CALDERWOOD ENGINEERING
222 RIVER ROAD
RICHMOND, ME 04357
PH. (207) 737-2007 FAX (207) 737-2008

Project: 11-Town of Bowdoinham-20; 01-Recycling Center-20
Client: Town of Bowdoinham

Construction Engineering Design: Calderwood Engineering
Design Computations by: Thad Chamberlain, EI
Design Check by: Eric Calderwood, PE

STATE OF MAINE
ERIC T. CALDERWOOD
No. 9099
LICENSED PROFESSIONAL ENGINEER

Project Notes:
Check Bowdoinham Recycling Building, determine capacity of additional members not addressed in the calculations and details dated December 2013.

References: NDS 2012, ASCE 7-14, IBC 2009

Check Design of additional beam supporting second floor:

(4) 2x8's at single span between additional support columns:

$b_{beam} := 1.5\ in$ $\quad d_{beam} := 7.25\ in$

$L_{beam} := 12\ ft$ — spacing between columns

$w_1 := 3\ ft + 2\ in$ — distance from center of column to center of exterior column

$w_2 := 8\ ft + 10.25\ in$ — distance from center of column to center of interior column

$\sigma_{LL} := 125\ psf$ — light storage warehouse (From Table 4-1, ASCE 7)

$\sigma_{floor} := 5\ psf$ — timber framing, assume 5psf (see pg 6 of 93 of original calculations)

Calculate total load applied by beam:

$$w_{beam} := (\sigma_{LL} + \sigma_{floor}) \cdot \left(\frac{w_1}{2} + \frac{w_2}{2}\right) + 45\ pcf \cdot (4 \cdot b_{beam} \cdot d_{beam}) = 794.948\ plf$$

$$P_{beam} := \frac{w_{beam} \cdot L_{beam}}{2} = 4.77\ kip$$

This is the reaction load at the end of the beam

Calculate beam in bearing:

$F_{cperp} := 335\ psi$ — SPF No.2 South, Ref. NDS 2012

$l_{bear} := 2.5\ in$ — length of bearing

$C_m := 1.0$ $\quad C_t := 1.0$ $\quad C_i := 1.0$ — NDS 4.3.3/4/8

$$C_b := \frac{l_{bear} + 0.375\ in}{l_{bear}} = 1.15$$

NDS 3.10-2

$F_{cperp}' := F_{cperp} \cdot C_m \cdot C_t \cdot C_i \cdot C_b = 385.25\ psi$ — NDS Table 4.3.1

$A_{bear} := l_{bear} \cdot b_{beam} = 3.75\ in^2$ — total bearing area of single beam

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Check Design of additional beam supporting second floor:

$$f_{cperp} := \frac{P_{beam}}{A_{bear} \cdot 4} = 317.979 \text{ psi}$$

$$\text{Check} := \begin{array}{l} \text{if } f_{cperp} \le F_{cperp}' \\ \quad \| \text{“Ok for bearing”} \\ \text{else} \\ \quad \| \text{“Check”} \end{array} = \text{“Ok for bearing”}$$

2.5 inches of bearing is required for the 2x8's, shift column as required to provide enough bearing for each beam, or install additional 6x6 column.

Check concrete blocks under Truss columns:

Existing 7"x7"x5" concrete blocks in compression:

$P_{vert} := 19440 \text{ lbf} = 19.44 \text{ kip}$ — See page 84 of 94 of previous design calculations
This is the factored load in the columns supporting the truss

$f'_c := 2 \text{ ksi}$ — no information on existing concrete blocks, assume 2ksi

$b_{conc} := 6 \text{ in}$ — concrete block dimension (7" square block, assume some section loss to 6" square block)

$t_{conc} := 5 \text{ in}$ — thickness of concrete block

$A_1 := b_{conc} \cdot b_{conc} = 36 \text{ in}^2$ — bearing area on concrete block

$B_n := 0.85 \cdot f'_c \cdot A_1 = 61.2 \text{ kip}$ — unfactored bearing capacity of block (ACI 318, Table 22.8.3.2)

$\phi_{bearing} := 0.65$ — (ACI 318, Table 21.2.1)

$\phi B_n := \phi_{bearing} \cdot B_n = 39.78 \text{ kip}$

$$\text{Check} := \begin{array}{l} \text{if } P_{vert} \le \phi B_n \\ \quad \| \text{“Concrete Ok for Bearing”} \\ \text{else} \\ \quad \| \text{“Check”} \end{array} = \text{“Concrete Ok for Bearing”}$$

Concete blocks are ok for bearing, should be monitored for section loss

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Capacities of Tapcon Blue Anchors:

Buildex Tapcon®
#1 CHOICE OF PROFESSIONALS

PERFORMANCE TABLES
BLUE, WHITE, AND STAINLESS

ULTIMATE TENSION AND SHEAR VALUES (LBS/KN) IN CONCRETE

ANCHOR DIA In.(mm)	MIN. DEPTH OF EMBEDMENT In.(mm)	f'c = 2000 PSI (13.8 MPa)		f'c = 3000 PSI (20.7 MPa)		f'c = 4000 PSI (27.6 MPa)		f'c = 5000 PSI (34.5 MPa)	
		TENSION Lbs. (kN)	SHEAR Lbs. (kN)	TENSION Lbs. (kN)	SHEAR Lbs. (kN)	TENSION Lbs. (kN)	SHEAR Lbs. (kN)	TENSION Lbs. (kN)	SHEAR Lbs. (kN)
3/16 (4.8)	1 (25.4)	600 (2.7)	720 (3.2)	625 (2.8)	720 (3.2)	650 (2.9)	720 (3.2)	800 (3.6)	860 (3.8)
	1-1/4 (31.8)	845 (3.7)	720 (3.2)	858 (3.8)	720 (3.2)	870 (3.9)	720 (3.2)	1,010 (4.5)	860 (3.8)
	1-1/2 (38.1)	1,090 (4.8)	860 (3.8)	1,090 (4.8)	860 (3.8)	1,090 (4.8)	860 (3.8)	1,220 (4.8)	860 (3.8)
	1-3/4 (44.5)	1,450 (6.5)	870 (3.9)	1455 (6.5)	870 (3.9)	1,460 (6.5)	990 (4.4)	1,730 (7.7)	990 (4.4)
1/4 (6.4)	1 (25.4)	750 (3.3)	900 (4.0)	775 (3.4)	900 (4.0)	800 (3.6)	1,360 (6.1)	950 (4.2)	1,440 (6.4)
	1-1/4 (31.8)	1,050 (4.7)	900 (4.0)	1,160 (5.2)	900 (4.0)	1,270 (5.6)	1,360 (6.1)	1,515 (6.7)	1,440 (6.4)
	1-1/2 (38.1)	1,380 (6.1)	1,200 (5.3)	1,600 (7.2)	1,200 (5.3)	1,820 (8.1)	1,380 (6.1)	2,170 (9.7)	1,670 (7.4)
	1-3/4 (44.5)	2,020 (9.0)	1,670 (7.4)	2,200 (9.8)	1,670 (7.4)	2,380 (10.6)	1,670 (7.4)	2,770 (12.3)	1,670 (7.4)

Safe working loads for single installation under static loading should not exceed 25% of the ultimate load capacity.

ULTIMATE TENSION AND SHEAR VALUES (LBS/KN) IN HOLLOW BLOCK

ANCHOR DIA In.(mm)	ANCHOR EMBEDMENT In.(mm)	LIGHTWEIGHT BLOCK		MEDIUM WEIGHT BLOCK	
		TENSION Lbs. (kN)	SHEAR Lbs. (kN)	TENSION Lbs. (kN)	SHEAR Lbs. (kN)
3/16 (4.8)	1 (25.4)	220 (1.0)	400 (1.8)	340 (1.5)	730 (3.2)
1/4 (6.4)	1 (25.4)	250 (1.1)	620 (1.8)	500 (2.2)	1,000 (4.4)

Safe working loads for single installation under static loading should not exceed 25% of the ultimate load capacity.
NOTE: 3/16" Tapcon requires 5/32" bit, 1/4" Tapcon requires 3/16" bit.

ALLOWABLE EDGE AND SPACING DISTANCES

PARAMETER	ANCHOR DIA. In.(mm)	NORMAL WEIGHT CONCRETE			CONCRETE MASONRY UNITS (CMU)		
		FULL CAPACITY (Critical Distance Inches)	REDUCED CAPACITY (Minimal Distnce Inches)	LOAD REDUCTION FACTOR	FULL CAPACITY (Critical Distance Inches)	REDUCED CAPACITY (Minimal Distance Inches)	LOAD REDUCTION FACTOR
Spacing Between Anchors - Tension	3/16 1/4	3 4	1-1/2 2	0.73 0.66	3 4	1-1/2 2	1.00 0.84
Spacing Between Anchors - Shear	3/16 1/4	3 4	1-1/2 2	0.83 0.82	3 4	1-1/2 2	1.00 0.81
Edge Distance - Tension	3/16 1/4	1-7/8 2-1/2	1 1-1/4	0.83 0.82	3 4	2 2	0.91 0.81
Edge Distance - Shear	3/16 1/4	2-1/4 3	1-1/8 1-1/2	0.70 0.59	3 4	2 2	0.93 0.80

For SI: 1 inch = 25.4 mm

Design connection for existing wall to foundation at swinging door:

Determine applied load:

$w_{gap} := 2.5\ ft$ width of section not connected to floor

$h_{gap} := 15\ ft + 3\ in$ height of section not connected to floor

Wind load on this section of wall:

$V_{wind} := 115\ mph$ wind speed (ASCE 7-16, Figure 26.5-1b) Category 2 building

$k_d := 0.85$ ASCE 7-16, Table 26.6-1

$k_z := 0.85$ ASCE 7-16, Table 26.10-1 (less than 15ft above ground level)

$k_e := 1.00$ ASCE 7-16, Table 26.9-1

$k_{zt} := 1.0$ ASCE 7-16, 26.8.2

$$q_z := 0.00256 \cdot k_z \cdot k_{zt} \cdot k_d \cdot k_e \cdot \left(\frac{V_{wind}}{mph}\right)^2 \cdot psf = 24.461\ psf$$ ASCE 7-16, 26.10-1

$G := 0.85$ Gust effect factor, ASCE 7-16, 26.11.1

$Gcp_i := -0.18$ ASCE 7-16, Table 26.13-1

$C_p := 0.8$ ASCE 7-16, Figure 27.3-1

$$P_{wind} := \left(q_z \cdot C_p \cdot G - q_z \cdot Gcp_i\right) \cdot \frac{h_{gap}}{2} \cdot w_{gap} = 0.401\ kip$$ ASCE 7-16, 27.3-1

This is the wind load reaction at each end of the wall section, design connection to footing to carry this applied load:
Install nail plate (2x4) into concrete slab with concrete screws, install 4x4 block under existing end wall column and nail column to block, and block to nail plate.

Design Nail Plate:

2x Nail plate, calculate number of concrete anchors required to carry wind loads:

(2) 1/4" Tapcon Blue Concrete Screws or equivalent:

$N_{screw} := 3$ $\phi_{screw} := 0.25\ in$

$V_{screw} := 900\ lbf$ assuming 1" of embedment into 2ksi concrete

$f_{red\_spacing} := 0.82$ Reduction for 2in spacing (minimum allowable)

$f_{red\_edge} := 0.59$ Reduction for 1-1/2" edge distance (Minimum allowable)

$$V_{allow} := V_{screw} \cdot f_{red\_spacing} \cdot f_{red\_edge} = 0.435\ kip$$

```
Check := if N_screw · V_allow ≥ P_wind      = "Screws Ok for applied loads"
           ‖ "Screws Ok for applied loads"
         else
           ‖ "Check"
```

Ok with (2) Anchor screws, check with Powder Actuated nails:

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Design Nail Plate:

Check design with Powder actuated nails:

$V_{nail} := 166\ \text{lbf}$ This is the shear strength of a single Powder Actuated nail, with 1" of embedment (assuming using the 1516SDC with 2-1/2" overall length, in 2ksi concrete)

$N_{nail} := 3$ Number of nails

$$\text{Check} := \begin{array}{l} \text{if } \left(N_{nail} \cdot V_{nail}\right) \geq P_{wind} \\ \quad \Vert \text{"Ok for Shear"} \\ \text{else} \\ \quad \Vert \text{"Check"} \end{array} = \text{"Ok for Shear"}$$

(3) Nails Required

Contractor may use either (3) 1516SDC Powder Actuated nails or 1/4" Tapcon Blue Concrete screws

Ramset®
DRIVING JOBSITE SPEED

PERFORMANCE/SUBMITTAL

Ramset fasteners may be specified by their type or catalog number to satisfy fastening requirements.

PIN SPECIFICATIONS

- Made from AISI 1060-1065 steel. Austempered to a core hardness of 52-56 Rc
- Typical tensile strength: 270,000 psi
- Typical shear strength: 162,000 psi
- **STANDARD FINISHES**
  Proprietary black
  Mechanical zinc plate to a minimum thickness of .0002 meets requirements of ASTM B695—Class 5 Type 1
  Ramguard

APPROVALS/LISTINGS

- **ICC Evaluation Service, Inc.**
  #ESR-2690 Sill Plate
  #ESR-1799 Powder Pins & Clips
- **City of Los Angeles**
  #RR-22668 Powder pins

ICC ES™

FASTENERS IN NORMAL WEIGHT CONCRETE

PART NUMBER SERIES	SHANK DIAMETER (INCH)	MINIMUM PENETRATION (INCH)	INSTALLED IN STONE AGGREGATE CONCRETE CONCRETE COMPRESSIVE STRENGTH ALLOWABLE LOAD - *Ultimate Load*					
			2000 PSI		4000 PSI		6000 PSI	
			TENSION (LBS)	SHEAR (LBS)	TENSION (LBS)	SHEAR (LBS)	TENSION (LBS)	SHEAR (LBS)
1500/1600 SERIES	0.145	3/4	50 *655*	66 *739*	100 *511*	104 *552*	....	....
		1	152 *943*	166 *1229*	157 *937*	182 *1342*	....	....
		1-1/4	159 *1078*	265 *1665*	179 *1043*	267 *1538*	....	....
		1-1/2	154 *1450*	340 *2027*	209 *1357*	342 *1712*	....	....
SP SERIES	0.150	3/4	....	....	150 *803*	105 *786*	81 *493*	82 *454*
SP SERIES	.150/.180	1	154 *1043*	200 *1173*	243 *1307*	175 *1037*	189 *1125*	210 *1177*
		1-1/4	207 *1553*	230 *1636*	298 *1749*	218 *1471*	213 *1568*	305 *1780*
		1-1/2	....	....	384 *2126*	391 *1957*	239 *1886*	594 *2968*
1900 SERIES	0.145	3/4	105 *694*	71 *458*	101 *685*	99 *627*	....	....

Note 1: **ALLOWABLE** loads are shown in the **LARGE BOLD** font, *Ultimate* loads are shown in *smaller italic* font. **Note 2:** Testing conducted in accordance with ICC AC70 & ASTM E1190. **Note 3:** Safety factors are based on coefficient of variation. In accordance with ICC AC70, the safety factor will be no less than 5. **Note 4:** Values shown in concrete are for the fastener only. Connected members must be investigated separately. **Note 5:** Cyclic, fatigue, shock loads, and other design criteria may require a different safety factor. **Note 6:** Job site testing may be required to determine actual job site values. **Note 7:** Minimum edge distance is 3 inches unless otherwise approved. **Note 8:** For SI: 1 lbf = 4.448 N, 1 inch = 25.4 mm, 1 ksi = 6.89MPa

INSTALLED IN CONCRETE - CONCRETE COMPRESSIVE STRENGTH

PART NUMBER SERIES	SHANK DIA	EMBED	4000psi Normal Wt		6000psi Normal Wt		3000 Lt Wt on W Deck Lower Flute	
			Tension	Shear	Tension	Shear	Tension	Shear
TE SERIES	0.157	3/4	71	137	109	142	106	265
		1	278	216	214	400	152	327
		1-1/4	377	317	415	349	164	330
		1-1/2	242	479	-----	-----	238	448
TEC100 90° Ceiling Clip	0.157	7/8	207	-----	-----	-----	88	-----

Notes:
1) Fasteners tested to ASTM E1190 & ICC-ES AC70 (March 1, 2010)
2) Allowable loads are shown
3) Allowable loads and safety factors are based on coefficient of variation in accordance with ICC AC70, the safety factor will be no less than 5
4) Values shown for steel base materials have the pointed end of the fastener driven through the steel plate

INSTALLED IN A36 STRUCTURAL STEEL

PART NUMBER SERIES	SHANK DIA	SHANK TYPE	3/16		1/4		3/8		1/2	
			Tension	Shear	Tension	Shear	Tension	Shear	Tension	Shear
TE SERIES	0.157	KNURLED	323	606	562	673	934	820	603	766

INSTALLED IN A572-GR50 STRUCTURAL STEEL

PART NUMBER SERIES	SHANK DIA	SHANK TYPE	3/16		1/4		3/8		1/2	
			Tension	Shear	Tension	Shear	Tension	Shear	Tension	Shear
TE SERIES	0.157	KNURLED	442	676	630	662	760	725	582	532

FASTENERS IN NORMAL WEIGHT CONCRETE / STEEL

WWW.RAMSET.COM MADE IN AMERICA Ramset 41

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Design connection from nail plate to block to wall end column:

#8 wood screw:

$N_{screw} := 3$

$W := 82 \ \frac{lbf}{in}$ Table 11.2b, NDS 2012 withdrawal of #8 wood screws

$Z := 78 \ \frac{lbf}{in}$ Table 11L, NDS 2012 for #8 wood screws with 1in side member thickness (conservatively)

$C_D := 1.6$ NDS Table 2.3.2, wind load factor

$C_M := 1.0$ $C_t := 1.0$ $C_g := 1.0$ $C_\Delta := 1.0$

$C_{eg} := 1.0$ $C_{di} := 1.0$ $C_{tn} := 1.0$ Toe nail factor for screws is 1.0

$Z' := Z \cdot C_D \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta \cdot C_{eg} \cdot C_{di} \cdot C_{tn} = 124.8 \ \frac{lbf}{in}$ Table 10.3.1, NDS 2012

$W' := W \cdot C_D \cdot C_t \cdot C_{eg} \cdot C_{tn} = 131.2 \ \frac{lbf}{in}$ Table 10.3.1, NDS 2012

$W_{applied} := \frac{P_{wind}}{\cos(45 \ deg)} = 567.109 \ lbf$ total withdrawal force

$L_{embed} := 1.5 \ in = 1.5 \ in$ this is the required embedment depth into the nail plate, at a 45deg angle

$W_{resist} := (W' \cdot L_{embed}) \cdot N_{screw} = 590.4 \ lbf$

$Check := \text{if } W_{applied} \le W_{resist}$ = "Ok for Withdrawal"
  "Ok for Withdrawal"
else
  "Check"

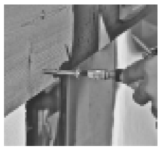
$Z_{resist} := (Z' \cdot L_{embed}) \cdot N_{screw} = 561.6 \ lbf$

$Check := \text{if } P_{wind} \le Z_{resist}$ = "Ok for Shear"
  "Ok for Shear"
else
  "Check"

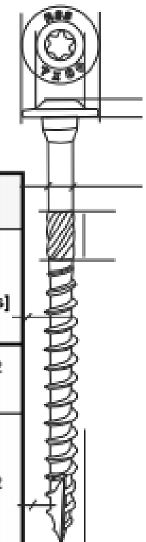
(3) #8 wood screws required to carry applied wind loads, conservatively use GRK-RSS 1/4" Diameter screws, these do not split the wood and are stronger than wood screws.

RSS™ Technical Data

Building Code Approved with a Limited Lifetime Warranty.
Scan to view to view for detailed IBC/IRC Code Compliant ESR #2442



RSS™ Rugged Structural Screws: Ideal for anywhere you would use a traditional lag screw and more. High tensile torque and shear strength means a 5/16" diameter RSS™ screw has the same strength as a 1/2" lag screw. Available from #10 up to 3/8" diameters in lengths from 1-1/2" to 16". Approved for use in all applications that include treated lumber. Also available in *PHE*INOX™ Stainless Steel, RSS™ JTS used for joists and trusses, RSS™ LPS for structural insulated panel systems and RSS™ LTF designed for log home and timber frames.



	FASTENER DESIGNATION	OVERALL LENGTH[1] (inches)	LENGTH OF THREAD[2] (inches)	MINOR THREAD DIAMETER[3] (inches)	SHANK DIAMETER[3] (inches)	OUTSIDE THREAD DIAMETER[3] (inches)	ALLOWABLE STEEL STRENGTH		
							Bending Yield Strength[4] F_{yb} (psi)	Tensile (psi) [pounds]	Shear (psi) [pounds]
RSS	1/4 x 2 1/2"	2 3/8	1 1/2	0.150	0.169	0.239	170,427	188,301 [3,336]	127,792 [2,264]
	1/4 x 3 1/8"	3 1/8	2						
	1/4 x 3 1/2"	3 1/2	2 3/8						
	5/16 x 2 1/2"	2 3/8	1 1/2	0.174	0.199	0.280	190,920	178,051 [4,247]	123,592 [2,948]
	5/16 x 2 3/4"	2 3/4	1 3/4						
	5/16 x 3 1/8"	3 1/8	2 1/8						
	5/16 x 3 1/2"	3 1/2	2 1/2						
	5/16 x 4"	3 7/8	2 3/4						
	5/16 x 5 1/8"	5	3 1/2						
	5/16 x 6"	5 7/8	3 7/8						
	3/8 x 3 1/8"	3 1/8	2 1/8	0.191	0.223	0.310	178,080	203,809 [5,824]	129,305 [3,695]
	3/8 x 4"	3 7/8	2 3/4						
	3/8 x 5 1/8"	5 1/8	3 1/2						
	3/8 x 6"	5 7/8	4						
	3/8 x 7 1/4"	7	4 1/2						
	3/8 x 8"	7 7/8	4 3/8						
	3/8 x 10"	9 3/4	5						
	3/8 x 12"	11 7/8	5 7/8						
	3/8 x 14 1/8"	14 1/8	5 7/8						
	3/8 x 16"	15 5/8	5 3/4						
LPS	1/4 x 8"	7 7/8	2 7/8	0.152	0.172	0.238	172,620	172,950 [3,155]	109,635 [2,000]
LTF	3/8 x 8"	7 7/8	3 7/8	0.191	0.220	0.310	167,580	179,390 [5,144]	114,525 [3,284]
	3/8 x 10"	9 7/8	3 7/8						
	3/8 x 12"	11 3/4	3 7/8						
PHE INOX	1/4 x 2 1/2"	2 3/8	1 1/2	0.152	0.170	0.237	111,460	103,799 [1,886]	90,260 [1,640]
	5/16 x 2 1/2"	2 3/8	1 5/8	0.171	0.195	0.276	118,360	104,767 [2,419]	86,880 [2,006]
	5/16 x 3 1/8"	3 1/8	2 1/8						
	5/16 x 4"	3 7/8	2 1/2						
	5/16 x 5 1/8"	5 1/8	3 3/8						
	5/16 x 6"	5 7/8	3 7/8						
JTS	1/4 x 3 3/8"	3 3/8	1 3/8	0.153	0.173	0.240	226,373	180,999 [3,312]	126,131 [2,308]
	1/4 x 5"	5	1 5/8						
	1/4 x 6 3/4"	6 3/4	1 1/2						

For **SI:** 1 inch = 25.4 mm; 1 psi = 6.9 kPa.

ULTIMATE LOAD VALUES TENSILE AND SHEAR

Determine max roof Snow Load:

Roof will be limited to a certain depth of snow, before it is removed, determine what depth of snow roof can carry:

$w_{lr} := 19 \text{ psf}$ — roof live load on structure, per pg 4 of 2013 calculations package

$w_{dead} := 1.5 \text{ psf}$ — dead load of truss member

$p_g := 9 \text{ psf}$ — design ground snow load is 60psf for this location in Maine, limit ground snow load to this amount

$\gamma_{snow} := 0.13 \cdot \frac{p_g}{ft} + 14 \text{ pcf} = 15.17 \text{ pcf}$ — ASCE 7-16, 7.7-1

$C_e := 1.0$ — ASCE 7-16, Table 7.3-1, partially exposed

$C_t := 1.2$ — Table 7.3-2, unheated structure

$I_s := 1.0$ — Table 1.5-2

$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 7.56 \text{ psf}$ — ASCE 7-16, 7.3-1

$C_s := 0.85$ — Figure 7.4-1, slippery surface with Ct=1.2

$p_s := C_s \cdot p_f = 6.426 \text{ psf}$ — sloped roof snow load, 7.4-1

$h_b := \frac{p_s}{\gamma_{snow}} = 5.083 \text{ in}$ — this is the depth of snow allowed on the roof

$p_{sunbalanced} := I_s \cdot p_g = 9 \text{ psf}$ — this is the unbalanced snow load for a gable roof (Figure 7.6-2)

Combined force on roof:

$S_{truss} := 3 \text{ ft}$ — Truss spacing

$w_{comb} := \left(w_{dead} + p_{sunbalanced}\right) \cdot S_{truss} = 31.5 \text{ plf}$ — D+S controls for combined loads

$L_1 := 11 \text{ ft} + 2 \text{ in}$ $\quad L_2 := 6 \text{ ft} + 10 \text{ in}$ — distance from exterior wall to support, to center of truss

$R_1 := \frac{w_{comb} \cdot L_1}{2} = 175.875 \text{ lbf}$ — Reaction at exterior wall

$R_3 := \frac{w_{comb} \cdot L_2}{2} = 107.625 \text{ lbf}$ — reaction at center of truss

$R_2 := R_1 + R_3 = 283.5 \text{ lbf}$ — Reaction at intermediate support

Determine max roof Snow Load:

Determine flexure and shear in sections:

$L_{long} := \dfrac{L_1}{\sin(71.5\ deg)} = 11.775\ ft$ actual length of long section, measured along slope of roof

$L_{short} := \dfrac{L_2}{\sin(71.5\ deg)} = 7.206\ ft$ actual length of short section, measured along slope of roof

$M_{ulong} := \dfrac{w_{comb} \cdot L_{long}^2}{8} = 6551.425\ lbf \cdot in$

$V_{ulong} := \dfrac{w_{comb} \cdot L_{long}}{2} = 185.459\ lbf$

$M_{ushort} := \dfrac{w_{comb} \cdot L_{short}^2}{8} = 2453.318\ lbf \cdot in$

$V_{ushort} := \dfrac{w_{comb} \cdot L_{short}}{2} = 113.49\ lbf$

$P_{comp} := R_3 \cdot \cos(71.5\ deg) = 34.15\ lbf$ compressive force induced by load applied at center of truss (no support at center)

Determine capacity of truss member:

$F'_c := 849.9\ psi$ $F_{cE} := 1328.8\ psi$ from pg 12 of 2013 calculations

$F'_b := 883.2\ psi$ from pg 15 of 2013 calculations

$F'_v := 155.3\ psi$ from pg 15 of 2013 calculations

$b_{timber} := 1.5\ in$ $d_{timber} := 5.5\ in$ 2x6 top chord

$S_{xchord} := \dfrac{b_{timber} \cdot d_{timber}^2}{6} = 7.563\ in^3$ $A_{chord} := b_{timber} \cdot d_{timber} = 8.25\ in^2$

$f_b := \dfrac{\max(M_{ushort}, M_{ulong})}{S_{xchord}} = 866.304\ psi$

$f_v := \dfrac{\max(V_{ulong}, V_{ushort}) \cdot 3}{2 \cdot A_{chord}} = 33.72\ psi$

$f_c := \dfrac{P_{comp}}{A_{chord}} = 4.139\ psi$

Determine max roof Snow Load:

Check shear, and combined flexure and compression:

Shear:

$$\text{Check} := \text{if } f_v \le F'_v \;\; \Big\| \;\text{“Ok for Shear”} \;\; \text{else} \;\; \Big\| \;\text{“Check”} \quad = \text{“Ok for Shear”}$$

Combined flexure and Compression:

$$\text{Check} := \text{if } \left(\frac{f_c}{F'_c}\right)^2 + \frac{f_b}{F'_b \cdot \left(1 - \frac{f_c}{F_{cE}}\right)} \le 1.0 \;\; \Big\| \;\text{“Ok for Combined forces”} \;\; \text{else} \;\; \Big\| \;\text{“Check”} \quad = \text{“Ok for Combined forces”}$$

Top chord of truss requires the snow live load to be limited to 9psf, which equates to 5" of snow depth, check roof girders:

Roof Girder Design:

$$w_{girder} := \frac{R_2}{S_{truss}} = 94.5 \text{ plf}$$ distributed load on girder

$$L_{girder} := 12 \text{ ft}$$ 12ft spacing of columns

$$M_{girder} := \frac{w_{girder} \cdot L_{girder}^2}{8} = 20412 \text{ lbf} \cdot \text{in}$$

$$V_{girder} := \frac{w_{girder} \cdot L_{girder}}{2} = 567 \text{ lbf}$$

$$b_{girder} := 1.5 \text{ in} \qquad d_{girder} := 7.25 \text{ in}$$

$$S_{xgirder} := 2 \cdot \frac{b_{girder} \cdot d_{girder}^2}{6} = 26.281 \text{ in}^3 \qquad A_{girder} := b_{girder} \cdot d_{girder} = 10.875 \text{ in}^2$$

$$f_{bgirder} := \frac{M_{girder}}{S_{xgirder}} = 776.675 \text{ psi}$$

$$f_{vgirder} := \frac{V_{girder}}{A_{girder} \cdot 2} = 26.069 \text{ psi}$$

$$F'_b := 885.8 \text{ psi} \qquad F'_v := 155.25 \text{ psi}$$ See pg 21 of 2013 calculations

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Roof Girder Design:

$$Check := \text{if } f_{bgirder} \le F'_b \;\; \| \text{"Ok for Flexure"} \;\; \text{else} \;\; \| \text{"Check"} = \text{"Ok for Flexure"}$$

$$Check := \text{if } f_{vgirder} \le F'_v \;\; \| \text{"Ok for Shear"} \;\; \text{else} \;\; \| \text{"Check"} = \text{"Ok for Shear"}$$

Roof girder is ok for flexure and shear with reduced snow load, all other members ok for applied snow load from roof by inspection, as they met 60psf limit from 2013 calculations:

Design floor load under existing apartment:

$\sigma_{LL} := 40\ psf$ — this is the design live load recommended for living space (ASCE 7-16, Table 4.3-1)

Floor Joists:
(2x8 @ 2'-0"):

$b_{joist} := 1.5\ in$ $\quad d_{joist} := 7.25\ in$ — joist dimensions

$S_{joist} := 2\ ft$ — spacing of joists

$$S_{xjoist} := \frac{b_{joist} \cdot d_{joist}^{\ 2}}{6} = 1.095\ ft \cdot in^2 \qquad A_{joist} := b_{joist} \cdot d_{joist} = 0.906\ ft \cdot in$$

$w_{LL} := \sigma_{LL} \cdot S_{joist} = 80\ plf$ — distributed live load on joist

$\sigma_{DL} := 5\ psf$ — assumed dead load on joist

$w_{DL} := \sigma_{DL} \cdot S_{joist} = 10\ plf$ — distributed dead load on joist

$F'_{bjoist} := 1069.5\ psi$ — page 32 of 2013 calculations

$L_{joist} := 12\ ft$ — span length of joists

$$M_{joist} := \frac{\left(w_{LL} + w_{DL}\right) \cdot L_{joist}^{\ 2}}{8} = 19440\ lbf \cdot in$$
Moment in joist

$$f_{bjoist} := \frac{M_{joist}}{S_{xjoist}} = 1479.382\ psi$$

$$f_{overstressed} := \frac{f_{bjoist} - F'_{bjoist}}{F'_{bjoist}} = 0.383$$
joists are overstressed for flexure, determine required repair to required capacity

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Design floor load under existing apartment:
Sister additional 2x8 to all floor joists:

$$f_{bjoistnew} := \frac{M_{joist}}{2 \cdot S_{xjoist}} = 739.691 \text{ psi}$$

$$Check := \text{if } f_{bjoistnew} \leq F'_{bjoist} \;\; \| \text{"Ok for Flexure"} \;\; \text{else} \;\; \| \text{"Check"} \quad = \text{"Ok for Flexure"}$$

Ok to sister additional 2x8 to each floor joist under apartment:

Check Carrying beams:
Existing (3) 2x8's are undersized, check with modified timber kicker repair called out in 2013:

$L_{trib} := 12 \text{ ft}$ tributary width of carrying beam

$w_{carry} := (7.8 \text{ psf} + \sigma_{LL}) \cdot L_{trib} = 573.6 \text{ plf}$ 7.8psf dead load per 2013 calculations, pg 37

$L_{carry} := 12 \text{ ft}$

Install kickers 3ft from supports (worst case moment is on middle section,
conservatively assuming simply supported, non-continuous)

$$L_{carrykick} := L_{carry} - 2 \cdot 3 \text{ ft} = 6 \text{ ft}$$

$$M_{carry} := \frac{w_{carry} \cdot L_{carrykick}^2}{8} = 30974.4 \text{ lbf} \cdot \text{in}$$

$V_{carry} := \frac{w_{carry} \cdot L_{carrykick}}{2} + \frac{w_{carry} \cdot 3 \text{ ft}}{2} = 2581.2 \text{ lbf}$ Reaction at kicker support

$b_{carry} := 1.5 \text{ in}$ $d_{carry} := 7.25 \text{ in}$

$S_{xcarry} := \frac{b_{carry} \cdot d_{carry}^2}{6} = 13.141 \text{ in}^3$ $A_{carry} := b_{carry} \cdot d_{carry} = 10.875 \text{ in}^2$

$$f_{bcarry} := \frac{M_{carry}}{3 \cdot S_{xcarry}} = 785.716 \text{ psi}$$

$F'_{bcarry} := 1005.6 \text{ psi}$ pg 39 of 2013 calcs

$$Check := \text{if } f_{bcarry} \leq F'_{bcarry} \;\; \| \text{"Ok for Flexure"} \;\; \text{else} \;\; \| \text{"Check"} \quad = \text{"Ok for Flexure"}$$

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Check Carrying beams:

$$f_{vcarry} := \frac{3 \cdot V_{carry}}{2 \cdot (3 \cdot A_{carry})} = 118.676\ psi$$

$$F'_{vcarry} := 135\ psi$$

$$Check := \text{if } f_{vcarry} \le F'_{vcarry} \;\Vert\; \text{"Ok for Shear"} \;\text{else}\; \Vert\; \text{"Check"} = \text{"Ok for Shear"}$$

Carrying beam is ok with kicker installed, check kicker

Kicker design:

$$P_{ukicker} := \frac{V_{carry}}{\cos(45\ deg)} = 3650.368\ lbf$$

$$l_e := \sqrt{(3\ ft)^2 + (3\ ft)^2} = 4.243\ ft$$

Use 4x4 kicker:

$d_{kicker} := 3.5\ in$ $\quad b_{kicker} := 3.5\ in$ $\quad A_{kicker} := d_{kicker} \cdot b_{kicker} = 12.25\ in^2$

$$f_{ckicker} := \frac{P_{ukicker}}{A_{kicker}} = 297.989\ psi$$

Determine compressive capacity of 4x4:

$F_c := 1000\ psi$ $\quad E_{min} := 400000\ psi$ NDS Table 4A

$C_D := 1.0$ NDS Table 2.3.2, for live load

$C_m := 1.0$ $\quad C_t := 1.0$ $\quad C_F := 1.15$ $\quad C_i := 1.0$ $\quad C_T := 1.0$

$c := 0.8$ For sawn lumber

$l_{ed} := \frac{l_e}{d_{timber}} = 9.257$ $\quad E_{min}' := E_{min} \cdot C_m \cdot C_t \cdot C_i \cdot C_T = 400000\ psi$

$F_{cE} := \frac{0.822 \cdot E_{min}'}{\left(l_{ed}^{2}\right)} = 3837.269\ psi$ $\quad F_{c''} := F_c \cdot C_D \cdot C_m \cdot C_t \cdot C_F \cdot C_i = 1150\ psi$

$$C_P := \frac{1 + \left(\frac{F_{cE}}{F_{c''}}\right)}{2 \cdot c} - \sqrt{\left(\frac{1 + \left(\frac{F_{cE}}{F_{c''}}\right)}{2 \cdot c}\right)^2 - \frac{\left(\frac{F_{cE}}{F_{c''}}\right)}{c}} = 0.928$$ NDS 3.7-1

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Kicker design:

$F'_c := F_{c''} \cdot C_P = 1067.681\ \text{psi}$

$\text{Check} := \begin{array}{l} \text{if } f_{ckicker} \le F'_c \\ \quad \| \text{“Ok for Compression”} \\ \text{else} \\ \quad \| \text{“Check”} \end{array} = \text{“Ok for Compression”}$

Kickers are ok for compression, check column for applied loads:

$P_{deadcolumn} := 1218.4\ \text{lbf}$ dead load applied to column

$P_{livecolumn} := 40\ \text{psf} \cdot (12\ \text{ft} \cdot 12\ \text{ft}) = 5760\ \text{lbf}$ live load applied to column

$P_{snowcolumn} := 7786\ \text{lbf}$ Snow load applied to column

for temporary, snow is limited on roof. However, this repair to the floor under the apartment is a permanent repair, include snow load combination check for when roof is repaired and snow load restrictions are no longer required:

$P_1 := P_{deadcolumn} + P_{livecolumn} = 6.978\ \text{kip}$ Load combination 1

$P_2 := P_{deadcolumn} + P_{snowcolumn} = 9.004\ \text{kip}$ Load combination 2

$P_3 := P_{deadcolumn} + 0.75 \cdot P_{livecolumn} + 0.75 \cdot P_{snowcolumn} = 11.378\ \text{kip}$ Load combination 3

$P_{column} := \max(P_1, P_2, P_3) = 11.378\ \text{kip}$

$P_{allow} := 17310.2\ \text{lbf}$ pg 43 of 2013 calculations

$\text{Check} := \begin{array}{l} \text{if } P_{column} \le P_{allow} \\ \quad \| \text{“Column Ok for applied loads”} \\ \text{else} \\ \quad \| \text{“Check”} \end{array} = \text{“Column Ok for applied loads”}$

Interior columns are ok for applied loads from apartment, check exterior wall with temporary lateral bracing without sheathing:

Exterior Sheathing:

Check exterior wall on 1st floor, worst case:
conservatively, check with 40psf live load on both floors, 2nd and third floor are limited to approximately 16psf, except where apartment has been improved, use 40psf design for both floors conservatively:

$P_{roof} := R_1 = 175.875\ \text{lbf}$ — snow and dead load from roof

$P_{roof\_dead} := \dfrac{1.5\ \text{psf} \cdot 11.17\ \text{ft}}{2} \cdot 2\ \text{ft} = 16.755\ \text{lbf}$ — dead load from roof

$P_{3rd\_live} := \sigma_{LL} \cdot (2\ \text{ft} \cdot 6\ \text{ft}) = 480\ \text{lbf}$ — Loads from 3rd floor

$P_{2nd\_live} := P_{3rd\_live} = 480\ \text{lbf}$ — Loads from 2nd floor

$P_{3rd\_dead} := 7.8\ \text{psf} \cdot (2\ \text{ft} \cdot 6\ \text{ft}) = 93.6\ \text{lbf}$ — dead loads from 3rd floor

$P_{2nd\_dead} := P_{3rd\_dead} = 93.6\ \text{lbf}$ — dead loads from 2nd floor

Load combinations:

$$P_{1ext} := P_{roof\_dead} + P_{3rd\_live} + P_{2nd\_live} + P_{3rd\_dead} + P_{2nd\_dead} = 1163.955\ \text{lbf}$$

$$P_{2ext} := P_{roof} + P_{roof\_dead} + P_{3rd\_dead} + P_{2nd\_dead} = 379.83\ \text{lbf}$$

$$P_{3ext} := P_{roof\_dead} + P_{3rd\_dead} + P_{2nd\_dead} + 0.75 \cdot \left(P_{roof} + P_{3rd\_live} + P_{2nd\_live}\right) = 1055.861\ \text{lbf}$$

$$P_{ext} := \max\left(P_{1ext}, P_{2ext}, P_{3ext}\right) = 1163.955\ \text{lbf}$$

Determine capacity of existing 2x4 studs, with bracing at midspan between floors:

$d_{ext} := 3.5\ \text{in}$ $\qquad$ $b_{ext} := 1.5\ \text{in}$

$A_{ext} := d_{ext} \cdot b_{ext} = 5.25\ \text{in}^2$

$f_{cext} := \dfrac{P_{ext}}{A_{ext}} = 221.706\ \text{psi}$

$C_F := 1.05$ — For 2x4's

$l_{e1} := (7\ \text{ft} + 10\ \text{in}) = 7.833\ \text{ft}$ — length of member unbraced between floors

$l_{e2} := 0.5 \cdot l_{e1} = 3.917\ \text{ft}$ — unbraced halfway between floors

$l_{ed1} := \dfrac{l_{e1}}{d_{ext}} = 26.857$ $\qquad$ $l_{ed2} := \dfrac{l_{e2}}{b_{ext}} = 31.333$

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Exterior Sheathing:

$$l_{ed} := \max\left(l_{ed1}, l_{ed2}\right) = 31.333$$

$$F_{cE} := \frac{0.822 \cdot E_{min}'}{\left(l_{ed}^{2}\right)} = 334.903 \text{ psi} \qquad F_{c''} := F_c \cdot C_D \cdot C_m \cdot C_t \cdot C_F \cdot C_i = 1050 \text{ psi}$$

$$C_P := \frac{1 + \left(\frac{F_{cE}}{F_{c''}}\right)}{2 \cdot c} - \sqrt{\left(\frac{1 + \left(\frac{F_{cE}}{F_{c''}}\right)}{2 \cdot c}\right)^2 - \frac{\left(\frac{F_{cE}}{F_{c''}}\right)}{c}} = 0.294$$

NDS 3.7-1

$$F'_c := F_{c''} \cdot C_P = 309.11 \text{ psi}$$

$$\text{Check} := \begin{array}{l} \text{if } f_{cext} \le F'_c \\ \quad \| \text{“Ok for Compression”} \\ \text{else} \\ \quad \| \text{“Check”} \end{array} = \text{“Ok for Compression”}$$

with bracing at midspan between floors, ok for temporary bracing,
with floor loads limited and snow loads reduced to max 5in depth