Blossom Restaurant

Structural Calculations

Engineer's seal applies to this entire calculation packet.

This engineering report is valid only for the aforementioned building located at 2082 North Hillcrest Road, Saratoga Springs, Utah. This report is to be used only once and may not be copied or reproduced without the written consent of LEI Engineers and Surveyors, Inc.

LEI

- A Utah Corporation -

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LEI Project #:

2020-2072

Location:

Saratoga Springs, Utah

Date:

2/24/2020

Engineered by:

J. Miller

Reviewed by:

STATE OF UTAH
JOSHUA K.S. ANDERSON
No. 4939007-2203
02-28-20
PROFESSIONAL STRUCTURAL ENGINEER

APPLIES TO PAGES 1-38

Structural Review for: **Blossom Restaurant**
Location: **Saratoga Springs, Utah**
Job #: **2020-2072**
Engineered by: **J. Miller**
Code: **2018 IBC**

Loadings

Risk Category: II

Ground Snow Load:

Elevation = 4642 ft
p_g = 33.0 psf

Roof Snow Load:

C_t = 1.0
Roof Exposure C_e = 1.0 Partially
I_s = 1.0
p_f = 23.1 psf

Roof Dead Load:

DL = 15 psf

Wind Loading:

Roofing Material = Shingle/Tile
Roof Pitch = 0.25 /12
Roof Angle = 1.2 degrees
Exposure Category = C
Mean Roof Height = 25
Wind Speed V = 103
Height & Exposure Factor λ = 1.35

p_{s30} Horizontal Pressures				p_{net30}	
zone A	zone B	zone C	zone D	zone 4	zone 5
16.86	-8.74	11.16	-5.20	17.82	23.88

p_s Horizontal Pressures				p_{net}	
zone A	zone B	zone C	zone D	zone 4	zone 5
22.8	**0.0**	**15.1**	**0.0**	**24.1**	**32.2**

Seismic Loading:

Number of Stories = 1
Roof diaphragm height h_r = 25 ft
I_e = 1.00
Fundamental Period T_a = 0.224 sec.
F = 1
Site Class = D **(Assumed)**
R factor = 6.5 Structural Sheathing
R factor = 6.5 Simpson Strong Wall
R factor = 6.5 Portal Frame
R factor = 2 Gypsum Sheathing
R factor = 5 Masonry Shear Wall
R factor = 4 Concrete Shear Wall
R factor = 2.5 Cantilever Steel Post
R factor = 4.5 Steel Moment Frame
S_S = 1.042
S_1 = 0.377
F_a = 1.2
F_v = 1.923
S_{MS} = 1.2504
S_{M1} = 0.724971
S_{DS} = 0.834
S_{D1} = 0.483
T_o = 0.115958 sec.
T_s = 0.579791 sec.
Seismic Design Category = D

Soil Bearing Capacity: 2500 psf **(Earthtec Project No. 131422 dated september 26, 2013)**

2018 Utah Ground Snow Load Map

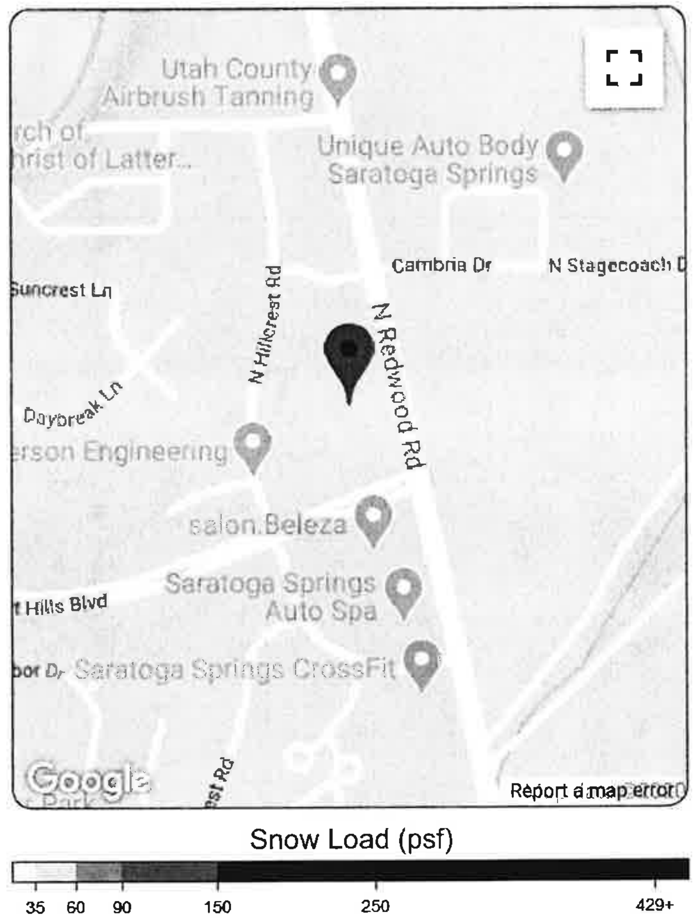
Blossom Restaurant

UtahState University

STRUCTURAL ENGINEERS ASSOCIATION OF UTAH

Latitude: 40.400
Longitude: -111.920
Elevation: 4,642 ft

Ground Snow Load:
33 psf / 1.59 kPa



*This document is not legally binding. The user is urged to verify ground snow load values with the local authority having jurisdiction.

These ground snow load values represent 50-year ground snow load estimated value at a 2% probability of exceedance for the location given. The grid used in the map is 3350ft by 3350ft. Elevations for these grid cells were estimated by aggregating data from 100ft by 100ft USGS digital elevation models and may not coincide with the actual site elevation. These predictions are calculated using the process outlined in The Utah Snow Load Study.1

Final predictions given are bounded at a lower limit for a minimum ground snow load of 21 psf to meet ASCE 7. Estimated values for snow loads at elevations significantly higher than all nearby stations lead to unreasonably high snow load estimates, therefore, the predictions in the map are not allowed to extend beyond the highest 50-year station ground snow load of 429 psf. Elevations over 9,000 ft are also considered less accurate due to the limited number of stations at these elevations. The results shown in this report have included a warning if the results have reached or exceeded the upper limit.

While great efforts have been made to ensure these predictions are as accurate as possible, designers must use expert judgement to ensure that such predictions are appropriate for their particular project. The SEAU and the authors cannot accept responsibility for prediction errors or any consequences resulting therefrom.

1 Bean, Brennan; Maguire, Marc; and Sun, Yan, "The Utah Snow Load Study" (2018). Civil and Environmental Engineering Faculty Publications. Paper 3589.

ASCE
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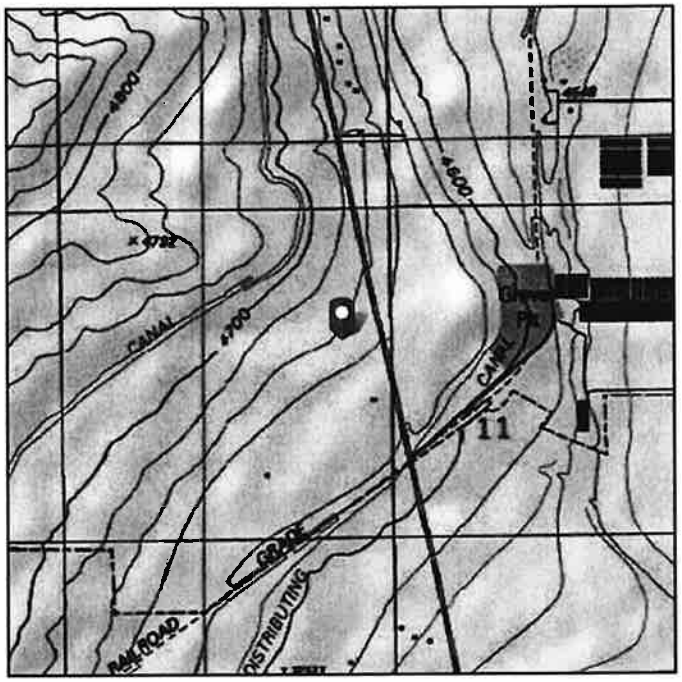
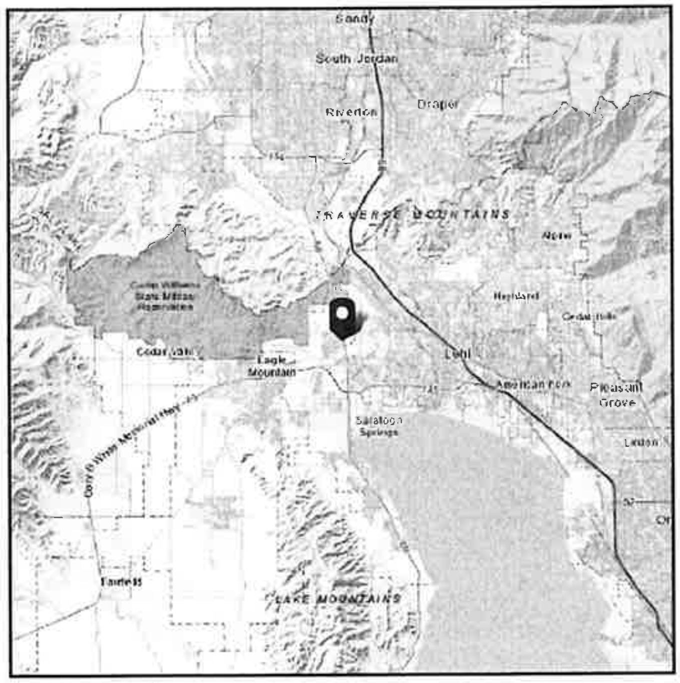
ASCE 7 Hazards Report

Address:
2082 N Hillcrest Rd
Saratoga Springs, Utah
84045

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Stiff Soil

Elevation: 4657.63 ft (NAVD 88)
Latitude: 40.400272
Longitude: -111.920957





Wind

Results:

Wind Speed:	103 Vmph
10-year MRI	73 Vmph
25-year MRI	79 Vmph
50-year MRI	84 Vmph
100-year MRI	89 Vmph
Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4
Date Accessed:	Fri Feb 21 2020

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Mountainous terrain, gorges, ocean promontories, and special wind regions should be examined for unusual wind conditions.

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Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	1.042	S_{D1} :	N/A
S_1 :	0.377	T_L :	8
F_a :	1.083	PGA :	0.457
F_v :	N/A	PGA_M :	0.523
S_{MS} :	1.128	F_{PGA} :	1.143
S_{M1} :	N/A	I_e :	1
S_{DS} :	0.752	C_v :	1.308

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Fri Feb 21 2020

Date Source: USGS Seismic Design Maps

Snow Drift Calculations

	Drift #1	Drift #2	Drift #3
Roofing Material =	Shingle/Tile	Shingle/Tile	Shingle/Tile
Ground Snow Load p_g (psf)=	33	33	33
Flat Roof Snow Load p_f (psf)=	23	23	23
Roof Pitch =	0.25	0.25	0.25
Angle =	1	1	1
C_s =	1.00	1.00	1.00
Sloped Roof Snow Load p_s (psf)=	23	23	23
λ (lb/ft^3)=	18.29	18.29	18.29
Height of normal Snow Load h_b (ft)=	1.26	1.26	1.26
Roof Height Difference h_c (ft)=	9	5	4
Does Drift Exist (h_c/h_b > 0.2)? =	Yes	Yes	Yes
Low Roof or Parapet =	Low Roof	Parapet	Parapet
Length of upper roof l_u (ft)=	42	52	24
Length of lower roof (ft)=	13		
Height of Drift h_d (ft)=	2.3	2.0	1.3
Drift tapers to zero @ **w** (ft)=	9	8	5
Drift Load p_d (psf)=	43	36	23
Total load (psf)=	**66**	**59**	**46**

Siesmic Weight

Additional Seismic Weight (psf)=	0.0
Total Seismic Weight (psf)=	**15.0**

Preface & Structural Notes

This engineering report is valid only for the following plan and location:

Blossom Restaurant

2082 North Hillcrest Road, Saratoga Springs, Utah

NOTE TO PLAN CHECKER AND BUILDING INSPECTOR:

If the above address does not match the intended building address, notify LEI immediately @ 801-798-0555. This engineering packet is to be used only once for the above mentioned location and is not to be copied or reproduced without written consent of LEI Consulting Engineers and Surveyors, Inc.

Structural Notes:

General Notes

1 If values and assumptions stated in this report are incorrect, or if changes in the field are noticed which are different from those stated in this report, the engineer must be notified in order for the necessary corrections to be made.
2 If there are any discrepancies between the calculations and the drawings, these calculations shall govern.
3 This engineering report deals only with the structural parts of the building and does not apply to the non-structural parts.
4 If drawings are stamped in conjunction with this engineering report, certification pertains only to the structural elements of the drawings.
5 The general contractor is responsible for the method, means, and sequence of all structural erection except when specifically noted otherwise on the drawings. General contractor shall provide temporary shoring and bracing as his method of erection requires to provide adequate vertical and lateral support during erection. This shoring and bracing shall remain in place until all permanent members are placed and all final connections are completed including all roof and floor attachments.

Site Preparation

1 Do not place footings or foundations on disturbed soils, undocumented fill, debris, frozen soil, or in ponded water.
2 All slabs on grade shall be underlain by 4 in. of free-draining granular material such as "pea" gravel or 3/4 - 1 in. minus clean gravel.
3 Footings, foundations, excavations, grading and fill shall comply with the geotechnical report.

Concrete

1 All concrete footings and slabs on grade shall have a 28 day minimum strength = 2500 psi.
2 All concrete foundation walls and retaining walls shall have a 28 day minimum strength = 3000 psi.
3 Concrete shall be thoroughly consolidated by suitable means during placement.
4 Footings shall be centered below the wall and/or column above, typical unless noted otherwise.
5 Exterior footings shall bear below the effects of frost.
6 Stagger footing construction joints from wall construction joints above by at least 6 feet.
7 Reinforcing in continuous footings shall be continuous at corners and/or intersections by providing proper lap lengths and/or corner bars.
8 Interior slabs on grade shall be a min. of 4" thick.
9 Place vertical reinforcing in the center of the wall (except for retaining walls or when each face is specified).
10 Vertical reinforcing shall be dowelled to footing or structure below and to structure above with the same size bar and spacing, typical U.N.O.
11 Provide corner bars at all intersections and corners. Use same size bar and spacing as the horizontal reinforcing.
12 Horizontal reinforcing shall terminate at the ends of the walls and at openings with a standard hook.
13 Provide drainage at the base of retaining walls.

Reinforcing Steel

1 Reinforcing steel shall be new stock deformed bars and shall conform to ASTM A615, grade 60, with a design yield strength = 60 ksi.
2 Reinforcing steel shall be free of loose, flaky rust, scale, grease, oil, dirt, and other materials which might affect or impair bond.
3 Splices in continuous reinforcing shall be made on areas of compression and/or at points of minimum stress, typical U.N.O.
4 Lap splices shall be 40 bar diameters or 24" long in concrete. Dowels shall have a minimum of 30 bar diameters embedment.
5 Bends shall be made cold; do not use heat. Do not un-bend or re-bend a previously bent bar.
6 Reinforcing steel in concrete shall be securely anchored and tied in place prior to placing concrete and shall be positioned with the following minimum cover:
concrete cast against and permanently exposed to earth = 3"
concrete exposed to earth or weather = 1 1/2"
slabs on grade = center of slab

Structural Steel

1 Structural steel W-shapes shall conform to ASTM A992 grade 50 enhanced steel. Structural steel plates shall conform to ASTM A36.
2 Structural steel HSS-shapes shall conform to ASTM A500, grade B, with a min. yield strength Fy = 46 ksi (rectangular) or Fy = 42 ksi (round).
3 Structural pipe shall conform to ASTM A53, with a min. yield strength Fy = 36 ksi.
4 High strength bolts shall conform to ASTM A325, all other bolts shall conform to ASTM A307 or better.
5 Welded anchor studs and deformed bar anchors shall conform to the manufacturer's specs.
6 Fabrication shall be done in an approved fabricator's shop.
7 Use high strength (8000 psi min. at 28 days), non shrink, liquid epoxy grout beneath all steel base plates and bearing plates.
8 Bolt shall be bearing type connections U.N.O.
9 Steel to steel bolted connections shall be made with ASTM A325 high strength bolts and nuts, U.N.O.
10 All other bolted connections shall be made with bolts and nuts conforming to ASTM A307 U.N.O., including anchor bolts.
11 Bolted connections shall be tightened and shall have washers as required by AISC U.N.O.
12 Enlarging of holes shall be accomplished by means of reaming. Do not use a torch on any bolt holes.
13 Welded connections shall be made using low hydrogen matching filler material electrodes, U.N.O.
14 Welders shall be currently certified according to AWS within the last year. All welding procedures shall be pre-qualified. Welders shall follow welding procedures.
15 Welding and gas cutting shall be done per AWS.
16 Welds shall have the slag removed.

Structural Notes (cont):

Masonry Veneer Anchor Ties

1 Masonry veneer ties shall be one of the following:
   a. Dovetail anchors
   b. DX-10 seismic clip interlock system by Hohmann & Barnard
   c. Engineer approved 2 piece adjustable hot-dipped galvanized ties.
2 Maximum spacing shall be 16" o.c. horizontal and vertical.
3 Provide continuous horizontal galvanized #9 wire in center third of mortar joints at 16" o.c. Engage #9 wire with all anchor ties in seismic zone category E.

Wood Truss

1 Bottom chords of trusses, acting as ceiling members must be able to support a 10 psf live load per IBC requirements.
2 The truss manufacturer shall be responsible for the design and fabrication of the pre-engineered trusses.
3 The trusses shall be designed as per the attached engineering specs.
4 The trusses shall be designed to carry any additional loads due to mechanical units, overhead doors, roof overbuilds, etc.
5 The trusses shall be designed per the IBC and local ordinances.
6 All members shall be designed for combined stresses based on the worst loading condition.
7 The truss manufacturer shall indicate proper bracing of compression chord members @ 6' long (or longer), as well as bracing for truss erection.
8 All dimensions shall be field verified prior to fabrication.
9 General contractor shall be responsible for the installation of the trusses per the truss manufacturer's recommendations and specs.
10 No web or chord members shall be modified in the field without approval from the truss engineer.
11 The engineer is not responsible for the pre-engineered trusses, nor for the installation of the trusses.
12 General contractor is to verify truss layout is consistent with these plans and notify engineer of any deviations.

General Framing

1 All joists, rafters, posts and headers shall be DF-L #2 or equal U.N.O. If TJI's or equal are used, they must be installed per manufacturer's specs.
2 All joists and rafters shall have solid blocking at their bearing points.
3 All wood/lumber placed onto concrete shall be pressure treated or redwood.
4 Verify all beam sizes on the drawings with this report.
5 All beams and headers over 6'-0" shall be supported by double trimmer studs U.N.O.
6 All headers over 8'-0" shall shall have double king studs at each end U.N.O.
7 All over frame areas are to have full roof sheathing below.
8 Provide solid blocking and continuous bearing to foundation at all bearing point loads from above.
9 Provide double floor joists below all parallel bearing walls above.
10 Glulam beams shall be 24F-V4 DF/DF for single spans and 24F-V8 DF/DF for multiple spans and cantilevered spans.
11 Microllam beams shall be Laminated Veneer Lumber (LVL) with the following minimum design values: E=1,900,000 psi, Fb=2,600 psi, Fv=285 psi.
12 Parallam beams shall be Parallel Strand Lumber (PSL) with the following minimum design values: E=2,000,000 psi, Fb=2,900 psi, Fv=290 psi.
13 TimberStrand beams shall be Laminated Strand Lumber (LSL) w/ the following minimum design values:
   - 1-1/4" wide (rim board): E=1,300,000 psi, Fb=1,700 psi, Fv=425 psi.
   - 1-3/4" wide: E=1,550,000 psi, Fb=2,325 psi, Fv=310 psi.
14 All rafters and joists over 3 ft long shall be hangered if not supported by bottom bearing.
15 All hangers and other wood connections must be designed to carry the capacity of the member that they are supporting.
16 No structural member shall be cut or notched unless specifically shown, noted or approved by engineer.
17 Lag screws shall be inserted in a drilled pilot hole 60 - 75% of the shank diameter by turning with a wrench, not by driving with a hammer.
18 Nails are to be common wire U.N.O.
19 All bolt holes shall be drilled with a bit 1/32" to 1/16" larger than the nominal bolt diameter.
20 All joints in wall sheathing shall occur in the middle of a plate or block and nailed on each side of the joint w/ edge nailing per the shearwall schedul
21 All over built roof rafters shall be braced vertically to the trusses below at 4' o.c. max.
22 Double top plates are to have a minimum 48" lap splice w/ (8) 16d nails U.N.O.
23 All fasteners and connectors in contact with treated lumber shall be galvanized G90 or better.

Summary

Roof:

RR1: 2x12 DF-L#2 @ 24" o.c. as noted on plans
RR2: 2x10 DF-L#2 @ 16" o.c. as noted on plans
Trusses by others
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field
Overbuild to be 2" x 6" Timber @ 24" o.c.

Other:

All bearing headers to be (2) 2x10 (DF L #2 or better) unless noted otherwise
All exterior sheathing to be Shear Wall #1 unless noted otherwise
All glulam beams are to be 24F-V4 unless noted otherwise
Strap end lengths for shear walls (see also Simpson Coiled strap specs.):
CS16 = 14" CMST14 = 34" CMSTC16 = 25"

Beam Schedule			
Desig.	**Qty.**	**Size**	**Type**
RB1	2	2 x 8	DF-L#2
RB2	3	1 3/4" x 11 7/8"	Microllam
RB3	1	5 1/2" x 19 1/2"	Glulam
RB4	1	5 1/2" x 30"	Glulam
RB5	3	1 3/4" x 14"	Microllam
RB6	1	5 1/2" x 28 1/2"	Glulam
RB7	2	1 3/4" x 14"	Microllam
RB8	3	1 3/4" x 11 7/8"	Microllam
RB9	2	1 3/4" x 11 7/8"	Microllam
RB10	2	1 3/4" x 11 1/4"	Microllam
RB11	1	W 10x49	A992-50 - Steel
RB12	3	1 3/4" x 11 7/8"	Microllam
RB13	1	W 10x49	A992-50 - Steel
RB14	2	1 3/4" x 11 7/8"	Microllam

Project: 2018-2072

Location: RR1
Roof Rafter

1.5 IN x 11.25 IN x 8.0 FT (4 + 4) @ 24 O.C.
#2 - Douglas-Fir-Larch - Dry Use
Section Adequate By: 420.2%
Controlling Factor: Moment

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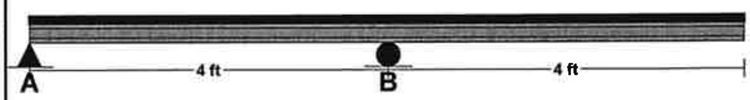
DEFLECTIONS	Center		Right	
Live Load	0.00	IN L/MAX	0.02	IN 2L/4606
Dead Load	0.00	in	0.01	in
Total Load	0.00	IN L/MAX	0.03	IN 2L/3354

Live Load Deflection Criteria: L/240 Total Load Deflection Criteria: L/180

REACTIONS	A		B	
Live Load	92	lb	368	lb
Dead Load	0	lb	160	lb
Total Load	92	lb	528	lb
Uplift (1.5 F.S)	**-92**	**lb**	**0**	**lb**
Bearing Length	0.10	in	0.56	in

SUPPORT LOADS	A		B	
Live Load	46	plf	184	plf
Dead Load	0	plf	80	plf
Total Load	46	plf	264	plf

LOADING DIAGRAM



MATERIAL PROPERTIES
#2 - Douglas-Fir-Larch

	Base Values			Adjusted		
Bending Stress:	Fb =	900	psi	Fb' =	1042	psi
	Cd=1.15 Cl=0.88 CF=1.00 Cr=1.15					
Shear Stress:	Fv =	180	psi	Fv' =	207	psi
	Cd=1.15					
Modulus of Elasticity:	E =	1600	ksi	E' =	1600	ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	625	psi	Fc - ⊥' =	625	psi

Controlling Moment: -528 ft-lb
3.999 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 3
Controlling Shear: -203 lb
At a distance d from right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2, 3

Comparisons with required sections:	Req'd	Provided
Section Modulus:	6.08 in3	31.64 in3
Area (Shear):	1.47 in2	16.88 in2
Moment of Inertia (deflection):	9.55 in4	177.98 in4
Moment:	-528 ft-lb	2747 ft-lb
Shear:	-203 lb	2329 lb

RAFTER DATA	Interior		Eave	
Span Length	4	ft	4	ft
Rafter Pitch	0.25	:12		

Roof sheathing applied to top of joists-top of rafters fully braced.

Roof Duration Factor	1.15
Peak Notch Depth	0.00
Base Notch Depth	0.00

RAFTER LOADING

Uniform Roof Loading			
Roof Live Load:	LL =	23	psf
Roof Dead Load:	DL =	10	psf
Slope Adjusted Spans And Loads			
Interior Span:	L-adj =	4	ft
Eave Span:	L-Eave-adj =	4	ft
Interior Live Load:	wL-adj =	46	plf
Eave Live Load:	wL-Eave-adj =	46	plf
Interior Dead Load:	wD-adj =	20	plf
Eave Dead Load:	wD-Eave-adj =	20	plf
Interior Total Load:	wT-adj =	66	plf
Eave Total Load:	wT-Eave-adj =	66	plf

Project: 2018-2072

Location: RR1 - Check
Multi-Loaded Multi-Span Beam

1.5 IN x 11.25 IN x 13.0 FT
#2 - Douglas-Fir-Larch - Dry Use
Section Adequate By: 21.0%
Controlling Factor: Moment

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DEFLECTIONS	Center	
Live Load	0.16	IN L/951
Dead Load	0.08	in
Total Load	0.24	IN L/650

Live Load Deflection Criteria: L/480 Total Load Deflection Criteria: L/360

REACTIONS	A		B	
Live Load	597	lb	388	lb
Dead Load	219	lb	219	lb
Total Load	816	lb	607	lb
Bearing Length	0.87	in	0.65	in

BEAM DATA	Center	
Span Length	13	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	13	ft
Live Load Duration Factor	1.00	
Notch Depth	0.00	

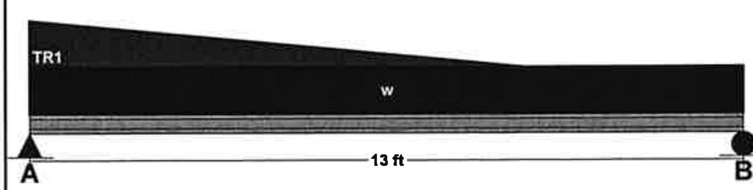
MATERIAL PROPERTIES
#2 - Douglas-Fir-Larch

	Base Values			Adjusted		
Bending Stress:	Fb =	900	psi	Fb' =	1035	psi
	Cd=1.00 CF=1.00 Cr=1.15					
Shear Stress:	Fv =	180	psi	Fv' =	180	psi
	Cd=1.00					
Modulus of Elasticity:	E =	1600	ksi	E' =	1600	ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	625	psi	Fc - ⊥' =	625	psi

Controlling Moment: 2255 ft-lb
5.98 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: 669 lb
At a distance d from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Comparisons with required sections:	Req'd	Provided
Section Modulus:	26.15 in3	31.64 in3
Area (Shear):	5.57 in2	16.88 in2
Moment of Inertia (deflection):	98.58 in4	177.98 in4
Moment:	2255 ft-lb	2729 ft-lb
Shear:	669 lb	2025 lb

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	46	plf
Uniform Dead Load	30	plf
Beam Self Weight	4	plf
Total Uniform Load	80	plf

TRAPEZOIDAL LOADS - CENTER SPAN	
Load Number	One
Left Live Load	86 plf
Left Dead Load	0 plf
Right Live Load	0 plf
Right Dead Load	0 plf
Load Start	0 ft
Load End	9 ft
Load Length	9 ft

Ledgers

	L1	
Roofing material =	Shingle/Tile	
Roof Pitch=	0.25	
Angle=	1.2	
C_S =	1.000	
Increase for Drift/Valley=	1.988	
Effective snow load (psf)=	46	
Roof dead load (psf)=	15	
Floor live load (psf)=	40	
Floor dead load (psf)=	15	
Fastener spacing (ft)=	1.33	
Trib. Area $_{roof}$=	6.5	
Trib. Area $_{floor}$=	0	
w_S (plf) =	299	
w_L (plf) =	0	
w_D (plf) =	101	
$w_{self\ weight}$ (plf) =	3.0	
Point Load: Snow (lb)=		
Live (lb)=		
Dead (lb)=		
a (ft)=	0.665	
b (ft)=	0.665	
Add. uniform load (plf)=		
Allowable Live Deflection =	L/240	
Allowable Total Deflection =	L/180	
Left/Right Reaction: Factored (lb)=	531	531
Snow (lb)=	397	397
Live (lb)=	0	0
Dead (lb)=	134	134
V_{max} (lb)=	265	
M_{max} (ftlb)=	88	
Size Factor (C_F)=	1.10	
Volume Factor (C_v)=	1.00	
Duration Factor (C_d) =	1.15	
Beam Type (t,t1,tss,g,m,p,ts,rb)	t	
d (in)=	9.25	
b (in)=	1.5	
	Section OK	
	Shear OK	
	Deflection OK	

Fastener Options:

Nails:	16d	
Nails into rim (1.25" min.) =	5	5
Nails into studs / beam >10D =	4	4
Lag Bolts:		
3/8" lag bolts into rim (1.25" min.) =	2	2
1/2" lag bolts into rim (1.25" min.) =	1	1
SDWS Screws:		
Through sheathing? (1/2" max.) =	Yes	
Through 5/8" gypsum? (# of layers) =	0	
Screws into rim (1.125" min.) =	2	2
	SDWS22400DB	
Screws into studs =	2	2
	SDWS22500DB	
Screws into beam / blocking =	N/A	N/A
	N/A	
Custom:		
	N/A	N/A
	N/A	N/A

Beams	RB1	RB2	RB2 - Check	RB3	RB4	RB5	RB6	RB7	RB8	RB9	RB10 - RB13	RB14
Roofing material =	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile		Shingle/Tile
Roof Pitch=	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25		0.25
Angle=	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2		1.2
C_S =	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000		1.000
Increase for Drift/Valley=	1.180	2.100	1.700	1.000	1.000	1.350	1.000	1.000	1.456	1.000		1.600
Effective snow load (psf)=	27	49	39	23	23	31	23	23	34	23		37
Roof dead load (psf)=	15	15	15	15	15	15	15	15	15	15		15
Floor live load (psf)=	40	40	40	40	40	40	40	40	40	40		40
Floor dead load (psf)=	15	15	15	15	15	15	15	15	15	15		15
Length (ft)=	4	11	18.5	21	30	14	28	10	18	12.5		9
Trib. Area $_{roof}$=	20.5	16.5	4	25	25	29.5	29.5	14	6.5	6.5		27
Trib. Area $_{floor}$=	0	0	0	0	0	0	0	0	0	0		0
w_S (plf) =	559	800	157	578	578	920	681	323	219	150		998
w_L (plf) =	0	0	0	0	0	0	0	0	0	0		0
w_D (plf) =	312	260	72	401	414	464	480	224	116	110		417
$w_{self\ weight}$ (plf) =	4.7	12.1	12.1	25.6	39.3	21.3	37.4	14.2	18.1	12.1		12.1
Point Load: Snow (lb)=												
Live (lb)=												
Dead (lb)=												
a (ft)=	2	5.5	9.25	10.5	15	7	13	5	9	6.25		4.5
b (ft)=	2	5.5	9.25	10.5	15	7	13	5	9	6.25		4.5
Add. uniform load (plf)=												
Allowable Live Deflection =	L/240	L/240	L/240	L/240	L/240	L/240	L/240	L/240	L/240	L/240		L/240
Allowable Total Deflection =	L/180	L/180	L/180	L/180	L/180	L/180	L/180	L/180	L/180	L/180		L/180
Left/Right Reaction: Factored (lb)=	**1742 1742**	**5830 5830**	**2120 2120**	**10270 10270**	**14877 14877**	**9687 9687**	**15097 15097**	**2738 2738**	**3008 3008**	**1623 1623**	See attached calculations	**6367 6367**
Snow (lb)=	1117 1117	4402 4402	1453 1453	6064 6064	8663 8663	6440 6440	8859 8859	1617 1617	1968 1968	938 938		4491 4491
Live (lb)=	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0		0 0
Dead (lb)=	624 624	1428 1428	667 667	4206 4206	6215 6215	3247 3247	6238 6238	1121 1121	1040 1040	685 685		1877 1877
V Redution Allowed (uniform load)=	No	No	No	No	No	No	No	No	No	No		No
V Redution Allowed (point load)=	No	No	No	No	No	No	No	No	No	No		No
V_{max} (lb)=	1742	5830	2120	10270	14877	9687	15097	2738	3008	1623		6367
Location of M_{max} (ft)=	2.00	5.50	9.25	10.50	15.00	7.00	13.00	5.00	9.00	6.25		4.50
M_{max} (ftlb)=	1742	16032	9803	53916	111580	33903	98131	6845	13537	5073		14327
Size Factor (C_F)=	1.20	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00
Volume Factor (C_v)=	1.00	1.00	1.00	0.95	0.87	0.98	0.89	0.98	1.00	1.00		1.00
Duration Factor (C_d) =	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15		1.15
Beam Type (t,t1,tss,g,m,p,ts,rb)	t	m	m	g	g	m	g	m	m	m		m
d (in)=	7.25	11.875	11.875	19.5	30	14	28.5	14	11.875	11.875		11.875
b (in)=	3	3.5	3.5	5.5	5.5	5.25	5.5	3.5	5.25	3.5		3.5
$F'_{c\perp}$ =	625	750	750	650	650	750	650	750	750	750		750
L/R Bearing Width (in)=	3 3	3.5 3.5	3.5 3.5	5.5 5.5	5.5 5.5	5.25 5.25	5.5 5.5	3.5 3.5	5.25 5.25	3.5 3.5		3.5 3.5
L/R Req'd Bearing Length (in)=	0.93 0.93	**2.22 2.22**	0.81 0.81	**2.87 2.87**	**4.16 4.16**	**2.46 2.46**	**4.22 4.22**	1.04 1.04	0.76 0.76	0.62 0.62		**2.43 2.43**
I (in^4)=	95.3	488.4	488.4	3398.5	12375.0	1200.5	10610.0	800.3	732.6	488.4		488.4
F_b	900	2600	2600	2400	2400	2600	2400	2600	2600	2600		2600
F'_b=	1242	2994	2994	2611	2413	2928	2460	2928	2994	2994		2994
S (in^3)=	26.3	82.3	82.3	348.6	825.0	171.5	744.6	114.3	123.4	82.3		82.3
S_{req}=	17	64	39	248	555	139	479	28	54	20		57
	Section OK	**Section OK**	**Section OK**	**Section OK**	**Section OK**	**Section OK**	**Section OK**	**Section OK**	**Section OK**	**Section OK**		**Section OK**
E (psi)=	1600000	1900000	1900000	1600000	1800000	1900000	1800000	1900000	1900000	1900000		1900000
F'_v (psi)=	207	327.75	327.75	304.75	304.75	327.75	304.75	327.75	327.75	327.75		327.75
f_v (psi)=	120	210	76	144	135	198	144	84	72	59		230
	Shear OK	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**		**Shear OK**
Location of Max Deflection=	2.00	5.50	9.25	10.50	15.00	7.00	13.00	5.00	9.00	6.25		4.50
Allowable Live Deflection (in)=	0.200	0.550	0.925	1.050	1.500	0.700	1.300	0.500	0.900	0.625		0.450
Live Deflection (in)=	0.021	0.284	0.446	0.413	0.473	0.349	0.367	0.048	0.371	0.089		0.159
Allowable Total Deflection (in)=	0.267	0.733	1.233	1.400	2.000	0.933	1.733	0.667	1.200	0.833		0.600
Total Deflection (in)=	0.033	0.376	0.651	0.700	0.811	0.524	0.625	0.081	0.567	0.154		0.225
	Deflection OK	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**		**Deflection OK**

Project: 2018-2072

Location: RB09 - Check
Multi-Loaded Multi-Span Beam

(2) 1.75 IN x 11.875 IN x 12.5 FT
1.9E Microllam - iLevel Trus Joist
Section Adequate By: 326.4%
Controlling Factor: Deflection

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CAUTIONS
* Laminations are to be fully connected to provide uniform transfer of loads to all members

DEFLECTIONS	Center	
Live Load	0.05	IN L/2752
Dead Load	0.04	in
Total Load	0.10	IN L/1535

Live Load Deflection Criteria: L/480   Total Load Deflection Criteria: L/360

REACTIONS	A	B
Live Load	719 lb	431 lb
Dead Load	550 lb	362 lb
Total Load	1269 lb	793 lb
Bearing Length	0.48 in	0.30 in

BEAM DATA	Center
Span Length	12.5 ft
Unbraced Length-Top	0 ft
Unbraced Length-Bottom	12.5 ft
Live Load Duration Factor	1.00
Notch Depth	0.00

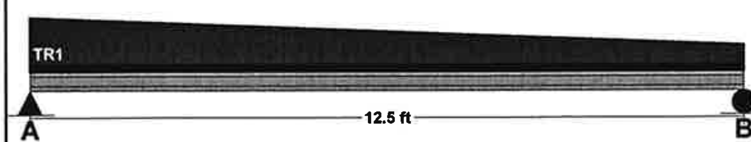
MATERIAL PROPERTIES
1.9E Microllam - iLevel Trus Joist

	Base Values		Adjusted	
Bending Stress:	Fb =	2600 psi	Fb' =	2604 psi
	Cd=1.00 CF=1.00			
Shear Stress:	Fv =	285 psi	Fv' =	285 psi
	Cd=1.00			
Modulus of Elasticity:	E =	1900 ksi	E' =	1900 ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	750 psi	Fc - ⊥' =	750 psi

Controlling Moment: 3264 ft-lb
5.5 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: 1032 lb
At a distance d from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Comparisons with required sections:	Req'd	Provided
Section Modulus:	15.04 in3	82.26 in3
Area (Shear):	5.43 in2	41.56 in2
Moment of Inertia (deflection):	114.56 in4	488.41 in4
Moment:	3264 ft-lb	17848 ft-lb
Shear:	1032 lb	7897 lb

LOADING DIAGRAM



UNIFORM LOADS	Center
Uniform Live Load	0 plf
Uniform Dead Load	0 plf
Beam Self Weight	13 plf
Total Uniform Load	13 plf

TRAPEZOIDAL LOADS - CENTER SPAN	
Load Number	One
Left Live Load	161 plf
Left Dead Load	105 plf
Right Live Load	23 plf
Right Dead Load	15 plf
Load Start	0 ft
Load End	12.5 ft
Load Length	12.5 ft

Project: 2018-2072

Location: RB10
Multi-Loaded Multi-Span Beam

(2) 1.75 IN x 11.25 IN x 17.0 FT
1.9E Microllam - iLevel Trus Joist
Section Adequate By: 89.8%
Controlling Factor: Deflection

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CAUTIONS
* Laminations are to be fully connected to provide uniform transfer of loads to all members

DEFLECTIONS	Center	
Live Load	0.34	IN L/594
Dead Load	0.25	in
Total Load	0.60	IN L/342

Live Load Deflection Criteria: L/240 Total Load Deflection Criteria: L/180

REACTIONS	A		B	
Live Load	1632	lb	816	lb
Dead Load	1170	lb	637	lb
Total Load	2802	lb	1453	lb
Bearing Length	1.07	in	0.55	in

BEAM DATA	Center	
Span Length	17	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	17	ft
Live Load Duration Factor	1.15	
Notch Depth	0.00	

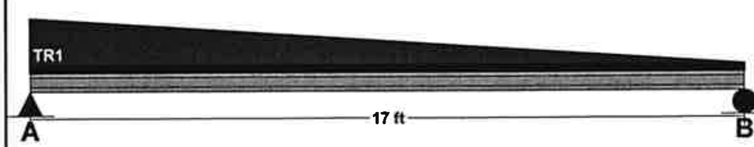
MATERIAL PROPERTIES
1.9E Microllam - iLevel Trus Joist

	Base Values			Adjusted		
Bending Stress:	Fb =	2600	psi	Fb' =	3016	psi
	Cd=1.15 CF=1.01					
Shear Stress:	Fv =	285	psi	Fv' =	328	psi
	Cd=1.15					
Modulus of Elasticity:	E =	1900	ksi	E' =	1900	ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	750	psi	Fc - ⊥' =	750	psi

Controlling Moment: 9258 ft-lb
7.31 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: 2397 lb
At a distance d from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Comparisons with required sections:	Req'd	Provided
Section Modulus:	36.83 in3	73.83 in3
Area (Shear):	10.97 in2	39.38 in2
Moment of Inertia (deflection):	218.76 in4	415.28 in4
Moment:	9258 ft-lb	18558 ft-lb
Shear:	2397 lb	8603 lb

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	0	plf
Uniform Dead Load	0	plf
Beam Self Weight	12	plf
Total Uniform Load	12	plf

TRAPEZOIDAL LOADS - CENTER SPAN	
Load Number	One
Left Live Load	288 plf
Left Dead Load	188 plf
Right Live Load	0 plf
Right Dead Load	0 plf
Load Start	0 ft
Load End	17 ft
Load Length	17 ft

Project: 2018-2072

Location: RB11
Multi-Loaded Multi-Span Beam

A992-50 W10x49 x 29.0 FT (17 + 12)
Section Adequate By: 18.2%
Controlling Factor: Deflection

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DEFLECTIONS	Center			Right	
Live Load	-0.04	IN	L/4566	0.40	IN 2L/718
Dead Load	-0.03	in		0.28	in
Total Load	-0.07	IN	L/2871	0.68	IN 2L/426

Live Load Deflection Criteria: L/480 Total Load Deflection Criteria: L/360

REACTIONS	A		B	
Live Load	196	lb	4892	lb
Dead Load	431	lb	4175	lb
Total Load	627	lb	9067	lb
Bearing Length	1.06	in	1.06	in

BEAM DATA	Center		Right	
Span Length	17	ft	12	ft
Unbraced Length-Top	0	ft	0	ft
Unbraced Length-Bottom	17	ft	12	ft

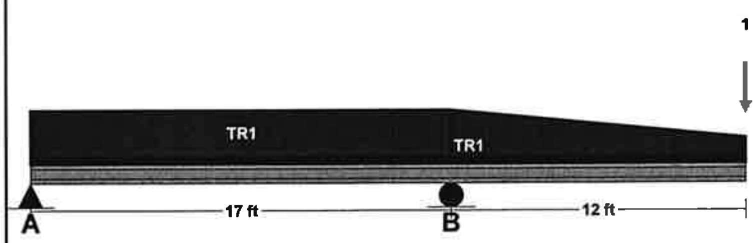
STEEL PROPERTIES
W10x49 - A992-50

Properties:			
Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	10	in
Web Thickness:	tw =	0.34	in
Flange Width:	bf =	10	in
Flange Thickness:	tf =	0.56	in
Distance to Web Toe of Fillet:	k =	1.06	in
Moment of Inertia About X-X Axis:	Ix =	272	in4
Section Modulus About X-X Axis:	Sx =	54.6	in3
Plastic Section Modulus About X-X Axis:	Zx =	60.4	in3
Design Properties per AISC 14th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	8.93	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	23.18	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	17	ft
Limiting Unbraced Length -			
for lateral-torsional buckling:	Lp =	8.97	ft
for Eqn. F2-2:	Lr =	31.63	ft
Nominal Flexural Strength w/ safety factor:	Mn =	131094	ft-lb
Controlling Equation:	F2-2		
Web height to thickness ratio:	h/tw =	23.18	
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95	
Cv Factor:	Cv =	1	
Controlling Equation:	G2-2		
Nominal Shear Strength w/ safety factor:	Vn =	68000	lb

Controlling Moment: -34860 ft-lb
Over right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2, 3
Controlling Shear: -4728 lb
At right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	230.13 in4	272 in4
Moment:	-34860 ft-lb	131094 ft-lb
Shear:	-4728 lb	68000 lb

LOADING DIAGRAM



UNIFORM LOADS	Center		Right	
Uniform Live Load	0	plf	0	plf
Uniform Dead Load	0	plf	0	plf
Beam Self Weight	49	plf	49	plf
Total Uniform Load	49	plf	49	plf

POINT LOADS - RIGHT SPAN	
Load Number	One *
Live Load	1247 lb
Dead Load	680 lb
Location	12 ft

* Load obtained from Load Tracker. See Summary Report for details.

TRAPEZOIDAL LOADS - CENTER SPAN	
Load Number	One
Left Live Load	161 plf
Left Dead Load	105 plf
Right Live Load	161 plf
Right Dead Load	105 plf
Load Start	0 ft
Load End	17 ft
Load Length	17 ft
RIGHT SPAN	
Load Number	One
Left Live Load	161 plf
Left Dead Load	105 plf
Right Live Load	23 plf
Right Dead Load	15 plf
Load Start	0 ft
Load End	12 ft
Load Length	12 ft

Project: 2018-2072

Location: RB12
Multi-Loaded Multi-Span Beam

(3) 1.75 IN x 11.875 IN x 21.0 FT (8.5 + 12.5)
1.9E Microllam - iLevel Trus Joist
Section Adequate By: 84.6%
Controlling Factor: Deflection

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CAUTIONS
* Laminations are to be fully connected to provide uniform transfer of loads to all members

DEFLECTIONS	Left		Center	
Live Load	0.39	IN 2L/528	-0.07	IN L/2073
Dead Load	0.23	in	-0.03	in
Total Load	0.61	IN 2L/332	-0.10	IN L/1533

Live Load Deflection Criteria: L/240 Total Load Deflection Criteria: L/180

REACTIONS	A	B
Live Load	2553 lb	431 lb
Dead Load	2008 lb	43 lb
Total Load	4561 lb	474 lb
Uplift (1.5 F.S)	**0 lb**	**-436 lb**
Bearing Length	1.16 in	0.12 in

BEAM DATA	Left	Center
Span Length	8.5 ft	12.5 ft
Unbraced Length-Top	0 ft	0 ft
Unbraced Length-Bottom	8.5 ft	12.5 ft
Live Load Duration Factor	1.15	
Notch Depth	0.00	

MATERIAL PROPERTIES
1.9E Microllam - iLevel Trus Joist

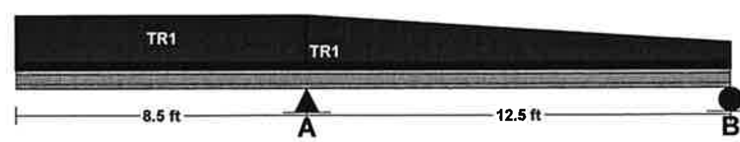
	Base Values		Adjusted	
Bending Stress:	Fb =	2600 psi	Fb' =	2930 psi
	Cd=1.15 Cl=0.98 CF=1.00			
Shear Stress:	Fv =	285 psi	Fv' =	328 psi
	Cd=1.15			
Modulus of Elasticity:	E =	1900 ksi	E' =	1900 ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	750 psi	Fc - ⊥' =	750 psi

Controlling Moment: -10313 ft-lb
Over left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 1, 2

Controlling Shear: -2160 lb
At a distance d from the right support of span 1 (Left Span)
Created by combining all dead loads and live loads on span(s) 1, 2

Comparisons with required sections:	Req'd	Provided
Section Modulus:	42.24 in3	123.39 in3
Area (Shear):	9.88 in2	62.34 in2
Moment of Inertia (deflection):	396.8 in4	732.62 in4
Moment:	-10313 ft-lb	30127 ft-lb
Shear:	-2160 lb	13622 lb

LOADING DIAGRAM



UNIFORM LOADS	Left	Center
Uniform Live Load	0 plf	0 plf
Uniform Dead Load	0 plf	0 plf
Beam Self Weight	19 plf	19 plf
Total Uniform Load	19 plf	19 plf

TRAPEZOIDAL LOADS - LEFT SPAN

Load Number	One
Left Live Load	161 plf
Left Dead Load	105 plf
Right Live Load	161 plf
Right Dead Load	105 plf
Load Start	0 ft
Load End	8.5 ft
Load Length	8.5 ft

CENTER SPAN

Load Number	One
Left Live Load	161 plf
Left Dead Load	105 plf
Right Live Load	23 plf
Right Dead Load	15 plf
Load Start	0 ft
Load End	12.5 ft
Load Length	12.5 ft

Project: 2018-2072

Location: RB13
Multi-Loaded Multi-Span Beam

A992-50 W10x49 x 29.0 FT (17 + 12)
Section Adequate By: 109.6%
Controlling Factor: Deflection

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DEFLECTIONS	Center		Right	
Live Load	-0.08	IN L/2520	0.49	IN 2L/590
Dead Load	-0.03	in	0.28	in
Total Load	-0.11	IN L/1919	0.76	IN 2L/378

Live Load Deflection Criteria: L/240 Total Load Deflection Criteria: L/180

REACTIONS	A	B
Live Load	1369 lb	4892 lb
Dead Load	431 lb	4175 lb
Total Load	1800 lb	9067 lb
Uplift (1.5 F.S)	**-885 lb**	**0 lb**
Bearing Length	1.06 in	1.06 in

BEAM DATA	Center	Right
Span Length	17 ft	12 ft
Unbraced Length-Top	0 ft	0 ft
Unbraced Length-Bottom	17 ft	12 ft

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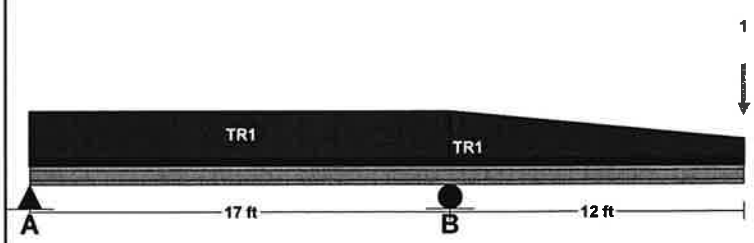

UNIFORM LOADS	Center	Right
Uniform Live Load	0 plf	0 plf
Uniform Dead Load	0 plf	0 plf
Beam Self Weight	49 plf	49 plf
Total Uniform Load	49 plf	49 plf

POINT LOADS - RIGHT SPAN

Load Number	One *
Live Load	1247 lb
Dead Load	680 lb
Location	12 ft

* Load obtained from Load Tracker. See Summary Report for details.

TRAPEZOIDAL LOADS - CENTER SPAN

Load Number	One
Left Live Load	161 plf
Left Dead Load	105 plf
Right Live Load	161 plf
Right Dead Load	105 plf
Load Start	0 ft
Load End	17 ft
Load Length	17 ft

RIGHT SPAN

Load Number	One
Left Live Load	161 plf
Left Dead Load	105 plf
Right Live Load	23 plf
Right Dead Load	15 plf
Load Start	0 ft
Load End	12 ft
Load Length	12 ft

STEEL PROPERTIES
W10x49 - A992-50

Properties:

Yield Stress:	Fy =	50 ksi
Modulus of Elasticity:	E =	29000 ksi
Depth:	d =	10 in
Web Thickness:	tw =	0.34 in
Flange Width:	bf =	10 in
Flange Thickness:	tf =	0.56 in
Distance to Web Toe of Fillet:	k =	1.06 in
Moment of Inertia About X-X Axis:	Ix =	272 in4
Section Modulus About X-X Axis:	Sx =	54.6 in3
Plastic Section Modulus About X-X Axis:	Zx =	60.4 in3

Design Properties per AISC 14th Edition Steel Manual:

Flange Buckling Ratio:	FBR =	8.93
Allowable Flange Buckling Ratio:	AFBR =	9.15
Web Buckling Ratio:	WBR =	23.18
Allowable Web Buckling Ratio:	AWBR =	90.55
Controlling Unbraced Length:	Lb =	17 ft
Limiting Unbraced Length -		
for lateral-torsional buckling:	Lp =	8.97 ft
for Eqn. F2-2:	Lr =	31.63 ft
Nominal Flexural Strength w/ safety factor:	Mn =	131094 ft-lb
Controlling Equation:	F2-2	
Web height to thickness ratio:	h/tw =	23.18
Limiting height to thickness ratio for eqn. G2-2:	h/tw-limit =	53.95
Cv Factor:	Cv =	1
Controlling Equation:	G2-2	
Nominal Shear Strength w/ safety factor:	Vn =	68000 lb

Controlling Moment: -34860 ft-lb
Over right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2, 3

Controlling Shear: -4728 lb
At right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	129.79 in4	272 in4
Moment:	-34860 ft-lb	131094 ft-lb
Shear:	-4728 lb	68000 lb

Shear Walls

Gridline 10
Front Covered Patio
Simpson Strong Wall
No anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic	Wind $TA_{Roof\text{-}end}$	Wind $TA_{Roof\text{-}int}$	Wind V_s	Seismic A_i	Seismic w_i	Seismic F	Seismic ρ	Seismic R	Seismic F_x	Seismic V_{final}	Check uplift DL	h	h'	Uplift $_w$	Uplift $_s$
panel 1							0	0	574	876	8270	1.000	1.300	6.5	1061	965					
panel 2																					
panel 3																					
panel 4																					
panel 5																					
panel 6																					
panel 7																					
panel 8																					
panel 9																					
panel 10																					
$ASW_{1,2}$=	0	0	Total=	0.00																	

$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$
42	0	574

$V_{additional}$ =

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
360 plf	530 plf	685 plf	895 plf	260 plf	350 plf	490 plf	640 plf
Total Resistance$_{wind}$				Total Resistance$_{seismic}$			
0	0	0	0	0	0	0	0

See the attached Simpson Strong Wall calculations

Perforated Shearwall 1:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline 9
Rear Covered Patio
Simpson Strong Wall
No anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic	Wind $TA_{Roof\text{-}end}$	Wind $TA_{Roof\text{-}int}$	Wind V_s	Seismic A_i	Seismic w_i	Seismic F	Seismic ρ	Seismic R	Seismic F_x	Seismic V_{final}	Check uplift DL	h	h'	Uplift $_w$	Uplift $_s$
panel 1							0	0	819	1909	16702.5	1.000	1.300	6.5	2142	1949					
panel 2																					
panel 3																					
panel 4																					
panel 5																					
panel 6																					
panel 7																					
panel 8																					
panel 9																					
panel 10																					
$ASW_{1,2}$=	0	0	Total=	0.00																	

$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$
60	0	2439

$V_{additional}$ = 1620 <= Parapet Load

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
360 plf	530 plf	685 plf	895 plf	260 plf	350 plf	490 plf	640 plf
Total Resistance$_{wind}$				Total Resistance$_{seismic}$			
0	0	0	0	0	0	0	0

See the attached Simpson Strong Wall calculations

Perforated Shearwall 1:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

SIMPSON STRONG-TIE COMPANY INC.
(800) 999-5099
5956 W. Las Positas Blvd., Pleasanton, CA 94588.
www.strongtie.com

Job Name: 2020-2072
Wall Name: Gridlines 10
Application: Garage Front

Design Criteria:

* 2018 International Bldg Code
* Seismic R=6.5
* 2500 psi concrete
* ASD Design Shear = 965 lbs
* Shearwall Height = 12' to underside of top plates

Selected Strong-Wall® Panel Solution:

Model	Type	W (in)	H (in)	T (in)	Sill Anchor	End Anchor Bolts	Total Axial Load (lbs)	Actual Uplift (lbs)
WSW12x12	Wood	12	141.25	3.5	N/A	2 - 7/8"	4500	6206 lb
WSW12x12	Wood	12	141.25	3.5	N/A	2 - 7/8"	4500	6206 lb

Actual Shear & Drift Distribution:

Model	RR Relative Rigidity	Actual Shear (lbs)		Allowable Shear (lbs)		Actual / Allow Shear	Actual Drift (in)	Drift Limit (in)
WSW12x12	0.50	483	≤	485	OK	0.99	0.62	0.63
WSW12x12	0.50	483	≤	485	OK	0.99	0.62	0.63

Notes:

1. Strong-Wall Wood Shearwalls have been evaluated to the 2018 IBC/IRC.
   See www.strongtie.com for additional design and installation information.
2. Anchor templates are recommended for proper anchor bolt placement, and are required in some jurisdictions.
3. Check that wall height "H" plus curb height (above slab) will attain overall rough header opening height (top of driveway slab to bottom of header).
4. The applied vertical load shall be a concentric point load or a uniformly distributed load not exceeding the allowable vertical load. Alternatively, the load may be applied anywhere along the width of the panel if imposed by a continuous bearing vertical load transfer element such as a rimboard or beam.
   For eccentric axial loads applied directly to the panel, the allowable vertical load shall be divided by two.
5. Panels may be trimmed to a minimum height of 74 1/2".
6. 2 ply headers may be used with Strong-Wall Wood Shearwalls. Minimum 9 1/4 inch deep nominal header is required with header desigı by others.

Disclaimer:

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(800) 999-5099
5956 W. Las Positas Blvd., Pleasanton, CA 94588.
www.strongtie.com

SIMPSON Strong-Tie

Job Name: 2020-2072
Wall Name: Gridlines 9
Application: Garage Front

Design Criteria:

* 2018 International Bldg Code
* Seismic R=6.5
* 2500 psi concrete
* ASD Design Shear = 1949 lbs
* Shearwall Height = 12' to underside of top plates

Selected Strong-Wall® Panel Solution:

Model	Type	W (in)	H (in)	T (in)	Sill Anchor	End Anchor Bolts	Total Axial Load (lbs)	Actual Uplift (lbs)
WSW18x12	Wood	18	141.25	3.5	N/A	2 - 7/8"	4600	7574 lb
WSW18x12	Wood	18	141.25	3.5	N/A	2 - 7/8"	4600	7574 lb

Actual Shear & Drift Distribution:

Model	RR Relative Rigidity	Actual Shear (lbs)		Allowable Shear (lbs)	Actual / Allow Shear	Actual Drift (in)	Drift Limit (in)
WSW18x12	0.50	974	≤	1340 OK	0.73	0.42	0.63
WSW18x12	0.50	974	≤	1340 OK	0.73	0.42	0.63

Notes:

1. Strong-Wall Wood Shearwalls have been evaluated to the 2018 IBC/IRC.
   See www.strongtie.com for additional design and installation information.
2. Anchor templates are recommended for proper anchor bolt placement, and are required in some jurisdictions.
3. Check that wall height "H" plus curb height (above slab) will attain overall rough header opening height (top of driveway slab to bottom of header).
4. The applied vertical load shall be a concentric point load or a uniformly distributed load not exceeding the allowable vertical load. Alternatively, the load may be applied anywhere along the width of the panel if imposed by a continuous bearing vertical load transfer element such as a rimboard or beam.
   For eccentric axial loads applied directly to the panel, the allowable vertical load shall be divided by two.
5. Panels may be trimmed to a minimum height of 74 1/2".
6. 2 ply headers may be used with Strong-Wall Wood Shearwalls. Minimum 9 1/4 inch deep nominal header is required with header desigi by others.

Disclaimer:

It is the Designer's responsibility to verify product suitability under applicable building codes. In order to verify code listed applications please refer to the appropriate product code reports at www.strongtie.com or contact Simpson Strong-Tie Company Inc. at 1-800-999-5099.

Shear Walls

Gridline 7.8
Front Dining

Structural Sheathing

5/8" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1	23		0.6 :1	1.00	269	95
panel 2	5		2.8 :1	0.71	192	68
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	28	**28**	Total=	26.57		

Wind

$TA_{Roof-end}$	$TA_{Roof-int}$	V_s
0	0	1537
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
0	170	**7140**
	$V_{additional}$ =	5508

<= Parapet Load

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
2822	28185	1.000	1.000	6.5	3615	**2530**
					$V_{additional}$ =	

Check uplift

DL	h	h'	$Uplift_w$	$Uplift_s$
100	14	14	2612	183
100	14	14	2437	702

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
360 plf	530 plf	685 plf	895 plf	260 plf	350 plf	490 plf	640 plf
Total $Resistance_{wind}$				Total $Resistance_{seismic}$			
9566	14083	18201	23781	6909	9300	13020	17006
Use SW1				Use SW1			

Use SW1
Use STHD10/10RJ holdowns each side of panel as noted on plans
Use 5/8" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline 6
Rear Dining

Structural Sheathing

5/8" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1	9.83		0.8 :1	1.00	450	146
panel 2	5		1.6 :1	1.00	450	146
panel 3	14.5		1.0 :1	1.00	450	146
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	29.33	**29.33**	Total=	29.33		

Wind

$TA_{Roof-end}$	$TA_{Roof-int}$	V_s
0	0	3281
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
0	363	**13205**
	$V_{additional}$ =	9720

<= Parapet Load

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
5016	47560	1.000	1.000	6.5	6099	**4270**
					$V_{additional}$ =	

Check uplift

DL	h	h'	$Uplift_w$	$Uplift_s$
150	14	8	2864	427
150	14	8	3227	790
150	14	14	5216	950

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
360 plf	530 plf	685 plf	895 plf	260 plf	350 plf	490 plf	640 plf
Total $Resistance_{wind}$				Total $Resistance_{seismic}$			
10559	15545	20091	26250	7626	10266	14372	18771
Use SW2				Use SW1			

Use SW2
Use STHD14 or HDU5-SDS2.5 holdowns each side of panel as noted on plans
Use 5/8" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		NOT USED		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Shear Walls

Gridline 2
Rear Kitchen

Structural Sheathing

5/8" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1	15.5		0.8 :1	1.00	295	83
panel 2	16		0.8 :1	1.00	295	83
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	31.5	31.5	Total=	31.50		

Wind

$TA_{Roof-end}$	$TA_{Roof-int}$	V_s
0	0	2764
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
117	129	9298
	$V_{additional}$ =	6534

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
3024	29280	1.000	1.000	6.5	3755	2629
					$V_{additional}$ =	

<= Parapet Load

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
360 plf	530 plf	685 plf	895 plf	260 plf	350 plf	490 plf	640 plf
Total Resistance$_{wind}$				Total Resistance$_{seismic}$			
11340	16695	21578	28193	8190	11025	15435	20160
Use SW1				Use SW1			

Check uplift

DL	h	h'	Uplift $_w$	Uplift $_s$
100	12	12	2767	226
100	12	12	2742	201

Use SW1
Use STHD10/10RJ holdowns each side of panel as noted on plans
Use 5/8" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Total Length =				$t = v =$	0
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				$t = v =$	0
Total Length =					
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				$t = v =$	0
Total Length =					
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline B
Left Dining

Structural Sheathing

5/8" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1	4		3.5 :1	0.57	246	129
panel 2	4		3.5 :1	0.57	246	129
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	8	8	Total=	4.57		

Wind

$TA_{Roof-end}$	$TA_{Roof-int}$	V_s
0	0	888
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
65	0	1968
	$V_{additional}$ =	1080

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
690	8835	1.000	1.300	6.5	1133	1031
					$V_{additional}$ =	

<= Parapet Load

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
360 plf	530 plf	685 plf	895 plf	260 plf	350 plf	490 plf	640 plf
Total Resistance$_{wind}$				Total Resistance$_{seismic}$			
1646	2423	3131	4091	1189	1600	2240	2926
Use SW2				Use SW1			

Check uplift

DL	h	h'	Uplift $_w$	Uplift $_s$
100	14	14	3243	1604
100	14	14	3243	1604

Use SW2
Use STHD10/10RJ holdowns each side of panel as noted on plans
Use 5/8" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Total Length =				$t = v =$	0
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				$t = v =$	0
Total Length =					
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				$t = v =$	0
Total Length =					
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Shear Walls

Gridline C.5
Left Restrooms

Structural Sheathing

5/8" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1	12		1.0 :1	1.00	188	87
panel 2	13.5		0.9 :1	1.00	188	87
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	25.5	25.5	Total=	25.50		

Wind

$TA_{Roof-end}$	$TA_{Roof-int}$	V_s
0	0	1226
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
60	45	4790
	$V_{additional}$ =	3564

<= Parapet Load

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
2328	24720	1.000	1.000	6.5	3170	2219
					$V_{additional}$ =	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
360 plf	530 plf	685 plf	895 plf	260 plf	350 plf	490 plf	640 plf
Total $Resistance_{wind}$				Total $Resistance_{seismic}$			
9180	13515	17468	22823	6630	8925	12495	16320
Use SW1				Use SW1			

Check uplift

DL	h	h'	$Uplift_w$	$Uplift_s$
100	12	12	1654	444
100	12	12	1579	369

Use SW1
Use LSTHD8/8RJ holdowns each side of panel as noted on plans
Use 5/8" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1: NOT USED

Total Length =		V_{wind}	V_{seis}
Height =		0	0
Max opening height=		Ratio	
C_o =		h:w	2w/h
segment 1			
segment 2			
segment 3			
segment 4			
segment 5			
		Total=	0.00

$t = v =$	0
$Uplift_w$	$Uplift_s$
0	0

Perforated Shearwall 2: NOT USED

Total Length =		V_{wind}	V_{seis}
Height =		0	0
Max opening height=		Ratio	
C_o =		h:w	2w/h
segment 1			
segment 2			
segment 3			
		Total=	0.00

$t = v =$	0
$Uplift_w$	$Uplift_s$
0	0

Perforated Shearwall 3: NOT USED

Total Length =		V_{wind}	V_{seis}
Height =		0	0
Max opening height=		Ratio	
C_o =		h:w	2w/h
segment 1			
segment 2			
segment 3			
		Total=	0.00

$t = v =$	0
$Uplift_w$	$Uplift_s$
0	0

Gridline D.7
Right Corridor

Structural Sheathing

5/8" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1	44		0.3 :1	1.00	401	111
panel 2						
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	44	44	Total=	44.00		

Wind

$TA_{Roof-end}$	$TA_{Roof-int}$	V_s
0	0	3797
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
0	420	17640
	$V_{additional}$ =	13608

<= Parapet Load

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
5959	54292.5	1.000	1.000	6.5	6963	4874
					$V_{additional}$ =	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
360 plf	530 plf	685 plf	895 plf	260 plf	350 plf	490 plf	640 plf
Total $Resistance_{wind}$				Total $Resistance_{seismic}$			
15840	23320	30140	39380	11440	15400	21560	28160
Use SW2				Use SW1			

Check uplift

DL	h	h'	$Uplift_w$	$Uplift_s$
324	12	12	-2317	-5799

Use SW2
No Uplift
Use 5/8" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1: NOT USED

Total Length =		V_{wind}	V_{seis}
Height =		0	0
Max opening height=		Ratio	
C_o =		h:w	2w/h
segment 1			
segment 2			
segment 3			
segment 4			
segment 5			
		Total=	0.00

$t = v =$	0
$Uplift_w$	$Uplift_s$
0	0

Perforated Shearwall 2: NOT USED

Total Length =		V_{wind}	V_{seis}
Height =		0	0
Max opening height=		Ratio	
C_o =		h:w	2w/h
segment 1			
segment 2			
segment 3			
		Total=	0.00

$t = v =$	0
$Uplift_w$	$Uplift_s$
0	0

Perforated Shearwall 3: NOT USED

Total Length =		V_{wind}	V_{seis}
Height =		0	0
Max opening height=		Ratio	
C_o =		h:w	2w/h
segment 1			
segment 2			
segment 3			
		Total=	0.00

$t = v =$	0
$Uplift_w$	$Uplift_s$
0	0

Shear Walls

Gridline G.1
Right Kitchen

Structural Sheathing

5/8" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1	36.5		0.3 :1	1.00	129	79
panel 2						
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	36.5	36.5	Total=	36.50		

Wind

$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int}$	V_s
0	0	1950
$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\,min}$
70	110	4704
	$V_{additional}$ =	2754

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
3264	32280	1.000	1.000	6.5	4140	2898

<= Parapet Load

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
360 plf	530 plf	685 plf	895 plf	260 plf	350 plf	490 plf	640 plf
Total Resistance$_{wind}$				Total Resistance$_{seismic}$			
13140	19345	25003	32668	9490	12775	17885	23360
Use SW1				Use SW1			

Check uplift

DL	h	h'	Uplift $_w$	Uplift $_s$
100	12	12	-278	-872

Use SW1
No Uplift
Use 5/8" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Total Length =				$t = v =$	0
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				$t = v =$	0
Total Length =					
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				$t = v =$	0
Total Length =					
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline H
Right Dining

Structural Sheathing

5/8" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1						
panel 2						
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	0	0	Total=	0.00		

Wind

$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int}$	V_s
0	0	888
$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\,min}$
65	0	1968
	$V_{additional}$ =	1080

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
476	6450	1.000	1.300	6.5	827	753

<= Parapet Load

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
360 plf	530 plf	685 plf	895 plf	260 plf	350 plf	490 plf	640 plf
Total Resistance$_{wind}$				Total Resistance$_{seismic}$			
2058	3030	3916	5116	1486	2001	2801	3659
Use SW1				Use SW1			

Check uplift

DL	h	h'	Uplift $_w$	Uplift $_s$

Use SW1
Use STHD14/14RJ holdowns each side of panel as noted on plans
Use 5/8" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				$t = v =$	295
Total Length =	24.5				
Height =	14	1968	753	4131	1580
Max opening height=	12	Ratio			
C_o =	0.556	h:w	2w/h		
segment 1	6	2.3 :1	0.86		
segment 2	6	2.3 :1	0.86		
segment 3					
segment 4					
segment 5					
		Total=	5.72		

Perforated Shearwall 2: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				$t = v =$	0
Total Length =					
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3: NOT USED

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				$t = v =$	0
Total Length =					
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Horiz. Diaphragm

Gridline 10
Front Covered Patio
Simpson Strong Wall
Roof
(Unblocked)
Load Case 3

	Wind				Seismic							Diaphragm				
	Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
L =	13	0	0	574	876	8270	1.000	1.300	6.5	1061	**965**	574	**7**	238	OK	**Wind**
b =	78	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		965	**12**	170	OK	**Seismic**
$b_{Collector}$=	78	42	0	**574**												
O =	18		$V_{additional}$ =													
$O_{Perf.}$ =	18															

Deflection
OK

Z =	141	lb
Z' =	248	lb

Diaphragm Chord Force		
M =	3136.7	ft-lb
T =	40	lb

Diaphragm Collector Force		
T =	223	lb
$T_{Perf.}$ =	0	lb

Top Plate Splice		
N =	0.9	nails
Use	8	16d nails between splice points

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Gridline 9
Rear Covered Patio
Simpson Strong Wall
Roof
(Unblocked)
Load Case 3

	Wind				Seismic							Diaphragm				
	Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
L =	13	0	0	819	1909	16703	1.000	1.300	6.5	2142	**1949**	2439	**31**	238	OK	**Wind**
b =	78	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		1949	**25**	170	OK	**Seismic**
$b_{Collector}$=	78	60	0	**2439**												
O =	26		$V_{additional}$ =	1620												
$O_{Perf.}$ =	26															

Deflection
OK

Z =	141	lb
Z' =	248	lb

Diaphragm Chord Force		
M =	7928	ft-lb
T =	102	lb

Diaphragm Collector Force		
T =	813	lb
$T_{Perf.}$ =	0	lb

Top Plate Splice		
N =	3.3	nails
Use	8	16d nails between splice points

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Gridline 7.8
Front Dining
Structural Sheathing
Roof
(Unblocked)
Load Case 3

	Wind				Seismic							Diaphragm				
	Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
L =	24	0	0	1537	2822	28185	1.000	1.000	6.5	3615	**2530**	7140	**162**	238	OK	**Wind**
b =	44	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		2530	**58**	170	OK	**Seismic**
$b_{Collector}$=	44	0	170	**7140**												
O =	0		$V_{additional}$ =	5508												
$O_{Perf.}$ =	0															

Deflection
OK

Z =	141	lb
Z' =	248	lb

Diaphragm Chord Force		
M =	42840	ft-lb
T =	974	lb

Diaphragm Collector Force		
T =	0	lb
$T_{Perf.}$ =	0	lb

Top Plate Splice		
N =	3.9	nails
Use	8	16d nails between splice points

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Horiz. Diaphragm

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 6	L =	54.5	0	0	3281	5016	47560	1.000	1.000	6.5	6099	**4270**	13205	**220**	238	**OK**	**Wind**
Rear Dining	b =	60	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$					$V_{additional}$ =			4270	**71**	170	**OK**	**Seismic**
	$b_{Collector}$=	60	0	363	**13205**								**Deflection**				
Structural Sheathing	O =	0	$V_{additional}$ =		9720					Z =	141 lb		**OK**				
	$O_{Perf.}$ =	0								Z' =	248 lb						

Roof (Unblocked) Load Case 3

Diaphragm Chord Force	
M =	179915 ft-lb
T =	2999 lb

Diaphragm Collector Force	
T =	0 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	12.1	nails
Use	**13**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 2	L =	54.5	0	0	2764	3024	29280	1.000	1.000	6.5	3755	**2629**	9298	**295**	322	**OK**	**Wind**
Rear Kitchen	b =	31.5	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$					$V_{additional}$ =			2629	**83**	230	**OK**	**Seismic**
	$b_{Collector}$=	31.5	117	129	**9298**								**Deflection**				
Structural Sheathing	O =	0	$V_{additional}$ =		6534					Z =	141 lb		**OK**				
	$O_{Perf.}$ =	0								Z' =	248 lb						

Roof (Unblocked) Load Case 1

Diaphragm Chord Force	
M =	126684 ft-lb
T =	4022 lb

Diaphragm Collector Force	
T =	0 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	16.2	nails
Use	**17**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline B	L =	13.5	0	0	888	690	8835	1.000	1.300	6.5	1133	**1031**	1968	**74**	322	**OK**	**Wind**
Left Dining	b =	26.5	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$					$V_{additional}$ =			1031	**39**	230	**OK**	**Seismic**
	$b_{Collector}$=	26.5	65	0	**1968**								**Deflection**				
Structural Sheathing	O =	18.5	$V_{additional}$ =		1080					Z =	141 lb		**OK**				
	$O_{Perf.}$ =	18.5								Z' =	248 lb						

Roof (Unblocked) Load Case 1

Diaphragm Chord Force	
M =	6640.9 ft-lb
T =	251 lb

Diaphragm Collector Force	
T =	1374 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	5.5	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Horiz. Diaphragm

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline C.5	L =	24.5	0	0	1226	2328	24720	1.000	1.000	6.5	3170	**2219**	4790	**188**	238	**OK**	**Wind**
Left Restrooms	b =	25.5	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		2219	**87**	170	**OK**	**Seismic**
	$b_{Collector}$=	25.5	60	45	**4790**								**Deflection**				
Structural Sheathing	O =	0		$V_{additional}$ =	3564					Z =	141 lb		**OK**				
	$O_{Perf.}$ =	0								Z' =	248 lb						

Roof
(Unblocked)
Load Case 3

Diaphragm Chord Force	
M =	29340 ft-lb
T =	1151 lb

Diaphragm Collector Force	
T =	0 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	4.6	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline D.7	L =	34	0	0	3797	5959	54293	1.000	1.000	6.5	6963	**4874**	17640	**226**	238	**OK**	**Wind**
Right Corridor	b =	78	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		4874	**62**	170	**OK**	**Seismic**
	$b_{Collector}$=	78	0	420	**17640**								**Deflection**				
Structural Sheathing	O =	0		$V_{additional}$ =	13608					Z =	141 lb		**OK**				
	$O_{Perf.}$ =	0								Z' =	248 lb						

Roof
(Unblocked)
Load Case 3

Diaphragm Chord Force	
M =	149940 ft-lb
T =	1922 lb

Diaphragm Collector Force	
T =	0 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	7.7	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline G.1	L =	34	0	0	1950	3264	32280	1.000	1.000	6.5	4140	**2898**	4704	**127**	238	**OK**	**Wind**
Right Kitchen	b =	37	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		2898	**78**	170	**OK**	**Seismic**
	$b_{Collector}$=	37	70	110	**4704**								**Deflection**				
Structural Sheathing	O =	0		$V_{additional}$ =	2754					Z =	141 lb		**OK**				
	$O_{Perf.}$ =	0								Z' =	248 lb						

Roof
(Unblocked)
Load Case 3

Diaphragm Chord Force	
M =	39987 ft-lb
T =	1081 lb

Diaphragm Collector Force	
T =	0 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	4.4	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Horiz. Diaphragm

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline H	L =	14	0	0	888	476	6450	1.000	1.300	6.5	827	753	1968	80	238	OK	Wind
Right Dining	b =	24.5	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		753	31	170	OK	Seismic
	$b_{Collector}$=	24.5	65	0	1968								Deflection				
Structural Sheathing	O =	12.5		$V_{additional}$ =	1080					Z =	141 lb		OK				
	$O_{Perf.}$ =	12.5								Z' =	248 lb						

Roof
(Unblocked)
Load Case 3

Diaphragm Chord Force	
M =	6886.9 ft-lb
T =	281 lb

Diaphragm Collector Force	
T =	1004 lb
$T_{Perf.}$ =	3688 lb

Top Plate Splice		
N =	14.9	nails
Use	15	16d nails between splice points

Use (4) 16d common toenails at full height truss blocking

Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

STUD WALL CALCULATION **Dining**

Wall Location =	Exterior	
Species =	DF-L #2	
Stud Width =	1.5	in
Stud Depth (d_x) =	5.5	in
L =	14	ft
stud spacing =	1.33	ft
F_b =	900	psi
F_c =	1350	psi
$F_{c\perp}$ =	625	psi
E =	1600000	psi
E_{min} =	580000	psi
C_F =	1.00	for bending
C_F =	1.00	for comp. II to grain
A =	8.25	in^2
S =	7.56	in^3
I =	20.80	in^4

Dead Loads:

Roof DL =	210	plf
Floor DL =	0	plf
w_{DL} =	350	plf

Live Loads:

Roof LL =	323.4	plf
Floor LL =	0	plf
W_{LL}=	323.40	

Load Case 1: Gravity Loads Only

Load Combinations:

D =	466	lbs
D+L =	466	lbs
D+S =	896	lbs
D+0.75(L)+0.75(S) =	788	lbs
C_D (D) =	0.9	
C_D (D+L) =	1	
C_D (D+S) =	1.15	
C_D (D+0.75(L)+0.75(S)) =	1.15	
$f_c = f_{c\perp}$ =	108.6	psi
$(l_e/d)_x$ =	29.7	in
E'_{min} =	580000	psi
c =	0.8	
F_{cE} =	539.5	
F^*_c =	1553	psi
F_{cE}/F^*_c =	0.348	psi
$(1+F_{cE}/F^*_c)/2c$ =	0.842	
C_p =	0.318	
F'_c =	493.5	
Check =	**OK**	psi

Bearing of stud on wall plates:

C_b =	1.25	
$F'_{c\perp}$ =	781	
Check =	**OK**	psi

Loadings

Roofing Material =	Shingle/Tile	
Roof Pitch =	0.25	
Angle =	1.2	
C_S =	1.000	
Increase for Drift=	1.000	
Effective snow load =	23	psf
Roof dead load =	15	psf
Floor live load =	40	psf
Floor dead load =	15	psf
Trib. Area $_{roof}$=	14	ft
Trib. Area $_{floor}$=	0	ft
Add. Uniform Load =	140	plf
Lateral Load =	14.43	psf

Use: 2x6 DF-L #2 Grade @ 16" o.c.

Deflection:

Allowable Deflection =	L/360	
Allowable Deflection =	0.45	in
Deflection =	0.26	in
Check =	**OK**	

Load Case 2: Gravity Loads + Lateral Loads

C_D =	1.6	
C_r =	1.35	
w =	19.2	plf
M =	5345.7	in.lb
f_b =	706.9	psi
F'_b =	1944.00	psi
Check =	**OK**	

Axial:

(l_e/d_x) =	29.7	in
E'_{min} =	580000	psi
c =	0.8	
F_{cE} =	539.5	psi
F^*_c =	2160	psi
F_{cE}/F^*_c =	0.250	
$(1+F_{cE}/F^*_c)/2c$ =	0.781	
C_p =	0.235	
F'_c =	508.2	psi

	D+0.75(W)+0.75(L)+0.75(S)	D+W	
f_c =	95.5	56.4	psi
Check =	OK	OK	
Combined Stress:			
F_{cEx} =	539.5	539.5	psi
Interaction Formula =	0.37	0.42	
Check =	OK	OK	

STUD WALL CALCULATION **Kitchen**

Wall Location =	Exterior	
Species =	DF-L #2	
Stud Width =	1.5	in
Stud Depth (d_x) =	5.5	in
L =	12	ft
stud spacing =	1.33	ft
F_b =	900	psi
F_c =	1350	psi
$F_{c\perp}$ =	625	psi
E =	1600000	psi
E_{min} =	580000	psi
C_F =	1.00	for bending
C_F =	1.00	for comp. II to grain
A =	8.25	in^2
S =	7.56	in^3
I =	20.80	in^4
Dead Loads:		
Roof DL =	307.5	plf
Floor DL =	0	plf
w_{DL} =	447.5	plf
Live Loads:		
Roof LL =	688.2	plf
Floor LL =	0	plf
W_{LL}=	688.17	
Load Case 1: Gravity Loads Only		
Load Combinations:		
D =	595	lbs
D+L =	595	lbs
D+S =	1510	lbs
D+0.75(L)+0.75(S) =	1282	lbs
C_D (D) =	0.9	
C_D (D+L) =	1	
C_D (D+S) =	1.15	
C_D (D+0.75(L)+0.75(S)) =	1.15	
$f_c = f_{c\perp}$ =	183.1	psi
$(l_e/d)_x$ =	25.4	in
E'_{min} =	580000	psi
c =	0.8	
F_{cE} =	741.1	
F^*_c =	1553	psi
F_{cE}/F^*_c =	0.477	psi
$(1+F_{cE}/F^*_c)/2c$ =	0.923	
C_p =	0.418	
F'_c =	648.2	
Check =	**OK**	psi
Bearing of stud on wall plates:		
C_b =	1.25	
$F'_{c\perp}$ =	781	
Check =	**OK**	psi

Loadings

Roofing Material =	Shingle/Tile	
Roof Pitch =	0.25	
Angle =	1.2	
C_S =	1.000	
Increase for Drift=	1.453	
Effective snow load =	34	psf
Roof dead load =	15	psf
Floor live load =	40	psf
Floor dead load =	15	psf
Trib. Area $_{roof}$=	20.5	ft
Trib. Area $_{floor}$=	0	ft
Add. Uniform Load =	140	plf
Lateral Load =	14.43	psf

Use: 2x6 DF-L #2 Grade @ 16" o.c.

Deflection:

Allowable Deflection =	L/360	
Allowable Deflection =	0.39	in
Deflection =	0.14	in
Check =	**OK**	

Load Case 2: Gravity Loads + Lateral Loads

C_D =	1.6	
C_r =	1.35	
w =	19.2	plf
M =	3891.5	in.lb
f_b =	514.6	psi
F'_b =	1944.00	psi
Check =	**OK**	
Axial:		
(l_e/d_x) =	25.4	in
E'_{min} =	580000	psi
c =	0.8	
F_{cE} =	741.1	psi
F^*_c =	2160	psi
F_{cE}/F^*_c =	0.343	
$(1+F_{cE}/F^*_c)/2c$ =	0.839	
C_p =	0.314	
F'_c =	678.9	psi

	D+0.75(W)+0.75(L)+0.75(S)	D+W	
f_c =	155.3	72.1	psi
Check =	OK	OK	
Combined Stress:			
F_{cEx} =	741.1	741.1	psi
Interaction Formula =	0.30	0.30	
Check =	OK	OK	

STUD WALL CALCULATION Corridor

Wall Location =	Interior	
Species =	DF-L #2	
Stud Width =	1.5	in
Stud Depth (d_x) =	5.5	in
L =	14	ft
stud spacing =	1.33	ft
F_b =	900	psi
F_c =	1350	psi
$F_{c\perp}$ =	625	psi
E =	1600000	psi
E_{min} =	580000	psi
C_F =	1.00	for bending
C_F =	1.00	for comp. II to grain
A =	8.25	in^2
S =	7.56	in^3
I =	20.80	in^4
Dead Loads:		
Roof DL =	435	plf
Floor DL =	0	plf
w_{DL} =	575	plf
Live Loads:		
Roof LL =	812.3	plf
Floor LL =	0	plf
W_{LL}=	812.33	
Load Case 1: Gravity Loads Only		
Load Combinations:		
D =	765	lbs
D+L =	765	lbs
D+S =	1845	lbs
D+0.75(L)+0.75(S) =	1575	lbs
C_D (D) =	0.9	
C_D (D+L) =	1	
C_D (D+S) =	1.15	
C_D (D+0.75(L)+0.75(S)) =	1.15	
$f_c = f_{c\perp}$ =	223.7	psi
$(l_e/d)_x$ =	29.7	in
E'_{min} =	580000	psi
c =	0.8	
F_{cE} =	539.5	
F^*_c =	1553	psi
F_{cE}/F^*_c =	0.348	psi
$(1+F_{cE}/F^*_c)/2c$ =	0.842	
C_p =	0.318	
F'_c =	493.5	
Check =	OK	psi
Bearing of stud on wall plates:		
C_b =	1.25	
$F'_{c\perp}$ =	781	
Check =	OK	psi

Loadings

Roofing Material =	Shingle/Tile	
Roof Pitch =	0.25	
Angle =	1.2	
C_S =	1.000	
Increase for Drift=	1.213	
Effective snow load =	28	psf
Roof dead load =	15	psf
Floor live load =	40	psf
Floor dead load =	15	psf
Trib. Area $_{roof}$=	29	ft
Trib. Area $_{floor}$=	0	ft
Add. Uniform Load =	140	plf
Lateral Load =	5.00	psf

Use: 2x6 DF-L #2 Grade @ 16" o.c.

Deflection:		
Allowable Deflection =	L/360	
Allowable Deflection =	0.45	in
Deflection =	0.11	in
Check =	OK	

Load Case 2: Gravity Loads + Lateral Loads

C_D =	1.6	
C_r =	1.15	
w =	6.7	plf
M =	1851.8	in.lb
f_b =	244.9	psi
F'_b =	1656.00	psi
Check =	OK	
Axial:		
(l_e/d_x) =	29.7	in
E'_{min} =	580000	psi
c =	0.8	
F_{cE} =	539.5	psi
F^*_c =	2160	psi
F_{cE}/F^*_c =	0.250	
$(1+F_{cE}/F^*_c)/2c$ =	0.781	
C_p =	0.235	
F'_c =	508.2	psi

	D+0.75(W)+0.75(L)+0.75(S)	D+W	
f_c =	190.9	92.7	psi
Check =	OK	OK	
Combined Stress:			
F_{cEx} =	539.5	539.5	psi
Interaction Formula =	0.31	0.21	
Check =	OK	OK	

KING STUD CALCULATION

Species =	DF-L #2	
Stud Width =	6	in
Stud Depth (d_x) =	5.5	in
L =	14	ft
opening width (OOP) =	18	ft
max. gravity span	18	ft
F_b =	900	psi
F_c =	1350	psi
$F_{c\perp}$ =	625	psi
E =	1600000	psi
E_{min} =	580000	psi
C_F =	1.00	for bending
C_F =	1.00	for comp. II to grain
A =	33	in^2
S =	30.25	in^3
I =	83.19	in^4
Dead Loads:		
Roof DL =	45	plf
Floor DL =	0	plf
w_{DL} =	45	plf
Live Loads:		
Roof LL =	69.3	plf
Floor LL =	0	plf
W_{LL}=	69.30	
Load Case 1: Gravity Loads Only		
Load Combinations:		
D =	435	lbs
D+L =	435	lbs
D+S =	1105	lbs
D+0.75(L)+0.75(S) =	938	lbs
C_D (D) =	0.9	
C_D (D+L) =	1	
C_D (D+S) =	1.15	
C_D (D+0.75(L)+0.75(S)) =	1.15	
$f_c = f_{c\perp}$ =	33.5	psi
$(l_e/d)_x$ =	29.7	in
E'_{min} =	580000	psi
c =	0.8	
F_{cE} =	539.5	
F^*_c =	1553	psi
F_{cE}/F^*_c =	0.348	psi
$(1+F_{cE}/F^*_c)/2c$ =	0.842	
C_p =	0.318	
F'_c =	493.5	
Check =	**OK**	psi
Bearing of stud on wall plates:		
C_b =	1.06	
$F'_{c\perp}$ =	664	
Check =	**OK**	psi

<u>**Loadings**</u>

Roofing Material =	Shingle/Tile	
Roof Pitch =	0.25	
Angle =	1.2	
C_S =	1.000	
Increase for Drift=	1.000	
Effective snow load =	23	psf
Roof dead load =	15	psf
Floor live load =	40	psf
Floor dead load =	15	psf
Trib. Area $_{roof}$=	3	ft
Trib. Area $_{floor}$=	0	ft
Add. Uniform Load =	0	plf
Lateral Load =	14.43	psf

Use: (2) 2x6 Full Height King Studs

Deflection:		
Allowable Deflection =	**L/175**	
Allowable Deflection =	0.93	in
Deflection =	0.47	in
Check =	**OK**	
Load Case 2: Gravity Loads + Lateral Loads		
C_D =	1.6	
C_r =	1.15	
w =	139.6	plf
M =	38867.2	in.lb
f_b =	1284.9	psi
F'_b =	1656.00	psi
Check =	**OK**	
Axial:		
(l_e/d_x) =	29.7	in
E'_{min} =	580000	psi
c =	0.8	
F_{cE} =	539.5	psi
F^*_c =	2160	psi
F_{cE}/F^*_c =	0.250	
$(1+F_{cE}/F^*_c)/2c$ =	0.781	
C_p =	0.235	
F'_c =	508.2	psi

	D+0.75(W)+0.75(L)+0.75(S)	D+W	
f_c =	28.4	13.2	psi
Check =	**OK**	**OK**	
Combined Stress:			
F_{cEx} =	539.5	539.5	psi
Interaction Formula =	0.62	0.80	
Check =	**OK**	**OK**	

Footing(s)

	FT1	FT2
Width of footing (in)=	20	18
Depth of footing (in)=	10	10
Height of wall (in)=	48	0
Width of wall (in)=	8	8
Roofing Material =	Shingle/Tile	Shingle/Tile
Roof Pitch=	0.25	0.25
Angle=	1.2	1.2
C_S=	1.000	1.000
Increase for Drift/Valley=	1.453	1.213
Effective snow load (psf)=	34	28
Roof dead load (psf)=	15	15
Floor live load (psf)=	40	40
Floor dead load (psf)=	15	15
Trib. $Area_{ROOF}$=	20.5	29
Trib. $Area_{FLOOR2}$=	0	0
Trib. $Area_{FLOOR1}$=	0	0
w_S (plf)=	688	812
w_L (plf)=	0	0
w_D (plf)=	307.5	435
$w_{CONC.}$ (plf)=	608	188
$w_{ADDITIONAL}$ (plf)=	140	140
w_{TOTAL} (plf)=	1744	1575
Req.Soil Bearing (psf)=	**1046**	**1050**
Footing Reinforcement:	**(2) #4 bars cont.**	**(2) #4 bars cont.**
Crosswise Reinforcement:	**None**	**None**

Post Calculations

	2"-4" Thick	5"x5"and Larger			
	Timber DF-L#2	Timber DF-L#2	Parallam	Glulam Comb #4	
F_c	1350	700	2500	2100	psi
F_{bx}	900	750	2400	1900	psi
F_{by}	900	750	2400	2200	psi
E_x	1600000	1300000	1800000	1900000	psi
E_y	1600000	1300000	1800000	1900000	psi

Example Calculations:

	lb	ft	ft	ft	in	in												
Post	Max P	l	le_x	le_y	e_x	e_y	C_d	$(le/d)_x$	$(le/d)_y$	A	S_x	S_y	f_c	F'c	F'_{bx}	F'_{by}	Comb.	Check
(2) 2x4	3725	8	8	1	0.61	0.00	1.15	27.4	4.0	10.5	6	5	355	582	1551	1708	0.6	OK
(2) 2x6	8990	8	8	1	0.96	0.00	1.15	17.5	4.0	16.5	15	8	545	1013	1344	1547	0.7	OK
(3) 2x4	5805	8	8	1	0.61	0.00	1.15	27.4	2.7	15.75	9	12	369	582	1785	1964	0.6	OK
(3) 2x6	14295	8	8	1	0.96	0.00	1.15	17.5	2.7	24.75	23	19	578	1019	1547	1779	0.7	OK
(4) 2x4	7745	8	8	1	0.61	0.00	1.15	27.4	2.0	21	12	21	369	582	1785	1964	0.6	OK
(4) 2x6	19080	8	8	1	0.96	0.00	1.15	17.5	2.0	33	30	33	578	1022	1547	1779	0.7	OK
(5) 2x4	9680	8	8	1	0.61	0.00	1.15	27.4	1.6	26.25	15	33	369	582	1785	1964	0.6	OK
(5) 2x6	23860	8	8	1	0.96	0.00	1.15	17.5	1.6	41.25	38	52	578	1023	1547	1779	0.7	OK
4x4	4340	8	8	1	0.61	0.00	1.15	27.4	3.4	12.25	7	7	354	571	1034	1035	0.7	OK
6x6	11200	8	8	1	0.96	0.00	1.15	17.5	2.2	30.25	28	28	370	663	862	863	0.8	OK
3 1/2" x 3 1/2" PLP	7440	8	1	8	0.00	0.61	1.15	3.4	27.4	12.25	7	7	607	953	3171	3174	1.0	OK
3 1/2" x 5 1/4" PLP	11035	8	1	8	0.00	0.61	1.15	2.3	27.4	18.375	16	11	601	953	3032	3036	1.0	OK
5 1/4" x 5 1/4" PLP	27915	8	1	8	0.00	0.92	1.15	2.3	18.3	27.563	24	24	1013	1889	3034	3036	1.0	OK
3 1/8" x 7 1/2" GLP	11495	8	1	8	0.00	0.55	1.15	1.6	30.7	23.438	29	12	490	802	2181	2935	0.9	OK
3 1/8" x 9" GLP	13790	8	1	8	0.00	0.55	1.15	1.3	30.7	28.125	42	15	490	802	2180	2935	0.9	OK
5 1/8" x 6" GLP	26595	8	1	8	0.00	0.90	1.15	2.0	18.7	30.75	31	26	865	1773	2184	2783	0.8	OK
5 1/8" x 7 1/2" GLP	33240	8	1	8	0.00	0.90	1.15	1.6	18.7	38.438	48	33	865	1773	2184	2783	0.8	OK
5 1/8" x 9" GLP	39890	8	1	8	0.00	0.90	1.15	1.3	18.7	46.125	69	39	865	1773	2183	2783	0.8	OK

Additional Post Calculations:

	0	8	8	8	0.61	0.61	1.15	27.4	27.4	12.25	7	7	0	571	1031	1035	0.0	OK
	0	8	8	8	0.61	0.61	1.15	27.4	27.4	12.25	7	7	0	571	1031	1035	0.0	OK
	0	8	8	8	0.61	0.61	1.15	27.4	27.4	12.25	7	7	0	571	1031	1035	0.0	OK

Load Charts:

Braced in One Direction

	Roof Loads				Floor Loads			
	7 ft	8 ft	9 ft	10 ft	7 ft	8 ft	9 ft	10 ft
(1) 2 x 4	2215	1855	1570	1340	2100	1775	1505	1290
(1) 2 x 6	5150	4630	4140	3695	4695	4270	3855	3470
(2) 2 x 4	4450	3725	3150	2690	4215	3560	3025	2595
(2) 2 x 6	9535	8990	8325	7430	8500	8080	7615	6970
(3) 2 x 4	6960	5805	4890	4160	6620	5560	4710	4025
(3) 2 x 6	15165	14295	13180	11720	13510	12845	12105	11020
(4) 2 x 4	9290	7745	6520	5550	8830	7415	6280	5365
(4) 2 x 6	20245	19080	17580	15630	18035	17145	16155	14700
(5) 2 x 4	11615	9680	8150	6935	11035	9265	7850	6710
(5) 2 x 6	25320	23860	21980	19535	22555	21440	20200	18375

Unbraced in Both Directions

	Roof Loads				Floor Loads			
	7 ft	8 ft	9 ft	10 ft	7 ft	8 ft	9 ft	10 ft
(2) 2 x 4	2905	2350	1930	1605	2800	2285	1885	1575
(2) 2 x 6	4670	3775	3095	2570	4500	3670	3025	2525
(3) 2 x 4	6605	5590	4750	4065	6205	5310	4550	3915
(3) 2 x 6	11575	9985	8575	7380	10745	9405	8170	7090
(4) 2 x 4	9290	7745	6520	5550	8830	7415	6280	5365
(4) 2 x 6	18155	16500	14830	13245	16425	15120	13760	12425
(5) 2 x 4	11615	9680	8150	6935	11035	9265	7850	6710
(5) 2 x 6	23935	22215	20425	18635	21465	20125	18695	17235
4 x 4	5185	4340	3670	3135	4915	4145	3525	3025
6 x 6	12040	11200	10330	9460	10790	10130	9430	8720
3 1/2" x 3 1/2" PLP	9000	7440	6225	5270	8595	7155	6015	5115
3 1/2" x 5 1/4" PLP	13330	11035	9245	7840	12720	10600	8930	7600
5 1/4" x 5 1/4" PLP	31850	27915	24355	21295	29340	26080	23000	20250
3 1/8" x 7 1/2"	13795	11495	9680	8245	13115	11005	9320	7970
3 1/8" x 9"	16555	13790	11620	9895	15735	13205	11185	9565
5 1/8" x 6"	29565	26595	23720	21095	26900	24510	22110	19840
5 1/8" x 7 1/2"	36955	33240	29650	26370	33625	30640	27640	24805
5 1/8" x 9"	44350	39890	35580	31645	40350	36765	33170	29765

Notes:
1. Example calculations show posts braced in one direction.
2. Loads have been adjusted to accommodate for the worst case of the following eccentric conditions: .175 of column thickness or .175 of column width.

Project: 2018-2072

Location: P8
Multi-Loaded Multi-Span Beam

HSS 5-1/2 x 5-1/2 x 3/8 x 14.0 FT (12 + 2) / ASTM A500-GR.B-46
Section Adequate By: 0.8%
Controlling Factor: Deflection

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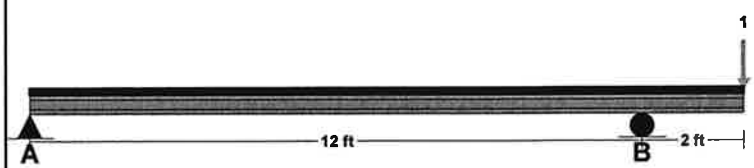
DEFLECTIONS	Center		Right	
Live Load	-0.39	IN L/367	0.40	IN 2L/120
Dead Load	0.01	in	-0.01	in
Total Load	-0.38	IN L/378	0.39	IN 2L/122

Live Load Deflection Criteria: L/120 Total Load Deflection Criteria: L/120

REACTIONS	A		B	
Live Load	-1767	lb	12367	lb
Dead Load	147	lb	206	lb
Total Load	-1620	lb	12573	lb
Uplift (1.5 F.S)	**-1669**	**lb**	**0**	**lb**
Bearing Length	0.70	in	1.19	in

BEAM DATA	Center		Right	
Span Length	12	ft	2	ft
Unbraced Length-Top	0	ft	0	ft
Unbraced Length-Bottom	12	ft	2	ft

LOADING DIAGRAM



UNIFORM LOADS	Center		Right	
Uniform Live Load	0	plf	0	plf
Uniform Dead Load	0	plf	0	plf
Beam Self Weight	25	plf	25	plf
Total Uniform Load	25	plf	25	plf

POINT LOADS - RIGHT SPAN	
Load Number	One
Live Load	10600 lb
Dead Load	0 lb
Location	2 ft

STEEL PROPERTIES

HSS 5-1/2 x 5-1/2 x 3/8 - A500-GR.B-46

Properties:

Steel Yield Strength:	Fy =	46	ksi
Modulus of Elasticity:	E =	29000	ksi
Tube Steel Section (X Axis):	dx =	5.5	in
Tube Steel Section (Y Axis):	dy =	5.5	in
Tube Steel Wall Thickness:	t =	0.349	in
Area:	A =	6.88	in2
Moment of Inertia (X Axis):	Ix =	29.7	in4
Section Modulus (X Axis):	Sx =	10.8	in3
Plastic Section Modulus (X Axis):	Zx =	13.1	in3
Plastic Section Modulus (Y Axis):	Zy =	13.1	in3

Design Properties per AISC 14th Edition Steel Manual:

Flange Buckling Ratio:	FBR =	12.76	
Allowable Flange Buckling Ratio:	AFBR =	28.12	
Allowable Flange Buckling Ratio non-compact:	AFBR_NC =	35.15	
Web Buckling Ratio:	WBR =	12.76	
Allowable Web Buckling Ratio for Eqn. F7-5:	AWBR =	60.76	
Nominal Flexural Strength w/ Safety Factor:	Mn =	30070	ft-lb
Controlling Equation:	F7-1		

Controlling Moment: -21250 ft-lb
Over right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2, 3
Controlling Shear: 10650 lb
At left support of span 3 (Right Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	29.47 in4	29.7 in4
Moment:	-21250 ft-lb	30070 ft-lb
Shear:	10650 lb	57191 lb

Project: 2018-2072

Location: P9
Column

HSS 4 x 4 x 1/4 x 12.0 FT /ASTM A500-GR.B-46
Section Adequate By: 50.9%

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VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	15299	lb
Dead Load:	Vert-DL-Rxn =	9633	lb
Total Load:	Vert-TL-Rxn =	24932	lb

COLUMN DATA

Total Column Length:	12	ft
Unbraced Length (X-Axis) Lx:	12	ft
Unbraced Length (Y-Axis) Ly:	12	ft
Column End Condition-K (e):	1	

COLUMN PROPERTIES

HSS 4 x 4 x 1/4 - Square

Steel Yield Strength:	Fy =	46 ksi		
Modulus of Elasticity:	E =	29000 ksi		
Column Section:	dx =	4 in	dy =	4 in
Column Wall Thickness:	t =	0.233 in		
Area:	A =	3.37 in		
Moment of Inertia (deflection):	Ix =	7.8 in4	Iy =	7.8 in4
Section Modulus:	Sx =	3.9 in3	Sy =	3.9 in3
Plastic Section Modulus:	Zx =	4.69 in3	Zy =	4.69 in3
Rad. of Gyration:	rx =	1.52 in	ry =	1.52 in

Column Compression Calculations:

KL/r Ratio: KLx/rx = 94.74 KLy/ry = 94.74

Controlling Direction for Compr. Calcs: (Y-Y Axis)

Flexural Buckling Stress: Fcr = 25.15 ksi

Controlling Equation F7-1

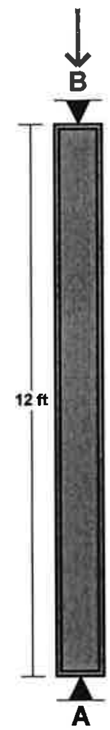
Nominal Compressive Strength: Pc = 51 kip

Combined Stress Calculations:

H1-1a Controls : 0.49

Controlling Combined Stress Factor: 0.49

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	15299 lb
Dead Load:	PD =	9485 lb
Column Self Weight:	CSW =	148 lb
Total Axial Load:	PT =	24932 lb

Project: 2018-2072

Location: P10
Multi-Loaded Multi-Span Beam

HSS 5 x 5 x 1/4 x 14.0 FT (12 + 2) / ASTM A500-GR.B-46
Section Adequate By: 47.5%
Controlling Factor: Deflection

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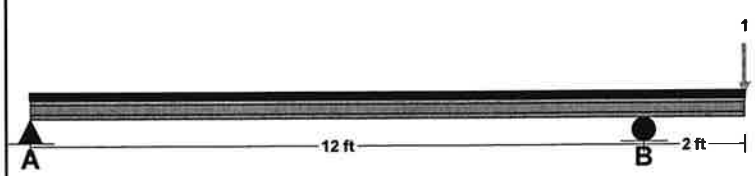
DEFLECTIONS	Center			Right		
Live Load	-0.27	IN	L/537	0.27	IN	2L/178
Dead Load	0.01	in		-0.01	in	
Total Load	-0.25	IN	L/567	0.26	IN	2L/182

Live Load Deflection Criteria: L/120 Total Load Deflection Criteria: L/120

REACTIONS	A		B	
Live Load	-650	lb	4550	lb
Dead Load	92	lb	128	lb
Total Load	-558	lb	4678	lb
Uplift (1.5 F.S)	**-589**	**lb**	**0**	**lb**
Bearing Length	0.47	in	0.66	in

BEAM DATA	Center		Right	
Span Length	12	ft	2	ft
Unbraced Length-Top	0	ft	0	ft
Unbraced Length-Bottom	12	ft	2	ft

LOADING DIAGRAM



UNIFORM LOADS	Center		Right	
Uniform Live Load	0	plf	0	plf
Uniform Dead Load	0	plf	0	plf
Beam Self Weight	16	plf	16	plf
Total Uniform Load	16	plf	16	plf

POINT LOADS - RIGHT SPAN	
Load Number	One
Live Load	3900 lb
Dead Load	0 lb
Location	2 ft

STEEL PROPERTIES

HSS 5 x 5 x 1/4 - A500-GR.B-46

Properties:

Steel Yield Strength:	Fy =	46	ksi
Modulus of Elasticity:	E =	29000	ksi
Tube Steel Section (X Axis):	dx =	5	in
Tube Steel Section (Y Axis):	dy =	5	in
Tube Steel Wall Thickness:	t =	0.233	in
Area:	A =	4.3	in2
Moment of Inertia (X Axis):	Ix =	16	in4
Section Modulus (X Axis):	Sx =	6.41	in3
Plastic Section Modulus (X Axis):	Zx =	7.61	in3
Plastic Section Modulus (Y Axis):	Zy =	7.61	in3

Design Properties per AISC 14th Edition Steel Manual:

Flange Buckling Ratio:	FBR =	18.46	
Allowable Flange Buckling Ratio:	AFBR =	28.12	
Allowable Flange Buckling Ratio non-compact:	AFBR_NC =	35.15	
Web Buckling Ratio:	WBR =	18.46	
Allowable Web Buckling Ratio for Eqn. F7-5:	AWBR =	60.76	
Nominal Flexural Strength w/ Safety Factor:	Mn =	17468	ft-lb
Controlling Equation:	F7-1		

Controlling Moment: -7831 ft-lb
Over right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2, 3

Controlling Shear: 3931 lb
At left support of span 3 (Right Span)
Created by combining all dead loads and live loads on span(s

Comparisons with required sections:	Req'd	Provided
Moment of Inertia (deflection):	10.84 in4	16 in4
Moment:	-7831 ft-lb	17468 ft-lb
Shear:	3931 lb	36878 lb

POST / SHEAR WALL / FOOTING / FOUNDATION WALL SCHEDULE

(not all are necessarily used)

Post Schedule

Designation	Post Size
P1	(1) 2x
P2	(2) 2x
P3	(3) 2x
P4	(4) 2x
P5	(5) 2x
P6	4x4
P7	6x6
P8	HSS 5 1/2x5 1/2x3/8 A500 Gr. B-46
P9	HSS 4x4x1/4 A500 Gr. B-46
P10	HSS 5x5x1/4 A500 Gr. B-46

Notes:
1. Posts indicate number of trimmer studs when specified at headers. All other post designations refer to full height king studs U.N.O.
2. Install (1) trimmer stud and (1) king stud each side of each opening U.N.O.
3. Install (2) trimmer studs each side of openings greater than 6'-0" U.N.O.
4. Install (2) king studs each side of openings greater than 8'-0" U.N.O.
5. 2x built-up posts shall be the same width of the wall in which they are framed U.N.O.
6. Nail each ply of 2x built-up posts w/ 16d nails @ 6" o.c. staggered U.N.O.
7. Posts that are not framed within a stud wall shall be braced with BC or AC post cap and PB or ABA post base U.N.O.

Shear Wall Schedule[1,3,4]

Designation	Material	1 1/2" 16 Gage Staples		8d Nails		Capacity		Note
		Edge	Field	Edge	Field	Wind	Seismic	
1	7/16" OSB or CDX plywood	3 1/2"	12"	6"	12"	360	260	2,5
2	7/16" OSB or CDX plywood	2" [6]	12"	4"	12"	530	350	2,5
3	7/16" OSB or CDX plywood	-	-	3"	12"	685	490	2,5,6
4	7/16" OSB or CDX plywood	-	-	2"	12"	895	640	2,5,6
S1	WSW12x12 Simpson Wood Strong Wall See Details13/S4.0, 15/S4.0, 30/S4.1, 12/S1.1, and 13/S1.1							
S2	WSW18x12 Simpson Wood Strong Wall See Details13/S4.0, 15/S4.0, 30/S4.1, 12/S1.1, and 13/S1.1							

Notes:
1. Wall studs are to be spaced at 16" o.c. U.N.O.
2. Sheath above and below openings in perforated shear walls as per the adjacent shear wall designation on each side of the opening.
3. Use (2) king studs at each end of shear panels (Shear Wall Chords) U.N.O.
4. All panel edges shall be blocked with 2x or wider framing with edge nailing at all supports and panel edges U.N.O.
5. Where panels are applied on both faces of a wall and nail spacing is less than 6" o.c. on either side, panel joints shall be offset to fall on different framing members.
6. Framing at adjoining panel edges and sill plates shall be 3x or wider for edge nailing 3" o.c. or less. Nails at adjoining panel edges and into sill plates shall be staggered. (Double 2x framing stitch-nailed with staggered 16d nails with spacing equal to the shear wall edge nailing is an adequate substitute for 3x framing.)
7. Fasteners for sheet rock shear walls shall be No. 6 Type S or W drywall screws 1-1/4" long in lieu of 8d nails.

Footing Schedule

Designation	Length	Width	Depth	Lengthwise Reinforcement				Crosswise Reinforcement				Capacity	Note
				Qty.	Size	Length	Spacing	Qty.	Size	Length	Spacing		
FT1	Cont.	20"	10"	2	#4	Cont.	EQ.	-	-	-	-	4167 PLF	
FT2	Cont.	18"	10"	2	#4	Cont.	EQ.	-	-	-	-	3750 PLF	See detail 10/S1.1
FT3	24"	24"	10"	3	#4	18"	EQ.	3	#4	18"	EQ.	10000 LBS	
FT4	30"	30"	10"	3	#4	24"	EQ.	3	#4	24"	EQ.	15625 LBS	
FT5	36"	36"	10"	4	#4	30"	EQ.	4	#4	30"	EQ.	22500 LBS	
FT6	42"	42"	10"	4	#4	36"	EQ.	4	#4	36"	EQ.	30625 LBS	
FT7	48"	48"	10"	5	#4	42"	EQ.	5	#4	42"	EQ.	40000 LBS	

Notes:
1. f'c= 2,500 psi, fy= 60,000 psi. No special inspection required.
2. Footings shall bear on undisturbed native soils or structural compacted fill (95% compaction), specified and tested by a registered geotechnical engineer.
3. All footings shall bear below the frost line of the locality. (30" U.N.O.) Provide 12" diameter sono-tube at exterior spot footings per detail 20/SD.1
4. Provide J-bars to match vertical foundation wall reinforcement with 24" minimum lap splice into foundation wall.
5. Center footing under foundation wall U.N.O.

Foundation Wall Schedule

Maximum Height	Reinforcement
3' Foundation Wall	#4 bars @ 24" o.c. vertical, (3) #4 bars horizontal

Notes:
1. Use 5/8" diameter x 14" long (7" embedment) anchor bolts @ 24" o.c. w/ 3"x3"x1/4" (0.229") plate washers at all exterior and shear walls U.N.O. (Edge of plate washer to be located within 1/2" of sheathed edge of sill plate.)
2. f'c= 3,000 psi, fy= 60,000 psi. No special inspection required.
3. Place (1) #4 bar below and on each side of each opening and (2) #4 bars above each opening. Bars shall be placed within 2" of the openings and extend 24" beyond the edge of the opening; vertical bars may terminate 3" from the top of the concrete. Opening reinforcement is in addition to standard wall reinforcement.
4. Top and bottom bars shall be within 4" of the top and bottom of the wall.
5. Place reinforcement in center of wall U.N.O.