LORAC DESIGN GROUP, LLC
Structural Engineers

Design Calculations

For

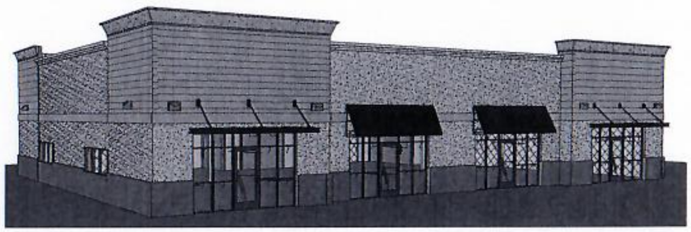
Summit Orchards
Lee's Summit, Missouri

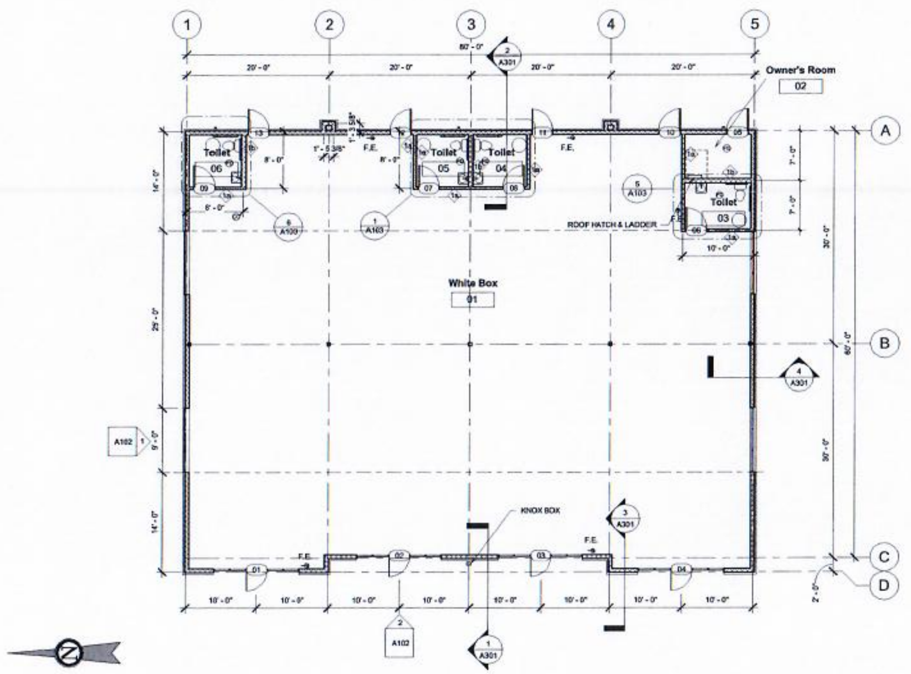
Architect,

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

Date: February 25, 2023

Joseph A. Towns, RA, PE, SE, AIA, NCARB, LEED AP, BD+C
Missouri Professional Engineer (Structural) #E-22017
Certification Applies to Pages Cover, through 51





1 Floor Plan
1/8" = 1'-0"

BUILDING CODE SUMMARY

APPLICABLE CODES
2018 INTERNATIONAL BUILDING CODE
2018 INTERNATIONAL MECHANICAL CODE
2018 INTERNATIONAL PLUMBING CODE
2018 INTERNATIONAL FIRE CODE
2018 INTERNATIONAL ENERGY CONSERVATION CODE
2017 NATIONAL ELECTRICAL CODE
2017 ICC/ANSI A117.1

SUMMARY OF WORK
NEW SHELL SPEC BUILDING. NO C.O. IS REQUESTED WITH THIS SUBMITTAL. SEPARATE TENANT FINISH PLANS WILL BE SUBMITTED FOR EACH SPACE AT A LATER DATE.

FIRE SUPPRESSION SUMMARY
NONE

OCCUPANCY CLASSIFICATION
M (RETAIL) & B (OFFICE)

TYPE OF CONSTRUCTION
V-B, NON - SPRINKLED

FLOOR AREA
BUILDING AREA: 4,880 SQ.FT.

OCCUPANT LOAD
TO BE DETERMINED

EXITS REQUIRED
TO BE DETERMINED

EXITS PROVIDED
TWO PER SPACE

TOILET FACILITIES REQUIRED
TO BE DETERMINED

TOILET FACILITIES PROVIDED
ONE UNISEX TOILET PER SPACE (ADA

DEFERRED SUBMITTALS TO BE COMPLETED BY OTHERS
ROOF TRUSS PLANS (PLANS BY SUBCONTRACTOR)

INTERIOR FINISH REQUIREMENTS
FLOOR FINISHES: CLASS I or CLASS II
WALL FINISHES: CLASS A (non-sprinkled)
CEILING FINISHES: CLASS A (non-sprinkled)

EGRESS DOORS SHALL BE READILY OPENABLE FROM THE EGRESS SIDE WITHOUT THE USE OF A KEY OR SPECIAL KNOWLEDGE OR EFFORT.

PREMISES SHALL BE IDENTIFIED ON ALL EXTERIOR DOORS, WITH NUMBERS AND/OR LETTERS. EACH CHARACTER SHALL BE NOT LESS THAN 6" HIGH WITH A MINIMUM STROKE WIDTH OF 1.0" INCHES. THEY SHOULD BE INSTALLED ON A CONTRASTING BACKGROUND. STREET FACING DOORS SHALL HAVE ADDRESSES THAT ARE PLAINLY LEGIBLE AND VISIBLE FROM THE STREET FRONTING THE PROPERTY. ADDRESS NUMBERS AND/OR LETTERS SHALL BE ARABIC NUMBERS OR ALPHABETIC LETTERS.

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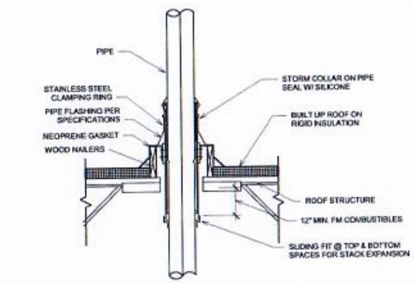
NEW BUILDING FOR
SUMMIT ORCHARDS
LEES SUMMIT, MO

No.	Description	Date
	Revision Schedule	

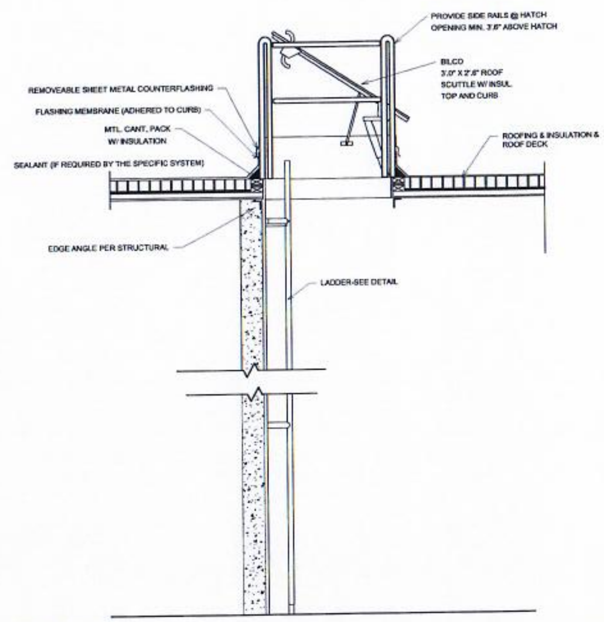
Floor Plan

Project number	2491
Date	03.01.2023
A101	
Scale	1/8" = 1'-0"

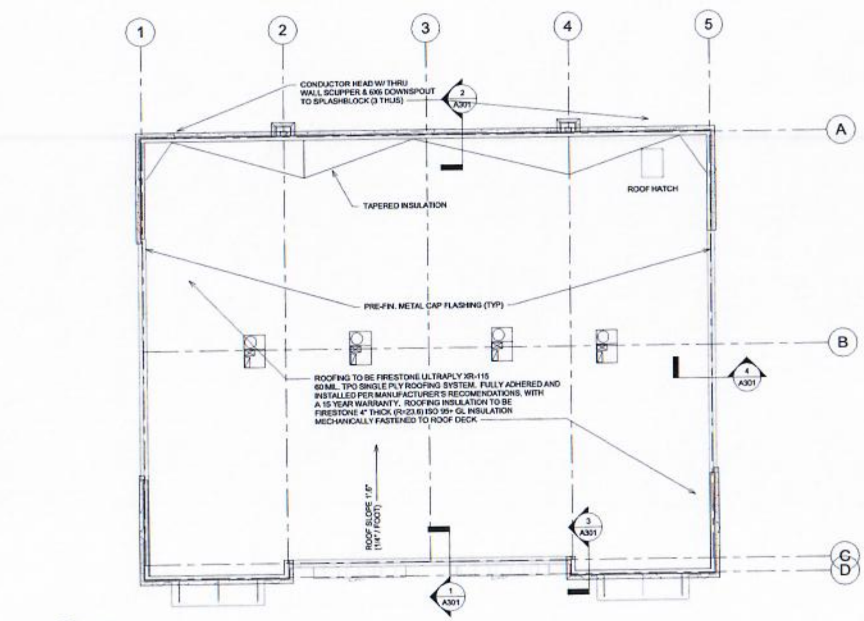
2



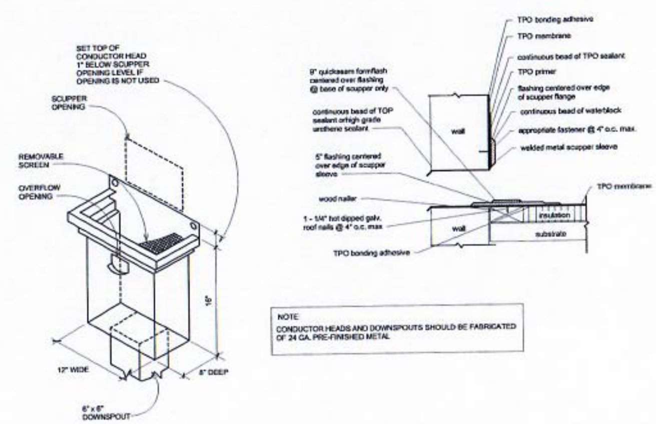
4 Pipe penetration
1" = 1'-0"



2 Roof hatch w/ guard rail
1" = 1'-0"



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



NOTE:
CONDUCTOR HEADS AND DOWNSPOUTS SHOULD BE FABRICATED OF 24 GA. PRE-FINISHED METAL

SCUPPER DETAIL AT PARAPET WALL
NOT TO SCALE

3 Scupper detail
1/2" = 1'-0"

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NEW BUILDING FOR
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LEES SUMMIT, MO

No.	Description	Date
	Revision Schedule	

Roof Plan

Project number	2491
Date	03.01.2023
A104	
Scale	As indicated

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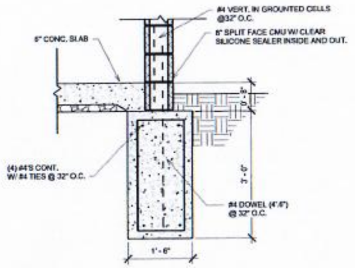
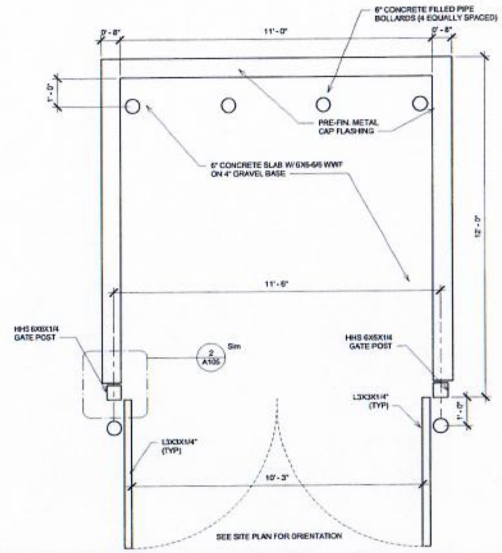
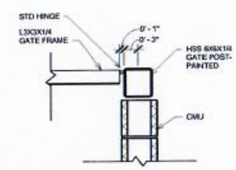
3



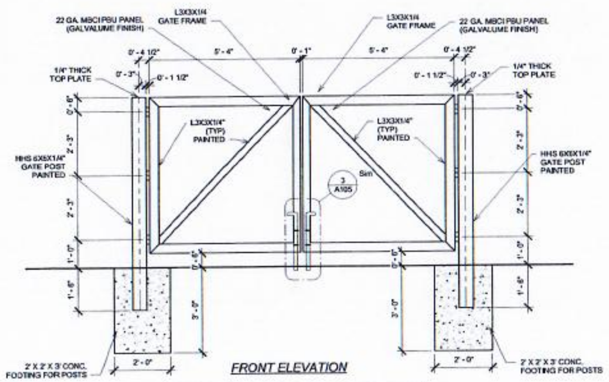
4 Trash Dumpster Detail Wall Section
3/4" = 1'-0"



PLAN VIEW



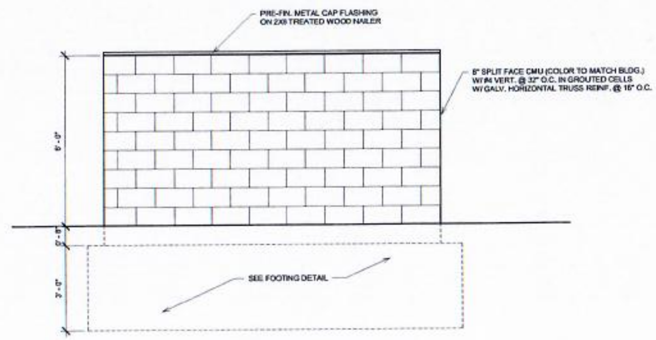
2 Hinge Detail Plan 12x11
1" = 1'-0"



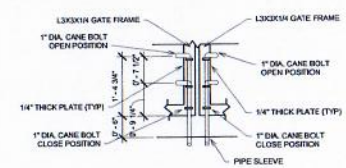
FRONT ELEVATION

ALL STEEL TO BE PAINTED
ALL CMU TO BE RECEIVE CLEAR SILICONE SEALANT @ EXTERIOR

1 Trash Dumpster Detail 11x12
1/2" = 1'-0"



SIDE ELEVATION



3 Trash Dumpster Detail - Hinge Detail
3/4" = 1'-0"

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NEW BUILDING FOR
SUMMIT ORCHARDS
LEES SUMMIT, MO

No.	Description	Date
	Revision Schedule	

Trash Dumpster Details- 11x12

Project number	2491
Date	03.01.2023
A105	
Scale	As indicated

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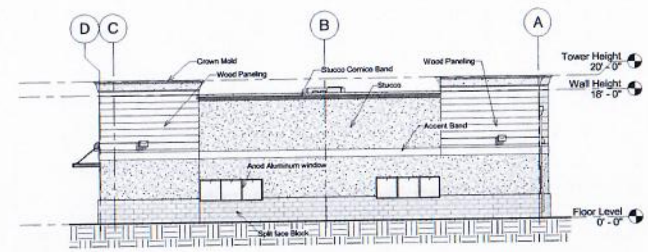
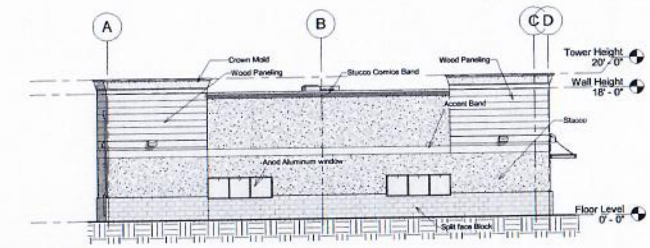
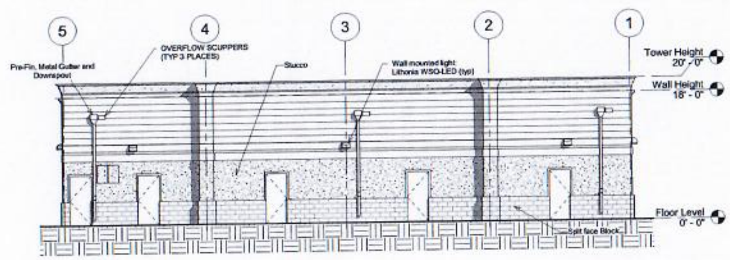


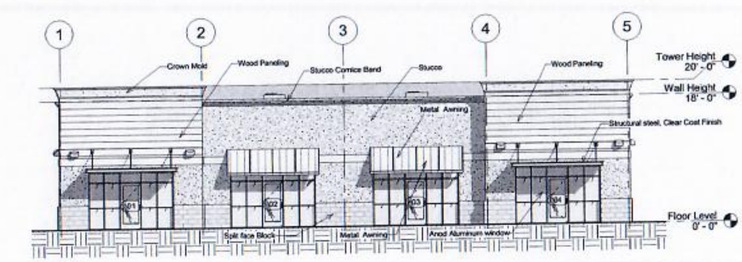
1 South Elevation
1/8" = 1'-0"



4 North Elevation
1/8" = 1'-0"



2 East Elevation
1/8" = 1'-0"



3 West Elevation
1/8" = 1'-0"

PROVIDE CLEAR SILICONE SEALER FOR BRICK

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NEW BUILDING FOR
SUMMIT ORCHARDS
LEES SUMMIT, MO

J.Jeffry Scharhag Mo. License A-4226
Herman Scharhag Co., Arch. Cert. of Authority A-22

No.	Description	Date
	Revision Schedule	

Elevations

Project number	2491
Date	03.01.2023

A201

Scale 1/8" = 1'-0"

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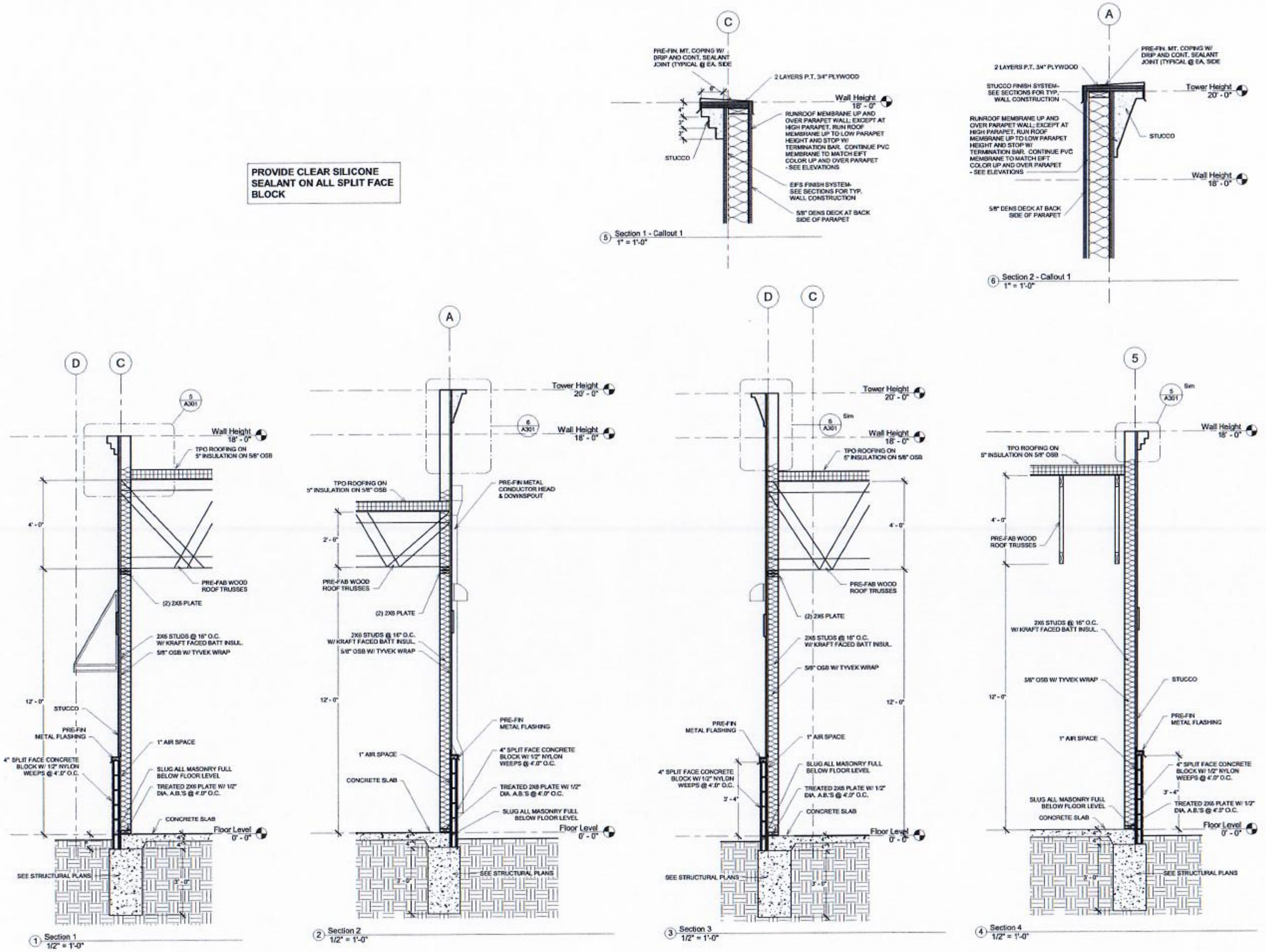

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NEW BUILDING FOR

SUMMIT ORCHARDS

LEES SUMMIT, MO

No.	Description	Date
	Revision Schedule	

Wall Sections

Project number	2491
Date	03.01.2023

A301

Scale: As indicated

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Loads and Codes

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems Office use Computer use	 50 100	 2,000 2,000
3. Armories and drill rooms	150[n]	—
4. Assembly areas Fixed seats (fastened to floor) Follow spot, projections and control rooms Lobbies Movable seats Stage floors Platforms (assembly) Other assembly areas	 60[m] 50 100[m] 100[m] 150[n] 100[m] 100[m]	—
5. Balconies and decks[h]	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 Other floors	 100 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 On single-family dwellings only	100 40	—
14. Garages (passenger vehicles only) Trucks and buses	40[e] See Section 1607.7	Note a
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals Corridors above first floor Operating rooms, laboratories Patient rooms	 80 60 40	 1,000 1,000 1,000
18. Hotels (see residential)	—	—
19. Libraries Corridors above first floor Reading rooms Stack rooms	 80 60 150[b, n]	 1,000 1,000 1,000
20. Manufacturing Heavy Light	 250[n] 125[n]	 3,000 2,000
21. Marquees, except one- and two-family dwellings	75	—
22. Office buildings Corridors above first floor File and computer rooms shall be designed for heavier loads based on anticipated occupancy Lobbies and first-floor corridors Offices	 80 — 100 50	 2,000 — 2,000 2,000

(continued)

TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
23. Penal institutions Cell blocks Corridors	 40 100	—
24. Recreational uses: Bowling alleys, poolrooms and similar uses Dance halls and ballrooms Gymnasiums Ice skating rink Reviewing stands, grandstands and bleachers Roller skating rink Stadiums and arenas with fixed seats (fastened to floor)	 75[m] 100[m] 100[m] 250[n] 100[c, m] 100[m] 60[c, m]	—
25. Residential One- and two-family dwellings Uninhabitable attics without storagei Uninhabitable attics with storage[i, j, k] Habitable attics and sleeping areas[k] Canopies, including marquees All other areas Hotels and multifamily dwellings Private rooms and corridors serving them Public roomsm and corridors serving them	 10 20 30 20 40 40 100	—
26. Roofs All roof surfaces subject to maintenance workers Awnings and canopies: Fabric construction supported by a skeleton structure All other construction, except one- and two-family dwellings Ordinary flat, pitched, and curved roofs (that are not occupiable) 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 All other primary roof members Occupiable roofs: Roof gardens Assembly areas All other similar areas	 5[m] 20 20 100 100[m] Note 1	 300 2,000 300 Note 1
27. Schools Classrooms Corridors above first floor First-floor corridors	 40 80 100	 1,000 1,000 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 All other	 40 100	 300[f] 300[f]

(continued)

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TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage) Heavy Light	 250[n] 125[n]	—
32. Stores Retail First floor Upper floors Wholesale, all floors	 100 75 125[o]	 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 mm^2,
1 square foot = 0.0929 m^2, 1 pound per square foot = 0.0479 kN/m^2,
1 pound = 0.004448 kN, 1 pound per cubic foot = 16 kg/m^3.

a. 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 $4^1/_2$ inches by $4^1/_2$ inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:
1. The nominal book stack unit height shall not exceed 90 inches.
2. The nominal shelf depth shall not exceed 12 inches for each face.
3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. 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.

g. 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).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. 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.

j. 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:

i. 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.

ii. 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_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

k. 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.

l. 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.

m. Live load reduction is not permitted.

n. Live load reduction is only permitted in accordance with Section 1607.11.1.2 or Item 1 of Section 1607.11.2.

o. 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:

1. A uniform live load, L, as specified in Items 1.1 and 1.2. This load shall not be reduced.
   1.1. 40 psf (1.92 kN/m^2) where the design basis helicopter has a maximum take-off weight of 3,000 pounds (13.35 kN) or less.
   1.2. 60 psf (2.87 kN/m^2) where the design basis helicopter has a maximum take-off weight greater than 3,000 pounds (13.35 kN).
2. 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.
3. 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.

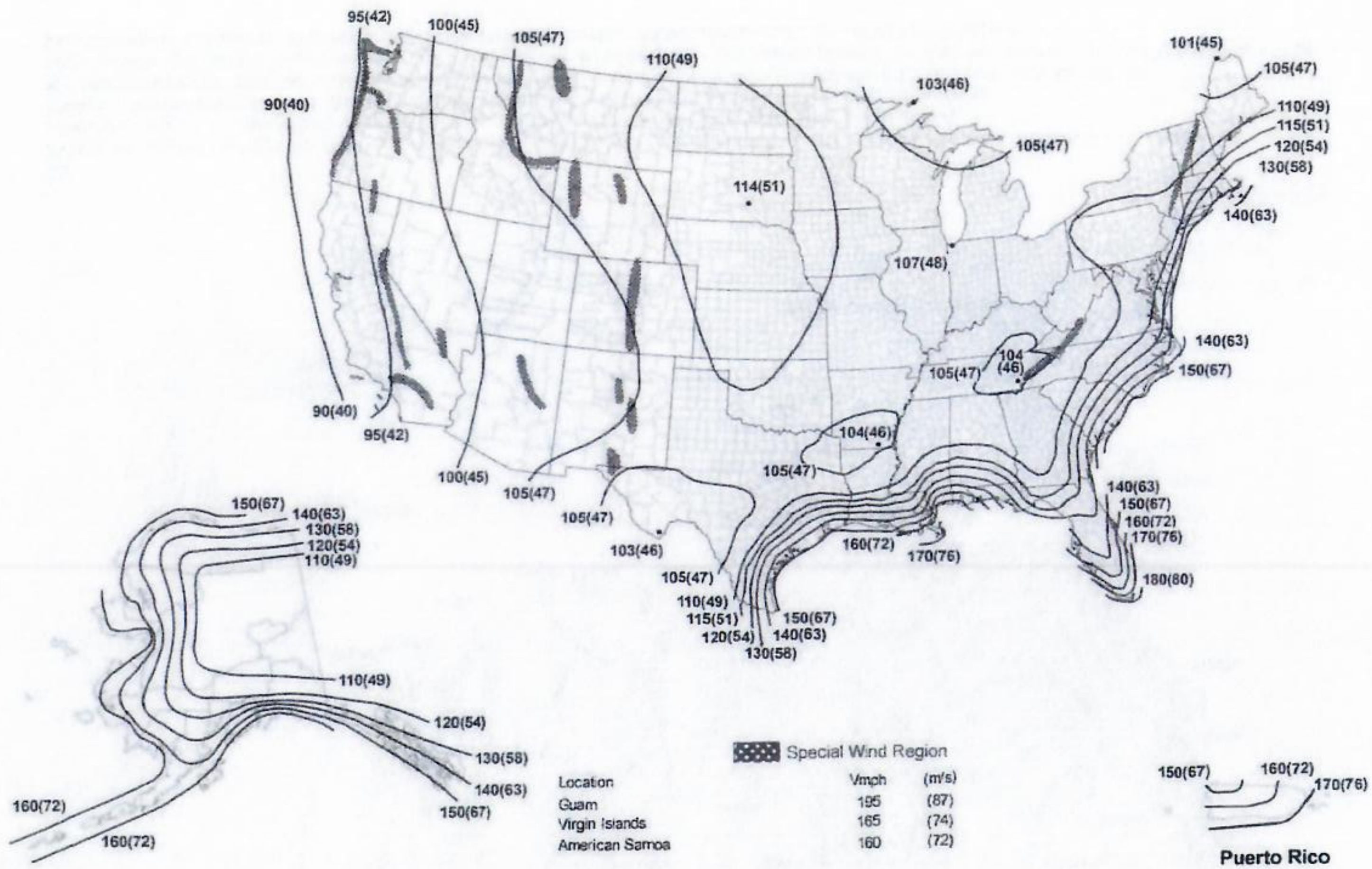
Landing areas designed for a design basis helicopter with maximum take-off weight of 3,000-pounds (13.35 kN) shall be identified with a 3,000 pound (13.34 kN) weight limitation. The landing area weight limitation shall be indicated by the numeral "3" (kips) located in the bottom right corner of the landing 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

9

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 permitted to 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

GENERAL NOTES

A. GENERAL

1. These notes shall be read in conjunction with the Specifications and the Drawings. In the event of a conflict, notify the Architect for clarification.
2. Before executing anything herein shown, examine actual job conditions. Report any discrepancy, dimensional or otherwise, between architectural and structural Drawings and any other error, omission, or difficulty affecting the work to the Architect and to the Structural Engineer for review.
3. The Owner or his Representative reserves the right to inspect any material, fabrication, or workmanship at any time in field or shop for conformance to the Specifications and Drawings.
4. All details and sections are intended to be typical and shall be construed to apply to any similar situation elsewhere, except where a different detail is shown.

B. DESIGN

1. Codes, specifications and standards (latest editions, U.N.O.)
   a. All design and construction shall conform to the International Building Code (currently adopted edition) as amended and adopted by the City of jurisdiction.
   b. All construction shall comply with the provisions of the following codes,specifications and standards, except where noted to the contrary on drawings and specifications or where more stringent requirements are specified or shown:
   - ACI 117 "Standard Specifications for Tolerance for Concrete Construction and Materials"
   - ACI 301 "Specifications for Structural Concrete for Buildings"
   - ACI 318 "Building Code Requirements for Reinforced Concrete"
   - ACI 530 "Building Code Requirements for Masonry Structures"
   - AISC "Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings"
   - SDI "Steel Deck Manual for Floor Decks and Roof Decks"
   - AWS D1.1 "Structural Welding Code - Steel"
2. Design Loads:
   a. Roof - Snow (incl. rain on snow)
      - Pf = 20 psf
      - Ce = 1.00
      - I = 1.00
      - Ct = 1.00
   b. Wind
      - Basic Wind Speed = 115 mph
      - I = 1.00
      - Wind Exposure B
      - Internal Pressure Coefficient = 0.3
   d. Floor Live Load - Office 50 psf
      - Entrances (exits), stairs 100 psf
      - Light Storage 125 psf
      - Heavy storage 250 psf
   e. Canopy Roof Design Dead Loads:
      - Roof Panels 30 psf
      - Steel Framing 5 psf
      - Roofing 5 psf
      - Total 40 psf
3. Foundations are designed for the following net allowable bearing capacities:
   a. Isolated Footings: 2 ksf
   b. Continuous Footings: 2 ksf
4. Foundations and retaining walls have been designed for an equivalent fluid pressure of 100 pcf.

C. CONCRETE

1. Concrete used in the Work shall have the following minimum 28-day ultimate compressive strengths:
   a. Columns 4000 psi
   b. Retaining walls, slabs on grade, and footings 4000 psi
   c. Framed slabs 4000 psi
2. Air entrain all exterior concrete (admixture: ASTM C 260).
3. Do not use calcium chloride admixtures under any circumstances.
4. Reinforcing bars: ASTM A 615 Specifications, Grade 60, deformed. Bend bars cold.
5. Welded wire fabric (WWF): ASTM A 185.
6. Maintain minimum concrete coverage for reinforcing as indicated, unless noted otherwise. Reference details 17/S1.0 and 18/S1.0 for placement of reinforcement in typical framed slabs.
   a. 3 in. clear where concrete is deposited directly against earth.
   b. 2 in. clear where concrete is exposed to earth or weather but poured against forms for bars larger than #5.
   c. 1-1/2 in. clear where concrete is exposed to earth or weather, but poured against forms for bars #5 or smaller.
   d. 3/4 in. clear for slabs and walls formed above grade not exposed to weather.
   e. 1-1/2 in. clear for beams and columns formed above grade and not exposed to weather.
7. Lap all bars at splices in accordance with ACI 318, unless specifically noted otherwise.
8. Top and bottom bars in continuous grade beams shall run continuous through multiple spans, where possible. Otherwise, top bars shall splice within the middle 1/3 span and bottom bars shall splice over supports.
9. Pour columns, walls, and pilasters to be monolithic.
10. All concrete walls shall be properly braced and held in line until supporting slabs or floors are in place.
11. All bar steel and WWF shall be properly supported and held accurately in place as recommended by the Concrete Reinforcing Steel Institute, except that maximum spacing of any bar or mesh support shall be 3 feet.
    a. Support top slab bars with continuous high chairs.
    b. Support beam bars on heavy beam bolsters.
    c. Support footing and grade beam bottom reinforcing on concrete bricks, concrete blocks, or mounds of poured concrete.
    d. Support WWF in slab-on-grade properly at the mid-depth of the slab.Hooking and pulling up mesh after concrete has started to take its initial set is prohibited.
    e. Supports for reinforcement for exposed-to-view concrete surfaces shall have legs that are in contact with forms plastic protected (CRSI, Class 1) or stainless steel (CRSI, Class 2).
12. Where slabs-on-grade make an abrupt change in direction, such as at doors and corners or ends of walls, provide 2-#4 by 4 feet across the reentrant corner.
13. Provide the following minimum concrete cover for fire rating:

Interior load bearing walls and columns	2 hrs	1 1/2" cover
Concrete beams	2 hrs	1/2" cover
Concrete joists	2 hrs	1 1/2" cover
Floor slab	2 hrs	3/4" cover

D. MASONRY

1. Concrete masonry units (CMU): ASTM C 90, lightweight units (105 pcf or less), with the minimum net area compressive strength of 2200 psi.
2. Mortar: Portland cement and lime, and proportioned in accordance with ASTM C 270 for the following types:
   Type N - for all walls above grade
   Type S - for all walls below grade, in contact with earth
3. f'm = 1500 psi.
4. Provide mortar bed on webs between grouted cells and hollow cells.
5. Grout: ASTM C 476, 2000 psi minimum 28-day compressive strength.
6. Grout all vertical cells and spaces containing reinforcing bars (as detailed) bond beams, and lintels.
7. Vertically reinforce walls as shown on drawings. However, if not indicated on the drawing, reinforce wall as indicated below, at each corner, at ends of 48 inches horizontally throughout the wall, of walls, each side of control joists and openings, and at a maximum spacing unless noted otherwise.
   8" or 6" wall #6
   12" or 10" wall (2) #6
8. Horizontally provide continuous bond beam with 2 #5 minimum for 12" or 10" CMU; 1 #5 minimum for 8" or 6" CMU at floor/roof, near midheight (10'-0 maximum spacing) and top of wall, unless noted otherwise. Provide #5 corner bar for each horizontal bond beam corners.
9. Place reinforcement prior to grouting. Hold vertical reinforcement in position with rebar positioner.
10. Provide horizontal joint reinforcement as indicated on the drawings and specifications, at a minimum provide at 16"o.c.
11. Lap joint reinforcement a minimum of 12 in.
12. In no case shall shores and forms at lintels be removed until it is certain that the masonry has hardened sufficiently to carry its own weight and all other reasonable temporary loads that may be placed on it during construction.
13. Do not wet concrete masonry units.
14. Do not use calcium chloride.
15. Do not use masonry cement.
16. Keep masonry walls shored during construction until the roof deck and floor slabs are in place to provide lateral stability.

E. STEEL

1. Qualifications for Welding Work:
   a. Perform all welding by a certified welder.
   b. Qualify welding processes and welding operators in accordance with AWS "Standard Qualification Procedure".
   c. Provide certifications that welders to be employed in work have satisfactorily passed AWS qualification tests within previous 12 months.
   d. If recertification of welders is required, retesting will be Contractor's responsibility.
2. Erector must examine areas and conditions under which structural steel work is to be installed, and notify Contractor in writing of conditions detrimental to proper and timely completion of Work.Do not proceed with work until unsatisfactory conditions have been corrected in a manner acceptable to the Erector.
3. Submit shop drawings prepared under supervision of a registered professional engineer,including complete details and schedules for fabrication and assembly of structural steel members procedures and diagrams.Include details of cuts, connections, camber, holes, and other pertinent data. Indicate welds by standard AWS symbols, and show size,length, and type of each weld. Show size and type of bolt for all bolted connections.
4. Provide setting drawings, templates, and directions for installation of anchor bolts and other anchorages to be installed by others.
5. Paragraph 4.2.1 of the (AISC) "Code of Standard Practice for Steel Buildings and Bridges" is hereby modified by deletion of the following sentence: "This approval constitutes the owner's acceptance of all responsibility for the design adequacy of any detail configuration of connections developed by the fabricator as a part of his preparation of these shop drawings."
6. If required cut edges of backing strips, extension bars, or run-off plates flush with edge of abutting parts.
7. Where framing members and/or connections for steel stairs are not indicated on either structural or architectural drawings. Design the members and/or connections and submit calculations or supporting data to verify their adequacy.A live load of 125 psf shall be used in the design. Fully detail stair connections, including attachments to supporting members.
8. Structural steel: ASTM A 572 - wide flange sections, ASTM A 36 - angles,channels, and plates, ASTM A 501 - pipes, and ASTM A 500, Grade B - tubes.
9. High Strength Bolts (steel-to-steel connections): ASTM A 325N, with twist-off load indicator type heads.
10. Anchor bolts: ASTM A 307, sizes indicated are based on preliminary reactions and spacing.
11. Welded connections: AWS Standards and Specifications using E70xx electrodes, unless noted otherwise.
12. Expansion Bolts: Stud type expansion anchors..(Hilti Kwik Bolt II).
13. Injection Adhesive: Hilti Dowelling Anchor (HY-150); Rawl/Sika Foil-Fast,Ramset/Redhead Epcon Ceramic 6.
14. Drill holes for anchors using a bit incapable of cutting steel. Do not cut existing concrete reinforcing steel. If, while drilling, reinforcing steel is encountered, notify the Structural Engineer for approval of new location. Cleaned and patch the abandoned hole grout.
15. Ends of beams which have copes to the extent that allowable shear or bending stress of steel is exceeded shall have web plates of sufficient size welded to the beam to reduce such stresses.
16. Provide holes required for securing other work to structural steel framing, and for passage of other work through steel framing members, as shown on final shop drawings.
17. Do not flame cut holes or enlarge holes by burning.
18. Set structural frames accurately to lines and elevations indicated.Align and adjust various members forming apart of a complete frame or structure before permanently fastening.Perform necessary adjustments to compensate for discrepancies in elevations and alignment.
19. Provide temporary shoring and bracing members with connections of sufficient strength to bear imposed loads.Remove temporary members and connections when permanent members are in place and final connections are made. Provide temporary guy line to achieve proper alignment of structure as erection proceeds.
20. Clean bearing surfaces of bond-reducing materials and roughen to improve bond to surfaces. Clean bottom surface of base plates.
21. Grout plates are prohibited. Tighten anchor bolts after supported members have been positioned and plumbed. Do not remove wedges or shims, but if protruding, cut off flush with edge of base plate prior to packing with grout.
22. Nonshrink grout: CRD-621 Type A, premixed, nonmetallic, noncorrosive, nonstaining.
23. Provide open-web joists (K-series), longspan joists (LH-series), and joist girders as indicated on the Drawings and in accordance with specifications of SJI.
    a. Weld K-series joists to supporting steel with 1/8 in. fillet welds in. long, each side, u.n.o.
    b. Weld LH-series joists to supporting steel with 1/4 in. fillet welds 2 in. long, each side, u.n.o.
    c. Bolt joists at or nearest a column to supporting steel in conformance with O.S.H.A.with erection bolts.
    d. Provide continuous horizontal bridging for joists (u.n.o.) and bottom chord braces for joist girders as required by SJI,except where the net uplift loading requires additional bridging.
    e. Provide horizontal bridging to resist 10psf uplift for main roof at service building and main building penthouse.
    f. Extend bottom cord to brace beam bottom flange at mid-span of beams in service building.
24. Form deck: 9/16 in.galvanized deck with the following minimum properties:
    Minimum thickness 0.0295
    Moment of Inertia 0.024 in ^4
    Section Modulus 0.070 in ^3
25. Composite floor deck: 1-1/2 in. galvanized deck with the following minimum properties:
    Minimum thickness 0.0358
    Moment of Inertia 0.195 in ^4
    Section Modulus 0.240 in ^3
26. Roof deck: 1-1/2" painted wide rib deck with the following minimum properties:
    Minimum thickness 0.358
    Moment of Inertia 0.212 in ^4
    Section Modulus 0.234 in ^3
27. Roof deck shall be welded to supports to resist a net uplift of 20 PSF.
28. Provide 2-1/2" x 2-1/2" x 1/4" angles as required to support deck at columns, ends of beams, around openings, etc. Except at noted otherwise.
29. Provide 1,500 # misc. steel for use by Engineer, as needed.

E. EPOXY AND MECHANICAL ANCHORS

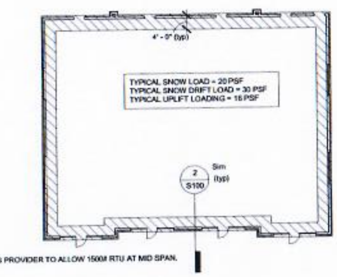
1. For concrete, grouted CMU, and solid masonry use Hilti HIT HY 150 two-part hybrid adhesive. For hollow CMU and masonry use Hilti HIT HY20 two-part hybrid adhesive with screen tubes. Equivalent adhesives may be used with prior written approval by the Structural Engineer.
2. Thoroughly clean holes with nylon brush and pressurized air per manufacturers instructions.
3. Drill holes to the embedment depths indicated on the drawings. If no depths are indicated, use 9 bolt or bar diameters with HY150 and 12 bolt diameters for HY 20.
4. "Wedge" or "Expansion" anchors shall be Hilti Kwik bolt II expansion anchors. Embed anchor 7 bolt diameters unless noted otherwise. Equivalent anchors may be substituted with prior written approval of the Structural Engineer.

F. METAL STUDS

1. Install cold-formed metal studs per drawings and manufacturer's recommendations. See Structural Plan for sizes and gauges.

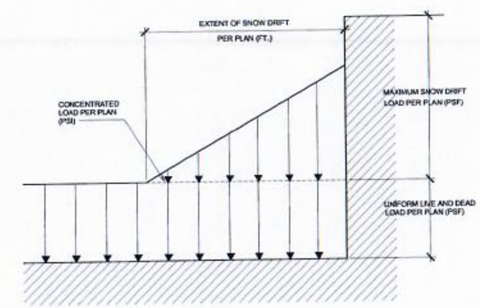
G. CONSTRUCTION

1. See architectural and mechanical requirements for embedded items not shown herein and to verify size and location of all openings.
2. Coordinate the sizes and locations of all miscellaneous metal items required for mechanical and electrical.
3. Requirements for embedded items, sleeves, block outs, duct openings,etc., in the concrete frame shall be submitted (plans and details) to the structural engineer for approval at least two weeks prior to the proposed date of casting concrete. No such items, other than those shown, shall be provided in the structure without the approval of the structural engineer.
4. Provide adequate shoring or bracing during construction to resist forces such as wind and unbalanced loading due to construction.
5. Field verify the location and depth (or height) of all utilities prior to beginning construction in order to provide adequate clearances and to insure noninterruption of service.



NOTE: TRUSS PROVIDER TO ALLOW 1500# RTU AT MID SPAN.

3 Snow Load Plan
1/16" = 1'-0"



2 Snow load
1/2" = 1'-0"

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NEW BUILDING FOR
SUMMIT ORCHARDS
LEES SUMMIT, MO

No.	Description	Date
	Revision Schedule	

Structural Notes

Project number	2491
Date	03.01.2023
	S100
Scale	As indicated

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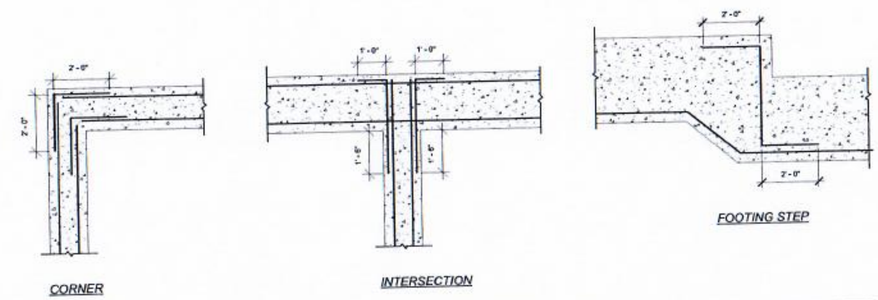
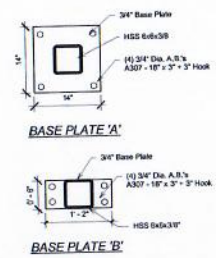
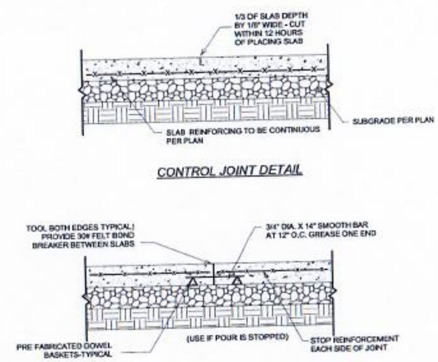


(5) Typical Footing Detail
1/2" = 1'-0"

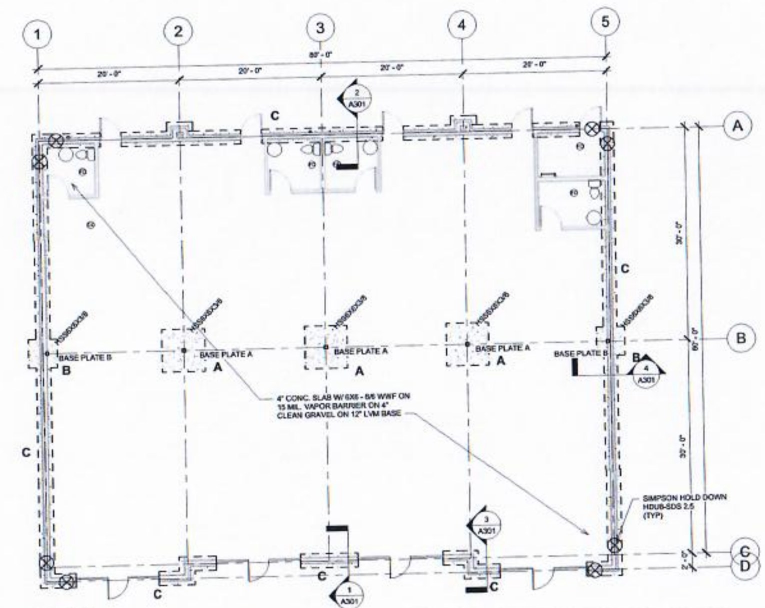


(2) Base Plate Details
1" = 1'-0"

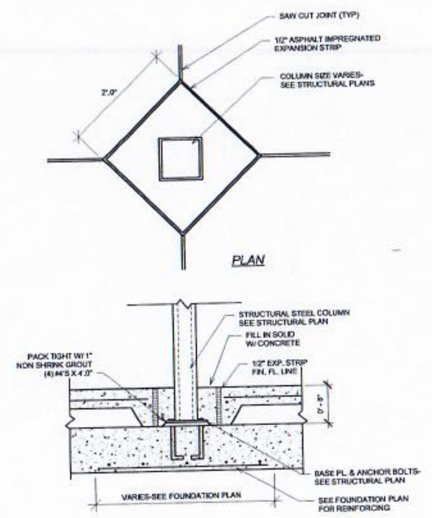


(3) Sawcut Floor Slab
3/4" = 1'-0"

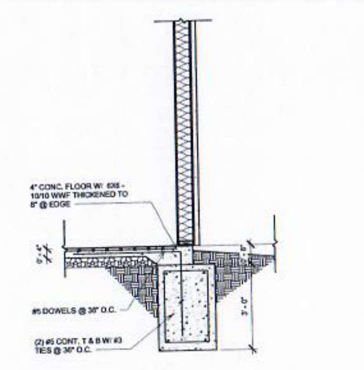
Structural Foundation Schedule		
Type Mark	Type	Type Comments
A	6'x 6' x 2'.0" deep	(7) #6's each way
B	4' x 4' x 3'.0" deep	(5) #5's each way at T &B w/ (1) #6 vert. @ each corner
C	Bearing Footing	2'.0" w x 3'.0" deep w2/ (4) #5's cont. & #3 ties # 36" O.C. w/ #5 dowel 3' x 3' into slab @ 36" O.C.



(1) Foundation Plan
1/8" = 1'-0"



(4) Column Detail
1" = 1'-0"



(6) Footing type 'C'
1/2" = 1'-0"

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NEW BUILDING FOR
SUMMIT ORCHARDS
LEES SUMMIT, MO

No.	Description	Date
	Revision Schedule	

Foundation

Project number	2491
Date	03.01.2023
S101	
Scale	As indicated

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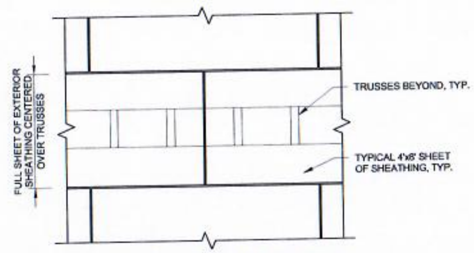
LINTEL SCHEDULE

LINTEL TYPE 'A'	TWO 2X12'S
LINTEL TYPE 'B'	THREE 2X12'S

PROVIDE (4) 2X8'S BEARING AT EACH END OF LINTELS

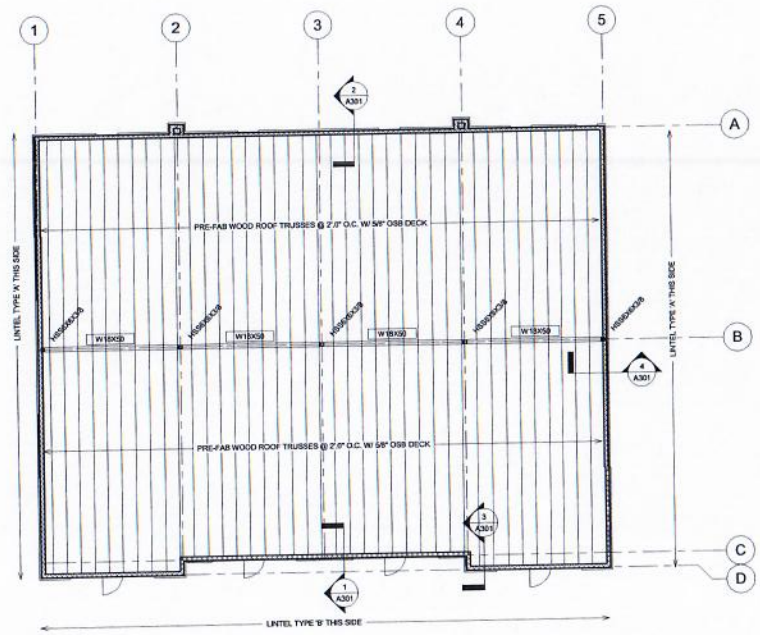
STRUCTURAL NOTES

1. TRUSS MANUFACTURER TO FURNISH ALL HOLD DOWNS AND CLIPS FOR WOOD TRUSSES
2. PROVIDE HEAVY DUTY CLIPS AT ALL PANEL EDGES PERPENDICULAR TO TRUSSES AT 2'-0" O.C. STAGGER END OF PANELS AND GAP ALL PANELS 1/16" AT ALL EDGES
3. PROVIDE SIMPSON H1 HOLD DOWN CLIPS FOR EACH TRUSS
4. ROOF SHEATHING TO BE 5/8" EXTERIOR APA PLYWOOD, NAILED WITH 10d NAILS AT 6" O.C. ALL AROUND PLYWOOD EDGES (BLOCKING AS REQUIRED) AND ALL AROUND ROOF PERIMETER WITH 10d NAILS AT 6" O.C. AT ALL INTERMEDIATE SUPPORTS
5. BRACING DESIGN BY TRUSS MANUFACTURER
6. TRUSS MANUFACTURER TO PROVIDE DESIGN DRAWINGS AND CALCULATIONS AND LAYOUT PLAN, SEALED BY REGISTERED ENGINEER, FOR REVIEW, AND FOR APPROVAL BY THE CITY
7. WALL SHEATHING TO BE 5/8" OSB, NAILED WITH 8d NAILS AT 6" O.C. AT ALL STUDS AND FULL PERIMETER OF EACH PLYWOOD SHEET (BLOCKING AS REQUIRED) AND ALL AROUND PERIMETER OF WALL AND AROUND ALL OPENINGS
8. SEE ARCHITECTURAL SHEETS FOR ALL OPENING HEIGHTS AND WIDTHS
9. ALL DOOR AND WINDOW OPENINGS TO BE SUPPORTED BY MIN. (3) 2X12 LINTEL ABOVE OPENING, BEARING ON (2) STUDS AT EACH SIDE
10. ALL STEEL LINTEL BEAMS TO BE SUPPORTED BY (4) 2X STUDS AT EACH END

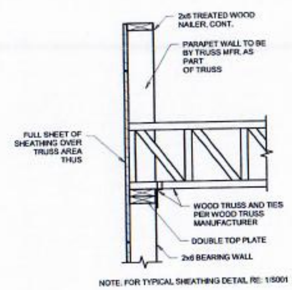


NOTE: ALL EXTERIOR SHEATHING TO HAVE NAILS @ 4" O.C. AROUND EDGE AND 6" O.C. INFIELD.

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

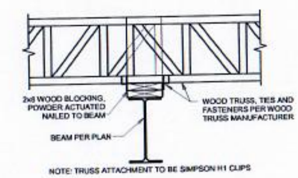


1 Roof Framing Plan
1/8" = 1'-0"



NOTE: FOR TYPICAL SHEATHING DETAIL RE: 1/S001

3 typical section at truss bearing wood
1" = 1'-0"



NOTE: TRUSS ATTACHMENT TO BE SIMPSON H1 CLIPS

2 typical section at truss bearing 2 wood
1" = 1'-0"

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NEW BUILDING FOR
SUMMIT ORCHARDS
LEES SUMMIT, MO

No.	Description	Date
	Revision Schedule	

Framing

Project number	2491
Date	03.01.2023
	S102
Scale	As indicated

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Design Calculations

Summit Orchards

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(Lr or S or R)	96.1	Eq 16-2
3	1.2(D +F) + 1.6(Lr or S or R) + 1.6H + (f1L or .5W)	79.6	Eq 16-3
4	1.2(D +F) + 1.0 W + f1L + 1.6H + .5(Lr or S or R)	72.1	Eq 16-4
5	1.2(D +F) + 1.0 E + f1L + 1.6H + f2S	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

Design Calculations

Summit Orchards

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: 80'x60'x19' Monoslope

Length:	80	feet	Re: Plan Drawings OUT TO OUT
Width:	60	feet	Re: Plan Drawings OUT TO OUT
Eave Height:	19	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 - 80'x60'x19'

Horizontal Wind Loads

Building Width	60 feet	Re: Plan Drawings
Building Eave Height	19 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	22.8 kips	By Calculation
Total Wind Load at Roof	-1.8 kips	By Calculation
Total Wind Shear (Longitudinal)	**21 kips**	**By Calculation**
Unit Shear Stress at Base	0.13 kips/ft	By Calculation
Theorhetical Chord Stress at T.O.W.		
Bending Moment:	157.5 k-ft	M=(wl^2)/8
Bending Stress:	2.0 k/in^2	Stress=M/d
Area for 2 Plate top chord:	10.5 in^2	By Calculation
Stress per Square Inch:	187.5 psi	By Calculation
OK		
Shear to Top of Wall:	10.5 kips	By Calculation
Theor. OT Reactions:	2.49 kips	By Calculation
Theor. Panel Shear Stress:	0.26 klf	By IBC
Lineal Feet Required:	9.78 feet	By Calculation
OK		

MWFRS - Transverse - 80'x60'x19'

Horizontal Wind Loads

Building Length	80 feet	Re: Plan Drawings
Building Eave Height	19 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	30.4 kips	By Calculation
Total Wind Load at Roof	-0.6 kips	By Calculation
Total Wind Shear (Transverse)	29.8 kips	By Calculation
Unit Stress at Base (Transverse)	0.248333 kips/ft	By Calculation
Theorhetical Chord Stress at T.O.W.		
Bending Moment:	298.0 k-ft	M=(wl^2)/8
Bending Stress:	5.0 k/in^2	Stress=M/d
Area for 2 Plate top chord:	10.5 in^2	By Calculation
Stress per Square Inch:	473.0 psi	By Calculation
	Check Design	
Shear to Top of Wall:	14.9 kips	By Calculation
Theor. OT Reactions:	3.5 kips	By Calculation
Theor. Panel Shear Stress:	0.3 klf	By IBC
Lineal Feet Required:	13.9 feet	By Calculation
	OK	

MWFRS - Longitudinal - **80'x60'x19'**

Vertical Wind Loads (Uplift)

Building Width	60 feet	Re: Plan Drawings
Building Eave Height	19 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)

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)	-0.3 kips/ft	By Calculation
Dead Load Forces		
Roof:	0.480 klf	By Calculation
Floors:	0.000 klf	By Calculation
Walls:	0.304 klf	By Calculation
Total:	0.784 klf	By Calculation
NET, Unit Stress at Foundation (Longitudinal)	0.463 kips/ft Dead Load Controls, Uplift < DL	By Calculation

MWFRS - Transverse - **80'x60'x19'**

Vertical Wind Loads (Uplift)

Building Length	80 feet	Re: Plan Drawings
Building Eave Height	19 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)	0.6 kips/ft	By Calculation
Dead Load Forces		
Roof:	0.48 klf	By Calculation
Floors:	0 klf	By Calculation
Walls:	0.304 klf	By Calculation
Total:	0.784 klf	By Calculation
NET, Unit Stress at Foundation (Transverse)	1.384 kips/ft Dead Load Controls, Uplift < DL	By Calculation

MWFRS - Longitudinal - 80'x60'x19'

OTM - Horizontal Wind Forces

Total Wind Shear (Longitudinal)	21.00 kips	By Calculation
Total Wind Force Vertical (Longitudinal)	-25.68 kips	By Calculation Controlling Value ONLY
Summing Moments at End	46.68 kips	By Calculation
Unit Stress at Extrema (longitudinal)	0.78 kips/ft Dead Load Controls	By Calculation

MWFRS - Transverse - **80'x60'x19'**

OTM - Horizontal Wind Forces

Total Wind Shear (Transverse)	29.80 kips	By Calculation
Total Wind Force Vertical (Transverse)	48.00 kips	By Calculation Controlling Value ONLY
Summing Moments at End	77.80 kips	By Calculation
Unit Stress at Extrema (Transverse)	0.97 kips/ft Hold Downs Required	By Calculation

Foundation Loads Due to Wind

Longitudinal Wind Loads

Shear Stress/foot	0.13 kips/ft	By Calculation
Vertical Force/foot	0.46 kips/ft	By Calculation
OT Force (Vertical)	0.78 kips/ft	By Calculation
Total Shear	21.00 kips	By Calculation
Shear @ Brace = Tshear/2=	**10.50** kips	By Calculation

Transverse Wind Loads

Shear Stress/foot	0.25	kips/ft	By Calculation
Vertical Force/foot	1.38	kips/ft	By Calculation
OT Force (Vertical)	0.97	kips/ft	By Calculation
Total Shear	29.80	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:	0.97	kips/ft	By Calculation
Hold Down Spacing:			

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	0.25	kips/ft	By Calculation

Design Edge Pattern **6 in. o.c Edge Spacing Required**

Shear Wall Quantity Confirmation

Transverse Loading

SW2	**1/2" Gyp blckd**	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		29.80	kips	By Calculation
% Shear Resisted		61%		
% Shear Remaining		39%	Additional Capacity Required	

SW1 15/32" Shtg blckd	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	29.80 kips	By Calculation
% Shear Resisted	185%	
% Shear Remaining	-85%	Shear Demand Met

Longitudinal Loading

SW2 1/2" Gyp blckd	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	21 kips	By Calculation
% Shear Resisted	96%	
% Shear Remaining	4%	Additional Capacity Required

SW1 15/32" Shtg blckd	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	21.00 kips	By Calculation
% Shear Resisted	291%	
% Shear Remaining	-191% Shear Demand Met	

Shear Wall Design, Wind

Longitudinal Loading

SW3 **1/2" Gyp blckd**	175 plf	Table 2306.3.1
Lineal feet available for SW1	115 lf	By Calculation
Shear Capacity	20.13 kips	By Calculation
Reduction Factor	1	
Factored Capacity	20.13 kips	By Calculation
Total Shear	21.00 kips	By Calculation
% Shear Resisted	96%	
% Shear Remaining	4%	

SW2 15/32" Shtg blckd	255 plf	Table 2306.3.1
Lineal feet available for SW1	115 lf	By Calculation
Shear Capacity	21.00 kips	By Calculation
Opening Reduction Factor	1	
Wind Increase Factor	1.40	IBC 2306.4.1
Factored Capacity	29.40 kips	By Calculation
Combined Capacity	49.53 kips	By Calculation
Total Shear	21.00 kips	By Calculation
% Shear Resisted	236%	
% Shear Remaining	-136% Capacity Demand Met	

Design Calculations

Summit Orchards

Lee's Summit, Missouri

Misc. Calcs - Continous 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
	fy =		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			

	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	in^2	
As, min =		1.71		ACI 10.5.1
Select =	1-#6's	0.44	in^2/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	

Design Calculations

Summit Orchards

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

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

Summit Orchards

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 =	100	kips	Conservative
			Ref:
rx = ry =	2.28	inches	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 =	12.00	feet	
	144.00		
rx = ry =	2.28	inches	
Fe =	71.75	ksi	E3-4
.44FY =	18.48	ksi	
Short Column, Eq. E3-2 Controls			
k = FY/Fe =	0.59		
Fcr =	32.87	ksi	E3-2
phi*Ag*Fc =	224.26	kips	
% Capacity =	45%		
	OK	Say OK... full load not likely	

Design Calculations

Summit Orchards

Lee's Summit, Missouri

Slab Calculation:

Slab thickness	4	inches
f'c	4000	ksi in 28 days
k	200	
desired safety factor	2.5	

Ec	3604996.5	ksi
I	64.0	inches^4
L	0.0	
Sec.Mod. per foot	32.0	Inches^3
allowable bending stress	227.7	Inches^2
Allowable load	1170.3	lbs per square foot

Design Calculations

Summit Orchards

Lee's Summit, Missouri

Wood Frame Building

Misc. Calcs - Spread Footing Calculations

Footing Carrying; **Corners**

*	Dead Load (conservative estimate)	4		By Design
*	Live Load (conservative estimate)	5		By Design
	Total Column Load	9	kips	Per Drawings
*	Bending Moment	1	kip-feet	By Design, Seismic
	Bending Moment	12	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)		3	feet	By Design
**	Footing Length (major axis)	L =	3	feet	By Design
*	Footing thickness		2.8	feet	By Design
	Direct Soil Pressure		473	psf	
	Axial Bearing Stress =		1420	psf	P/A
	P/3Af*(4L/L-2e) =		1998	psf	

Solve for "x" (length of bearing)=		4.27 feet	=((Sum of Loads))*1000)/(0.5*Pa*Fw
Check Overturning =	Mo =	9 inch-kips	(Mdes*.75)
	Mr =	208 inch-kips	=((Sum of Loads))*1000)/(0.5*Pa*Fm
	S.F. =	**23.06**	Mr/Mo

Check Punching Shear

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

Punching Shear Failure Perimeter:

* Flexural Shear Critical Section =		23.76 inches	Estimated @ (.7071*t)
Punching Shear Critical Section		11.88 inches	=Flex Cr/2
	bo=	143.03	

Ultimate Load to Footing:

	Pu =	14.10 kips	=1.4DL + 1.7LL

Ultimate Punching Shear:

Soil Load due to Column Load Only

	Pc =	1000.00 psf	=P/A ftg
	Vu=	26.56 kips	=P/Lin at edge

Calculate Depth Required:

Vu/phi*bo*Vc =	d=	0.86 inches

Check Flexural Shear

	Vc =	126 ksi

Soil Pressure at Critical Section:

	Psoil =	2456.44
Total Shear Force		6680.96 lbs
	Vu =	12107.24 lbs
	d =	3.13 inch

Flexural Shear Does NOT Control Design

Check Bending

M=	35.71 kip-feet
Mu=	60.71 kip-feet

Select Reinforcement

	Mu/ftg length =		20 kip-ft/ft
	Minimum As =		0.73 inches^2
**	Try	5 - #6's	2.20 inches^2/foot Each Way!!
	Mu Capacity =		282 kip-ft/ft

OK

Check if Tensile Steel is Required at Top of Footing:

*	Overburden =	2.50 feet
**	Soil weight =	0.11 pcf
	Mu =	0.59 kip-feet/foot
	fc = Mc/I=	3.14 psi
	fr =	205.55 psi

OK, No Top Steel Required

Design Calculations

Summit Orchards

Lee's Summit, Missouri

Wood Frame Building with Ctr Line Beam

Misc. Calcs - Spread Footing Calculations

Footing Carrying;

*	Dead Load (conservative estimate)		29.11	kips	By Design
*	Live Load (conservative estimate)		24.26	kips	By Design
	Total Column Load		53.37	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)	L =	7	feet	By Design
*	Footing thickness		2	feet	By Design
	Direct Soil Pressure			psf	
	Axial Bearing Stress =		1389	psf	P/A
	P/3Af*(4L/L-2e) =		1950	psf	
	Footing Weight =		15	kips	

Solve for "x" (length of bearing)=		9.97 feet	=((Sum of Loads))*1000)/(0.5*Pa*Fw
Check Overturning =	Mo =	108 inch-kips	(Mdes*.75)
	Mr =	2604 inch-kips	=((Sum of Loads))*1000)/(0.5*Pa*Fm
	S.F. =	**24.11**	Mr/Mo

Check Punching Shear

	Vc =	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
	bo=	115.88	

Ultimate Load to Footing:

	Pu =	81.99 kips	=1.4DL + 1.7LL

Ultimate Punching Shear:

Soil Load due to Column Load Only

	Pc =	1089.13 psf	=P/A ftg
	Vu=	28.09 kips	=P/Lin at edge

Calculate Depth Required:

Vu/phi*bo*Vc =	d=	1.13 inches

Check Flexural Shear

	Vc =	126 ksi

Soil Pressure at Critical Section:

	Psoil =	1640.22
Total Shear Force		12567.21 lbs
	Vu =	18533.60 lbs
	d =	2.05 inch

Flexural Shear Does NOT Control Design

Check Bending

M=	443.02	kip-feet
Mu=	753.13	kip-feet

Select Reinforcement

	Mu/ftg length =		108	kip-ft/ft	
	Minimum As =		0.52	inches^2	
**	Try	6- #6's	2.64	inches^2/foot	Each Way!!
	Mu Capacity =		220	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

Summit Orchards

Lee's Summit, Missouri

Misc. Calcs - Window Headers 9'-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*.025+1.4*.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*l)*span/2

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

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
Actual fb=	**1859.69**	**psi**
	OK	

Design Calculations

Summit Orchards

Lee's Summit, Missouri

Misc. Calcs - Studs @ Windows - Max Load

Column Capacity - 2x6 Studs

	Column Designation	None		
			BLOCKING REQ'D	
*	**Column Length**	**unbraced length**	8.0 feet	Re: Plan Drawings
*	Trib Width		14.4 feet	Re: Plan Drawings
	Trib Length		33.8 feet	Re: Plan Drawings
*	Uniform Load	Loads (LL + DL)	0.1 ksf	Load & Codes
		Additional Load (xxx)	0.0 kips	
	Column Loading		29.1 kips	Sum(Loads above)
*	Column "d"	12-2x6's	13.0 inches	Re: Plan Drawings
*	Column "b"		5.5 inches	Re: Plan Drawings
	Wind Loading	Considered in Other Calcs	0 kips	By Design & Drawings
			0 kips	By Design & Drawings
			0 kips	By Design & Drawings
			0 kips	By Design & Drawings
			0 kips	By Design & Drawings

For Conservatism Column Consider Unbraced !!

Total Design Load **29.10938 kips** **Sum (Loads above)**

Adjustment Factors

	Cd	0.9	Table 2.3.2, pg. 9
	Cm	1	Table 4B, NDS Sup, pg 36, **
	Ct	1	Table 2.3.3, pg. 9
	Cf	1	ASD Wood Design Manual
*	Bearing Stress =	825	Bearing Stress Unfactored
	Fc=	742.5	H.2, pg 157, See definition

	l = le		
	l/d Ratio =	7.384615	H.2, pg 157, NDS
		OK	
*	Kce =	0.3	Equation H3, pg 157, NDS
	E=E'	1500000	psi
	Fce=	8251.953	Equation H2, pg 157, NDS
	F'c=	728.396	Equation H1, pg 157, NDS
* Logic Statement	Design Stress	728.396 psi	Lesser Value of F'c or Fce
*	c=	0.80	Equation H2, pg 157, NDS
	Cp =	0.68	Equation 3.7-1, pg 19, NDS
Capacity	**P'=**	**36.33 kips**	**=((Fc*Cp)*(d*b))/1000**
	Capacity Check	OK	

Design Calculations

Summit Orchards

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		1 kips
(as applicable)	a =	10 ft
	b =	10 ft
Point Load 2		0 kips
(as applicable)	a =	0 ft
	b =	20 ft
Point Load 3		0 kips
(as applicable)	a =	0 ft
	b =	20 ft
Point Load 4		0 kips
(as applicable)	a =	0 ft
	b =	20 ft

Total Moment		Mu=.9*Mn =	115.20 kip-feet	Mu=.9*Mn =
			1382.40 kip-inches	
Reactions		Right Reaction =	25.10 kips	
		Left Reaction =	25.10 kips	
		Zx req'd =	30.72 in^3	=(Mu*12)/(0.9*Fy)
		OK		
Select	W21x44			
		Depth =	21.00 Inch	
		Area =	13.00 Inch^2	
		Zx =	95.40 Inch^3	
		Ixx =	843.00 inch^4	
		Fy =	50.00 ksi	
Allowable Defl.	L/240		1.00 inches	Span/Allowable
Deflection	Point & Uniform Ldg		0.37 inches	AISC Simple Bm
		Zx act =	95.4 in^3	
		Zx req'd =	30.72 in^3	
		OK		
		M(plastic) = Fy*Z =	4770 kip-in	AISC LRFD
		My = Fy*S =	4770 kip-in	AISC LRFD
		Mp Limit = 1.5*My =	7155 kip-in	AISC LRFD
		OK		
		J =	0.77	
		Lp =	10.10596	F10-5
		Lp (actual) =	5.0000	

Design Calculations

Summit Orchards

Lee's Summit, Missouri

Wind Posts

Supporting Front Wall (Store Front)

Building Height	16	4877 mm	Dim. Converted to mm
Trib Width	Building Length/4=	16.875 ft	Per Drawings
Wind Post Height		16 ft	Per Drawings
Beam Loading Live =	LL = (1.6*.025)	0.04 kips/ft^2	Gravity Load Calcs
Trib Area	(half wall width, per bm)	8.44 feet	
	Total Loading (Vertical)	0.34 kips/lin. ft	
Point Load 1		1 kips	
(as applicable)	a =	8 ft	
	b =	8 ft	
Point Load 2		0 kips	
(as applicable)	a =	0 ft	
	b =	16 ft	
Point Load 3		0 kips	
(as applicable)	a =	0 ft	
	b =	16 ft	
Point Load 4		0 kips	
(as applicable)	a =	0 ft	
	b =	16 ft	

Total Moment	Mu=.9*Mn =	13.32	kip-feet	Mu=.9*Mn =
		159.84	kip-inches	
Reactions	Right Reaction =	3.20	kips	
	Left Reaction =	3.20	kips	
	Zx req'd =	3.55	in^3	=(Mu*12)/(0.9*Fy)
	OK			
Select	W12x26			
	Depth =	12.00	Inch	
	Area =	7.85	Inch^2	
	Zx =	37.20	Inch^3	
	Ixx =	204.00	inch^4	
	Fy =	50.00	ksi	
Allowable Defl.	L/240	0.80	inches	Span/Allowable
Deflection	Point & Uniform Ldg	0.11	inches	AISC Simple Bm
	Zx act =	37.2	in^3	
	Zx req'd =	3.55	in^3	
	OK			
	M(plastic) = Fy*Z =	1860	kip-in	AISC LRFD
	My = Fy*S =	1860	kip-in	AISC LRFD
	Mp Limit = 1.5*My =	2790	kip-in	AISC LRFD
	OK			
	J =	0.3		
	Lp =	7.183287		F10-5
	Lp (actual) =	5.0000		

Attachment to Footing

Mu =	13.32	kip-ft	
Bolt Centers	16	inches	

Couple Force = Mu/Ctrs	9.99	Kips
Number of Bolts Anchors	2	bolts
Force Per Bolt Force/no.	4.995	kips
Select Bolt	3/4" A307	
Capacity	14	kips
	OK	

Seismic

SEISMIC LOADS - ASCE 7-2010

Site Class Soil Definition	D		Per Geotechnical recommendation
	.2 Second Response Ss=	0.12	Map Figure 22-1 p.211
	Fa =	1.6	Table 11.4-1
SMS =	Fa x Ss =	0.192	Eq. 11.4-1
SDS =	2/3 SMS =	0.128	Eq. 11.4-3
	1 Second Response S1=	0.06	Map Figure 22-2 p.213
	Fv =	2.4	Table 11.4-2
SM1 =	Fv x S1 =	0.144	Eq. 11.4-2
SD1 =	2/3 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 Wo =		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 = Cs W			Eq. 12.8-1 (p129)
Cs=	SDS = R / I	0.042667 Use	Eq. 12.8-2
Max. Cs=	SD1 = (R / I) T	0.336359	Eq. 12.8-3
	Ta=	CT hnx = 0.1	Eq. 12.8-7
		CT = 0	Table 12.8-2 -All other systems
		hn= 8	Taken at Median height
		x= 0.8	Table 12.8-2-All other systems
	T<=	CuTa= 0.2	OK
		Cu= 1.6	Table 12.8-1 (p129) for SD1<.1
	(See Eq. 12.8-4 for period T> TL of 12 sec for Max Cs)		
Min. Cs=	.5 S1 = R / I	0.01 <= .0	Eq. 12.8-5 and 12.8-6
Dead Load =	80x60x.035		168.00
Dead Load =			

168.00

Total Dead Load W for Bldg=	168 k	
Total Base Shear V = Cs W =	7.17 k	To building Frame

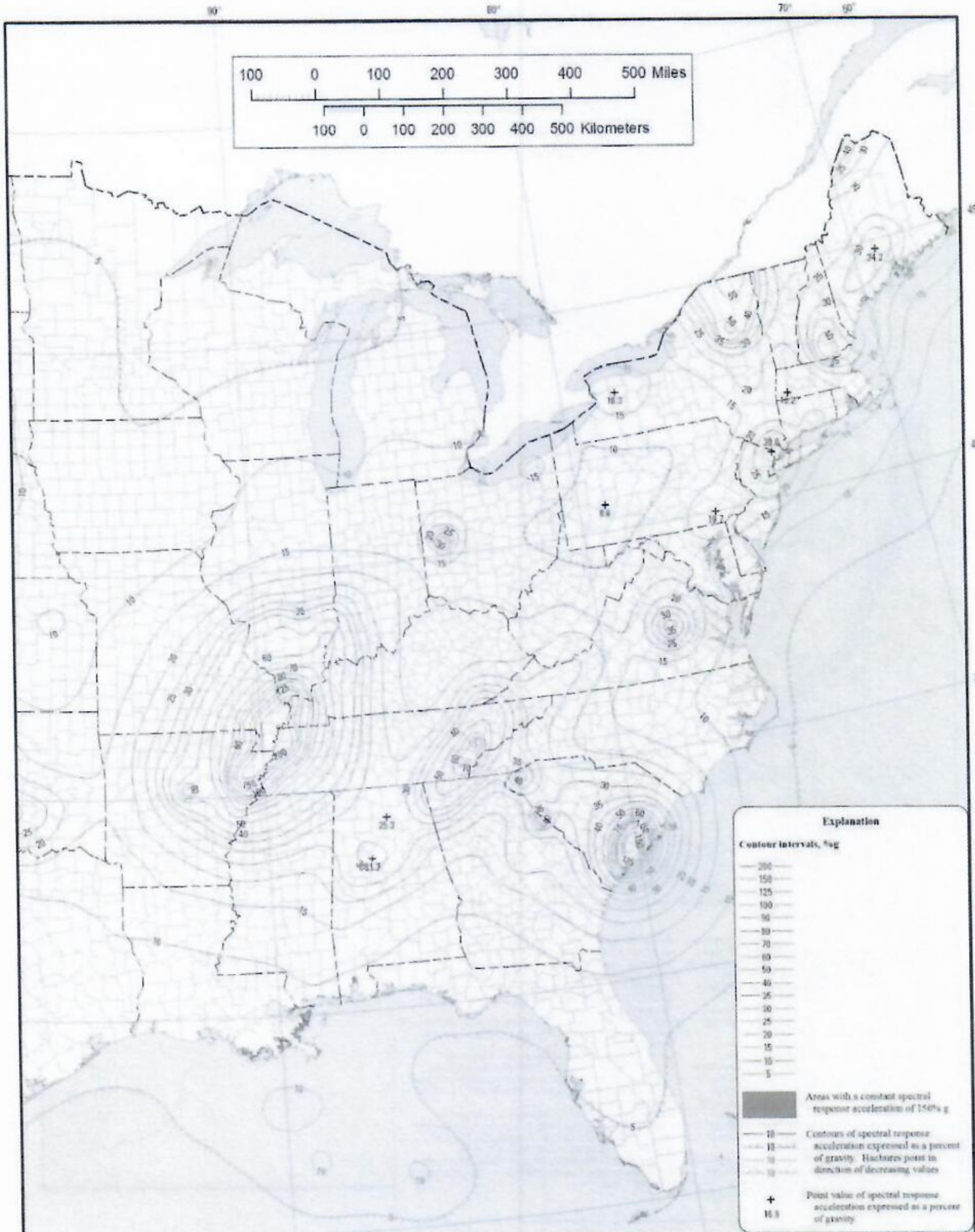

FIGURE 1613.2.1(1)—continued
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)

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

End of Review