

Design Calculations

For

***Douglas Road Center, Lot 4
NE Victoria Drive and NE Douglas, Lee's Summit, Missouri***

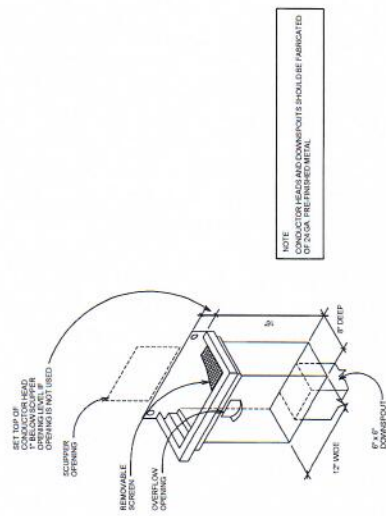
Architect,

Scharhag Architects
6247 Brookside Blvd., Suite #204
Kansas City, Missouri, 64113

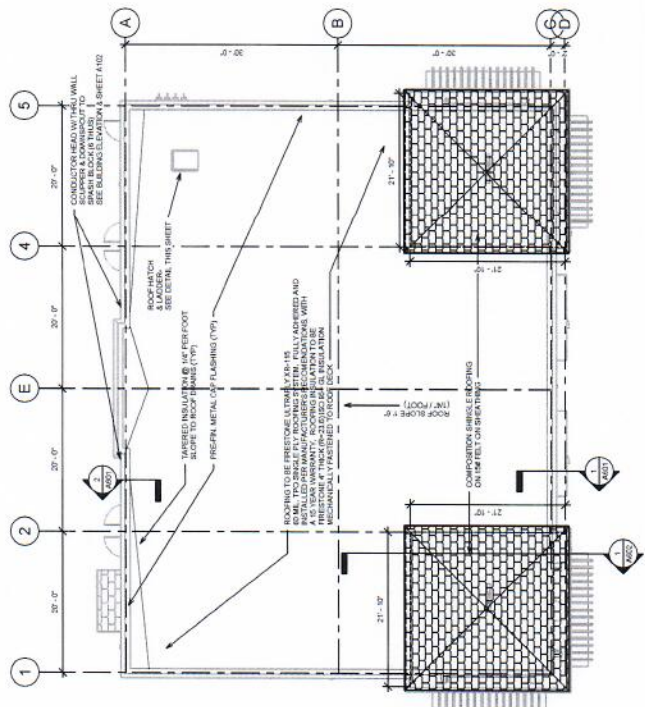
Date: November 1, 2020



Joseph A. Towns, RA, PE, SE, AIA, NCARB, LEED AP, BD+C
Missouri Professional Engineer (Structural) #E-22017
Certification Applies to Pages Cover, through **52**



SCUPPER DETAIL AT PARAPET WALL
NOT TO SCALE



ALL ROOF PARAPETS ARE MIN. 3" ABOVE ROOF AND WILL ACT AS FALL PROTECTION.

③ Roof hatch w/ guard rail
4.8 - 4.9

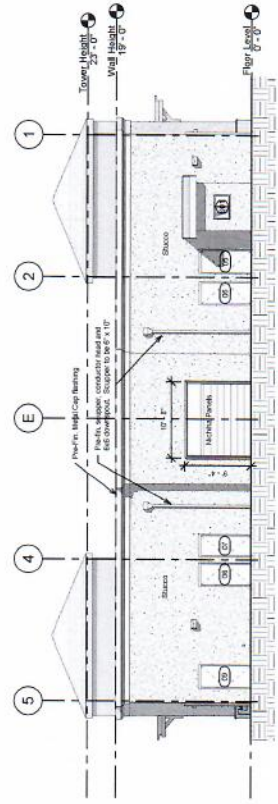
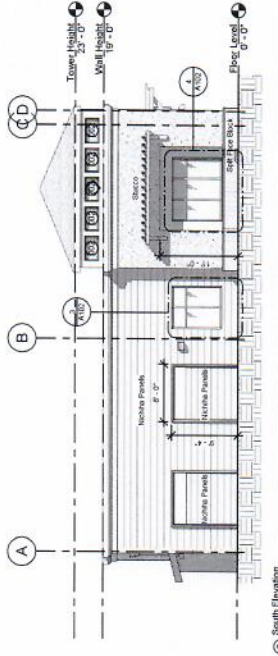
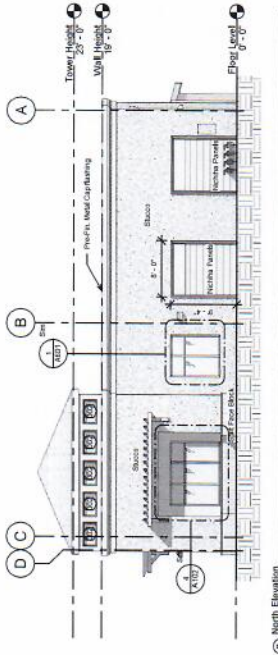
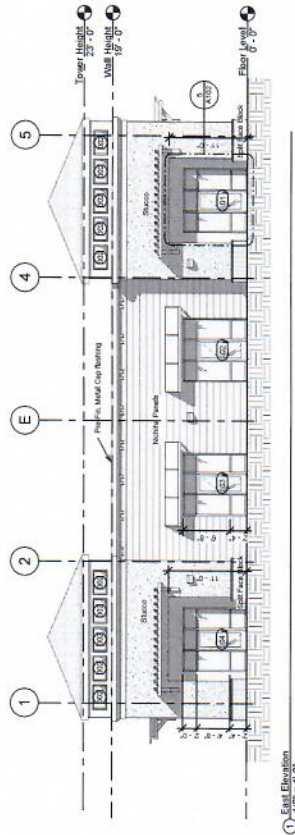
No.	Description	Date
1	Revision Schedule	

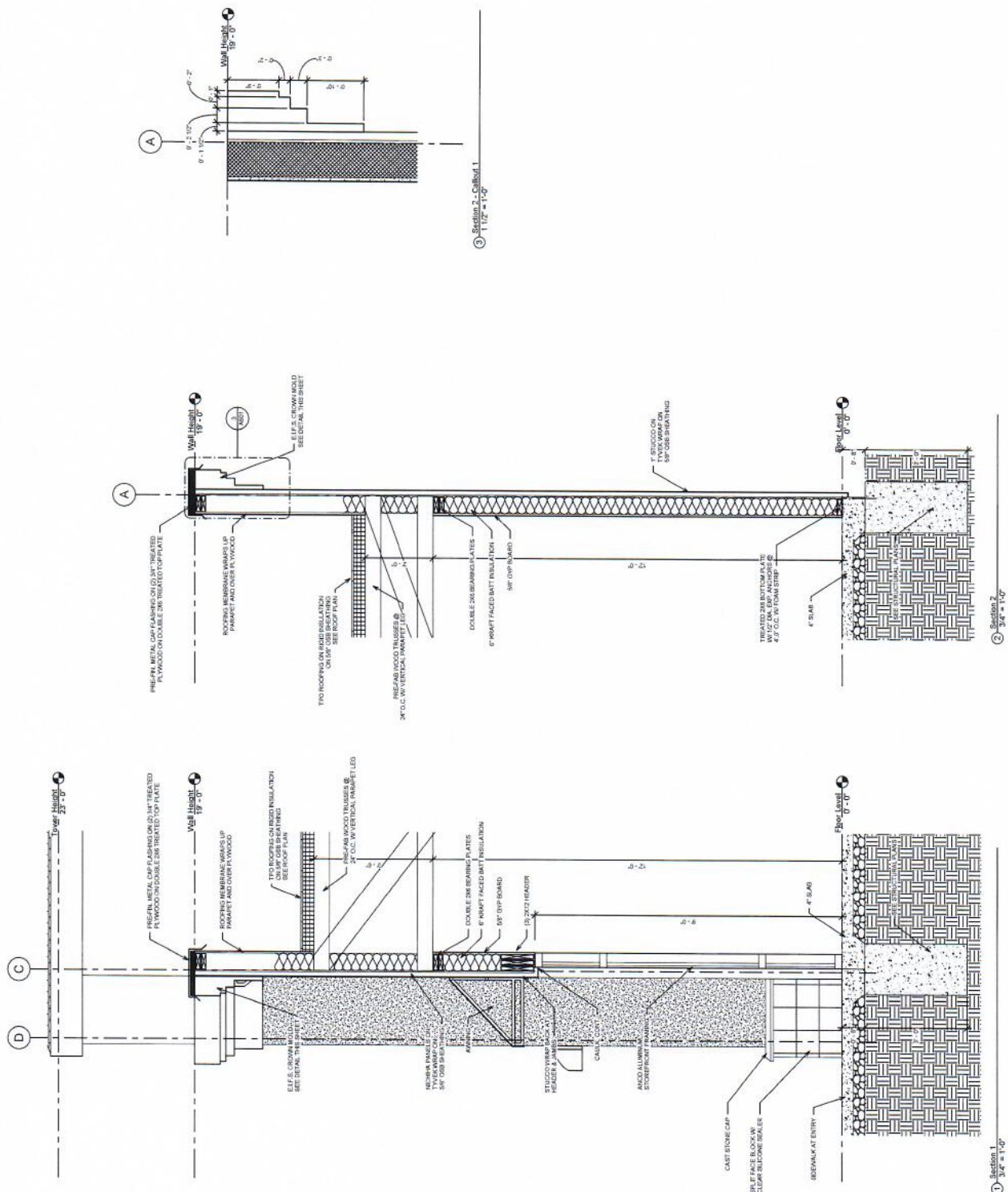
Elevations

Project Number: 2222
Date: 11/02/2020

A201

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



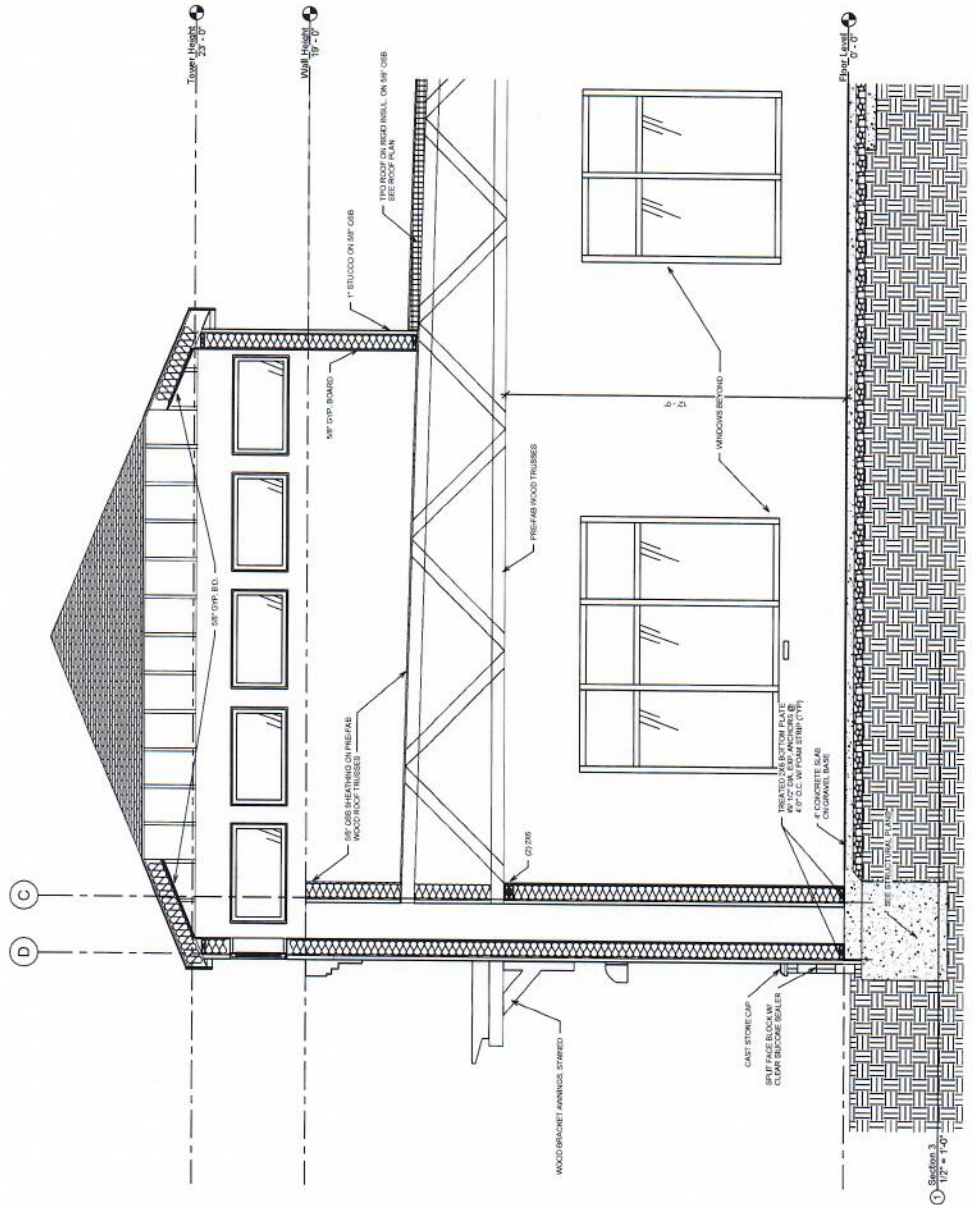


Wall Sections

No.	Description	Date
	Revisions Schedule	

NEW BUILDING FOR
 DOUGLAS ROAD CENTER Lot 4
 NE VICTORIA DRIVE & NE DOUGLAS ST.
 (Auto Generated by AutoCAD LT 2019)
 (AutoCAD LT 2019)

scharhay
 SCHARHAY ARCHITECTS
 6247 Brookside Blvd, #204 Kansas City, Mo 64113
 Phone: 816-656-5055 Scharhayarch@gmail.com



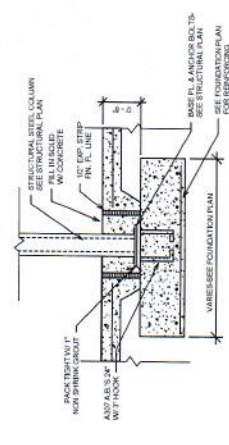
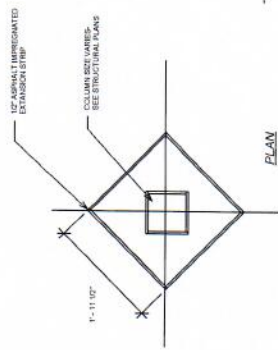
No.	Description	Date
	Revision Schedule	

Foundation

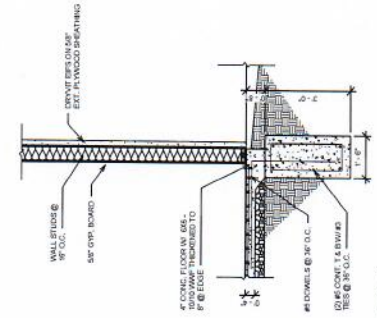
Project number	2222
Date	11.02.2020

S101

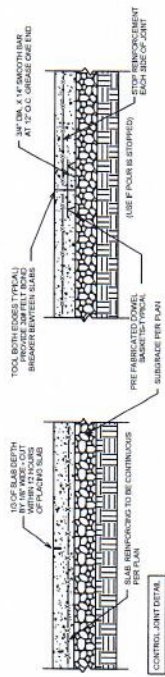
Scale: As indicated



① Column Detail
 1/2" = 1'-0"

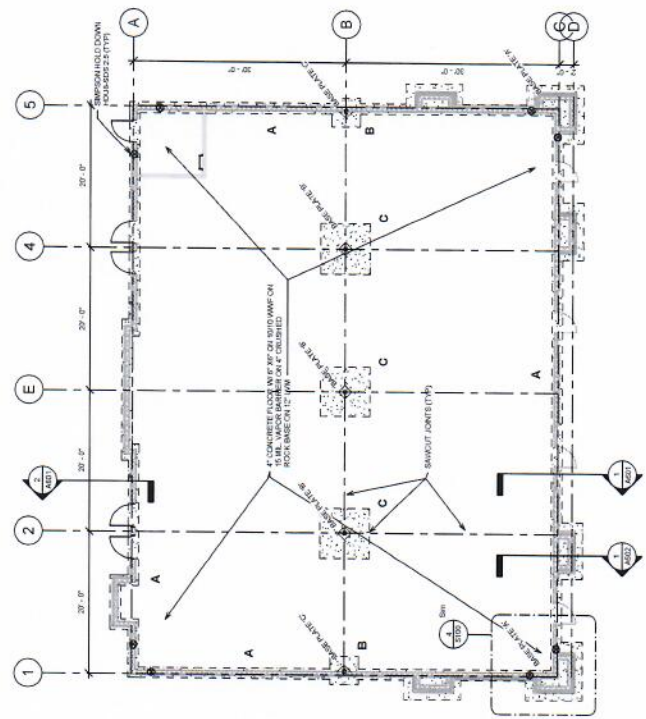


③ Foundation 3
 1/2" = 1'-0"



② Sawcut Floor Step
 3/4" = 1'-0"

Type Mark	Type	Type Comments
A	18" x 3' 0" deep	
B	4' x 4' x 3' 0" deep	(6) #6's Each way at top and bottom w/ (1) #6 @ each corner
C	7' x 7' x 2' 0" deep	(7) #6's each way at bottom

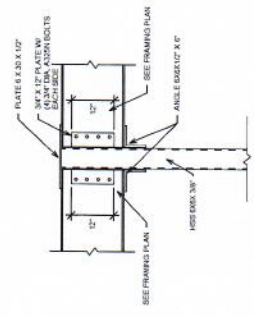


④ Foundation Plan
 1/8" = 1'-0"

No.	Description	Date
1	Revision Schedule	

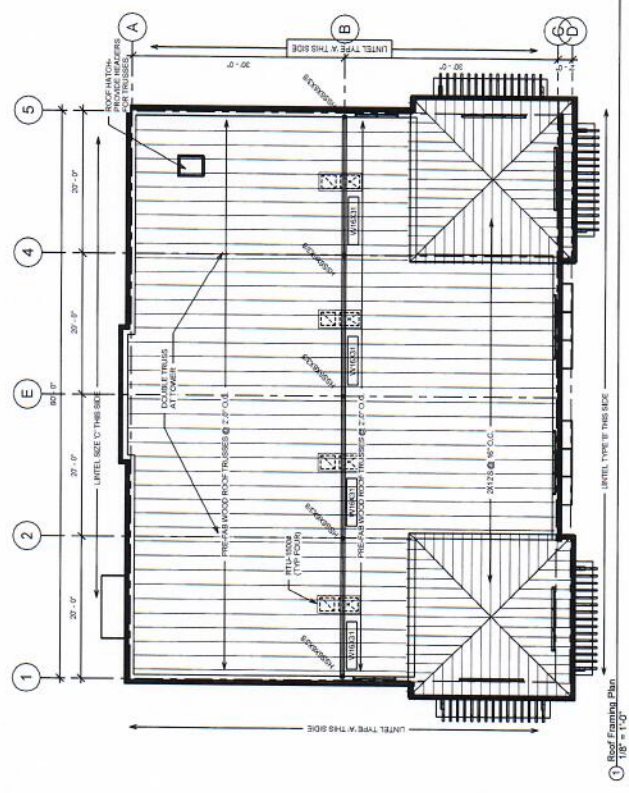
Framing	
Project number	2222
Date	11.02.2020
S102	
Scale	As indicated

11/2/2020 11:32:12 AM

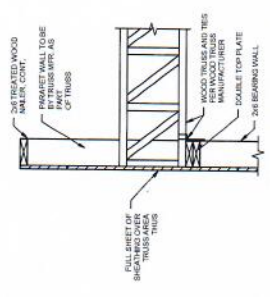


2 Beam Connection Detail
 3/4" x 1'-0"
 ALL WELLS TO BE W/ FLEETS

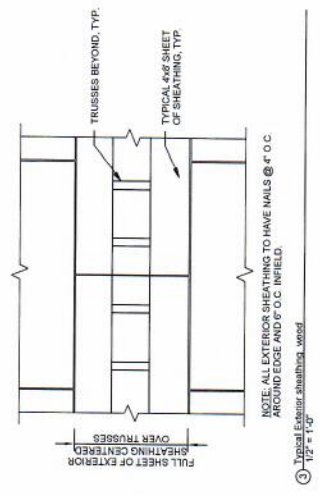
LINTEL SCHEDULE
TWO SIZES
LINTEL TYPE 'A'
LINTEL TYPE 'B'
LINTEL TYPE 'C'
PROVIDE (6) 2X12S BEARING AT EACH END OF LINTELS



1 Roof Framing Plan
 1/8" = 1'-0"



3 Typical section at truss bearing wall
 1/2" = 1'-0"



4 Typical exterior sheathing, wood
 1/2" = 1'-0"

NOTE: ALL EXTERIOR SHEATHING TO HAVE NAILS @ 4" O.C. AROUND EDGE AND 6" O.C. INTERIOR.

Loads and Codes

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TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o
AND MINIMUM CONCENTRATED LIVE LOADS^a

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2,000
Computer use	100	2,000
3. Armories and drill rooms	150 ^m	—
4. Assembly areas		
Fixed seats (fastened to floor)	60 ^m	—
Follow spot, projections and control rooms	50	—
Lobbies	100 ^m	—
Movable seats	100 ^m	—
Stage floors	150 ⁿ	—
Platforms (assembly)	100 ^m	—
Other assembly areas	100 ^m	—
5. Balconies and decks ^b	1.5 times the live load for the area served, not required to exceed 100	—
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors		
First floor	100	—
Other floors	Same as occupancy served except as indicated	—
9. Dining rooms and restaurants	100 ^m	—
10. Dwellings (see residential)	—	—
11. Elevator machine room and controlroom grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes	100	—
On single-family dwellings only	40	—
14. Garages (passenger vehicles only)	40 ⁿ	Note a
Trucks and buses	See Section 1607.7	
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals		
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000
18. Hotels (see residential)	—	—
19. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150 ^{b, n}	1,000
20. Manufacturing		
Heavy	250 ⁿ	3,000
Light	125 ⁿ	2,000
21. Marquees, except one- and two-family dwellings	75	—
22. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	—	—
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000

(continued)

TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o
AND MINIMUM CONCENTRATED LIVE LOADS^a

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
23. Penal institutions		
Cell blocks	40	—
Corridors	100	—
24. Recreational uses:		
Bowling alleys, poolrooms and similar uses	75 ^m	—
Dance halls and ballrooms	100 ^m	—
Gymnasiums	100 ^m	—
Ice skating rink	250 ⁿ	—
Reviewing stands, grandstands and bleachers	100 ^{c, m}	—
Roller skating rink	100 ^m	—
Stadiums and arenas with fixed seats (fastened to floor)	60 ^{c, m}	—
25. Residential		
One- and two-family dwellings		
Uninhabitable attics without storage ⁱ	10	—
Uninhabitable attics with storage ^{j, k}	20	—
Habitable attics and sleeping areas ^k	30	—
Canopies, including marquees	20	—
All other areas	40	—
Hotels and multifamily dwellings		
Private rooms and corridors serving them	40	—
Public rooms and corridors serving them	100	—
26. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies:		
Fabric construction supported by a skeleton structure	5 ^m	—
All other construction, except one- and two-family dwellings	20	—
Ordinary flat, pitched, and curved roofs (that are not occupiable)	20	—
Primary roof members exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000
All other primary roof members		300
Occupiable roofs:		
Roof gardens	100	—
Assembly areas	100 ^m	—
All other similar areas	Note I	Note I
27. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
28. Scuttles, skylight ribs and accessible ceilings	—	200
29. Sidewalks, vehicular driveways and yards, subject to trucking	250 ^{d, n}	8,000 ^e
30. Stairs and exits		
One- and two-family dwellings	40	300 ^f
All other	100	300 ^f

(continued)

TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_u
AND MINIMUM CONCENTRATED LIVE LOADS^a

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage) Heavy Light	250 ^b 125 ^b	—
32. Stores Retail First floor Upper floors Wholesale, all floors	100 75 125 ^b	1,000 1,000 1,000
33. Vehicle barriers	See Section 1607.9	
34. Walkways and elevated platforms (other than exitways)	60	—
35. Yards and terraces, pedestrians	100 ^m	—

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm²,
 1 square foot = 0.0929 m², 1 pound per square foot = 0.0479 kN/m²,
 1 pound = 0.004448 kN, 1 pound per cubic foot = 16 kg/m³.

- Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of this table or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4½ inches by 4½ inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.
- The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:
 - The nominal book stack unit height shall not exceed 90 inches.
 - The nominal shelf depth shall not exceed 12 inches for each face.
 - Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.
- Design in accordance with ICC 300.
- Other uniform loads in accordance with an approved method containing provisions for truck loadings shall be considered where appropriate.
- The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.
- The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.
- Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).
- See Section 1604.8.3 for decks attached to exterior walls.
- Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.
- Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.
 The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:
 - The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is not less than 30 inches.
 - The slopes of the joists or truss bottom chords are not greater than two units vertical in 12 units horizontal.
 The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.

(continued)

TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_u
AND MINIMUM CONCENTRATED LIVE LOADS^a

- Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.
- Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.13.3.
- Live load reduction is not permitted.
- Live load reduction is only permitted in accordance with Section 1607.11.1.2 or Item 1 of Section 1607.11.2.
- Live load reduction is only permitted in accordance with Section 1607.11.1.3 or Item 2 of Section 1607.11.2.

1607.6 Helipads. Helipads shall be designed for the following live loads:

- A uniform live load, L , as specified in Items 1.1 and 1.2. This load shall not be reduced.
 - 40 psf (1.92 kN/m²) where the design basis helicopter has a maximum take-off weight of 3,000 pounds (13.35 kN) or less.
 - 60 psf (2.87 kN/m²) where the design basis helicopter has a maximum take-off weight greater than 3,000 pounds (13.35 kN).
- A single concentrated live load, L , of 3,000 pounds (13.35 kN) applied over an area of 4.5 inches by 4.5 inches (114 mm by 114 mm) and located so as to produce the maximum load effects on the structural elements under consideration. The concentrated load is not required to act concurrently with other uniform or concentrated live loads.
- Two single concentrated live loads, L , 8 feet (2438 mm) apart applied on the landing pad (representing the helicopter's two main landing gear, whether skid type or wheeled type), each having a magnitude of 0.75 times the maximum take-off weight of the helicopter, and located so as to produce the maximum load effects on the structural elements under consideration. The concentrated loads shall be applied over an area of 8 inches by 8 inches (203 mm by 203 mm) and are not required to act concurrently with other uniform or concentrated live loads.

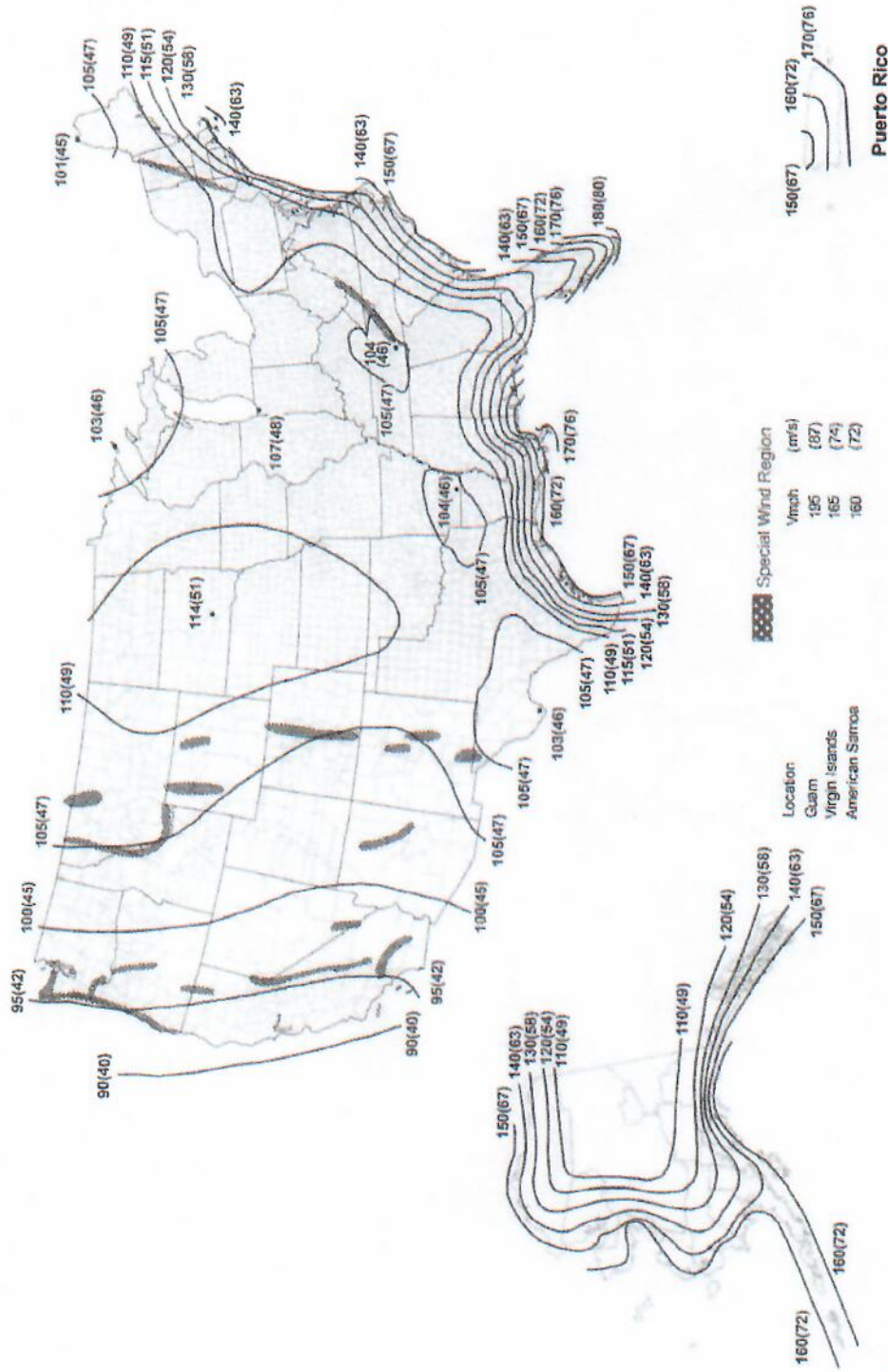
Landing areas designed for a design basis helicopter with maximum take-off weight of 3,000-pounds (13.35 kN) shall be identified with a 3,000 pound (13.34 kN) weight limitation. The landing area weight limitation shall be indicated by the numeral "3" (kips) located in the bottom right corner of the landing area as viewed from the primary approach path. The indication for the landing area weight limitation shall be a minimum 5 feet (1524 mm) in height.

1607.7 Heavy vehicle loads. Floors and other surfaces that are intended to support vehicle loads greater than a 10,000-pound (4536 kg) gross vehicle weight rating shall comply with Sections 1607.7.1 through 1607.7.5.

1607.7.1 Loads. Where any structure does not restrict access for vehicles that exceed a 10,000-pound (4536 kg) gross vehicle weight rating, those portions of the structure subject



FIGURE 1608.2—continued
GROUND SNOW LOADS, p_g , FOR THE UNITED STATES (psf)



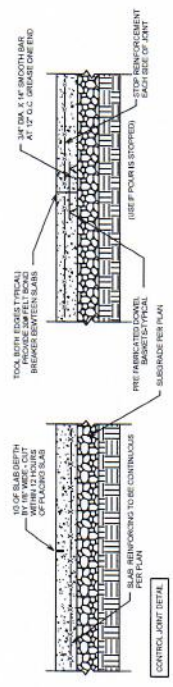
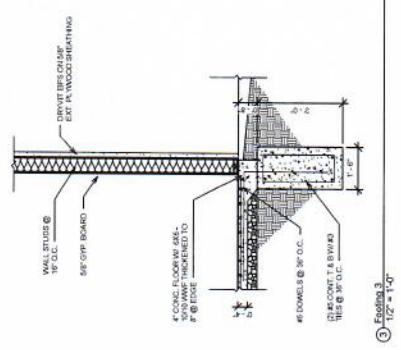
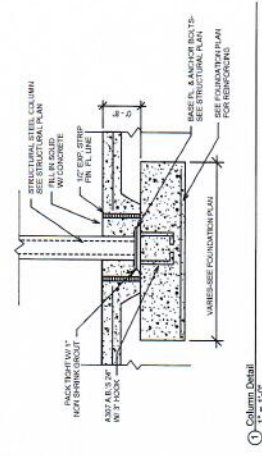
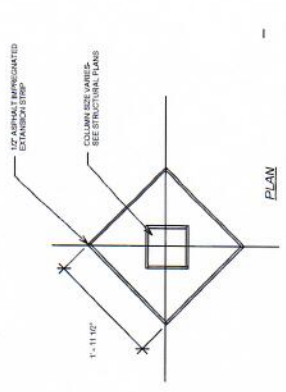
Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).
6. Location-specific basic wind speeds shall be determined using www.atcouncil.org/windspeed

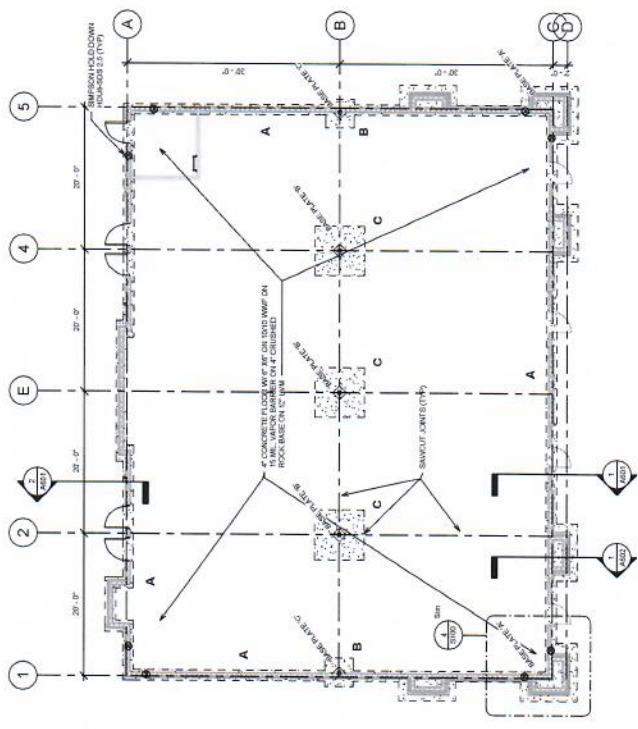
FIGURE 1609.3(1)
BASIC DESIGN WIND SPEEDS, V, FOR RISK CATEGORY II BUILDINGS AND OTHER STRUCTURES

Misc. Calculations

Foundation



Structural Foundation Schedule		
Type Mark	Type	Type Comments
A	16 x 3' 0\"	
B	4 x 4 x 2' 0\"	(6) #5's Each way at top and bottom w/ (1) #6 @ each corner
C	7' x 7' x 2' 0\"	(7) #5's each way at bottom



4 Foundation Plan 1/8" = 1'-0"

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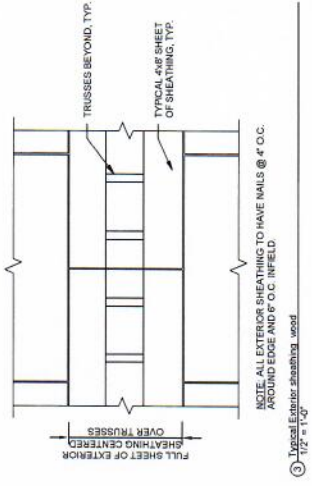
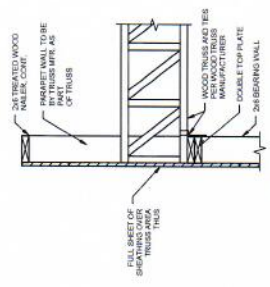
Framing

No.	Description	Date
	Revision Schedule	

NEW BUILDING FOR
DOUGLAS ROAD CENTER Lot 4
 NE VICTORIA DRIVE & NE DOUGLAS ST.

Scharhay
 KENNETH SCHARHAY COMPANY ARCHITECTS
 6247 Brookside Blvd. #204 Kansas City, Mo 64113
 Phone: 816-565-5055 Scharhayarcht@gmail.com

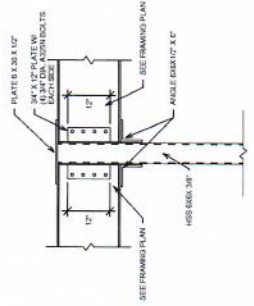
① Typical Section at Truss bearing wood
 1" = 1'-0"



③ Typical Exterior sheathing wood
 1/2" = 1'-0"

W21x44 truss

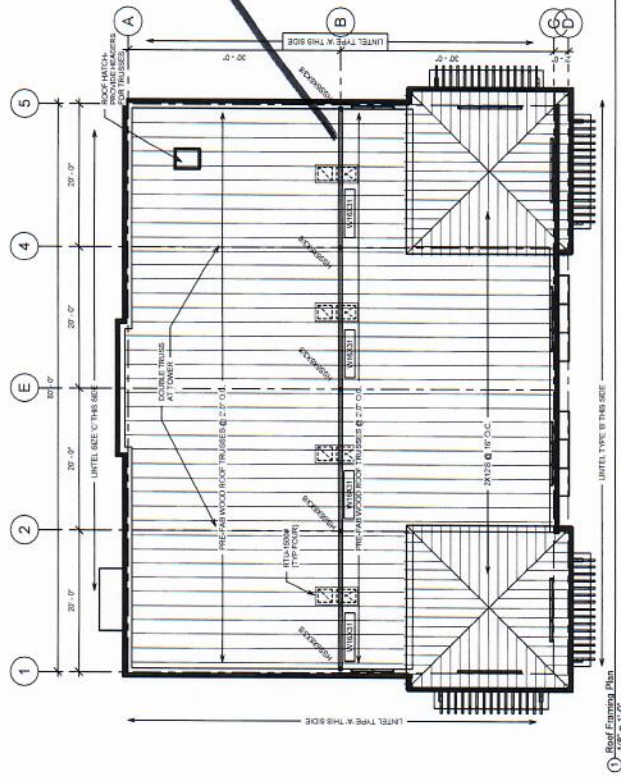
② Beam Connection Detail
 3/4" = 1'-0"
 ALL WELDS TO BE W/ FILLERS



UNTEL SCHEDULE

UNTEL TYPE A	TWO 2X12S
UNTEL TYPE B	TWO 2X12S
UNTEL TYPE C	(2) 2X12S

PROVIDE (6) 2X12S BEAMS AT EACH END OF UNTEL B



① Roof Framing Plan
 1/8" = 1'-0"

Design Calculations

Douglas Road Center, Lot 4

NE Victoria Drive and NE Douglas Road
Lee's Summit, Missouri

The following calculations are based on "out-to-out" dimensions as determined by the designing engineer.

All values are IBC, "simplified method" for structures under 30 feet in height.

Unit Description:	60'x80'x22'	Monoslope
Length:	60 feet	Re: Plan Drawings OUT TO OUT
Width:	80 feet	Re: Plan Drawings OUT TO OUT
Eave Height:	22 feet	Re: Plan Drawings
Roof Mean Height above eave:	1 feet	Re: Plan Drawings
Height Adjustment Factor:	1	Height Adjustmt Lamda, Fig 28.6-1
Design Wind Speed:	115 mph	IBC Table 1609.6.2.1 (1), pg 294
Roof Slope:	14.5 degrees	By Calculation
Exposure:	B	ASCE, pg195
Floors Considered in Calcs:	1 Floor	Re: Plan Drawings

For Simplicity and 120 Loading, Wall Forces Rounded to 20 psf
Roof Forces take as Exp C Rounded to -30

From IBC: Longitudinal		(Wind Force at End of Building)
Wind Pressure to Eave:	20 psf	ASCE Table 27.6-1
Wind Pressure to Roof:	-30 psf	ASCE Table 27.6-2
Vertical Pressures:		
Windward:	20 psf	ASCE Table 27.6-1
Leeward:	0 psf	
From IBC: Transverse		(Wind Force at Length of Building)
Wind Pressure to Eave:	20 psf	ASCE Table 27.6-1
Wind Pressure to Roof:	-30 psf	ASCE Table 27.6-2
Vertical Pressures:		
Windward:	20 psf	ASCE Table 27.6-1
Leeward:	0 psf	

Pressumed Dead Loads

Roof:	16 psf	Re: Plan Drawings
Floor:	0 psf	Re: Plan Drawings, Slab on Grade
Ext. Walls:	16 psf	Re: Plan Drawings

MWFRS - Longitudinal -

60'x80'x22'

Horizontal Wind Loads

Building Width	80 feet	Re: Plan Drawings
Building Eave Height	22 feet	Re: Elevation Drawings
Roof Mean Height above eave	1 feet	Re: Elevation Drawings
Wind Pressure to Eave	20 psf	IBC Table 1609.6.2.1(1)
Wind Pressure to Roof	-30 psf	IBC Table 1609.6.2.1(1)
Height Adjustment Factor	1	IBC Table 1609.6.2.1 (4)
Total Wind Load at Walls	35.2 kips	By Calculation
Total Wind Load at Roof	-2.4 kips	By Calculation

Total Wind Shear (Longitudinal) 32.8 kips By Calculation

Unit Shear Stress at Base 0.27 kips/ft By Calculation

Theoretical Chord Stress at T.O.W.

Bending Moment:	328.0 k-ft	$M=(wl^2)/8$
Bending Stress:	5.5 k/in ²	$Stress=M/d$
Area for 2 Plate top chord:	10.5 in ²	By Calculation
Stress per Square Inch:	520.6 psi	By Calculation

Check Design

Shear to Top of Wall: 16.4 kips By Calculation

Theor. OT Reactions: 6.01 kips By Calculation

Theor. Panel Shear Stress: 0.26 klf By IBC

Lineal Feet Required: 23.58 feet By Calculation

OK

MWFRS - Transverse -

60'x80'x22'

24

Horizontal Wind Loads

Building Length	60 feet	Re: Plan Drawings
Building Eave Height	22 feet	Re: Elevation Drawings
Roof Mean Height above eave	1 feet	Re: Elevation Drawings
Wind Pressure to Eave	20 psf	IBC Table 1609.6.2.1(1)
Wind Pressure to Roof	-30 psf	IBC Table 1609.6.2.1(1)

Height Adjustment Factor	1	IBC Table 1609.6.2.1 (4)
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Total Wind Load at Walls	26.4 kips	By Calculation
Total Wind Load at Roof	-0.69 kips	By Calculation

Total Wind Shear (Transverse)	25.71 kips	By Calculation
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Unit Stress at Base (Transverse)	0.161 kips/ft	By Calculation
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Theoretical Chord Stress at T.O.W.

Bending Moment:	192.8 k-ft	$M=(wl^2)/8$
Bending Stress:	2.4 k/in ²	$\text{Stress}=M/d$
Area for 2 Plate top chord:	10.5 in ²	By Calculation
Stress per Square Inch:	229.6 psi	By Calculation

OK

Shear to Top of Wall:	12.855 kips	By Calculation
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Theor. OT Reactions:	4.7 kips	By Calculation
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Theor. Panel Shear Stress:	0.3 klf	By IBC
----------------------------	---------	--------

Lineal Feet Required:	18.5 feet	By Calculation
-----------------------	-----------	----------------

OK

MWFRS - Longitudinal - 60'x80'x22'

Vertical Wind Loads (Uplift)

Building Width	80 feet	Re: Plan Drawings
Building Eave Height	22 feet	Re: Elevation Drawings
Roof Mean Height above eave	1 feet	Re: Elevation Drawings
Wind Pressure to Leeward	-10.7 psf	IBC Table 1609.6.2.1(1)

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Height Adjustment Factor	1	IBC Table 1609.6.2.1 (4)
Total Wind Load at Roof	-25.7 kips	By Calculation
Total Wind Force Vertical (Longitudinal)	-0.4 kips/ft	By Calculation
Dead Load Forces		
Roof:	0.640 klf	By Calculation
Floors:	0.000 klf	By Calculation
Walls:	0.352 klf	By Calculation
Total:	0.992 klf	By Calculation
NET, Unit Stress at Foundation (Longitudinal)	0.564 kips/ft	By Calculation Dead Load Controls, Uplift < DL

MWFRS - Transverse -

60'x80'x22'

Vertical Wind Loads (Uplift)

Building Length	60 feet	Re: Plan Drawings
Building Eave Height	22 feet	Re: Elevation Drawings
Roof Mean Height above eave	1 feet	Re: Elevation Drawings
Wind Pressure to Windward	20 psf	IBC Table 1609.6.2.1(1)
Height Adjustment Factor	1	IBC Table 1609.6.2.1 (4)
Total Wind Windward Pressure	48 kips	By Calculation
Total Wind Force Vertical (Transverse)	0.8 kips/ft	By Calculation
Dead Load Forces		
Roof:	0.64 klf	By Calculation
Floors:	0 klf	By Calculation
Walls:	0.352 klf	By Calculation
Total:	0.992 klf	By Calculation
NET, Unit Stress at Foundation (Transverse)	1.792 kips/ft	By Calculation Dead Load Controls, Uplift < DL

MWFRS - Longitudinal -

60'x80'x22'

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OTM - Horizontal Wind Forces

Total Wind Shear (Longitudinal)	32.80 kips	By Calculation
Total Wind Force Vertical (Longitudinal)	-25.68 kips	By Calculation Controlling Value ONLY
Summing Moments at End	58.48 kips	By Calculation
Unit Stress at Extrema (longitudinal)	0.73 kips/ft Dead Load Controls	By Calculation

MWFRS - Transverse - 60'x80'x22'

OTM - Horizontal Wind Forces

Total Wind Shear (Transverse)	25.71 kips	By Calculation
Total Wind Force Vertical (Transverse)	48.00 kips	By Calculation Controlling Value ONLY
Summing Moments at End	73.71 kips	By Calculation
Unit Stress at Extrema (Transverse)	1.23 kips/ft Hold Downs Required	By Calculation

Foundation Loads Due to Wind

Longitudinal Wind Loads

Shear Stress/foot	0.27 kips/ft	By Calculation
Vertical Force/foot	0.56 kips/ft	By Calculation
OT Force (Vertical)	0.73 kips/ft	By Calculation
Total Shear	32.80 kips	By Calculation
Shear @ Brace = $T_{\text{shear}}/2$	16.40 kips	By Calculation

Transverse Wind Loads

Shear Stress/foot	0.16 kips/ft	By Calculation
Vertical Force/foot	1.79 kips/ft	By Calculation
OT Force (Vertical)	1.23 kips/ft	By Calculation
Total Shear	25.71 kips	By Calculation

Simpson Hold Down Confirmation

By Standard, Check Simpson HD8A, w/ 5/8" A307, Bolt Embed 12" min.	3.66 kips/conn. Simpson, pg 29
--	--------------------------------

Controlling Vertical Load: Hold Down Spacing:	1.23 kips/ft	By Calculation
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OK, Hold Downs per Code and Calculation

Nail Pattern Confirmation, for Structural Sheathing

By Standard, Check	0.27 kips/ft	6" o.c. at Edges
Edge Condition for 15/32" Plywood with 2x4 studs, and 8d, 1 3/8" nails.	0.36 kips/ft	4" o.c. at Edges

Controlling Shear Stress	0.27 kips/ft	By Calculation
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Design Edge Pattern 4 in. o.c Edge Spacing Required

Shear Wall Quantity Confirmation

Transverse Loading

SW2	1/2" Gyp blkcd	134 plf	Table 2306.3.1
Lineal feet available for SW1		135 lf	By Calculation
Shear Capacity		18.09 kips	By Calculation
Factored Capacity		18.09 kips	By Calculation
Total Shear		25.71 kips	By Calculation
% Shear Resisted		70%	
% Shear Remaining		30%	Additional Capacity Required

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SW1	15/32" Shtg blkcd	255 plf	Table 2306.3.1
Lineal feet available for SW1	135 lf		By Calculation
Shear Capacity	34.43 kips		By Calculation
Opening Reduction Factor	0.77		Table 2305.3.7.2
Wind Increase Factor	1.40		IBC 2306.4.1
Factored Capacity	37.11 kips		By Calculation
Combined Capacity	55.20 kips		By Calculation
Total Shear	25.71 kips		By Calculation
% Shear Resisted	215%		
% Shear Remaining	-115%	Shear Demand Met	

Longitudinal Loading

SW2	1/2" Gyp blkcd	175 plf	Table 2306.3.1
Lineal feet available for SW1	115 lf		By Calculation
Shear Capacity	20.125 kips		By Calculation
Reduction Factor	1		
Factored Capacity	20.125 kips		By Calculation
Total Shear	32.8 kips		By Calculation
% Shear Resisted	61%		
% Shear Remaining	39%	Additional Capacity Required	

SW1	15/32" Shtg blkcd	255 plf	Table 2306.3.1
Lineal feet available for SW1	115 lf		By Calculation
Shear Capacity	29.33 kips		By Calculation
Opening Reduction Factor	1		
Wind Increase Factor	1.40		IBC 2306.4.1
Factored Capacity	41.06 kips		By Calculation
Combined Capacity	61.18 kips		By Calculation

Total Shear	32.80 kips	By Calculation
% Shear Resisted	187%	
% Shear Remaining	-87% Shear Demand Met	

Shear Wall Design, Wind

Longitudinal Loading

SW3	1/2" Gyp blkcd	175 plf	Table 2306.3.1
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Lineal feet available for SW1	115 lf	By Calculation
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Shear Capacity	20.13 kips	By Calculation
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Reduction Factor	1	
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Factored Capacity	20.13 kips	By Calculation
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Total Shear	32.80 kips	By Calculation
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% Shear Resisted	61%	
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% Shear Remaining	39%	
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SW2	15/32" Shtg blkcd	255 plf	Table 2306.3.1
-----	-------------------	---------	----------------

Lineal feet available for SW1	115 lf	By Calculation
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Shear Capacity	32.80 kips	By Calculation
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Opening Reduction Factor	1	
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Wind Increase Factor	1.40	IBC 2306.4.1
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Factored Capacity	45.92 kips	By Calculation
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Combined Capacity	66.05 kips	By Calculation
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Total Shear	32.80 kips	By Calculation
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% Shear Resisted	201%	
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% Shear Remaining	-101% Capacity Demand Met	
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Design Calculations

Douglas Road Center, Lot 4

Lee's Summit, Missouri

Load Combinations: IBC 1605.2, IBC 2018

Dead Load	D	15 psf/total
Wind Load	W	20 psf/total
Flood Load	F	0 psf/total
Live Load (Floor)	L	40 psf/total
Height Load	H	1 unit factor
Snow Load	S	25 psf/total (Ice on Snow, IBC 1607)
Rain Load	R	15 psf/total
Earthquake Load	E	15 psf/total
LL adjustment	f1	0.5 other LL's
Snow adjustment	f2	0 Non-Saw Tooth Roofs

Load Cases:

1	$1.4(D + F) =$	21 Eq 16-1
2	$1.2(D + F) + 1.6(L + H) + .5(L_r \text{ or } S \text{ or } R)$	96.1 Eq 16-2
3	$1.2(D + F) + 1.6(L_r \text{ or } S \text{ or } R) + 1.6H + (f_1L \text{ or } .5W)$	79.6 Eq 16-3
4	$1.2(D + F) + 1.0 W + f_1L + 1.6H + .5(L_r \text{ or } S \text{ or } R)$	72.1 Eq 16-4
5	$1.2(D + F) + 1.0 E + f_1L + 1.6H + f_2S$	39.6 Eq 16-5
6	$.9D + 1.0 W + 1.6H$	35.1 Eq 16-6
7	$.9(D + F) + 1.0 E + 1.6H$	30.1 Eq 16-7
8	optional case	
9	optional case	
10	optional case	

Controlling Case: 2

96.1 psf/total

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Design Calculations

Douglas Road Center, Lot 4

Lee's Summit, Missouri

Wood Frame Building with Ctr Line Beam

Misc. Calcs - Spread Footing Calculations

Footing Carrying;

*	Dead Load (conservative estimate)	25.00 kips	By Design
*	Live Load (conservative estimate)	25.00 kips	By Design
	Total Column Load	50.00 kips	Per Drawings
*	Bending Moment	12 kip-feet	By Design, Seismic
	Bending Moment	144 in-kips	M*12
**	Allow. Soil Pressure	2000 psf	Conservative Estimated (IBC)
*	f'c =	4000 psi	By Design
	fy =	60000 psi	By Design

Select Footing Size

*	Footing Width (minor axis)	7 feet	By Design
**	Footing Length (major axis)	7 feet	By Design
*	Footing thickness	2 feet	By Design

Direct Soil Pressure psf

Axial Bearing Stress = 1320 psf P/A
 $P/3Af*(4L/L-2e) =$ 1859 psf

Footing Weight = 15 kips

$$\text{Solve for "x" (length of bearing)} = 9.94 \text{ feet} = ((\text{Sum of Loads}) * 1000) / (0.5 * P_a * F_w)$$

$$\text{Check Overturning} = M_o = 108 \text{ inch-kips} \quad (M_{des} * .75)$$

$$M_r = 2455 \text{ inch-kips} = ((\text{Sum of Loads}) * 1000) / (0.5 * P_a * F_r)$$

$$S.F. = 22.73 \quad M_r / M_o$$

Check Punching Shear

	$V_c =$	253 psi	ACI
Column Size	Length =	12 inches	By Design
	Width =	12 inches	By Design

Punching Shear Failure Perimeter:

* Flexural Shear Critical Section = 16.97 inches Estimated @ (.7071*t)

Punching Shear Critical Section = 8.49 inches = Flex Cr/2

$$b_o = 115.88$$

Ultimate Load to Footing:

$$P_u = 77.50 \text{ kips} = 1.4DL + 1.7LL$$

Ultimate Punching Shear:

Soil Load due to Column Load Only

$P_c =$	1020.41 psf	= P/A ftg
$V_u =$	28.09 kips	= P/Lin at edge

Calculate Depth Required:

$$V_u / \phi * b_o * V_c = d = 1.13 \text{ inches}$$

Check Flexural Shear

$$V_c = 126 \text{ ksi}$$

Soil Pressure at Critical Section:

$$P_{soil} = 1562.58$$

$$\text{Total Shear Force} = 11975.73 \text{ lbs}$$

$$V_u = 17528.08 \text{ lbs}$$

$$d = 1.94 \text{ inch}$$

Flexural Shear Does NOT Control Design

Check Bending

M= 419.45 kip-feet

Mu= 713.07 kip-feet

Select Reinforcement

Mu/ftg length = 102 kip-ft/ft

Minimum As = 0.52 inches²

** Try 8- #6's 3.52 inches²/foot Each Way!!

Mu Capacity = 284 kip-ft/ft

OK

Check if Tensile Steel is Required at Top of Footing:

* Overburden = 2.50 feet

** Soil weight = 0.11 pcf

Mu = 4.40 kip-feet/foot

fc = Mc/I= 45.82 psi

fr = 205.55 psi

OK, No Top Steel Required

Design Calculations

Douglas Road Center, Lot 4

Lee's Summit, Missouri

End of Girder Foundations

Misc. Calcs - Spread Footing Calculations

Footings Carrying; Sides

*	Dead Load (conservative estimate)	12.5	By Design
*	Live Load (conservative estimate)	12.5	By Design
	Total Column Load	25 kips	Per Drawings
*	Bending Moment	1 kip-feet	By Design, Seismic
	Bending Moment	12 in-kips	M*12
**	Allow. Soil Pressure	2010 psf	Conservative Estimated (IBC)
*	f'c =	4000 psi	By Design
	fy =	60000 psi	By Design

Select Footing Size

*	Footing Width (minor axis)	4 feet	By Design
**	Footing Length (major axis) L =	6 feet	By Design
*	Footing thickness	3 feet	By Design

Direct Soil Pressure 497 psf

Axial Bearing Stress = 1492 psf P/A
P/3Af*(4L/L-2e) = 2008 psf

Solve for "x" (length of bearing)= 8.92 feet $=((\text{Sum of Loads})*1000)/(0.5*Pa*Fw)$

Check Overturning = $M_o =$ 9 inch-kips $(M_{des}*.75)$

$M_r =$ 1176 inch-kips $=((\text{Sum of Loads})*1000)/(0.5*Pa*F_r)$
S.F. = 130.70 M_r/M_o

Check Punching Shear

Column Size $V_c =$ 253 psi ACI
 $\text{Length} =$ 12 inches By Design
 $\text{Width} =$ 12 inches By Design

Punching Shear Failure Perimeter:

* Flexural Shear Critical Section = 25.46 inches Estimated @ $(.7071*t)$
Punching Shear Critical Section 12.73 inches $=\text{Flex Cr}/2$

$b_o =$ 149.82

Ultimate Load to Footing:

$P_u =$ 38.75 kips $=1.4DL + 1.7LL$

Ultimate Punching Shear:

Soil Load due to Column Load Only

$P_c =$ 1041.67 psf $=P/A \text{ ftg}$
 $V_u =$ 26.13 kips $=P/Lin \text{ at edge}$

Calculate Depth Required:

$V_u/\phi*b_o*V_c =$ $d =$ 0.81 inches

Check Flexural Shear

$V_c =$ 126 ksi

Soil Pressure at Critical Section:

$P_{soil} =$ 1922.31

Total Shear Force 7859.79 lbs

$V_u =$ 12975.37 lbs

$d =$ 2.51 inch

Flexural Shear Does NOT Control Design

Check Bending

M= 207.18 kip-feet

Mu= 352.20 kip-feet

Select Reinforcement

Mu/ftg length = 88 kip-ft/ft

Minimum As = 0.78 inches²

** Try 5 - #6's 2.20 inches²/foot Each Way!!

Mu Capacity = 306 kip-ft/ft

OK

Check if Tensile Steel is Required at Top of Footing:

* Overburden = 2.50 feet

** Soil weight = 0.11 pcf

Mu = 3.85 kip-feet/foot

fc = Mc/I= 17.83 psi

fr = 205.55 psi

OK, No Top Steel Required

Design Calculations

Douglas Road Center, Lot 4

Lee's Summit, Missouri

Steel Girder at Mid Line of Building

Supporting Roof

Cap. Check - Max Span=	20	6096 mm	Dim. Converted to mm
Trib Width	Per Beam (2 total)	60 ft	Per Drawings
Beam Span		20 ft	Per Drawings
Beam Loading Live =	LL = (1.6*.025)	0.04 kips/ft^2	Gravity Load Calcs
Beam Loading Dead=	DL = (1.4*.03)	0.04 kips/ft^2	Gravity Load Calcs
Trib Area	(half wall width, per bm)	30.00 feet	
Total Beam Loading		2.46 kips/lin. ft	
Point Load 1 (as applicable)	a =	1 kips 20 ft	
	b =	0 ft	
Point Load 2 (as applicable)	a =	0 kips 0 ft	
	b =	20 ft	
Point Load 3 (as applicable)	a =	0 kips 0 ft	
	b =	20 ft	
Point Load 4 (as applicable)	a =	0 kips 0 ft	
	b =	20 ft	

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Total Moment

$$\begin{aligned} \mu_u = .9 * M_n &= 110.70 \text{ kip-feet} & \mu_u = .9 * M_n &= \\ &= 1328.40 \text{ kip-inches} \end{aligned}$$

Reactions

$$\begin{aligned} \text{Right Reaction} &= 25.60 \text{ kips} \\ \text{Left Reaction} &= 24.60 \text{ kips} \end{aligned}$$

$$Z_x \text{ req'd} = 29.52 \text{ in}^3 = (\mu_u * 12) / (0.9 * F_y)$$

OK

Select

W21x44

$$\begin{aligned} \text{Depth} &= 21.00 \text{ Inch} \\ \text{Area} &= 13.00 \text{ Inch}^2 \\ Z_x &= 95.40 \text{ Inch}^3 \\ I_{xx} &= 959.00 \text{ inch}^4 \\ F_y &= 50.00 \text{ ksi} \end{aligned}$$

Allowable Defl.

L/360

$$0.67 \text{ inches}$$

Span/Allowable

Deflection

Point & Uniform Ldg

$$0.33 \text{ inches}$$

AISC Simple Bm

$$Z_x \text{ act} = 95.4 \text{ in}^3$$

$$Z_x \text{ req'd} = 29.52 \text{ in}^3$$

OK

$$\begin{aligned} M(\text{plastic}) = F_y * Z &= 4770 \text{ kip-in} & \text{AISC LRFD} \\ M_y = F_y * S &= 4770 \text{ kip-in} & \text{AISC LRFD} \\ M_p \text{ Limit} = 1.5 * M_y &= 7155 \text{ kip-in} & \text{AISC LRFD} \end{aligned}$$

OK

$$J = 0.803$$

$$\begin{aligned} L_p &= 10.32024 & \text{F10-5} \\ L_p (\text{actual}) &= 5.0000 \end{aligned}$$

Design Calculations

Douglas Road Center, Lot 4

Lee's Summit, Missouri

Misc. Calcs - Continuous Grade Beam Check

Concrete Beam Calcs - 18"x36" Grade Beam

Typical Strip -

*	Section Trib Width (Selected)	3 ft	Per Drawings
*	Max Span	10 ft	Per Drawings
*	Loading Dead	1.5 kips/ft ²	Per Drawings
	Loading Live	0.5 kips/ft ²	Per Drawings
	Trib Area	1.5 feet	= 3feet/2000
	$f'_c =$	4000 psi	By Design
	$f_y =$	60000 psi	By Design
	Beam Ldg (Dead)	2.25 kips/lin. ft	
	Beam Ldg (Live)	0.75 kips/lin. ft	
	Moment (Dead)	28.13 kip-feet	$M=wl^2/2$
	Moment (Live)	9.38 kip-feet	$M=wl^2/2$
	Total Moment	37.50 kip-feet	summation
	Factored Moment (Dead)	39.38 kip-feet	$M=wl^2/2 * 1.4$
	Factored Moment (Live)	15.94 kip-feet	$M=wl^2/2 * 1.6$
	Total Factored Mom.	55.31 kip-feet	summation
	Reactions (Dead)	11.25 kips	
	Reactions (Live)	3.75 kips	
	Total Reaction		

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b = 18.00 inch By Design
d = 30.00 inch By Design

Omega = 0.0114
Omega' = 0.0113

rho = 0.0008

As, Rqd As = 0.41 in²

As, min = 1.71 ACI 10.5.1
Select = 1-#6's 0.44 in²/ft

OK

Beam Check

T = Asfy = 26.40 kips

a = Asfy / .85f'cb = 0.43 inches

Moment Strength = 707.68 in-kips
or 58.97 kip-feet

OK

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Design Calculations

Douglas Road Center, Lot 4

Lee's Summit, Missouri

Foundation Calcs - Continuous Footing

Typical

Wall Perimeter,

Roof Loads

1st Total Trib Width	12 feet	From Drawings
2nd Total Trib Width	0 feet	From Drawings
Dead Load	35 psf	Loads and Codes
Live Load	30 psf	Loads and Codes
Design Load	0.390 klf	Sum of Loads

Second Floor

1st Total Trib Width	0 feet	From Drawings
2nd Total Trib Width	0 feet	From Drawings
Dead Load	0 psf	Loads and Codes
Live Load	0 psf	Loads and Codes
Design Load	0 klf	Sum of Loads

First Floor

1st Total Trib Width	8.5 feet	From Drawings
2nd Total Trib Width	0 feet	From Drawings
Dead Load	60 psf	Loads and Codes
Live Load	100 psf	Loads and Codes
Total Wall Height (This section)	12 feet	From Drawings
Estimated Wall Weight	45 psf	Area * Load

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Design Load	1.22 klf	Sum of Loads
Total Design Load	1.61 klf	Summation of Loads
Allowable Bearing Stress	2.00 k/ft ²	OK By IBC
Footing Width	0.81 ft	Load/Stress
Footing Depth	2.00 ft	By Design

Design Calculations

Douglas Road Center, Lot 4

Lee's Summit, Missouri

LRFD, Short Hss Column Calculation

CITED TEXT: STRUCTURAL STEEL DESIGN, LRFD 3rd Edition, A. WILLIAMS PhD, SE

Member =	HSS6x6x3/8	AISC
Factored Load =	50 kips	Conservative
rx = ry =	2.28 inches	Ref. AISC
phi =	0.90	LRFD
Ag =	7.58 inches^2	AISC
Fy =	42.00 ksi	AISC
E =	29000.00 ksi	AISC
K =	1.00	
L =	16.00 feet	
	192.00	
rx = ry =	2.28 inches	
Fe =	40.36 ksi	E3-4
.44FY =	18.48 ksi	
Short Column, Eq. E3-2 Controls		
k = FY/Fe =	1.04	
Fcr =	27.17 ksi	E3-2
phi*Ag*Fc =	185.35 kips	
% Capacity =	27%	
	OK	

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Design Calculations

Douglas Road Center, Lot 4

Lee's Summit, Missouri

Misc. Calcs - Window Headers 12'-0"

Supporting - Roof - Wall - Parapet

Beam Designation None

Beam Span	Max. Typical	12 feet	Re: Plan Drawings
Uniform Load Width		16.88 feet	Re: Plan Drawings
Uniform Load (total load LL+DL)	$1.6 \cdot .025 + 1.4 \cdot .035$	0.075 ksf	Load & Codes
Beam Loading		1.27 klf	ULW * UL
Point Load 1		0.0 kips	
Point Load 1 Location		6.0 feet	
Point Load 2		0.0 kips	By Design
Point Load 2 Location		6.0 feet	
Point Load 3		0.0 kips	
Point Load 3 Location		6.0 feet	

For Conservatism All Moments Consider at Midspan !!
All beams considered continually braced at top flange !!

Moments	Uniform Load	22.781 k-ft	$M = (wl^2)/8$
	Point Load 1	0.00 k-ft	$M = (wl^2)/8$
	Point Load 2	0.00 k-ft	$M = (wl^2)/8$
	Point Load 2	0.00 k-ft	$M = (wl^2)/8$
	Total	22.78 k-ft	Summation
Allow Deflection	L/360	0.400 inches	Span/allowable
Reaction		7.59375 kips	$= (w \cdot l) \cdot \text{span} / 2$

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Size Beam

By Design

b =	4.5 inches
d =	14.0 inches
Ixx =	1029.0 inches ⁴
E =	1800 ksi
Deflection, Uniform	0.3188 inches
Deflection, PL 1	0.0000 inches
Deflection, PL 2	0.0000 inches
Deflection, PL 3	0.0000 inches
Deflection Total	0.3188 inches
	OK
Shear Stress	0.121 ksi

Check Bending Stresses

Sxx=	147.0 inches ³
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Adjustment Factors

Cd	0.95 Section 2.3.2 Duration
Cm	1.00 ASD Wood Design Manual
Ct	1.00 ASD Wood Design Manual
Cl	Braced Section 4.4.1 Stability
Cf	1.00 ASD Wood Design Manual
Cv	1.03 Table 5A Wood Supplement page 57
Adjusted Cv	1.00 Max Value from Suppl. page 57
Cfu	1.00 ASD Wood Design Manual
Cr	1.15 ASD Rep Factor, NDS Section 4.3.9 Ref: Page 30
Fb=	1800 #2 GRD, DF

F'b=	2019.87 psi
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Actual fb=	1859.69 psi
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OK

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Seismic

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SEISMIC LOADS - ASCE 7-2010

Site Class Soil Definition	D	Per Geotechnical recommendation
	.2 Second Response $S_s =$	0.12 Map Figure 22-1 p.211
	$F_a =$	1.6 Table 11.4-1
SMS =	$F_a \times S_s =$	0.192 Eq. 11.4-1
SDS =	$\frac{2}{3} \text{ SMS} =$	0.128 Eq. 11.4-3
	1 Second Response $S_1 =$	0.06 Map Figure 22-2 p.213
	$F_v =$	2.4 Table 11.4-2
SM1 =	$F_v \times S_1 =$	0.144 Eq. 11.4-2
SD1 =	$\frac{2}{3} \text{ SM1} =$	0.096 Eq. 11.4-4
Occupancy	II	Table 1-1 (p3)
Seismic Design Category	A	Table 11.4-1 (p115)
	B	Table 11.4-2 (p115)
Importance Factor $I =$	1	Table 11.5-1, Category I (p116)
Redundancy $r =$	1	12.3.4.1 (Design Category B or C) (p1
Ordinary Steel Concentrically Braced Frame		
Overstrength factor $W_o =$	6.5	Table 12.2-1 Ordinary Wood Framing
Response Modification Coeff $R =$	3	Table 12.2-1 Ordinary Wood Framing
Base Shear for Building $V = C_s W$		Eq. 12.8-1 (p129)
$C_s =$	$\text{SDS} = \frac{R}{I}$	0.042667 Use Eq. 12.8-2
Max. $C_s =$	$\text{SD1} = \frac{(R/I) T}{T_a}$	0.336359 Eq. 12.8-3
	$T_a =$	CT $h_{nx} = 0.1$ Eq. 12.8-7
		CT = 0 Table 12.8-2 -All other systems
		$h_n = 8$ Taken at Median height
		$x = 0.8$ Table 12.8-2-All other systems
	$T \leq$	$C_u T_a = 0.2$ OK
		$C_u = 1.6$ Table 12.8-1 (p129) for $\text{SD1} < .1$
		(See Eq. 12.8-4 for period $T > T_L$ of 12 sec for Max C_s)
Min. $C_s =$	$.5 S_1 = \frac{R}{I}$	0.01 $\leq .0$ Eq. 12.8-5 and 12.8-6
Dead Load =	67.5x57.5x.035	135.84
Dead Load =		

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Total Dead Load W for Bldg=	135.84375 k	135.84
Total Base Shear V = Cs W =	5.80 k	To building Frame

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FIGURE 1613.2.1(1)—continued
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE
CONTIGUOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)



FIGURE 1613.2.1(2)—continued
 RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_E) GROUND MOTION RESPONSE ACCELERATIONS FOR THE
 CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)

End of Review

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