R	2.8+62R	3.4+57R
	TSW @ 18"	q	993	981	861	872	879	804	818	829	774
		F	-5+172R	-2.9+137R	-1.1+114R	-0.2+98R	0.5+86R	1.4+76R	1.8+69R	2.1+62R	2.6+57R
	TSW @ 12"	q	1127	1093	1069	1051	1037	1026	1018	1010	912
		F	-5.5+172R	-3.3+137R	-1.9+115R	-0.8+98R	0+86R	0.6+76R	1+69R	1.4+62R	1.8+57R
	TSW @ 6"	q	1435	1422	1412	1406	1400	1396	1313	1085	912
		F	-6.2+172R	-4.1+138R	-2.7+115R	-1.7+98R	-0.9+86R	-0.3+76R	0.1+69R	0.5+63R	0.8+57R
18	BP @ 24"	q	704	592	487	438	379	353	314	300	275
		F	6.3+80R	9.1+63R	11.9+51R	13.9+42R	16.3+35R	17.8+30R	20+26R	21.3+22R	23.3+19R
	BP @ 12"	q	794	666	579	517	470	434	405	383	366
		F	5.5+81R	8.2+63R	10.4+52R	12.2+43R	13.9+37R	15.3+32R	16.7+28R	17.9+24R	19+22R
	TSW @ 24"	q	1272	1293	1121	1160	1040	1081	989	1028	955
		F	0+84R	0.8+67R	1.9+56R	2.2+48R	2.9+42R	3+37R	3.5+33R	3.5+30R	3.9+28R
	TSW @ 18"	q	1513	1486	1306	1316	1323	1210	1227	1241	1160
		F	-0.7+84R	0.3+67R	1.3+56R	1.7+48R	2+42R	2.6+37R	2.7+33R	2.9+30R	3.2+28R
	TSW @ 12"	q	1705	1648	1607	1577	1554	1535	1520	1508	1394
		F	-1.1+84R	-0.1+67R	0.7+56R	1.2+48R	1.6+42R	1.9+37R	2.1+34R	2.3+30R	2.5+28R
	TSW @ 6"	q	2150	2127	2111	2099	2090	2083	2007	1659	1394
		F	-1.8+84R	-0.8+67R	-0.1+56R	0.4+48R	0.8+42R	1.1+37R	1.3+34R	1.5+31R	1.7+28R
16	BP @ 24"	q	912	778	641	584	506	477	425	408	371
		F	7.1+44R	9.2+34R	11.5+27R	13.1+22R	15.1+18R	16.4+15R	18.3+12R	19.4+10R	21.2+8R
	BP @ 12"	q	1041	893	784	707	649	604	568	538	514
		F	6.4+45R	8.4+35R	10.1+28R	11.6+23R	13+19R	14.2+16R	15.3+14R	16.3+12R	17.3+10R
	TSW @ 24"	q	1643	1679	1460	1515	1361	1417	1299	1352	1257
		F	1.4+48R	1.8+38R	2.6+32R	2.7+27R	3.2+24R	3.2+21R	3.6+19R	3.5+17R	3.8+16R
	TSW @ 18"	q	1957	1929	1702	1718	1731	1586	1610	1630	1525
		F	0.8+48R	1.3+38R	2.1+32R	2.3+27R	2.4+24R	2.8+21R	2.9+19R	2.9+17R	3.2+16R
	TSW @ 12"	q	2203	2136	2088	2053	2026	2004	1986	1971	1941
		F	0.4+48R	1+38R	1.5+32R	1.8+27R	2+24R	2.2+21R	2.3+19R	2.4+17R	2.5+16R
	TSW @ 6"	q	2753	2727	2710	2696	2686	2678	2671	2310	1941
		F	-0.2+48R	0.4+38R	0.8+32R	1.1+27R	1.3+24R	1.5+21R	1.6+19R	1.7+17R	1.8+16R

See footnotes on page 28.

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Table IV – 9c

Screws

Nominal Shear Strength Limited by Tilting and Bearing, P_{nv}, kips[1]
Shear of Sheet - F_u = 45 ksi
Joist/Stud or Track Design Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

43 mil

Screw Designation	Diameter in.	Thickness of member in contact with screw head, in.	Thickness of member not in contact with the screw head, in.							
			0.0188	0.0283	0.0312	0.0346	0.0451	0.0566	0.0713	0.1017
#6	0.138	0.0188	0.181	0.315	0.315	0.315	0.315	0.315	0.315	0.315
		0.0283	0.181	0.334	0.393	0.455	0.475	0.475	0.475	0.475
		0.0312	0.181	0.334	0.387	0.457	0.523	0.523	0.523	0.523
		0.0346	0.181	0.334	0.387	0.452	0.580	0.580	0.580	0.580
		0.0451	0.181	0.334	0.387	0.452	0.672	0.756	0.756	0.756
		0.0566	0.181	0.334	0.387	0.452	0.672	0.945	0.949	0.949
		0.0713	0.181	0.334	0.387	0.452	0.672	0.945	1.20	1.20
		0.1017	0.181	0.334	0.387	0.452	0.672	0.945	1.20	1.71
#8	0.164	0.0188	0.197	0.368	0.375	0.375	0.375	0.375	0.375	0.375
		0.0283	0.197	0.364	0.432	0.503	0.564	0.564	0.564	0.564
		0.0312	0.197	0.364	0.422	0.502	0.622	0.622	0.622	0.622
		0.0346	0.197	0.364	0.422	0.493	0.689	0.689	0.689	0.689
		0.0451	0.197	0.364	0.422	0.493	0.733	0.899	0.899	0.899
		0.0566	0.197	0.364	0.422	0.493	0.733	1.03	1.13	1.13
		0.0713	0.197	0.364	0.422	0.493	0.733	1.03	1.42	1.42
		0.1017	0.197	0.364	0.422	0.493	0.733	1.03	1.42	2.03
#10	0.190	0.0188	0.212	0.406	0.434	0.434	0.434	0.434	0.434	0.434
		0.0283	0.212	0.392	0.468	0.548	0.653	0.653	0.653	0.653
		0.0312	0.212	0.392	0.454	0.544	0.720	0.720	0.720	0.720
		0.0346	0.212	0.392	0.454	0.530	0.791	0.799	0.799	0.799
		0.0451	0.212	0.392	0.454	0.530	0.789	1.04	1.04	1.04
		0.0566	0.212	0.392	0.454	0.530	0.789	1.11	1.31	1.31
		0.0713	0.212	0.392	0.454	0.530	0.789	1.11	1.57	1.65
		0.1017	0.212	0.392	0.454	0.530	0.789	1.11	1.57	2.35
#12	0.216	0.0188	0.226	0.444	0.488	0.493	0.493	0.493	0.493	0.493
		0.0283	0.226	0.418	0.502	0.592	0.743	0.743	0.743	0.743
		0.0312	0.226	0.418	0.484	0.584	0.819	0.819	0.819	0.819
		0.0346	0.226	0.418	0.484	0.565	0.855	0.908	0.908	0.908
		0.0451	0.226	0.418	0.484	0.565	0.841	1.18	1.18	1.18
		0.0566	0.226	0.418	0.484	0.565	0.841	1.18	1.49	1.49
		0.0713	0.226	0.418	0.484	0.565	0.841	1.18	1.67	1.87
		0.1017	0.226	0.418	0.484	0.565	0.841	1.18	1.67	2.67
1/4 in.	0.250	0.0188	0.244	0.491	0.543	0.571	0.571	0.571	0.571	0.571
		0.0283	0.244	0.450	0.544	0.646	0.860	0.860	0.860	0.860
		0.0312	0.244	0.450	0.521	0.633	0.918	0.948	0.948	0.948
		0.0346	0.244	0.450	0.521	0.608	0.935	1.05	1.05	1.05
		0.0451	0.244	0.450	0.521	0.608	0.905	1.29	1.37	1.37
		0.0566	0.244	0.450	0.521	0.608	0.905	1.27	1.72	1.72
		0.0713	0.244	0.450	0.521	0.608	0.905	1.27	1.80	2.17
		0.1017	0.244	0.450	0.521	0.608	0.905	1.27	1.80	3.06

10ga = 0.128"
12ga = 0.104"
14ga = 0.080"
16ga = 0.064"
18ga = 0.048"
20ga = 0.036"
22ga = 0.028"
26ga = 0.018"
29ga = 0.014"

Table IV - 10a

Screws
Pull-Out - F_u = 45 ksi
Purlin/Girt Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Nominal Pull-Out Strength, P_{not}, kips[1]

Screw Designation	Diameter in.	Thickness of member not in contact with the screw head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.138	0.190	0.253	0.317	0.396	0.475	0.554	0.713
#8	0.164	0.226	0.301	0.376	0.470	0.565	0.659	0.847
#10	0.190	0.262	0.349	0.436	0.545	0.654	0.763	0.981
#12	0.216	0.297	0.397	0.496	0.620	0.744	0.868	1.12
1/4 in.	0.250	0.344	0.459	0.574	0.717	0.861	1.00	1.29

a shear specimen and joint behavior is dependent upon the shear strength of the fastener. Shear failure of the screw shank occurs at the wood-metal interface.

c) The metal-critical joint may fail in one of two ways. Failure occurs when the resistance of the screw head to embedment is greater than the resistance of the metal to lateral and/or withdrawal load, and the screw tears through or away from the metal. Failure also occurs when thin metal in a metal-to-panel joint crushes or tears away from the screw.

The following test data are presented for **plywood** only.

Tables 1 and 2 present average ultimate lateral loads for wood- and sheet-metal-screw connections in plywood-and-metal joints. The end distance of the loaded-edge in these tests was one inch. Plywood face grain was parallel to the load since this direction yields the lowest lateral loads when the joint is plywood-critical. All wood-screw specimens were tested with a 3/16-inch-thick steel side plate, and values should be modified if thinner steel is used.

TABLE 1

SCREWS: METAL-TO-PLYWOOD CONNECTIONS[a]

Depth of Threaded Penetration (inch)	Average Ultimate Lateral Load (lbf)[b]					
	Wood Screws			Sheet Metal Screws		
	#8	#10	#12	#8	#10	#12
1/2	415	(500)	590	465	(565)	670
5/8	–	–	–	500	(600)	705
3/4	–	–	–	590	(655)	715

(a) Plywood was C-D grade with exterior glue (all plies Group 1), face grain parallel to load. Side plate was 3/16"-thick steel.

(b) Values are **not** design values. Values in parentheses are estimates based on other tests.

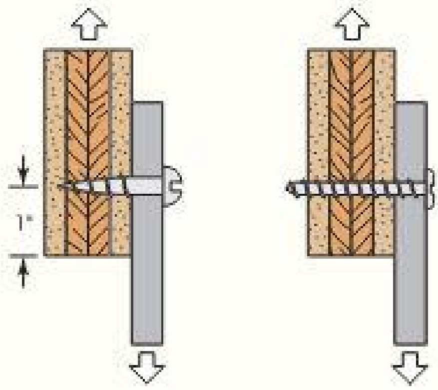


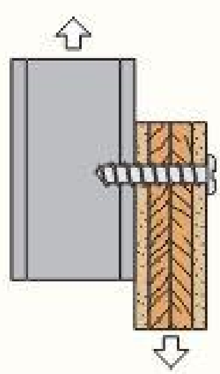
TABLE 2

SHEET METAL SCREWS: PLYWOOD-TO-METAL CONNECTIONS[a]

Framing	Plywood Performance Category	Average Ultimate Lateral Load (lbf)[b]				
		Screw Size				1/4"-20 Self Tapping Screw
		#8	#10	#12	#14	
0.080-inch Aluminum	1/4	330	360	390	410	590
	1/2	630	850*	860	920	970
	3/4	910*	930*	1250	1330	1440
0.078-inch Galvanized Steel (14 gage)	1/4	360	380	400	410	650
	1/2	700*	890*	900	920	970
	3/4	700*	950*	1300*	1390*	1500

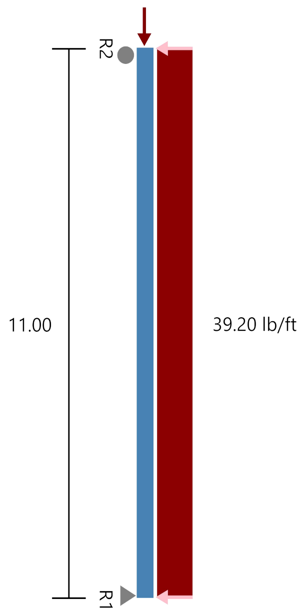
(a) Plywood was A-C EXT (all plies Group 1), face grain parallel to load.

(b) Values are **not** design values. Loads denoted by an asterisk(*) were limited by screw-to-framing strength; others were limited by plywood strength.



Project Name: 22-04-13_cold formed steel calc_eh
Model: Wall Stud – PH
Code: 2012 NASPEC [AISI S100-2012]

Date: 04/28/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Section : 600S162-43 (33 ksi) @ 16" o.c. Single C Stud (punched)		
Maxo = 1390.0 ft-lb	**Va =** 1415.7 lb	**I =** 2.32 in^4

Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	48.0", 48.0"	48.0", 132.0"	LSUBH3.25 (Min)	0.66

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	215.6	--Shear Connection w/ clip--				NO
R1	215.6	--Stud/Track Design, Ref Connectors--				NO

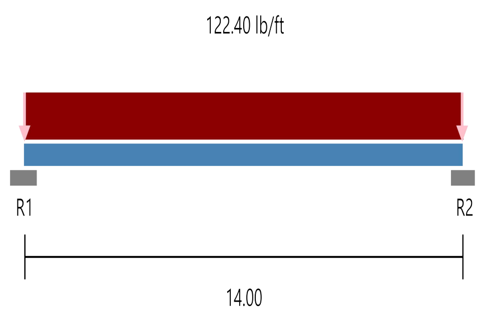
Gravity Load

Type	Load (lb)
Uniform	0.00plf
P1y	433lb @ 11ft

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	433.0(c)	3507.8(c)	12%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	215.6	1240.3	17%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	592.9	1205.1	49%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	592.9	1221.7	49%	
	Shear/Moment	0.43	1.00	43%	Shear 0.0, Moment 592.9
	Axial/Moment	0.62	1.00	62%	Axial 433.0(c), Moment 592.9
	Deflection Span, in	0.132	--meets L/998--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	215.6	0.0	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	35.34 %	19.34 %
R1	215.6	433.0	600T125-33 (33) & (1) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	52.58 %	97.91 %

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements



Reactions

Support	Reactions (lb)
R2	856.80
R1	856.80

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	30.9% Stressed @R2
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	Yes @R1,R2

Section: 1200S162-68 (50 ksi) Single C Stud (punched)
Maxo = 6599.5 ft-lb **Va =** 2770.7 lb **I =** 18.39 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	2998.8	0.454	2998.8	48.0	5659.2	0.530	0.195	L/861

Distortional Buckling

	K-phi (lb-in/in)	Lm brace (in)	Ma-dist (ft-lb)	Mmax/ Ma-dist
Span	0.00	168.0	5511.5	0.544

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		Checked	
Project No.	40.2021.105	Date	

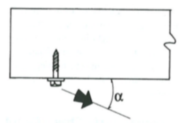
~ COMBINED SHEAR AND TENSION CONNECTION PER NDS 2018, CHAPTER 12 ~

INPUT: Design for shear wall loading (track to blocking)

Method: ASD

Forces:

Tension/Withdrawal:	R_t =	0.0 lbs
Shear:	R_s =	269.0 lbs
Resultant Load:	R_R =	269.0 lbs
Direction:	α =	0.0°



0 ° is Shear Only
90° is Tension Only

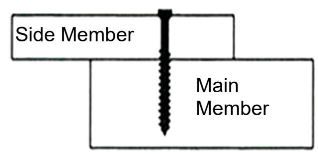
Fastener: Lag Screw

# of Fasteners, *n* =	3
Size:	1/4 in
Length, *L* =	4.00 in
Nominal Diameter[B], *D* =	0.250 in
Head Diameter, D_H =	NA

Wood:

Side Member Material[C]:	Steel: Gr. 33
Main Member Material[C]:	Wood

		NDS Ref.:
ASD		
Tension Load Duration:	10 Min/Wind or EQ Load	11.3.2
Shear Load Duration:	10 Min/Wind or EQ Load	11.3.2
ASD & LRFD		
Moisture Content[A]:	≤19% @ fab. & ≤19% in service	11.3.3
Temperature:	In Service Dry & Temp.≤100°F	11.3.4
Group Action:	Input C_g below	11.3.6
Geometry:	Input C_Δ below	12.5.1
End Grain:	NO End Grain	12.5.2
Diaphragm:	NOT a Diaphragm	12.5.3
Toe-nail:	NO Toe-Nailing	12.5.4
LRFD		
Format Conversion:	2.16/Φ	11.3.7
Φ Resistance:	0.65	11.3.8
Time Effect:	λ approximated below, see Appendix N.3.3	11.3.9



Side Member Specific Gravity, G_s=	0.46	Ref Section 11.2.3, NA
Main Member Specific Gravity, G_m=	0.46	

Lead and Clearance Holes Required 12.1.4.2

Side Member Thickness, t_{ns}=	0.05 in	
Main Member Thickness, t_m=	9.25 in	Main Member Pen: 3.96 in *(15.8 D)*
*Only used for toe-nailing:	2.00 in	

Shear:

Main Member Shear Action Angle, θ_m=	0°	0° = // to grain
Side Member Shear Action Angle, θ_s=	0°	90° = ⊥ to grain

Connection OK 44%

Adjustment Factors: NDS Pg #

	Tension/Withdrawal	Shear		NDS Pg #	NDS Ref.:
Load Duration Factor:	C_D = 1.60	C_D = 1.60	ASD only	pg. 11	11.3.2
Wet Service Factor:	C_M = 1.00	$C_M{}^1$ = 1.00	ASD & LRFD	pg. 61	10.3.3 & 11.5.4
Temperature Factor:	C_t = 1.00	C_t = 1.00	ASD & LRFD	pg. 66	11.3.4
Group Action Factor:	C_g = 1.00	C_g = 1.00	ASD & LRFD	pg. 68-72	11.3.6
Geometry Factor:	C_Δ = 1.00	C_Δ = 1.00	ASD & LRFD	pg. 89-91	12.5.1
End Grain Factor:	C_{eg} = 1.00	C_{eg} = 1.00	ASD & LRFD	pg. 91	12.5.2
Diaphragm Factor:	C_{di} = 1.00	C_{di} = 1.00	ASD & LRFD	pg. 91	12.5.3
Toe-nail Factor:	C_{tn} = 1.00	C_{tn} = 1.00	ASD & LRFD	pg. 91	12.5.4
Format Conversion Factc	K_F = 3.32	K_F = 3.32	LRFD only	pg. 187	11.3.7
Resistance Factor:	Φ_z = 0.65	Φ_z = 0.65	LRFD only	pg. 187	11.3.8
Time Effect Factor:	λ = 1.00	λ = 1.00	LRFD only	pg. 187	11.3.9

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		Checked	
Project No.	40.2021.105	Date	

~ COMBINED SHEAR AND TENSION CONNECTION PER NDS 2018, CHAPTER 12 ~

INPUT: Design for shear wall loading (track to blocking)

Withdrawal Capacity: (NDS Ref.:)

NDS Ref.: 12.2

Withdrawal Values, W		
Lag Screw	Wood Screw	Nail
199 lbs/in	N/A	N/A

NDS Ref.: Eq 12.2-1, 12.2-2, & 12.2-3

Length of Fastener Engaged/Thread Length, T			
Lag Screw		Wood Screw	Nail
T	E	T	T
2.500 in	0.156 in	N/A	N/A

NDS Ref.: Appendix L

W= 199 lbs/in/fastener

W' = Cd*Cm*Ct*Cg*CΔ*Ceg*Ctn*W*n= 953 lbs/in

Unthreaded Shank Length: S = L-T= 1.50 in

Max Penetration allowed by Fastener: p_{max} = IF(Lag Screw, use T-E; else T)= 2.34 in

Penetration into Main Member: p = MIN(t_m, IF(t_{ns}<S, MIN(t_{sm}+t_m-S, p_{max}), S+p_{max}-t_{sm})= 2.34 in — 12.1.4.6

W' = W * p = 2234 lbs

12.2.5

Pull-Through Values, W_H		
Lag Screw	Wood Screw	Nail
NA	N/A	N/A

W_H = 0 lbs/fastener

WH' = WH*Cd*Cm*Ct*n = NA

Controlling W' = 2234 lbs

Shear Capacity: (NDS Ref.:)

12.3.2

p_m= 3.955 in
D_r = 0.173 in
$F_{em\perp}$= 3,977 psi
$F_{es\perp}$= 61,850 psi
R_e= 0.064
$F_{em\theta}$= 3,977 psi

ℓ_s= 0.045 in
F_{yb}= 70,000 psi
$F_{em\parallel}$= 3,977 psi
$F_{es\parallel}$= 61,850 psi
R_t= 87.889
$F_{es\theta}$= 61,850 psi

ℓ_m=

Lag Screw*	Wood Screw*	Nail
3.96 in	N/A	N/A

*screw tapered tip not included when P<10D

Tbl 12.3.2

Yield Mode — Tbl 12.3.1A

I_m: $Z = \dfrac{D\ell_m F_{em}}{R_d}$ R_d= 2.230 Z= 1220 lbs/fastener

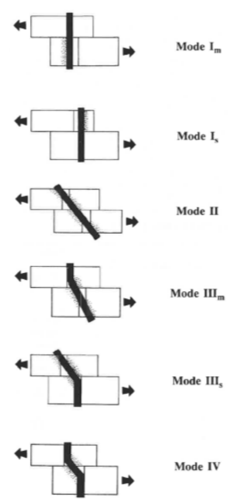
I_s: $Z = \dfrac{D\ell_s F_{es}}{R_d}$ R_d= 2.230 Z= 216 lbs/fastener

II: $Z = \dfrac{k_1 D\ell_s F_{es}}{R_d}$ k_1= 2.305, R_d= 2.230 Z= 498 lbs/fastener

III_m: $Z = \dfrac{k_2 D\ell_m F_{em}}{(1+2R_e)R_d}$ k_2= 0.468, R_d= 2.230 Z= 506 lbs/fastener

III_s: $Z = \dfrac{k_3 D\ell_s F_{em}}{(2+R_e)R_d}$ k_3= 18.776, R_d= 2.230 Z= 126 lbs/fastener

IV: $Z = \dfrac{D^2}{R_d}\sqrt{\dfrac{2F_{em}F_{yb}}{3(1+R_e)}}$ R_d= 2.230 Z= 177 lbs/fastener



Controlling Z= 126 lbs/fastener — 12.3

Z' = Cd*Cm*Ct*Cg*CΔ*Ceg*Ctn*Z*n= 606 lbs

Combined Capacity: (NDS Ref.:)

$Z_{\alpha}'_{Screws}$: $Z'_{\alpha} = \dfrac{(W'p)Z'}{(W'p)\cos^2\alpha + Z'\sin^2\alpha}$ **606 lbs** — 12.4-1

$Z_{\alpha}'_{Nails}$: $Z'_{\alpha} = \dfrac{(W'p)Z'}{(W'p)\cos\alpha + Z'\sin\alpha}$ N/A — 12.4-2

A. If Moisture Content is >19% at time of fabrication and ≤19% in service, 0.4 is a conservative factor. For a more precise factor see NDS 11.3.3.

B. Diameter of lag screws are for "reduced body diameter" lag screws. (Similar to tables 11J & 11k in NDS)

C. The adujsted values for $F_{e\perp}$ and $F_{e\parallel}$ are not supported by NDS for diameters of fasteners >1/4" in plywood or OSB & therefore not recommended. F_e is assumed for all fastener sizes herein.

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		Checked	
Project No.	40.2021.105	Date	

~ COMBINED SHEAR AND TENSION CONNECTION PER NDS 2018, CHAPTER 12 ~

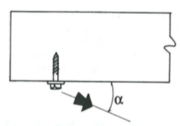
INPUT: **Shear and tension at track to blocking connection**

Method: **ASD**

Forces:

Tension/Withdrawal:	R_t =	1710.0 lbs
Shear:	R_s =	269.0 lbs
Resultant Load:	R_R =	1731.0 lbs
Direction:	α =	81.1°

0 ° is Shear Only
90° is Tension Only



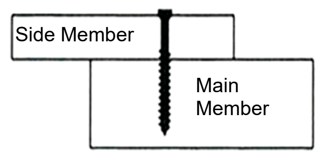
Fastener: Lag Screw

# of Fasteners, *n* =	3
Size:	1/4 in
Length, *L* =	4.00 in
Nominal Diameter[B], *D* =	0.250 in
Head Diameter, D_H =	NA

Wood:

Side Member Material[C]:	Steel: Gr. 33
Main Member Material[C]:	Wood

ASD	Tension Load Duration:	10 Min/Wind or EQ Load	11.3.2
ASD	Shear Load Duration:	10 Min/Wind or EQ Load	11.3.2
ASD & LRFD	Moisture Content[A]:	≤19% @ fab. & ≤19% in service	11.3.3
ASD & LRFD	Temperature:	In Service Dry & Temp.≤100°F	11.3.4
ASD & LRFD	Group Action:	Input C_g below	11.3.6
ASD & LRFD	Geometry:	Input C_Δ below	12.5.1
ASD & LRFD	End Grain:	NO End Grain	12.5.2
ASD & LRFD	Diaphragm:	NOT a Diaphragm	12.5.3
ASD & LRFD	Toe-nail:	NO Toe-Nailing	12.5.4
LRFD	Format Conversion:	2.16/Φ	11.3.7
LRFD	Φ Resistance:	0.65	11.3.8
LRFD	Time Effect:	λ approximated below, see Appendix N.3.3	11.3.9



Side Member Specific Gravity, G_s=	0.46	Ref Section 11.2.3, NA
Main Member Specific Gravity, G_m=	0.46	

No Lead Hole Required 12.1.4.3

Side Member Thickness, t_{ns}=	0.05 in	
Main Member Thickness, t_m=	9.25 in	Main Member Pen: 3.96 in *(15.8 D)*
*Only used for toe-nailing:	2.00 in	

Shear:

Main Member Shear Action Angle, θ_m=	0°	0° = // to grain
Side Member Shear Action Angle, θ_s=	0°	90° = ⊥ to grain

Connection OK 83%

Adjustment Factors:

	Tension/Withdrawal	Shear		NDS Pg #	NDS Ref.:
Load Duration Factor:	C_D = 1.60	C_D = 1.60	ASD only	pg. 11	11.3.2
Wet Service Factor:	C_M = 1.00	$C_M{}^1$ = 1.00	ASD & LRFD	pg. 61	10.3.3 & 11.5.4
Temperature Factor:	C_t = 1.00	C_t = 1.00	ASD & LRFD	pg. 66	11.3.4
Group Action Factor:	C_g = 1.00	C_g = 1.00	ASD & LRFD	pg. 68-72	11.3.6
Geometry Factor:	C_Δ = 1.00	C_Δ = 1.00	ASD & LRFD	pg. 89-91	12.5.1
End Grain Factor:	C_{eg} = 1.00	C_{eg} = 1.00	ASD & LRFD	pg. 91	12.5.2
Diaphragm Factor:	C_{di} = 1.00	C_{di} = 1.00	ASD & LRFD	pg. 91	12.5.3
Toe-nail Factor:	C_{tn} = 1.00	C_{tn} = 1.00	ASD & LRFD	pg. 91	12.5.4
Format Conversion Factc	K_F = 3.32	K_F = 3.32	LRFD only	pg. 187	11.3.7
Resistance Factor:	Φ_z = 0.65	Φ_z = 0.65	LRFD only	pg. 187	11.3.8
Time Effect Factor:	λ = 1.00	λ = 1.00	LRFD only	pg. 187	11.3.9

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~ COMBINED SHEAR AND TENSION CONNECTION PER NDS 2018, CHAPTER 12 ~

INPUT: **Shear and tension at track to blocking connection**

Withdrawal Capacity: (NDS Ref.: 12.2)

Withdrawal Values, W		
Lag Screw	Wood Screw	Nail
199 lbs/in	N/A	N/A

NDS Ref.: Eq 12.2-1, 12.2-2, & 12.2-3

Length of Fastener Engaged/Thread Length, T			
Lag Screw		Wood Screw	Nail
T	E	T	T
2.500 in	0.156 in	N/A	N/A

NDS Ref.: Appendix L

W= 199 lbs/in/fastener

W' = Cd*Cm*Ct*Cg*CΔ*Ceg*Ctn*W*n= 953 lbs/in

Unthreaded Shank Length: S = L-T= 1.50 in

Max Penetration allowed by Fastener: p_{max} = IF(Lag Screw, use T-E; else T)= 2.34 in

Penetration into Main Member: p = MIN(t_m, IF(t_{ns}<S, MIN(t_{sm}+t_m-S, p_{max}), S+p_{max}-t_{sm})= 2.34 in (12.1.4.6)

W' = W * p = 2234 lbs

Pull-Through Values, W_H		
Lag Screw	Wood Screw	Nail
NA	N/A	N/A

(12.2.5)

W_H = 0 lbs/fastener

WH' = WH*Cd*Cm*Ct*n = NA

Controlling W' = 2234 lbs

Shear Capacity: (NDS Ref.: 12.3.2)

p_m= 3.955 in	ℓ_s= 0.045 in
D_r = 0.173 in	F_{yb}= 70,000 psi
$F_{em\perp}$= 3,977 psi	$F_{em\parallel}$= 3,977 psi
$F_{es\perp}$= 61,850 psi	$F_{es\parallel}$= 61,850 psi
R_e= 0.064	R_t= 87.889
$F_{em\theta}$= 3,977 psi	$F_{es\theta}$= 61,850 psi

ℓ_m=	Lag Screw*	Wood Screw*	Nail
	3.96 in	N/A	N/A

*screw tapered tip not included when P<10D (Tbl 12.3.2)

Yield Mode (Tbl 12.3.1A)

I_m: $Z = \dfrac{D\ell_m F_{em}}{R_d}$ — R_d= 2.230 — Z= 1220 lbs/fastener

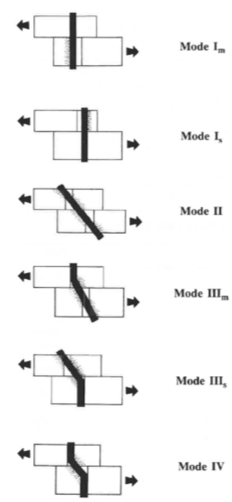
I_s: $Z = \dfrac{D\ell_s F_{es}}{R_d}$ — R_d= 2.230 — Z= 216 lbs/fastener

II: $Z = \dfrac{k_1 D\ell_s F_{es}}{R_d}$ — k_1= 2.305, R_d= 2.230 — Z= 498 lbs/fastener

III_m: $Z = \dfrac{k_2 D\ell_m F_{em}}{(1+2R_e)R_d}$ — k_2= 0.468, R_d= 2.230 — Z= 506 lbs/fastener

III_s: $Z = \dfrac{k_3 D\ell_s F_{em}}{(2+R_e)R_d}$ — k_3= 18.776, R_d= 2.230 — Z= 126 lbs/fastener

IV: $Z = \dfrac{D^2}{R_d}\sqrt{\dfrac{2F_{em}F_{yb}}{3(1+R_e)}}$ — R_d= 2.230 — Z= 177 lbs/fastener



Controlling Z= 126 lbs/fastener (12.3)

Z' = Cd*Cm*Ct*Cg*CΔ*Ceg*Ctn*Z*n= 606 lbs

Combined Capacity: (NDS Ref.:)

$Z_\alpha'_{Screws}$: $$Z'_\alpha = \frac{(W'p)Z'}{(W'p)\cos^2\alpha + Z'\sin^2\alpha}$$ **2098 lbs** (12.4-1)

$Z_\alpha'_{Nails}$: $$Z'_\alpha = \frac{(W'p)Z'}{(W'p)\cos\alpha + Z'\sin\alpha}$$ N/A (12.4-2)

A. If Moisture Content is >19% at time of fabrication and ≤19% in service, 0.4 is a conservative factor. For a more precise factor see NDS 11.3.3.

B. Diameter of lag screws are for "reduced body diameter" lag screws. (Similar to tables 11J & 11k in NDS)

C. The adujsted values for $F_{e\perp}$ and $F_{e\parallel}$ are not supported by NDS for diameters of fasteners >1/4" in plywood or OSB & therefore not recommended.
F_e is assumed for all fastener sizes herein.

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Engineer:
Project ID:
Project Descr:

Steel Beam

File: snowden.ec6

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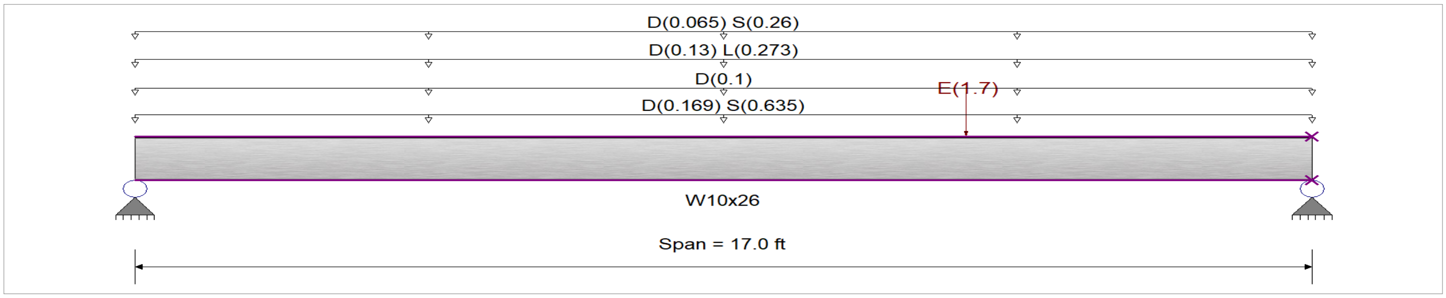
DESCRIPTION: Grid E Girder - W/ holdown load

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	36.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.1690, S = 0.6350 k/ft, Tributary Width = 1.0 ft, (load from joists with drift)

Uniform Load : D = 0.10 k/ft, Tributary Width = 1.0 ft, (wall weight)

Uniform Load : D = 0.020, L = 0.0420 ksf, Tributary Width = 6.50 ft, (PH Floor)

Uniform Load : D = 0.010, S = 0.040 ksf, Tributary Width = 6.50 ft, (PH Roof)

Point Load : E = 1.70 k @ 12.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.918 : 1	Maximum Shear Stress Ratio =	0.317 : 1
Section used for this span	W10x26	Section used for this span	W10x26
Ma : Applied	51.600 k-ft	Va : Applied	12.240 k
Mn / Omega : Allowable	56.228 k-ft	Vn/Omega : Allowable	38.563 k
Load Combination	+D+0.750L+0.750S+0.5250E	Load Combination	+D+0.750L+0.750S+0.5250E
Location of maximum on span	8.694 ft	Location of maximum on span	17.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.404 in	Ratio =	504 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.647 in	Ratio =	315 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 17.00 ft	1	0.315	0.108	17.70		17.70	93.90	56.23	1.00	1.00	4.16	57.84	38.56
+D+L													
Dsgn. L = 17.00 ft	1	0.490	0.168	27.56		27.56	93.90	56.23	1.00	1.00	6.48	57.84	38.56
+D+S													
Dsgn. L = 17.00 ft	1	0.890	0.305	50.03		50.03	93.90	56.23	1.00	1.00	11.77	57.84	38.56
+D+0.750L													
Dsgn. L = 17.00 ft	1	0.446	0.153	25.09		25.09	93.90	56.23	1.00	1.00	5.90	57.84	38.56
+D+0.750L+0.750S													
Dsgn. L = 17.00 ft	1	0.878	0.301	49.34		49.34	93.90	56.23	1.00	1.00	11.61	57.84	38.56
+D+0.70E													
Dsgn. L = 17.00 ft	1	0.370	0.130	20.80		20.80	93.90	56.23	1.00	1.00	5.00	57.84	38.56
+D+0.750L+0.750S+0.5250E													

Steel Beam

DESCRIPTION: Grid E Girder - W/ holdown load

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 17.00 ft	1	0.918	0.317	51.60		51.60	93.90	56.23	1.00	1.00	12.24	57.84	38.56
+0.60D													
Dsgn. L = 17.00 ft	1	0.189	0.065	10.62		10.62	93.90	56.23	1.00	1.00	2.50	57.84	38.56
+0.60D+0.70E													
Dsgn. L = 17.00 ft	1	0.245	0.087	13.80		13.80	93.90	56.23	1.00	1.00	3.34	57.84	38.56

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S+0.5250E	1	0.6471	8.549		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	11.873	12.240
Overall MINimum	0.500	1.200
D Only	4.164	4.164
+D+L	6.485	6.485
+D+S	11.772	11.772
+D+0.750L	5.905	5.905
+D+0.750L+0.750S	11.610	11.610
+D+0.70E	4.514	5.004
+D+0.750L+0.750S+0.5250E	11.873	12.240
+0.60D	2.499	2.499
+0.60D+0.70E	2.849	3.339
L Only	2.321	2.321
S Only	7.608	7.608
E Only	0.500	1.200

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~ COMBINED SHEAR AND TENSION CONNECTION PER NDS 2018, CHAPTER 12 ~

INPUT: **wind uplift and shear check - track to blocking**

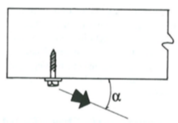
Method: **ASD**

Forces:

Tension/Withdrawal:	R_t =	323.0 lbs
Shear:	R_s =	269.0 lbs
Resultant Load:	R_R =	420.3 lbs
Direction:	α =	50.2°

0 ° is Shear Only

90° is Tension Only



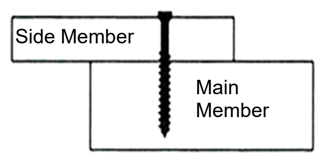
			NDS Ref.:
ASD	Tension Load Duration:	10 Min/Wind or EQ Load	11.3.2
ASD	Shear Load Duration:	10 Min/Wind or EQ Load	11.3.2
ASD & LRFD	Moisture Content[A]:	≤19% @ fab. & ≤19% in service	11.3.3
ASD & LRFD	Temperature:	In Service Dry & Temp.≤100°F	11.3.4
ASD & LRFD	Group Action:	Input C_g below	11.3.6
ASD & LRFD	Geometry:	Input C_Δ below	12.5.1
ASD & LRFD	End Grain:	NO End Grain	12.5.2
ASD & LRFD	Diaphragm:	NOT a Diaphragm	12.5.3
ASD & LRFD	Toe-nail:	NO Toe-Nailing	12.5.4
LRFD	Format Conversion:	2.16/Φ	11.3.7
LRFD	Φ Resistance:	0.65	11.3.8
LRFD	Time Effect:	λ approximated below, see Appendix N.3.3	11.3.9

Fastener: Lag Screw

# of Fasteners, *n* =	3
Size:	1/4 in
Length, *L* =	4.00 in
Nominal Diameter[B], *D* =	0.250 in
Head Diameter, D_H =	NA

Wood:

Side Member Material[C]:	Steel: Gr. 33
Main Member Material[C]:	Wood



Side Member Specific Gravity, G_s=	0.46	Ref Section 11.2.3, NA
Main Member Specific Gravity, G_m=	0.46	

No Lead Hole Required 12.1.4.3

Side Member Thickness, t_{ns}=	0.05 in	
Main Member Thickness, t_m=	9.25 in	Main Member Pen: 3.96 in *(15.8 D)*
*Only used for toe-nailing:	2.00 in	

Shear:

Main Member Shear Action Angle, θ_m=	0°	0° = // to grain
Side Member Shear Action Angle, θ_s=	0°	90° = ⊥ to grain

Connection OK 40%

Adjustment Factors: NDS Pg # **NDS Ref.:**

	Tension/Withdrawal	Shear		NDS Pg #	NDS Ref.
Load Duration Factor:	C_D = 1.60	C_D = 1.60	ASD only	pg. 11	11.3.2
Wet Service Factor:	C_M = 1.00	C_M^1 = 1.00	ASD & LRFD	pg. 61	10.3.3 & 11.5.4
Temperature Factor:	C_t = 1.00	C_t = 1.00	ASD & LRFD	pg. 66	11.3.4
Group Action Factor:	C_g = 1.00	C_g = 1.00	ASD & LRFD	pg. 68-72	11.3.6
Geometry Factor:	C_Δ = 1.00	C_Δ = 1.00	ASD & LRFD	pg. 89-91	12.5.1
End Grain Factor:	C_{eg} = 1.00	C_{eg} = 1.00	ASD & LRFD	pg. 91	12.5.2
Diaphragm Factor:	C_{di} = 1.00	C_{di} = 1.00	ASD & LRFD	pg. 91	12.5.3
Toe-nail Factor:	C_{tn} = 1.00	C_{tn} = 1.00	ASD & LRFD	pg. 91	12.5.4
Format Conversion Factc	K_F = 3.32	K_F = 3.32	LRFD only	pg. 187	11.3.7
Resistance Factor:	Φ_z = 0.65	Φ_z = 0.65	LRFD only	pg. 187	11.3.8
Time Effect Factor:	λ = 1.00	λ = 1.00	LRFD only	pg. 187	11.3.9

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~ COMBINED SHEAR AND TENSION CONNECTION PER NDS 2018, CHAPTER 12 ~

INPUT: wind uplift and shear check - track to blocking

Withdrawal Capacity:

NDS Ref.: 12.2

Withdrawal Values, W		
Lag Screw	Wood Screw	Nail
199 lbs/in	N/A	N/A

NDS Ref.: Eq 12.2-1, 12.2-2, & 12.2-3

Length of Fastener Engaged/Thread Length, T			
Lag Screw		Wood Screw	Nail
T	E	T	T
2.500 in	0.156 in	N/A	N/A

NDS Ref.: Appendix L

W= 199 lbs/in/fastener

W' = Cd*Cm*Ct*Cg*CΔ*Ceg*Ctn*W*n= 953 lbs/in

Unthreaded Shank Length: S = L-T= 1.50 in

Max Penetration allowed by Fastener: p_{max} = IF(Lag Screw, use T-E; else T)= 2.34 in

Penetration into Main Member: p = MIN(t_m, IF(t_{ns}<S, MIN(t_{sm}+t_m-S, p_{max}), S+p_{max}-t_{sm})= 2.34 in — 12.1.4.6

W' = W * p = 2234 lbs

12.2.5

Pull-Through Values, W_H		
Lag Screw	Wood Screw	Nail
NA	N/A	N/A

WH' = WH*Cd*Cm*Ct*n = NA

W_H = 0 lbs/fastener

Controlling W' = 2234 lbs

Shear Capacity:

NDS Ref.: 12.3.2

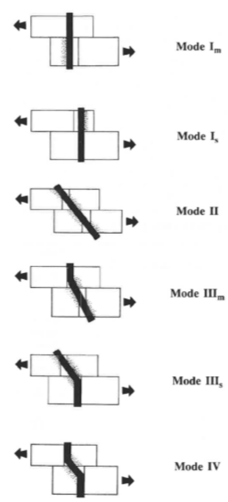
p_m= 3.955 in
ℓ_s= 0.045 in
D_r = 0.173 in
F_{yb}= 70,000 psi
$F_{em\perp}$= 3,977 psi
$F_{em||}$= 3,977 psi
$F_{es\perp}$= 61,850 psi
$F_{es||}$= 61,850 psi
R_e= 0.064
R_t= 87.889
$F_{em\theta}$= 3,977 psi
$F_{es\theta}$= 61,850 psi

ℓ_m=

Lag Screw*	Wood Screw*	Nail
3.96 in	N/A	N/A

*screw tapered tip not included when P<10D — Tbl 12.3.2

Yield Mode — Tbl 12.3.1A



$\mathrm{I_m}$ $$Z=\frac{D\ell_m F_{em}}{R_d}$$ R_d= 2.230 Z= 1220 lbs/fastener

$\mathrm{I_s}$ $$Z=\frac{D\ell_s F_{es}}{R_d}$$ R_d= 2.230 Z= 216 lbs/fastener

II $$Z=\frac{k_1 D\ell_s F_{es}}{R_d}$$ k_1= 2.305, R_d= 2.230 Z= 498 lbs/fastener

$\mathrm{III_m}$ $$Z=\frac{k_2 D\ell_m F_{em}}{(1+2R_e)R_d}$$ k_2= 0.468, R_d= 2.230 Z= 506 lbs/fastener

$\mathrm{III_s}$ $$Z=\frac{k_3 D\ell_s F_{em}}{(2+R_e)R_d}$$ k_3= 18.776, R_d= 2.230 Z= 126 lbs/fastener

IV $$Z=\frac{D^2}{R_d}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$$ R_d= 2.230 Z= 177 lbs/fastener

Controlling Z= 126 lbs/fastener — 12.3

Z' = Cd*Cm*Ct*Cg*CΔ*Ceg*Ctn*Z*n= 606 lbs

Combined Capacity:

NDS Ref.:

Z_α'Screws: $$Z_\alpha'=\frac{(W'p)Z'}{(W'p)\cos^2\alpha+Z'\sin^2\alpha}$$ **1064 lbs** — 12.4-1

Z_α'Nails: $$Z_\alpha'=\frac{(W'p)Z'}{(W'p)\cos\alpha+Z'\sin\alpha}$$ N/A — 12.4-2

A. If Moisture Content is >19% at time of fabrication and ≤19% in service, 0.4 is a conservative factor. For a more precise factor see NDS 11.3.3.

B. Diameter of lag screws are for "reduced body diameter" lag screws. (Similar to tables 11J & 11k in NDS)

C. The adujsted values for $F_{e\perp}$ and $F_{e||}$ are not supported by NDS for diameters of fasteners >1/4" in plywood or OSB & therefore not recommended. F_e is assumed for all fastener sizes herein.

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ASCE 7-16 PARAPET DESIGN (Enclosed, Partially Enclosed, and Partially Open Buildings)

I.D.: parapet at new penthouse

INPUT:

ASCE 7-16, Section 30.8

$p = q_p \; (GC_p \; - GC_{pi} \;)$

Eq 30.8-1

NOTE: Internal pressure is ignored as it produces no net effect on a single stud wall. If parapet geometry is such that each face of the parapet needs to be considered separately, include internal pressure as given by the porosity of the parapet and Table 26.13-1

			Ref:
Top of Parapet Height above ground:	37.0 ft		
Exterior Parapet Height, h_e =	30 in		
Roof Insulation Thickness, h_{ins} =	12 in		
Remaining Interior Parapet Height =	18 in		
Roof Pitch (Slope):	0 : 12	*0.0 deg*	
Exposure Classification:	B		Art. 26.7.3
Meets conditions specified in Section 26.8.1:	☐ Yes	(Wind speedup due to topography)	
Ground Elevation Factor, K_e =	1.000		Table 26.9-1
Basic Wind Speed, V =	130 mph	(3 second gust at 33 feet)	
Roof Type:	Monoslope		Fig. 30.3-2A to 30.5-1

OUTPUT:

			Ref:
K_h =	0.74		Table 26.10-1
K_{zt} =	1.00		Art. 26.8.2, Eq 26.8-1
K_d =	0.85	(Comp. and Cladding)	Table 26.6-1
q_p =	27.4 psf		Eq. 26.10-1

Positive GC_p: ***Using Fig. 30.3-1:*** — Figure 30.3-1 or 30.5-1

	(Front Face EWA)	(Back Face EWA)
Zones 4 & 5:	0.90	0.90

Negative GC_p: ***Using Fig. 30.3-5A, Note 5 -> Fig. 30.3-2A:*** — Fig. 30.3-2A to 30.5-1

	(Front Face EWA)	(Back Face EWA)
Zone 2 Roof Eave Side:	-2.30	-2.30
Zone 2' Roof Peak Side:	-2.30	-2.30
Zone 3 Roof Eave Side:	-3.20	-3.20
Zone 3' Roof Peak Side:	-3.20	-3.20

Using Fig. 30.3-1:

	(Back Face EWA)
Zone 4:	-0.99
Zone 5:	-1.26

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ASCE 7-16 PARAPET DESIGN (Enclosed, Partially Enclosed, and Partially Open Buildings)

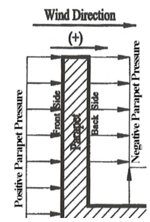
I.D.: parapet at new penthouse

FORCE ON PARPET STUDS & CONNECTIONS:

Stud Spacing, s_s =	12 in
Front Face Effective Wind Area (EWA):	2.5 sq ft
Back Face Effective Wind Area (EWA):	1.5 sq ft

Case A (windward parapet):

	Positive Pressure	Negative Pressure
Eave Side Interior =	24.6 psf	-62.9 psf
Peak Side Interior =	24.6 psf	-62.9 psf
Eave Side Corner =	24.6 psf	-87.5 psf
Peak Side Corner =	24.6 psf	-87.5 psf

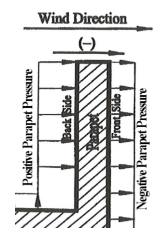


	$V_{wind,\ base}$	$M_{wind,\ base}$
Eave Side Interior =	156 lb/stud	242 lbf-ft/stud
Peak Side Interior =	156 lb/stud	242 lbf-ft/stud
Eave Side Corner =	193 lb/stud	307 lbf-ft/stud
Peak Side Corner =	193 lb/stud	307 lbf-ft/stud

$V_{wind,\ base}$ = Pos. Press. x F.F. EWA - Neg. Press. x B.F. EWA . M = (Pos. Press. x s_s x h_e^2) / 2 - (Neg. Press. x s_s x (h_e^2 - h_{ins}^2)) / 2

Case B (leeward parapet):

	Positive Pressure	Negative Pressure
Interior =	24.6 psf	-27.1 psf
Corner =	24.6 psf	-34.5 psf



	$V_{wind,\ base}$	$M_{wind,\ base}$
Interior =	-105 lb/stud	-149 lbf-ft/stud
Corner =	-123 lb/stud	-172 lbf-ft/stud

$V_{wind,\ base}$ = Neg. Press. x F.F. EWA - Pos. Press. x B.F. EWA . M = (Neg. Press. x s_s x h_e^2) / 2 - (Pos. Press. x s_s x (h_e^2 - h_{ins}^2)) / 2

Connection at new penthouse:

V = 193 lb/stud stud spacing = 12 in
M = 307 lbf-ft/stud

BOTTOM TRACK TO DECK CONN
stud = 43 mil
deck = 33 mil

Rn = 608 lb (AISI table 9c, #14 screws)
ϕ Rn = 304 lb
DCR shear = 0.63 (1 qty #14 can take all shear)

T = M / d

d = 5 in
T = 736.1 lb/stud
Rn = 344 lb (AISI table 10 a (governs over pull-over), #14 screws)
ϕ Rn = 172 lb
screw spacing = 2 inches
load/screw = 123 lb
DCR tension = 0.71

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ASCE 7-16 PARAPET DESIGN (Enclosed, Partially Enclosed, and Partially Open Buildings)

I.D.: parapet at new penthouse

STUD TO TRACK CONNECTION

Shear = M / d

d =	6	in
Shear =	613	lb/stud

43 mil track and studs

2 x 1/4" screw each side

per screw Rn =	905	lb (AISI table 9c, #14 screw)
qty =	2	
ϕ Rn =	905	
DCR =	0.68	

ABANDONED ROOF CURBS

Snowden Mech Upgrades | EH
40-21-105 | 4/19/2022

Connection between (N) ledger and (E) mechanical curb

Loads	DL =	20	psf
	S =	40	psf
	LL =	20	psf (does not govern)
	Span L =	4	ft
	spacing =	0.5	ft (anchor spacing)

size =	#14	
nominal shear capacity =	645	lb (AISI Table IV-9a)
	608	lb (AISI Table IV-9c)
governing nominal =	608	lb
allowable shear =	202.7	lb

Rim Joist to Curb connection

# screws =	1	
allowable capacity =	202.6667	lb/ft
DL + S reaction =	60	lb
DCR =	0.30	

CMU INFILL

Snowden Mechanical Upgrades EH
Project: 40-21-105 4/19/2022

Design of CMU Infill

Per ASCE 7-16, section 12.11.1, structural walls and anchorage must be designed for load $F_p = 0.4S_{DS}I_eW_p$

S_{DS} = 1.2 reference ATC hazards
I_e = 1 Table 1.5-2, Risk Category II

Wp ==> trib w = 22 in (1/2 times maximum infill width of 44" per architect)
trib w = 1.8 ft

height = 0.667 ft spacing of reinforcement
weight (psf) = 86 psf (using weight of CMU grouted at 8", see NCMA.org printout)
W_p = 105.1 lbs per 22 inches

F_p = 50 lbs per 22 inches

Check to see if horizontal bars embed into existing grouted cells can take this load.
Hilti ESR 4143
Table 7
Allowable load = 620 lb

Convert Fp load to ASD (x 0.7)
ASD Fp = 35.3 lb

$\boxed{\text{DCR} = 0.06}$

Check to see that existing vertical reinforcing can take the load.

Existing reinforcing = #5 @ 32" OC (assumed)

Check to see if #5 @ 32 inches, height = 12 ft, can take Fp load

Fp per 32 in width = 110.1 lb/ft

See enercalc printout, existing reinforcing is wall is adequate to take load.

From NCMA.org, printed 10/10/2019

EH
10/10/19

Table 4—8-in. (203-mm) Single Wythe Wall Weights

Units	Vertical grout spacing, in. (mm)	Mortar bedding	Wall weight, lb/ft² (kg/m²) for concrete densities, lb/ft³ (kg/m³) of: 85 (1,362)	95 (1,522)	105 (1,682)	115 (1,842)	125 (2,003)	135 (2,163)
Hollow	No grout	Face shell	25 (122)	28 (137)	31 (151)	33 (161)	36 (176)	39 (191)
Hollow	No grout	Full	26 (127)	28 (137)	31 (151)	34 (166)	37 (181)	39 (191)
Solid	No grout	Full	56 (274)	62 (303)	68 (332)	74 (362)	80 (391)	86 (420)
Hollow	8 (203)	Full	73 (357)	76 (371)	78 (381)	81 (396)	84 (411)	86 (420)
Hollow	16 (406)	Face shell	49 (239)	52 (254)	55 (269)	57 (279)	60 (293)	63 (308)
Hollow	24 (610)	Face shell	41 (200)	44 (215)	47 (230)	49 (239)	52 (254)	55 (269)
Hollow	32 (812)	Face shell	37 (181)	40 (195)	43 (210)	45 (220)	48 (235)	51 (249)
Hollow	40 (1,016)	Face shell	35 (171)	38 (186)	40 (195)	43 (210)	46 (225)	48 (235)
Hollow	48 (1,219)	Face shell	33 (161)	36 (176)	39 (191)	41 (200)	44 (215)	47 (230)
Hollow	56 (1,422)	Face shell	32 (156)	35 (171)	38 (186)	40 (195)	43 (210)	46 (225)
Hollow	64 (1,626)	Face shell	31 (151)	34 (166)	37 (181)	39 (191)	42 (205)	45 (220)
Hollow	72 (1,829)	Face shell	31 (151)	33 (161)	36 (176)	39 (191)	41 (200)	44 (215)
Hollow	80 (2,032)	Face shell	30 (147)	33 (161)	35 (171)	38 (186)	41 (200)	44 (215)
Hollow	88 (2,235)	Face shell	30 (147)	32 (156)	35 (171)	38 (186)	40 (195)	43 (210)
Hollow	96 (2,438)	Face shell	29 (142)	32 (156)	35 (171)	37 (181)	40 (195)	43 (210)
Hollow	104 (2,642)	Face shell	29 (142)	32 (156)	34 (166)	37 (181)	40 (195)	42 (205)
Hollow	112 (2,845)	Face shell	29 (142)	31 (151)	34 (166)	37 (181)	39 (191)	42 (205)
Hollow	120 (3,048)	Face shell	28 (137)	31 (151)	34 (166)	37 (181)	39 (191)	42 (205)

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Project Title: Gruening MS Earthquake Recovery
Engineer: EH
Project ID: 40-19-050
Project Descr:

Masonry Slender Wall

File: 22-04-19_CMU infill jambs_eh.ec6

Lic. # : KW-06001667 **REID MIDDLETON, INC.**

DESCRIPTION: Check jamb at masonry infill

Code References

Calculations per ACI 530-11, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Calculations per ACI 530-11, IBC 2012, CBC 2013, ASCE 7-10

Construction Type : Grouted Hollow Concrete Masonry

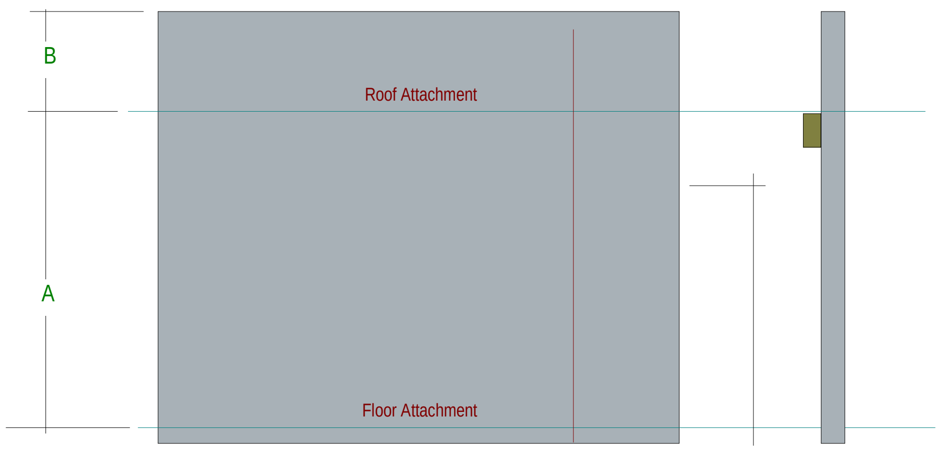
F'm	=	1.50 ksi	Nom. Wall Thickness	8 in	Temp Diff across thickness	=	deg F
Fy - Yield	=	40.0 ksi	Actual Thickness	7.625 in	Min Allow Out-of-plane Defl Ratio	=	0.0
Fr - Rupture	=	61.0 psi	Rebar "d" distance	3.750 in			
Em = f'm *	=	900.0	Lower Level Rebar . . .		Minimum Vertical Steel %	=	0.0020
Max % of ρ bal.	=	0.01640	Bar Size #	5			
Grout Density	=	140 pcf	Bar Spacing	32 in			
Block Weight		Normal Weight					
Wall Weight	=	58.0 psf					

Wall is grouted at rebar cells only

One-Story Wall Dimensions

A Clear Height	=	12.0	ft
B Parapet height	=		ft

Wall Support Condition Top & Bottom Pinned



Lateral Loads

Wind Loads :

Full area WIND load 15.0 psf

Seismic Loads :

Wall Weight Seismic Load Input Method : Direct entry of Lateral Wall Weight

Seismic Wall Lateral Load 42.0 psf

Fp 1.0 = 42.0 psf

(Applied to full "STRIP Width")

	D	Lr	L	E	W		Endpoints from Base top	bottom	
Distributed Lateral Load				0.1100		k/ft			ft

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Project Descr:

Masonry Slender Wall

File: 22-04-19_CMU infill jambs_eh.ec6

Lic. # : KW-06001667 REID MIDDLETON, INC.

DESCRIPTION: Check jamb at masonry infill

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check	Maximum Bending Stress Ratio =	**0.6048**		
	+0.90D+E+0.90H	Max Mu	0.7598 k-ft	Phi * Mn	1.256 k-ft
PASS	Service Deflection Check	Actual Defl. Ratio L/	537	Allowable Defl. Ratio	150.0
	E Only	Max. Deflection	0.2681 in		
PASS	Axial Load Check	Max Pu / Ag	7.575 psi	Max. Allow. Defl.	0.960 in
	+1.20D+0.50L+0.20S+E+1.60H	Location	5.80 ft	0.2 * f'm	300.0 psi
PASS	Reinforcing Limit Check	Actual As/bd	0.00250	Max Allow As/bd	0.01640

Maximum Reactions . . .	*for Load Combination....*	
Top Horizontal	E Only	0.2520 k
Base Horizontal	E Only	0.2520 k
Vertical Reaction	+D+0.750L+0.750S+0.5250E+H	0.6960 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu (k)	Axial Load 0.2*f'm*b*t (k)	Mcr (k-ft)	Moment Values Mu (k-ft)	Phi	Phi Mn (k-ft)	As (in^2)	As Ratio	0.6 * rho bal
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
+1.20D+1.60Lr+0.50W+1.60H at 5.60 to 6.00	0.445	17.640	0.46	0.13	0.90	1.29	0.116	0.0025	0.0162
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
+1.20D+1.60S+0.50W+1.60H at 5.60 to 6.00	0.445	17.640	0.46	0.13	0.90	1.29	0.116	0.0025	0.0162
+1.20D+0.50Lr+0.50L+W+1.60H at 5.60 to 6.	0.445	17.640	0.46	0.27	0.90	1.29	0.116	0.0025	0.0162
+1.20D+0.50L+0.50S+W+1.60H at 5.60 to 6.0	0.445	17.640	0.46	0.27	0.90	1.29	0.116	0.0025	0.0162
+1.20D+0.50L+0.20S+E+1.60H at 5.60 to 6.0	0.445	17.640	0.46	0.76	0.90	1.29	0.116	0.0025	0.0162
+0.90D+W+0.90H at 5.60 to 6.00	0.334	17.640	0.46	0.27	0.90	1.26	0.116	0.0025	0.0162
+0.90D+E+0.90H at 5.60 to 6.00	0.334	17.640	0.46	0.76	0.90	1.26	0.116	0.0025	0.0162

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu (k)	Moment Values Mcr (k-ft)	Moment Values Mactual (k-ft)	Stiffness I gross (in^4)	Stiffness I cracked (in^4)	Stiffness I effective (in^4)	Deflections Deflection (in)	Deflections Defl. Ratio
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.60W+H at 5.60 to 6.00	0.371	0.46	0.16	342.40	26.09	342.400	0.009	15,861.9
+D+0.70E+H at 6.00 to 6.40	0.348	0.46	0.53	342.40	25.99	36.087	0.067	2,146.4
+D+0.750Lr+0.750L+0.450W+H at 5.60 to 6.0	0.371	0.46	0.12	342.40	26.09	342.400	0.007	21,149.2
+D+0.750L+0.750S+0.450W+H at 5.60 to 6.00	0.371	0.46	0.12	342.40	26.09	342.400	0.007	21,149.2
+D+0.750L+0.750S+0.5250E+H at 5.60 to 6.0	0.371	0.46	0.40	342.40	26.09	342.400	0.022	6,474.2
+0.60D+0.60W+0.60H at 5.60 to 6.00	0.223	0.46	0.16	342.40	25.48	342.400	0.009	15,872.0
+0.60D+0.70E+0.60H at 6.00 to 6.40	0.209	0.46	0.53	342.40	25.42	35.414	0.067	2,135.1
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0

Masonry Slender Wall

DESCRIPTION: Check jamb at masonry infill

	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
W Only at 6.00 to 6.40	0.000	0.46	0.27	342.40	24.55	342.400	0.015	9,532.3
E Only at 6.00 to 6.40	0.000	0.46	0.75	342.40	24.55	26.272	0.268	537.1

Masonry Slender Wall

DESCRIPTION: Check jamb at masonry infill

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu (k)	Moment Values Mcr (k-ft)	Moment Values Mactual (k-ft)	Stiffness I gross (in^4)	Stiffness I cracked (in^4)	Stiffness I effective (in^4)	Deflections Deflection (in)	Deflections Defl. Ratio
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
+D+H	0.0 k	0.00 k	0.696 k
+D+L+H	0.0 k	0.00 k	0.696 k
+D+Lr+H	0.0 k	0.00 k	0.696 k
+D+S+H	0.0 k	0.00 k	0.696 k
+D+0.750Lr+0.750L+H	0.0 k	0.00 k	0.696 k
+D+0.750L+0.750S+H	0.0 k	0.00 k	0.696 k
+D+0.60W+H	0.1 k	0.05 k	0.696 k
+D+0.70E+H	0.2 k	0.18 k	0.696 k
+D+0.750Lr+0.750L+0.450W+H	0.0 k	0.04 k	0.696 k
+D+0.750L+0.750S+0.450W+H	0.0 k	0.04 k	0.696 k
+D+0.750L+0.750S+0.5250E+H	0.1 k	0.13 k	0.696 k
+0.60D+0.60W+0.60H	0.1 k	0.05 k	0.418 k
+0.60D+0.70E+0.60H	0.2 k	0.18 k	0.418 k
D Only	0.0 k	0.00 k	0.696 k
Lr Only	0.0 k	0.00 k	0.000 k
L Only	0.0 k	0.00 k	0.000 k
S Only	0.0 k	0.00 k	0.000 k
W Only	0.1 k	0.09 k	0.000 k
E Only	0.3 k	0.25 k	0.000 k
H Only	0.0 k	0.00 k	0.000 k

NEW OPENINGS IN EXISTING CONC SLAB

Project: Snowden Mech Upgrades — EH
40.2021.105 — 6/15/2022

NEW OPENING ON SECOND FLOOR ADJACENT TO EXISTING

	concrete beam to beam =	8.08	ft	
	opening w =	3.67	in	(use width of existing opening)
	opening L =	10	in	
loads: DL:	t =	5	in concrete	
	DL =	62.5	psf	
LL:	LL =	40	psf, assumed	
	DL + LL =	102.5	psf	

header:

header span =	3.50	ft	
trib =	4.04	ft	conservative
loading =	414.3	plf	
R1 = R2 =	725.0	lb	
M =	634.4	lbft	
	0.6	kft	
C6x13 (w/Fy = 36 ksi)			
Vn/Ω =	33.90	k	
shear DCR =	0.02		
Mp/Ω =	13.10	kft	
moment DCR =	0.05		

at concrete beam, anchor using 1/2" diameter x 3 1/2" concrete screw anchors (see Hilti printout)

LRFD: 1.2D + 1.6L = 139 psf

R = 983.1 lb

at perpendicular channel, weld 3/16" all around

steel member from concrete beam to concrete beam:

span =	8.08	ft
point load, p =	725.0	lb
a =	4.6	ft
b =	3.5	ft
R1 = p*a/span =	411.1	lb
R2 = p*b/span =	313.9	lb
M = pab/span =	1438.7	lbft
	1.4	kft

C6x1e (w/Fy = 36 ksi)

V_n/Ω =	33.90	k
shear DCR =	0.01	
M_p/Ω =	13.10	kft
moment DCR =	0.11	

at ends of channel (at concrete beam, use same anchorage as header)

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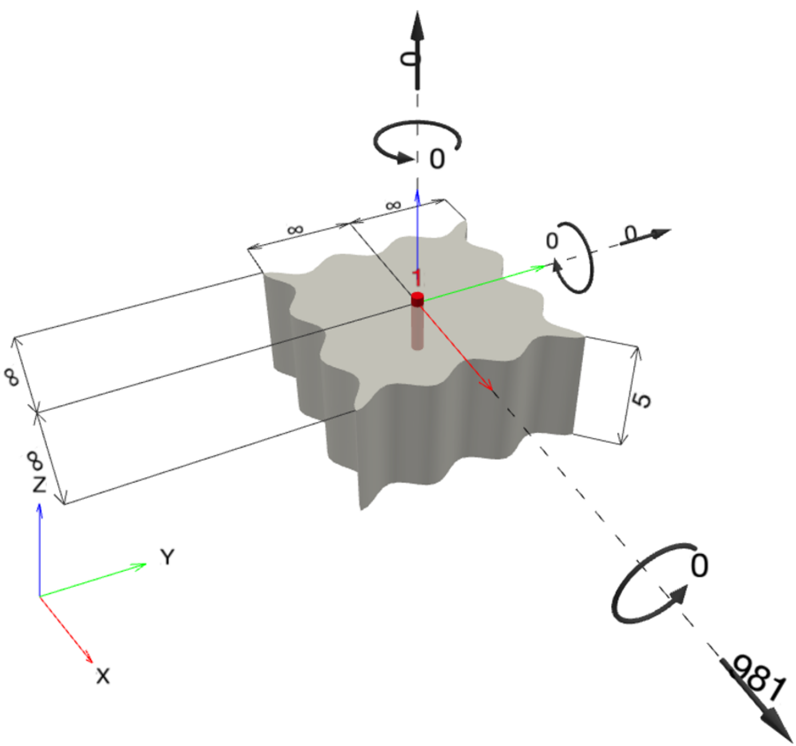
Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	New opening adjacent to existing	Date:	6/17/2022
Fastening point:			

Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 1/2 (3)**
Item number:	418072 KH-EZ 1/2"x3 1/2"
Effective embedment depth:	h_{ef} = 2.160 in., h_{nom} = 3.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	4/1/2022 | 12/1/2023
Proof:	Design Method ACI 318 / AC193
Stand-off installation:	
Profile:	
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 5.000 in.
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]



Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	New opening adjacent to existing	Date:	6/17/2022
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0; V_x = 981; V_y = 0;$ $M_x = 0; M_y = 0; M_z = 0;$	no	52

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	981	981	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	981	5,547	18	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	981	1,889	52	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3027

ϕ $V_{steel} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.16	112,540	0.600

Calculations

V_{sa} [lb]
9,245

Results

V_{sa} [lb]	ϕ $_{steel}$	ϕ V_{sa} [lb]	V_{ua} [lb]
9,245	0.600	5,547	981

4.2 Pryout Strength

$V_{cp} = k_{cp}\left[\left(\frac{A_{Nc}}{A_{Nc0}}\right)\psi_{ed,N}\,\psi_{c,N}\,\psi_{cp,N}\,N_b\right]$	ACI 318-08 Eq. (D-30)
$\phi\ V_{cp} \geq V_{ua}$	ACI 318-08 Eq. (D-2)
A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-08 Eq. (D-6)
$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-08 Eq. (D-11)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-08 Eq. (D-13)
$N_b = k_c\ \lambda\ \sqrt{f'_c}\ h_{ef}^{1.5}$	ACI 318-08 Eq. (D-7)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1	2.160	∞	1.000

c_{ac} [in.]	k_c	λ	f'_c [psi]
3.750	17	1	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
41.99	41.99	1.000	1.000	2,698

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
2,698	0.700	1,889	981

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!

Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	New opening adjacent to existing	Date:	6/17/2022
Fastening point:			

6 Installation data

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 1/2 (3)

Profile: -

Item number: 418072 KH-EZ 1/2"x3 1/2"

Hole diameter in the fixture: -

Maximum installation torque: 540 in.lb

Plate thickness (input): -

Hole diameter in the base material: 0.500 in.

Hole depth in the base material: 3.375 in.

Drilling method: Hammer drilled

Minimum thickness of the base material: 4.750 in.

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti KH-EZ screw anchor with 3 in embedment, 1/2 (3), Carbon steel, installation per ESR-3027

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	-	-	-	-

Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	New opening adjacent to existing	Date:	6/17/2022
Fastening point:			

7 Remarks; Your Cooperation Duties

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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Project Title: Engineer: Project ID: Project Descr:

NEW OPNG IN (E) STL DECK

Steel Beam

File: snowden.ec6

Lic. # : KW-06001667 **REID MIDDLETON, INC.**

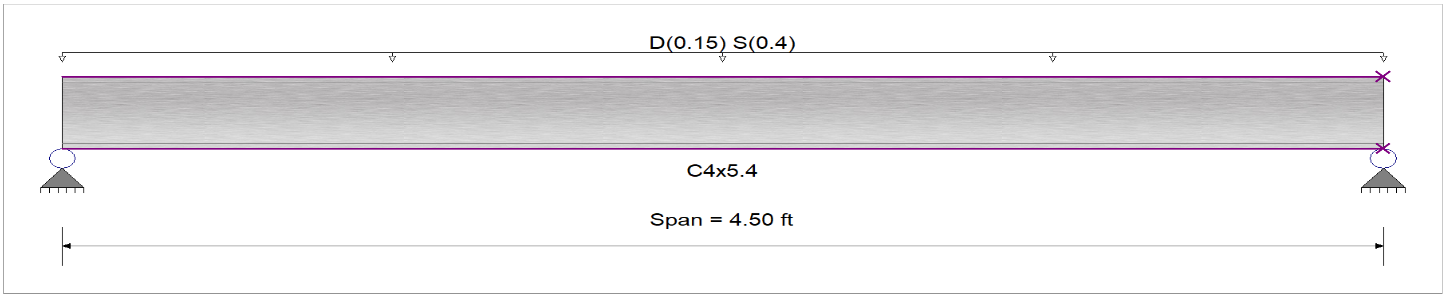
DESCRIPTION: (N) opening in (E) steel deck; C4x5.4

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	36.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.0150, S = 0.040 ksf, Tributary Width = 10.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.338** : 1	Maximum Shear Stress Ratio =	**0.130** : 1
Section used for this span	**C4x5.4**	Section used for this span	**C4x5.4**
Ma : Applied	1.392 k-ft	Va : Applied	1.238 k
Mn / Omega : Allowable	4.114 k-ft	Vn/Omega : Allowable	9.520 k
Load Combination	+D+S+H	Load Combination	+D+S+H
Location of maximum on span	2.250 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.033 in	Ratio = 1,626 >=360
Max Upward Transient Deflection	0.000 in	Ratio = 0 <360
Max Downward Total Deflection	0.046 in	Ratio = 1183 >=180
Max Upward Total Deflection	0.000 in	Ratio = 0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 4.50 ft	1	0.092	0.035	0.38		0.38	6.87	4.11	1.00	1.00	0.34	15.90	9.52
+D+L+H													
Dsgn. L = 4.50 ft	1	0.092	0.035	0.38		0.38	6.87	4.11	1.00	1.00	0.34	15.90	9.52
+D+Lr+H													
Dsgn. L = 4.50 ft	1	0.092	0.035	0.38		0.38	6.87	4.11	1.00	1.00	0.34	15.90	9.52
+D+S+H													
Dsgn. L = 4.50 ft	1	0.338	0.130	1.39		1.39	6.87	4.11	1.00	1.00	1.24	15.90	9.52
+D+0.750Lr+0.750L+H													
Dsgn. L = 4.50 ft	1	0.092	0.035	0.38		0.38	6.87	4.11	1.00	1.00	0.34	15.90	9.52
+D+0.750L+0.750S+H													
Dsgn. L = 4.50 ft	1	0.277	0.106	1.14		1.14	6.87	4.11	1.00	1.00	1.01	15.90	9.52
+D+0.60W+H													
Dsgn. L = 4.50 ft	1	0.092	0.035	0.38		0.38	6.87	4.11	1.00	1.00	0.34	15.90	9.52
+D+0.750Lr+0.750L+0.450W+H													
Dsgn. L = 4.50 ft	1	0.092	0.035	0.38		0.38	6.87	4.11	1.00	1.00	0.34	15.90	9.52
+D+0.750L+0.750S+0.450W+H													
Dsgn. L = 4.50 ft	1	0.277	0.106	1.14		1.14	6.87	4.11	1.00	1.00	1.01	15.90	9.52
+0.60D+0.60W+0.60H													
Dsgn. L = 4.50 ft	1	0.055	0.021	0.23		0.23	6.87	4.11	1.00	1.00	0.20	15.90	9.52
+D+0.70E+0.60H													
Dsgn. L = 4.50 ft	1	0.092	0.035	0.38		0.38	6.87	4.11	1.00	1.00	0.34	15.90	9.52
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 4.50 ft	1	0.277	0.106	1.14		1.14	6.87	4.11	1.00	1.00	1.01	15.90	9.52

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Engineer:
Project ID:
Project Descr:

Steel Beam

File: snowden.ec6

Lic. # : KW-06001667 REID MIDDLETON, INC.

DESCRIPTION: (N) opening in (E) steel deck; C4x5.4

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+0.60D+0.70E+H													
Dsgn. L = 4.50 ft	1	0.055	0.021	0.23		0.23	6.87	4.11	1.00	1.00	0.20	15.90	9.52

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.0457	2.263		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.238	1.238
Overall MINimum	0.203	0.203
+D+H	0.338	0.338
+D+L+H	0.338	0.338
+D+Lr+H	0.338	0.338
+D+S+H	1.238	1.238
+D+0.750Lr+0.750L+H	0.338	0.338
+D+0.750L+0.750S+H	1.013	1.013
+D+0.60W+H	0.338	0.338
+D+0.750Lr+0.750L+0.450W+H	0.338	0.338
+D+0.750L+0.750S+0.450W+H	1.013	1.013
+0.60D+0.60W+0.60H	0.203	0.203
+D+0.70E+0.60H	0.338	0.338
+D+0.750L+0.750S+0.5250E+H	1.013	1.013
+0.60D+0.70E+H	0.203	0.203
D Only	0.338	0.338
S Only	0.900	0.900
H Only		

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Engineer:
Project ID:
Project Descr:

NEW OPNG IN (E) WOOD FRAMING

Wood Beam

File: snowden.ec6

Lic. # : KW-06001667 REID MIDDLETON, INC.

DESCRIPTION: CASE 1 - Header for (N) opening at (E) wood framing; double header (use 1/2 of trib width)

CODE REFERENCES

case 1 - opening adjacent to girder

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

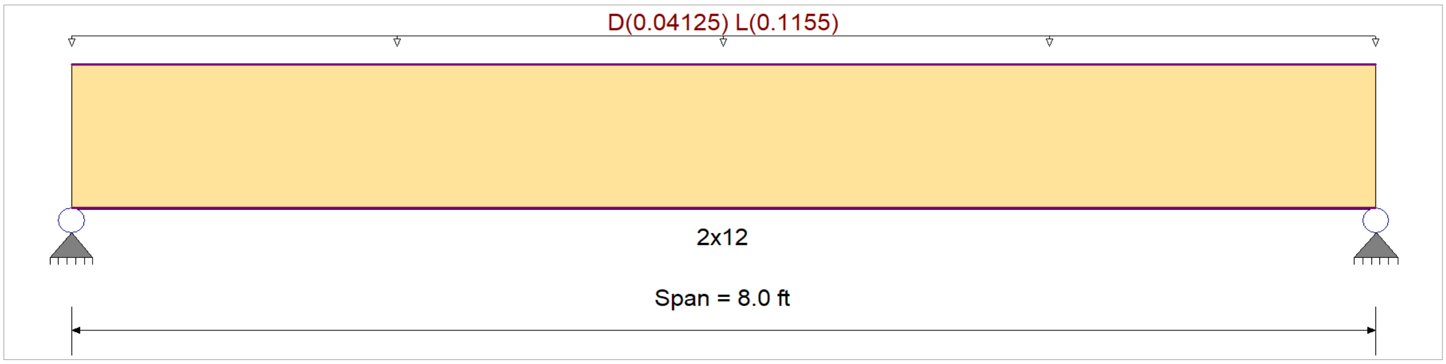
Wood Species : Hem-Fir
Wood Grade : No.2

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	850.0	psi
Fb -	850.0	psi
Fc - Prll	1,300.0	psi
Fc - Perp	405.0	psi
Fv	150.0	psi
Ft	525.0	psi

E : Modulus of Elasticity

Ebend- xx	1,300.0	ksi
Eminbend - xx	470.0	ksi
Density	26.840	pcf



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.0420 ksf, Tributary Width = 2.750 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.560** : 1	Maximum Shear Stress Ratio	=	**0.285** : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
fb: Actual	=	475.59 psi	fv: Actual	=	42.72 psi
Fb: Allowable	=	850.00 psi	Fv: Allowable	=	150.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	7.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.046 in	Ratio =	2074 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.063 in	Ratio =	1528 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.164	0.083	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.33	125.16	765.00	0.13	11.24	135.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.560	0.285	1.00	1.000	1.00	1.00	1.00	1.00	1.00	1.25	475.59	850.00	0.48	42.72	150.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.118	0.060	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.33	125.16	1062.50	0.13	11.24	187.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.128	0.065	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.33	125.16	977.50	0.13	11.24	172.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.365	0.186	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.02	387.98	1062.50	0.39	34.85	187.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.397	0.202	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.02	387.98	977.50	0.39	34.85	172.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.092	0.047	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.33	125.16	1360.00	0.13	11.24	240.00

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Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

File: snowden.ec6

Lic. # : KW-06001667 REID MIDDLETON, INC.

DESCRIPTION: CASE 1 - Header for (N) opening at (E) wood framing; double header (use 1/2 of trib width)

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.285	0.145	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.02	387.98	1360.00	0.39	34.85	240.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.285	0.145	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.02	387.98	1360.00	0.39	34.85	240.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.055	0.028	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.20	75.09	1360.00	0.08	6.74	240.00
+D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.092	0.047	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.33	125.16	1360.00	0.13	11.24	240.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.285	0.145	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.02	387.98	1360.00	0.39	34.85	240.00
+0.60D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.055	0.028	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.20	75.09	1360.00	0.08	6.74	240.00

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.627	0.627
Overall MINimum	0.462	0.462
+D+H	0.165	0.165
+D+L+H	0.627	0.627
+D+Lr+H	0.165	0.165
+D+S+H	0.165	0.165
+D+0.750Lr+0.750L+H	0.512	0.512
+D+0.750L+0.750S+H	0.512	0.512
+D+0.60W+H	0.165	0.165
+D+0.750Lr+0.750L+0.450W+H	0.512	0.512
+D+0.750L+0.750S+0.450W+H	0.512	0.512
+0.60D+0.60W+0.60H	0.099	0.099
+D+0.70E+0.60H	0.165	0.165
+D+0.750L+0.750S+0.5250E+H	0.512	0.512
+0.60D+0.70E+H	0.099	0.099
D Only	0.165	0.165
L Only	0.462	0.462
H Only		

Wood Beam

DESCRIPTION: CASE 1 - Joist at (N) opening at (E) wood framing; opening size = 8'-0"; (3) joists per side therefore take reaction from header

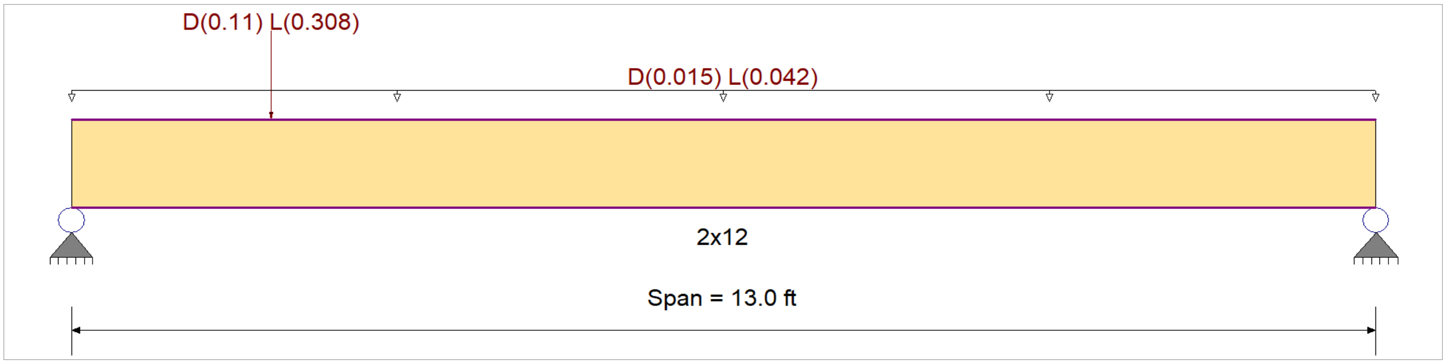
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	850.0 psi	Ebend- xx	1,300.0 ksi
		Fc - Prll	1,300.0 psi	Eminbend - xx	470.0 ksi
Wood Species	: Hem-Fir	Fc - Perp	405.0 psi		
Wood Grade	: No.2	Fv	150.0 psi		
		Ft	525.0 psi	Density	26.840 pcf

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.0420 ksf, Tributary Width = 1.0 ft
Point Load : D = 0.110, L = 0.3080 k @ 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.740** : 1	Maximum Shear Stress Ratio	=	**0.399** : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
fb: Actual	=	628.96 psi	fv: Actual	=	59.81 psi
Fb: Allowable	=	850.00 psi	Fv: Allowable	=	150.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	5.361 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.165 in	Ratio =	945 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.224 in	Ratio =	696 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.216	0.117	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.44	165.52	765.00	0.18	15.74	135.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.740	0.399	1.00	1.000	1.00	1.00	1.00	1.00	1.00	1.66	628.96	850.00	0.67	59.81	150.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.156	0.084	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.44	165.52	1062.50	0.18	15.74	187.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.169	0.091	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.44	165.52	977.50	0.18	15.74	172.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.483	0.260	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.35	513.10	1062.50	0.55	48.79	187.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.525	0.283	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.35	513.10	977.50	0.55	48.79	172.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

DESCRIPTION: CASE 1 - Joist at (N) opening at (E) wood framing; opening size = 8'-0"; (3) joists per side therefore take reaction from header

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 13.0 ft	1	0.122	0.066	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.44	165.52	1360.00	0.18	15.74	240.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.377	0.203	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.35	513.10	1360.00	0.55	48.79	240.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.377	0.203	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.35	513.10	1360.00	0.55	48.79	240.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.073	0.039	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.26	99.31	1360.00	0.11	9.44	240.00
+D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.122	0.066	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.44	165.52	1360.00	0.18	15.74	240.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.377	0.203	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.35	513.10	1360.00	0.55	48.79	240.00
+0.60D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.073	0.039	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.26	99.31	1360.00	0.11	9.44	240.00

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.724	0.435
Overall MINimum	0.534	0.320
+D+H	0.191	0.114
+D+L+H	0.724	0.435
+D+Lr+H	0.191	0.114
+D+S+H	0.191	0.114
+D+0.750Lr+0.750L+H	0.591	0.355
+D+0.750L+0.750S+H	0.591	0.355
+D+0.60W+H	0.191	0.114
+D+0.750Lr+0.750L+0.450W+H	0.591	0.355
+D+0.750L+0.750S+0.450W+H	0.591	0.355
+0.60D+0.60W+0.60H	0.114	0.069
+D+0.70E+0.60H	0.191	0.114
+D+0.750L+0.750S+0.5250E+H	0.591	0.355
+0.60D+0.70E+H	0.114	0.069
D Only	0.191	0.114
L Only	0.534	0.320
H Only		

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Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

File: snowden.ec6
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Lic. # : KW-06001667 **REID MIDDLETON, INC.**

DESCRIPTION: CASE 2 - Header for (N) opening at (E) wood framing; double header (use 1/2 of trib width)

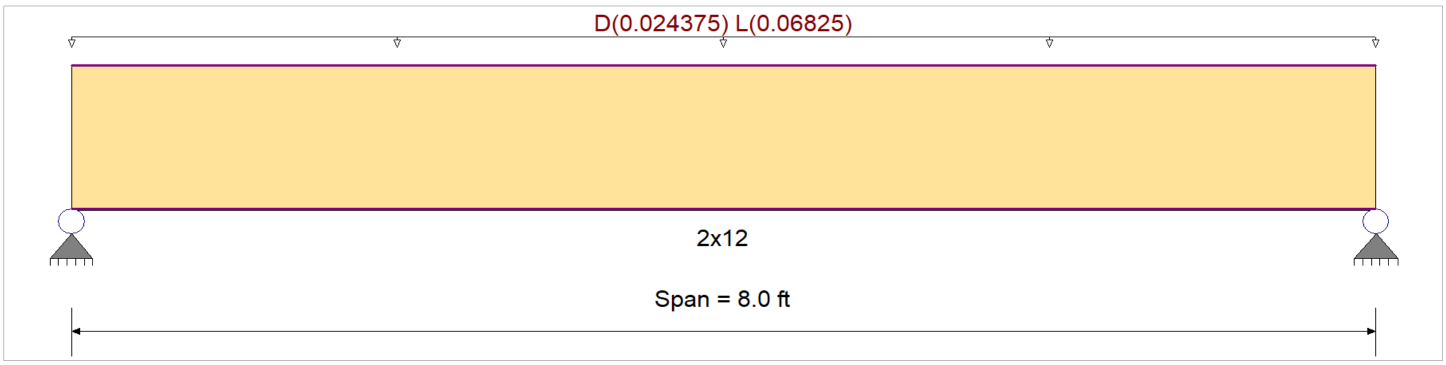
CODE REFERENCES

Case 2 - opening at midspan

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	850.0 psi	Ebend- xx	1,300.0 ksi
		Fc - Prll	1,300.0 psi	Eminbend - xx	470.0 ksi
Wood Species	: Hem-Fir	Fc - Perp	405.0 psi		
Wood Grade	: No.2	Fv	150.0 psi		
		Ft	525.0 psi	Density	26.840 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.0420 ksf, Tributary Width = 1.625 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.331** : 1	Maximum Shear Stress Ratio	=	**0.168** : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
fb: Actual	=	281.03 psi	fv: Actual	=	25.24 psi
Fb: Allowable	=	850.00 psi	Fv: Allowable	=	150.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	7.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.027 in	Ratio =	3510 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.037 in	Ratio =	2586 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.097	0.049	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.20	73.96	765.00	0.07	6.64	135.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.331	0.168	1.00	1.000	1.00	1.00	1.00	1.00	1.00	0.74	281.03	850.00	0.28	25.24	150.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.070	0.035	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.20	73.96	1062.50	0.07	6.64	187.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.076	0.039	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.20	73.96	977.50	0.07	6.64	172.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.216	0.110	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.60	229.26	1062.50	0.23	20.59	187.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.235	0.119	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.60	229.26	977.50	0.23	20.59	172.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.054	0.028	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.20	73.96	1360.00	0.07	6.64	240.00

Wood Beam

File: snowden.ec6

Lic. # : KW-06001667 **REID MIDDLETON, INC.**

DESCRIPTION: CASE 2 - Header for (N) opening at (E) wood framing; double header (use 1/2 of trib width)

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.169	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.60	229.26	1360.00	0.23	20.59	240.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.169	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.60	229.26	1360.00	0.23	20.59	240.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.033	0.017	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.12	44.37	1360.00	0.04	3.99	240.00
+D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.054	0.028	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.20	73.96	1360.00	0.07	6.64	240.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.169	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.60	229.26	1360.00	0.23	20.59	240.00
+0.60D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.033	0.017	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.12	44.37	1360.00	0.04	3.99	240.00

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.371	0.371
Overall MINimum	0.273	0.273
+D+H	0.098	0.098
+D+L+H	0.371	0.371
+D+Lr+H	0.098	0.098
+D+S+H	0.098	0.098
+D+0.750Lr+0.750L+H	0.302	0.302
+D+0.750L+0.750S+H	0.302	0.302
+D+0.60W+H	0.098	0.098
+D+0.750Lr+0.750L+0.450W+H	0.302	0.302
+D+0.750L+0.750S+0.450W+H	0.302	0.302
+0.60D+0.60W+0.60H	0.059	0.059
+D+0.70E+0.60H	0.098	0.098
+D+0.750L+0.750S+0.5250E+H	0.302	0.302
+0.60D+0.70E+H	0.059	0.059
D Only	0.098	0.098
L Only	0.273	0.273
H Only		

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Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

File: snowden.ec6

Lic. # : KW-06001667 **REID MIDDLETON, INC.**

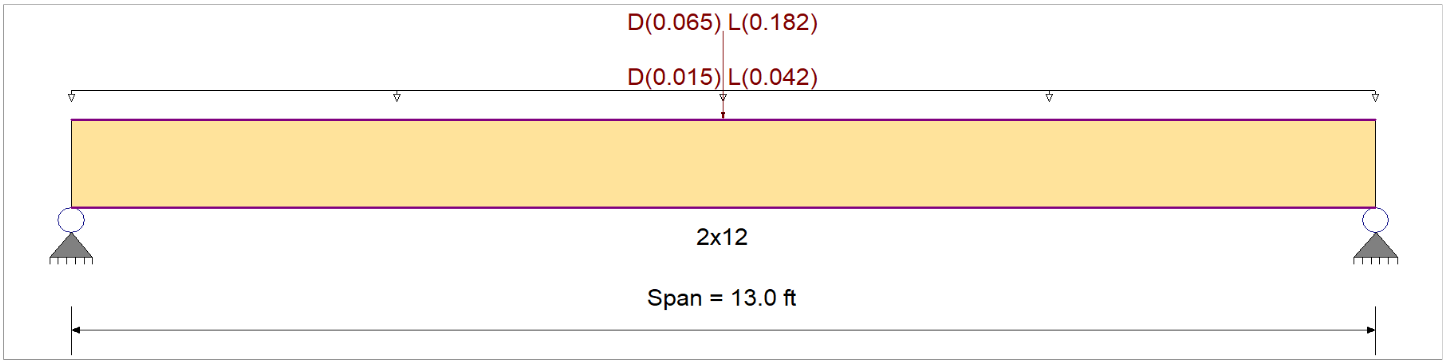
DESCRIPTION: CASE 2 - Joist at (N) opening at (E) wood framing; (3) joists per side therefore take reaction from header calc x 2/3; load at

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	850.0 psi	Ebend- xx	1,300.0 ksi
		Fc - Prll	1,300.0 psi	Eminbend - xx	470.0 ksi
Wood Species	: Hem-Fir	Fc - Perp	405.0 psi		
Wood Grade	: No.2	Fv	150.0 psi		
		Ft	525.0 psi	Density	26.840 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.0150, L = 0.0420 ksf, Tributary Width = 1.0 ft
Point Load : D = 0.0650, L = 0.1820 k @ 6.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.895** : 1	Maximum Shear Stress Ratio	=	**0.262** : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
fb: Actual	=	761.13 psi	fv: Actual	=	39.34 psi
Fb: Allowable	=	850.00 psi	Fv: Allowable	=	150.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	6.500 ft	Location of maximum on span	=	12.099 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.180 in	Ratio =	867 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.244 in	Ratio =	638 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.262	0.077	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.53	200.30	765.00	0.12	10.35	135.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.895	0.262	1.00	1.000	1.00	1.00	1.00	1.00	1.00	2.01	761.13	850.00	0.44	39.34	150.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.189	0.055	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.53	200.30	1062.50	0.12	10.35	187.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.205	0.060	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.53	200.30	977.50	0.12	10.35	172.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.584	0.171	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.64	620.92	1062.50	0.36	32.10	187.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.635	0.186	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.64	620.92	977.50	0.36	32.10	172.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

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Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

File: snowden.ec6

Lic. # : KW-06001667
REID MIDDLETON, INC.

DESCRIPTION: CASE 2 - Joist at (N) opening at (E) wood framing; (3) joists per side therefore take reaction from header calc x 2/3; load at

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 13.0 ft	1	0.147	0.043	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.53	200.30	1360.00	0.12	10.35	240.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.457	0.134	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.64	620.92	1360.00	0.36	32.10	240.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.457	0.134	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.64	620.92	1360.00	0.36	32.10	240.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.088	0.026	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.32	120.18	1360.00	0.07	6.21	240.00
+D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.147	0.043	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.53	200.30	1360.00	0.12	10.35	240.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.457	0.134	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.64	620.92	1360.00	0.36	32.10	240.00
+0.60D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.088	0.026	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.32	120.18	1360.00	0.07	6.21	240.00

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.494	0.494
Overall MINimum	0.364	0.364
+D+H	0.130	0.130
+D+L+H	0.494	0.494
+D+Lr+H	0.130	0.130
+D+S+H	0.130	0.130
+D+0.750Lr+0.750L+H	0.403	0.403
+D+0.750L+0.750S+H	0.403	0.403
+D+0.60W+H	0.130	0.130
+D+0.750Lr+0.750L+0.450W+H	0.403	0.403
+D+0.750L+0.750S+0.450W+H	0.403	0.403
+0.60D+0.60W+0.60H	0.078	0.078
+D+0.70E+0.60H	0.130	0.130
+D+0.750L+0.750S+0.5250E+H	0.403	0.403
+0.60D+0.70E+H	0.078	0.078
D Only	0.130	0.130
L Only	0.364	0.364
H Only		