Yehuda Residence

Structural Calculations

Engineer's seal applies to this entire calculation packet. This packet is void if binding seal is broken or if engineer's seal is not an original signature in red ink.

This engineering report is valid only for the aforementioned building located at Lot #65, Summit Powder Mountain Subdivision, Eden, 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

ENGINEERS

SURVEYORS

PLANNERS

3302 N. Main Street
Spanish Fork, UT 84660
Phone: 801.798.0555
Fax: 801.798.9393
office@lei-eng.com
www.lei-eng.com

LEI Project #:

2017-2259

Location:

Eden, Utah

Date:

7/20/2017

Engineered by:

K. Christensen

Reviewed by:

STATE OF UTAH
JOSHUA K.S. ANDERSON
No. 4939007-2203
08/14/2017
PROFESSIONAL STRUCTURAL ENGINEER

APPLIES TO PAGES 1-112

Structural Review for: **Yehuda Residence**
Location: **Eden, Utah**
Job #: **2017-2259**
Engineered by: **K. Christensen**
Code: **2015 IBC**

Loadings

Risk Category: II

Ground Snow Load:

Elevation = 8580 ft
County = Weber
A_o = 4.5
S = 63
P_o = 43
P_g = 260.6 psf

Roof Snow Load:

C_t = 1.1
Roof Exposure C_e = 0.9 Full
I = 1.0
P_f = 180.6 psf

Roof Dead Load:

DL = 15 psf

Floor Loadings:

Dead Load = 15 psf
Live Load = 40 psf

Wind Loading:

Roofing Material = Shingle/Tile
Roof Pitch = 0.5 /12
Roof Angle = 2.4 degrees

Exposure Category = **C**
Mean Roof Height = 25
Wind Speed V = 115

Height & Exposure Factor λ = 1.35

p_{s30} Horizontal Pressures				p_{net30}	
zone A	zone B	zone C	zone D	zone 4	zone 5
21.00	-10.90	13.90	-6.50	23.30	26.90

p_s Horizontal Pressures				p_{net}	
zone A	zone B	zone C	zone D	zone 4	zone 5
28.4	**0.0**	**18.8**	**0.0**	**31.5**	**36.3**

Seismic Loading:

Number of Stories = 2
Roof diaphragm height h_r = 25 ft
I_E = 1.00
Fundamental Period T_a = 0.224 sec.
F = 1.1
Site Class = D
R factor = 6.5 Structural Sheathing
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 = 8.0 Special Moment Frame
S_S = 0.813
S_1 = 0.27
F_a = 1.1748
F_v = . 1.86
S_{MS} = 0.9551124
S_{M1} = 0.5022
S_{DS} = 0.637
S_{D1} = 0.335
T_o = 0.1051604 sec.
T_s = 0.525802 sec.
Seismic Design Category = **D**

Soil Bearing Capacity: 1500 psf **Assumed**

Snow Drift Calculations

Roofing Material =	Shingle/Tile	
Ground Snow Load **p_g** =	241	psf
Flat Roof Snow Load **p_f** =	167	psf
Roof Pitch =	0.5	
Angle =	2	
C_s =	1.00	
Sloped Roof Snow Load **p_s** =	167	psf
λ =	30.00	
Height of normal Snow Load **h_b** =	5.58	ft

	Drift #1	**Not Used**
Roof Height Difference **h_c** (ft)=	20	0
Does Drift Exist (h_c/h_b < .2)?	**Yes**	**No**
Length of upper roof **l_u** (ft)=	52	0
Height of Drift **h_d** (ft)=	4.9	-1.5
w (ft)=	20	-6
Max drift width (ft)=	160	0
Drift tapers to zero @ **w** (ft)=	**20**	**-6**
Drift Load **p_d** (psf)=	147	0
Total load (psf)=	**314**	**167**

Siesmic Weight

Additional Seismic Weight	48.4	psf
Total Seismic Weight	**63.4**	psf

Preface & Structural Notes

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

Yehuda Residence

Lot #65, Summit Powder Mountain Subdivision, Eden, 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.

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 supercede.
3 This engineering report deals only with the structural parts of the building and does not provide liability to the non-structural parts.
4 If plans are stamped in conjunction with this engineering packet, certification pertains only to the structural elements of the plans.
5 The general contractor is responsible for the method, means, and sequence of all structural erection except when specifically noted otherwise on the drawings. He 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 connetions 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 be performed as per 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 The 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 project engineer is not responsible for the pre-engineered trusses, nor for the installation of the trusses.
12 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 with engineering specs.
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

Floor Joists:

FJ1: 11 7/8" TJI/210 @ 16" o.c. as noted on plans
FJ2: 11 7/8" TJI/560 @ 12" o.c. as noted on plans
3/4" APA rated T&G flooring to be nailed with 10d nails @ 6" o.c. edge, 12" o.c. field

Deck Joists:

DJ1: 2x8 DF-L#2 @ 16" o.c. as noted on plans
DJ2: 4x10 DF-L#2 @ 12" o.c. as noted on plans

Roof:

RR1: 11 7/8" TJI/360 @ 12" o.c. as noted on plans
Trusses by others
Use 7/8" APA rated OSB sheathing w/ 10d 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 6	Timber
RB2	2	2 x 10	Timber
RB3	3	2 x 10	Timber
RB4	1	W10x54	A992-50
RB5	1	5 1/8" x 27"	Glulam
RB6	1	W10x54	A992-50
RB7	1	W10x54	A992-50
RB8	1	1 3/4" x 11 7/8"	Microllam

Beam Schedule

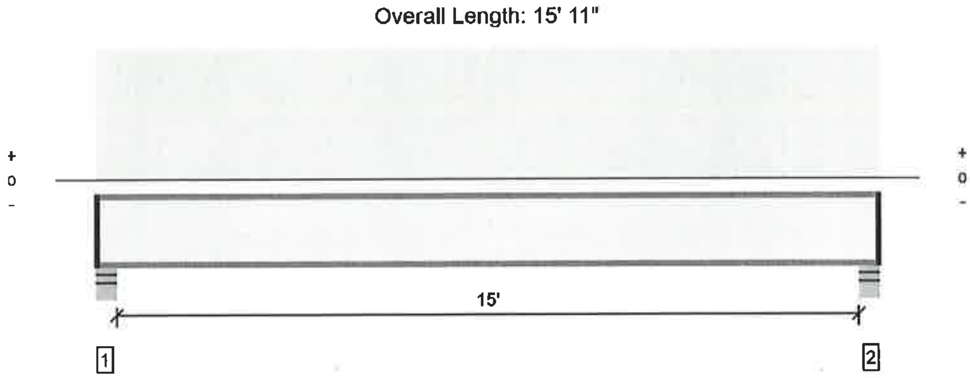
Desig.	Qty.	Size	Type
SB1	2	2 x 6	Timber
SB2	2	2 x 10	Timber
SB3	1	W8x48	A992-50
SB4	2	1 3/4" x 11 7/8"	Microllam
SB5	1	W10x19	A992-50
SB6	1	W10x49	A992-50
SB7	2	1 3/4" x 11 7/8"	Microllam
SB8	2	1 3/4" x 9 1/2"	Microllam
SB9	1	W8x48	A992-50
SB10	1	W8x48	A992-50

Beam Schedule

Desig.	Qty.	Size	Type
MB1	2	2 x 6	Timber
MB2	3	1 3/4" x 11 7/8"	Microllam
MB3	2	2 x 10	Timber
MB4	3	2 x 10	Timber
MB5	4	1 3/4" x 14"	Microllam
MB6	1	W8x15	A992-50
MB7	1	W10x54	A992-50
MB8	1	W10x54	A992-50
MB9	1	W10x54	A992-50

FORTE® **MEMBER REPORT** *Level, FJ1* **PASSED**

1 piece(s) 11 7/8" TJI® 210 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	576 @ 4 1/2"	1460 (3.50")	Passed (39%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	550 @ 5 1/2"	1655	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2109 @ 7' 11 1/2"	3795	Passed (56%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.192 @ 7' 11 1/2"	0.379	Passed (L/950)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.263 @ 7' 11 1/2"	0.758	Passed (L/691)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	48	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 4' 4 11/16" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	5.50"	4.25"	1.75"	159	424	583	1 1/4" Rim Board
2 - Stud wall - SPF	5.50"	4.25"	1.75"	159	424	583	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 15' 11"	16"	15.0	40.0	Residential - Living Areas

Weyerhaeuser Notes

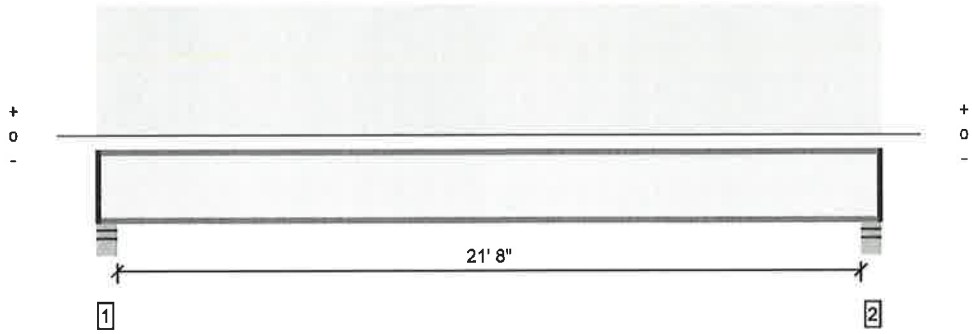
Forte Software Operator	Job Notes
Kelly Christensen LEI Consulting Engineers (801) 798-0555 kchristensen@lei-eng.com	Page 6 of 112

FORTE® **MEMBER REPORT** *Level, FJ2*

1 piece(s) 11 7/8" TJI® 560 @ 12" OC

PASSED

Overall Length: 22' 7"



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	615 @ 4 1/2"	1725 (3.50")	Passed (36%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	596 @ 5 1/2"	2050	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3277 @ 11' 3 1/2"	9500	Passed (34%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.327 @ 11' 3 1/2"	0.546	Passed (L/800)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.450 @ 11' 3 1/2"	1.092	Passed (L/582)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	43	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 8' 6" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	5.50"	4.25"	1.75"	169	452	621	1 1/4" Rim Board
2 - Stud wall - SPF	5.50"	4.25"	1.75"	169	452	621	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 22' 7"	12"	15.0	40.0	Residential - Living Areas

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by Forte Software Operator

Forte Software Operator	Job Notes
Kelly Christensen LEI Consulting Engineers (801) 798-0555 kchristensen@lei-eng.com	Page 7 of 112

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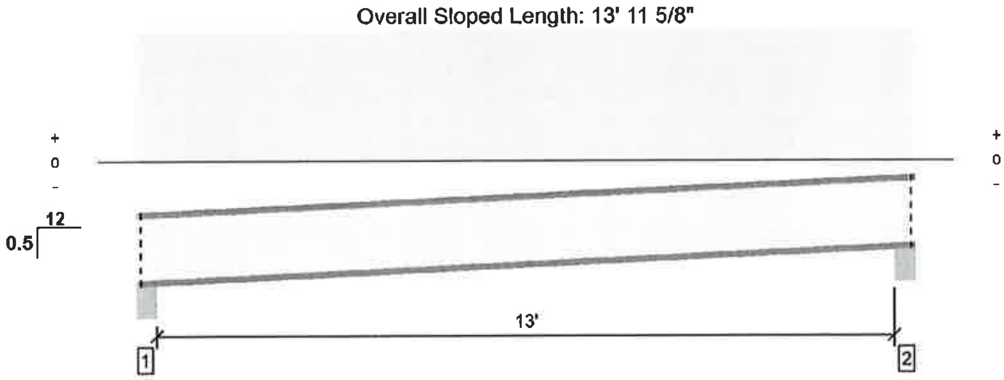
FORTE® **MEMBER REPORT** *Level, RR1* **PASSED**

1 piece(s) 11 7/8" TJI® 360 @ 12" OC



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1357 @ 4 1/2"	1731 (3.50")	Passed (78%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1268 @ 5 1/2"	1961	Passed (65%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	4226 @ 6' 11 1/2"	7107	Passed (59%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.361 @ 6' 11 1/2"	0.439	Passed (L/438)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.391 @ 6' 11 1/2"	0.659	Passed (L/404)	--	1.0 D + 1.0 S (All Spans)

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD
Member Pitch: 0.5/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Bracing (Lu): All compression edges (top and bottom) must be braced at 3' 11 13/16" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Beveled Plate - SPF	5.50"	5.50"	2.16"	104	1253	1357	Blocking
2 - Beveled Plate - SPF	5.50"	5.50"	2.16"	104	1253	1357	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 13' 11"	12"	15.0	180.0	Roof

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by Forte Software Operator

SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Kelly Christensen LEI Consulting Engineers (801) 798-0555 kchristensen@lei-eng.com	

6/5/2017 8:00:25 AM
Forte v5.1, Design Engine: V6.5.1.1
2017-2259.4te

Project: 2017-2259

Location: Outlookers
Roof Rafter
[2015 International Building Code(2012 NDS)]
TJI 210 / 11.875 - iLevel Trus Joist x 5.0 FT (2.5 + 2.5) @ 16 O.C.
Section Adequate By: 11.7%
Controlling Factor: End Reaction

Jack Miller
LEI Surveyors and Engineers
3302 North Main Street
Spanish Fork, Utah

page / of

StruCalc Version 10.0.1.4 6/2/2017 11:05:29 AM

DEFLECTIONS	Center		Right	
Live Load	-0.01	IN L/2213	0.01	IN 2L/4640
Dead Load	0.00	in	0.00	in
Total Load	-0.01	IN L/2043	0.01	IN 2L/4286

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

REACTIONS	B	
Live Load	1207	lb
Dead Load	100	lb
Total Load	1307	lb
Bearing Length	5.50	in
Web Stiffeners	No	

SUPPORT LOADS	B	
Live Load	905	plf
Dead Load	75	plf
Total Load	980	plf

I-JOIST PROPERTIES

TJI 210 / 11.875 - iLevel Trus Joist

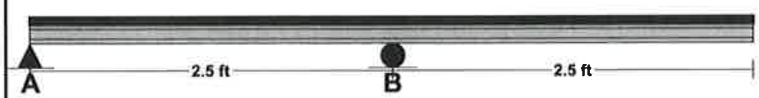
	Base Values	Adjusted
Moment Cap:	Mcap = 3795 ft-lb *Cd = 1.00*	Mcap' = 3795 ft-lb
Shear Stress:	Vcap = 1655 lb *Cd = 1.00*	Vcap' = 1655 lb
Reaction A:	Rcap = 0 lb	Rcap' = 0 lb
Reaction B:	Rcap = 1460 lb	Rcap' = 1460 lb
E.I.:	EI = 315 lb-in2	EI' = 315 lb-in2

Controlling Moment: -817 ft-lb
2.5 Ft from left support of span 3 (Right Span)
Created by combining all dead loads and live loads on span(s) 2, 3

Controlling Shear: 653 lb
0.0 Ft from left support of span 3 (Right Span)
Created by combining all dead loads and live loads on span(s) 2, 3

Comparisons with required sections:	Req'd	Provided
E.I.:	34 in2-lb E6	315 in2-lb xE6
Moment:	-817 ft-lb	3795 ft-lb
Shear:	653 lb	1655 lb

LOADING DIAGRAM



RAFTER DATA	Interior	Eave
Span Length	2.5 ft	2.5 ft
Rafter Pitch	0 :12	

Roof sheathing applied to top of joists-top of rafters fully braced.
Sheathing/sheetrock applied to bottom of joists-bottom of rafters fully braced.
Roof Duration Factor 1.00

RAFTER LOADING

Uniform Roof Loading

Roof Live Load:	LL =	181	psf
Roof Dead Load:	DL =	15	psf

Slope Adjusted Spans And Loads

Interior Span:	L-adj =	2.5	ft
Eave Span:	L-Eave-adj =	2.5	ft
Interior Live Load:	wL-adj =	241	plf
Eave Live Load:	wL-Eave-adj =	241	plf
Interior Dead Load:	wD-adj =	20	plf
Eave Dead Load:	wD-Eave-adj =	20	plf
Interior Total Load:	wT-adj =	261	plf
Eave Total Load:	wT-Eave-adj =	261	plf

NOTES

Project: 2017-2259

Location: Diag Outlooker
Multi-Loaded Multi-Span Beam
[2015 International Building Code(2012 NDS)]
1.75 IN x 11.875 IN x 7.08 FT (3.5 + 3.5)
1.9E Microllam - iLevel Trus Joist
Section Adequate By: 277.2%
Controlling Factor: Shear

Jack Miller
LEI Surveyors and Engineers
3302 North Main Street
Spanish Fork, Utah

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DEFLECTIONS	Center			Right		
Live Load	-0.01	IN	L/7507	0.05	IN	2L/1806
Dead Load	0.00	in		0.00	in	
Total Load	-0.01	IN	L/7165	0.05	IN	2L/1664

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	1068	lb	2670	lb
Dead Load	44	lb	267	lb
Total Load	1112	lb	2937	lb
Uplift (1.5 F.S)	**-504**	**lb**	**0**	**lb**
Bearing Length	0.85	in	2.24	in

BEAM DATA	Center		Right	
Span Length	3.54	ft	3.54	ft
Unbraced Length-Top	0	ft	0	ft
Unbraced Length-Bottom	3.54	ft	3.54	ft
Live Load Duration Factor	1.00			
Notch Depth	0.00			

MATERIAL PROPERTIES

1.9E Microllam - iLevel Trus Joist

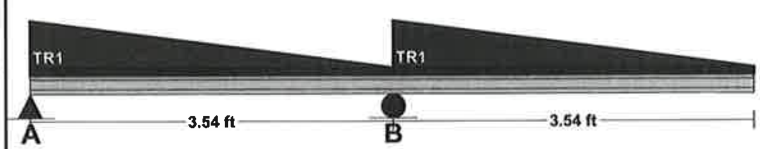
	Base Values			Adjusted		
Bending Stress:	Fb =	2600	psi	Fb' =	2346	psi
	Cd=1.00 Cl=0.90 CF=1.00					
Shear Stress:	Fv =	285	psi	Fv' =	285	psi
	Cd=1.00					
Modulus of Elasticity:	E =	1900	ksi	E' =	1900	ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	750	psi	Fc - ⊥' =	750	psi

Controlling Moment: -2088 ft-lb
Over right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 3

Controlling Shear: -1047 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:	10.68	in3	41.13	in3
Area (Shear):	5.51	in2	20.78	in2
Moment of Inertia (deflection):	48.68	in4	244.21	in4
Moment:	-2088	ft-lb	8042	ft-lb
Shear:	-1047	lb	3948	lb

LOADING DIAGRAM



UNIFORM LOADS	Center		Right	
Uniform Live Load	0	plf	0	plf
Uniform Dead Load	0	plf	0	plf
Beam Self Weight	6	plf	6	plf
Total Uniform Load	6	plf	6	plf

TRAPEZOIDAL LOADS - CENTER SPAN

Load Number	One
Left Live Load	905 plf
Left Dead Load	75 plf
Right Live Load	0 plf
Right Dead Load	0 plf
Load Start	0 ft
Load End	3.54 ft
Load Length	3.54 ft

RIGHT SPAN

Load Number	One
Left Live Load	905 plf
Left Dead Load	75 plf
Right Live Load	0 plf
Right Dead Load	0 plf
Load Start	0 ft
Load End	3.54 ft
Load Length	3.54 ft

NOTES

Project: 2017-2259

Location: DJ1
Multi-Loaded Multi-Span Beam
[2015 International Building Code(2012 NDS)]
1.5 IN x 7.25 IN x 5.0 FT (3.5 + 1.5)
#2 - Douglas-Fir-Larch - Dry Use
Section Adequate By: 104.2%
Controlling Factor: Shear

Jack Miller
LEI Surveyors and Engineers
3302 North Main Street
Spanish Fork, Utah

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DEFLECTIONS	Center		Right	
Live Load	0.01	IN L/3821	0.00	IN 2L/15632
Dead Load	0.00	in	0.00	in
Total Load	0.01	IN L/3636	0.00	IN 2L/15368

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	615	lb	1435	lb
Dead Load	32	lb	80	lb
Total Load	647	lb	1515	lb
Bearing Length	0.69	in	1.62	in

BEAM DATA	Center		Right	
Span Length	3.5	ft	1.5	ft
Unbraced Length-Top	0	ft	0	ft
Unbraced Length-Bottom	3.5	ft	1.5	ft
Live Load Duration Factor	1.00			
Notch Depth	0.00			

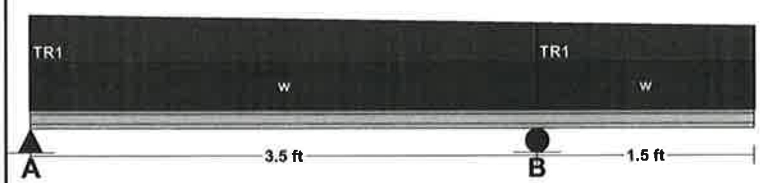
MATERIAL PROPERTIES
#2 - Douglas-Fir-Larch

	Base Values			Adjusted		
Bending Stress:	Fb =	900	psi	Fb' =	1043	psi
	Cd=1.00 Cl=0.97 CF=1.20					
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: -464 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: -639 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:	5.34 in3	13.14 in3
Area (Shear):	5.33 in2	10.88 in2
Moment of Inertia (deflection):	4.49 in4	47.63 in4
Moment:	-464 ft-lb	1142 ft-lb
Shear:	-639 lb	1305 lb

LOADING DIAGRAM



UNIFORM LOADS	Center		Right	
Uniform Live Load	240	plf	240	plf
Uniform Dead Load	20	plf	20	plf
Beam Self Weight	2	plf	2	plf
Total Uniform Load	262	plf	262	plf

TRAPEZOIDAL LOADS - CENTER SPAN

Load Number	One
Left Live Load	195 plf
Left Dead Load	0 plf
Right Live Load	160 plf
Right Dead Load	0 plf
Load Start	0 ft
Load End	3.5 ft
Load Length	3.5 ft

RIGHT SPAN

Load Number	One
Left Live Load	160 plf
Left Dead Load	0 plf
Right Live Load	145 plf
Right Dead Load	0 plf
Load Start	0 ft
Load End	1.5 ft
Load Length	1.5 ft

NOTES

Project: 2017-2259

Location: DJ2
Floor Joist
[2015 International Building Code(2012 NDS)]
3.5 IN x 9.25 IN x 13.0 FT @ 12 O.C.
#2 - Douglas-Fir-Larch - Dry Use
Section Adequate By: 3.2%
Controlling Factor: Deflection

Jack Miller
LEI Surveyors and Engineers
3302 North Main Street
Spanish Fork, Utah

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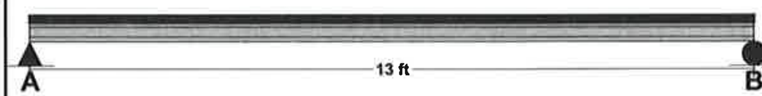
DEFLECTIONS	Center	
Live Load	0.31	IN L/495
Dead Load	0.03	in
Total Load	0.34	IN L/458

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

REACTIONS	A		B	
Live Load	1177	lb	1177	lb
Dead Load	98	lb	98	lb
Total Load	1275	lb	1275	lb
Bearing Length	0.58	in	0.58	in

SUPPORT LOADS	A		B	
Live Load	1177	plf	1177	plf
Dead Load	98	plf	98	plf
Total Load	1275	plf	1275	plf

LOADING DIAGRAM



MATERIAL PROPERTIES
#2 - Douglas-Fir-Larch

	Base Values			Adjusted		
Bending Stress:	Fb =	900	psi	Fb' =	1242	psi
	Cd=1.00 CF=1.20 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: 4141 ft-lb
6.5 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: 1147 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:	40 in3	49.91 in3
Area (Shear):	9.55 in2	32.38 in2
Moment of Inertia (deflection):	223.65 in4	230.84 in4
Moment:	4141 ft-lb	5166 ft-lb
Shear:	1147 lb	3885 lb

JOIST DATA	Center	
Span Length	13	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	0	ft

Floor sheathing applied to top of joists-top of joists fully braced.
Floor Duration Factor 1.00

JOIST LOADING

Uniform Floor Loading		Center	
Live Load	LL =	181	psf
Dead Load	DL =	15	psf
Total Load	TL =	196	psf
TL Adj. For Joist Spacing	wT =	196	plf

NOTES

Ledger L1 Calculations

Loads/Reactions	**Roof**		**Floor**	
Dead load:	15	psf	15	psf
Live load:	181	psf	40	psf
Increase for drift:	1.508			
Effective snow load:	272	psf		
Span length of rafter/truss/joist:	3.5	ft	0	ft
Roof rafter/truss/joist spacing:	1.33	ft	1.33	ft
Uniform load on rafter/truss/joist:	**382.2**	plf	**73.2**	plf
End reaction on rafter/truss/joist:	**668.8**	lbs	**0.0**	lbs
Ledger loading:	**502.8**	plf	**0.0**	plf
Additional uniform load:	0	plf		
Final ledger loading:	**502.8**	plf		

Number of Required Screws		
SDWS22400DB Wood Screw	250	(per Simpson)
C_D=	1.00	
SDWS22400DB Wood Screw	250	lb
Number of required screws:	**2.0**	screws/ft
Spacing:	1	ft
Required screws at specified spacing:	**2.0**	

Use 2 SDWS22400DB Wood Screws minimum at 12" o.c.

Use 2x8 Ledger

Beams	RB1	RB2	RB3	RB4	RB5	RB5 - OP	RB6	RB7	RB8
Roofing material =	Shingle/Tile	Shingle/Tile	Shingle/Tile		Shingle/Tile	Shingle/Tile			
Roof Pitch=	0.5	0.5	0.5		0.5	0.5			
Angle=	2.4	2.4	2.4		2.4	2.4			
C_S =	1.000	1.000	1.000		1.000	1.000			
Increase for Drift/Valley=	1.000	1.000	1.000		1.000	1.000			
Effective snow load (psf)=	181	181	181		181	181			
Roof dead load (psf)=	15	15	15		15	15			
Floor live load (psf)=	40	40	40		40	40			
Floor dead load (psf)=	15	15	15		15	15			
Length (ft)=	3	3	3		17.5	17.5			
Trib. Area $_{roof}$=	5.5	9	15		13.5	0			
Trib. Area $_{floor}$=	0	0	0		0	0			
w_S (plf) =	993	1625	2709		2438	0			
w_L (plf) =	0	0	0		0	0			
w_D (plf) =	86	141	234		235	169			
$w_{self\ weight}$ (plf) =	3.6	6.0	9.0		33.0	33.0			
Point Load (lb)=									
a (ft)=	1.5	1.5	1.5		8.75	8.75			
b (ft)=	1.5	1.5	1.5		8.75	8.75			
Add. uniform load (plf)=						136			
Allowable Live Deflection =	L/240	L/240	L/240		L/240	L/180			
Allowable Total Deflection =	L/180	L/180	L/180		L/180	L/120			
Left Reaction (lb)=	1619	2650	4415		23394	1479			
Right Reaction (lb)=	1619	2650	4415		23394	1479			
V_{max} (lb)=	1619	2650	4415		23394	1479			
Location of M_{max} (ft)=	1.5	1.5	1.5		8.8	8.8			
M_{max} (ftlb)=	1214	1987	3311	See attached calculations	102350	6469	See attached calculations	See attached calculations	See attached calculations
Size Factor (C_F)=	1.30	1.10	1.10		1.00	1.00			
Volume Factor (C_v)=	1.00	1.00	1.00		0.94	0.94			
Duration Factor (C_d) =	1.00	1.00	1.00		1.00	1.00			
Beam Type (t,g,m,p,ts,rb)	t	t	t		g	g			
d (in)=	5.5	9.25	9.25		27	5.125			
b (in)=	3	3	4.5		**5.125**	27			
$F'_{c\perp}$ =	625	625	625		650	650			
Bearing Width (in)=	3	3	4.5		5.125	27			
Req'd Bearing Length (in)=	0.86	1.41	**1.57**		**7.02**	0.08			
I (in^4)=	41.6	197.9	296.8		8406.3	302.9			
F_b	900	900	900		2400	2400			
F'_b=	1170	990	990		2254	2254			
S (in^3)=	15.1	42.8	64.2		622.7	118.2			
S_{req}=	12	24	40		545	34			
	Section OK	**Section OK**	**Section OK**		**Section OK**	**Section OK**			
E (psi)=	1600000	1600000	1600000		1800000	1800000			
F'_v (psi)=	180	180	180		265	265			
f_v (psi)=	147	143	159		254	16			
	Shear OK	**Shear OK**	**Shear OK**		**Shear OK**	**Shear OK**			
Location of Max Deflection=	1.50	1.50	1.50		8.75	8.75			
Allowable Live Deflection (in)=	0.150	0.150	0.150		0.875	1.167			
Live Deflection (in)=	0.027	0.009	0.010		0.340	0.000			
Allowable Total Deflection (in)=	0.200	0.200	0.200		1.167	1.750			
Total Deflection (in)=	0.030	0.010	0.011		0.373	0.654			
	Deflection OK	**Deflection OK**	**Deflection OK**		**Deflection OK**	**Deflection OK**			

Project: 2017-2259

Location: RB8
Multi-Loaded Multi-Span Beam
[2015 International Building Code(2012 NDS)]
1.75 IN x 11.875 IN x 10.0 FT (7 + 3)
1.9E Microllam - iLevel Trus Joist
Section Adequate By: 506.1%
Controlling Factor: Moment

Kelly Christensen
LEI Consulting Engineers and Surveyers Inc.
3302 N. Main St.
Spanish Fork, UT 84660

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DEFLECTIONS	Center		Right	
Live Load	0.01	IN L/7082	0.00	IN 2L/MAX
Dead Load	0.00	in	0.00	in
Total Load	0.01	IN L/6326	0.00	IN 2L/MAX

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

REACTIONS	A		B	
Live Load	514	lb	1286	lb
Dead Load	61	lb	154	lb
Total Load	575	lb	1440	lb
Bearing Length	0.44	in	1.10	in

BEAM DATA	Center		Right	
Span Length	7	ft	3	ft
Unbraced Length-Top	0	ft	0	ft
Unbraced Length-Bottom	7	ft	3	ft
Live Load Duration Factor	1.00			
Notch Depth	0.00			

LOADING DIAGRAM

UNIFORM LOADS	Center		Right	
Uniform Live Load	180	plf	180	plf
Uniform Dead Load	15	plf	15	plf
Beam Self Weight	6	plf	6	plf
Total Uniform Load	201	plf	201	plf

MATERIAL PROPERTIES

1.9E Microllam - iLevel Trus Joist

	Base Values			Adjusted		
Bending Stress:	Fb =	2600	psi	Fb' =	1604	psi
	Cd=1.00 Cl=0.62 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: -907 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: -637 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:	6.79 in3	41.13 in3
Area (Shear):	3.35 in2	20.78 in2
Moment of Inertia (deflection):	8.28 in4	244.21 in4
Moment:	-907 ft-lb	5496 ft-lb
Shear:	-637 lb	3948 lb

NOTES

Beams	SB1	SB2	SB3	SB4	SB5	SB6	SB7	SB8	SB9-SB10
Roofing material =	Shingle/Tile	Shingle/Tile		Shingle/Tile			Shingle/Tile	Shingle/Tile	
Roof Pitch=	0.5	0.5		0.5			0.5	0.5	
Angle=	2.4	2.4		2.4			2.4	2.4	
C_S =	1.000	1.000		1.000			1.000	1.000	
Increase for Drift/Valley=	1.000	1.000		1.000			1.000	1.000	
Effective snow load (psf)=	181	181		181			181	181	
Roof dead load (psf)=	15	15		15			15	15	
Floor live load (psf)=	40	40		40			40	40	
Floor dead load (psf)=	15	15		15			15	15	
Length (ft)=	3	6		12			9	7.75	
Trib. Area $_{roof}$=	6	3		0			0	5	
Trib. Area $_{floor}$=	2.5	1		12			12	1	
w_S (plf) =	1084	542		0			0	903	
w_L (plf) =	100	40		480			480	40	
w_D (plf) =	221	156		192			192	180	
$w_{self\ weight}$ (plf) =	3.6	6.0		12.1			12.1	9.7	
Point Load (lb)=									
a (ft)=	1.5	3		6			4.5	3.875	
b (ft)=	1.5	3		6			4.5	3.875	
Add. uniform load (plf)=	90	90						80	
Allowable Live Deflection =	L/480	L/480		L/480			L/480	L/480	
Allowable Total Deflection =	L/360	L/360		L/360			L/360	L/360	
Left Reaction (lb)=	1957	2093		4032			3024	4195	
Right Reaction (lb)=	1957	2093		4032			3024	4195	
V_{max} (lb)=	1957	2093		4032			3024	4195	
Location of M_{max} (ft)=	1.5	3.0	See attached calculations	6.0	See attached calculations	See attached calculations	4.5	3.9	See attached calculations
M_{max} (ftlb)=	1468	3140		12097			6805	8129	
Size Factor (C_F)=	1.30	1.10		1.00			1.00	1.00	
Volume Factor (C_v)=	1.00	1.00		1.00			1.00	1.03	
Duration Factor (C_d) =	1.00	1.00		1.00			1.00	1.00	
Beam Type (t,g,m,p,ts,rb)	t	t		m			m	m	
d (in)=	5.5	9.25		11.875			11.875	9.5	
b (in)=	3	3		3.5			3.5	3.5	
$F'_{c\perp}$ =	625	625		750			750	750	
Bearing Width (in)=	3	3		3.5			3.5	3.5	
Req'd Bearing Length (in)=	1.04	1.12		**1.54**			1.15	**1.60**	
I (in^4)=	41.6	197.9		488.4			488.4	250.1	
F_b	900	900		2600			2600	2600	
F'_b=	1170	990		2604			2604	2684	
S (in^3)=	15.1	42.8		82.3			82.3	52.6	
S_{req}=	15	38		56			31	36	
	Section OK	**Section OK**		**Section OK**			**Section OK**	**Section OK**	
E (psi)=	1600000	1600000		1900000			1900000	1900000	
F'_v (psi)=	180	180		285			285	285	
f_v (psi)=	178	113		146			109	189	
	Shear OK	**Shear OK**		**Shear OK**			**Shear OK**	**Shear OK**	
Location of Max Deflection=	1.50	3.00		6.00			4.50	3.88	
Allowable Live Deflection (in)=	0.075	0.150		0.300			0.225	0.194	
Live Deflection (in)=	0.030	0.050		0.241			0.076	0.154	
Allowable Total Deflection (in)=	0.100	0.200		0.400			0.300	0.258	
Total Deflection (in)=	0.036	0.064		0.338			0.107	0.185	
	Deflection OK	**Deflection OK**		**Deflection OK**			**Deflection OK**	**Deflection OK**	

Project: 2017-2259

Location: SB5
Multi-Loaded Multi-Span Beam
[2015 International Building Code(AISC 14th Ed ASD)]
A992-50 W10x19 x 17.33 FT
Section Adequate By: 52.9%
Controlling Factor: Deflection

Jack Miller
LEI Surveyors and Engineers
3302 North Main Street
Spanish Fork, Utah

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DEFLECTIONS	Center	
Live Load	0.38	IN L/550
Dead Load	0.16	in
Total Load	0.53	IN L/390

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	4506	lb	4506	lb
Dead Load	1854	lb	1854	lb
Total Load	6360	lb	6360	lb
Bearing Length	0.70	in	0.70	in

BEAM DATA	Center	
Span Length	17.33	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	17.33	ft

LOADING DIAGRAM

UNIFORM LOADS	Center	
Uniform Live Load	520	plf
Uniform Dead Load	195	plf
Beam Self Weight	19	plf
Total Uniform Load	734	plf

STEEL PROPERTIES
W10x19 - A992-50

Properties:			
Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	10.2	in
Web Thickness:	tw =	0.25	in
Flange Width:	bf =	4.02	in
Flange Thickness:	tf =	0.4	in
Distance to Web Toe of Fillet:	k =	0.7	in
Moment of Inertia About X-X Axis:	Ix =	96.3	in4
Section Modulus About X-X Axis:	Sx =	18.8	in3
Plastic Section Modulus About X-X Axis:	Zx =	21.6	in3
Design Properties per AISC 14th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	5.09	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	35.24	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	3.09	ft
Nominal Flexural Strength w/ safety factor:	Mn =	53892	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	35.24	
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 =	51000	lb

Controlling Moment: 27555 ft-lb
8.66 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: 6360 lb
At left 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):	62.98 in4	96.3 in4
Moment:	27555 ft-lb	53892 ft-lb
Shear:	6360 lb	51000 lb

NOTES

Project: 2017-2259

Location: SB6
Multi-Loaded Multi-Span Beam
[2015 International Building Code(AISC 14th Ed ASD)]
A992-50 W10x49 x 24.0 FT
Section Adequate By: 131.6%
Controlling Factor: Deflection

Jack Miller
LEI Surveyors and Engineers
3302 North Main Street
Spanish Fork, Utah

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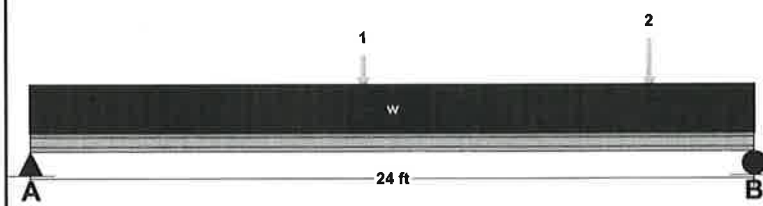
DEFLECTIONS	Center	
Live Load	0.34	IN L/839
Dead Load	0.18	in
Total Load	0.52	IN L/556

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	2731	lb	5704	lb
Dead Load	1612	lb	2728	lb
Total Load	4343	lb	8432	lb
Bearing Length	1.06	in	1.06	in

BEAM DATA	Center	
Span Length	24	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	24	ft

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	40	plf
Uniform Dead Load	15	plf
Beam Self Weight	49	plf
Total Uniform Load	104	plf

POINT LOADS - CENTER SPAN

Load Number	One	Two
Live Load	2932 lb	4543 lb
Dead Load	1100 lb	1704 lb
Location	11 ft	20.5 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 =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	8.97	ft
Nominal Flexural Strength w/ safety factor:	Mn =	150699	ft-lb
Controlling Equation:	F2-1		
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: 41448 ft-lb
11.04 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Controlling Shear: -8432 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):	117.44 in4	272 in4
Moment:	41448 ft-lb	150699 ft-lb
Shear:	-8432 lb	68000 lb

NOTES

Beams	MB1	MB2	MB3	MB4	MB5	MB6	MB7	MB8	MB9
Roofing material =	Shingle/Tile		Shingle/Tile	Shingle/Tile	Shingle/Tile				
Roof Pitch=	0.5		0.5	0.5	0.5				
Angle=	2.4		2.4	2.4	2.4				
C_S =	1.000		1.000	1.000	1.000				
Increase for Drift/Valley=	1.000		1.000	1.000	1.000				
Effective snow load (psf)=	181		181	100	100				
Roof dead load (psf)=	15		15	15	15				
Floor live load (psf)=	40		40	40	40				
Floor dead load (psf)=	15		15	15	15				
Length (ft)=	3		7	7	13				
Trib. Area $_{roof}$=	0		0	5.5	4.33				
Trib. Area $_{floor}$=	13		1	0	2				
w_S (plf) =	0		0	550	433				
w_L (plf) =	520		40	0	80				
w_D (plf) =	199		21	92	323				
$w_{self\ weight}$ (plf) =	3.6		6.0	9.0	28.5				
Point Load (lb)=			2959		13921				
a (ft)=	1.5		6.25	3.5	11				
b (ft)=	1.5		0.75	3.5	2				
Add. uniform load (plf)=					200				
Allowable Live Deflection =	L/480		L/480	L/240	L/360				
Allowable Total Deflection =	L/360		L/360	L/180	L/240				
Left Reaction (lb)=	1078	See attached calculations	531	2245	7058	See attached calculations	See attached calculations	See attached calculations	See attached calculations
Right Reaction (lb)=	1078		2855	2245	16696				
V_{max} (lb)=	1078		2855	2245	16696				
Location of M_{max} (ft)=	1.5		6.3	3.5	9.3				
M_{max} (ftlb)=	808		2124	3929	32932				
Size Factor (C_F)=	1.30		1.10	1.10	1.00				
Volume Factor (C_v)=	1.00		1.00	1.00	0.98				
Duration Factor (C_d) =	1.00		1.00	1.00	1.00				
Beam Type (t,g,m,p,ts,rb)	t		t	t	m				
d (in)=	5.5		9.25	9.25	14				
b (in)=	3		3	4.5	7				
$F'_{c\perp}$ =	625		625	625	750				
Bearing Width (in)=	3		3	4.5	7				
Req'd Bearing Length (in)=	0.57		**1.52**	0.80	**3.18**				
I (in^4)=	41.6		197.9	296.8	1600.7				
F_b	900		900	900	2600				
F'_b=	1170		990	990	2546				
S (in^3)=	15.1		42.8	64.2	228.7				
S_{req}=	8		26	48	155				
	Section OK		**Section OK**	**Section OK**	**Section OK**				
E (psi)=	1600000		1600000	1600000	1900000				
F'_v (psi)=	180		180	180	285				
f_v (psi)=	98		154	81	256				
	Shear OK		**Shear OK**	**Shear OK**	**Shear OK**				
Location of Max Deflection=	1.50		3.50	3.50	6.50				
Allowable Live Deflection (in)=	0.075		0.175	0.350	0.433				
Live Deflection (in)=	0.014		0.043	0.063	0.253				
Allowable Total Deflection (in)=	0.100		0.233	0.467	0.650				
Total Deflection (in)=	0.020		0.047	0.073	0.322				
	Deflection OK		**Deflection OK**	**Deflection OK**	**Deflection OK**				

Project: 2017-2259

Location: MB2
Multi-Loaded Multi-Span Beam
[2015 International Building Code(2015 NDS)]
(3) 1.75 IN x 11.875 IN x 17.5 FT (17 + 0.5)
1.9E Microllam - iLevel Trus Joist
Section Adequate By: 9.3%
Controlling Factor: Deflection

Kelly Christensen
LEI
3302 N. Main St.
Spanish Fork, UT 84660

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CAUTIONS

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

DEFLECTIONS	Center		Right	
Live Load	0.32	IN L/630	-0.03	IN 2L/394
Dead Load	0.14	in	-0.01	in
Total Load	0.46	IN L/444	-0.04	IN 2L/284

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	2040	lb	5180	lb
Dead Load	897	lb	2119	lb
Total Load	2937	lb	7299	lb
Bearing Length	0.75	in	1.85	in

BEAM DATA	Center		Right	
Span Length	17	ft	0.5	ft
Unbraced Length-Top	0	ft	0	ft
Unbraced Length-Bottom	17	ft	0.5	ft
Live Load Duration Factor	1.00			
Notch Depth	0.00			

MATERIAL PROPERTIES

1.9E Microllam - iLevel Trus Joist

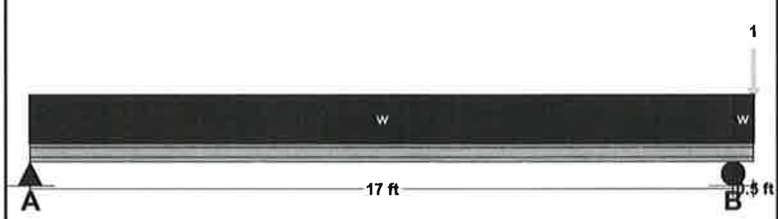
	Base Values			Adjusted		
Bending Stress:	Fb =	2600	psi	Fb' =	2604	psi
	Cd=1.00 CF=1.00					
Shear Stress:	Fv =	285	psi	Fv' =	285	psi
	Cd=1.00					
Modulus of Elasticity:	E =	1900	ksi	E' =	1900	ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	750	psi	Fc - ⊥' =	750	psi

Controlling Moment: 12344 ft-lb
8.33 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Controlling Shear: -2795 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:	56.89 in3	123.39 in3
Area (Shear):	14.71 in2	62.34 in2
Moment of Inertia (deflection):	670.25 in4	732.62 in4
Moment:	12344 ft-lb	26772 ft-lb
Shear:	-2795 lb	11845 lb

LOADING DIAGRAM



UNIFORM LOADS	Center		Right	
Uniform Live Load	240	plf	240	plf
Uniform Dead Load	90	plf	90	plf
Beam Self Weight	19	plf	19	plf
Total Uniform Load	349	plf	349	plf

POINT LOADS - RIGHT SPAN	
Load Number	One
Live Load	2932 lb
Dead Load	1100 lb
Location	0.5 ft

NOTES

Project: 2017-2259

Location: MB6
Multi-Loaded Multi-Span Beam
[2015 International Building Code(AISC 14th Ed ASD)]
A992-50 W8x15 x 19.486 FT (4.7 + 12.7 + 2.2)
Section Adequate By: 25.5%
Controlling Factor: Moment

Mike Larrabee
LEI Consulting Engineers
3302 N Main Street
Spanish Fork, UT 84660

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DEFLECTIONS	Left		Center		Right	
Live Load	-0.13	IN 2L/888	0.26	IN L/584	-0.14	IN 2L/372
Dead Load	-0.01	in	0.02	in	-0.01	in
Total Load	-0.14	IN 2L/826	0.28	IN L/544	-0.15	IN 2L/346

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

REACTIONS	A		B	
Live Load	12036	lb	8074	lb
Dead Load	886	lb	595	lb
Total Load	12922	lb	8669	lb
Bearing Length	0.62	in	0.62	in

BEAM DATA	Left		Center		Right	
Span Length	4.66	ft	12.66	ft	2.17	ft
Unbraced Length-Top	0	ft	0	ft	0	ft
Unbraced Length-Bottom	4.66	ft	12.66	ft	2.17	ft

LOADING DIAGRAM

UNIFORM LOADS	Left		Center		Right	
Uniform Live Load	1032	plf	1032	plf	1032	plf
Uniform Dead Load	61	plf	61	plf	61	plf
Beam Self Weight	15	plf	15	plf	15	plf
Total Uniform Load	1108	plf	1108	plf	1108	plf

STEEL PROPERTIES

W8x15 - A992-50

Properties:

Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	8.11	in
Web Thickness:	tw =	0.25	in
Flange Width:	bf =	4.01	in
Flange Thickness:	tf =	0.32	in
Distance to Web Toe of Fillet:	k =	0.62	in
Moment of Inertia About X-X Axis:	Ix =	48	in4
Section Modulus About X-X Axis:	Sx =	11.8	in3
Plastic Section Modulus About X-X Axis:	Zx =	13.6	in3
Design Properties per AISC 14th Edition Steel Manual:			
Flange Buckling Ratio:	FBR =	6.37	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	28.08	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	12.66	ft
Limiting Unbraced Length -			
for lateral-torsional buckling:	Lp =	3.09	ft
for Eqn. F2-2:	Lr =	10.05	ft
Elastic lateral-torsional buckling stress:	Fcr =	25.64	ksi
Nominal Flexural Strength w/ safety factor:	Mn =	15095	ft-lb
Controlling Equation:	F2-3		
Web height to thickness ratio:	h/tw =	28.08	
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 =	39739	lb

Controlling Moment: -12030 ft-lb
Over left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 1, 2, 3
Controlling Shear: 7759 lb
At left 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):	31.03 in4	48 in4
Moment:	-12030 ft-lb	15095 ft-lb
Shear:	7759 lb	39739 lb

NOTES

Project: 2017-2259

Location: MB9
Multi-Loaded Multi-Span Beam
[2015 International Building Code(AISC 14th Ed ASD)]
A992-50 W10x45 x 9.0 FT
Section Adequate By: 503.7%
Controlling Factor: Moment

Kelly Christensen
LEI Consulting Engineers and Surveyors Inc.
3302 N. Main St.
Spanish Fork, UT 84660

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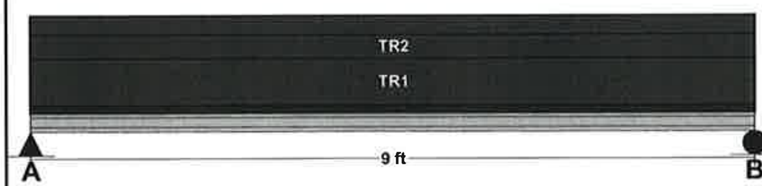
DEFLECTIONS	Center	
Live Load	0.04	IN L/3077
Dead Load	0.01	in
Total Load	0.05	IN L/2348

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A		B	
Live Load	7695	lb	7695	lb
Dead Load	2390	lb	2390	lb
Total Load	10085	lb	10085	lb
Bearing Length	1.12	in	1.12	in

BEAM DATA	Center	
Span Length	9	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	9	ft

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	0	plf
Uniform Dead Load	0	plf
Beam Self Weight	45	plf
Total Uniform Load	45	plf

TRAPEZOIDAL LOADS - CENTER SPAN

Load Number	One	Two	Three
Left Live Load	1220 plf	490 plf	0 plf
Left Dead Load	102 plf	184 plf	200 plf
Right Live Load	1220 plf	490 plf	0 plf
Right Dead Load	102 plf	184 plf	200 plf
Load Start	0 ft	0 ft	0 ft
Load End	9 ft	9 ft	9 ft
Load Length	9 ft	9 ft	9 ft

STEEL PROPERTIES

W10x45 - A992-50

Properties:

Yield Stress:	Fy =	50	ksi
Modulus of Elasticity:	E =	29000	ksi
Depth:	d =	10.1	in
Web Thickness:	tw =	0.35	in
Flange Width:	bf =	8.02	in
Flange Thickness:	tf =	0.62	in
Distance to Web Toe of Fillet:	k =	1.12	in
Moment of Inertia About X-X Axis:	Ix =	248	in4
Section Modulus About X-X Axis:	Sx =	49.1	in3
Plastic Section Modulus About X-X Axis:	Zx =	54.9	in3

Design Properties per AISC 14th Edition Steel Manual:

Flange Buckling Ratio:	FBR =	6.47	
Allowable Flange Buckling Ratio:	AFBR =	9.15	
Web Buckling Ratio:	WBR =	22.46	
Allowable Web Buckling Ratio:	AWBR =	90.55	
Controlling Unbraced Length:	Lb =	0	ft
Limiting Unbraced Length - for lateral-torsional buckling:	Lp =	7.1	ft
Nominal Flexural Strength w/ safety factor:	Mn =	136976	ft-lb
Controlling Equation:	F2-1		
Web height to thickness ratio:	h/tw =	22.46	
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 =	70700	lb

Controlling Moment: 22690 ft-lb
4.5 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Controlling Shear: -10085 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):	29.01 in4	248 in4
Moment:	22690 ft-lb	136976 ft-lb
Shear:	-10085 lb	70700 lb

NOTES

Shear Walls

Gridline A
Front Upper
Special Moment Frame
No anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind
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

$TA_{Roof-end}$	$TA_{Roof-int}$	V_s
0	0	3535
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
36	121	3535
	$V_{additional}$ =	

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1430	48529.9	1.100	1.300	8	4249	**5524**
					$V_{additiona}$ =	

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

Check uplift

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

See the attached Special Moment Frame calculations

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

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

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

Gridline E
Rear Upper
Structural Sheathing
No anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind
panel 1	6		1.4 :1	1.00	170	99
panel 2	15.5		0.5 :1	1.00	170	99
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	21.5	21.5	Total=	21.50		

Wind

$TA_{Roof-end}$	$TA_{Roof-int}$	V_s
0	0	2121
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
36	121	2121
	$V_{additional}$ =	

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1430	48529.9	1.100	1.000	6.5	5229	**3661**
					$V_{additiona}$ =	

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$_{seismic}$			
7740	11395	14728	19243	5590	7525	10535	13760
Use SW1				Use SW1			

Check uplift

DL	h	h'	Uplift $_s$	Uplift $_w$
100	8.5	8.5	1147	539
100	8.5	8.5	672	64

Use SW1
Use CS16 straps each side of panel as noted on plans

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

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

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

Shear Walls

Gridline 2
Left Upper

Structural Sheathing

No anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind	Check uplift DL	h	h'	$Uplift_s$	$Uplift_w$
panel 1	8.833		0.8 :1	1.00	173	53	130	7.5	7.5	723	-180
panel 2	12.33		0.6 :1	1.00	173	53	130	7.5	7.5	496	-408
panel 3											
panel 4											
panel 5											
panel 6											
panel 7											
panel 8											
panel 9											
panel 10											
$ASW_{1,2}$=	21.163	21.163	Total=	21.16							

Wind

$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int}$	V_s
0	0	1111
$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$
37	36	1111

$V_{additional}$ =

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1430	48529.9	1.100	1.000	6.5	5229	3661

$V_{additiona}$ =

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 $Resistance_{seismic}$			
7619	11216	14497	18941	5502	7407	10370	13544
Use SW1				Use SW1			

Use SW1
Use CS16 straps each side of panel as noted on plans

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

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

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

Gridline 5
Right Upper

Structural Sheathing

No anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind	Check uplift DL	h	h'	$Uplift_s$	$Uplift_w$
panel 1	8.833		1.0 :1	1.00	130	42	130	8.5	8.5	534	-214
panel 2	19.25		0.4 :1	1.00	130	42	130	8.5	8.5	-143	-891
panel 3											
panel 4											
panel 5											
panel 6											
panel 7											
panel 8											
panel 9											
panel 10											
$ASW_{1,2}$=	28.083	28.083	Total=	28.08							

Wind

$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int}$	V_s
0	0	1190
$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$
40	38	1190

$V_{additional}$ =

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1430	48529.9	1.100	1.000	6.5	5229	3661

$V_{additiona}$ =

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 $Resistance_{seismic}$			
10110	14884	19237	25134	7302	9829	13761	17973
Use SW1				Use SW1			

Use SW1
Use CS16 straps each side of panel as noted on plans

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

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

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

Shear Walls

Gridline A
Front Main
Special Moment Frame
No anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind
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	5646
$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\,min}$
57	194	**9181**
	$V_{additional}$ =	3535

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1222	16100	1.100	1.300	8	1410	**7356**
					$V_{additiona}$ =	5524

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

Check uplift

DL	h	h'	$Uplift_s$	$Uplift_w$

See the attached Special Moment Frame calculations

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

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

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

Gridline E
Rear Main
Structural Sheathing
No anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind
panel 1	6		1.5 :1	1.00	510	576
panel 2	4		2.3 :1	0.89	453	512
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	10	10	Total=	9.56		

Wind

$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int}$	V_s
0	0	3387
$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\,min}$
57	194	**5508**
	$V_{additional}$ =	2121

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1222	16100	1.100	1.000	6.5	1735	**4875**
					$V_{additiona}$ =	3661

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$_{seismic}$			
3440	5064	6546	8552	2484	3344	4682	6116
Use SW3				Use SW4			

Check uplift

DL	h	h'	$Uplift_s$	$Uplift_w$
100	9	9	**5439**	**5427**
100	9	9	**4554**	**4475**

Use SW4
Use HDU5-SDS2.5 holdowns or MST72 straps each side of panel as noted on plans

3x framing required at all panel edges and sill plates. Stagger edge nailing.

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

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

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

Shear Walls

Gridline 2
Left Main

Structural Sheathing

No anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind	Wind $TA_{Roof-end}$	Wind $TA_{Roof-int}$	Wind V_s	Seismic A_i	w_i	F	ρ	R	F_x	V_{final}	Check uplift DL	h	h'	$Uplift_s$	$Uplift_w$
panel 1	8.833		1.0 :1	1.00	230	147	0	0	2010	1222	16100	1.100	1.000	6.5	1735	**4875**	180	9	9	**2001**	**352**
panel 2	12.33		0.7 :1	1.00	230	147	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additiona}$ =	3661	180	9	9	**1459**	**-190**
panel 3							67	65	**3121**												
panel 4								$V_{additional}$ =	1111												
panel 5																					
panel 6																					
panel 7																					
panel 8																					
panel 9																					
panel 10																					
$ASW_{1,2}$=	21.163	**21.163**	Total=	21.16																	

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$_{seismic}$**			
7619	11216	14497	18941	5502	7407	10370	13544
Use SW1				Use SW1			

Use SW1
Use MST37 straps each side of panel as noted on plans

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

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

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

Gridline 5
Right Main

Structural Sheathing

1/2" anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind	Wind $TA_{Roof-end}$	Wind $TA_{Roof-int}$	Wind V_s	Seismic A_i	w_i	F	ρ	R	F_x	V_{final}	Check uplift DL	h	h'	$Uplift_s$	$Uplift_w$
panel 1	8.833		1.0 :1	1.00	259	170	0	0	2010	1222	16100	1.100	1.000	6.5	1735	**4875**	130	9	9	**2289**	**741**
panel 2	10		0.9 :1	1.00	259	170	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additiona}$ =	3661	130	9	9	1680	879
panel 3							67	65	**3200**												
panel 4								$V_{additional}$ =	1190												
panel 5																					
panel 6																					
panel 7																					
panel 8																					
panel 9																					
panel 10																					
$ASW_{1,2}$=	18.833	**18.833**	Total=	18.83																	

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$_{seismic}$**			
6780	9981	12901	16856	4897	6592	9228	12053
Use SW1				Use SW1			

Use SW1
Use STHD10/10RJ holdowns each side of panel as noted on plans
Use 1/2" anchor bolts @ 32" o.c.

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

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

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

Shear Walls

Gridline A
Front Lower

Special Moment Frame

No anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind
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

$TA_{Roof-end}$	$TA_{Roof-int}$	V_s
0	0	5687
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
61	190	**14868**
	$V_{additional}$=	9181

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1300	16970	1.100	1.300	8	1486	**9288**
					$V_{additiona}$ =	7356

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

Check uplift

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

See the attached Special Moment Frame calculations

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

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

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

Gridline 5
Left Lower

Structural Sheathing

1/2" anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind
panel 1	8.833		0.9 :1	1.00	302	186
panel 2	18.75		0.4 :1	1.00	302	186
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	27.583	**27.583**	Total=	27.58		

Wind

$TA_{Roof-end}$	$TA_{Roof-int}$	V_s
0	0	2010
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
67	65	**5132**
	$V_{additional}$=	3121

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1222	45688.6	1.100	1.000	6.5	4923	**8321**
					$V_{additiona}$ =	4875

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$_{seismic}$			
9930	14619	18894	24687	7172	9654	13516	17653
Use SW1				Use SW2			

Check uplift

DL	h	h'	Uplift$_s$	Uplift$_w$
140	8	8	**3796**	**1222**
140	8	8	**2560**	**-14**

Use SW2
Use STHD14/14RJ holdowns each side of panel as noted on plans
Use 1/2" anchor bolts @ 32" o.c.

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

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

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

Shear Walls

Gridline 11
Rear Lower

Structural Sheathing

1/2" anchor bolts

	Length	Inside	Ratio h:w	2w/h	SWS Seismic	SWS Wind
panel 1	6		1.3 :1	1.00	166	224
panel 2						
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10	15.5	15.5	0.0 :1	1.00	166	224
$ASW_{1,2}$=	21.5	37	Total=	37.00		

Wind

$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int}$	V_s
0	0	3177
$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$
61	190	**8305**
	$V_{additional}$ =	5129

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1300	16970	1.100	1.000	6.5	1829	**6155**
					$V_{additiona}$ =	4875

Check uplift

DL	h	h'	$Uplift_s$	$Uplift_w$
100	8	8	**6470**	**6507**

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 $Resistance_{seismic}$			
13320	19610	25345	33115	9620	12950	18130	23680
Use SW1				Use SW1			

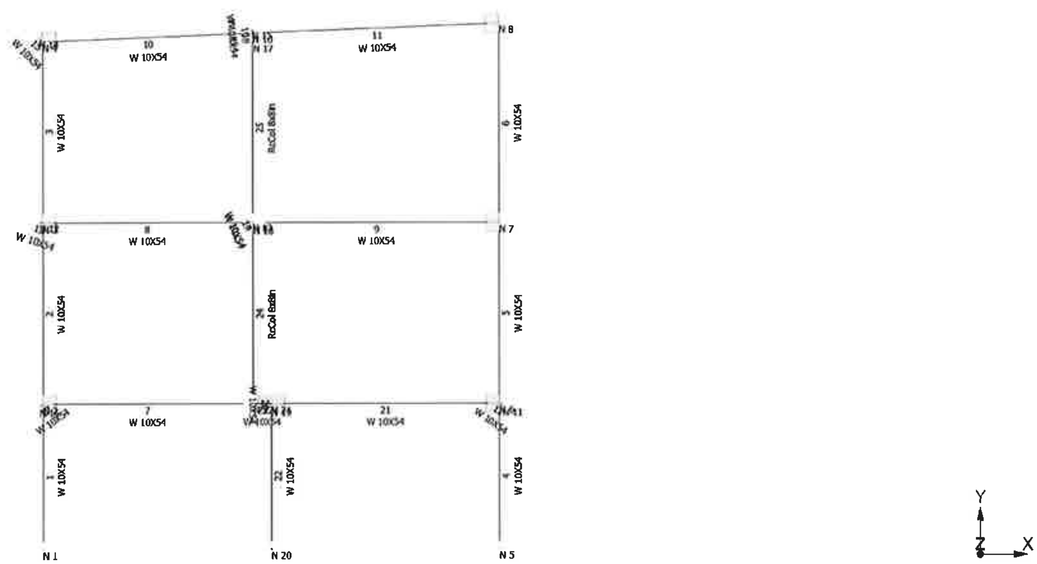
<<Foundation Wall

Use SW1
Use HDU8-SDS2.5 holdowns each side of panel as noted on plans
Use 1/2" anchor bolts @ 32" o.c.

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

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

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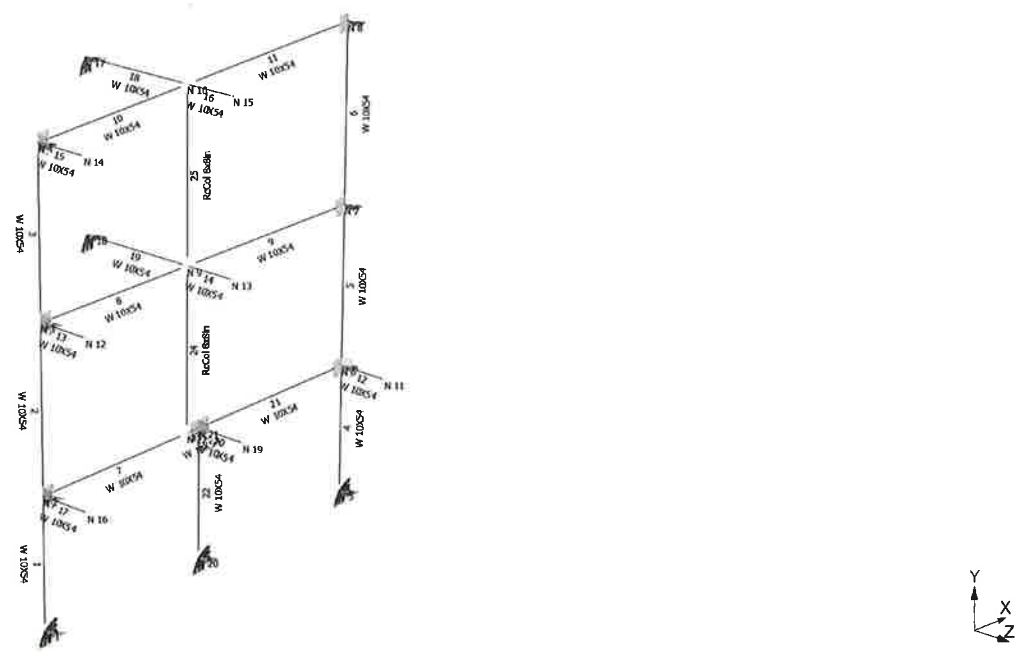
X
Z


W 10X54
N 15
N 14
N 13
N 12
N 11
N 19
N 16
Y
X
Z

Bentley

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Geometry data

GLOSSARY

Cb22, Cb33	: Moment gradient coefficients
Cm22, Cm33	: Coefficients applied to bending term in interaction formula
d0	: Tapered member section depth at J end of member
DJX	: Rigid end offset distance measured from J node in axis X
DJY	: Rigid end offset distance measured from J node in axis Y
DJZ	: Rigid end offset distance measured from J node in axis Z
DKX	: Rigid end offset distance measured from K node in axis X
DKY	: Rigid end offset distance measured from K node in axis Y
DKZ	: Rigid end offset distance measured from K node in axis Z
dL	: Tapered member section depth at K end of member
Ig factor	: Inertia reduction factor (Effective Inertia/Gross Inertia) for reinforced concrete members
K22	: Effective length factor about axis 2
K33	: Effective length factor about axis 3
L22	: Member length for calculation of axial capacity
L33	: Member length for calculation of axial capacity
LB pos	: Lateral unbraced length of the compression flange in the positive side of local axis 2
LB neg	: Lateral unbraced length of the compression flange in the negative side of local axis 2
RX	: Rotation about X
RY	: Rotation about Y
RZ	: Rotation about Z
TO	: 1 = Tension only member 0 = Normal member
TX	: Translation in X
TY	: Translation in Y
TZ	: Translation in Z

Nodes

Node	X [ft]	Y [ft]	Z [ft]	Rigid Floor
1	0.00	0.00	0.00	0
2	0.00	8.00	0.00	0
3	0.00	18.00	0.00	0
4	0.00	28.00	0.00	0
5	25.00	0.00	0.00	0
6	25.00	8.00	0.00	0
7	25.00	18.00	0.00	0
8	25.00	29.00	0.00	0
9	11.50	18.00	0.00	0
10	11.50	28.46	0.00	0
11	25.00	8.00	3.25	0
12	0.00	18.00	3.25	0
13	11.50	18.00	3.25	0
14	0.00	28.00	3.25	0
15	11.50	28.46	3.25	0
16	0.00	8.00	3.25	0
17	11.50	28.46	-7.50	0
18	11.50	18.00	-7.50	0
19	12.50	8.00	3.25	0
20	12.50	0.00	0.00	0
21	12.50	8.00	0.00	0
22	11.50	8.00	0.00	0

Restraints

Node	TX	TY	TZ	RX	RY	RZ
1	1	1	1	0	0	0
2	0	0	1	0	0	0
3	0	0	1	0	0	0
4	0	0	1	0	0	0
5	1	1	1	0	0	0
6	0	0	1	0	0	0
7	0	0	1	0	0	0
8	0	0	1	0	0	0
17	0	1	1	0	0	0
18	0	1	1	0	0	0
20	1	1	1	0	0	0

Members

Member	NJ	NK	Description	Section	Material	d0 [in]	dL [in]	Ig factor
1	1	2	Column	W 10X54	A992 Gr50	0.00	0.00	0.00
2	2	3	Column	W 10X54	A992 Gr50	0.00	0.00	0.00
3	3	4	Column	W 10X54	A992 Gr50	0.00	0.00	0.00
4	5	6	Column	W 10X54	A992 Gr50	0.00	0.00	0.00
5	6	7	Column	W 10X54	A992 Gr50	0.00	0.00	0.00
6	7	8	Column	W 10X54	A992 Gr50	0.00	0.00	0.00
7	2	22	Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
8	3	9	Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
9	9	7	Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
10	4	10	Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
11	10	8	Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
12	6	11	Cantilever Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
13	3	12	Cantilever Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
14	9	13	Cantilever Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
15	4	14	Cantilever Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
16	10	15	Cantilever Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
17	2	16	Cantilever Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
18	10	17	Cantilever Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
19	9	18	Cantilever Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
20	21	19	Cantilever Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
21	21	6	Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
22	20	21	Column	W 10X54	A992 Gr50	0.00	0.00	0.00
23	22	21	Beam	W 10X54	A992 Gr50	0.00	0.00	0.00
24	22	9	Wood Post	RcCol 8x8in	DFir-L_No2_col	0.00	0.00	0.00
25	9	10	Wood Post	RcCol 8x8in	DFir-L_No2_col	0.00	0.00	0.00

Orientation of local axes

Member	Rotation [Deg]	Axes23	NX	NY	NZ
16	2.3859	0	0.00	0.00	0.00

Hinges

	Node-J				Node-K						
Member	M33	M22	V3	V2	M33	M22	V3	V2	TOR	AXL	Axial rigidity
24	1	1	0	0	1	1	0	0	0	0	Full
25	1	1	0	0	1	1	0	0	0	0	Full

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Load data

GLOSSARY

Comb : Indicates if load condition is a load combination

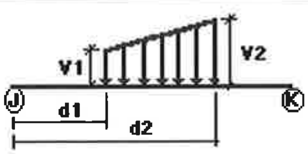
Load conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
LL	Live Load	No	LL
SL	Snow Load	No	SNOW
Wx	Wind in X	No	WIND
EQx	Seismic in X	No	EQ

Load on nodes

Condition	Node	FX [Kip]	FY [Kip]	FZ [Kip]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
Wx	2	5.687	0.00	0.00	0.00	0.00	0.00
	3	5.646	0.00	0.00	0.00	0.00	0.00
	4	1.768	0.00	0.00	0.00	0.00	0.00
	8	1.768	0.00	0.00	0.00	0.00	0.00
EQx	2	1.931	0.00	0.00	0.00	0.00	0.00
	3	1.832	0.00	0.00	0.00	0.00	0.00
	4	2.762	0.00	0.00	0.00	0.00	0.00
	8	2.762	0.00	0.00	0.00	0.00	0.00

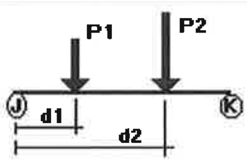
Distributed force on members



Condition	Member	Dir1	Val1 [Kip/ft]	Val2 [Kip/ft]	Dist1 [ft]	%	Dist2 [ft]	%
DL	7	y	-0.105	-0.105	0.00	No	100.00	Yes
	8	Y	-0.03	-0.03	0.00	No	100.00	Yes
	9	Y	-0.015	-0.015	0.00	No	100.00	Yes
	10	Y	-0.03	-0.03	0.00	No	100.00	Yes
	11	Y	-0.03	-0.03	0.00	No	100.00	Yes
	12	y	-0.383	-0.383	0.00	No	100.00	Yes
	13	y	-0.09	-0.09	0.00	No	100.00	Yes

	14	y	-0.09	-0.09	0.00	No	100.00	Yes
	15	y	-0.128	-0.125	0.00	No	100.00	Yes
	16	y	-0.188	-0.188	0.00	No	100.00	Yes
	17	y	-0.083	-0.083	0.00	No	100.00	Yes
	18	y	-0.188	-0.188	0.00	No	100.00	Yes
	19	y	-0.188	-0.188	0.00	No	100.00	Yes
	20	y	-0.188	-0.188	0.00	No	100.00	Yes
	21	y	-0.03	-0.03	0.00	No	100.00	Yes
	23	y	-0.03	-0.03	0.00	No	100.00	Yes
LL	9	y	-0.04	-0.04	0.00	No	100.00	Yes
	19	y	-0.26	-0.26	0.00	No	100.00	Yes
	21	y	-0.04	-0.04	0.00	No	100.00	Yes
	23	y	-0.04	-0.04	0.00	No	100.00	Yes
SL	7	y	-1.171	-1.171	0.00	No	100.00	Yes
	8	y	-0.335	-0.335	0.00	No	100.00	Yes
	10	y	-0.335	-0.335	0.00	No	100.00	Yes
	11	y	-0.335	-0.335	0.00	No	100.00	Yes
	12	y	-2.259	-2.259	0.00	No	100.00	Yes
	13	y	-1.004	-1.004	0.00	No	100.00	Yes
	14	y	-1.004	-1.004	0.00	No	100.00	Yes
	15	y	-1.422	-1.422	0.00	No	100.00	Yes
	16	y	-2.091	-2.091	0.00	No	100.00	Yes
	17	y	-0.921	-0.921	0.00	No	100.00	Yes
	18	y	-2.091	-2.091	0.00	No	100.00	Yes
	19	y	-1.004	-1.004	0.00	No	100.00	Yes
	20	y	-2.091	-2.091	0.00	No	100.00	Yes
	21	y	-0.167	-0.167	0.00	No	100.00	Yes
	23	y	-0.167	-0.167	0.00	No	100.00	Yes

Concentrated forces on members



Condition	Member	Dir1	Value1 [Kip]	Dist1 [ft]	%
DL	7	y	-0.292	11.25	No
SL	7	y	-3.259	11.25	No

Self weight multipliers for load conditions

		Self weight multiplier			
Condition	Description	Comb.	MultX	MultY	MultZ
DL	Dead Load	No	0.00	0.00	0.00
LL	Live Load	No	0.00	0.00	0.00
SL	Snow Load	No	0.00	0.00	0.00
Wx	Wind in X	No	0.00	0.00	0.00

EQx	Seismic in X	No	0.00	0.00	0.00

Earthquake (Dynamic analysis only)

Condition	a/g	Ang. [Deg]	Damp. [%]
DL	0.00	0.00	0.00
LL	0.00	0.00	0.00
SL	0.00	0.00	0.00
Wx	0.00	0.00	0.00
EQx	0.00	0.00	0.00

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Analysis result

Translations

	Translations [in]			Rotations [Rad]		
Node	**TX**	**TY**	**TZ**	**RX**	**RY**	**RZ**
Condition **DL=Dead Load**						
1	0.00000	0.00000	0.00000	-0.00001	-0.00023	0.00002
2	-0.00053	-0.00050	0.00000	0.00003	-0.00023	-0.00004
3	0.00182	-0.00085	0.00000	0.00000	0.00000	-0.00003
4	0.00323	-0.00104	0.00000	0.00008	0.00000	-0.00005
5	0.00000	0.00000	0.00000	-0.00007	0.00023	0.00001
6	-0.00050	-0.00039	0.00000	0.00014	0.00023	-0.00001
7	0.00186	-0.00054	0.00000	-0.00004	0.00000	0.00000
8	0.00311	-0.00064	0.00000	0.00002	0.00000	0.00004
9	0.00184	-0.00660	0.00000	0.00004	0.00000	0.00000
10	0.00349	-0.00894	0.00000	0.00009	0.00000	-0.00001
11	0.00857	-0.00739	0.00000	0.00017	0.00023	-0.00001
12	0.00182	-0.00134	0.00000	0.00001	0.00000	-0.00003
13	0.00182	-0.00854	0.00000	0.00005	0.00000	0.00000
14	0.00335	-0.00470	0.00000	0.00009	0.00000	-0.00005
15	0.00360	-0.01322	0.00000	0.00011	0.00000	-0.00001
16	-0.00963	-0.00192	0.00000	0.00003	-0.00023	-0.00004
17	0.00318	0.00000	0.00000	0.00014	0.00000	-0.00001
18	0.00190	0.00000	0.00000	0.00012	0.00000	0.00000
19	-0.00052	-0.01618	0.02340	0.00039	0.00000	0.00007
20	0.00000	0.00000	0.00000	0.00018	0.00000	-0.00002
21	-0.00054	-0.00089	0.02340	0.00037	0.00000	0.00007
22	-0.00054	-0.00283	0.02318	0.00034	-0.00004	0.00009
Condition **LL=Live Load**						
1	0.00000	0.00000	0.00000	0.00000	0.00000	0.00001
2	-0.00083	-0.00003	0.00000	0.00000	0.00000	0.00001
3	-0.00248	-0.00005	0.00000	0.00000	0.00000	0.00001
4	-0.00374	-0.00007	0.00000	0.00000	0.00000	0.00000
5	0.00000	0.00000	0.00000	0.00000	0.00000	0.00001
6	-0.00081	-0.00012	0.00000	0.00000	0.00000	0.00001
7	-0.00248	-0.00021	0.00000	0.00000	0.00000	0.00003
8	-0.00377	-0.00022	0.00000	0.00000	0.00000	0.00001
9	-0.00248	-0.00226	0.00000	-0.00005	0.00000	-0.00003
10	-0.00367	-0.00212	0.00000	0.00002	0.00000	0.00000
11	-0.00081	-0.00012	0.00000	0.00000	0.00000	0.00001
12	-0.00248	-0.00005	0.00000	0.00000	0.00000	0.00001
13	-0.00248	-0.00031	0.00000	-0.00005	0.00000	-0.00003
14	-0.00374	-0.00007	0.00000	0.00000	0.00000	0.00000
15	-0.00364	-0.00304	0.00000	0.00002	0.00000	0.00000
16	-0.00083	-0.00003	0.00000	0.00000	0.00000	0.00001
17	-0.00376	0.00000	0.00000	0.00002	0.00000	0.00000
18	-0.00248	0.00000	0.00000	0.00010	0.00000	-0.00003
19	-0.00081	-0.00028	0.00000	0.00000	0.00000	0.00001
20	0.00000	0.00000	0.00000	0.00000	0.00000	0.00001
21	-0.00081	-0.00028	0.00000	0.00000	0.00000	0.00001
22	-0.00082	-0.00070	0.00000	0.00000	0.00000	0.00001
Condition **SL=Snow Load**						
1	0.00000	0.00000	0.00000	-0.00015	-0.00290	0.00024

2	-0.00275	-0.00550	0.00000	0.00030	-0.00290	-0.00049
3	0.03020	-0.00933	0.00000	0.00003	0.00000	-0.00033
4	0.05092	-0.01140	0.00000	0.00091	0.00004	-0.00052
5	0.00000	0.00000	0.00000	-0.00040	0.00289	0.00010
6	-0.00249	-0.00255	0.00000	0.00081	0.00289	-0.00016
7	0.03067	-0.00381	0.00000	-0.00024	0.00001	-0.00012
8	0.04963	-0.00483	0.00000	0.00012	0.00001	0.00043
9	0.03042	-0.06398	-0.00001	0.00066	-0.00001	0.00015
10	0.05360	-0.09063	0.00000	0.00089	0.00003	-0.00009
11	0.11017	-0.04393	0.00000	0.00103	0.00289	-0.00016
12	0.03013	-0.01462	0.00000	0.00012	0.00000	-0.00033
13	0.03012	-0.09381	-0.00001	0.00075	-0.00001	0.00015
14	0.05234	-0.05279	0.00000	0.00104	0.00004	-0.00052
15	0.05458	-0.13435	0.00000	0.00109	0.00002	-0.00009
16	-0.11602	-0.02125	0.00000	0.00039	-0.00290	-0.00049
17	0.05046	0.00000	0.00000	0.00142	0.00003	-0.00009
18	0.03111	0.00000	0.00000	0.00091	-0.00001	0.00015
19	-0.00243	-0.19165	0.29100	0.00466	0.00001	0.00073
20	0.00000	0.00000	0.00000	0.00233	0.00001	-0.00025
21	-0.00293	-0.00873	0.29100	0.00446	0.00001	0.00073
22	-0.00291	-0.02864	0.28839	0.00413	-0.00043	0.00093

Condition **Wx=Wind in X**

1	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00420
2	0.35345	0.00220	0.00000	0.00000	0.00000	-0.00240
3	0.70687	0.00329	0.00000	0.00000	0.00000	-0.00202
4	0.93713	0.00368	0.00000	0.00000	0.00000	-0.00110
5	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00418
6	0.35161	-0.00195	0.00000	0.00000	0.00000	-0.00238
7	0.70473	-0.00273	0.00000	0.00000	0.00000	-0.00203
8	0.93773	-0.00300	0.00000	0.00000	0.00000	-0.00100
9	0.70589	0.00094	0.00000	-0.00001	0.00000	0.00090
10	0.93743	0.00014	0.00000	0.00000	0.00000	0.00045
11	0.35161	-0.00195	0.00000	0.00000	0.00000	-0.00238
12	0.70687	0.00329	0.00000	0.00000	0.00000	-0.00202
13	0.70589	0.00135	0.00000	-0.00001	0.00000	0.00090
14	0.93713	0.00368	0.00000	0.00000	0.00000	-0.00110
15	0.93743	0.00020	0.00000	0.00000	0.00000	0.00045
16	0.35345	0.00220	0.00000	0.00000	0.00000	-0.00240
17	0.93743	0.00000	0.00000	0.00000	0.00000	0.00045
18	0.70589	0.00000	0.00000	-0.00001	0.00000	0.00090
19	0.35122	-0.00025	0.00000	0.00000	0.00000	-0.00067
20	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00486
21	0.35122	-0.00025	0.00000	0.00000	0.00000	-0.00067
22	0.35140	0.00267	0.00000	0.00000	0.00000	-0.00020

Condition **EQx=Seismic in X**

1	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00276
2	0.23713	0.00185	0.00000	0.00000	0.00000	-0.00176
3	0.53101	0.00298	0.00000	0.00000	0.00000	-0.00197
4	0.79394	0.00345	0.00000	0.00000	0.00000	-0.00135
5	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00275
6	0.23653	-0.00161	0.00000	0.00000	0.00000	-0.00176
7	0.53004	-0.00243	0.00000	0.00000	0.00000	-0.00194
8	0.79459	-0.00278	0.00000	0.00000	0.00000	-0.00123
9	0.53056	-0.00078	0.00000	0.00001	0.00000	0.00087
10	0.79433	-0.00163	0.00000	0.00002	0.00000	0.00056
11	0.23653	-0.00161	0.00000	0.00000	0.00000	-0.00176
12	0.53101	0.00298	0.00000	0.00000	0.00000	-0.00197

13	0.53057	-0.00112	0.00000	0.00001	0.00000	0.00087
14	0.79394	0.00345	0.00000	0.00000	0.00000	-0.00135
15	0.79436	-0.00234	0.00000	0.00002	0.00000	0.00056
16	0.23713	0.00185	0.00000	0.00000	0.00000	-0.00176
17	0.79427	0.00000	0.00000	0.00002	0.00000	0.00056
18	0.53056	0.00000	0.00000	0.00001	0.00000	0.00087
19	0.23593	-0.00024	0.00000	0.00000	0.00000	-0.00040
20	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00329
21	0.23593	-0.00024	0.00000	0.00000	0.00000	-0.00040
22	0.23602	0.00093	0.00000	0.00000	0.00000	-0.00008

Condition **S1=DL**

1	0.00000	0.00000	0.00000	-0.00001	-0.00023	0.00002
2	-0.00053	-0.00050	0.00000	0.00003	-0.00023	-0.00004
3	0.00182	-0.00085	0.00000	0.00000	0.00000	-0.00003
4	0.00323	-0.00104	0.00000	0.00008	0.00000	-0.00005
5	0.00000	0.00000	0.00000	-0.00007	0.00023	0.00001
6	-0.00050	-0.00039	0.00000	0.00014	0.00023	-0.00001
7	0.00186	-0.00054	0.00000	-0.00004	0.00000	0.00000
8	0.00311	-0.00064	0.00000	0.00002	0.00000	0.00004
9	0.00184	-0.00660	0.00000	0.00004	0.00000	0.00000
10	0.00349	-0.00894	0.00000	0.00009	0.00000	-0.00001
11	0.00857	-0.00739	0.00000	0.00017	0.00023	-0.00001
12	0.00182	-0.00134	0.00000	0.00001	0.00000	-0.00003
13	0.00182	-0.00854	0.00000	0.00005	0.00000	0.00000
14	0.00335	-0.00470	0.00000	0.00009	0.00000	-0.00005
15	0.00360	-0.01322	0.00000	0.00011	0.00000	-0.00001
16	-0.00963	-0.00192	0.00000	0.00003	-0.00023	-0.00004
17	0.00318	0.00000	0.00000	0.00014	0.00000	-0.00001
18	0.00190	0.00000	0.00000	0.00012	0.00000	0.00000
19	-0.00052	-0.01618	0.02340	0.00039	0.00000	0.00007
20	0.00000	0.00000	0.00000	0.00018	0.00000	-0.00002
21	-0.00054	-0.00089	0.02340	0.00037	0.00000	0.00007
22	-0.00054	-0.00283	0.02318	0.00034	-0.00004	0.00009

Condition **S2=DL+LL**

1	0.00000	0.00000	0.00000	-0.00001	-0.00023	0.00003
2	-0.00136	-0.00053	0.00000	0.00003	-0.00023	-0.00004
3	-0.00067	-0.00091	0.00000	0.00000	0.00000	-0.00002
4	-0.00052	-0.00111	0.00000	0.00008	0.00000	-0.00005
5	0.00000	0.00000	0.00000	-0.00007	0.00023	0.00002
6	-0.00130	-0.00051	0.00000	0.00014	0.00023	0.00000
7	-0.00062	-0.00075	0.00000	-0.00004	0.00000	0.00003
8	-0.00066	-0.00085	0.00000	0.00002	0.00000	0.00005
9	-0.00064	-0.00885	0.00000	-0.00001	0.00000	-0.00003
10	-0.00018	-0.01107	0.00000	0.00011	0.00000	-0.00002
11	0.00780	-0.00751	0.00000	0.00017	0.00023	0.00000
12	-0.00067	-0.00139	0.00000	0.00001	0.00000	-0.00002
13	-0.00067	-0.00886	0.00000	0.00000	0.00000	-0.00003
14	-0.00039	-0.00477	0.00000	0.00009	0.00000	-0.00005
15	-0.00005	-0.01626	0.00000	0.00013	0.00000	-0.00002
16	-0.01049	-0.00195	0.00000	0.00003	-0.00023	-0.00004
17	-0.00058	0.00000	0.00000	0.00016	0.00000	-0.00002
18	-0.00059	0.00000	0.00000	0.00022	0.00000	-0.00003
19	-0.00133	-0.01651	0.02350	0.00039	0.00000	0.00007
20	0.00000	0.00000	0.00000	0.00018	0.00000	-0.00001
21	-0.00136	-0.00117	0.02350	0.00037	0.00000	0.00007
22	-0.00136	-0.00353	0.02328	0.00034	-0.00004	0.00010

Condition **S3=DL+SL**

1	0.00000	0.00000	0.00000	-0.00016	-0.00321	0.00026
2	-0.00327	-0.00600	0.00000	0.00033	-0.00321	-0.00053
3	0.03210	-0.01018	0.00000	0.00003	0.00000	-0.00035
4	0.05427	-0.01245	0.00000	0.00099	0.00004	-0.00057
5	0.00000	0.00000	0.00000	-0.00047	0.00319	0.00011
6	-0.00297	-0.00294	0.00000	0.00095	0.00319	-0.00017
7	0.03261	-0.00435	0.00000	-0.00028	0.00001	-0.00012
8	0.05286	-0.00546	0.00000	0.00014	0.00001	0.00047
9	0.03233	-0.07058	-0.00001	0.00070	-0.00001	0.00016
10	0.05721	-0.09957	0.00000	0.00098	0.00004	-0.00010
11	0.12151	-0.05134	0.00000	0.00120	0.00319	-0.00017
12	0.03202	-0.01595	0.00000	0.00013	0.00000	-0.00035
13	0.03200	-0.10236	-0.00001	0.00080	-0.00001	0.00016
14	0.05582	-0.05750	0.00000	0.00113	0.00004	-0.00057
15	0.05830	-0.14757	0.00000	0.00120	0.00002	-0.00010
16	-0.12846	-0.02318	0.00000	0.00042	-0.00321	-0.00053
17	0.05377	0.00000	0.00000	0.00155	0.00004	-0.00010
18	0.03310	0.00000	0.00000	0.00103	-0.00001	0.00016
19	-0.00288	-0.21084	0.32159	0.00512	0.00001	0.00080
20	0.00000	0.00000	0.00000	0.00258	0.00001	-0.00027
21	-0.00346	-0.00962	0.32159	0.00491	0.00001	0.00080
22	-0.00344	-0.03147	0.31871	0.00454	-0.00048	0.00101

Condition **S4=DL+0.75LL**

1	0.00000	0.00000	0.00000	-0.00001	-0.00023	0.00003
2	-0.00115	-0.00053	0.00000	0.00003	-0.00023	-0.00004
3	-0.00004	-0.00089	0.00000	0.00000	0.00000	-0.00002
4	0.00042	-0.00109	0.00000	0.00008	0.00000	-0.00005
5	0.00000	0.00000	0.00000	-0.00007	0.00023	0.00002
6	-0.00110	-0.00048	0.00000	0.00014	0.00023	0.00000
7	0.00000	-0.00070	0.00000	-0.00004	0.00000	0.00002
8	0.00028	-0.00080	0.00000	0.00002	0.00000	0.00005
9	-0.00002	-0.00829	0.00000	0.00000	0.00000	-0.00002
10	0.00074	-0.01053	0.00000	0.00011	0.00000	-0.00001
11	0.00799	-0.00748	0.00000	0.00017	0.00023	0.00000
12	-0.00005	-0.00138	0.00000	0.00001	0.00000	-0.00002
13	-0.00005	-0.00878	0.00000	0.00001	0.00000	-0.00002
14	0.00055	-0.00475	0.00000	0.00009	0.00000	-0.00005
15	0.00086	-0.01550	0.00000	0.00012	0.00000	-0.00001
16	-0.01028	-0.00194	0.00000	0.00003	-0.00023	-0.00004
17	0.00036	0.00000	0.00000	0.00015	0.00000	-0.00001
18	0.00003	0.00000	0.00000	0.00019	0.00000	-0.00002
19	-0.00113	-0.01643	0.02347	0.00039	0.00000	0.00007
20	0.00000	0.00000	0.00000	0.00018	0.00000	-0.00001
21	-0.00115	-0.00110	0.02347	0.00037	0.00000	0.00007
22	-0.00115	-0.00335	0.02326	0.00034	-0.00004	0.00009

Condition **S5=DL+0.75SL**

1	0.00000	0.00000	0.00000	-0.00012	-0.00239	0.00020
2	-0.00260	-0.00463	0.00000	0.00025	-0.00239	-0.00041
3	0.02444	-0.00785	0.00000	0.00002	0.00000	-0.00027
4	0.04136	-0.00960	0.00000	0.00076	0.00003	-0.00044
5	0.00000	0.00000	0.00000	-0.00037	0.00238	0.00009
6	-0.00237	-0.00231	0.00000	0.00075	0.00238	-0.00013
7	0.02484	-0.00340	0.00000	-0.00022	0.00001	-0.00009
8	0.04028	-0.00425	0.00000	0.00011	0.00000	0.00036
9	0.02462	-0.05458	-0.00001	0.00053	-0.00001	0.00012
10	0.04364	-0.07691	0.00000	0.00076	0.00003	-0.00008
11	0.09047	-0.04034	0.00000	0.00094	0.00238	-0.00013

12	0.02438	-0.01231	0.00000	0.00010	0.00000	-0.00027
13	0.02438	-0.07890	-0.00001	0.00061	-0.00001	0.00012
14	0.04256	-0.04429	0.00000	0.00087	0.00003	-0.00044
15	0.04448	-0.11397	0.00000	0.00092	0.00002	-0.00008
16	-0.09591	-0.01785	0.00000	0.00033	-0.00239	-0.00041
17	0.04098	0.00000	0.00000	0.00120	0.00003	-0.00008
18	0.02519	0.00000	0.00000	0.00080	-0.00001	0.00012
19	-0.00236	-0.15914	0.23977	0.00386	0.00001	0.00062
20	0.00000	0.00000	0.00000	0.00191	0.00001	-0.00021
21	-0.00274	-0.00744	0.23977	0.00370	0.00001	0.00062
22	-0.00273	-0.02431	0.23761	0.00342	-0.00036	0.00078

Condition **S6=DL+0.75LL+0.75SL**

1	0.00000	0.00000	0.00000	-0.00013	-0.00240	0.00021
2	-0.00323	-0.00465	0.00000	0.00025	-0.00240	-0.00041
3	0.02256	-0.00789	0.00000	0.00002	0.00000	-0.00027
4	0.03853	-0.00965	0.00000	0.00076	0.00003	-0.00044
5	0.00000	0.00000	0.00000	-0.00037	0.00239	0.00009
6	-0.00299	-0.00240	0.00000	0.00075	0.00239	-0.00012
7	0.02296	-0.00356	0.00000	-0.00022	0.00001	-0.00007
8	0.03743	-0.00442	0.00000	0.00011	0.00000	0.00037
9	0.02274	-0.05627	-0.00001	0.00049	-0.00001	0.00010
10	0.04086	-0.07850	0.00000	0.00078	0.00003	-0.00008
11	0.09018	-0.04043	0.00000	0.00094	0.00239	-0.00012
12	0.02250	-0.01235	0.00000	0.00010	0.00000	-0.00027
13	0.02250	-0.07914	-0.00001	0.00057	-0.00001	0.00010
14	0.03973	-0.04434	0.00000	0.00087	0.00003	-0.00044
15	0.04173	-0.11626	0.00000	0.00094	0.00002	-0.00008
16	-0.09687	-0.01787	0.00000	0.00033	-0.00240	-0.00041
17	0.03813	0.00000	0.00000	0.00122	0.00003	-0.00008
18	0.02332	0.00000	0.00000	0.00088	-0.00001	0.00010
19	-0.00298	-0.15970	0.24061	0.00387	0.00001	0.00062
20	0.00000	0.00000	0.00000	0.00192	0.00001	-0.00020
21	-0.00336	-0.00765	0.24061	0.00371	0.00001	0.00062
22	-0.00335	-0.02483	0.23844	0.00343	-0.00036	0.00079

Condition **S7=DL+0.6Wx**

1	0.00000	0.00000	0.00000	-0.00001	-0.00024	-0.00250
2	0.21198	0.00082	0.00000	0.00003	-0.00024	-0.00148
3	0.42672	0.00112	0.00000	0.00000	0.00000	-0.00124
4	0.56648	0.00117	0.00000	0.00008	0.00000	-0.00071
5	0.00000	0.00000	0.00000	-0.00007	0.00023	-0.00250
6	0.21090	-0.00157	0.00000	0.00014	0.00023	-0.00144
7	0.42548	-0.00218	0.00000	-0.00004	0.00000	-0.00122
8	0.56672	-0.00244	0.00000	0.00002	0.00000	-0.00056
9	0.42615	-0.00603	0.00000	0.00003	0.00000	0.00054
10	0.56693	-0.00886	0.00000	0.00009	0.00000	0.00026
11	0.22004	-0.00857	0.00000	0.00017	0.00023	-0.00144
12	0.42672	0.00064	0.00000	0.00001	0.00000	-0.00124
13	0.42613	-0.00773	0.00000	0.00004	0.00000	0.00054
14	0.56661	-0.00249	0.00000	0.00009	0.00000	-0.00071
15	0.56703	-0.01309	0.00000	0.00011	0.00000	0.00026
16	0.20280	-0.00059	0.00000	0.00003	-0.00024	-0.00148
17	0.56662	0.00000	0.00000	0.00014	0.00000	0.00026
18	0.42621	0.00000	0.00000	0.00011	0.00000	0.00054
19	0.21066	-0.01641	0.02358	0.00039	0.00000	-0.00033
20	0.00000	0.00000	0.00000	0.00018	0.00000	-0.00294
21	0.21063	-0.00104	0.02358	0.00037	0.00000	-0.00033
22	0.21074	-0.00122	0.02337	0.00035	-0.00004	-0.00003

Condition **S8=DL+0.7EQx**

1	0.00000	0.00000	0.00000	-0.00001	-0.00023	-0.00191
2	0.16582	0.00079	0.00000	0.00003	-0.00023	-0.00128
3	0.37422	0.00124	0.00000	0.00000	0.00000	-0.00141
4	0.55990	0.00137	0.00000	0.00008	0.00000	-0.00100
5	0.00000	0.00000	0.00000	-0.00007	0.00023	-0.00192
6	0.16543	-0.00152	0.00000	0.00014	0.00023	-0.00124
7	0.37358	-0.00225	0.00000	-0.00004	0.00000	-0.00136
8	0.56023	-0.00259	0.00000	0.00002	0.00000	-0.00082
9	0.37392	-0.00714	0.00000	0.00005	0.00000	0.00061
10	0.56044	-0.01008	0.00000	0.00010	0.00000	0.00038
11	0.17454	-0.00852	0.00000	0.00017	0.00023	-0.00124
12	0.37421	0.00075	0.00000	0.00001	0.00000	-0.00141
13	0.37390	-0.00933	0.00000	0.00005	0.00000	0.00061
14	0.56003	-0.00228	0.00000	0.00009	0.00000	-0.00100
15	0.56056	-0.01485	0.00000	0.00012	0.00000	0.00038
16	0.15667	-0.00062	0.00000	0.00003	-0.00023	-0.00128
17	0.56008	0.00000	0.00000	0.00015	0.00000	0.00038
18	0.37398	0.00000	0.00000	0.00013	0.00000	0.00061
19	0.16500	-0.01640	0.02351	0.00039	0.00000	-0.00021
20	0.00000	0.00000	0.00000	0.00018	0.00000	-0.00233
21	0.16497	-0.00106	0.02351	0.00037	0.00000	-0.00021
22	0.16504	-0.00218	0.02330	0.00035	-0.00004	0.00003

Condition **S9=DL+0.75LL+0.45Wx+0.75SL**

1	0.00000	0.00000	0.00000	-0.00012	-0.00242	-0.00171
2	0.15863	-0.00364	0.00000	0.00025	-0.00242	-0.00150
3	0.34574	-0.00639	0.00000	0.00002	0.00000	-0.00119
4	0.46672	-0.00797	0.00000	0.00076	0.00003	-0.00094
5	0.00000	0.00000	0.00000	-0.00037	0.00240	-0.00182
6	0.15804	-0.00329	0.00000	0.00075	0.00240	-0.00121
7	0.34517	-0.00480	0.00000	-0.00022	0.00001	-0.00100
8	0.46589	-0.00579	0.00000	0.00011	0.00000	-0.00009
9	0.34548	-0.05585	-0.00001	0.00049	-0.00001	0.00051
10	0.46919	-0.07844	0.00000	0.00078	0.00003	0.00012
11	0.25176	-0.04134	0.00000	0.00094	0.00240	-0.00121
12	0.34568	-0.01085	0.00000	0.00010	0.00000	-0.00119
13	0.34523	-0.07853	-0.00001	0.00057	-0.00001	0.00051
14	0.46792	-0.04266	0.00000	0.00087	0.00003	-0.00094
15	0.47006	-0.11617	0.00000	0.00094	0.00002	0.00012
16	0.06433	-0.01686	0.00000	0.00033	-0.00242	-0.00150
17	0.46646	0.00000	0.00000	0.00122	0.00003	0.00012
18	0.34605	0.00000	0.00000	0.00087	-0.00001	0.00051
19	0.15793	-0.16044	0.24214	0.00389	0.00001	0.00032
20	0.00000	0.00000	0.00000	0.00193	0.00001	-0.00243
21	0.15748	-0.00776	0.24214	0.00372	0.00001	0.00032
22	0.15758	-0.02361	0.23998	0.00345	-0.00036	0.00070

Condition **S10=DL+0.525EQx**

1	0.00000	0.00000	0.00000	-0.00001	-0.00023	-0.00143
2	0.12423	0.00047	0.00000	0.00003	-0.00023	-0.00097
3	0.28111	0.00071	0.00000	0.00000	0.00000	-0.00106
4	0.42073	0.00077	0.00000	0.00008	0.00000	-0.00076
5	0.00000	0.00000	0.00000	-0.00007	0.00023	-0.00144
6	0.12395	-0.00124	0.00000	0.00014	0.00023	-0.00093
7	0.28064	-0.00182	0.00000	-0.00004	0.00000	-0.00102
8	0.42095	-0.00210	0.00000	0.00002	0.00000	-0.00060
9	0.28090	-0.00701	0.00000	0.00004	0.00000	0.00046

10	0.42120	-0.00980	0.00000	0.00010	0.00000	0.00028
11	0.13305	-0.00824	0.00000	0.00017	0.00023	-0.00093
12	0.28111	0.00023	0.00000	0.00001	0.00000	-0.00106
13	0.28087	-0.00913	0.00000	0.00005	0.00000	0.00046
14	0.42085	-0.00289	0.00000	0.00009	0.00000	-0.00076
15	0.42132	-0.01444	0.00000	0.00012	0.00000	0.00028
16	0.11510	-0.00094	0.00000	0.00003	-0.00023	-0.00097
17	0.42085	0.00000	0.00000	0.00015	0.00000	0.00028
18	0.28095	0.00000	0.00000	0.00012	0.00000	0.00046
19	0.12362	-0.01635	0.02349	0.00039	0.00000	-0.00014
20	0.00000	0.00000	0.00000	0.00018	0.00000	-0.00175
21	0.12359	-0.00102	0.02349	0.00037	0.00000	-0.00014
22	0.12364	-0.00234	0.02327	0.00034	-0.00004	0.00004

Condition **S11=DL+0.75SL**

1	0.00000	0.00000	0.00000	-0.00012	-0.00239	0.00020
2	-0.00260	-0.00463	0.00000	0.00025	-0.00239	-0.00041
3	0.02444	-0.00785	0.00000	0.00002	0.00000	-0.00027
4	0.04136	-0.00960	0.00000	0.00076	0.00003	-0.00044
5	0.00000	0.00000	0.00000	-0.00037	0.00238	0.00009
6	-0.00237	-0.00231	0.00000	0.00075	0.00238	-0.00013
7	0.02484	-0.00340	0.00000	-0.00022	0.00001	-0.00009
8	0.04028	-0.00425	0.00000	0.00011	0.00000	0.00036
9	0.02462	-0.05458	-0.00001	0.00053	-0.00001	0.00012
10	0.04364	-0.07691	0.00000	0.00076	0.00003	-0.00008
11	0.09047	-0.04034	0.00000	0.00094	0.00238	-0.00013
12	0.02438	-0.01231	0.00000	0.00010	0.00000	-0.00027
13	0.02438	-0.07890	-0.00001	0.00061	-0.00001	0.00012
14	0.04256	-0.04429	0.00000	0.00087	0.00003	-0.00044
15	0.04448	-0.11397	0.00000	0.00092	0.00002	-0.00008
16	-0.09591	-0.01785	0.00000	0.00033	-0.00239	-0.00041
17	0.04098	0.00000	0.00000	0.00120	0.00003	-0.00008
18	0.02519	0.00000	0.00000	0.00080	-0.00001	0.00012
19	-0.00236	-0.15914	0.23977	0.00386	0.00001	0.00062
20	0.00000	0.00000	0.00000	0.00191	0.00001	-0.00021
21	-0.00274	-0.00744	0.23977	0.00370	0.00001	0.00062
22	-0.00273	-0.02431	0.23761	0.00342	-0.00036	0.00078

Condition **S12=DL+0.525EQx+0.75SL**

1	0.00000	0.00000	0.00000	-0.00012	-0.00240	-0.00127
2	0.12412	-0.00364	0.00000	0.00025	-0.00240	-0.00135
3	0.30755	-0.00626	0.00000	0.00002	0.00000	-0.00132
4	0.46405	-0.00776	0.00000	0.00076	0.00003	-0.00116
5	0.00000	0.00000	0.00000	-0.00037	0.00239	-0.00138
6	0.12403	-0.00316	0.00000	0.00075	0.00239	-0.00107
7	0.30743	-0.00469	0.00000	-0.00022	0.00001	-0.00112
8	0.46330	-0.00573	0.00000	0.00011	0.00000	-0.00029
9	0.30749	-0.05499	-0.00001	0.00054	-0.00001	0.00058
10	0.46653	-0.07778	0.00000	0.00077	0.00003	0.00022
11	0.21721	-0.04121	0.00000	0.00094	0.00239	-0.00107
12	0.30749	-0.01073	0.00000	0.00010	0.00000	-0.00132
13	0.30725	-0.07949	-0.00001	0.00062	-0.00001	0.00058
14	0.46524	-0.04245	0.00000	0.00087	0.00003	-0.00116
15	0.46739	-0.11521	0.00000	0.00093	0.00002	0.00022
16	0.03039	-0.01686	0.00000	0.00033	-0.00240	-0.00135
17	0.46383	0.00000	0.00000	0.00121	0.00003	0.00022
18	0.30806	0.00000	0.00000	0.00081	-0.00001	0.00058
19	0.12377	-0.15966	0.24073	0.00387	0.00001	0.00040
20	0.00000	0.00000	0.00000	0.00192	0.00001	-0.00197

21	0.12334	-0.00756	0.24073	0.00371	0.00001	0.00040
22	0.12340	-0.02381	0.23857	0.00343	-0.00036	0.00074

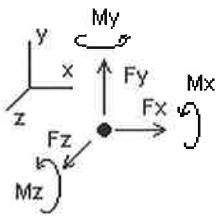
Condition **S13=0.6DL+0.6Wx**

1	0.00000	0.00000	0.00000	-0.00001	-0.00014	-0.00251
2	0.21201	0.00102	0.00000	0.00002	-0.00014	-0.00146
3	0.42567	0.00146	0.00000	0.00000	0.00000	-0.00123
4	0.56478	0.00158	0.00000	0.00005	0.00000	-0.00069
5	0.00000	0.00000	0.00000	-0.00004	0.00014	-0.00250
6	0.21092	-0.00141	0.00000	0.00008	0.00014	-0.00144
7	0.42441	-0.00196	0.00000	-0.00002	0.00000	-0.00122
8	0.56507	-0.00218	0.00000	0.00001	0.00000	-0.00058
9	0.42509	-0.00339	0.00000	0.00002	0.00000	0.00054
10	0.56512	-0.00528	0.00000	0.00005	0.00000	0.00026
11	0.21638	-0.00561	0.00000	0.00010	0.00014	-0.00144
12	0.42567	0.00117	0.00000	0.00001	0.00000	-0.00123
13	0.42508	-0.00431	0.00000	0.00002	0.00000	0.00054
14	0.56486	-0.00061	0.00000	0.00006	0.00000	-0.00069
15	0.56518	-0.00781	0.00000	0.00006	0.00000	0.00026
16	0.20653	0.00017	0.00000	0.00002	-0.00014	-0.00146
17	0.56494	0.00000	0.00000	0.00008	0.00000	0.00026
18	0.42513	0.00000	0.00000	0.00007	0.00000	0.00054
19	0.21068	-0.00987	0.01408	0.00023	0.00000	-0.00036
20	0.00000	0.00000	0.00000	0.00011	0.00000	-0.00293
21	0.21067	-0.00068	0.01408	0.00022	0.00000	-0.00036
22	0.21077	-0.00009	0.01395	0.00021	-0.00002	-0.00007

Condition **S14=0.6DL+0.7EQx**

1	0.00000	0.00000	0.00000	-0.00001	-0.00014	-0.00192
2	0.16589	0.00099	0.00000	0.00002	-0.00014	-0.00126
3	0.37321	0.00158	0.00000	0.00000	0.00000	-0.00139
4	0.55824	0.00179	0.00000	0.00005	0.00000	-0.00098
5	0.00000	0.00000	0.00000	-0.00004	0.00014	-0.00192
6	0.16549	-0.00136	0.00000	0.00008	0.00014	-0.00124
7	0.37255	-0.00203	0.00000	-0.00002	0.00000	-0.00136
8	0.55862	-0.00233	0.00000	0.00001	0.00000	-0.00084
9	0.37291	-0.00450	0.00000	0.00003	0.00000	0.00061
10	0.55867	-0.00651	0.00000	0.00007	0.00000	0.00039
11	0.17092	-0.00556	0.00000	0.00010	0.00014	-0.00124
12	0.37321	0.00128	0.00000	0.00001	0.00000	-0.00139
13	0.37289	-0.00591	0.00000	0.00004	0.00000	0.00061
14	0.55832	-0.00040	0.00000	0.00006	0.00000	-0.00098
15	0.55875	-0.00957	0.00000	0.00008	0.00000	0.00039
16	0.16043	0.00014	0.00000	0.00002	-0.00014	-0.00126
17	0.55844	0.00000	0.00000	0.00009	0.00000	0.00039
18	0.37294	0.00000	0.00000	0.00008	0.00000	0.00061
19	0.16506	-0.00988	0.01404	0.00023	0.00000	-0.00024
20	0.00000	0.00000	0.00000	0.00011	0.00000	-0.00232
21	0.16504	-0.00070	0.01404	0.00022	0.00000	-0.00024
22	0.16511	-0.00104	0.01391	0.00021	-0.00002	0.00000

Reactions



Direction of positive forces and moments

	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **DL=Dead Load**						
1	0.12743	2.40190	0.02619	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.05250	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.06534	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.10098	0.00000	0.00000	0.00000
5	0.03928	1.88452	0.13272	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.07582	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.13935	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.02044	0.00000	0.00000	0.00000
17	0.00000	0.57273	-0.00002	0.00000	0.00000	0.00000
18	0.00000	0.64180	-0.00036	0.00000	0.00000	0.00000
20	-0.16671	4.25367	0.12433	0.00000	0.00000	0.00000
SUM	0.00000	9.75463	0.00000	0.00000	0.00000	0.00000
Condition **LL=Live Load**						
1	0.00945	0.14254	0.00001	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.00007	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.00005	0.00000	0.00000	0.00000
4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
5	-0.01451	0.56131	0.00001	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.00006	0.00000	0.00000	0.00000
7	0.00000	0.00000	0.00006	0.00000	0.00000	0.00000
8	0.00000	0.00000	-0.00001	0.00000	0.00000	0.00000
17	0.00000	0.00007	0.00000	0.00000	0.00000	0.00000
18	0.00000	0.97485	0.00000	0.00000	0.00000	0.00000
20	0.00506	1.35123	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	3.03000	0.00000	0.00000	0.00000	0.00000
Condition **SL=Snow Load**						
1	1.38455	26.24058	0.29118	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.67044	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.74510	0.00000	0.00000	0.00000
4	0.00000	0.00000	-1.13771	0.00000	0.00000	0.00000
5	0.49429	12.16879	0.78406	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.85311	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.82294	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.12032	0.00000	0.00000	0.00000
17	0.00000	6.36994	-0.00022	0.00000	0.00000	0.00000
18	0.00000	3.06029	0.04812	0.00000	0.00000	0.00000
20	-1.87884	41.65439	1.49564	0.00000	0.00000	0.00000
SUM	0.00000	89.49400	0.00000	0.00000	0.00000	0.00000

Condition **Wx=Wind in X**

1	-3.48186	-10.50497	0.00000	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.00001	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
4	0.00000	0.00000	0.00001	0.00000	0.00000	0.00000
5	-3.39287	9.32712	0.00000	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.00001	0.00000	0.00000	0.00000
7	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.00001	0.00000	0.00000	0.00000
17	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
18	0.00000	-0.00003	0.00000	0.00000	0.00000	0.00000
20	-7.99427	1.17789	0.00000	0.00000	0.00000	0.00000
SUM	-14.86900	0.00000	0.00000	0.00000	0.00000	0.00000

Condition **EQx=Seismic in X**

1	-1.91768	-8.80800	0.00000	0.00000	0.00000	0.00000
2	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.00003	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.00003	0.00000	0.00000	0.00000
5	-1.87047	7.66068	0.00000	0.00000	0.00000	0.00000
6	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
7	0.00000	0.00000	0.00003	0.00000	0.00000	0.00000
8	0.00000	0.00000	-0.00003	0.00000	0.00000	0.00000
17	0.00000	0.00006	0.00000	0.00000	0.00000	0.00000
18	0.00000	0.00003	0.00000	0.00000	0.00000	0.00000
20	-5.49885	1.14724	0.00000	0.00000	0.00000	0.00000
SUM	-9.28700	0.00000	0.00000	0.00000	0.00000	0.00000

Condition **S1=DL**

1	0.12743	2.40190	0.02619	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.05250	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.06534	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.10098	0.00000	0.00000	0.00000
5	0.03928	1.88452	0.13272	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.07582	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.13935	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.02044	0.00000	0.00000	0.00000
17	0.00000	0.57273	-0.00002	0.00000	0.00000	0.00000
18	0.00000	0.64180	-0.00036	0.00000	0.00000	0.00000
20	-0.16671	4.25367	0.12433	0.00000	0.00000	0.00000
SUM	0.00000	9.75463	0.00000	0.00000	0.00000	0.00000

Condition **S2=DL+LL**

1	0.13688	2.54449	0.02620	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.05285	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.06540	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.10097	0.00000	0.00000	0.00000
5	0.02477	2.44578	0.13273	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.07614	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.13929	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.02043	0.00000	0.00000	0.00000
17	0.00000	0.57280	-0.00002	0.00000	0.00000	0.00000
18	0.00000	1.61665	-0.00015	0.00000	0.00000	0.00000
20	-0.16165	5.60491	0.12466	0.00000	0.00000	0.00000
SUM	0.00000	12.78463	0.00000	0.00000	0.00000	0.00000

Condition **S3=DL+SL**

1	1.51193	28.64127	0.31738	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.74256	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.81061	0.00000	0.00000	0.00000
4	0.00000	0.00000	-1.23869	0.00000	0.00000	0.00000
5	0.53379	14.05409	0.91667	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.94777	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.96259	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.14085	0.00000	0.00000	0.00000
17	0.00000	6.94267	-0.00024	0.00000	0.00000	0.00000
18	0.00000	3.70209	0.06010	0.00000	0.00000	0.00000
20	-2.04572	45.90850	1.64624	0.00000	0.00000	0.00000
SUM	0.00000	99.24863	0.00000	0.00000	0.00000	0.00000

Condition **S4=DL+0.75LL**

1	0.13452	2.50884	0.02620	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.05276	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.06538	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.10097	0.00000	0.00000	0.00000
5	0.02840	2.30547	0.13273	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.07606	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.13931	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.02044	0.00000	0.00000	0.00000
17	0.00000	0.57279	-0.00002	0.00000	0.00000	0.00000
18	0.00000	1.37294	-0.00020	0.00000	0.00000	0.00000
20	-0.16292	5.26710	0.12458	0.00000	0.00000	0.00000
SUM	0.00000	12.02713	0.00000	0.00000	0.00000	0.00000

Condition **S5=DL+0.75SL**

1	1.16584	22.08286	0.24457	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.55025	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.62413	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.95425	0.00000	0.00000	0.00000
5	0.40993	11.01073	0.72071	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.71053	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.75661	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.11070	0.00000	0.00000	0.00000
17	0.00000	5.35018	-0.00019	0.00000	0.00000	0.00000
18	0.00000	2.93702	0.03269	0.00000	0.00000	0.00000
20	-1.57577	35.49434	1.23903	0.00000	0.00000	0.00000
SUM	0.00000	76.87513	0.00000	0.00000	0.00000	0.00000

Condition **S6=DL+0.75LL+0.75SL**

1	1.17289	22.19004	0.24459	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.55267	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.62418	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.95425	0.00000	0.00000	0.00000
5	0.39908	11.43143	0.72072	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.71275	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.75660	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.11070	0.00000	0.00000	0.00000
17	0.00000	5.35024	-0.00019	0.00000	0.00000	0.00000
18	0.00000	3.66816	0.03442	0.00000	0.00000	0.00000
20	-1.57197	36.50776	1.24186	0.00000	0.00000	0.00000
SUM	0.00000	79.14763	0.00000	0.00000	0.00000	0.00000

Condition **S7=DL+0.6Wx**

1	-1.95185	-3.91212	0.02620	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.05232	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.06535	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.10097	0.00000	0.00000	0.00000
5	-2.00567	7.49062	0.13268	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.07626	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.13942	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.02047	0.00000	0.00000	0.00000
17	0.00000	0.57273	-0.00002	0.00000	0.00000	0.00000
18	0.00000	0.64178	-0.00022	0.00000	0.00000	0.00000
20	-4.96388	4.96162	0.12451	0.00000	0.00000	0.00000
SUM	-8.92140	9.75463	0.00000	0.00000	0.00000	0.00000

Condition **S8=DL+0.7EQx**

1	-1.20980	-3.77370	0.02619	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.05240	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.06537	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.10099	0.00000	0.00000	0.00000
5	-1.27452	7.25587	0.13269	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.07621	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.13940	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.02044	0.00000	0.00000	0.00000
17	0.00000	0.57277	-0.00002	0.00000	0.00000	0.00000
18	0.00000	0.64182	-0.00020	0.00000	0.00000	0.00000
20	-4.01658	5.05787	0.12453	0.00000	0.00000	0.00000
SUM	-6.50090	9.75462	0.00000	0.00000	0.00000	0.00000

Condition **S9=DL+0.75LL+0.45Wx+0.75SL**

1	-0.37654	17.39172	0.24466	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.55156	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.62428	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.95426	0.00000	0.00000	0.00000
5	-1.14823	15.69173	0.72055	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.71680	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.75692	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.11079	0.00000	0.00000	0.00000
17	0.00000	5.35024	-0.00019	0.00000	0.00000	0.00000
18	0.00000	3.66814	0.03572	0.00000	0.00000	0.00000
20	-5.16628	37.04580	1.24373	0.00000	0.00000	0.00000
SUM	-6.69105	79.14763	0.00000	0.00000	0.00000	0.00000

Condition **S10=DL+0.525EQx**

1	-0.87348	-2.22979	0.02619	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.05243	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.06536	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.10099	0.00000	0.00000	0.00000
5	-0.94783	5.91304	0.13270	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.07612	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.13938	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.02044	0.00000	0.00000	0.00000
17	0.00000	0.57276	-0.00002	0.00000	0.00000	0.00000
18	0.00000	0.64181	-0.00024	0.00000	0.00000	0.00000
20	-3.05437	4.85680	0.12448	0.00000	0.00000	0.00000
SUM	-4.87568	9.75463	0.00000	0.00000	0.00000	0.00000

Condition **S11=DL+0.75SL**

1	1.16584	22.08286	0.24457	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.55025	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.62413	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.95425	0.00000	0.00000	0.00000
5	0.40993	11.01073	0.72071	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.71053	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.75661	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.11070	0.00000	0.00000	0.00000
17	0.00000	5.35018	-0.00019	0.00000	0.00000	0.00000
18	0.00000	2.93702	0.03269	0.00000	0.00000	0.00000
20	-1.57577	35.49434	1.23903	0.00000	0.00000	0.00000
SUM	0.00000	76.87513	0.00000	0.00000	0.00000	0.00000

Condition **S12=DL+0.525EQx+0.75SL**

1	0.17196	17.39505	0.24463	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.54970	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.62419	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.95427	0.00000	0.00000	0.00000
5	-0.58497	15.08873	0.72057	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.71406	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.75689	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.11077	0.00000	0.00000	0.00000
17	0.00000	5.35021	-0.00019	0.00000	0.00000	0.00000
18	0.00000	2.93703	0.03406	0.00000	0.00000	0.00000
20	-4.46266	36.10410	1.24087	0.00000	0.00000	0.00000
SUM	-4.87568	76.87513	0.00000	0.00000	0.00000	0.00000

Condition **S13=0.6DL+0.6Wx**

1	-2.00302	-4.86843	0.01572	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.03120	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.03921	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.06058	0.00000	0.00000	0.00000
5	-2.02106	6.73287	0.07961	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.04558	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.08365	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.01228	0.00000	0.00000	0.00000
17	0.00000	0.34364	-0.00001	0.00000	0.00000	0.00000
18	0.00000	0.38506	-0.00025	0.00000	0.00000	0.00000
20	-4.89731	3.25964	0.07445	0.00000	0.00000	0.00000
SUM	-8.92140	5.85278	0.00000	0.00000	0.00000	0.00000

Condition **S14=0.6DL+0.7EQx**

1	-1.26100	-4.73043	0.01572	0.00000	0.00000	0.00000
2	0.00000	0.00000	-0.03125	0.00000	0.00000	0.00000
3	0.00000	0.00000	0.03923	0.00000	0.00000	0.00000
4	0.00000	0.00000	-0.06060	0.00000	0.00000	0.00000
5	-1.29005	6.49851	0.07962	0.00000	0.00000	0.00000
6	0.00000	0.00000	-0.04555	0.00000	0.00000	0.00000
7	0.00000	0.00000	-0.08362	0.00000	0.00000	0.00000
8	0.00000	0.00000	0.01226	0.00000	0.00000	0.00000
17	0.00000	0.34368	-0.00001	0.00000	0.00000	0.00000
18	0.00000	0.38510	-0.00024	0.00000	0.00000	0.00000
20	-3.94986	3.35593	0.07447	0.00000	0.00000	0.00000
SUM	-6.50090	5.85278	0.00000	0.00000	0.00000	0.00000

Bentley

Current Date: 5/15/2017 1:42 PM
Units system: English
File name: T:\Structural\2017 Structural Jobs\2017-2259_BA 1606 Yehuda Res\2017-2259.etz\

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+1.6LL
D3=1.2DL+0.5SL
D4=1.2DL+1.6LL+0.5SL
D5=1.2DL+1.6SL
D6=1.2DL+0.5Wx
D7=1.2DL+1.6SL+LL
D8=1.2DL+1.6SL+0.5Wx
D9=1.2DL+Wx
D10=1.2DL+Wx+0.5SL
D11=1.2DL+Wx+LL
D12=1.2DL+Wx+LL+0.5SL
D13=1.2DL+0.2SL
D14=1.2DL+EQx
D15=1.2DL+LL+0.2SL
D16=1.2DL+EQx+0.2SL
D17=1.2DL+EQx+LL
D18=1.2DL+EQx+LL+0.2SL
D19=0.9DL+Wx
D20=0.9DL+EQx

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
Beam	***W 10X54***	**7**	D1 at 75.00%	0.02	OK	Eq. H1-1b
			D10 at 0.00%	0.24	OK	Eq. H1-1b
			D11 at 0.00%	0.29	OK	Eq. H1-1b
			D12 at 0.00%	0.24	OK	Eq. H1-1b
			D13 at 75.00%	0.05	OK	Eq. H1-1b
			D14 at 0.00%	0.20	OK	Eq. H1-1b
			D15 at 75.00%	0.05	OK	Eq. H1-1b
			D16 at 0.00%	0.18	OK	Eq. H1-1b
			D17 at 0.00%	0.20	OK	Eq. H1-1b
			D18 at 0.00%	0.18	OK	Eq. H1-1b
			D19 at 0.00%	0.29	OK	Eq. H1-1b
			D2 at 81.25%	0.02	OK	Eq. H1-1b
			D20 at 0.00%	0.20	OK	Eq. H1-1b
			D3 at 75.00%	0.10	OK	Eq. H1-1b
			D4 at 75.00%	0.11	OK	Eq. H1-1b
			D5 at 75.00%	0.31	OK	Eq. H1-1b
			D6 at 0.00%	0.14	OK	Eq. H1-1b
			D7 at 81.25%	**0.32**	**OK**	Eq. H1-1b
			D8 at 62.50%	0.31	OK	Eq. H1-1b
			D9 at 0.00%	0.29	OK	Eq. H1-1b
		8	D1 at 0.00%	0.01	OK	Eq. H1-1b
			D10 at 0.00%	0.15	OK	Eq. H1-1b
			D11 at 0.00%	0.18	OK	Eq. H1-1b
			D12 at 0.00%	0.15	OK	Eq. H1-1b
			D13 at 0.00%	0.02	OK	Eq. H1-1b
			D14 at 0.00%	0.17	OK	Eq. H1-1b
			D15 at 0.00%	0.02	OK	Eq. H1-1b
			D16 at 0.00%	0.16	OK	Eq. H1-1b

	D17 at 0.00%	0.17	OK	Eq. H1-1b
	D18 at 0.00%	0.16	OK	Eq. H1-1b
	D19 at 0.00%	**0.18**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.01	OK	Eq. H1-1b
	D20 at 0.00%	0.18	OK	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
	D4 at 0.00%	0.04	OK	Eq. H1-1b
	D5 at 0.00%	0.10	OK	Eq. H1-1b
	D6 at 0.00%	0.09	OK	Eq. H1-1b
	D7 at 0.00%	0.11	OK	Eq. H1-1b
	D8 at 68.75%	0.10	OK	Eq. H1-1b
	D9 at 0.00%	0.18	OK	Eq. H1-1b
9	D1 at 100.00%	0.01	OK	Eq. H1-1b
	D10 at 100.00%	0.20	OK	Eq. H1-1b
	D11 at 100.00%	0.18	OK	Eq. H1-1b
	D12 at 100.00%	**0.21**	**OK**	Eq. H1-1b
	D13 at 100.00%	0.02	OK	Eq. H1-1b
	D14 at 100.00%	0.16	OK	Eq. H1-1b
	D15 at 100.00%	0.02	OK	Eq. H1-1b
	D16 at 100.00%	0.18	OK	Eq. H1-1b
	D17 at 100.00%	0.17	OK	Eq. H1-1b
	D18 at 100.00%	0.18	OK	Eq. H1-1b
	D19 at 100.00%	0.17	OK	Eq. H1-1b
	D2 at 100.00%	0.01	OK	Eq. H1-1b
	D20 at 100.00%	0.16	OK	Eq. H1-1b
	D3 at 100.00%	0.04	OK	Eq. H1-1b
	D4 at 100.00%	0.05	OK	Eq. H1-1b
	D5 at 100.00%	0.11	OK	Eq. H1-1b
	D6 at 100.00%	0.09	OK	Eq. H1-1b
	D7 at 100.00%	0.11	OK	Eq. H1-1b
	D8 at 100.00%	0.19	OK	Eq. H1-1b
	D9 at 100.00%	0.17	OK	Eq. H1-1b
10	D1 at 0.00%	0.01	OK	Eq. H1-1b
	D10 at 18.75%	0.07	OK	Eq. H1-1b
	D11 at 0.00%	0.09	OK	Eq. H1-1b
	D12 at 25.00%	0.07	OK	Eq. H1-1b
	D13 at 0.00%	0.02	OK	Eq. H1-1b
	D14 at 0.00%	0.12	OK	Eq. H1-1b
	D15 at 0.00%	0.02	OK	Eq. H1-1b
	D16 at 0.00%	0.11	OK	Eq. H1-1b
	D17 at 0.00%	0.11	OK	Eq. H1-1b
	D18 at 0.00%	0.10	OK	Eq. H1-1b
	D19 at 0.00%	0.10	OK	Eq. H1-1b
	D2 at 0.00%	0.01	OK	Eq. H1-1b
	D20 at 0.00%	**0.12**	**OK**	Eq. H1-1b
	D3 at 0.00%	0.03	OK	Eq. H1-1b
	D4 at 0.00%	0.04	OK	Eq. H1-1b
	D5 at 0.00%	0.10	OK	Eq. H1-1b
	D6 at 0.00%	0.04	OK	Eq. H1-1b
	D7 at 0.00%	0.10	OK	Eq. H1-1b
	D8 at 75.00%	0.08	OK	Eq. H1-1b
	D9 at 0.00%	0.09	OK	Eq. H1-1b
11	D1 at 100.00%	0.01	OK	Eq. H1-1b
	D10 at 100.00%	0.12	OK	Eq. H1-1b
	D11 at 100.00%	0.09	OK	Eq. H1-1b
	D12 at 100.00%	0.12	OK	Eq. H1-1b
	D13 at 100.00%	0.02	OK	Eq. H1-1b
	D14 at 100.00%	0.11	OK	Eq. H1-1b
	D15 at 100.00%	0.02	OK	Eq. H1-1b
	D16 at 100.00%	0.12	OK	Eq. H1-1b

	D17 at 100.00%	0.11	OK	Eq. H1-1b
	D18 at 100.00%	0.12	OK	Eq. H1-1b
	D19 at 100.00%	0.08	OK	Eq. H1-1b
	D2 at 100.00%	0.01	OK	Eq. H1-1b
	D20 at 100.00%	0.10	OK	Eq. H1-1b
	D3 at 100.00%	0.04	OK	Eq. H1-1b
	D4 at 100.00%	0.05	OK	Eq. H1-1b
	D5 at 100.00%	0.12	OK	Eq. H1-1b
	D6 at 100.00%	0.05	OK	Eq. H1-1b
	D7 at 100.00%	0.12	OK	Eq. H1-1b
	D8 at 100.00%	**0.16**	**OK**	Eq. H1-1b
	D9 at 100.00%	0.09	OK	Eq. H1-1b
21	D1 at 0.00%	0.03	OK	Eq. H1-1b
	D10 at 100.00%	0.30	OK	Eq. H1-1b
	D11 at 100.00%	0.29	OK	Eq. H1-1b
	D12 at 100.00%	0.30	OK	Eq. H1-1b
	D13 at 0.00%	0.06	OK	Eq. H1-1b
	D14 at 100.00%	0.21	OK	Eq. H1-1b
	D15 at 0.00%	0.07	OK	Eq. H1-1b
	D16 at 100.00%	0.21	OK	Eq. H1-1b
	D17 at 100.00%	0.21	OK	Eq. H1-1b
	D18 at 100.00%	0.22	OK	Eq. H1-1b
	D19 at 100.00%	0.29	OK	Eq. H1-1b
	D2 at 0.00%	0.03	OK	Eq. H1-1b
	D20 at 100.00%	0.21	OK	Eq. H1-1b
	D3 at 0.00%	0.12	OK	Eq. H1-1b
	D4 at 0.00%	0.13	OK	Eq. H1-1b
	D5 at 0.00%	0.37	OK	Eq. H1-1b
	D6 at 100.00%	0.15	OK	Eq. H1-1b
	D7 at 0.00%	**0.38**	**OK**	Eq. H1-1b
	D8 at 0.00%	0.27	OK	Eq. H1-1b
	D9 at 100.00%	0.29	OK	Eq. H1-1b
23	D1 at 100.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1				
	D10 at 100.00%	0.38	OK	Eq. H1-1b
	D11 at 100.00%	0.23	OK	Eq. H1-1b
	D12 at 100.00%	0.39	OK	Sec. G2, Sec. G2.1(a), T.
B4.1				
	D13 at 100.00%	0.14	OK	Sec. G2, Sec. G2.1(a), T.
B4.1				
	D14 at 100.00%	0.17	OK	Eq. H1-1b
	D15 at 100.00%	0.16	OK	Sec. G2, Sec. G2.1(a), T.
B4.1				
	D16 at 100.00%	0.22	OK	Eq. H1-1b
	D17 at 100.00%	0.17	OK	Eq. H1-1b
	D18 at 100.00%	0.23	OK	Sec. G2, Sec. G2.1(a), T.
B4.1				
	D19 at 100.00%	0.22	OK	Eq. H1-1b
	D2 at 100.00%	0.08	OK	Sec. G2, Sec. G2.1(a), T.
B4.1				
	D20 at 100.00%	0.16	OK	Eq. H1-1b
	D3 at 100.00%	0.27	OK	Sec. G2, Sec. G2.1(a), T.
B4.1				
	D4 at 100.00%	0.30	OK	Sec. G2, Sec. G2.1(a), T.
B4.1				
	D5 at 100.00%	0.75	OK	Sec. G2, Sec. G2.1(a), T.
B4.1				
	D6 at 100.00%	0.13	OK	Eq. H1-1b
	D7 at 100.00%	0.77	OK	Sec. G2, Sec. G2.1(a), T.
B4.1				
	D8 at 100.00%	**0.81**	**OK**	Sec. G2, Sec. G2.1(a), T.

B4.1		D9 at 100.00%	0.23	OK	Eq. H1-1b
<u>Cantilever Beam</u>	**12**	D1 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D10 at 0.00%	0.07	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D11 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D12 at 0.00%	0.07	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D13 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D14 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D15 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D16 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D17 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D18 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D19 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D2 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D20 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D3 at 0.00%	0.07	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D4 at 0.00%	0.07	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D5 at 0.00%	0.18	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D6 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D7 at 0.00%	**0.18**	**OK**	Sec. G2, Sec. G2.1(a), T.
B4.1		D8 at 0.00%	0.18	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D9 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1	**13**	D1 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D10 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D11 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D12 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D13 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D14 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D15 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D16 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D17 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D18 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.

B4.1		D19 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D2 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D20 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D3 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D4 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D5 at 0.00%	0.07	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D6 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D7 at 0.00%	**0.07**	**OK**	Sec. G2, Sec. G2.1(a), T.
B4.1		D8 at 0.00%	0.07	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D9 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1	**14**	D1 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D10 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D11 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D12 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D13 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D14 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D15 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D16 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D17 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D18 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D19 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D2 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D20 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D3 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D4 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D5 at 0.00%	0.07	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D6 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D7 at 0.00%	0.07	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D8 at 0.00%	**0.07**	**OK**	Sec. G2, Sec. G2.1(a), T.
B4.1		D9 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1					

B4.1	15	D1 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D10 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D11 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D12 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D13 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D14 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D15 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D16 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D17 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D18 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D19 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D2 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D20 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D3 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D4 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D5 at 0.00%	0.11	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D6 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D7 at 0.00%	**0.11**	**OK**	Sec. G2, Sec. G2.1(a), T.
B4.1		D8 at 0.00%	0.11	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D9 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1	16	D1 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D10 at 0.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D11 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D12 at 0.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D13 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D14 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D15 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D16 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D17 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D18 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D19 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.

B4.1		D2 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D20 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D3 at 0.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D4 at 0.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D5 at 0.00%	0.16	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D6 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D7 at 0.00%	0.16	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D8 at 0.00%	**0.16**	**OK**	Sec. G2, Sec. G2.1(a), T.
B4.1		D9 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1	**17**	D1 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D10 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D11 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D12 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D13 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D14 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D15 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D16 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D17 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D18 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D19 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D2 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D20 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D3 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D4 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D5 at 0.00%	**0.07**	**OK**	Sec. G2, Sec. G2.1(a), T.
B4.1		D6 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D7 at 0.00%	0.07	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D8 at 0.00%	0.07	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D9 at 0.00%	0.00	OK	Sec. G2, Sec. G2.1(a), T.
B4.1	**18**	D1 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
		D10 at 0.00%	0.08	OK	Sec. G2, Sec. G2.1(a), T.

B4.1		D11 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D12 at 0.00%	0.08	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D13 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D14 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D15 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D16 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D17 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D18 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D19 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D2 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D20 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D3 at 0.00%	0.08	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D4 at 0.00%	0.08	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D5 at 0.00%	0.21	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D6 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D7 at 0.00%	0.21	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D8 at 0.00%	**0.21**	**OK**	Sec. G2, Sec. G2.1(a), T.
B4.1		D9 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1					
	19	D1 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D10 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D11 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D12 at 0.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D13 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D14 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D15 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D16 at 0.00%	0.02	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D17 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D18 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D19 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D2 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D20 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.

B4.1		D3 at 0.00%	0.04	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D4 at 0.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D5 at 0.00%	0.11	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D6 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D7 at 0.00%	**0.12**	**OK**	Sec. G2, Sec. G2.1(a), T.
B4.1		D8 at 0.00%	0.11	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D9 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1	**20**	D1 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D10 at 0.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D11 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D12 at 0.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D13 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D14 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D15 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D16 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D17 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D18 at 0.00%	0.03	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D19 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D2 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D20 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D3 at 0.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D4 at 0.00%	0.06	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D5 at 0.00%	**0.16**	**OK**	Sec. G2, Sec. G2.1(a), T.
B4.1		D6 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D7 at 0.00%	0.16	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D8 at 0.00%	0.16	OK	Sec. G2, Sec. G2.1(a), T.
B4.1		D9 at 0.00%	0.01	OK	Sec. G2, Sec. G2.1(a), T.
B4.1					
Column	**1**	D1 at 100.00%	0.01	OK	Eq. H1-1b
		D10 at 100.00%	0.15	OK	Eq. H1-1b
		D11 at 100.00%	0.17	OK	Eq. H1-1b
		D12 at 100.00%	0.15	OK	Eq. H1-1b
		D13 at 100.00%	0.03	OK	Eq. H1-1b
		D14 at 100.00%	0.09	OK	Eq. H1-1b

	D15 at 100.00%	0.03	OK	Eq. H1-1b
	D16 at 100.00%	0.08	OK	Eq. H1-1b
	D17 at 100.00%	0.09	OK	Eq. H1-1b
	D18 at 100.00%	0.08	OK	Eq. H1-1b
	D19 at 100.00%	0.17	OK	Eq. H1-1b
	D2 at 100.00%	0.01	OK	Eq. H1-1b
	D20 at 100.00%	0.10	OK	Eq. H1-1b
	D3 at 100.00%	0.06	OK	Eq. H1-1b
	D4 at 100.00%	0.06	OK	Eq. H1-1b
	D5 at 100.00%	0.22	OK	Eq. H1-1b
	D6 at 100.00%	0.08	OK	Eq. H1-1b
	D7 at 100.00%	**0.22**	**OK**	Eq. H1-1b
	D8 at 100.00%	0.13	OK	Eq. H1-1b
	D9 at 100.00%	0.17	OK	Eq. H1-1b
2	D1 at 0.00%	0.01	OK	Eq. H1-1b
	D10 at 100.00%	0.15	OK	Eq. H1-1b
	D11 at 100.00%	0.15	OK	Eq. H1-1b
	D12 at 100.00%	0.15	OK	Eq. H1-1b
	D13 at 0.00%	0.02	OK	Eq. H1-1b
	D14 at 0.00%	0.12	OK	Eq. H1-1b
	D15 at 0.00%	0.02	OK	Eq. H1-1b
	D16 at 0.00%	0.12	OK	Eq. H1-1b
	D17 at 0.00%	0.12	OK	Eq. H1-1b
	D18 at 0.00%	0.12	OK	Eq. H1-1b
	D19 at 100.00%	**0.15**	**OK**	Eq. H1-1b
	D2 at 0.00%	0.01	OK	Eq. H1-1b
	D20 at 0.00%	0.12	OK	Eq. H1-1b
	D3 at 0.00%	0.04	OK	Eq. H1-1b
	D4 at 0.00%	0.04	OK	Eq. H1-1b
	D5 at 0.00%	0.14	OK	Eq. H1-1b
	D6 at 100.00%	0.08	OK	Eq. H1-1b
	D7 at 0.00%	0.14	OK	Eq. H1-1b
	D8 at 100.00%	0.10	OK	Eq. H1-1b
	D9 at 100.00%	0.15	OK	Eq. H1-1b
3	D1 at 100.00%	0.02	OK	Eq. H1-1b
	D10 at 100.00%	0.13	OK	Eq. H1-1b
	D11 at 100.00%	0.10	OK	Eq. H1-1b
	D12 at 100.00%	0.13	OK	Eq. H1-1b
	D13 at 100.00%	0.05	OK	Eq. H1-1b
	D14 at 100.00%	0.13	OK	Eq. H1-1b
	D15 at 100.00%	0.05	OK	Eq. H1-1b
	D16 at 100.00%	0.13	OK	Eq. H1-1b
	D17 at 100.00%	0.12	OK	Eq. H1-1b
	D18 at 100.00%	0.13	OK	Eq. H1-1b
	D19 at 100.00%	0.10	OK	Eq. H1-1b
	D2 at 100.00%	0.02	OK	Eq. H1-1b
	D20 at 100.00%	0.13	OK	Eq. H1-1b
	D3 at 100.00%	0.10	OK	Eq. H1-1b
	D4 at 100.00%	0.10	OK	Eq. H1-1b
	D5 at 100.00%	0.27	OK	Eq. H1-1b
	D6 at 100.00%	0.05	OK	Eq. H1-1b
	D7 at 100.00%	**0.28**	**OK**	Eq. H1-1b
	D8 at 100.00%	0.22	OK	Eq. H1-1b
	D9 at 100.00%	0.10	OK	Eq. H1-1b
4	D1 at 100.00%	0.02	OK	Eq. H1-1b
	D10 at 100.00%	0.23	OK	Eq. H1-1b
	D11 at 100.00%	0.18	OK	Eq. H1-1b
	D12 at 100.00%	**0.23**	**OK**	Eq. H1-1b
	D13 at 100.00%	0.04	OK	Eq. H1-1b
	D14 at 100.00%	0.11	OK	Eq. H1-1b

	D15 at 100.00%	0.04	OK	Eq. H1-1b
	D16 at 100.00%	0.12	OK	Eq. H1-1b
	D17 at 100.00%	0.11	OK	Eq. H1-1b
	D18 at 100.00%	0.12	OK	Eq. H1-1b
	D19 at 100.00%	0.18	OK	Eq. H1-1b
	D2 at 100.00%	0.02	OK	Eq. H1-1b
	D20 at 100.00%	0.10	OK	Eq. H1-1b
	D3 at 100.00%	0.08	OK	Eq. H1-1b
	D4 at 100.00%	0.08	OK	Eq. H1-1b
	D5 at 100.00%	0.21	OK	Eq. H1-1b
	D6 at 100.00%	0.10	OK	Eq. H1-1b
	D7 at 100.00%	0.21	OK	Eq. H1-1b
	D8 at 100.00%	0.22	OK	Eq. H1-1b
	D9 at 100.00%	0.18	OK	Eq. H1-1b
5	D1 at 0.00%	0.02	OK	Eq. H1-1b
	D10 at 0.00%	0.20	OK	Eq. H1-1b
	D11 at 100.00%	0.16	OK	Eq. H1-1b
	D12 at 0.00%	0.20	OK	Eq. H1-1b
	D13 at 0.00%	0.04	OK	Eq. H1-1b
	D14 at 0.00%	0.14	OK	Eq. H1-1b
	D15 at 0.00%	0.04	OK	Eq. H1-1b
	D16 at 0.00%	0.16	OK	Eq. H1-1b
	D17 at 0.00%	0.14	OK	Eq. H1-1b
	D18 at 0.00%	0.16	OK	Eq. H1-1b
	D19 at 100.00%	0.16	OK	Eq. H1-1b
	D2 at 0.00%	0.02	OK	Eq. H1-1b
	D20 at 0.00%	0.13	OK	Eq. H1-1b
	D3 at 0.00%	0.07	OK	Eq. H1-1b
	D4 at 0.00%	0.07	OK	Eq. H1-1b
	D5 at 0.00%	0.18	OK	Eq. H1-1b
	D6 at 100.00%	0.08	OK	Eq. H1-1b
	D7 at 0.00%	0.19	OK	Eq. H1-1b
	D8 at 0.00%	**0.25**	**OK**	Eq. H1-1b
	D9 at 100.00%	0.16	OK	Eq. H1-1b
6	D1 at 100.00%	0.01	OK	Eq. H1-1b
	D10 at 100.00%	0.13	OK	Eq. H1-1b
	D11 at 100.00%	0.09	OK	Eq. H1-1b
	D12 at 100.00%	0.13	OK	Eq. H1-1b
	D13 at 100.00%	0.02	OK	Eq. H1-1b
	D14 at 100.00%	0.11	OK	Eq. H1-1b
	D15 at 100.00%	0.02	OK	Eq. H1-1b
	D16 at 100.00%	0.12	OK	Eq. H1-1b
	D17 at 100.00%	0.11	OK	Eq. H1-1b
	D18 at 100.00%	0.12	OK	Eq. H1-1b
	D19 at 100.00%	0.09	OK	Eq. H1-1b
	D2 at 0.00%	0.01	OK	Eq. H1-1b
	D20 at 100.00%	0.11	OK	Eq. H1-1b
	D3 at 100.00%	0.05	OK	Eq. H1-1b
	D4 at 100.00%	0.05	OK	Eq. H1-1b
	D5 at 100.00%	0.13	OK	Eq. H1-1b
	D6 at 100.00%	0.05	OK	Eq. H1-1b
	D7 at 100.00%	0.13	OK	Eq. H1-1b
	D8 at 100.00%	**0.17**	**OK**	Eq. H1-1b
	D9 at 100.00%	0.09	OK	Eq. H1-1b
22	D1 at 100.00%	0.03	OK	Eq. H1-1b
	D10 at 100.00%	0.56	OK	Eq. H1-1b
	D11 at 100.00%	0.41	OK	Eq. H1-1b
	D12 at 100.00%	0.56	OK	Eq. H1-1b
	D13 at 100.00%	0.07	OK	Eq. H1-1b
	D14 at 100.00%	0.29	OK	Eq. H1-1b

D15 at 100.00%	0.07	OK	Eq. H1-1b
D16 at 100.00%	0.34	OK	Eq. H1-1b
D17 at 100.00%	0.29	OK	Eq. H1-1b
D18 at 100.00%	0.34	OK	Eq. H1-1b
D19 at 100.00%	0.40	OK	Eq. H1-1b
D2 at 100.00%	0.02	OK	Eq. H1-1b
D20 at 100.00%	0.28	OK	Eq. H1-1b
D3 at 100.00%	0.17	OK	Eq. H1-1b
D4 at 100.00%	0.17	OK	Eq. H1-1b
D5 at 100.00%	0.48	OK	Eq. H1-1b
D6 at 100.00%	0.22	OK	Eq. H1-1b
D7 at 100.00%	0.48	OK	Eq. H1-1b
D8 at 100.00%	**0.68**	**OK**	Eq. H1-1b
D9 at 100.00%	0.41	OK	Eq. H1-1b

Bentley

Current Date: 5/15/2017 1:43 PM
Units system: English
File name: T:\Structural\2017 Structural Jobs\2017-2259_BA 1606 Yehuda Res\2017-2259.etz\

Steel connections

Results

Connection name : **DW BCF**
Connection ID : **14**

Family: Beam - Column flange (BCF)
Type: Directly welded flanges
Description: Smart DW 1

Design code: AISC 360-10 LRFD

DEMANDS

	Beam			Right beam		Left beam		Column	Panel	
Description	**Ru** [Kip]	**Pu** [Kip]	**Mu** [Kip*ft]	**PufTop** [Kip]	**PufBot** [Kip]	**PufTop** [Kip]	**PufBot** [Kip]	**Pu** [Kip]	**Vu** [Kip]	**Load type**
DL	0.00	-0.04	-1.57	1.97	-2.01	0.00	0.00	-2.40	1.88	Design
LL	0.00	0.03	-0.31	0.41	-0.38	0.00	0.00	-0.14	0.41	Design
SL	0.00	-0.55	-16.25	20.28	-20.83	0.00	0.00	-26.24	19.44	Design
Wx	0.00	-6.81	48.20	-64.39	57.58	0.00	0.00	10.50	60.90	Design
EQx	0.00	-3.66	34.62	-45.63	41.97	0.00	0.00	8.81	43.71	Design
D1	0.00	-0.05	-2.20	2.76	-2.81	0.00	0.00	-3.36	2.63	Design
D2	0.00	0.01	-2.39	3.03	-3.02	0.00	0.00	-3.11	2.86	Design
D3	0.00	-0.32	-10.01	12.51	-12.83	0.00	0.00	-16.00	11.98	Design
D4	0.00	-0.26	-10.52	13.17	-13.44	0.00	0.00	-16.23	12.58	Design
D5	0.00	-0.92	-27.84	34.76	-35.68	0.00	0.00	-44.86	33.31	Design
D6	0.00	-3.46	22.27	-29.90	26.45	0.00	0.00	2.38	28.33	Design
D7	0.00	-0.89	-28.16	35.18	-36.07	0.00	0.00	-45.00	33.69	Design
D8	0.00	-4.37	-2.98	1.59	-5.96	0.00	0.00	-39.45	5.30	Design
D9	0.00	-6.86	46.42	-62.16	55.31	0.00	0.00	7.64	58.83	Design
D10	0.00	-7.15	38.73	-52.57	45.42	0.00	0.00	-5.39	49.95	Design
D11	0.00	-6.82	46.13	-61.78	54.95	0.00	0.00	7.51	58.46	Design
D12	0.00	-7.12	38.44	-52.19	45.07	0.00	0.00	-5.52	49.57	Design
D13	0.00	-0.15	-5.14	6.42	-6.58	0.00	0.00	-8.13	6.15	Design
D14	0.00	-3.71	32.81	-43.37	39.66	0.00	0.00	5.94	41.60	Design
D15	0.00	-0.12	-5.45	6.84	-6.96	0.00	0.00	-8.27	6.52	Design
D16	0.00	-3.83	29.68	-39.47	35.64	0.00	0.00	0.72	37.99	Design
D17	0.00	-3.68	32.51	-42.98	39.30	0.00	0.00	5.80	41.22	Design
D18	0.00	-3.79	29.39	-39.08	35.28	0.00	0.00	0.58	37.60	Design
D19	0.00	-6.84	46.87	-62.72	55.87	0.00	0.00	8.36	59.35	Design
D20	0.00	-3.70	33.26	-43.93	40.23	0.00	0.00	6.66	42.13	Design

GEOMETRIC CONSIDERATIONS

Dimensions	**Unit**	**Value**	**Min. value**	**Max. value**	**Sta.**	**References**
Transverse stiffeners						
Length	[in]	8.87	4.43	--	✔	Sec. J10.8
Width	[in]	4.50	3.15	--	✔	Sec. J10.8
Thickness	[in]	0.38	0.31	--	✔	Sec. J10.8
Weld size	[1/16in]	4	3	--	✔	DG 13 Eq. 4.3-6
Doublers						
Recommended thickness for beveling and welding	[in]	0.50	0.26	--	✔	Sec. G2.1,

WARNINGS

- Width of beam flange should be shorter than available with on support

DESIGN CHECK

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Support						
Panel web shear	[Kip]	327.58	60.90	Wx	0.19	Sec. J10-6, Eq. J10-11
Support - right side						
Top local flange bending	[Kip]	191.43	35.18	D7	0.18	Eq. J10-1
Bottom local flange bending	[Kip]	191.43	57.58	Wx	0.30	Eq. J10-1
Local web yielding	[Kip]	423.77	64.39	Wx	0.15	Eq. J10-2
Transverse stiffeners - top						
Yielding strength due to axial load	[Kip]	85.05	0.00	DL	0.00	Eq. J4-1
Compression	[Kip]	73.25	0.00	DL	0.00	Sec. J4.4
Flange weld capacity	[Kip]	108.59	0.00	DL	0.00	Eq. J2-4
Web weld capacity	[Kip]	169.73	0.00	DL	0.00	Eq. J2-4
Transverse stiffeners - bottom						
Yielding strength due to axial load	[Kip]	85.05	0.00	DL	0.00	Eq. J4-1
Compression	[Kip]	73.25	0.00	DL	0.00	Sec. J4.4
Flange weld capacity	[Kip]	108.59	0.00	DL	0.00	Eq. J2-4
Web weld capacity	[Kip]	169.73	0.00	DL	0.00	Eq. J2-4

Global critical strength ratio **0.30**

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Steel connections

Results

Connection name : DW BCF
Connection ID : 14

Family: Beam - Column flange (BCF)
Type: Directly welded flanges
Description: Smart DW 1

Design code: AISC 360-10 LRFD, AISC 341-10 LRFD

DEMANDS

	Beam			Right beam		Left beam		Column	Panel	
Description	Ru [Kip]	Pu [Kip]	Mu [Kip*ft]	PufTop [Kip]	PufBot [Kip]	PufTop [Kip]	PufBot [Kip]	Pu [Kip]	Vu [Kip]	Load type
DL	0.00	-0.04	-1.57	1.97	-2.01	0.00	0.00	-2.40	341.70	Design
LL	0.00	0.03	-0.31	0.41	-0.38	0.00	0.00	-0.14	341.70	Design
SL	0.00	-0.55	-16.25	20.28	-20.83	0.00	0.00	-26.24	341.70	Design
Wx	0.00	-6.81	48.20	-64.39	57.58	0.00	0.00	10.50	341.70	Design
EQx	0.00	-3.66	34.62	-45.63	41.97	0.00	0.00	8.81	341.70	Design
D1	0.00	-0.05	-2.20	2.76	-2.81	0.00	0.00	-3.36	341.70	Design
D2	0.00	0.01	-2.39	3.03	-3.02	0.00	0.00	-3.11	341.70	Design
D3	0.00	-0.32	-10.01	12.51	-12.83	0.00	0.00	-16.00	341.70	Design
D4	0.00	-0.26	-10.52	13.17	-13.44	0.00	0.00	-16.23	341.70	Design
D5	0.00	-0.92	-27.84	34.76	-35.68	0.00	0.00	-44.86	341.70	Design
D6	0.00	-3.46	22.27	-29.90	26.45	0.00	0.00	2.38	341.70	Design
D7	0.00	-0.89	-28.16	35.18	-36.07	0.00	0.00	-45.00	341.70	Design
D8	0.00	-4.37	-2.98	1.59	-5.96	0.00	0.00	-39.45	341.70	Design
D9	0.00	-6.86	46.42	-62.16	55.31	0.00	0.00	7.64	341.70	Design
D10	0.00	-7.15	38.73	-52.57	45.42	0.00	0.00	-5.39	341.70	Design
D11	0.00	-6.82	46.13	-61.78	54.95	0.00	0.00	7.51	341.70	Design
D12	0.00	-7.12	38.44	-52.19	45.07	0.00	0.00	-5.52	341.70	Design
D13	0.00	-0.15	-5.14	6.42	-6.58	0.00	0.00	-8.13	341.70	Design
D14	0.00	-3.71	32.81	-43.37	39.66	0.00	0.00	5.94	341.70	Design
D15	0.00	-0.12	-5.45	6.84	-6.96	0.00	0.00	-8.27	341.70	Design
D16	0.00	-3.83	29.68	-39.47	35.64	0.00	0.00	0.72	341.70	Design
D17	0.00	-3.68	32.51	-42.98	39.30	0.00	0.00	5.80	341.70	Design
D18	0.00	-3.79	29.39	-39.08	35.28	0.00	0.00	0.58	341.70	Design
D19	0.00	-6.84	46.87	-62.72	55.87	0.00	0.00	8.36	341.70	Design
D20	0.00	-3.70	33.26	-43.93	40.23	0.00	0.00	6.66	341.70	Design

GEOMETRIC CONSIDERATIONS

Dimensions	Unit	Value	Min. value	Max. value	Sta.	References
Transverse stiffeners						
Length	[in]	8.87	4.43	--	✔	Sec. J10.8
Width	[in]	4.50	3.15	--	✔	Sec. J10.8
Thickness	[in]	0.38	0.31	--	✔	Sec. J10.8
Weld size	[1/16in]	4	3	--	✔	DG 13 Eq. 4.3-6
Doublers						
Recommended thickness for beveling and welding	[in]	0.50	0.26	--	✔	Sec. G2.1,

SEISMIC PREQUALIFICATION REQUIREMENTS (ANSI/AISC 358-10)

Beam					
Beam weight	[Kip/ft]	0.05	--	0.30	✔
Reduced beam section (RBS)					
Horizontal distance to start of RBS cut (a)	[in]	6.00	5.00	7.50	✔
Length of RBS cut (b)	[in]	7.60	6.57	8.58	✔
Length of RBS cut (b)	[in]	2.00	1.00	2.50	✔

WARNINGS

- Width of beam flange should be shorter than available with on support

Requirement	Value	Allowable values	Sta.
Beam			No
Material	A992	A36, A529, A572 Grade 42/50/55, A588, A913 Grade 50/60/65, A992	Yes
Support			No
Material	A992	A36, A529, A572 Grade 42/50/55, A588, A913 Grade 50/60/65, A992	Yes

Protected zone from column face = 13.6 [in]

DESIGN CHECK

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Panel web shear	[Kip]	363.97	341.70	DL	0.94	Sec. J10-6, Eq. J10-11
Support - right side						
Top local flange bending	[Kip]	212.70	35.18	D7	0.17	Eq. J10-1
Bottom local flange bending	[Kip]	212.70	57.58	Wx	0.27	Eq. J10-1
Local web yielding	[Kip]	433.22	64.39	Wx	0.15	Eq. J10-2
Transverse stiffeners - top						
Yielding strength due to axial load	[Kip]	94.50	0.00	DL	0.00	Eq. J4-1
Compression	[Kip]	81.39	0.00	DL	0.00	Sec. J4.4
Flange weld capacity	[Kip]	130.30	0.00	DL	0.00	Eq. J2-4
Web weld capacity	[Kip]	203.67	0.00	DL	0.00	Eq. J2-4
Transverse stiffeners - bottom						
Yielding strength due to axial load	[Kip]	94.50	0.00	DL	0.00	Eq. J4-1
Compression	[Kip]	81.39	0.00	DL	0.00	Sec. J4.4
Flange weld capacity	[Kip]	130.30	0.00	DL	0.00	Eq. J2-4
Web weld capacity	[Kip]	203.67	0.00	DL	0.00	Eq. J2-4
Seismic forces						
Mf vs. Mpe at column face	[Kip*ft]	305.25	270.08	DL	0.88	AISC 358-10 Eq. 5.8-7, AISC 358-05 Eq. 2.4.3-1, AISC 358-05 Eq. 5.8-6
Mpr: Probable peak plastic hinge moment	[Kip*ft]	228.05				AISC 358-05 Eq. 2.4.3-1
Mc: Maximum probable moment at column centerline	[Kip*ft]	291.74				AISC 358-05 Eq. 2.4.3-1
Vp: Plastic hinge shear force	[Kip]	51.47				AISC 358-10 Eq. 5.8-9
Mf: Maximum probable moment at column face	[Kip*ft]	270.08				AISC 358-05 Eq. 2.4.3-1, AISC 358-05 Eq. 5.8-6

Global critical strength ratio **0.94**

NOTES

CJP groove welds are required for the beam web to column connection, Sec. 5.6 (a) of AISC 358

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Steel connections

Results

Connection name : **SP_BCF_1/4PL_2B3/4**
Connection ID : **1**

Family: Beam - Column flange (BCF)
Type: Single plate
Description: Basic SP 2

Design code: AISC 360-10 LRFD

DEMANDS

	Beam		Column			
Description	**Ru** [Kip]	**Pu** [Kip]	**Pu** [Kip]	**Mu22** [Kip*ft]	**Mu33** [Kip*ft]	**Load type**
DL	0.79	-0.04	-2.40	0.00	1.02	Design
LL	0.06	0.03	-0.14	0.00	0.08	Design
SL	8.62	-0.55	-26.24	0.00	11.08	Design
Wx	-6.36	-6.81	10.50	0.00	-27.55	Design
EQx	-4.47	-3.66	8.81	0.00	-15.17	Design
D1	1.11	-0.05	-3.36	0.00	1.43	Design
D2	1.05	0.01	-3.11	0.00	1.34	Design
D3	5.26	-0.32	-16.00	0.00	6.76	Design
D4	5.36	-0.26	-16.23	0.00	6.89	Design
D5	14.74	-0.92	-44.86	0.00	18.96	Design
D6	-2.23	-3.46	2.38	0.00	-12.59	Design
D7	14.80	-0.89	-45.00	0.00	19.04	Design
D8	11.46	-4.37	-39.45	0.00	4.66	Design
D9	-5.42	-6.86	7.64	0.00	-26.40	Design
D10	-1.17	-7.15	-5.39	0.00	-21.16	Design
D11	-5.36	-6.82	7.51	0.00	-26.34	Design
D12	-1.11	-7.12	-5.52	0.00	-21.10	Design
D13	2.68	-0.15	-8.13	0.00	3.44	Design
D14	-3.53	-3.71	5.94	0.00	-13.99	Design
D15	2.74	-0.12	-8.27	0.00	3.52	Design
D16	-1.82	-3.83	0.72	0.00	-11.85	Design
D17	-3.47	-3.68	5.80	0.00	-13.93	Design
D18	-1.76	-3.79	0.58	0.00	-11.79	Design
D19	-5.65	-6.84	8.36	0.00	-26.69	Design
D20	-3.76	-3.70	6.66	0.00	-14.29	Design

GEOMETRIC CONSIDERATIONS

Dimensions	**Unit**	**Value**	**Min. value**	**Max. value**	**Sta.**	**References**
Shear plate						
Length	[in]	6.00	3.93	7.86	✔	p. 10-104
Thickness	[in]	0.38	--	0.44	✔	p. 10-102
Number of bolts		2	2	12	✔	p 10-102
Distance from the bolt line to the weld line	[in]	3.00	--	3.50	✔	p 10-102
Minimum plate or beam web thickness	[in]	0.37	--	0.44	✔	Table 10-9
Vertical edge distance	[in]	1.50	1.00	--	✔	Tables J3.4,

						J3.5
Horizontal edge distance	[in]	1.50	1.50	--	✔	p. 10-103
Vertical center-to-center spacing (pitch)	[in]	3.00	2.00	8.88	✔	Sec. J3.3, Sec. J3.5
Beam						
Vertical edge distance	[in]	3.55	1.00	--	✔	Tables J3.4, J3.5
Horizontal edge distance	[in]	2.50	1.50	--	✔	p. 10-103
Support						
Weld size	[1/16in]	4	4	--	✔	p. 10-101
Weld length	[in]	6.00	1.00	--	✔	Sec. J2.2b

DESIGN CHECK

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Shear plate						
Bolts shear	[Kip]	24.75	14.83	D7	0.60	Tables (7-1..14)
Bolt bearing under shear load	[Kip]	29.60	14.80	D7	0.50	Eq. J3-6, p. 7-18
Shear yielding	[Kip]	48.60	14.80	D7	0.30	Eq. J4-3
Shear rupture	[Kip]	41.60	14.80	D7	0.36	Eq. J4-4
Block shear	[Kip]	44.67	14.80	D7	0.33	Eq. J4-5
Bolt bearing under axial load	[Kip]	28.65	0.03	LL	0.00	Eq. J3-6, p. 7-18
Tension yielding	[Kip]	72.90	0.03	LL	0.00	Eq. J4-1
Tension rupture	[Kip]	69.33	0.03	LL	0.00	Eq. J4-2
Tear out under axial load	[Kip]	52.89	0.03	LL	0.00	Eq. J4-5
Plate (support side)						
Weld capacity	[Kip]	67.31	14.83	D7	0.22	Tables 8-4 .. 8-11
Web crippling	[Kip]	180.54	7.15	D10	0.04	Eq. J10-4
Beam						
Bolt bearing under shear load	[Kip]	44.89	14.80	D7	0.33	Eq. J3-6, p. 7-18
Shear yielding	[Kip]	112.11	14.80	D7	0.13	Eq. J4-3
Bolt bearing under axial load	[Kip]	64.94	0.03	LL	0.00	Eq. J3-6
Yielding strength due to axial load	[Kip]	711.00	0.03	LL	0.00	Eq. D2-1
Tension rupture	[Kip]	247.97	0.03	LL	0.00	Eq. J4-2
Tear out under axial load	[Kip]	79.95	0.03	LL	0.00	Eq. J4-5
Support						
Welds rupture	[Kip/ft]	287.82	19.63	D7	0.07	p. 9-5

Global critical strength ratio **0.60**

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Steel connections

Results

Connection name : SP_BCF_1/2PL_3B1
Connection ID : 13

Family: Beam - Column flange (BCF)
Type: Single plate
Description: Basic SP 2

Design code: AISC 360-10 LRFD

DEMANDS

	Beam		Column			
Description	**Ru** [Kip]	**Pu** [Kip]	**Pu** [Kip]	**Mu22** [Kip*ft]	**Mu33** [Kip*ft]	**Load type**
DL	-3.35	-0.04	-4.25	0.00	-1.33	Design
LL	-1.06	0.03	-1.35	0.00	0.04	Design
SL	-32.78	-0.54	-41.65	0.00	-15.02	Design
Wx	-7.56	-6.80	-1.18	0.00	-63.99	Design
EQx	-5.66	-3.66	-1.15	0.00	-44.01	Design
D1	-4.68	-0.05	-5.96	0.00	-1.87	Design
D2	-5.72	0.01	-7.27	0.00	-1.53	Design
D3	-20.40	-0.31	-25.93	0.00	-9.11	Design
D4	-22.10	-0.26	-28.09	0.00	-9.04	Design
D5	-56.46	-0.90	-71.75	0.00	-25.63	Design
D6	-7.80	-3.45	-5.69	0.00	-33.67	Design
D7	-57.53	-0.87	-73.11	0.00	-25.58	Design
D8	-60.36	-4.28	-72.36	0.00	-58.64	Design
D9	-11.59	-6.84	-6.28	0.00	-65.73	Design
D10	-28.05	-7.10	-27.12	0.00	-73.82	Design
D11	-12.66	-6.81	-7.64	0.00	-65.72	Design
D12	-29.11	-7.06	-28.47	0.00	-73.81	Design
D13	-10.57	-0.15	-13.44	0.00	-4.60	Design
D14	-9.69	-3.70	-6.25	0.00	-45.72	Design
D15	-11.63	-0.12	-14.79	0.00	-4.56	Design
D16	-16.26	-3.80	-14.59	0.00	-48.88	Design
D17	-10.75	-3.66	-7.61	0.00	-45.70	Design
D18	-17.33	-3.77	-15.94	0.00	-48.86	Design
D19	-10.58	-6.83	-5.01	0.00	-65.30	Design
D20	-8.68	-3.69	-4.98	0.00	-45.29	Design

GEOMETRIC CONSIDERATIONS

Dimensions	**Unit**	**Value**	**Min. value**	**Max. value**	**Sta.**	**References**
<u>Shear plate</u>						
Length	[in]	7.50	3.93	7.86	✔	p. 10-104
Thickness	[in]	0.63	--	0.69	✔	p. 10-102
Number of bolts		2	2	12	✔	p 10-102
Distance from the bolt line to the weld line	[in]	3.00	--	3.50	✔	p 10-102
Minimum plate or beam web thickness	[in]	0.37	--	0.69	✔	Table 10-9
Vertical edge distance	[in]	2.00	1.63	--	✔	Tables J3.4,

						J3.5
Horizontal edge distance	[in]	2.50	2.50	--	✔	p. 10-103
Vertical center-to-center spacing (pitch)	[in]	3.50	3.33	8.88	✔	Sec. J3.3, Sec. J3.5
Beam						
Vertical edge distance	[in]	3.30	1.63	--	✔	Tables J3.4, J3.5
Horizontal edge distance	[in]	2.50	2.50	--	✔	p. 10-103
Support						
Weld size	[1/16in]	7	7	--	✔	p. 10-101
Weld length	[in]	7.50	1.75	--	✔	Sec. J2.2b

DESIGN CHECK

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Shear plate						
Bolts shear	[Kip]	73.95	60.51	D8	0.82	Tables (7-1..14)
Bolt bearing under shear load	[Kip]	65.08	60.36	D8	0.93	Eq. J3-6, p. 7-18
Shear yielding	[Kip]	101.25	60.36	D8	0.60	Eq. J4-3
Shear rupture	[Kip]	77.48	60.36	D8	0.78	Eq. J4-4
Block shear	[Kip]	104.96	60.36	D8	0.58	Eq. J4-5
Bolt bearing under axial load	[Kip]	89.50	0.03	LL	0.00	Eq. J3-6, p. 7-18
Tension yielding	[Kip]	151.88	0.03	LL	0.00	Eq. J4-1
Tension rupture	[Kip]	129.14	0.03	LL	0.00	Eq. J4-2
Tear out under axial load	[Kip]	108.40	0.03	LL	0.00	Eq. J4-5
Plate (support side)						
Weld capacity	[Kip]	147.55	60.51	D8	0.41	Tables 8-4 .. 8-11
Web crippling	[Kip]	189.38	7.10	D10	0.04	Eq. J10-4
Beam						
Bolt bearing under shear load	[Kip]	70.29	60.36	D8	0.86	Eq. J3-6, p. 7-18
Shear yielding	[Kip]	112.11	60.36	D8	0.54	Eq. J4-3
Bolt bearing under axial load	[Kip]	79.82	0.03	LL	0.00	Eq. J3-6
Yielding strength due to axial load	[Kip]	711.00	0.03	LL	0.00	Eq. D2-1
Tension rupture	[Kip]	310.31	0.03	LL	0.00	Eq. J4-2
Tear out under axial load	[Kip]	77.56	0.03	LL	0.00	Eq. J4-5
Support						
Welds rupture	[Kip/ft]	287.82	63.95	D8	0.22	p. 9-5
Global critical strength ratio	**0.93**					

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

Project Title:
Engineer:
Project Descr:

Project ID:

Steel Base Plate

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ENERCALC, INC. 1983-2016, Build:6.16.5.11, Ver:6.16.5.11
Lic. # : KW-06004645
Licensee : LEI CONSULTING ENGINEERS

Description : BP

Code References

Calculations per AISC Design Guide # 1, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : IBC 2015

General Information

Material Properties

AISC Design Method Load Resistance Factor Design

Φ_c : LRFD Resistance Factor 0.60

Steel Plate Fy = 36.0 ksi
Concrete Support f'c = 2.50 ksi
Assumed Bearing Area :Full Bearing

Allowable Bearing Fp per J8 1.50 ksi

Column & Plate

Column Properties

Steel Section : W10x54

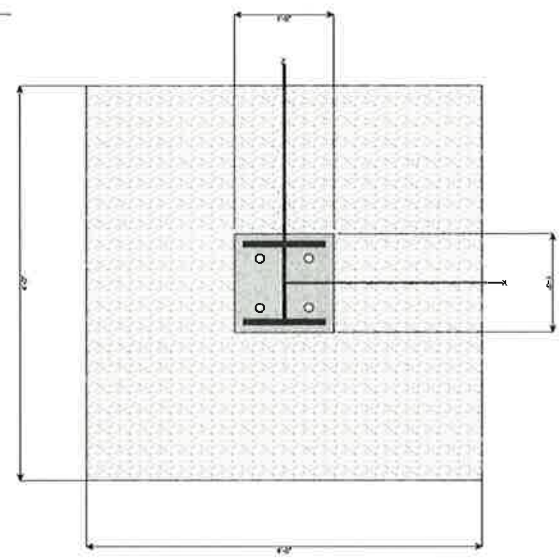
Depth	10.1 in	Area	15.8 in^2
Width	10 in	Ixx	303 in^4
Flange Thickness	0.615 in	Iyy	103 in^4
Web Thickness	0.37 in		

Plate Dimensions

N : Length 12.0 in
B : Width 12.0 in
Thickness 0.6250 in
Column assumed welded to base plate.

Support Dimensions

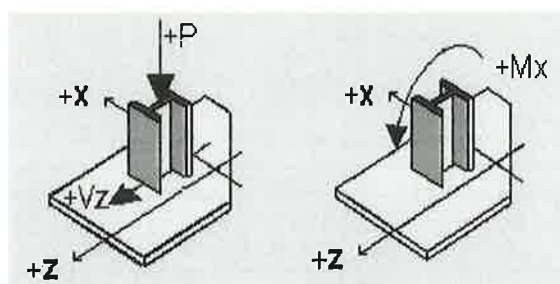
Width along "X" 48.0 in
Length along "Z" 48.0 in



Applied Loads

	P-Y	V-Z	M-X
D : Dead Load	2.40 k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	k	k	k-ft
S : Snow	k	k	k-ft
W : Wind	10.50 k	3.480 k	k-ft
E : Earthquake	8.810 k	1.920 k	k-ft
H : Lateral Earth	k	k	k-ft

" P " = Gravity load, "+" sign is downward.
"+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.



GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method Load Resistance Factor Design
Governing Load Combination +1.20D+0.50Lr+0.50L+W+1.60H
Governing Load Case Type Axial Load Only
Design Plate Size **1'-0" x 1'-0" x 0 -5/8**
Pu : Axial .... 13.380 k
Mu : Moment .... 0.000 k-ft

Mu : Max. Moment .... 0.186 k-in
fb : Max. Bending Stress .... 1.903 ksi
Fb : Allowable : 32.400 ksi

Bending Stress Ratio 0.059
Bending Stress OK

fu : Max. Plate Bearing Stress .... 0.093 ksi
Fp : Allowable : 1.500 ksi

Bearing Stress Ratio 0.062
Bearing Stress OK

SIMPSON Strong-Tie® **Anchor Designer™ Software** Version 2.3.5555.2

Company:		Date:	5/15/2017
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Moment Frame Outside Columns
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 10.13
C_{min} (inch): 1.50
S_{min} (inch): 3.00

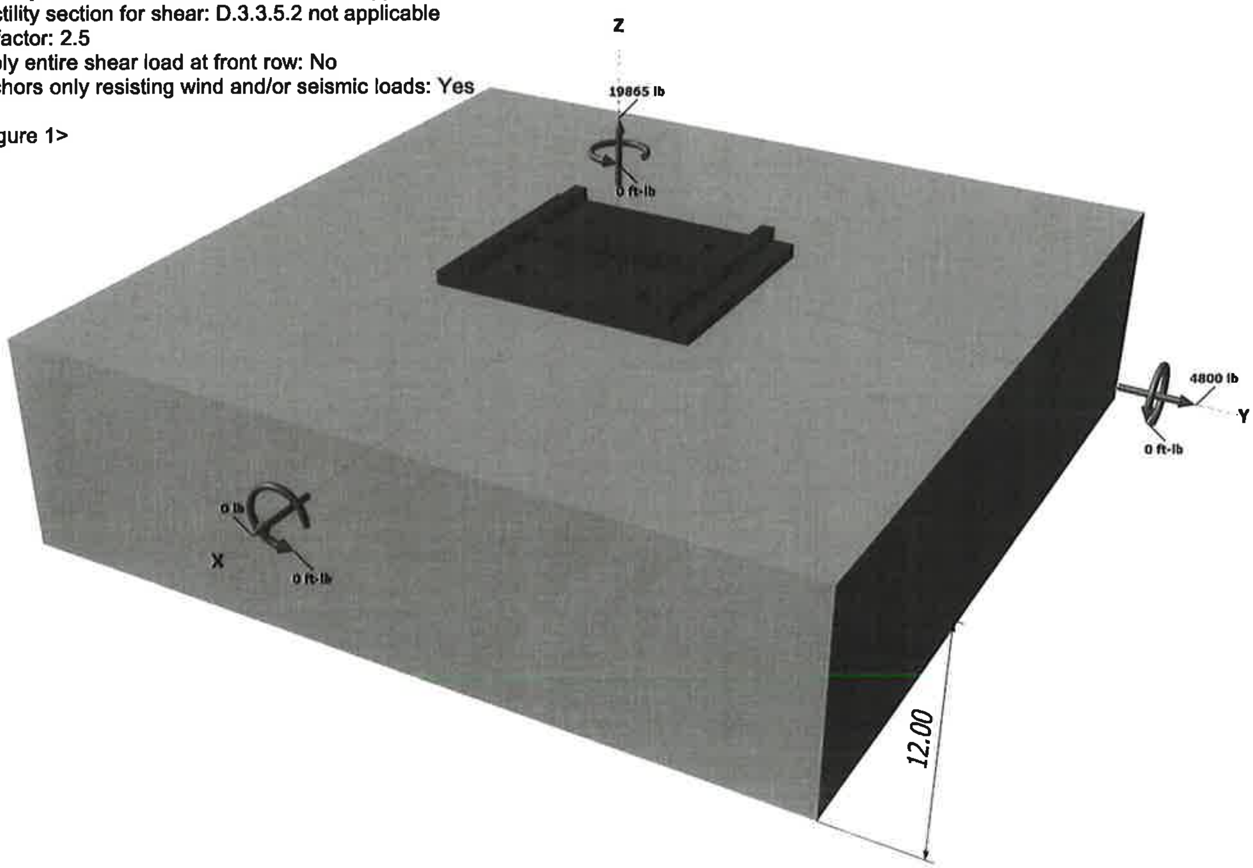
Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: 2.5
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Do not evaluate concrete breakout in tension: No
Do not evaluate concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: Yes

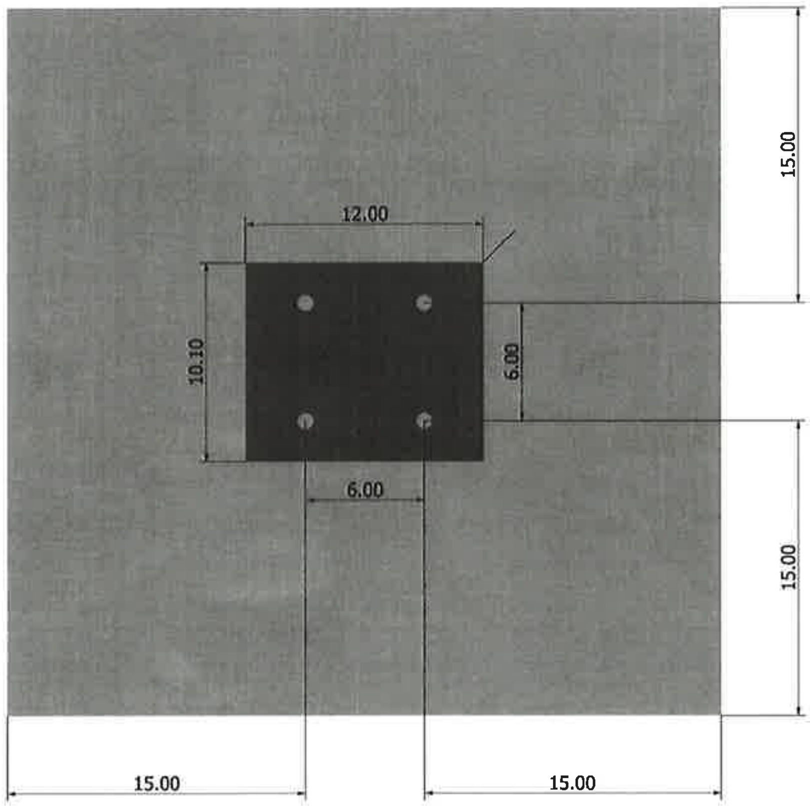
Base Plate
Length x Width x Thickness (inch): 10.10 x 12.00 x 0.63
Yield stress: 36000 psi

Profile type/size: W10X54

<Figure 1>



<Figure 2>



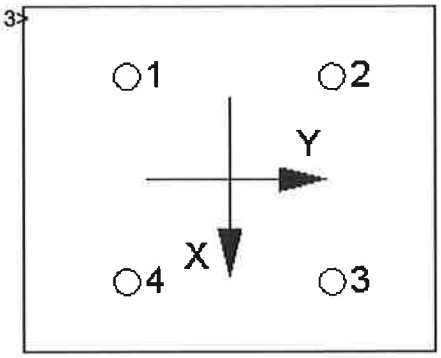
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB6 (3/4"Ø)

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4966.3	0.0	1200.0	1200.0
2	4966.3	0.0	1200.0	1200.0
3	4966.3	0.0	1200.0	1200.0
4	4966.3	0.0	1200.0	1200.0
Sum	19865.0	0.0	4800.0	4800.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 19865
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension(Sec. D.5.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

5. Concrete Breakout Strength of Anchor in Tension (Sec. D.5.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. D-6)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	2500	8.000	27153

$0.75\phi N_{cbg} = 0.75\phi\ (A_{Nc} / A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. D.4.1 & Eq. D-4)

A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cbg}$ (lb)
1024.00	576.00	1.000	1.000	1.00	1.000	27153	0.75	27153

6. Pullout Strength of Anchor in Tension (Sec. D.5.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. D.4.1, Eq. D-13 & D-14)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	3.56	2500	0.70	37361

8. Steel Strength of Anchor in Shear (Sec. D.6.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
11625	0.8	0.65	6045

9. Concrete Breakout Strength of Anchor in Shear (Sec. D.6.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.75	1.00	2500	10.00	14230

$\phi V_{cbgy} = \phi\ (A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. D.4.1 & Eq. D-31)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
432.00	450.00	1.000	1.000	1.000	1.118	14230	0.75	11455

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.75	1.00	2500	10.00	14230

$\phi V_{cbgy} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. D.4.1 & Eq. D-31)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
432.00	450.00	1.000	1.000	1.000	1.118	14230	0.75	22910

10. Concrete Pryout Strength of Anchor in Shear (Sec. D.6.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Eq. D-41)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	1024.00	576.00	1.000	1.000	1.000	1.000	27153	0.70	67581

11. Results

Interaction of Tensile and Shear Forces (Sec. D.7)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	4966	14528	0.34	Pass
Concrete breakout	**19865**	**27153**	**0.73**	**Pass (Governs)**
Pullout	4966	37361	0.13	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	1200	6045	0.20	Pass
T Concrete breakout y+	**4800**	**11455**	**0.42**	**Pass (Governs)**
|| Concrete breakout x-	**2400**	**22910**	**0.10**	**Pass (Governs)**
Pryout	4800	67581	0.07	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. D.7.3	0.73	0.42	115.1 %	1.2	Pass

PAB6 (3/4"Ø) with hef = 8.000 inch meets the selected design criteria.

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections D.8.1 and D.8.2 for torqued cast-in-place anchor is waived per designer option.

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of D.3.3.4.3 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of D.3.3.5.3 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

SIMPSON Strong-Tie®	Anchor Designer™ Software Version 2.3.5555.2

Company:		Date:	5/15/2017
Engineer:		Page:	1/4
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Moment Frame Center Column
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-11
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 10.13
C_{min} (inch): 1.50
S_{min} (inch): 3.00

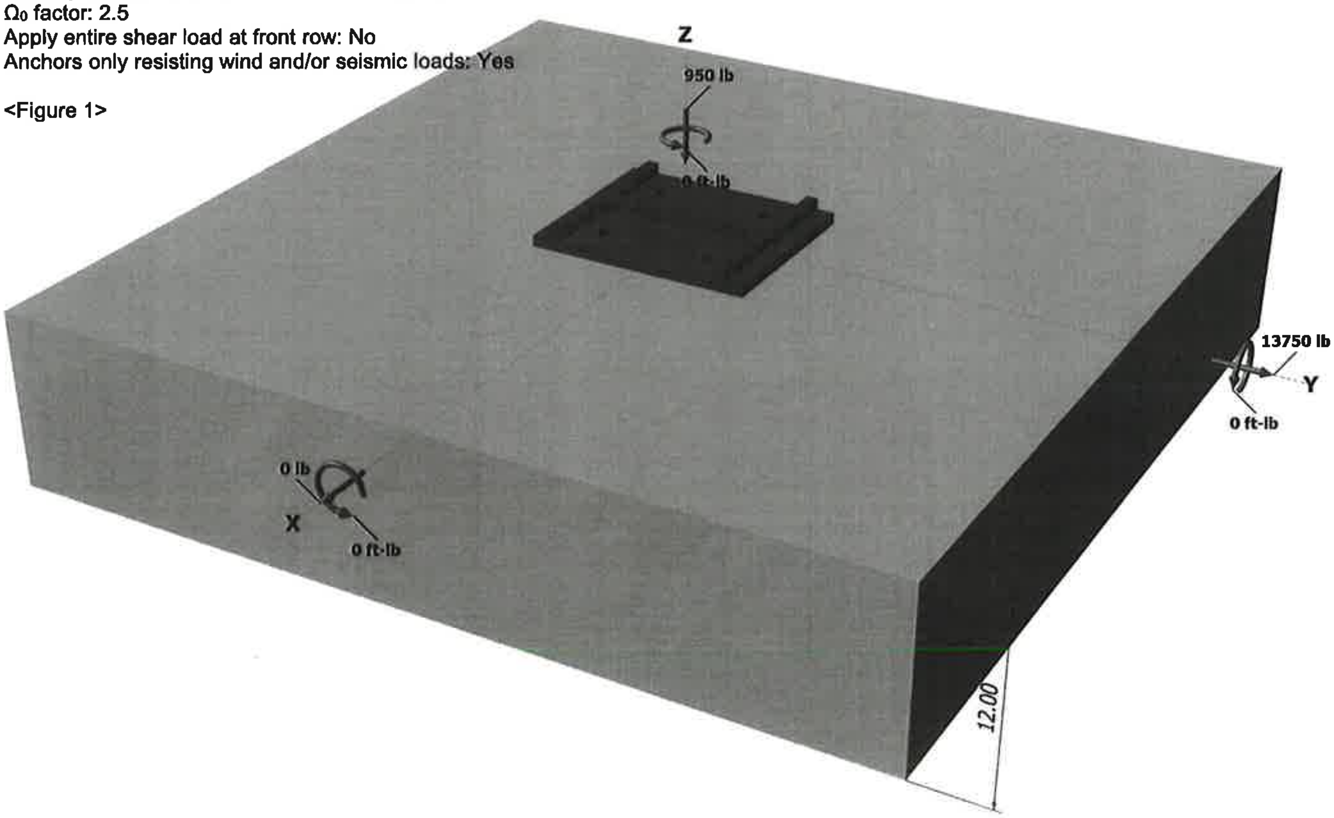
Load and Geometry
Load factor source: ACI 318 Section 9.2
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: D.3.3.4.2 not applicable
Ductility section for shear: D.3.3.5.2 not applicable
Ω_0 factor: 2.5
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Do not evaluate concrete breakout in tension: No
Do not evaluate concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: Yes

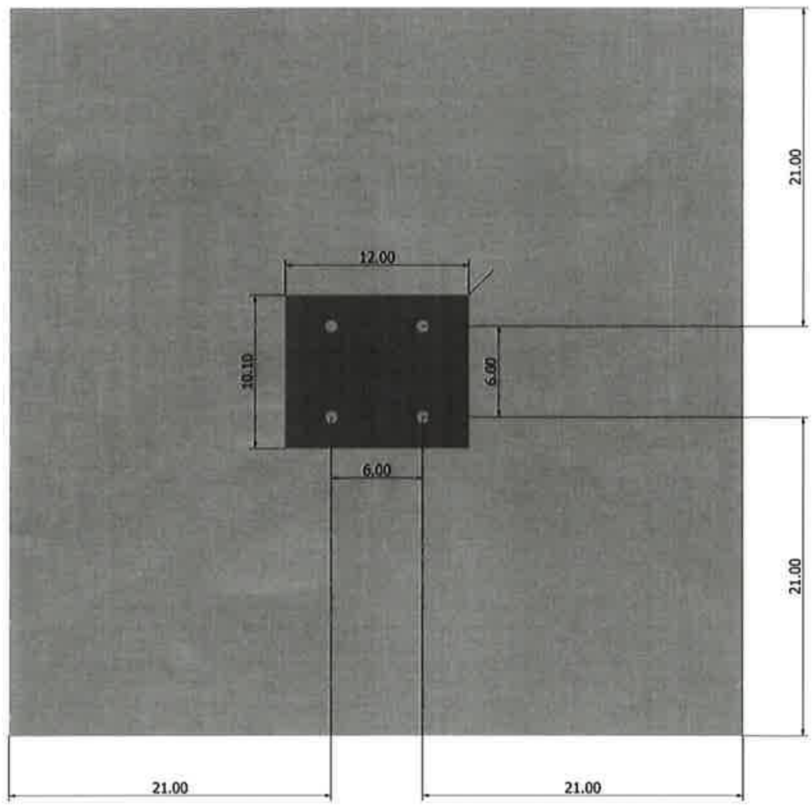
Base Plate
Length x Width x Thickness (inch): 10.10 x 12.00 x 0.63
Yield stress: 36000 psi

Profile type/size: W10X54

<Figure 1>



<Figure 2>



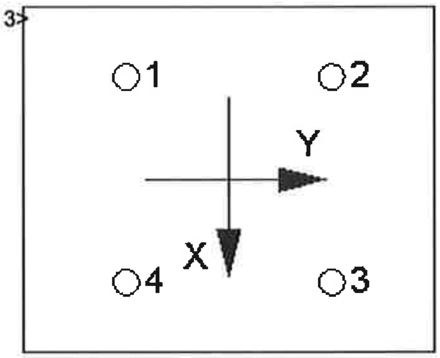
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB6 (3/4"Ø)

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	0.0	3437.5	3437.5
2	0.0	0.0	3437.5	3437.5
3	0.0	0.0	3437.5	3437.5
4	0.0	0.0	3437.5	3437.5
Sum	0.0	0.0	13750.0	13750.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



8. Steel Strength of Anchor in Shear (Sec. D.6.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
11625	0.8	0.65	6045

9. Concrete Breakout Strength of Anchor in Shear (Sec. D.6.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.75	1.00	2500	14.00	23572

$\phi V_{cbgy} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. D.4.1 & Eq. D-31)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
576.00	882.00	1.000	1.000	1.000	1.323	23572	0.75	15274

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. D-33 & Eq. D-34)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
6.00	0.75	1.00	2500	14.00	23572

$\phi V_{cbgy} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. D.4.1 & Eq. D-31)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
576.00	882.00	1.000	1.000	1.000	1.323	23572	0.75	30547

10. Concrete Pryout Strength of Anchor in Shear (Sec. D.6.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Eq. D-41)

k_{cp}	A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	1024.00	576.00	1.000	1.000	1.000	1.000	27153	0.70	67581

11. Results

Company:		Date:	5/15/2017
Engineer:		Page:	4/4
Project:			
Address:			
Phone:			
E-mail:			

Interaction of Tensile and Shear Forces (Sec. D.7)

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	3438	6045	0.57	Pass
T Concrete breakout y+	**13750**	**15274**	**0.90**	**Pass (Governs)**
|| Concrete breakout x-	**6875**	**30547**	**0.23**	**Pass (Governs)**
Pryout	13750	67581	0.20	Pass

PAB6 (3/4"Ø) with hef = 8.000 inch meets the selected design criteria.

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections D.8.1 and D.8.2 for torqued cast-in-place anchor is waived per designer option.

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of D.3.3.4.3 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of D.3.3.5.3 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

Horiz. Diaphragm

Gridline A — Front Upper — Special Moment Frame — Heavy Roof (Unblocked)

	Length	Wind: $TA_{Roof\text{-}end}$	Wind: $TA_{Roof\text{-}int}$	Wind: V_s	Seismic: A_i	w_i	F	ρ	R	F_x	V_{final}	Diaphragm: V	$v_{diaphragm}$	v_{allow}	Check	
Wall L =	46.25	0	0	2121	1430	48529.9	1.100	1.300	8	4249	**3866**	2121	**85**	399	**OK**	**Wind**
Wall b =	25	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$					$V_{additional}$ =			3866	**155**	285	**OK**	**Siesmic**
Opening	0	36	121	**2121**												

$V_{additional}$ =

Deflection: OK

Diaphragm Chord Force		
w =	167	plf
M =	44706	ft-lb
T =	1788	lb

Diaphragm Collector Force		
Opening b_{total} =	0	ft
T =	0	lb

Z =	141	lb
Z' =	248	lb

Top Plate Splice		
N =	7.2	nails
Use	8	16d nails between splice points

Use A35 clips or direct nailing at full height truss blocking as specified on plans
Use 3/4" APA rated OSB sheathing w/ 10d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Gridline E — Rear Upper — Structural Sheathing — Heavy Roof (Unblocked)

	Length	Wind: $TA_{Roof\text{-}end}$	Wind: $TA_{Roof\text{-}int}$	Wind: V_s	Seismic: A_i	w_i	F	ρ	R	F_x	V_{final}	Diaphragm: V	$v_{diaphragm}$	v_{allow}	Check	
Wall L =	46.25	0	0	2121	1430	48530	1.100	1.000	6.5	5229	**3661**	2121	**85**	399	**OK**	**Wind**
Wall b =	25	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$					$V_{additional}$ =			3661	**146**	285	**OK**	**Siesmic**
Opening	3.5	36	121	**2121**												

$V_{additional}$ =

Deflection: OK

Diaphragm Chord Force		
w =	158	plf
M =	42326	ft-lb
T =	1693	lb

Diaphragm Collector Force		
Opening b_{total} =	3.5	ft
T =	512	lb

Z =	141	lb
Z' =	248	lb

Top Plate Splice		
N =	6.8	nails
Use	8	16d nails between splice points

Use (4) 16d common toenails at full height truss blocking
Use 3/4" APA rated OSB sheathing w/ 10d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Gridline 2 — Left Upper — Structural Sheathing — Heavy Roof (Unblocked)

	Length	Wind: $TA_{Roof\text{-}end}$	Wind: $TA_{Roof\text{-}int}$	Wind: V_s	Seismic: A_i	w_i	F	ρ	R	F_x	V_{final}	Diaphragm: V	$v_{diaphragm}$	v_{allow}	Check	
Wall L =	27	0	0	1111	1430	48530	1.100	1.000	6.5	5229	**3661**	1111	**24**	399	**OK**	**Wind**
Wall b =	46.25	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$					$V_{additional}$ =			3661	**79**	285	**OK**	**Siesmic**
Opening	3	37	36	**1111**												

$V_{additional}$ =

Deflection: OK

Diaphragm Chord Force		
w =	271	plf
M =	24709	ft-lb
T =	534	lb

Diaphragm Collector Force		
Opening b_{total} =	3	ft
T =	237	lb

Z =	141	lb
Z' =	248	lb

Top Plate Splice		
N =	2.2	nails
Use	8	16d nails between splice points

Use (4) 16d common toenails at full height truss blocking
Use 3/4" APA rated OSB sheathing w/ 10d 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 5	Wall L =	27	0	0	1190	1430	48530	1.100	1.000	6.5	5229	**3661**	1190	**26**	399	**OK**	**Wind**
Right Upper	Wall b =	46.25	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		3661	**79**	285	**OK**	**Siesmic**
	Opening	1.5	40	38	**1190**										**Deflection**		
Structural Sheathing				$V_{additional}$ =											**OK**		

Heavy Roof
(Unblocked)

Diaphragm Chord Force		
w =	271	plf
M =	24709	ft-lb
T =	534	lb

Diaphragm Collector Force		
Opening b_{total} =	1.5	ft
T =	119	lb

Z =	141	lb
Z' =	248	lb

Top Plate Splice		
N =	2.2	nails
Use	8	16d nails between splice points

Use (4) 16d common toenails at full height truss blocking
Use 3/4" APA rated OSB sheathing w/ 10d 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 A	Wall L =	46.25	0	0	3387	1222	16100	1.100	1.300	8	1410	**1283**	3387	**135**	399	**OK**	**Wind**
Front Main	Wall b =	25	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		1283	**51**	285	**OK**	**Siesmic**
	Opening	0	57	194	**3387**										**Deflection**		
Special Moment Frame				$V_{additional}$ =											**OK**		

Floor
(Unblocked)

Diaphragm Chord Force		
w =	146	plf
M =	39167	ft-lb
T =	1567	lb

Diaphragm Collector Force		
Opening b_{total} =	0	ft
T =	0	lb

Z =	141	lb
Z' =	248	lb

Top Plate Splice		
N =	6.3	nails
Use	8	16d nails between splice points

Use (4) 16d common toenails at full height truss blocking
Use 3/4" APA rated OSB sheathing w/ 10d 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 E	Wall L =	46.25	0	0	3387	1222	16100	1.100	1.000	6.5	1735	**1214**	3387	**135**	399	**OK**	**Wind**
Rear Main	Wall b =	25	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		1214	**49**	285	**OK**	**Siesmic**
	Opening	15	57	194	**3387**										**Deflection**		
Structural Sheathing				$V_{additional}$ =											**OK**		

Floor
(Unblocked)

Diaphragm Chord Force		
w =	146	plf
M =	39167	ft-lb
T =	1567	lb

Diaphragm Collector Force		
Opening b_{total} =	15	ft
T =	2032	lb

Z =	141	lb
Z' =	248	lb

Top Plate Splice		
N =	8.2	nails
Use	9	16d nails between splice points

Use (4) 16d common toenails at full height truss blocking
Use 3/4" APA rated OSB sheathing w/ 10d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Horiz. Diaphragm

Gridline 2
Left Main

Structural Sheathing

Floor
(Unblocked)

	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	
		Wind			Seismic							Diaphragm				
Wall L =	27	0	0	2010	1222	16100	1.100	1.000	6.5	1735	**1214**	2010	**43**	399	**OK**	**Wind**
Wall b =	46.25	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$					$V_{additional}$ =			1214	**26**	285	**OK**	**Siesmic**
Opening	3	67	65	**2010**								**Deflection**				

$V_{additional}$ =

Z =	141 lb
Z' =	248 lb

Deflection: **OK**

Diaphragm Chord Force	
w =	149 plf
M =	13568 ft-lb
T =	293 lb

Diaphragm Collector Force	
Opening b_{total} =	3 ft
T =	130 lb

Top Plate Splice		
N =	1.2	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 3/4" APA rated OSB sheathing w/ 10d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Gridline 5
Right Main

Structural Sheathing

Floor
(Unblocked)

	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	
		Wind			Seismic							Diaphragm				
Wall L =	27	0	0	2010	1222	16100	1.100	1.000	6.5	1735	**1214**	2010	**43**	399	**OK**	**Wind**
Wall b =	46.25	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$					$V_{additional}$ =			1214	**26**	285	**OK**	**Siesmic**
Opening	1.5	67	65	**2010**								**Deflection**				

$V_{additional}$ =

Z =	141 lb
Z' =	248 lb

Deflection: **OK**

Diaphragm Chord Force	
w =	149 plf
M =	13568 ft-lb
T =	293 lb

Diaphragm Collector Force	
Opening b_{total} =	1.5 ft
T =	65 lb

Top Plate Splice		
N =	1.2	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 3/4" APA rated OSB sheathing w/ 10d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Gridline A
Front Lower

Special Moment Frame

Floor
(Unblocked)

	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	
		Wind			Seismic							Diaphragm				
Wall L =	46.25	0	0	3412	1300	16970	1.100	1.300	8	1486	**1352**	3412	**136**	399	**OK**	**Wind**
Wall b =	25	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$					$V_{additional}$ =			1352	**54**	285	**OK**	**Siesmic**
Opening	0	61	190	**3412**								**Deflection**				

$V_{additional}$ =

Z =	141 lb
Z' =	248 lb

Deflection: **OK**

Diaphragm Chord Force	
w =	148 plf
M =	39453 ft-lb
T =	1578 lb

Diaphragm Collector Force	
Opening b_{total} =	0 ft
T =	0 lb

Top Plate Splice		
N =	6.4	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 3/4" APA rated OSB sheathing w/ 10d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Horiz. Diaphragm

Gridline 5
Left Lower

		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	
Wall L =	27	0	0	2010	1222	45689	1.100	1.000	6.5	4923	**3446**	2010	**43**	399	**OK**	**Wind**
Wall b =	46.25	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		3446	**75**	285	**OK**	**Siesmic**
Opening	0	67	65	**2010**												

Deflection	
OK	

Structural Sheathing

$V_{additional}$ =

Z =	141 lb
Z' =	248 lb

Floor
(Unblocked)

Diaphragm Chord Force	
w =	255 plf
M =	23262 ft-lb
T =	503 lb

Diaphragm Collector Force	
Opening b_{total} =	0 ft
T =	0 lb

Top Plate Splice		
N =	2.0	nails
Use	8	16d nails between splice points

Use (4) 16d common toenails at full height truss blocking
Use 3/4" APA rated OSB sheathing w/ 10d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

STUD WALL CALCULATION **Upper**

Wall Location =	Exterior	
Species =	DF-L Stud	
Stud Width =	1.5	in
Stud Depth (d_x) =	5.5	in
L =	8	ft
stud spacing =	1.33	ft
F_b =	700	psi
F_c =	850	psi
$F_{c\perp}$ =	625	psi
E =	1400000	psi
E_{min} =	510000	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
Dead Loads:		
Roof DL =	195	plf
Floor DL =	0	plf
w_{DL} =	275	plf
Live Loads:		
Roof LL =	2347.9	plf
Floor LL =	0	plf
W_{LL}=	2347.85	
Load Case 1: Gravity Loads Only		
Load Combinations:		
D =	366	lbs
D+L =	366	lbs
D+S =	3488	lbs
D+0.75(L)+0.75(S) =	2708	lbs
C_D (D) =	0.9	
C_D (D+L) =	1	
C_D (D+S) =	1	
C_D (D+0.75(L)+0.75(S)) =	1	
$f_c = f_{c\perp}$ =	422.8	psi
$(l_e/d)_x$ =	17.5	in
E'_{min} =	510000	psi
c =	0.8	
F_{cE} =	1376.0	
F^*_c =	850	psi
F_{cE}/F^*_c =	1.619	psi
$(1+F_{cE}/F^*_c)/2c$ =	1.637	
C_p =	0.827	
F'_c =	703.1	
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 =	0.5	
Angle =	2.4	
C_S =	1.000	
Increase for Drift=	1.000	
Effective snow load =	181	psf
Roof dead load =	15	psf
Floor live load =	40	psf
Floor dead load =	15	psf
Trib. Area $_{roof}$=	13	ft
Trib. Area $_{floor}$=	0	ft
Add. Uniform Load =	80	plf
Lateral Load =	21.79	psf

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

Load Case 2: Gravity Loads + Lateral Loads

C_D =	1.6	
C_r =	1.35	
w =	29.0	plf
M =	2782.0	in.lb
f_b =	367.9	psi
F'_b =	1512.00	psi
Check =	**OK**	
Axial:		
(l_e/d_x) =	17.5	in
E'_{min} =	510000	psi
c =	0.8	
F_{cE} =	1376.0	psi
F^*_c =	1360	psi
F_{cE}/F^*_c =	1.012	
$(1+F_{cE}/F^*_c)/2c$ =	1.257	
C_p =	0.695	
F'_c =	945.2	psi

	D+0.75(W)+0.75(L)+0.75(S)	**D+W**	
f_c =	328.2	44.3	psi
Check =	**OK**	**OK**	
Combined Stress:			
F_{cEx} =	1376.0	1376.0	psi
Interaction Formula =	0.36	0.25	
Check =	**OK**	**OK**	

STUD WALL CALCULATION **Main**

Wall Location =	Exterior	
Species =	DF-L Stud	
Stud Width =	1.5	in
Stud Depth (d_x) =	5.5	in
L =	9	ft
stud spacing =	1.33	ft
F_b =	700	psi
F_c =	850	psi
$F_{c\perp}$ =	625	psi
E =	1400000	psi
E_{min} =	510000	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

Dead Loads:

Roof DL =	195	plf
Floor DL =	165	plf
w_{DL} =	440	plf

Live Loads:

Roof LL =	2347.9	plf
Floor LL =	440	plf
W_{LL}=	2787.85	

Load Case 1: Gravity Loads Only

Load Combinations:

D =	585	lbs
D+L =	1170	lbs
D+S =	3708	lbs
D+0.75(L)+0.75(S) =	3366	lbs
C_D (D) =	0.9	
C_D (D+L) =	1	
C_D (D+S) =	1	
C_D (D+0.75(L)+0.75(S)) =	1	
$f_c = f_{c\perp}$ =	449.4	psi
$(l_e/d)_x$ =	19.6	in
E'_{min} =	510000	psi
c =	0.8	
F_{cE} =	1087.2	
F^*_c =	850	psi
F_{cE}/F^*_c =	1.279	psi
$(1+F_{cE}/F^*_c)/2c$ =	1.424	
C_p =	0.769	
F'_c =	653.3	
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 =	0.5	
Angle =	2.4	
C_S =	1.000	
Increase for Drift=	1.000	
Effective snow load =	181	psf
Roof dead load =	15	psf
Floor live load =	40	psf
Floor dead load =	15	psf
Trib. Area $_{roof}$=	13	ft
Trib. Area $_{floor}$=	11	ft
Add. Uniform Load =	80	plf
Lateral Load =	21.79	psf

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

Load Case 2: Gravity Loads + Lateral Loads

C_D =	1.6	
C_r =	1.35	
w =	29.0	plf
M =	3521.0	in.lb
f_b =	465.6	psi
F'_b =	1512.00	psi
Check =	**OK**	

Axial:

(l_e/d_x) =	19.6	in
E'_{min} =	510000	psi
c =	0.8	
F_{cE} =	1087.2	psi
F^*_c =	1360	psi
F_{cE}/F^*_c =	0.799	
$(1+F_{cE}/F^*_c)/2c$ =	1.125	
C_p =	0.609	
F'_c =	828.7	psi

	D+0.75(W)+0.75(L)+0.75(S)	D+W	
f_c =	408.0	70.9	psi
Check =	**OK**	**OK**	
Combined Stress:			
F_{cEx} =	1087.2	1087.2	psi
Interaction Formula =	0.61	0.34	
Check =	**OK**	**OK**	

STUD WALL CALCULATION Basement

Wall Location =	Exterior	
Species =	DF-L Stud	
Stud Width =	1.5	in
Stud Depth (d_x) =	5.5	in
L =	8	ft
stud spacing =	1.33	ft
F_b =	700	psi
F_c =	850	psi
$F_{c\perp}$ =	625	psi
E =	1400000	psi
E_{min} =	510000	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
Dead Loads:		
Roof DL =	195	plf
Floor DL =	277.5	plf
w_{DL} =	552.5	plf
Live Loads:		
Roof LL =	2347.9	plf
Floor LL =	740	plf
W_{LL}=	3087.85	

Load Case 1: Gravity Loads Only

Load Combinations:		
D =	735	lbs
D+L =	1719	lbs
D+S =	3857	lbs
D+0.75(L)+0.75(S) =	3815	lbs
C_D (D) =	0.9	
C_D (D+L) =	1	
C_D (D+S) =	1	
C_D (D+0.75(L)+0.75(S)) =	1	
$f_c = f_{c\perp}$ =	467.6	psi
$(l_e/d)_x$ =	17.5	in
E'_{min} =	510000	psi
c =	0.8	
F_{cE} =	1376.0	
F^*_c =	850	psi
F_{cE}/F^*_c =	1.619	psi
$(1+F_{cE}/F^*_c)/2c$ =	1.637	
C_p =	0.827	
F'_c =	703.1	
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 =	0.5	
Angle =	2.4	
C_S =	1.000	
Increase for Drift=	1.000	
Effective snow load =	181	psf
Roof dead load =	15	psf
Floor live load =	40	psf
Floor dead load =	15	psf
Trib. Area $_{roof}$=	13	ft
Trib. Area $_{floor}$=	18.5	ft
Add. Uniform Load =	80	plf
Lateral Load =	21.79	psf

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

Load Case 2: Gravity Loads + Lateral Loads

C_D =	1.6	
C_r =	1.35	
w =	29.0	plf
M =	2782.0	in.lb
f_b =	367.9	psi
F'_b =	1512.00	psi
Check =	**OK**	
Axial:		
(l_e/d_x) =	17.5	in
E'_{min} =	510000	psi
c =	0.8	
F_{cE} =	1376.0	psi
F^*_c =	1360	psi
F_{cE}/F^*_c =	1.012	
$(1+F_{cE}/F^*_c)/2c$ =	1.257	
C_p =	0.695	
F'_c =	945.2	psi

	D+0.75(W)+0.75(L)+0.75(S)	D+W	
f_c =	462.4	89.1	psi
Check =	**OK**	**OK**	
Combined Stress:			
F_{cEx} =	1376.0	1376.0	psi
Interaction Formula =	0.51	0.27	
Check =	**OK**	**OK**	

KING STUD CALCULATION **Rear Main**

Species =	DF-L Stud	
Stud Width =	6	in
Stud Depth (d_x) =	5.5	in
L =	9	ft
opening width =	15	ft
stud spacing =	8.17	ft
F_b =	700	psi
F_c =	850	psi
$F_{c\perp}$ =	625	psi
E =	1400000	psi
E_{min} =	510000	psi
C_F =	1.00	for bending
C_F =	1.00	for comp. ll to grain
A =	33	in^2
S =	30.25	in^3

Dead Loads:

Roof DL =	75	plf
Floor DL =	15	plf
w_{DL} =	170	plf

Live Loads:

Roof LL =	903.0	plf
Floor LL =	40	plf
W_{LL}=	943.02	

Load Case 1: Gravity Loads Only

Load Combinations:

D =	1389	lbs
D+L =	1716	lbs
D+S =	8767	lbs
D+0.75(L)+0.75(S) =	7167	lbs
C_D (D) =	0.9	
C_D (D+L) =	1	
C_D (D+S) =	1	
C_D (D+0.75(L)+0.75(S)) =	1	
$f_c = f_{c\perp}$ =	265.7	psi
$(l_e/d)_x$ =	19.6	in
E'_{min} =	510000	psi
c =	0.8	
F_{cE} =	1087.2	
F^*_c =	850	psi
F_{cE}/F^*_c =	1.279	psi
$(1+F_{cE}/F^*_c)/2c$ =	1.424	
C_p =	0.769	
F'_c =	653.3	
Check =	**OK**	psi

Bearing of stud on wall plates:

C_b =	1.06	
$F'_{c\perp}$ =	664	
Check =	**OK**	psi

Loadings

Roofing Material =	Shingle/Tile	
Roof Pitch =	0.5	
Angle =	2.4	
C_S =	1.000	
Increase for Drift=	1.000	
Effective snow load =	181	psf
Roof dead load =	15	psf
Floor live load =	40	psf
Floor dead load =	15	psf
Trib. Area $_{roof}$=	5	ft
Trib. Area $_{floor}$=	1	ft
Add. Uniform Load =	80	plf
Lateral Load =	18.87	psf

Use: (2) 2x6 Full Height King Studs

Load Case 2: Gravity Loads + Lateral Loads

C_D =	1.6	
C_r =	1.15	
w =	154.2	plf
M =	18734.4	in.lb
f_b =	619.3	psi
F'_b =	1288.00	psi
Check =	**OK**	

Axial:

(l_e/d_x) =	19.6	in
E'_{min} =	510000	psi
c =	0.8	
F_{cE} =	1087.2	psi
F^*_c =	1360	psi
F_{cE}/F^*_c =	0.799	
$(1+F_{cE}/F^*_c)/2c$ =	1.125	
C_p =	0.609	
F'_c =	828.7	psi

	D+0.75(W)+0.75(L)+0.75(S)	D+W	
f_c =	217.2	42.1	psi
Check =	**OK**	**OK**	
Combined Stress:			
F_{cEx} =	1087.2	1087.2	psi
Interaction Formula =	0.52	0.50	
Check =	**OK**	**OK**	

Footing(s)

	FT1A	FT1B	FT1C	FT2
Width of footing (in)=	20	32	36	20
Depth of footing (in)=	10	10	10	10
Height of wall (in)=	132	60	132	0
Width of wall (in)=	8	8	8	8
Roofing Material =	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile
Roof Pitch=	0.5	0.5	0.5	0.5
Angle=	2.4	2.4	2.4	2.4
C_S=	1.000	1.000	1.000	1.000
Increase for Drift/Valley=	1.000	1.000	1.000	1.000
Effective snow load (psf)=	181	181	181	181
Roof dead load (psf)=	15	15	15	15
Floor live load (psf)=	40	40	40	40
Floor dead load (psf)=	15	15	15	15
Trib. $Area_{ROOF}$=	5	13	12.5	0
Trib. $Area_{FLOOR2}$=	2	11	7	0
Trib. $Area_{FLOOR1}$=	2	7.5	5.5	12.5
w_S (plf)=	885	2348	2258	0
w_L (plf)=	160	740	500	500
w_D (plf)=	133.5	472.5	375	187.5
$w_{CONC.}$ (plf)=	1308	833	1475	208
$w_{ADDITIONAL}$ (plf)=	170	260	200	300
w_{TOTAL} (plf)=	2497	3914	4308	1196
Req.Soil Bearing (psf)=	**1498**	**1468**	**1436**	**718**
Footing Reinforcement:	**(2) #4 bars cont.**	**(3) #4 bars cont.**	**(4) #4 bars cont.**	**(2) #4 bars cont.**
Crosswise Reinforcement:	**None**	**#4 bars @ 12" o.c.**	**#5 bars @ 12" o.c.**	**None**

Project: 2017-2259

Location: FT8
Footing
[2015 International Building Code(2015 NDS)]
Footing Size: 5.0 FT x 5.0 FT x 12.00 IN
Reinforcement: #4 Bars @ 8.00 IN. O.C. E/W / (7) min.
Section Footing Design Adequate

Kelly Christensen
LEI Consulting Engineers and Surveyers Inc.
3302 N. Main St.
Spanish Fork, UT 84660

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FOOTING PROPERTIES

Allowable Soil Bearing Pressure:	Qs =	1500	psf
Concrete Compressive Strength:	F'c =	2500	psi
Reinforcing Steel Yield Strength:	Fy =	60000	psi
Concrete Reinforcement Cover:	c =	3	in

FOOTING SIZE

Width:	W =	5	ft
Length:	L =	5	ft
Depth:	Depth =	12	in
Effective Depth to Top Layer of Steel:	d =	8.25	in

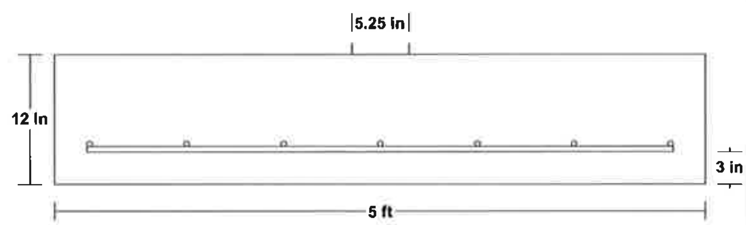
COLUMN AND BASEPLATE SIZE

Column Type:	Wood		
Column Width:	m =	5.25	in
Column Depth:	n =	5.25	in

FOOTING CALCULATIONS

Bearing Calculations:			
Ultimate Bearing Pressure:	Qu =	1179	psf
Effective Allowable Soil Bearing Pressure:	Qe =	1350	psf
Required Footing Area:	Areq =	21.83	sf
Area Provided:	A =	25.00	sf
Baseplate Bearing:			
Bearing Required:	Bear =	46193	lb
Allowable Bearing:	Bear-A =	76141	lb
Beam Shear Calculations (One Way Shear):			
Beam Shear:	Vu1 =	16745	lb
Allowable Beam Shear:	Vc1 =	37125	lb
Punching Shear Calculations (Two Way Shear):			
Critical Perimeter:	Bo =	54	in
Punching Shear:	Vu2 =	43854	lb
Allowable Punching Shear (ACI 11-35):	vc2-a =	100238	lb
Allowable Punching Shear (ACI 11-36):	vc2-b =	135506	lb
Allowable Punching Shear (ACI 11-37):	vc2-c =	66825	lb
Controlling Allowable Punching Shear:	vc2 =	66825	lb
Bending Calculations:			
Factored Moment:	Mu =	346446	in-lb
Nominal Moment Strength:	Mn =	588027	in-lb
Reinforcement Calculations:			
Concrete Compressive Block Depth:	a =	0.65	in
Steel Required Based on Moment:	As(1) =	0.80	in2
Min. Code Req'd Reinf. Shrink./Temp. (ACI-10.5.4):	As(2) =	1.30	in2
Controlling Reinforcing Steel:	As-reqd =	1.30	in2
Selected Reinforcement:	#4's @ 8.0 in. o.c. e/w (7) Min.		
Reinforcement Area Provided:	As =	1.37	in2
Development Length Calculations:			
Development Length Required:	Ld =	15	in
Development Length Supplied:	Ld-sup =	27	in

LOADING DIAGRAM



FOOTING LOADING

Live Load:	PL =	27051	lb
Dead Load:	PD =	2426	lb
Total Load:	PT =	29477	lb
Ultimate Factored Load:	Pu =	46193	lb
Weight to resist uplift w/ 1.5 F.S.:	U.R. =	2417	lb

NOTES

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

General Footing

Description : FT11 at Right Column

Code References

Calculations per ACI 318-11, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : IBC 2015

General Information

Material Properties

f'c : Concrete 28 day strength	=	2.50 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	Yes
Use Pedestal wt for stability, mom & shear	:	Yes

Soil Design Values

Allowable Soil Bearing	=	1.50 ksf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Footing base depth below soil surface	=	3.0 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

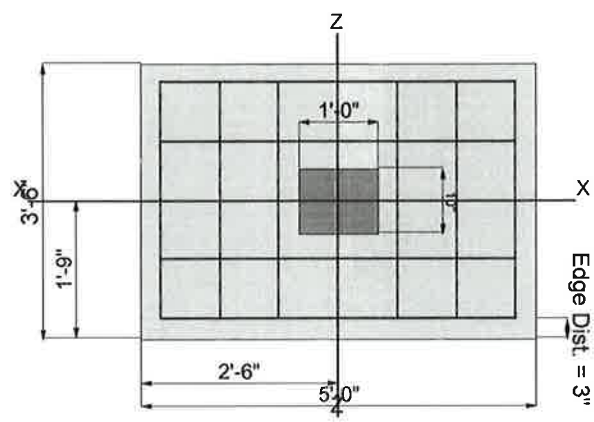
Width parallel to X-X Axis	=	5.0 ft
Length parallel to Z-Z Axis	=	3.50 ft
Footing Thickness	=	12.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	12.0 in
pz : parallel to Z-Z Axis	=	10.0 in
Height	=	36.0 in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in



Reinforcing

Bars parallel to X-X Axis

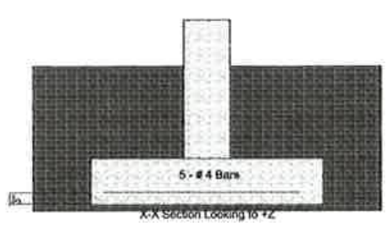
Number of Bars	=	5
Reinforcing Bar Size	=	# 4

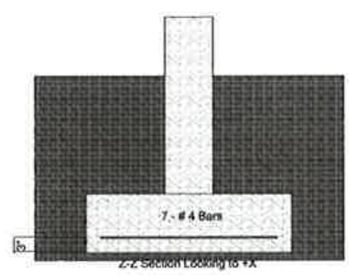
Bars parallel to Z-Z Axis

Number of Bars	=	7
Reinforcing Bar Size	=	# 4

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	ıg Z-Z Axis
# Bars required within zone	82.4 %
# Bars required on each side of zone	17.6 %





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	1.885		0.5610	12.169				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=					3.393	1.870		k
V-z	=								k

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

Project Title:
Engineer:
Project Descr:

Project ID:

Printed: 5 JUN 2017, 9:04AM

General Footing

File = T:\Structural\2017 Structural Jobs\2017-2259_BA 1606 Yehuda Res\2017-2259.ec6
ENERCALC, INC. 1983-2016, Build:6.16.5.11, Ver:6.16.5.11

Lic. # : KW-06004645

Licensee : LEI CONSULTING ENGINEERS

Description : FT11 at Right Column

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.960	Soil Bearing	1.440 ksf	1.50 ksf	+D+0.750L+0.750S+0.450W+H about Z-Z
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	1.557	Overturning - Z-Z	8.143 k-ft	12.678 k-ft	+0.60D+0.60W+0.60H
PASS	1.822	Sliding - X-X	2.036 k	3.709 k	+0.60D+0.60W+0.60H
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.2864	Z Flexure (+X)	3.191 k-ft	11.139 k-ft	+1.20D+1.60S+0.50W+1.60H
PASS	0.2281	Z Flexure (-X)	2.540 k-ft	11.139 k-ft	+1.20D+0.50L+1.60S+1.60H
PASS	0.1033	X Flexure (+Z)	1.129 k-ft	10.925 k-ft	+1.20D+0.50L+1.60S+1.60H
PASS	0.1033	X Flexure (-Z)	1.129 k-ft	10.925 k-ft	+1.20D+0.50L+1.60S+1.60H
PASS	0.2474	1-way Shear (+X)	18.555 psi	75.0 psi	+1.20D+1.60S+0.50W+1.60H
PASS	0.1960	1-way Shear (-X)	14.701 psi	75.0 psi	+1.20D+0.50L+1.60S+1.60H
PASS	0.09147	1-way Shear (+Z)	6.860 psi	75.0 psi	+1.20D+0.50L+1.60S+1.60H
PASS	0.09147	1-way Shear (-Z)	6.860 psi	75.0 psi	+1.20D+0.50L+1.60S+1.60H
PASS	0.1706	2-way Punching	25.590 psi	150.0 psi	+1.20D+0.50L+1.60S+1.60H

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, +D+H	1.50	n/a	0.0	0.4830	0.4830	n/a	n/a	0.322
X-X, +D+L+H	1.50	n/a	0.0	0.5150	0.5150	n/a	n/a	0.343
X-X, +D+Lr+H	1.50	n/a	0.0	0.4830	0.4830	n/a	n/a	0.322
X-X, +D+S+H	1.50	n/a	0.0	1.178	1.178	n/a	n/a	0.785
X-X, +D+0.750Lr+0.750L+H	1.50	n/a	0.0	0.5070	0.5070	n/a	n/a	0.338
X-X, +D+0.750L+0.750S+H	1.50	n/a	0.0	1.029	1.029	n/a	n/a	0.686
X-X, +D+0.60W+H	1.50	n/a	0.0	0.4830	0.4830	n/a	n/a	0.322
X-X, +D+0.70E+H	1.50	n/a	0.0	0.4830	0.4830	n/a	n/a	0.322
X-X, +D+0.750Lr+0.750L+0.450W+H	1.50	n/a	0.0	0.5070	0.5070	n/a	n/a	0.338
X-X, +D+0.750L+0.750S+0.450W+H	1.50	n/a	0.0	1.029	1.029	n/a	n/a	0.686
X-X, +D+0.750L+0.750S+0.5250E+H	1.50	n/a	0.0	1.029	1.029	n/a	n/a	0.686
X-X, +0.60D+0.60W+0.60H	1.50	n/a	0.0	0.2898	0.2898	n/a	n/a	0.193
X-X, +0.60D+0.70E+0.60H	1.50	n/a	0.0	0.2898	0.2898	n/a	n/a	0.193
Z-Z, +D+H	1.50	0.0	n/a	n/a	n/a	0.4830	0.4830	0.322
Z-Z, +D+L+H	1.50	0.0	n/a	n/a	n/a	0.5150	0.5150	0.343
Z-Z, +D+Lr+H	1.50	0.0	n/a	n/a	n/a	0.4830	0.4830	0.322
Z-Z, +D+S+H	1.50	0.0	n/a	n/a	n/a	1.178	1.178	0.785
Z-Z, +D+0.750Lr+0.750L+H	1.50	0.0	n/a	n/a	n/a	0.5070	0.5070	0.338
Z-Z, +D+0.750L+0.750S+H	1.50	0.0	n/a	n/a	n/a	1.029	1.029	0.686
Z-Z, +D+0.60W+H	1.50	11.562	n/a	n/a	n/a	0.0	1.038	0.692
Z-Z, +D+0.70E+H	1.50	7.434	n/a	n/a	n/a	0.1299	0.8360	0.557
Z-Z, +D+0.750Lr+0.750L+0.450W+H	1.50	8.260	n/a	n/a	n/a	0.09518	0.9188	0.613
Z-Z, +D+0.750L+0.750S+0.450W+H	1.50	4.072	n/a	n/a	n/a	0.6167	1.440	0.960
Z-Z, +D+0.750L+0.750S+0.5250E+H	1.50	2.618	n/a	n/a	n/a	0.7637	1.293	0.862
Z-Z, +0.60D+0.60W+0.60H	1.50	19.270	n/a	n/a	n/a	0.0	1.063	0.709
Z-Z, +0.60D+0.70E+0.60H	1.50	12.390	n/a	n/a	n/a	0.0	0.6520	0.435

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, +D+H	None	0.0 k-ft	Infinity	OK
X-X, +D+L+H	None	0.0 k-ft	Infinity	OK
X-X, +D+Lr+H	None	0.0 k-ft	Infinity	OK
X-X, +D+S+H	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr+0.750L+H	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750L+0.750S+H	None	0.0 k-ft	Infinity	OK
X-X, +D+0.60W+H	None	0.0 k-ft	Infinity	OK
X-X, +D+0.70E+H	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr+0.750L+0.450W+H	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750L+0.750S+0.450W+H	None	0.0 k-ft	Infinity	OK

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

Project Title:
Engineer:
Project Descr:

Project ID:

General Footing

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ENERCALC, INC. 1983-2016, Build:6.16.5.11, Ver:6.16.5.11

Lic. # : KW-06004645 Licensee : LEI CONSULTING ENGINEERS

Description : FT11 at Right Column

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, +D+0.750L+0.750S+0.5250E+H	None	0.0 k-ft	Infinity	OK
X-X, +0.60D+0.60W+0.60H	None	0.0 k-ft	Infinity	OK
X-X, +0.60D+0.70E+0.60H	None	0.0 k-ft	Infinity	OK
Z-Z, +D+H	None	0.0 k-ft	Infinity	OK
Z-Z, +D+L+H	None	0.0 k-ft	Infinity	OK
Z-Z, +D+Lr+H	None	0.0 k-ft	Infinity	OK
Z-Z, +D+S+H	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.750Lr+0.750L+H	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.750L+0.750S+H	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.60W+H	8.143 k-ft	21.129 k-ft	2.595	OK
Z-Z, +D+0.70E+H	5.236 k-ft	21.129 k-ft	4.035	OK
Z-Z, +D+0.750Lr+0.750L+0.450W+H	6.107 k-ft	22.181 k-ft	3.632	OK
Z-Z, +D+0.750L+0.750S+0.450W+H	6.107 k-ft	44.998 k-ft	7.368	OK
Z-Z, +D+0.750L+0.750S+0.5250E+H	3.927 k-ft	44.998 k-ft	11.459	OK
Z-Z, +0.60D+0.60W+0.60H	8.143 k-ft	12.678 k-ft	1.557	OK
Z-Z, +0.60D+0.70E+0.60H	5.236 k-ft	12.678 k-ft	2.421	OK

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
X-X, +D+H	0.0 k	4.723 k	No Sliding	OK
X-X, +D+L+H	0.0 k	4.891 k	No Sliding	OK
X-X, +D+Lr+H	0.0 k	4.723 k	No Sliding	OK
X-X, +D+S+H	0.0 k	8.374 k	No Sliding	OK
X-X, +D+0.750Lr+0.750L+H	0.0 k	4.849 k	No Sliding	OK
X-X, +D+0.750L+0.750S+H	0.0 k	7.587 k	No Sliding	OK
X-X, +D+0.60W+H	2.036 k	4.723 k	2.320	OK
X-X, +D+0.70E+H	1.309 k	4.723 k	3.608	OK
X-X, +D+0.750Lr+0.750L+0.450W+H	1.527 k	4.849 k	3.176	OK
X-X, +D+0.750L+0.750S+0.450W+H	1.527 k	7.587 k	4.969	OK
X-X, +D+0.750L+0.750S+0.5250E+H	0.9818 k	7.587 k	7.728	OK
X-X, +0.60D+0.60W+0.60H	2.036 k	3.709 k	1.822	OK
X-X, +0.60D+0.70E+0.60H	1.309 k	3.709 k	2.833	OK
Z-Z, +D+H	0.0 k	5.661 k	No Sliding	OK
Z-Z, +D+L+H	0.0 k	5.829 k	No Sliding	OK
Z-Z, +D+Lr+H	0.0 k	5.661 k	No Sliding	OK
Z-Z, +D+S+H	0.0 k	9.311 k	No Sliding	OK
Z-Z, +D+0.750Lr+0.750L+H	0.0 k	5.787 k	No Sliding	OK
Z-Z, +D+0.750L+0.750S+H	0.0 k	8.525 k	No Sliding	OK
Z-Z, +D+0.750L+0.750S+0.450W+H	0.0 k	8.525 k	No Sliding	OK
Z-Z, +D+0.750L+0.750S+0.5250E+H	0.0 k	8.525 k	No Sliding	OK
Z-Z, +0.60D+0.60W+0.60H	0.0 k	4.646 k	No Sliding	OK
Z-Z, +0.60D+0.70E+0.60H	0.0 k	4.646 k	No Sliding	OK
Z-Z, +D+0.60W+H	0.0 k	5.661 k	No Sliding	OK
Z-Z, +D+0.70E+H	0.0 k	5.661 k	No Sliding	OK
Z-Z, +D+0.750Lr+0.750L+0.450W+H	0.0 k	5.787 k	No Sliding	OK

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D+1.60H	0.1468	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.40D+1.60H	0.1468	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+0.50Lr+1.60L+1.60H	0.1714	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+0.50Lr+1.60L+1.60H	0.1714	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+1.60L+0.50S+1.60H	0.4804	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+1.60L+0.50S+1.60H	0.4804	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+1.60Lr+0.50L+1.60H	0.1401	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+1.60Lr+0.50L+1.60H	0.1401	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+1.60Lr+0.50W+1.60H	0.1258	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+1.60Lr+0.50W+1.60H	0.1258	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+0.50L+1.60S+1.60H	1.129	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+0.50L+1.60S+1.60H	1.129	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

Project Title:
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Project Descr:

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General Footing

Description : FT11 at Right Column

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.20D+1.60S+0.50W+1.60H	1.115	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+1.60S+0.50W+1.60H	1.115	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+0.50Lr+0.50L+W+1.60H	0.1401	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+0.50Lr+0.50L+W+1.60H	0.1401	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+0.50L+0.50S+W+1.60H	0.4491	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+0.50L+0.50S+W+1.60H	0.4491	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+0.50L+0.70S+E+1.60H	0.5727	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +1.20D+0.50L+0.70S+E+1.60H	0.5727	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +0.90D+W+0.90H	0.09436	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +0.90D+W+0.90H	0.09436	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +0.90D+E+0.90H	0.09436	+Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
X-X, +0.90D+E+0.90H	0.09436	-Z	Bottom	0.2592	Min Temp %	0.280	10.925	OK
Z-Z, +1.40D+1.60H	0.3303	-X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.40D+1.60H	0.3303	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+0.50Lr+1.60L+1.60H	0.3857	-X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+0.50Lr+1.60L+1.60H	0.3857	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+1.60L+0.50S+1.60H	1.081	-X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+1.60L+0.50S+1.60H	1.081	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+1.60Lr+0.50L+1.60H	0.3151	-X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+1.60Lr+0.50L+1.60H	0.3151	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+1.60Lr+0.50W+1.60H	0.3992	-X	Top	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+1.60Lr+0.50W+1.60H	0.9653	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+0.50L+1.60S+1.60H	2.540	-X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+0.50L+1.60S+1.60H	2.540	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+1.60S+0.50W+1.60H	1.826	-X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+1.60S+0.50W+1.60H	3.191	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+0.50Lr+0.50L+W+1.60H	0.8598	-X	Top	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+0.50Lr+0.50L+W+1.60H	1.823	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+0.50L+0.50S+W+1.60H	0.3540	-X	Top	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+0.50L+0.50S+W+1.60H	2.375	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+0.50L+0.70S+E+1.60H	0.5366	-X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +1.20D+0.50L+0.70S+E+1.60H	2.041	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +0.90D+W+0.90H	0.6570	-X	Top	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +0.90D+W+0.90H	2.133	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +0.90D+E+0.90H	0.5267	-X	Top	0.2592	Min Temp %	0.2857	11.139	OK
Z-Z, +0.90D+E+0.90H	0.9734	+X	Bottom	0.2592	Min Temp %	0.2857	11.139	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D+1.60H	1.911 psi	1.911 psi	0.8919 psi	0.8919 psi	1.911 psi	75 psi	0.02548	OK
+1.20D+0.50Lr+1.60L+1.60H	2.232 psi	2.232 psi	1.042 psi	1.042 psi	2.232 psi	75 psi	0.02976	OK
+1.20D+1.60L+0.50S+1.60H	6.256 psi	6.256 psi	2.919 psi	2.919 psi	6.256 psi	75 psi	0.08341	OK
+1.20D+1.60Lr+0.50L+1.60H	1.824 psi	1.824 psi	0.8511 psi	0.8511 psi	1.824 psi	75 psi	0.02432	OK
+1.20D+1.60Lr+0.50W+1.60H	2.401 psi	5.678 psi	0.7645 psi	0.7645 psi	5.678 psi	75 psi	0.0757	OK
+1.20D+0.50L+1.60S+1.60H	14.701 psi	14.701 psi	6.86 psi	6.86 psi	14.701 psi	75 psi	0.196	OK
+1.20D+1.60S+0.50W+1.60H	10.476 psi	18.555 psi	6.774 psi	6.774 psi	18.555 psi	75 psi	0.2474	OK
+1.20D+0.50Lr+0.50L+W+1.60H	5.069 psi	10.777 psi	0.8511 psi	0.8511 psi	10.777 psi	75 psi	0.1437	OK
+1.20D+0.50L+0.50S+W+1.60H	2.231 psi	13.926 psi	2.729 psi	2.729 psi	13.926 psi	75 psi	0.1857	OK
+1.20D+0.50L+0.70S+E+1.60H	3.005 psi	11.91 psi	3.48 psi	3.48 psi	11.91 psi	75 psi	0.1588	OK
+0.90D+W+0.90H	3.802 psi	12.808 psi	0.5734 psi	0.5734 psi	12.808 psi	75 psi	0.1708	OK
+0.90D+E+0.90H	3.181 psi	5.736 psi	0.5734 psi	0.5734 psi	5.736 psi	75 psi	0.07648	OK

Punching Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D+1.60H	3.327 psi	150 psi	0.02218	OK
+1.20D+0.50Lr+1.60L+1.60H	3.885 psi	150 psi	0.0259	OK
+1.20D+1.60L+0.50S+1.60H	10.89 psi	150 psi	0.0726	OK
+1.20D+1.60Lr+0.50L+1.60H	3.175 psi	150 psi	0.02116	OK
+1.20D+1.60Lr+0.50W+1.60H	2.852 psi	150 psi	0.01901	OK
+1.20D+0.50L+1.60S+1.60H	25.59 psi	150 psi	0.1706	OK
+1.20D+1.60S+0.50W+1.60H	25.267 psi	150 psi	0.1684	OK
+1.20D+0.50Lr+0.50L+W+1.60H	3.552 psi	150 psi	0.02368	OK
+1.20D+0.50L+0.50S+W+1.60H	10.179 psi	150 psi	0.06786	OK

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

General Footing

Description : FT11 at Right Column

Punching Shear All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.20D+0.50L+0.70S+E+1.60H	12.981 psi	150psi	0.08654	OK
+0.90D+W+0.90H	3.389 psi	150psi	0.0226	OK
+0.90D+E+0.90H	2.156 psi	150psi	0.01437	OK

Concrete Column

Description : Concrete Pier Check

Code References

Calculations per ACI 318-11, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : IBC 2015

General Information

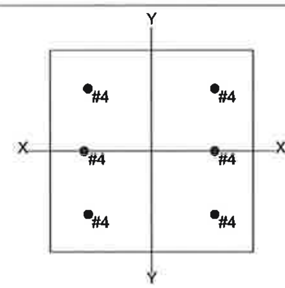
f'c : Concrete 28 day strength	=	2.50 ksi
E =	=	3,122.0 ksi
Density	=	150.0 pcf
β	=	0.850
fy - Main Rebar	=	60.0 ksi
E - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits		*ASTM A615 Bars Used*
Min. Reinf.	=	1.0 %
Max. Reinf.	=	8.0 %

Overall Column Height = 3.0 ft
End Fixity Top Free, Bottom Fixed
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 3.0 ft, K = 2.10
Y-Y (depth) axis :
Unbraced Length for X-X Axis buckling = 3.0 ft, K = 2.10

Column Cross Section

Column Dimensions : 12.0in Square Column, Column Edge to Rebar Edge Cover = 2.0in

Column Reinforcing : 4 - #4 bars @ corners,, 1 - #4 bars left & right between corner bars



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included : 450.0 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 3.0 ft above base, D = 4.254, L = 1.351, S = 41.654 k
BENDING LOADS . . .
Lat. Point Load at 3.0 ft creating Mx-x, W = 7.994, E = 5.499 k

DESIGN SUMMARY

Load Combination +0.90D+W+0.90H
Location of max.above base 2.980 ft

Maximum Stress Ratio **0.895 : 1**
Ratio = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5

Pu =	4.234 k	φ * Pn =	5.568 k
Mu-x =	-23.982 k-ft	φ * Mn-x =	27.173 k-ft
Mu-y =	0.0 k-ft	φ * Mn-y =	0.2288 k-ft
Mu Angle =	180.0 deg		
Mu at Angle =	23.982 k-ft	φMn at Angle =	26.789 k-ft

Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . . .

Pnmax : Nominal Max. Compressive Axial Capacity	375.450 k
Pnmin : Nominal Min. Tension Axial Capacity	-72.0 k
φ Pn, max : Usable Compressive Axial Capacity	195.234 k
φ Pn, min : Usable Tension Axial Capacity	-46.80 k

Maximum SERVICE Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	7.994 k

Maximum SERVICE Load Deflections . . .

Along Y-Y 0.02293 in at 3.0 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination :

General Section Information . φ = 0.650 β = 0.850 θ = 0.80

ρ : % Reinforcing	0.8333 %	Rebar < Min of 1.0 %
Reinforcing Area	1.20 in^2	
Concrete Area	144.0 in^2	

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	φ * Pn	δx	δx * Mux	δy	δy * Muy	Bending Analysis k-ft Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.40D+1.60H			2.98	6.59	195.23					0.000			**0.034**

Concrete Column

Description : Concrete Pier Check

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	φ * Pn	δ x	δ x * Mux	δ y	δ y * Muy	Bending Analysis k-ft Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.20D+0.50Lr+1.60L+1.60H			2.98	7.81	195.23					0.000			**0.040**
+1.20D+1.60L+0.50S+1.60H			2.98	28.63	195.23					0.000			**0.147**
+1.20D+1.60Lr+0.50L+1.60H			2.98	6.32	195.23					0.000			**0.032**
+1.20D+1.60Lr+0.50W+1.60H	Actual		2.98	5.64	13.49	1.000	-11.99			180.000	11.99	29.31	**0.409**
+1.20D+0.50L+1.60S+1.60H			2.98	72.97	195.23					0.000			**0.374**
+1.20D+1.60S+0.50W+1.60H	Actual		2.98	72.29	158.13	1.000	-11.99			180.000	11.99	26.45	**0.454**
+1.20D+0.50Lr+0.50L+W+1.60H	Actual		2.98	6.32	7.34	1.000	-23.98			180.000	23.98	27.36	**0.876**
+1.20D+0.50L+0.50S+W+1.60H	Actual		2.98	27.15	41.18	1.000	-23.98			180.000	23.98	36.74	**0.653**
+1.20D+0.50L+0.70S+E+1.60H	Actual		2.98	35.48	73.12	1.000	-16.50			180.000	16.50	33.95	**0.486**
+0.90D+W+0.90H	Actual		2.98	4.23	5.57	1.000	-23.98			180.000	23.98	26.79	**0.895**
+0.90D+E+0.90H	Actual		2.98	4.23	7.34	1.000	-16.50			180.000	16.50	27.36	**0.603**

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Reaction along X-X Axis @ Base	@ Top	Reaction along Y-Y Axis @ Base	@ Top	Axial Reaction @ Base
+D+H		k		k	4.704 k
+D+L+H		k		k	6.055 k
+D+Lr+H		k		k	4.704 k
+D+S+H		k		k	46.358 k
+D+0.750Lr+0.750L+H		k		k	5.717 k
+D+0.750L+0.750S+H		k		k	36.958 k
+D+0.60W+H		k	4.796	k	4.704 k
+D+0.70E+H		k	3.849	k	4.704 k
+D+0.750Lr+0.750L+0.450W+H		k	3.597	k	5.717 k
+D+0.750L+0.750S+0.450W+H		k	3.597	k	36.958 k
+D+0.750L+0.750S+0.5250E+H		k	2.887	k	36.958 k
+0.60D+0.60W+0.60H		k	4.796	k	2.822 k
+0.60D+0.70E+0.60H		k	3.849	k	2.822 k
D Only		k		k	4.704 k
Lr Only		k		k	k
L Only		k		k	1.351 k
S Only		k		k	41.654 k
W Only		k	7.994	k	k
E Only		k	5.499	k	k
H Only		k		k	k

Maximum Moments

Note: Only non-zero reactions are listed.

Load Combination	Moment About X-X Axis @ Base	@ Top	Moment About Y-Y Axis @ Base	@ Top
+D+H		k-ft		k-ft
+D+L+H		k-ft		k-ft
+D+Lr+H		k-ft		k-ft
+D+S+H		k-ft		k-ft
+D+0.750Lr+0.750L+H		k-ft		k-ft
+D+0.750L+0.750S+H		k-ft		k-ft
+D+0.60W+H		k-ft		14.389 k-ft
+D+0.70E+H		k-ft		11.548 k-ft
+D+0.750Lr+0.750L+0.450W+H		k-ft		10.792 k-ft
+D+0.750L+0.750S+0.450W+H		k-ft		10.792 k-ft
+D+0.750L+0.750S+0.5250E+H		k-ft		8.661 k-ft
+0.60D+0.60W+0.60H		k-ft		14.389 k-ft
+0.60D+0.70E+0.60H		k-ft		11.548 k-ft
D Only		k-ft		k-ft
Lr Only		k-ft		k-ft
L Only		k-ft		k-ft
S Only		k-ft		k-ft
W Only		k-ft		23.982 k-ft
E Only		k-ft		16.497 k-ft
H Only		k-ft		k-ft

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

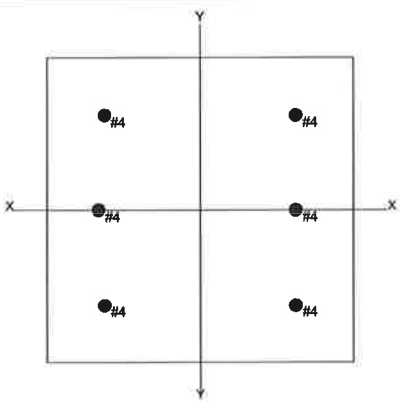
Concrete Column

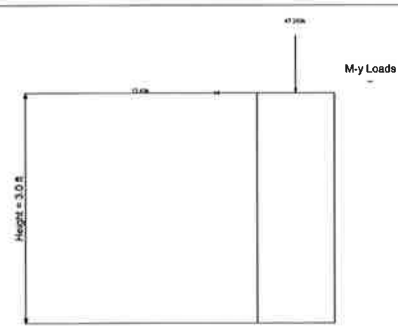
Description : Concrete Pier Check

Maximum Deflections for Load Combinations

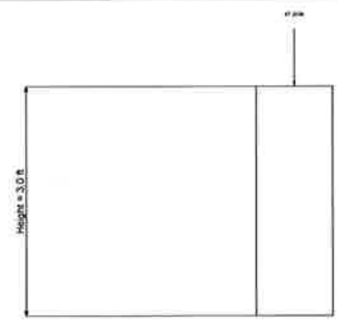
Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
+D+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.60W+H	0.0000	in	0.000	ft	0.014	in	3.000	ft
+D+0.70E+H	0.0000	in	0.000	ft	0.011	in	3.000	ft
+D+0.750Lr+0.750L+0.450W+H	0.0000	in	0.000	ft	0.010	in	3.000	ft
+D+0.750L+0.750S+0.450W+H	0.0000	in	0.000	ft	0.010	in	3.000	ft
+D+0.750L+0.750S+0.5250E+H	0.0000	in	0.000	ft	0.008	in	3.000	ft
+0.60D+0.60W+0.60H	0.0000	in	0.000	ft	0.014	in	3.000	ft
+0.60D+0.70E+0.60H	0.0000	in	0.000	ft	0.011	in	3.000	ft
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
W Only	0.0000	in	0.000	ft	0.023	in	3.000	ft
E Only	0.0000	in	0.000	ft	0.016	in	3.000	ft
H Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Sketches





Looking along X-X Axis



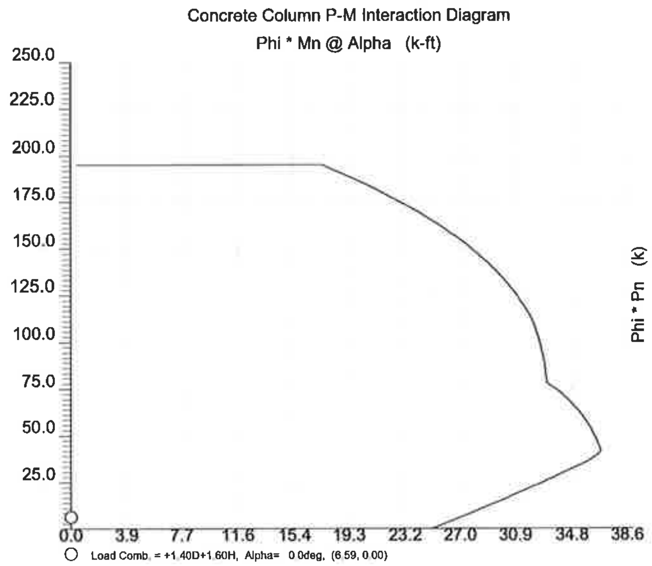
Looking along Y-Y Axis

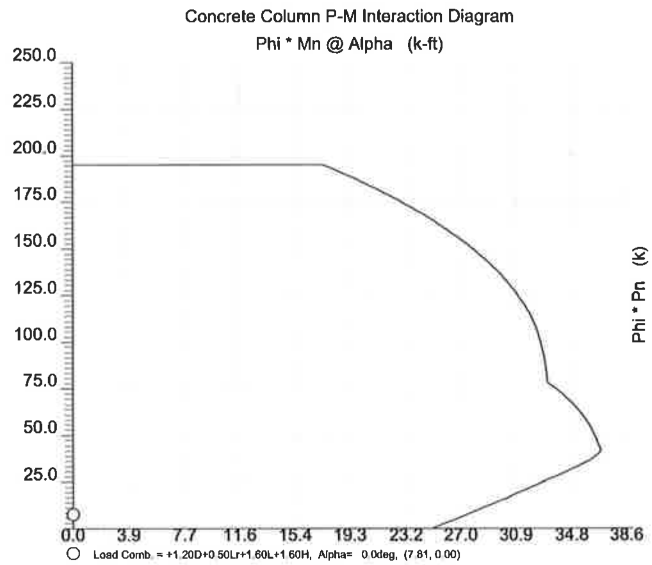
Interaction Diagrams

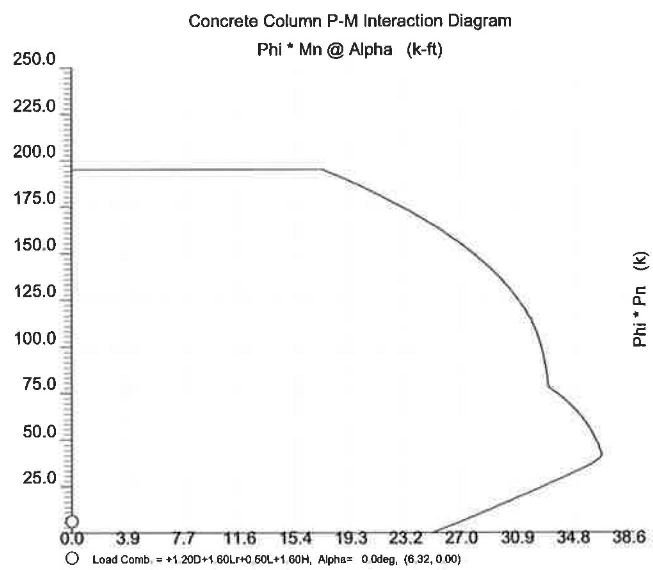
Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

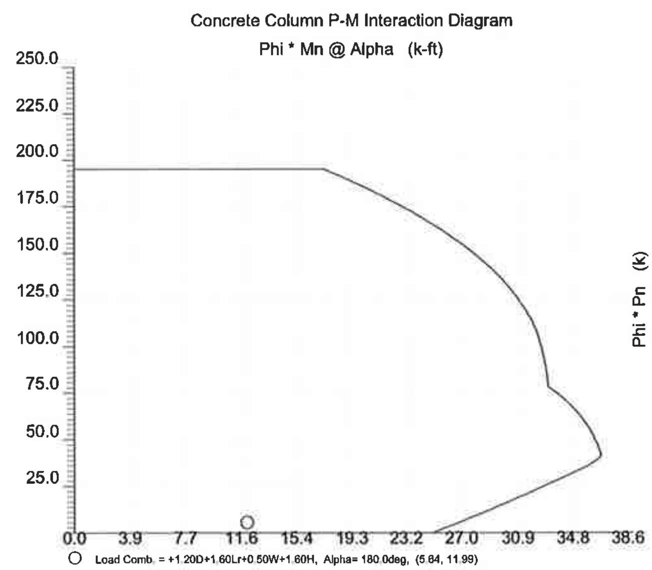
Concrete Column

Description : Concrete Pier Check





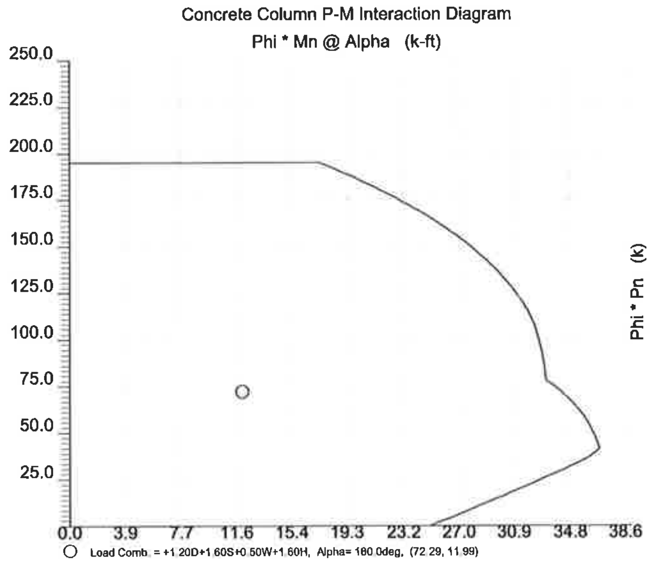


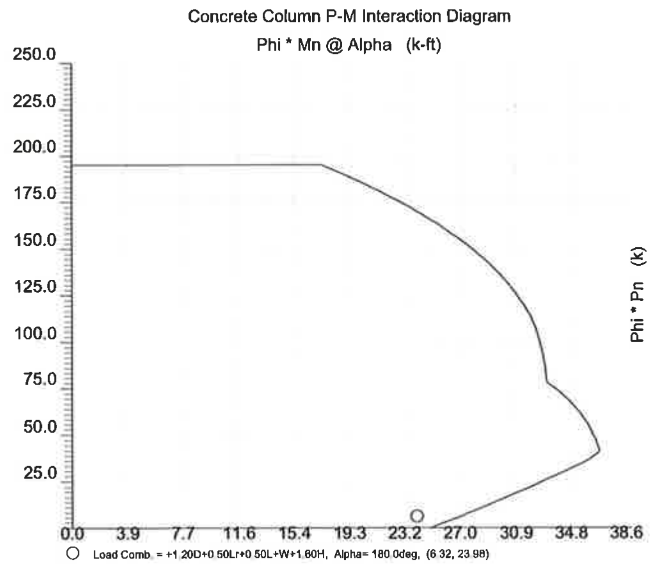


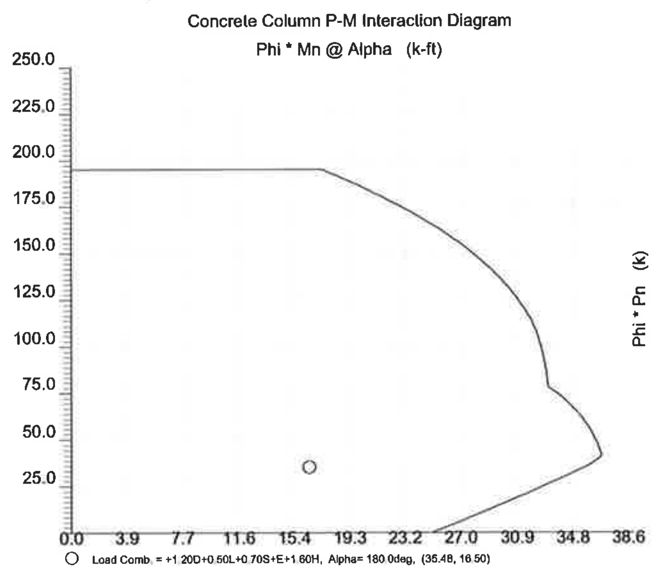
Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

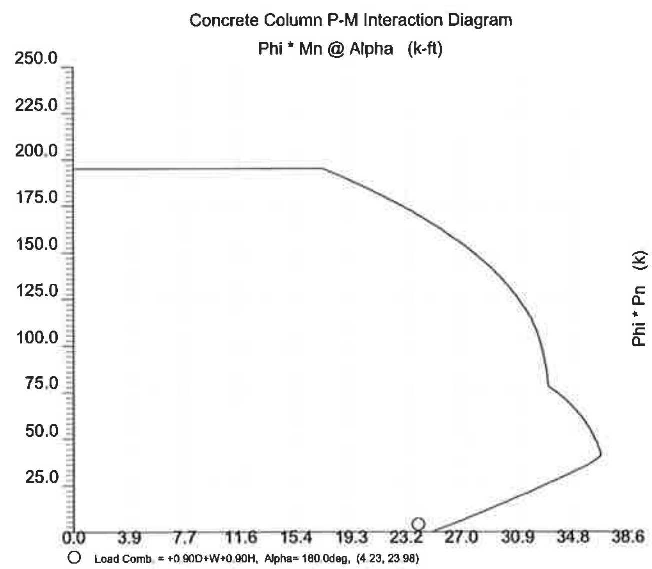
Concrete Column

Description : Concrete Pier Check









Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

Concrete Column

Description : FW11 at Point Load

Code References

Calculations per ACI 318-11, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

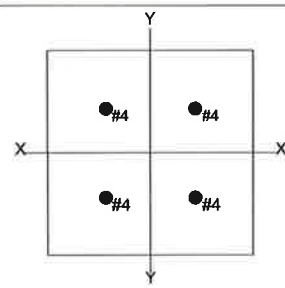
f'c : Concrete 28 day strength	=	2.50 ksi
E =	=	3,122.0 ksi
Density	=	150.0 pcf
β	=	0.850
fy - Main Rebar	=	60.0 ksi
E - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits		*ASTM A615 Bars Used*
Min. Reinf.	=	1.0 %
Max. Reinf.	=	8.0 %

Overall Column Height = 11.0 ft
End Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Fully braced against buckling along X-X Axis
Y-Y (depth) axis :
Fully braced against buckling along Y-Y Axis

Column Cross Section

Column Dimensions : 8.0in Square Column, Column Edge to Rebar Edge Cover = 2.0in

Column Reinforcing : 4 - #4 bars @ corners,



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included : 733.33 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 11.0 ft above base, D = 4.009, L = 6.045, S = 19.423 k

DESIGN SUMMARY

Load Combination +1.20D+0.50L+1.60S+1.60H
Location of max.above base 10.926 ft

Maximum Stress Ratio **0.420 : 1**
Ratio = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5

Pu =	39.790 k	φ * Pn =	94.796 k
Mu-x =	0.0 k-ft	φ * Mn-x =	0.0 k-ft
Mu-y =	0.0 k-ft	φ * Mn-y =	0.0 k-ft
Mu Angle =	0.0 deg		
Mu at Angle =	0.0 k-ft	φMn at Angle =	0.0 k-ft

Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . . .

Pnmax : Nominal Max. Compressive Axial Capacity	182.30 k
Pnmin : Nominal Min. Tension Axial Capacity	-48.0 k
φ Pn, max : Usable Compressive Axial Capacity	94.796 k
φ Pn, min : Usable Tension Axial Capacity	-31.20 k

Maximum SERVICE Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination :

Along X-X 0.0 in at 0.0 ft above base
for load combination :

General Section Information . φ = 0.650 β = 0.850 θ = 0.80

ρ : % Reinforcing	1.250 %	Rebar % Ok
Reinforcing Area	0.80 in^2	
Concrete Area	64.0 in^2	

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	Axial Load k φ * Pn	δ x	δ x * Mux	δ y	δ y * Muy	Bending Analysis k-ft Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.40D+1.60H			10.93	6.64	94.80					0.000			**0.070**
+1.20D+0.50Lr+1.60L+1.60H			10.93	15.36	94.80					0.000			**0.162**
+1.20D+1.60L+0.50S+1.60H			10.93	25.07	94.80					0.000			**0.265**

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

Project Title:
Engineer:
Project Descr:

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Concrete Column

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Description : FW11 at Point Load

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	Axial Load k φ * Pn	δ x	δ x * Mux	δ y	δ y * Muy	Bending Analysis k-ft Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.20D+1.60Lr+0.50L+1.60H			10.93	8.71	94.80					0.000			**0.092**
+1.20D+1.60Lr+0.50W+1.60H			10.93	5.69	94.80					0.000			**0.060**
+1.20D+0.50L+1.60S+1.60H			10.93	39.79	94.80					0.000			**0.420**
+1.20D+1.60S+0.50W+1.60H			10.93	36.77	94.80					0.000			**0.388**
+1.20D+0.50Lr+0.50L+W+1.60H			10.93	8.71	94.80					0.000			**0.092**
+1.20D+0.50L+0.50S+W+1.60H			10.93	18.42	94.80					0.000			**0.194**
+1.20D+0.50L+0.20S+E+1.60H			10.93	12.60	94.80					0.000			**0.133**
+0.90D+W+0.90H			10.93	4.27	94.80					0.000			**0.045**
+0.90D+E+0.90H			10.93	4.27	94.80					0.000			**0.045**

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Reaction along X-X Axis @ Base	Reaction along X-X Axis @ Top	Reaction along Y-Y Axis @ Base	Reaction along Y-Y Axis @ Top	Axial Reaction @ Base
+D+H		k		k	4.742 k
+D+L+H		k		k	10.787 k
+D+Lr+H		k		k	4.742 k
+D+S+H		k		k	24.165 k
+D+0.750Lr+0.750L+H		k		k	9.276 k
+D+0.750L+0.750S+H		k		k	23.843 k
+D+0.60W+H		k		k	4.742 k
+D+0.70E+H		k		k	4.742 k
+D+0.750Lr+0.750L+0.450W+H		k		k	9.276 k
+D+0.750L+0.750S+0.450W+H		k		k	23.843 k
+D+0.750L+0.750S+0.5250E+H		k		k	23.843 k
+0.60D+0.60W+0.60H		k		k	2.845 k
+0.60D+0.70E+0.60H		k		k	2.845 k
D Only		k		k	4.742 k
Lr Only		k		k	k
L Only		k		k	6.045 k
S Only		k		k	19.423 k
W Only		k		k	k
E Only		k		k	k
H Only		k		k	k

Maximum Moments

Note: Only non-zero reactions are listed.

Load Combination	Moment About X-X Axis @ Base	Moment About X-X Axis @ Top	Moment About Y-Y Axis @ Base	Moment About Y-Y Axis @ Top
+D+H		k-ft		k-ft
+D+L+H		k-ft		k-ft
+D+Lr+H		k-ft		k-ft
+D+S+H		k-ft		k-ft
+D+0.750Lr+0.750L+H		k-ft		k-ft
+D+0.750L+0.750S+H		k-ft		k-ft
+D+0.60W+H		k-ft		k-ft
+D+0.70E+H		k-ft		k-ft
+D+0.750Lr+0.750L+0.450W+H		k-ft		k-ft
+D+0.750L+0.750S+0.450W+H		k-ft		k-ft
+D+0.750L+0.750S+0.5250E+H		k-ft		k-ft
+0.60D+0.60W+0.60H		k-ft		k-ft
+0.60D+0.70E+0.60H		k-ft		k-ft
D Only		k-ft		k-ft
Lr Only		k-ft		k-ft
L Only		k-ft		k-ft
S Only		k-ft		k-ft
W Only		k-ft		k-ft
E Only		k-ft		k-ft
H Only		k-ft		k-ft

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

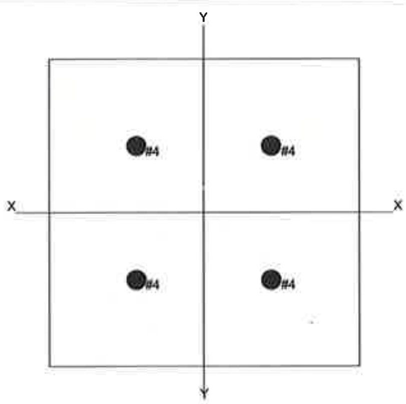
Concrete Column

Description : FW11 at Point Load

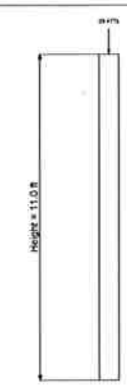
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
+D+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.60W+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.70E+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L+0.450W+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S+0.450W+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S+0.5250E+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D+0.60W+0.60H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D+0.70E+0.60H	0.0000	in	0.000	ft	0.000	in	0.000	ft
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
W Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
E Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
H Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Sketches



Looking along X-X Axis



Looking along Y-Y Axis

Interaction Diagrams

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

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Concrete Column

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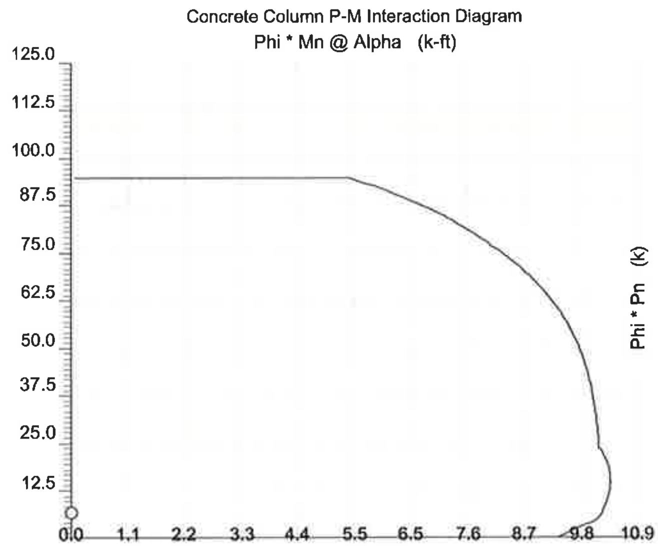
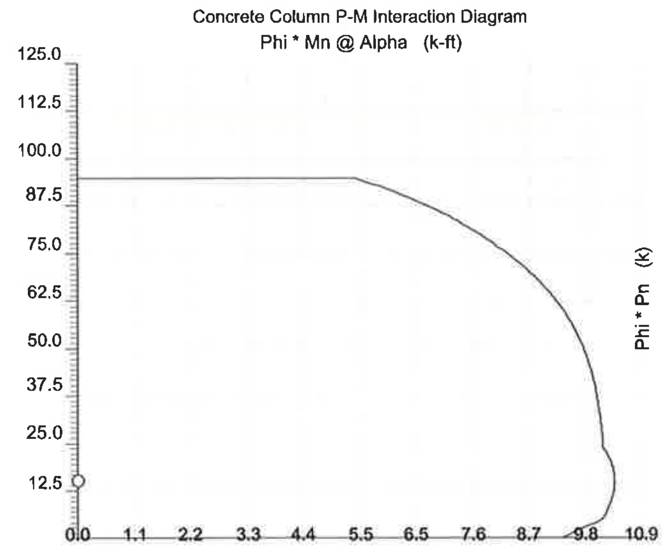
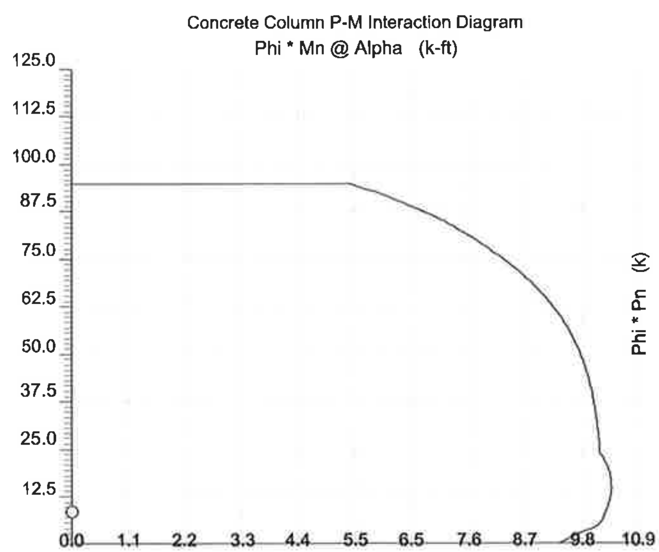
Description : FW11 at Point Load



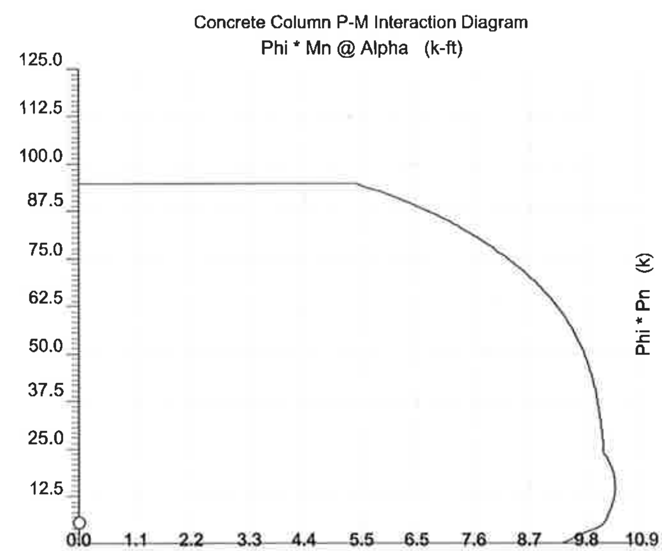
Load Comb. = +1.40D+1.60H, Alpha= 0.0deg, (6.64, 0.00)



Load Comb. = +1.20D+0.50Lr+1.60L+1.60H, Alpha= 0.0deg, (15.36, 0.00)



Load Comb. = +1.20D+1.60Lr+0.50L+1.60H, Alpha= 0.0deg, (8.71, 0.00)

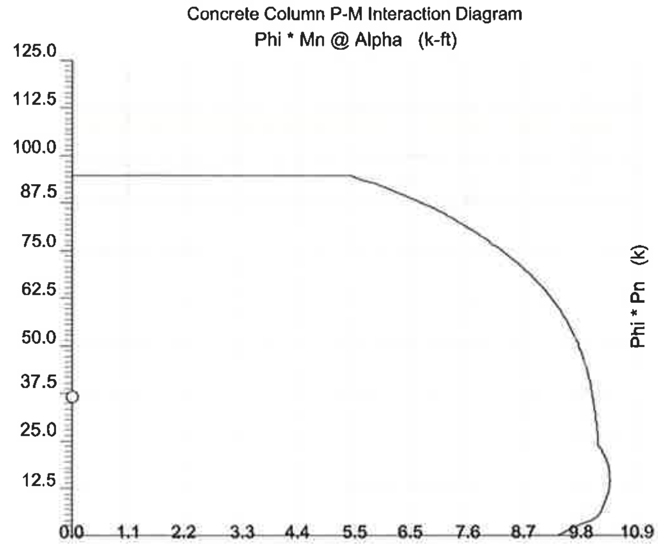


Load Comb. = +1.20D+1.60Lr+0.50W+1.60H, Alpha= 0.0deg, (5.69, 0.00)

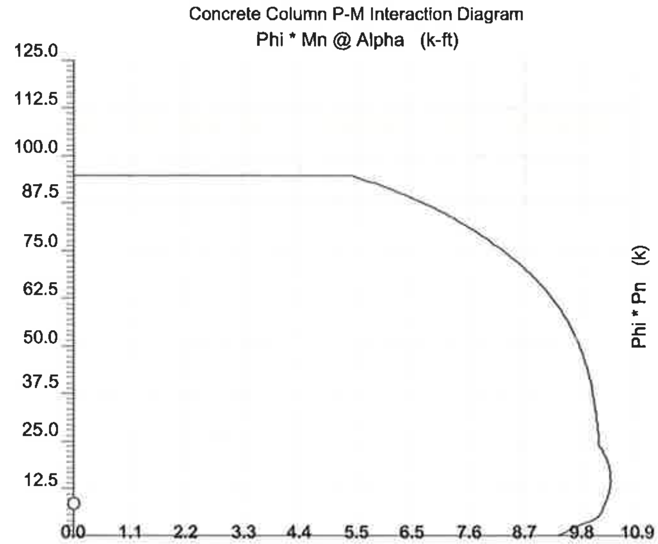
Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

Concrete Column

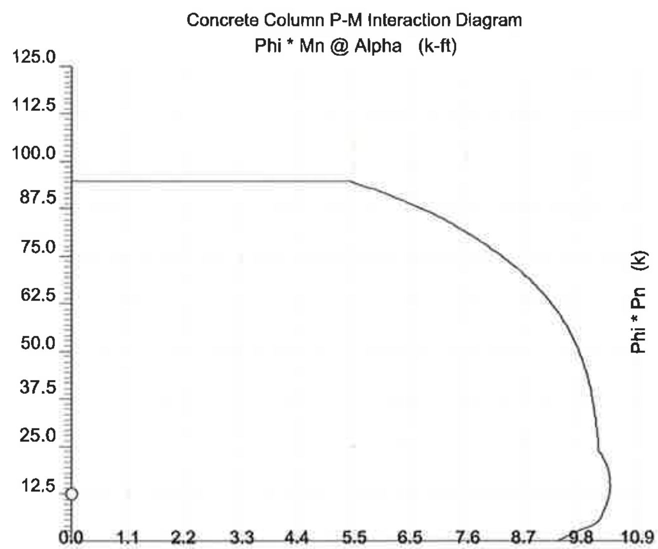
Description : FW11 at Point Load



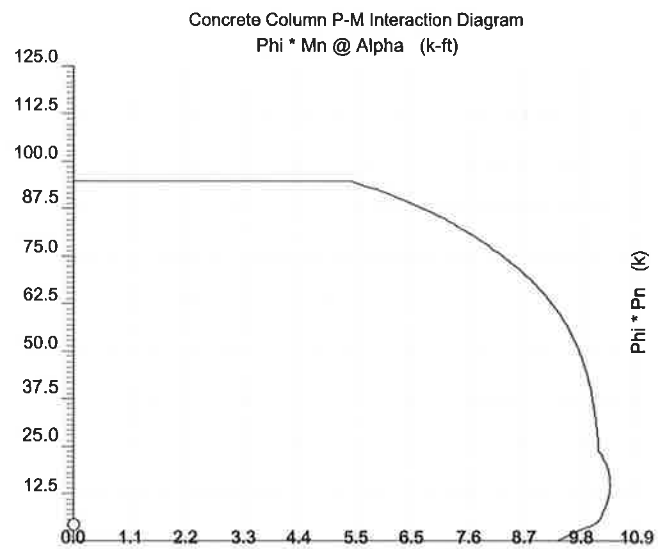
Load Comb. = +1.20D+1.60S+0.50W+1.60H, Alpha= 0.0deg, (36.77, 0.00)



Load Comb. = +1.20D+0.50Lr+0.50L+W+1.60H, Alpha= 0.0deg, (8.71, 0.00)



Load Comb. = +1.20D+0.50L+0.20S+E+1.60H, Alpha= 0.0deg, (12.60, 0.00)



Load Comb. = +0.90D+W+0.90H, Alpha= 0.0deg, (4.27, 0.00)

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

Concrete Beam

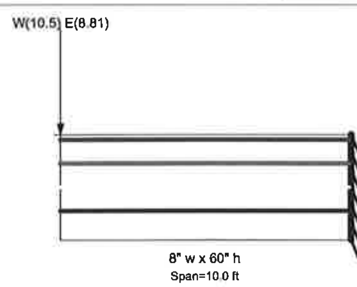
Description : Uplift Check at FW5

CODE REFERENCES

Calculations per ACI 318-11, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

f'c	=	2.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2} * 7.50$	=	375.0 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup =		2



Cross Section & Reinforcing Details

Rectangular Section, Width = 8.0 in, Height = 60.0 in
Span #1 Reinforcing....
- 1-#4 at 3.0 in from Top, from 0.0 to 10.0 ft in this span
- 1-#4 at 30.0 in from Top, from 0.0 to 10.0 ft in this span
- 1-#4 at 30.0 in from Bottom, from 0.0 to 10.0 ft in this span
- 1-#4 at 16.50 in from Top, from 0.0 to 10.0 ft in this span
- 1-#4 at 16.50 in from Bottom, from 0.0 to 10.0 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Point Load : W = 10.50, E = 8.810 k @ 0.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.885** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	-133.991	k-ft
Mn * Phi : Allowable	151.359	k-ft
Location of maximum on span	10.000	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection			
Max Downward Transient Deflection	0.013 in	Ratio =	17840 >=36
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.000 in	Ratio =	999 <180
Max Upward Total Deflection	0.000 in	Ratio =	999 <180

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum		11.133
Overall MINimum		4.833
+D+H		4.833
+D+L+H		4.833
+D+Lr+H		4.833
+D+S+H		4.833
+D+0.750Lr+0.750L+H		4.833
+D+0.750L+0.750S+H		4.833
+D+0.60W+H		11.133
+D+0.70E+H		11.000
+D+0.750Lr+0.750L+0.450W+H		9.558
+D+0.750L+0.750S+0.450W+H		9.558

Kelly Christensen
LEI Consulting Engineers
3302 N Main St.
Spanish Fork, UT
(801) 798-0555

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Concrete Beam

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Description : Uplift Check at FW5

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.750L+0.750S+0.5250E+H		9.459
+0.60D+0.60W+0.60H		9.200
+0.60D+0.70E+0.60H		9.067
D Only		4.833
Lr Only		
L Only		
S Only		
W Only		10.500
E Only		8.810
H Only		

Shear Stirrup Requirements

Between 0.00 to 7.43 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 11.4.6.1, use stirrups spaced at 0.000 in
Between 7.45 to 9.98 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 22.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	10.000	-133.99	151.36	0.89
+1.40D+1.60H					
Span # 1	1	10.000	-33.83	151.36	0.22
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	10.000	-29.00	151.36	0.19
+1.20D+1.60L+0.50S+1.60H					
Span # 1	1	10.000	-29.00	151.36	0.19
+1.20D+1.60Lr+0.50L+1.60H					
Span # 1	1	10.000	-29.00	151.36	0.19
+1.20D+1.60Lr+0.50W+1.60H					
Span # 1	1	10.000	-81.50	151.36	0.54
+1.20D+0.50L+1.60S+1.60H					
Span # 1	1	10.000	-29.00	151.36	0.19
+1.20D+1.60S+0.50W+1.60H					
Span # 1	1	10.000	-81.50	151.36	0.54
+1.20D+0.50Lr+0.50L+W+1.60H					
Span # 1	1	10.000	-133.99	151.36	0.89
+1.20D+0.50L+0.50S+W+1.60H					
Span # 1	1	10.000	-133.99	151.36	0.89
+1.20D+0.50L+0.20S+E+1.60H					
Span # 1	1	10.000	-117.09	151.36	0.77
+0.90D+W+0.90H					
Span # 1	1	10.000	-126.74	151.36	0.84
+0.90D+E+0.90H					
Span # 1	1	10.000	-109.84	151.36	0.73

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.0135	0.000		0.0000	0.000

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd	Suggest
+0.90D+E+0.90H	1	0.00	43.50	0.00	0.00	0.00	1.00	25.92	Vu < PhiVc/2	Not Reqd 1	25.9	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.02	50.25	-10.51	10.51	0.18	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.04	50.25	-10.52	10.52	0.37	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.05	50.25	-10.53	10.53	0.57	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.07	50.25	-10.54	10.54	0.76	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.09	50.25	-10.55	10.55	0.95	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.11	50.25	-10.56	10.56	1.14	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.13	50.25	-10.57	10.57	1.33	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.15	50.25	-10.58	10.58	1.53	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.16	50.25	-10.60	10.60	1.72	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.18	50.25	-10.61	10.61	1.91	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0
+1.20D+0.50L+0.50S+W+1.60	1	0.20	50.25	-10.62	10.62	2.11	1.00	30.14	Vu < PhiVc/2	Not Reqd 1	30.1	0.0	0.0

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	18.38	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	27.56	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	23.44	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	28.13	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	38.44	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	46.13	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: 2017-2259

Location: P8
Column
[2015 International Building Code(2015 NDS)]
5.25 IN x 5.25 IN x 9 FT
1.8E Parallam Column - iLevel Trus Joist
Section Adequate By: 11.0%

Kelly Christensen
LEI Consulting Engineers and Surveyers Inc.
3302 N. Main St.
Spanish Fork, UT 84660

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VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	27051	lb
Dead Load:	Vert-DL-Rxn =	2504	lb
Total Load:	Vert-TL-Rxn =	29555	lb

COLUMN DATA

Total Column Length:	9	ft
Unbraced Length (X-Axis) Lx:	9	ft
Unbraced Length (Y-Axis) Ly:	9	ft
Column End Condition-K (e):	1	
Load Eccentricity (X-Axis) - ex:	0.3	in
Load Eccentricity (Y-Axis) - ey:	0	in
Axial Load Duration Factor	1.00	

COLUMN PROPERTIES

1.8E Parallam Column - iLevel Trus Joist

	Base Values	Adjusted
Compressive Stress:	Fc = 2500 psi *Cd=1.00 Cp=0.61*	Fc' = 1534 psi
Bending Stress (X-X Axis):	Fbx = 2400 psi *Cd=1.00 CF=1.10*	Fbx' = 2631 psi
Bending Stress (Y-Y Axis):	Fby = 2400 psi *Cd=1.00 CF=1.10*	Fby' = 2631 psi
Modulus of Elasticity:	E = 1800 ksi	E' = 1800 ksi

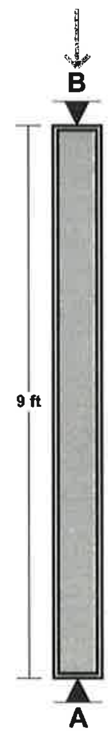
Column Section (X-X Axis):	dx =	5.25	in
Column Section (Y-Y Axis):	dy =	5.25	in
Area:	A =	27.56	in2
Section Modulus (X-X Axis):	Sx =	24.12	in3
Section Modulus (Y-Y Axis):	Sy =	24.12	in3
Slenderness Ratio:	Lex/dx =	20.57	
	Ley/dy =	20.57	

Column Calculations (Controlling Case Only):

Controlling Load Case: Axial Total Load Only (L + D)

Actual Compressive Stress:	Fc =	1072	psi
Allowable Compressive Stress:	Fc' =	1534	psi
Eccentricity Moment (X-X Axis):	Mx-ex =	737	ft-lb
Eccentricity Moment (Y-Y Axis):	My-ey =	0	ft-lb
Moment Due to Lateral Loads (X-X Axis):	Mx =	0	ft-lb
Moment Due to Lateral Loads (Y-Y Axis):	My =	0	ft-lb
Bending Stress Lateral Loads Only (X-X Axis):	Fbx =	0	psi
Allowable Bending Stress (X-X Axis):	Fbx' =	2631	psi
Bending Stress Lateral Loads Only (Y-Y Axis):	Fby =	0	psi
Allowable Bending Stress (Y-Y Axis):	Fby' =	2631	psi
Combined Stress Factor:	**CSF =**	**0.89**	

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	27051	lb
Dead Load:	PD =	2426	lb
Column Self Weight:	CSW =	78	lb
Total Axial Load:	PT =	29555	lb

NOTES

Project: 2017-2259

Location: P8 - Check
Column
[2015 International Building Code(2015 NDS)]
5.25 IN x 5.25 IN x 9 FT
1.8E Parallam Column - iLevel Trus Joist
Section Adequate By: 3.5%

Kelly Christensen
LEI Consulting Engineers and Surveyers Inc.
3302 N. Main St.
Spanish Fork, UT 84660

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VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	24690	lb
Dead Load:	Vert-DL-Rxn =	2598	lb
Total Load:	Vert-TL-Rxn =	27288	lb

COLUMN DATA

Total Column Length:	9	ft
Unbraced Length (X-Axis) Lx:	9	ft
Unbraced Length (Y-Axis) Ly:	9	ft
Column End Condition-K (e):	1	
Load Eccentricity (X-Axis) - ex:	0.5	in
Load Eccentricity (Y-Axis) - ey:	0	in
Axial Load Duration Factor	1.00	

COLUMN PROPERTIES

1.8E Parallam Column - iLevel Trus Joist

	Base Values	Adjusted
Compressive Stress:	Fc = 2500 psi *Cd=1.00 Cp=0.61*	Fc' = 1534 psi
Bending Stress (X-X Axis):	Fbx = 2400 psi *Cd=1.00 CF=1.10*	Fbx' = 2631 psi
Bending Stress (Y-Y Axis):	Fby = 2400 psi *Cd=1.00 CF=1.10*	Fby' = 2631 psi
Modulus of Elasticity:	E = 1800 ksi	E' = 1800 ksi

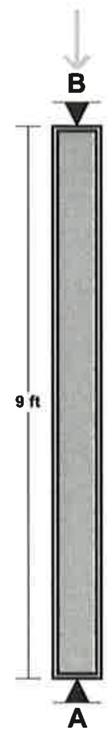
Column Section (X-X Axis):	dx =	5.25	in
Column Section (Y-Y Axis):	dy =	5.25	in
Area:	A =	27.56	in2
Section Modulus (X-X Axis):	Sx =	24.12	in3
Section Modulus (Y-Y Axis):	Sy =	24.12	in3
Slenderness Ratio:	Lex/dx =	20.57	
	Ley/dy =	20.57	

Column Calculations (Controlling Case Only):

Controlling Load Case: Axial Total Load Only (L + D)

Actual Compressive Stress:	Fc =	990	psi
Allowable Compressive Stress:	Fc' =	1534	psi
Eccentricity Moment (X-X Axis):	Mx-ex =	1134	ft-lb
Eccentricity Moment (Y-Y Axis):	My-ey =	0	ft-lb
Moment Due to Lateral Loads (X-X Axis):	Mx =	0	ft-lb
Moment Due to Lateral Loads (Y-Y Axis):	My =	0	ft-lb
Bending Stress Lateral Loads Only (X-X Axis):	Fbx =	0	psi
Allowable Bending Stress (X-X Axis):	Fbx' =	2631	psi
Bending Stress Lateral Loads Only (Y-Y Axis):	Fby =	0	psi
Allowable Bending Stress (Y-Y Axis):	Fby' =	2631	psi
Combined Stress Factor:	**CSF =**	**0.96**	

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	24690 lb
Dead Load:	PD =	2520 lb
Column Self Weight:	CSW =	78 lb
Total Axial Load:	PT =	27288 lb

NOTES

Project: 2017-2259

Location: P10
Column
[2015 International Building Code(AISC 14th Ed ASD)]
HSS 4 x 4 x 1/4 x 8.0 FT /ASTM A500-GR.B-46
Section Adequate By: 90.8%

Mike Larrabee
LEI Consulting Engineers
3302 N Main Street
Spanish Fork, UT 84660

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VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	12036	lb
Dead Load:	Vert-DL-Rxn =	985	lb
Total Load:	Vert-TL-Rxn =	13021	lb

COLUMN DATA

Total Column Length:	8	ft
Unbraced Length (X-Axis) Lx:	8	ft
Unbraced Length (Y-Axis) Ly:	8	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 = 63.16 KLy/ry = 63.16

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

Flexural Buckling Stress: Fcr = 35.17 ksi

Controlling Equation F7-1

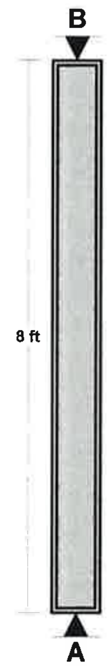
Nominal Compressive Strength: Pc = 71 kip

Combined Stress Calculations:

H1-1b Controls : 0.09

Controlling Combined Stress Factor: 0.09

LOADING DIAGRAM



NOTES

AXIAL LOADING

Live Load:	PL =	12036 lb
Dead Load:	PD =	886 lb
Column Self Weight:	CSW =	99 lb
Total Axial Load:	PT =	13021 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	5 1/4" x 5 1/4" Parallam Post
P9	W10x54 A992-50
P10	HSS 4 x 4 x 1/4 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]

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	2,4,5
2	7/16" OSB or CDX plywood	-	-	4"	12"	530	350	2,4,5

Notes:
1. Wall studs are to be spaced at 16" o.c. U.N.O.
2. Unit shear capacities are based on AF&PA SDPWS Table 4.3A (IBC 2306.3)
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 2-inch nominal or wider framing with edge nailing at all supports and panel edges U.N.O. (AF&PA SDPWS 4.3.7.1 note 1)
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 3-inch nominal or wider for edge nailing 3" o.c. or less. Nails at adjoining panel edges and into sill plates shall be staggered. (AF&PA SDPWS 4.3.7.1 note 3)

Footing Schedule

Designation	Length	Width	Depth	Lengthwise Reinforcement				Crosswise Reinforcement				Capacity	Note
				Qty.	Size	Length	Spacing	Qty.	Size	Length	Spacing		
FT1A	Cont.	20"	10"	2	#4	Cont.	EQ.	-	-	-	-	2500 PLF	
FT1B	Cont.	32"	10"	3	#4	Cont.	EQ.	-	#4	26"	12" o.c.	4000 PLF	
FT1C	Cont.	36"	10"	4	#4	Cont.	EQ.	-	#5	30"	12" o.c.	4500 PLF	
FT2	Cont.	20"	10"	2	#4	Cont.	EQ.	-	-	-	-	2500 PLF	See detail 19/SD.1
FT3	24"	24"	10"	3	#4	18"	EQ.	3	#4	18"	EQ.	6000 LBS	
FT4	30"	30"	10"	3	#4	24"	EQ.	3	#4	24"	EQ.	9375 LBS	
FT5	36"	36"	10"	4	#4	30"	EQ.	4	#4	30"	EQ.	13500 LBS	
FT6	42"	42"	10"	4	#4	36"	EQ.	4	#4	36"	EQ.	18375 LBS	
FT7	48"	48"	10"	5	#4	42"	EQ.	5	#4	42"	EQ.	24000 LBS	
FT8	60"	60"	12"	7	#4	54"	EQ.	7	#4	54"	EQ.	37500 LBS	
FT9	36"	36"	12"	4	#4	30"	EQ.	4	#4	30"	EQ.	-	
FT10	48"	48"	12"	6	#4	42"	EQ.	6	#4	42"	EQ.	-	
FT11	60"	42"	12"	5	#4	54"	EQ.	7	#4	36"	EQ.	-	

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. (36" 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

Designation	Thickness	Max Height	Vert. Reinforcement		Horizontal Reinforcement			Note
			Size	Spacing	Qty.	Size	Spacing	
FW3A	8"	3'-2"	#4	24"	3	#4	EQ.	
FW3B	12"	3'-2"	#4	24"	3	#4	EQ.	(2) mats of reinforcement. See 33/SD.2
FW5	8"	5'-0"	#4	24"	-	#4	12"	
FW12	8"	12'-0"	#4	9"	-	#4	12"	

Notes:
1. Use 1/2" diameter x 7" embedment anchor bolts @ 32" o.c. w/ 3"x3"x1/4" (0.229") plate washers at all exterior and shear walls U.N.O.
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.