STRUCTURAL CALCULATION
Prepared By

Urbala Development, LLC.

Residential Design*Development*Consulting
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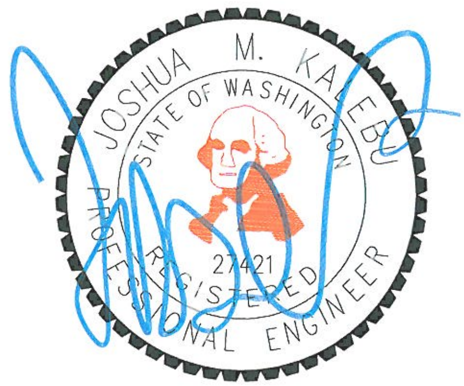
Job # 20-29E
(Revision)



Prepared For

Wind Speed = 110 mph, Exposure: "C"
YOUN CHUNG DADU
NEW CONSTRUCTION
7002 78th ST. SE.
MERCER ISLAND, WA. 98040

JUNE 12, 2021

Urbala Development LLC.
***Residential Design*Development*Consulting**
33505 13th PL. S
Federal Way, WA 98003

BASIS OF DESIGN

BUILDING CODE:	2015 Edition of the International Building Code
GRAVITY LOADS:	Roof Live Load = 25 psf. (snow) Roof Dead Load = 15 psf. Floor Live Load = 40 psf. (reducible) DECK Floor Live Load = 60 psf. Floor Dead Load = 12 psf.
LATERAL LOAD CRITERIA:	Wind Speed = 110 mph, Exposure: C Seismic Site Class: D Seismic Design Category: D
FOUNDATION:	Spead footings shall bear on firm undisturbed soil 18" minimum below finished grade. Allowable Soil Bearing Pressure = 1,500 psf.

DESIGN LOADS

ROOF LIVE LOAD:		**25.00 PSF** (snow load)
ROOF DEAD LOAD:	Roofing	7.00 psf (15 psf = tile roof)
	1/2" Plywood Sheating	2.00 psf
	Insulation	2.00 psf
	Trusses & 1/2" GWB	3.00 psf (6 psf = tile roof)
	Lighting, Mechanical, Electrical, Misc.	1.00 psf
	Solar-ready zone system, Misc.	4.00 psf

		15.00 PSF Total Dead Load
Solar-ready zone system, Misc.		**4.00 psf**
FLOOR LIVE LOAD:		**40.00 PSF** (Reducible)
FLOOR DEAD LOAD:	Flooring	2.00 psf
	3/4" Plywood Sheathing	3.00 psf
	Insulation	1.00 psf
	Floor Joists & Beams	5.00 psf
	Lighting, Mechanical, Electrical, Misc.	1.00 pst

		12.00 PSF Total Dead Load

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WOOD:

JOISTS & FAFTERS:	2X4	HF#2
	2X6 or Larger	HF#2
BEAMS :	Width of 4" and Less	DF#2
	Width of greater than 4"	DF#2
LEDGERS AND TOP PLATES:		
	Width of 4" and Less	HF#2
STUDS:	2X4	HF#2
	2X6 or Larger	HF#2
POSTS:	4X4	HF#2 (unless note on plan)
	4X6 or Larger	HF#2 (unless note on plan)
	6X6 or Larger	HF#2 (unless note on plan)

GLUED-LAMINATED (GLB) BEAM & HEADER:
Fb=2,400 PSI, Fv=165 PSI, Fc (Perp) =650 PSI, E=1,800,000 PSI.

PARALLAM (PSL) 2.0E BEAM & HEADER:
Fb=2,900 PSI, Fv=290 PSI, Fc (Perp) =750 PSI, E=2,000,000 PSI.

MICROLAM (LVL) 1.9E BEAM & HEADER:
Fb=2,600 PSI, Fv=285 PSI, Pc (Perp) =750 PSI, E=1,900,000 PSI.

TIMBERSTRAND (LSL) 1.3E BEAM, HEADER, & RIM BOARD:
Fb=1,700 PSI, Fv=400 PSI, Pc (Perp) =680 PSI, E=1,300,000 PSI.

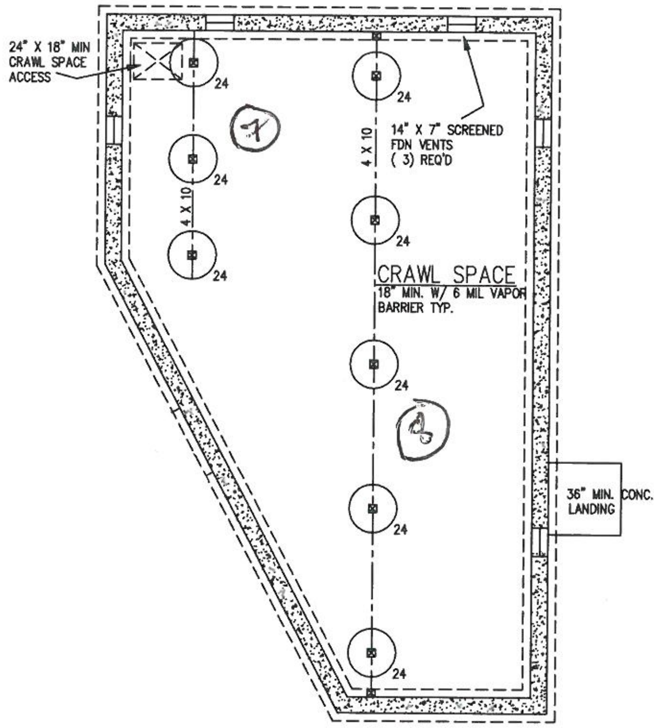
TRUSSES:

PREFABRICATED WOOD TRUSSES SHALL BE DESIGNED BY A REGISTERED DESIGN PROFESSIONAL REGISTERED IN THE STATE OF WASHINGTON.

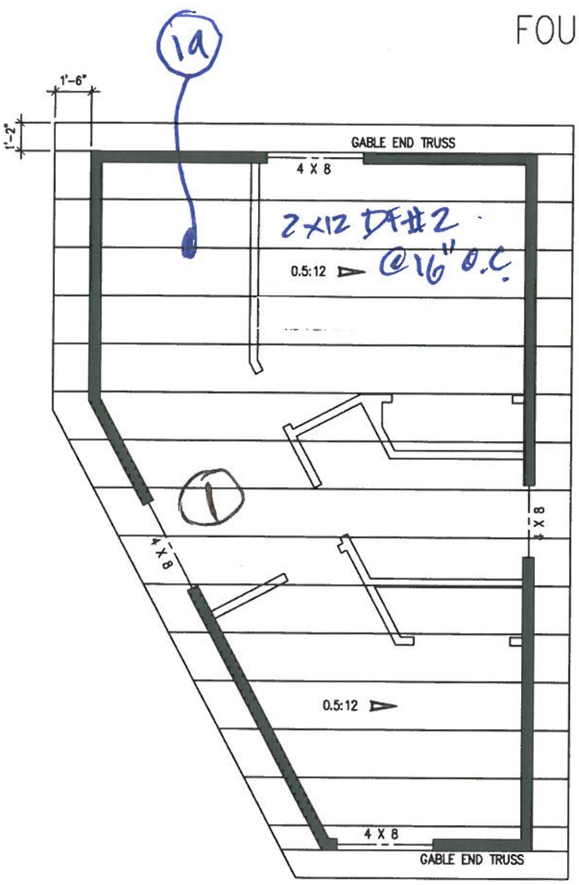
UNLESS OTHERWISE SPECIFIED BY LOCAL BUILDING OFFICIAL OR STATUTE, TRUSS DESIGNS BEARING THE SEAL AND SIGNATURE OF THE TRUSS DESIGNER SHALL BE AVAILABLE AT TIME OF INSPECTION.

ENGINEERED I-JOISTS

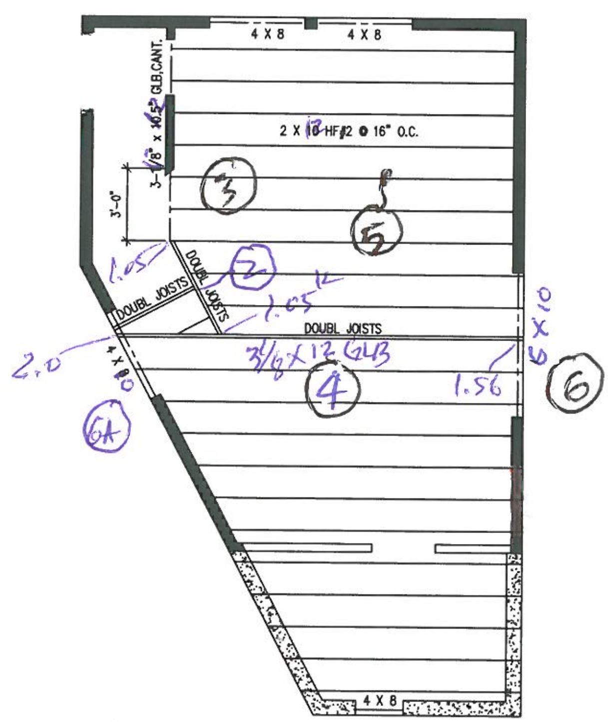
-FLOOR JOISTS & BEAMS OF EQUAL OR BETTER CAPACITY MAY BE SUBSTITUTED FOR THOSE SHOWN ON THIS PLAN, "EQUAL" IS DEFINED AS HAVING MOMENT CAPACITY, SHEAR CAPACITY, AND STIFFNESS WITHIN 3% OF THE SPECIFIED JOISTS OR BEAMS.



FOUNDATION PLAN



ROOF FRAMING PLAN



UPPER FLOOR FRAMING PLAN

1/8" = 1'-0"

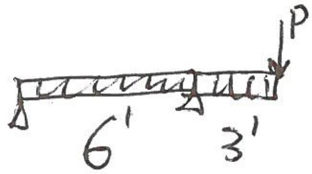
3

① $15(11) = 165^{\#}$
$25(11) = 275^{\#}$

4×8 DF#2

③ $12(9) + 80 = 188^{\#}$
$40(9) = 360^{\#}$

P = 1055 lb



4×12 DF#2

② $12(8) = 96^{\#}$
$40(8) = 320^{\#}$

(2) 2×12 HF#2

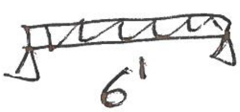
④ $12(1.33) + 80 = 96^{\#}$
$40(1.33) = 52$

$3\frac{1}{8} \times 10\frac{1}{2}$" GLB

⑤ $16^{\#} \neq 52^{\#}$

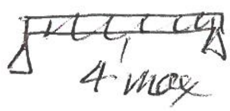
2×12 HF#2

⑥ $15(11) + 80 + 12(8) = 341^{\#}$
$25(11) + 40(8) = 595^{\#}$



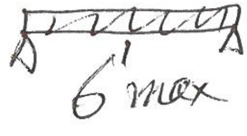
6×10 DF#2

⑦ $12(10) + 80 + 12(8) = 296^{\#}$
$40(10) + 40(8) = 720^{\#}$



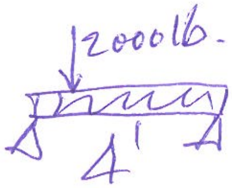
4×10 DF#2

⑧ $12(8) + 80 = 176^{\#}$
$40(8) = 320^{\#}$



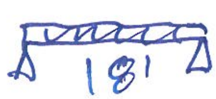
4×10 DF#2

6A Same load #6



4×10 DF#2

1a Rafter: $15(3/2) = 22.5^{\#}$
$25(3/2) = 37.5^{\#}$



2×12 DF#2 @16" O.C.

Rev: 580004

General Timber Beam

Description 1. HEADER

General Information

Section Name	**4x8**	Center Span	4.00 ft	.....Lu	0.00 ft
Beam Width	3.500 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth	7.250 in	Right Cantilever	ft	.....Lu	0.00 ft
Member Type	Sawn	Douglas Fir - Larch, No.2			
Bm Wt. Added to Loads		Fb Base Allow	900.0 psi		
Load Dur. Factor	1.150	Fv Allow	180.0 psi		
Beam End Fixity	Pin-Pin	Fc Allow	625.0 psi		
Wood Density	32.500 pcf	E	1,600.0 ksi		

Full Length Uniform Loads

Center	DL	165.00 #/ft	LL	275.00 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Summary

Beam Design OK

Span= 4.00ft, Beam Width = 3.500in x Depth = 7.25in, Ends are Pin-Pin

Max Stress Ratio 0.259 : 1

Maximum Moment	0.9 k-ft	Maximum Shear * 1.5	0.9 k
Allowable	3.4 k-ft	Allowable	5.3 k

Max. Positive Moment	0.89 k-ft	at	2.000 ft	Shear:	@ Left	0.89 k
Max. Negative Moment	0.00 k-ft	at	0.000 ft		@ Right	0.89 k
Max @ Left Support	0.00 k-ft			Camber:	@ Left	0.000 in
Max @ Right Support	0.00 k-ft				@ Center	0.008 in
Max. M allow	3.44				@ Right	0.000 in

Reactions...

fb	348.89 psi	fv	37.10 psi	Left DL	0.34 k	Max	0.89 k
Fb	1,345.50 psi	Fv	207.00 psi	Right DL	0.34 k	Max	0.89 k

Deflections

Center Span...	Dead Load	Total Load	**Left Cantilever...**	Dead Load	Total Load
Deflection	-0.006 in	-0.014 in	Deflection	0.000 in	0.000 in
...Location	2.000 ft	2.000 ft	...Length/Defl	0.0	0.0
...Length/Defl	8,680.6	3,324.93	**Right Cantilever...**		
Camber (using 1.5 * D.L. Defl) ...			Deflection	0.000 in	0.000 in
@ Center	0.008 in		...Length/Defl	0.0	0.0
@ Left	0.000 in				
@ Right	0.000 in				

Stress Calcs

Bending Analysis

Ck	31.887	Le	0.000 ft	Sxx	30.661 in3	Area	25.375 in2
Cf	1.300	Rb	0.000	Cl	891.454		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	0.89 k-ft	7.95 in3	1,345.50 psi
@ Left Support	0.00 k-ft	0.00 in3	1,345.50 psi
@ Right Support	0.00 k-ft	0.00 in3	1,345.50 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	0.94 k	0.94 k
Area Required	4.548 in2	4.548 in2
Fv: Allowable	207.00 psi	207.00 psi

Bearing @ Supports

Max. Left Reaction	0.89 k	Bearing Length Req'd	0.408 in
Max. Right Reaction	0.89 k	Bearing Length Req'd	0.408 in

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Rev: 580004

General Timber Beam

Description 1a- roof rafter

General Information

Section Name	**2x12**	Center Span	18.00 ft	.....Lu	0.00 ft
Beam Width	1.500 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth	11.250 in	Right Cantilever	ft	.....Lu	0.00 ft
Member Type	Sawn	Douglas Fir - Larch, No.2			
Bm Wt. Added to Loads		Fb Base Allow	900.0 psi		
Load Dur. Factor	1.150	Fv Allow	180.0 psi		
Beam End Fixity	Pin-Pin	Fc Allow	625.0 psi		**Repetitive Member**
Wood Density	10.000 pcf	E	1,600.0 ksi		

Full Length Uniform Loads

Center	DL	22.50 #/ft	LL	37.50 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Summary

Beam Design OK

Span= 18.00ft, Beam Width = 1.500in x Depth = 11.25in, Ends are Pin-Pin

Max Stress Ratio	0.789 : 1				
Maximum Moment	2.5 k-ft		Maximum Shear * 1.5		0.7 k
Allowable	3.1 k-ft		Allowable		3.5 k

Max. Positive Moment	2.48 k-ft	at	9.000 ft	Shear:		@ Left	0.55 k
Max. Negative Moment	0.00 k-ft	at	0.000 ft			@ Right	0.55 k
Max @ Left Support	0.00 k-ft			Camber:		@ Left	0.000 in
Max @ Right Support	0.00 k-ft					@ Center	0.295 in
Max. M allow	3.14			Reactions...		@ Right	0.000 in
fb 939.60 psi	fv	43.85 psi		Left DL	0.21 k	Max	0.55 k
Fb 1,190.25 psi	Fv	207.00 psi		Right DL	0.21 k	Max	0.55 k

Deflections

Center Span...	Dead Load	Total Load	**Left Cantilever...**	Dead Load	Total Load
Deflection	-0.196 in	-0.507 in	Deflection	0.000 in	0.000 in
...Location	9.000 ft	9.000 ft	...Length/Defl	0.0	0.0
...Length/Defl	1,100.1	425.72	**Right Cantilever...**		
Camber (using 1.5 * D.L. Defl) ...			Deflection	0.000 in	0.000 in
@ Center	0.295 in		...Length/Defl	0.0	0.0
@ Left	0.000 in				
@ Right	0.000 in				

Stress Calcs

Bending Analysis

Ck	31.887	Le	0.000 ft	Sxx	31.641 in3	Area	16.875 in2
Cf	1.000	Rb	0.000	Cl	550.547		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	2.48 k-ft	24.98 in3	1,190.25 psi
@ Left Support	0.00 k-ft	0.00 in3	1,190.25 psi
@ Right Support	0.00 k-ft	0.00 in3	1,190.25 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	0.74 k	0.74 k
Area Required	3.575 in2	3.575 in2
Fv: Allowable	207.00 psi	207.00 psi

Bearing @ Supports

Max. Left Reaction	0.55 k	Bearing Length Req'd	0.587 in
Max. Right Reaction	0.55 k	Bearing Length Req'd	0.587 in

5b (2009 Revision)

Rev: 580004

General Timber Beam

Description 2. DOUBLE JOIST

General Information

Section Name	**2-2x12**	Center Span	5.00 ft	.....Lu	0.00 ft
Beam Width	3.000 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth	11.250 in	Right Cantilever	ft	.....Lu	0.00 ft
Member Type	Sawn	Douglas Fir - Larch, No.2			
Bm Wt. Added to Loads		Fb Base Allow	900.0 psi		
Load Dur. Factor	1.150	Fv Allow	180.0 psi		
Beam End Fixity	Pin-Pin	Fc Allow	625.0 psi		
Wood Density	32.500 pcf	E	1,600.0 ksi		

Full Length Uniform Loads

Center	DL	96.00 #/ft	LL	320.00 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Summary

Beam Design OK

Span= 5.00ft, Beam Width = 3.000in x Depth = 11.25in, Ends are Pin-Pin

Max Stress Ratio	0.243 : 1			
Maximum Moment	1.3 k-ft	Maximum Shear * 1.5		1.0 k
Allowable	5.5 k-ft	Allowable		7.0 k

Max. Positive Moment	1.32 k-ft	at 2.500 ft	Shear:	@ Left	1.06 k
Max. Negative Moment	0.00 k-ft	at 0.000 ft		@ Right	1.06 k
Max @ Left Support	0.00 k-ft		Camber:	@ Left	0.000 in
Max @ Right Support	0.00 k-ft			@ Center	0.004 in
Max. M allow	5.46		Reactions...	@ Right	0.000 in
fb 251.03 psi	fv 29.75 psi		Left DL 0.26 k	Max	1.06 k
Fb 1,035.00 psi	Fv 207.00 psi		Right DL 0.26 k	Max	1.06 k

Deflections

Center Span...	Dead Load	Total Load	**Left Cantilever...**	Dead Load	Total Load
Deflection	-0.003 in	-0.010 in	Deflection	0.000 in	0.000 in
...Location	2.500 ft	2.500 ft	...Length/Defl	0.0	0.0
...Length/Defl	23,452.3	5,736.46	**Right Cantilever...**		
Camber (using 1.5 * D.L. Defl) ...			Deflection	0.000 in	0.000 in
@ Center	0.004 in		...Length/Defl	0.0	0.0
@ Left	0.000 in				
@ Right	0.000 in				

Stress Calcs

Bending Analysis

Ck	31.887	Le	0.000 ft	Sxx	63.281 in3	Area	33.750 in2
Cf	1.000	Rb	0.000	Cl	1059.043		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	1.32 k-ft	15.35 in3	1,035.00 psi
@ Left Support	0.00 k-ft	0.00 in3	1,035.00 psi
@ Right Support	0.00 k-ft	0.00 in3	1,035.00 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	1.00 k	1.00 k
Area Required	4.850 in2	4.850 in2
Fv: Allowable	207.00 psi	207.00 psi

Bearing @ Supports

Max. Left Reaction	1.06 k	Bearing Length Req'd	0.565 in
Max. Right Reaction	1.06 k	Bearing Length Req'd	0.565 in

Rev: 580004

General Timber Beam

Description 3. CANTILEVER BEAM SUPPORT #2

General Information

Section Name	4x12	Center Span	6.00 ft	.....Lu	0.00 ft
Beam Width	3.500 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth	11.250 in	Right Cantilever	3.00 ft	.....Lu	0.00 ft
Member Type	Sawn	Douglas Fir - Larch, No.2			
Bm Wt. Added to Loads		Fb Base Allow	900.0 psi		
Load Dur. Factor	1.150	Fv Allow	180.0 psi		
Beam End Fixity	Pin-Pin	Fc Allow	625.0 psi		
Wood Density	32.500 pcf	E	1,600.0 ksi		

Full Length Uniform Loads

Center	DL	188.00 #/ft	LL	360.00 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	188.00 #/ft	LL	360.00 #/ft

Point Loads

Dead Load	1,055.0 lbs	lbs	lbs	lbs	lbs	lbs	lbs
Live Load	lbs	lbs	lbs	lbs	lbs	lbs	lbs
...distance	9.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft

Summary

Beam Design OK

Span= 6.00ft, Right Cant= 3.00ft, Beam Width = 3.500in x Depth = 11.25in, Ends are Pin-Pin

Max Stress Ratio	0.810 : 1				
Maximum Moment	-5.7 k-ft	Maximum Shear * 1.5		3.3 k	
Allowable	7.0 k-ft	Allowable		8.2 k	
Max. Positive Moment	0.89 k-ft at 1.796 ft	Shear:	@ Left	1.00 k	
Max. Negative Moment	-4.05 k-ft at 6.000 ft		@ Right	2.71 k	
Max @ Left Support	0.00 k-ft	Camber:	@ Left	0.000 in	
Max @ Right Support	-5.67 k-ft		@ Center	0.024 in	
Max. M allow	7.00		@ Right	0.117 in	

				Reactions...			
fb	921.76 psi	fv	83.90 psi	Left DL	-0.08 k	Max	1.00 k
Fb	1,138.50 psi	Fv	207.00 psi	Right DL	2.91 k	Max	5.34 k

Deflections

Center Span...	Dead Load	Total Load
Deflection	0.016 in	0.004 in
...Location	3.737 ft	4.922 ft
...Length/Defl	4,478.5	19,279.72
Camber (using 1.5 * D.L. Defl) ...		
@ Center	0.024 in	
@ Left	0.000 in	
@ Right	0.117 in	

Left Cantilever...	Dead Load	Total Load
Deflection	0.000 in	0.000 in
...Length/Defl	0.0	0.0
Right Cantilever...		
Deflection	-0.078 in	-0.112 in
...Length/Defl	923.6	640.6

Rev: 580004

General Timber Beam

Description 3. CANTILEVER BEAM SUPPORT #2

Stress Calcs

Bending Analysis

Ck	31.887	Le	0.000 ft	Sxx	73.828 in3	Area	39.375 in2
Cf	1.100	Rb	0.000	Cl	995.495		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	4.05 k-ft	42.70 in3	1,138.50 psi
@ Left Support	0.00 k-ft	0.00 in3	1,138.50 psi
@ Right Support	5.67 k-ft	59.77 in3	1,138.50 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	0.81 k	3.30 k
Area Required	3.902 in2	15.959 in2
Fv: Allowable	207.00 psi	207.00 psi

Bearing @ Supports

Max. Left Reaction	1.00 k	Bearing Length Req'd	0.455 in
Max. Right Reaction	5.34 k	Bearing Length Req'd	2.442 in

Rev: 580004

General Timber Beam

Description 4. BEAM SUPPORT #2

General Information

Section Name	**3.125x10.5**	Center Span	17.00 ft	.....Lu	0.00 ft
Beam Width	3.125 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth	10.500 in	Right Cantilever	ft	.....Lu	0.00 ft
Member Type	GluLam	Douglas Fir, 24F - V4			
Bm Wt. Added to Loads		Fb Base Allow	2,400.0 psi		
Load Dur. Factor	1.150	Fv Allow	240.0 psi		
Beam End Fixity	Pin-Pin	Fc Allow	650.0 psi		
Wood Density	32.500 pcf	E	1,800.0 ksi		

Full Length Uniform Loads

Center	DL	96.00 #/ft	LL	52.00 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Point Loads

Dead Load	1,055.0 lbs	lbs	lbs	lbs	lbs	lbs	lbs
Live Load	lbs	lbs	lbs	lbs	lbs	lbs	lbs
...distance	4.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft

Summary

Beam Design OK

Span= 17.00ft, Beam Width = 3.125in x Depth = 10.5in, Ends are Pin-Pin

Max Stress Ratio	0.600 : 1					
Maximum Moment	7.9 k-ft		Maximum Shear * 1.5			3.0 k
Allowable	13.2 k-ft		Allowable			9.1 k
Max. Positive Moment	7.92 k-ft	at 6.936 ft	Shear:		@ Left	2.13 k
Max. Negative Moment	0.00 k-ft	at 17.000 ft			@ Right	1.57 k
Max @ Left Support	0.00 k-ft		Camber:		@ Left	0.000 in
Max @ Right Support	0.00 k-ft				@ Center	0.877 in
Max. M allow	13.21				@ Right	0.000 in
			Reactions...			
fb 1,655.58 psi	fv	91.47 psi	Left DL	1.69 k	Max	2.13 k
Fb 2,760.00 psi	Fv	276.00 psi	Right DL	1.13 k	Max	1.57 k

Deflections

Center Span...	Dead Load	Total Load	**Left Cantilever...**	Dead Load	Total Load
Deflection	-0.584 in	-0.764 in	Deflection	0.000 in	0.000 in
...Location	8.092 ft	8.228 ft	...Length/Defl	0.0	0.0
...Length/Defl	349.1	266.99	**Right Cantilever...**		
Camber (using 1.5 * D.L. Defl) ...			Deflection	0.000 in	0.000 in
@ Center	0.877 in		...Length/Defl	0.0	0.0
@ Left	0.000 in				
@ Right	0.000 in				

Rev: 580004

General Timber Beam

Description 4. BEAM SUPPORT #2

Stress Calcs

Bending Analysis

Ck	20.711	Le	0.000 ft	Sxx	57.422 in3	Area	32.813 in2
Cv	1.000	Rb	0.000	Cl	2127.712		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	7.92 k-ft	34.44 in3	2,760.00 psi
@ Left Support	0.00 k-ft	0.00 in3	2,760.00 psi
@ Right Support	0.00 k-ft	0.00 in3	2,760.00 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	3.00 k	2.16 k
Area Required	10.874 in2	7.839 in2
Fv: Allowable	276.00 psi	276.00 psi

Bearing @ Supports

Max. Left Reaction	2.13 k	Bearing Length Req'd	1.047 in
Max. Right Reaction	1.57 k	Bearing Length Req'd	0.773 in

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General Timber Beam

Description 5. FLOOR JOIST

General Information

Section Name **2x12**		Center Span	17.00 ft	.....Lu	0.00 ft
Beam Width	1.500 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth	11.250 in	Right Cantilever	ft	.....Lu	0.00 ft
Member Type	Sawn	Hem Fir, No.2			
Bm Wt. Added to Loads		Fb Base Allow	850.0 psi		
Load Dur. Factor	1.150	Fv Allow	150.0 psi		
Beam End Fixity	Pin-Pin	Fc Allow	405.0 psi		**Repetitive Member**
Wood Density	10.000 pcf	E	1,300.0 ksi		

Full Length Uniform Loads

Center	DL	16.00 #/ft	LL	52.00 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Summary

Beam Design OK

Span= 17.00ft, Beam Width = 1.500in x Depth = 11.25in, Ends are Pin-Pin

Max Stress Ratio 0.843 : 1

Maximum Moment	2.5 k-ft		Maximum Shear * 1.5		0.8 k	
Allowable	3.0 k-ft		Allowable		2.9 k	
Max. Positive Moment	2.50 k-ft	at 8.500 ft	Shear:	@ Left	0.59 k	
Max. Negative Moment	0.00 k-ft	at 0.000 ft		@ Right	0.59 k	
Max @ Left Support	0.00 k-ft		Camber:	@ Left	0.000 in	
Max @ Right Support	0.00 k-ft			@ Center	0.209 in	
Max. M allow	2.96			@ Right	0.000 in	

				Reactions...				
fb	947.71 psi	fv	46.83 psi	Left DL	0.15 k	Max	0.59 k	
Fb	1,124.13 psi	Fv	172.50 psi	Right DL	0.15 k	Max	0.59 k	

Deflections

Center Span...	Dead Load	Total Load	**Left Cantilever...**	Dead Load	Total Load
Deflection	-0.139 in	-0.562 in	Deflection	0.000 in	0.000 in
...Location	8.500 ft	8.500 ft	...Length/Defl	0.0	0.0
...Length/Defl	1,462.7	363.12	**Right Cantilever...**		
Camber (using 1.5 * D.L. Defl) ...			Deflection	0.000 in	0.000 in
@ Center	0.209 in		...Length/Defl	0.0	0.0
@ Left	0.000 in				
@ Right	0.000 in				

Stress Calcs

Bending Analysis

Ck	29.576	Le	0.000 ft	Sxx	31.641 in3	Area	16.875 in2
Cf	1.000	Rb	0.000	Cl	587.961		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	2.50 k-ft	26.67 in3	1,124.13 psi
@ Left Support	0.00 k-ft	0.00 in3	1,124.13 psi
@ Right Support	0.00 k-ft	0.00 in3	1,124.13 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	0.79 k	0.79 k
Area Required	4.581 in2	4.581 in2
Fv: Allowable	172.50 psi	172.50 psi

Bearing @ Supports

Max. Left Reaction	0.59 k	Bearing Length Req'd	0.968 in
Max. Right Reaction	0.59 k	Bearing Length Req'd	0.968 in

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General Timber Beam

Description 6.header support #4

General Information

Section Name	**6x10**	Center Span	6.00 ft	.....Lu	0.00 ft
Beam Width	5.500 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth	9.500 in	Right Cantilever	ft	.....Lu	0.00 ft
Member Type	Sawn	Douglas Fir - Larch, No.2			
Bm Wt. Added to Loads		Fb Base Allow	900.0 psi		
Load Dur. Factor	1.150	Fv Allow	180.0 psi		
Beam End Fixity	Pin-Pin	Fc Allow	625.0 psi		
Wood Density	32.500 pcf	E	1,600.0 ksi		

Full Length Uniform Loads

Center	DL	341.00 #/ft	LL	595.00 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Point Loads

Dead Load	1,569.0 lbs	lbs	lbs	lbs	lbs	lbs	lbs
Live Load	lbs	lbs	lbs	lbs	lbs	lbs	lbs
...distance	3.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft

Summary

Beam Design OK

Span= 6.00ft, Beam Width = 5.500in x Depth = 9.5in, Ends are Pin-Pin

Max Stress Ratio 0.928 : 1

Maximum Moment	6.6 k-ft		Maximum Shear * 1.5		4.3 k
Allowable	7.1 k-ft		Allowable		10.8 k
Max. Positive Moment	6.62 k-ft	at 3.000 ft	Shear:	@ Left	3.63 k
Max. Negative Moment	0.00 k-ft	at 0.000 ft		@ Right	3.63 k
Max @ Left Support	0.00 k-ft		Camber:	@ Left	0.000 in
Max @ Right Support	0.00 k-ft			@ Center	0.054 in
Max. M allow	7.14			@ Right	0.000 in

				Reactions...			
fb	960.03 psi	fv	83.25 psi	Left DL	1.84 k	Max	3.63 k
Fb	1,035.00 psi	Fv	207.00 psi	Right DL	1.84 k	Max	3.63 k

Deflections

Center Span...	Dead Load	Total Load
Deflection	-0.036 in	-0.063 in
...Location	3.000 ft	3.000 ft
...Length/Defl	2,013.1	1,136.36
Camber (using 1.5 * D.L. Defl) ...		
@ Center	0.054 in	
@ Left	0.000 in	
@ Right	0.000 in	

Left Cantilever...	Dead Load	Total Load
Deflection	0.000 in	0.000 in
...Length/Defl	0.0	0.0
Right Cantilever...		
Deflection	0.000 in	0.000 in
...Length/Defl	0.0	0.0

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General Timber Beam

Description 6.header support #4

Stress Calcs

Bending Analysis

Ck	31.887	Le	0.000 ft	Sxx	82.729 in3	Area	52.250 in2
Cf	1.000	Rb	0.000	Cl	3627.876		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	6.62 k-ft	76.74 in3	1,035.00 psi
@ Left Support	0.00 k-ft	0.00 in3	1,035.00 psi
@ Right Support	0.00 k-ft	0.00 in3	1,035.00 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	4.35 k	4.35 k
Area Required	21.014 in2	21.014 in2
Fv: Allowable	207.00 psi	207.00 psi

Bearing @ Supports

Max. Left Reaction	3.63 k	Bearing Length Req'd	1.055 in
Max. Right Reaction	3.63 k	Bearing Length Req'd	1.055 in

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General Timber Beam

Description 6a.header support #4

General Information

Section Name	**4x10**	Center Span	4.00 ft	.....Lu	0.00 ft
Beam Width	3.500 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth	9.250 in	Right Cantilever	ft	.....Lu	0.00 ft
Member Type	Sawn	Douglas Fir - Larch, No.2			
Bm Wt. Added to Loads		Fb Base Allow	900.0 psi		
Load Dur. Factor	1.150	Fv Allow	180.0 psi		
Beam End Fixity	Pin-Pin	Fc Allow	625.0 psi		
Wood Density	32.500 pcf	E	1,600.0 ksi		

Full Length Uniform Loads

Center	DL	341.00 #/ft	LL	595.00 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Point Loads

Dead Load	2,159.0 lbs	lbs	lbs	lbs	lbs	lbs	lbs
Live Load	lbs	lbs	lbs	lbs	lbs	lbs	lbs
...distance	1.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft	0.000 ft

Summary

Beam Design OK

Span= 4.00ft, Beam Width = 3.500in x Depth = 9.25in, Ends are Pin-Pin

Max Stress Ratio	0.623 : 1					
Maximum Moment	3.1 k-ft		Maximum Shear * 1.5			4.2 k
Allowable	5.2 k-ft		Allowable			6.7 k
Max. Positive Moment	3.12 k-ft	at 1.424 ft	Shear:		@ Left	3.51 k
Max. Negative Moment	0.00 k-ft	at 4.000 ft			@ Right	2.43 k
Max @ Left Support	0.00 k-ft		Camber:		@ Left	0.000 in
Max @ Right Support	0.00 k-ft				@ Center	0.022 in
Max. M allow	5.17		Reactions...		@ Right	0.000 in
fb 750.25 psi	fv	128.87 psi	Left DL	2.32 k	Max	3.51 k
Fb 1,242.00 psi	Fv	207.00 psi	Right DL	1.24 k	Max	2.43 k

Deflections

Center Span...	Dead Load	Total Load	**Left Cantilever...**	Dead Load	Total Load
Deflection	-0.015 in	-0.024 in	Deflection	0.000 in	0.000 in
...Location	1.856 ft	1.904 ft	...Length/Defl	0.0	0.0
...Length/Defl	3,246.5	1,997.79	**Right Cantilever...**		
Camber (using 1.5 * D.L. Defl) ...			Deflection	0.000 in	0.000 in
@ Center	0.022 in		...Length/Defl	0.0	0.0
@ Left	0.000 in				
@ Right	0.000 in				

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General Timber Beam

Description 6a.header support #4

Stress Calcs

Bending Analysis

Ck	31.887	Le	0.000 ft	Sxx	49.911 in3	Area	32.375 in2
Cf	1.200	Rb	0.000	Cl	3505.863		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	3.12 k-ft	30.15 in3	1,242.00 psi
@ Left Support	0.00 k-ft	0.00 in3	1,242.00 psi
@ Right Support	0.00 k-ft	0.00 in3	1,242.00 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	4.17 k	2.55 k
Area Required	20.155 in2	12.333 in2
Fv: Allowable	207.00 psi	207.00 psi

Bearing @ Supports

Max. Left Reaction	3.51 k	Bearing Length Req'd	1.603 in
Max. Right Reaction	2.43 k	Bearing Length Req'd	1.109 in

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General Timber Beam

Description 7. foundation beam with load above

General Information

Section Name	**4x10**		Center Span	4.00 ft	.....Lu	0.00 ft
Beam Width		3.500 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth		9.250 in	Right Cantilever	ft	.....Lu	0.00 ft
Member Type		Sawn	Douglas Fir - Larch, No.2			
Bm Wt. Added to Loads			Fb Base Allow	900.0 psi		
Load Dur. Factor		1.150	Fv Allow	180.0 psi		
Beam End Fixity		Pin-Pin	Fc Allow	625.0 psi		
Wood Density		32.500 pcf	E	1,600.0 ksi		

Full Length Uniform Loads

Center	DL	296.00 #/ft	LL	720.00 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Summary

Beam Design OK

Span= 4.00ft, Beam Width = 3.500in x Depth = 9.25in, Ends are Pin-Pin

Max Stress Ratio	0.396 : 1				
Maximum Moment	2.0 k-ft		Maximum Shear * 1.5		1.9 k
Allowable	5.2 k-ft		Allowable		6.7 k
Max. Positive Moment	2.05 k-ft	at 2.000 ft	Shear:	@ Left	2.05 k
Max. Negative Moment	0.00 k-ft	at 0.000 ft		@ Right	2.05 k
Max @ Left Support	0.00 k-ft		Camber:	@ Left	0.000 in
Max @ Right Support	0.00 k-ft			@ Center	0.007 in
Max. M allow	5.17		Reactions...	@ Right	0.000 in
fb 492.06 psi	fv	58.41 psi	Left DL 0.61 k	Max	2.05 k
Fb 1,242.00 psi	Fv	207.00 psi	Right DL 0.61 k	Max	2.05 k

Deflections

Center Span...	Dead Load	Total Load	**Left Cantilever...**	Dead Load	Total Load
Deflection	-0.005 in	-0.016 in	Deflection	0.000 in	0.000 in
...Location	2.000 ft	2.000 ft	...Length/Defl	0.0	0.0
...Length/Defl	10,148.0	3,007.85	**Right Cantilever...**		
Camber (using 1.5 * D.L. Defl) ...			Deflection	0.000 in	0.000 in
@ Center	0.007 in		...Length/Defl	0.0	0.0
@ Left	0.000 in				
@ Right	0.000 in				

Stress Calcs

Bending Analysis

Ck	31.887	Le	0.000 ft	Sxx	49.911 in3	Area	32.375 in2
Cf	1.200	Rb	0.000	Cl	2046.613		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	2.05 k-ft	19.77 in3	1,242.00 psi
@ Left Support	0.00 k-ft	0.00 in3	1,242.00 psi
@ Right Support	0.00 k-ft	0.00 in3	1,242.00 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	1.89 k	1.89 k
Area Required	9.136 in2	9.136 in2
Fv: Allowable	207.00 psi	207.00 psi

Bearing @ Supports

Max. Left Reaction	2.05 k	Bearing Length Req'd	0.936 in
Max. Right Reaction	2.05 k	Bearing Length Req'd	0.936 in

Rev: 580004

General Timber Beam

Description 8. foundation beam without load above

General Information

Section Name	**4x10**		Center Span	6.00 ft	.....Lu	0.00 ft
Beam Width		3.500 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth		9.250 in	Right Cantilever	ft	.....Lu	0.00 ft
Member Type		Sawn	Douglas Fir - Larch, No.2			
Bm Wt. Added to Loads			Fb Base Allow	900.0 psi		
Load Dur. Factor		1.150	Fv Allow	180.0 psi		
Beam End Fixity		Pin-Pin	Fc Allow	625.0 psi		
Wood Density		32.500 pcf	E	1,600.0 ksi		

Full Length Uniform Loads

Center	DL	176.00 #/ft	LL	320.00 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Summary

Beam Design OK

Span= 6.00ft, Beam Width = 3.500in x Depth = 9.25in, Ends are Pin-Pin

Max Stress Ratio 0.438 : 1

Maximum Moment	2.3 k-ft	Maximum Shear * 1.5	1.7 k
Allowable	5.2 k-ft	Allowable	6.7 k

Max. Positive Moment	2.26 k-ft	at	3.000 ft	Shear:	@ Left	1.51 k
Max. Negative Moment	0.00 k-ft	at	0.000 ft		@ Right	1.51 k
Max @ Left Support	0.00 k-ft			Camber:	@ Left	0.000 in
Max @ Right Support	0.00 k-ft				@ Center	0.022 in
Max. M allow	5.17			Reactions...	@ Right	0.000 in

fb	544.54 psi	fv	52.05 psi	Left DL	0.55 k	Max	1.51 k
Fb	1,242.00 psi	Fv	207.00 psi	Right DL	0.55 k	Max	1.51 k

Deflections

Center Span...	Dead Load	Total Load	**Left Cantilever...**	Dead Load	Total Load
Deflection	-0.014 in	-0.040 in	Deflection	0.000 in	0.000 in
...Location	3.000 ft	3.000 ft	...Length/Defl	0.0	0.0
...Length/Defl	4,975.2	1,811.99	**Right Cantilever...**		
Camber (using 1.5 * D.L. Defl) ...			Deflection	0.000 in	0.000 in
@ Center	0.022 in		...Length/Defl	0.0	0.0
@ Left	0.000 in				
@ Right	0.000 in				

Stress Calcs

Bending Analysis

Ck	31.887	Le	0.000 ft	Sxx	49.911 in3	Area	32.375 in2
Cf	1.200	Rb	0.000	Cl	1509.920		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	2.26 k-ft	21.88 in3	1,242.00 psi
@ Left Support	0.00 k-ft	0.00 in3	1,242.00 psi
@ Right Support	0.00 k-ft	0.00 in3	1,242.00 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	1.69 k	1.69 k
Area Required	8.140 in2	8.140 in2
Fv: Allowable	207.00 psi	207.00 psi

Bearing @ Supports

Max. Left Reaction	1.51 k	Bearing Length Req'd	0.690 in
Max. Right Reaction	1.51 k	Bearing Length Req'd	0.690 in

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LATERAL ANALYSIS

BASED ON 2015 INTERNATIONAL BUILDING CODE AS MODIFIED BY LOCAL JURISDICTION.

Lateral Forces will be distributed along lines of Force/Resistance. Lines of Force/Resistance will be investigated for both wind and seismic lateral loads.(Vertical component of earthquake ground motion neglected for Lateral Force Resisting System w/ Allowable Stress Design)

SEISMIC DESIGN

SEISMIC DESIGN BASED ASCE 7-10 CHAPTER 12 SECTION 12.14
SIMPLIFIED ALTERNATIVE STRUCTURAL DESIGN CRITERIA FOR SIMPLE BEARING WALL SINGLE FAMILY DWELLING, LIGHT FRAME CONSTRUCTION LESS THAN THREE STORIES IN HEIGHT (EXCLUDING BASEMENT).

Seismic Design Data:

--Soils Site Class D (Assumed)
--Seismic Design Category D_1/D_2

$I_E := 1.0$	For Seismic Use Group I occupancy (ASCE 7-10 Table 11.5-1)
$R := 6.5$	Light Framed Walls w/ Wood Shear Panels (ASCE 7-10 Table 12.14-1)
$S_s := 1.59$	Mapped Maximum Considered Earthquake Spectral Response Acceleration Short-Period
$S_1 := 0.552$	Mapped Maximum Considered Earthquake Spectral Response Acceleration 1-Second Period
$F_a := 1.00$	Site Coefficient based on Site Class & S_s (ASCE 7-10 Table 11.4-1)
$F_v := 1.5$	Site Coefficient based on Site Class & S_1 (ASCE 7-10 Table 11.4-2)
W, w_x	Seismic Weight of Overall Structure, Seismic Weight of Structure above Level x (LB.)

ASCE 7-05 Eq. 11.4-1 $S_{ms} := S_s \cdot F_a$ $S_{ms} = 1.59$

ASCE 7-05 Eq. 11.4-2 $S_{m1} := S_1 \cdot F_v$ $S_{m1} = 0.83$

ASCE 7-05 Eq. 11.4-3 $S_{DS} := \frac{2}{3} \cdot S_{ms}$ $S_{DS} = 1.06$

ASCE 7-05 Eq. 11.4-4 $S_{D1} := \frac{2}{3} \cdot S_{m1}$ $S_{D1} = 0.55$

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SEISMIC WEIGHT OF STRUCTURE CHECK FOR EACH LEVEL:

Roof Slope Adjustment Factor: $S := \dfrac{1}{\cos\left(\operatorname{atan}\left(\dfrac{0.5}{12}\right)\right)}$ $\quad S = 1$

ROOF PLAN AREA EACH LEVEL:

$R_2 := 574\text{ft}^2 \cdot S$

(Upper Roof Area)

$R_1 := 0\text{ft}^2 \cdot S$

(Lower Roof Area)

FLOOR AREA EACH LEVEL:

$A_2 := 445\text{ft}^2$

(2nd Floor)

PLAN PERIMETER FOR EACH LEVEL:

$P_2 := 2(18.5\text{ft}) + 2(29\text{ft})$

(2nd Floor)

$P_1 := 2(18.5\text{ft}) + 2(29\text{ft})$

(1st Floor)

Weight of Structure at Each Level:

Story Weight at Upper level:

$w_2 := 15 \cdot \text{psf} \cdot (R_2) + 10 \cdot \text{psf} \cdot 8 \cdot \text{ft} \cdot (P_2)$ (weight of upper roof, and wall)

Story Weight at Main level:

$w_1 := 15 \cdot \text{psf} \cdot (R_1) + 12 \cdot \text{psf} \cdot (A_2) + 10 \cdot \text{psf} \cdot (8 \cdot \text{ft} \cdot P_1)$ (weight of lower roof, floor, & wall)

Shear at each Level:

F=1.0 for one-story building
F=1.1 for two-story building
F=1.2 for three-story building

$F := 1.1$

$V_{2E} := \dfrac{(F \cdot S_{DS} \cdot w_2)}{R}$ $\quad V_{2E} = 2909.16\,\text{lb}$ Story Shear at upper Floor

$V_{1E} := \dfrac{[F \cdot S_{DS} \cdot (w_1)]}{R}$ $\quad V_{1E} = 2321.24\,\text{lb}$ Story Shear at main Floor

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WIND DESIGN

USE ANALYTICAL PROCEDURE OF ASCE 7-10 SECTION 6.5
ENCLOSED LOW-RISE BUILDING LESS THAN 60 FEET IN HEIGHT

WIND EXPOSURE = C   90 mph WIND SPEED

Equation 6-18 $p=q_h[(GC_{pf}) - (GC_{pi})]$

Mean Roof Height $h := 23.5 \cdot ft$

$q_h = 0.00256K_zK_{zt}K_dv^2I$ = Velocity Pressure Evaluated at mean Roof Height h (Equation 6-15)

GC_{pf} = External Pressure Coefficients per Figure 6-10

GC_{pi} = Internal Pressure Coefficients per Figure 6-5

$K_{15} := 0.85$ Velocity Pressure Exposure Coefficients at z < 15ft (Table 6-3)

$K_{20} := 0.90$ Velocity Pressure Exposure Coefficients at 15ft < z < 20ft (Table 6-3)

$K_{25} := 0.94$ Velocity Pressure Exposure Coefficients at 20ft < z < 25ft (Table 6-3)

$K_{30} := 0.98$ Velocity Pressure Exposure Coefficients at 25ft < z < 30ft (Table 6-3)

$K_{40} := 1.04$ Velocity Pressure Exposure Coefficients at 30ft < z < 40ft (Table 6-3)

$K_d := 0.85$ Wind Directionality Factor (Table 6-4).

$v := 110$ Wind Speed per Hours (Figure 6-1).

$I := 1.0$ Important Factor (Table 6-1).

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Velocity Pressure (q_z) Evaluated at Height z (Equation 6-15)

$q_{15} := 0.00256 \cdot K_{15} \cdot K_d \cdot v^2 \cdot I$ $\quad q_{15} = 22.38$

$q_{20} := 0.00256 \cdot K_{20} \cdot K_d \cdot v^2 \cdot I$ $\quad q_{20} = 23.7$

$q_{25} := 0.00256 \cdot K_{25} \cdot K_d \cdot v^2 \cdot I$ $\quad q_{25} = 24.75$

$q_{30} := 0.00256 \cdot K_{30} \cdot K_d \cdot v^2 \cdot I$ $\quad q_{30} = 25.8$

$q_{40} := 0.00256 \cdot K_{40} \cdot K_d \cdot v^2 \cdot I$ $\quad q_{40} = 27.38$

Internal Pressure Coefficients (Figure 6-5)

$GC_{pi} := .18$ +/- The Internal Pressures on Windward and Leeward Walls will offset each other for the lateral design of the overall building and will therefore be ignored for this application.

External Pressure Coefficients w/ Roof Pitch = 7/12 (30 degrees) to 12/12 (45 degrees)
(Conservatively taken from Figure 6-10)

$GC_{pf1} := .56$ $\quad GC_{pf1E} := .69$

$GC_{pf2} := .21$ $\quad GC_{pf2E} := .27$

$GC_{pf3} := -.43$ $\quad GC_{pf3E} := -.53$

$GC_{pf4} := -.37$ $\quad GC_{pf4E} := -.48$

$P_{15.1} := q_{15} \cdot (GC_{pf1}) \cdot psf$ $P_{15.1} = 12.53\,lb\,ft^{-2}$

$P_{15.2} := q_{15} \cdot (GC_{pf2}) \cdot psf$ $P_{15.2} = 4.7\,lb\,ft^{-2}$

$P_{15.3} := q_{15} \cdot (GC_{pf3}) \cdot psf$ $P_{15.3} = -9.62\,lb\,ft^{-2}$

$P_{15.4} := q_{15} \cdot (GC_{pf4}) \cdot psf$ $P_{15.4} = -8.28\,lb\,ft^{-2}$

$P_{15.1E} := q_{15} \cdot (GC_{pf1E}) \cdot psf$ $P_{15.1E} = 15.44\,lb\,ft^{-2}$

$P_{15.2E} := q_{15} \cdot (GC_{pf2E}) \cdot psf$ $P_{15.2E} = 6.04\,lb\,ft^{-2}$

$P_{15.3E} := q_{15} \cdot (GC_{pf3E}) \cdot psf$ $P_{15.3E} = -11.86\,lb\,ft^{-2}$

$P_{15.4E} := q_{15} \cdot (GC_{pf4E}) \cdot psf$ $P_{15.4E} = -10.74\,lb\,ft^{-2}$

$P_{20.1} := q_{20} \cdot (GC_{pf1}) \cdot psf$ $P_{20.1} = 13.27\,lb\,ft^{-2}$

$P_{20.2} := q_{20} \cdot (GC_{pf2}) \cdot psf$ $P_{20.2} = 4.98\,lb\,ft^{-2}$

$P_{20.3} := q_{20} \cdot (GC_{pf3}) \cdot psf$ $P_{20.3} = -10.19\,lb\,ft^{-2}$

$P_{20.4} := q_{20} \cdot (GC_{pf4}) \cdot psf$ $P_{20.4} = -8.77\,lb\,ft^{-2}$

$P_{20.1E} := q_{20} \cdot (GC_{pf1E}) \cdot psf$ $P_{20.1E} = 16.35\,lb\,ft^{-2}$

$P_{20.2E} := q_{20} \cdot (GC_{pf2E}) \cdot psf$ $P_{20.2E} = 6.4\,lb\,ft^{-2}$

$P_{20.3E} := q_{20} \cdot (GC_{pf3E}) \cdot psf$ $P_{20.3E} = -12.56\,lb\,ft^{-2}$

$P_{20.4E} := q_{20} \cdot (GC_{pf4E}) \cdot psf$ $P_{20.4E} = -11.37\,lb\,ft^{-2}$

$P_{25.1} := q_{25} \cdot (GC_{pf1}) \cdot psf$ $P_{25.1} = 13.86\,lb\,ft^{-2}$

$P_{25.2} := q_{25} \cdot (GC_{pf2}) \cdot psf$ $P_{25.2} = 5.2\,lb\,ft^{-2}$

$P_{25.3} := q_{25} \cdot (GC_{pf3}) \cdot psf$ $P_{25.3} = -10.64\,lb\,ft^{-2}$

$P_{25.4} := q_{25} \cdot (GC_{pf4}) \cdot psf$ $P_{25.4} = -9.16\,lb\,ft^{-2}$

$P_{25.1E} := q_{25} \cdot (GC_{pf1E}) \cdot psf$ $P_{25.1E} = 17.08\,lb\,ft^{-2}$

$P_{25.2E} := q_{25} \cdot (GC_{pf2E}) \cdot psf$ $P_{25.2E} = 6.68\,lb\,ft^{-2}$

$P_{25.3E} := q_{25} \cdot (GC_{pf3E}) \cdot psf$ $P_{25.3E} = -13.12\,lb\,ft^{-2}$

$P_{25.4E} := q_{25} \cdot (GC_{pf4E}) \cdot psf$ $P_{25.4E} = -11.88\,lb\,ft^{-2}$

$P_{30.1} := q_{30} \cdot (GC_{pf1}) \cdot psf$ $P_{30.1} = 14.45\,lb\,ft^{-2}$

$P_{30.2} := q_{30} \cdot (GC_{pf2}) \cdot psf$ $P_{30.2} = 5.42\,lb\,ft^{-2}$

$P_{30.3} := q_{30} \cdot (GC_{pf3}) \cdot psf$ $P_{30.3} = -11.1\,lb\,ft^{-2}$

$P_{30.4} := q_{30} \cdot (GC_{pf4}) \cdot psf$ $P_{30.4} = -9.55\,lb\,ft^{-2}$

$P_{30.1E} := q_{30} \cdot (GC_{pf1E}) \cdot psf$ $P_{30.1E} = 17.8\,lb\,ft^{-2}$

$P_{30.2E} := q_{30} \cdot (GC_{pf2E}) \cdot psf$ $P_{30.2E} = 6.97\,lb\,ft^{-2}$

$P_{30.3E} := q_{30} \cdot (GC_{pf3E}) \cdot psf$ $P_{30.3E} = -13.68\,lb\,ft^{-2}$

$P_{30.4E} := q_{30} \cdot (GC_{pf4E}) \cdot psf$ $P_{30.4E} = -12.39\,lb\,ft^{-2}$

$P_{40.1} := q_{40} \cdot (GC_{pf1}) \cdot psf$ $P_{40.1} = 15.33\,lb\,ft^{-2}$

$P_{40.2} := q_{40} \cdot (GC_{pf2}) \cdot psf$ $P_{40.2} = 5.75\,lb\,ft^{-2}$

$P_{40.3} := q_{40} \cdot (GC_{pf3}) \cdot psf$ $P_{40.3} = -11.77\,lb\,ft^{-2}$

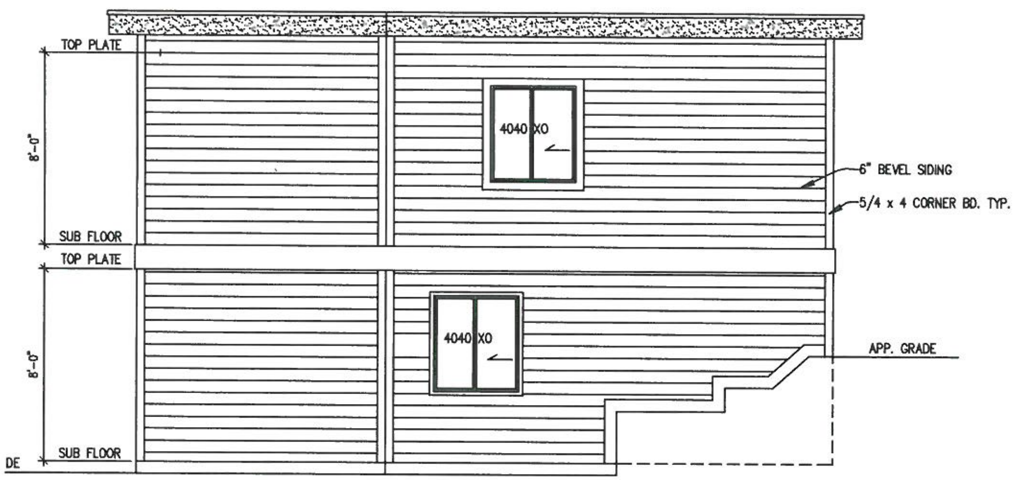
$P_{40.4} := q_{40} \cdot (GC_{pf4}) \cdot psf$ $P_{40.4} = -10.13\,lb\,ft^{-2}$

$P_{40.1E} := q_{40} \cdot (GC_{pf1E}) \cdot psf$ $P_{40.1E} = 18.89\,lb\,ft^{-2}$

$P_{40.2E} := q_{40} \cdot (GC_{pf2E}) \cdot psf$ $P_{40.2E} = 7.39\,lb\,ft^{-2}$

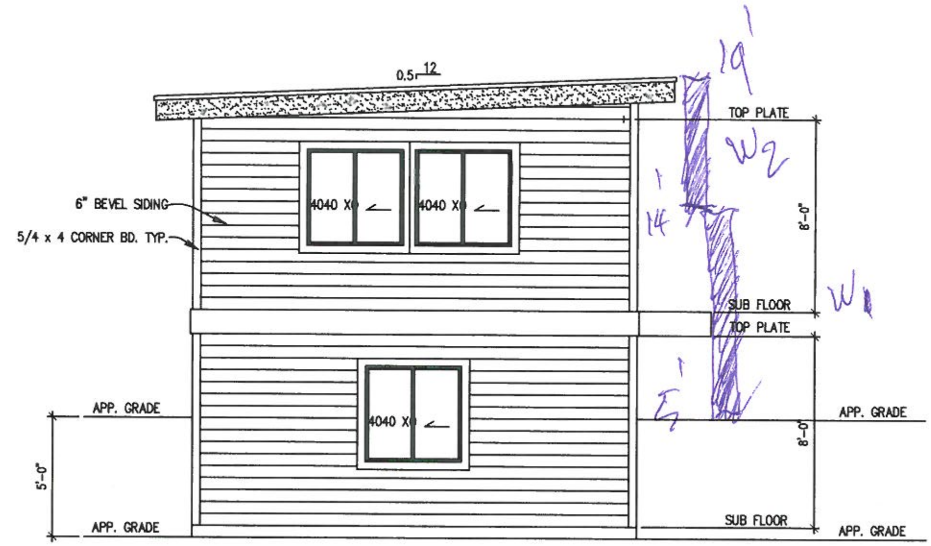
$P_{40.3E} := q_{40} \cdot (GC_{pf3E}) \cdot psf$ $P_{40.3E} = -14.51\,lb\,ft^{-2}$

$P_{40.4E} := q_{40} \cdot (GC_{pf4E}) \cdot psf$ $P_{40.4E} = -13.14\,lb\,ft^{-2}$



FRONT ELEVATION

SCALE : 1/4"= 1'-0"



RIGHT ELEVATION

Roof 574 SF.
upper Floor 445 SF.

SCALE : 1/4"= 1'-0"

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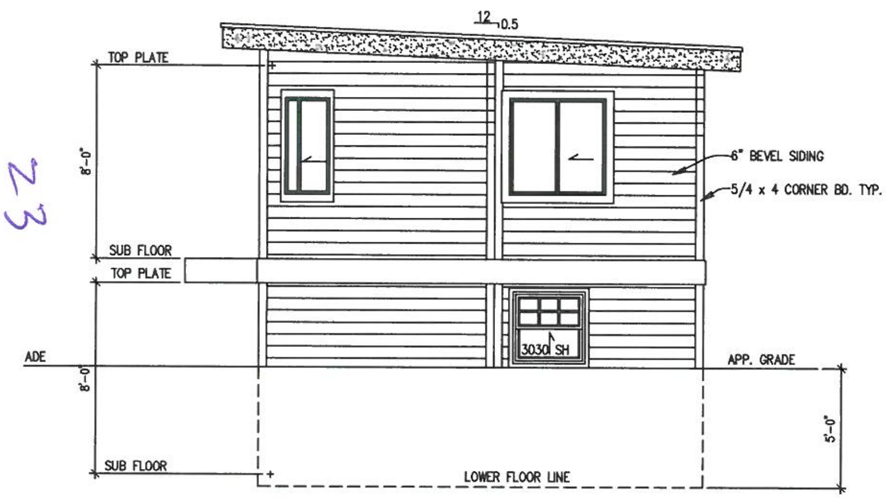
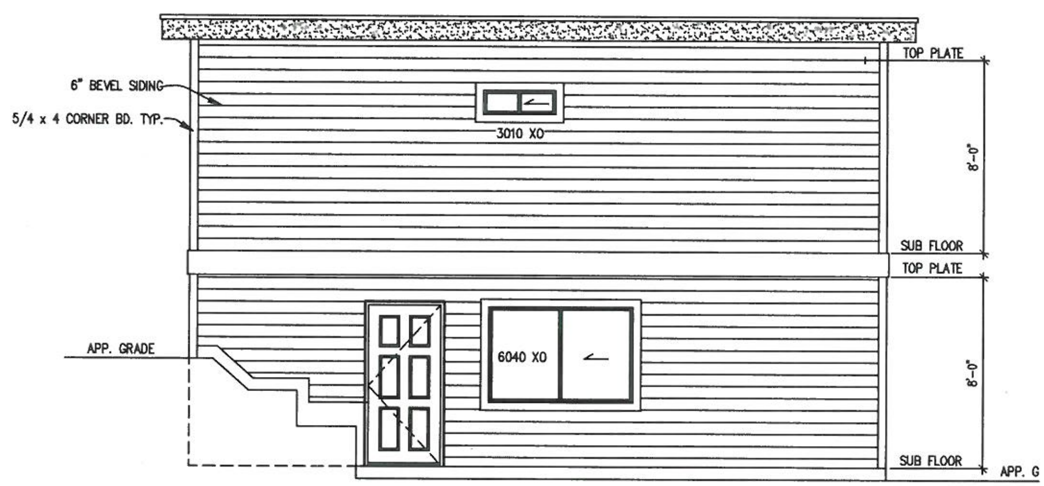


:FT ELEVATION

SCALE : 1/4"= 1'-0"



BACK ELEVATION

SCALE : 1/4"= 1'-0"

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WIND AT UPPER LEVEL------(All sides)----------

$$W_2 := (P_{20.1} - P_{20.4})\cdot 4ft + (P_{15.1} - P_{15.4})\cdot 1ft$$

END ZONE WIND AT UPPER LEVEL--------(All sides)----------

$$W_{2E} := (P_{20.1E} - P_{20.4E})\cdot 4ft + (P_{15.1E} - P_{15.4E})\cdot 1ft$$

WIND AT MAIN LEVEL------(all sides)----------

$$W_1 := (P_{15.1} - P_{15.4})\cdot 9ft$$

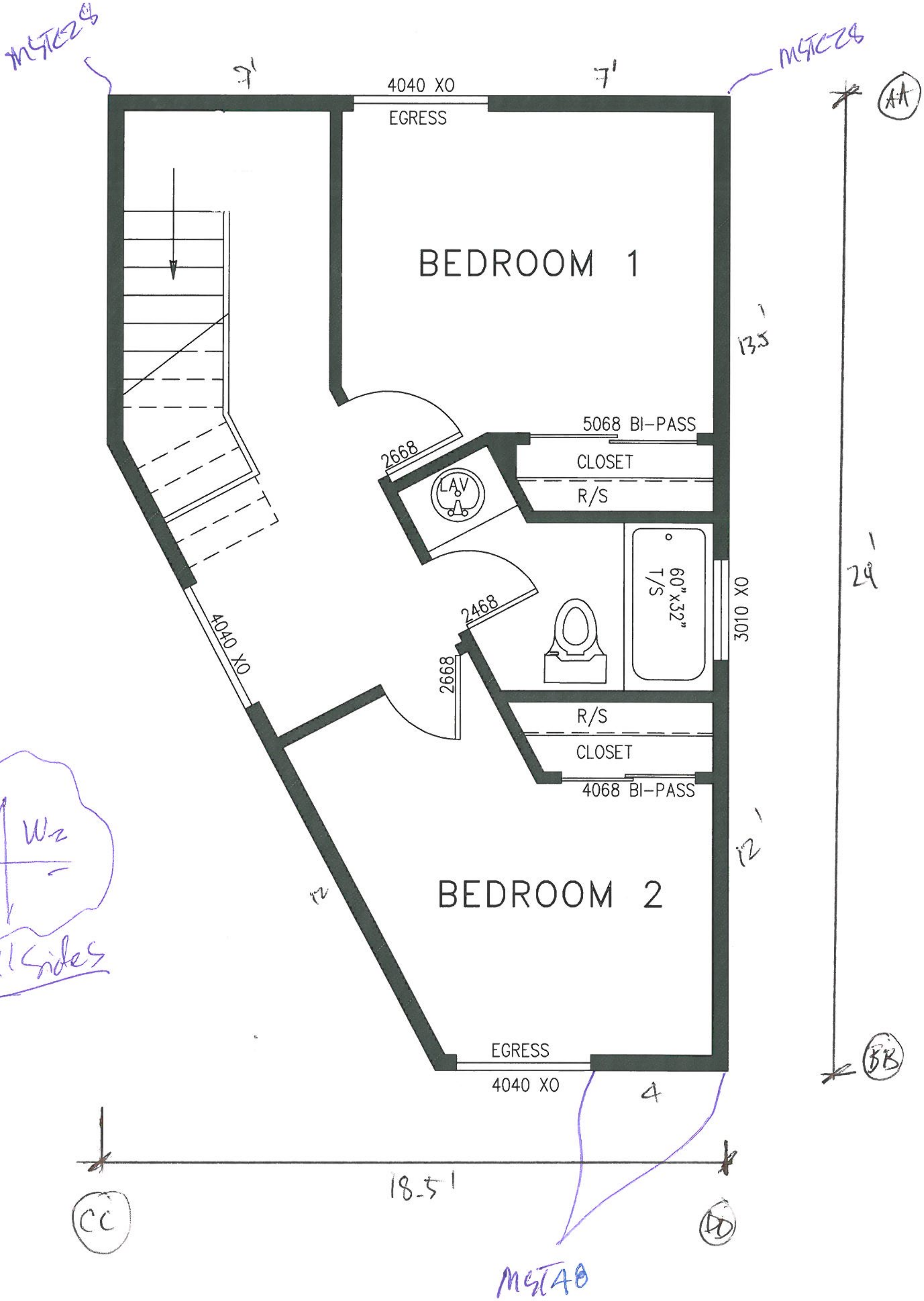
END ZONE WIND AT ROOF LEVEL--------(all sides)----------

$$W_{1E} := (P_{15.1E} - P_{15.4E})\cdot 9ft$$

But not less than 10 psf over the projected vertical plane.

$W_2 = 108.97\,lb\,ft^{-1}$ $\qquad$ $W_{2E} = 137.09\,lb\,ft^{-1}$

$W_1 = 187.32\,lb\,ft^{-1}$ $\qquad$ $W_{1E} = 235.66\,lb\,ft^{-1}$



UPPER FLOOR PLAN $\frac{1}{4}'' = 1'-0''$

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Shear wall line AA:

Step 1. Wind analysis on wall

Wind loads per foot: $W_2 = 108.97\,lb\,ft^{-1}$ $W_{2E} = 137.09\,lb\,ft^{-1}$

Distance between shear wall: $L1 := 29 \cdot ft$

Shear wall panels: $Laa := (7 + 7)ft$ $Laa = 14\,ft$

Percent full height sheathing: $\% := \left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100$ $\% = 100$ Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per IBC Table 2305.3.8.2

Wind Force: $vaa := \dfrac{\frac{(W_2 + W_{2E})}{2} \cdot \frac{L1}{2}}{Laa}$ $vaa = 127.42\,lb\,ft^{-1}$ $\frac{vaa}{C_o} = 127.42\,lb\,ft^{-1}$ Wind loads per foot

Step 2. Seismic analysis on wall

Seismic Weight per level: $V_{2E} = 2909.16\,lb$ Assumed $\rho := 1.0$ Bldg Width in direction of Load: $Lt := 29 \cdot ft$

Seismic Force: $E_{aa} := \dfrac{0.7\rho \cdot \frac{V_{2E}}{Lt} \cdot \frac{L1}{2}}{Laa}$ $E_{aa} = 72.73\,lb\,ft^{-1}$ $\frac{E_{aa}}{C_o} = 72.73\,lb\,ft^{-1}$ Seismic loads per foot

Step 3. Checking Overturning Moment & Holdown on Wall

Overturning Moment on Wall: Plate Height: $Pt := 8.0 \cdot ft$

$L_{aa} := 7 \cdot ft$ $OTM := vaa \cdot L_{aa} \cdot Pt$ $OTM = 7135.45\,lb\,ft$

Dead Load Resisting Overturning:

$W_R := 0.6(15 \cdot psf) \cdot 2 \cdot ft \cdot L_{aa} + 0.6 \cdot (10 \cdot psf) \cdot Pt \cdot 1 \cdot L_{aa}$

$DLRM := W_R \cdot \frac{L_{aa}}{2}$ $DLRM = 1617\,lb\,ft$

Holdown Force & Net Uplift:

$HDFaa := \dfrac{OTM - DLRM}{C_o \cdot L_{aa}}$ $HDFaa = 788.35\,lb$

Base Plate Nail Spacing (2003 NDS Table11N)
16d Common Nails & 1-1/2" Plate Hem-Fir

$Z_N := 122 \cdot lb$ $C_D := 1.33$ $Z'_N := Z_N \cdot C_D$ $Z'_N = 162.26\,lb$

$B_p := \frac{Z'_N}{vaa}$ $B_p = 1.27\,ft$ Per Nail

Anchor Bolt Spacing (2003 NDS Table 11E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 830 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1328\,lb$

$As := \frac{Z_B \cdot C_o}{vaa}$ $As = 10.42\,ft$ Per Bolt

Step 4. Shear Wall Summary:

Wind Force:	Seismic Force:	B.P. Nailing Spacing	A.B. Spacing	Holdown Force:	Holdown Types:
$\frac{vaa}{C_o} = 127.42\,lb\,ft^{-1}$	$\frac{E_{aa}}{C_o} = 72.73\,lb\,ft^{-1}$	$B_p = 1.27\,ft$	$As = 10.42\,ft$	$HDFaa = 788.35\,lb$	No Holdown
		16d @ 15" o.c.	**5/8" A.B. @ 60" o.c.**		

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

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Shear wall line BB:

Step 1. Wind analysis on wall

Wind loads per foot: $W_2 = 108.97\,lb\,ft^{-1}$ $W_{2E} = 137.09\,lb\,ft^{-1}$

Distance between shear wall: $L1 := 29\cdot ft$

Shear wall panels: $Lbb := (4)ft$ $Lbb = 4\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$ Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per IBC Table 2305.3.8.2

Wind Force: $vbb := \dfrac{\frac{(W_2 + W_{2E})}{2}\cdot\frac{L1}{2}}{Lbb}$ $vbb = 445.97\,lb\,ft^{-1}$ $\dfrac{vbb}{C_o} = 445.97\,lb\,ft^{-1}$ Wind loads per foot

Step 2. Seismic analysis on wall

Seismic Weight per level: $V_{2E} = 2909.16\,lb$ Assumed $\rho := 1.0$ Bldg Width in direction of Load: $Lt := 29\cdot ft$

Seismic Force: $E_{bb} := \dfrac{0.7\rho\cdot\frac{V_{2E}}{Lt}\cdot\frac{L1}{2}}{Lbb}$ $E_{bb} = 254.55\,lb\,ft^{-1}$ $\dfrac{E_{bb}}{C_o} = 254.55\,lb\,ft^{-1}$ Seismic loads per foot

Step 3. Checking Overturning Moment & Holdown on Wall

Overturning Moment on Wall: Plate Height: $Pt := 8.0\cdot ft$

$L_{bb} := 4\cdot ft$ $OTM := vbb\cdot L_{bb}\cdot Pt$ $OTM = 14270.91\,lb\,ft$

Dead Load Resisting Overturning:

$W_R := 0.6(15\cdot psf)\cdot 2\cdot ft\cdot L_{bb} + 0.6\cdot(10\cdot psf)\cdot Pt\cdot 1\cdot L_{bb}$

$DLRM := W_R\cdot\dfrac{L_{bb}}{2}$ $DLRM = 528\,lb\,ft$

Holdown Force & Net Uplift:

$HDFbb := \dfrac{OTM - DLRM}{C_o\cdot L_{bb}}$ $HDFbb = 3435.73\,lb$

Base Plate Nail Spacing (2003 NDS Table11N)
16d Common Nails & 1-1/2" Plate Hem-Fir

$Z_N := 122\cdot lb$ $C_D := 1.6$ $Z'_N := Z_N\cdot C_D$ $Z'_N = 195.2\,lb$

$B_p := \dfrac{Z'_N}{vbb}$ $B_p = 0.44\,ft$ Per Nail

Anchor Bolt Spacing (2003 NDS Table 11E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 830\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1328\,lb$

$As := \dfrac{Z_B\cdot C_o}{vbb}$ $As = 2.98\,ft$ Per Bolt

Step 4. Shear Wall Summary:

Wind Force:	Seismic Force:	B.P. Nailing Spacing	A.B. Spacing	Holdown Force:	Holdown Types:
$\frac{vbb}{C_o} = 445.97\,lb\,ft^{-1}$	$\frac{E_{bb}}{C_o} = 254.55\,lb\,ft^{-1}$	$B_p = 0.44\,ft$	$As = 2.98\,ft$	$HDFbb = 3435.73\,lb$	MST48
		16d @ 5" o.c.	**5/8" AB. @ 36" o.c.**		

P1-4: 7/16" Sheathing w/ 8d nails @ 4" O.C.
Wind Capacity = 494 plf
Seismic Capacity = 353 plf

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Shear wall line CC:

Step 1. Wind analysis on wall

Wind loads per foot: $W_2 = 108.97\,lb\,ft^{-1}$ $W_{2E} = 137.09\,lb\,ft^{-1}$

Distance between shear wall: $L1 := 18.5 \cdot ft$

Shear wall panels: $Lcc := (12)ft$ $Lcc = 12\,ft$

Percent full height sheathing: $\% := \left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per IBC Table 2305.3.8.2

Wind Force: $vcc := \frac{\frac{(W_2 + W_{2E})}{2} \cdot \frac{L1}{2}}{Lcc}$ $vcc = 94.83\,lb\,ft^{-1}$ $\frac{vcc}{C_o} = 94.83\,lb\,ft^{-1}$ Wind loads per foot

Step 2. Seismic analysis on wall

Seismic Weight per level: $V_{2E} = 2909.16\,lb$ Assumed $\rho := 1.0$ Bldg Width in direction of Load: $Lt := 18.5 \cdot ft$

Seismic Force: $E_{cc} := \frac{0.7\rho \cdot \frac{V_{2E}}{Lt} \cdot \frac{L1}{2}}{Lcc}$ $E_{cc} = 84.85\,lb\,ft^{-1}$ $\frac{E_{cc}}{C_o} = 84.85\,lb\,ft^{-1}$ Seismic loads per foot

Step 3. Checking Overturning Moment & Holdown on Wall

Overturning Moment on Wall: Plate Height: $Pt := 8.0 \cdot ft$

$L_{cc} := 12 \cdot ft$ $OTM := vcc \cdot L_{cc} \cdot Pt$ $OTM = 9103.85\,lb\,ft$

Dead Load Resisting Overturning:

$W_R := 0.6(15 \cdot psf) \cdot 6 \cdot ft \cdot L_{cc} + 0.6 \cdot (10 \cdot psf) \cdot Pt \cdot 1 \cdot L_{cc}$

$DLRM := W_R \cdot \frac{L_{cc}}{2}$ $DLRM = 7344\,lb\,ft$

Holdown Force & Net Uplift:

$HDFcc := \frac{OTM - DLRM}{C_o \cdot L_{cc}}$ $HDFcc = 146.65\,lb$

Base Plate Nail Spacing (2003 NDS Table11N)
16d Common Nails & 1-1/2" Plate Hem-Fir

$Z_N := 122 \cdot lb$ $C_D := 1.33$ $Z'_N := Z_N \cdot C_D$ $Z'_N = 162.26\,lb$

$B_p := \frac{Z'_N}{vcc}$ $B_p = 1.71\,ft$ Per Nail

Anchor Bolt Spacing (2003 NDS Table 11E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 830 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1328\,lb$

$As := \frac{Z_B \cdot C_o}{vcc}$ $As = 14\,ft$ Per Bolt

Step 4. Shear Wall Summary:

Wind Force:	Seismic Force:	B.P. Nailing Spacing	A.B. Spacing	Holdown Force:	Holdown Types:
$\frac{vcc}{C_o} = 94.83\,lb\,ft^{-1}$	$\frac{E_{cc}}{C_o} = 84.85\,lb\,ft^{-1}$	$B_p = 1.71\,ft$	$As = 14\,ft$	$HDFcc = 146.65\,lb$	NO HOLDOWN
		16d @ 16" o.c.	**5/8" A.B. @ 60" o.c.**		

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

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Shear wall line DD:

Step 1. Wind analysis on wall

Wind loads per foot: $W_2 = 108.97\,lb\,ft^{-1}$ $W_{2E} = 137.09\,lb\,ft^{-1}$

Distance between shear wall: $L1 := 18.5\cdot ft$

Shear wall panels: $Ldd := (12 + 13.5)ft$ $Ldd = 25.5\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per IBC Table 2305.3.8.2

Wind Force: $vdd := \dfrac{\frac{(W_2 + W_{2E})}{2}\cdot\frac{L1}{2}}{Ldd}$ $vdd = 44.63\,lb\,ft^{-1}$ $\dfrac{vdd}{C_o} = 44.63\,lb\,ft^{-1}$ Wind loads per foot

Step 2. Seismic analysis on wall

Seismic Weight per level: $V_{2E} = 2909.16\,lb$ Assumed $\rho := 1.0$ Bldg Width in direction of Load: $Lt := 18.5\cdot ft$

Seismic Force: $E_{dd} := \dfrac{0.7\rho\cdot\frac{V_{2E}}{Lt}\cdot\frac{L1}{2}}{Ldd}$ $E_{dd} = 39.93\,lb\,ft^{-1}$ $\dfrac{E_{dd}}{C_o} = 39.93\,lb\,ft^{-1}$ Seismic loads per foot

Step 3. Checking Overturning Moment & Holdown on Wall

Overturning Moment on Wall: Plate Height: $Pt := 8.0\cdot ft$

$L_{dd} := 12\cdot ft$ $OTM := vdd\cdot L_{dd}\cdot Pt$ $OTM = 4284.17\,lb\,ft$

Dead Load Resisting Overturning:

$W_R := 0.6(15\cdot psf)\cdot 8\cdot ft\cdot L_{dd} + 0.6\cdot(10\cdot psf)\cdot Pt\cdot 1\cdot L_{dd}$

$DLRM := W_R\cdot\dfrac{L_{dd}}{2}$ $DLRM = 8640\,lb\,ft$

Holdown Force & Net Uplift:

$HDFdd := \dfrac{OTM - DLRM}{C_o\cdot L_{dd}}$ $HDFdd = -362.99\,lb$

Base Plate Nail Spacing (2003 NDS Table11N)
16d Common Nails & 1-1/2" Plate Hem-Fir

$Z_N := 122\cdot lb$ $C_D := 1.33$ $Z'_N := Z_N\cdot C_D$ $Z'_N = 162.26\,lb$

$B_p := \dfrac{Z'_N}{vdd}$ $B_p = 3.64\,ft$ Per Nail

Anchor Bolt Spacing (2003 NDS Table 11E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

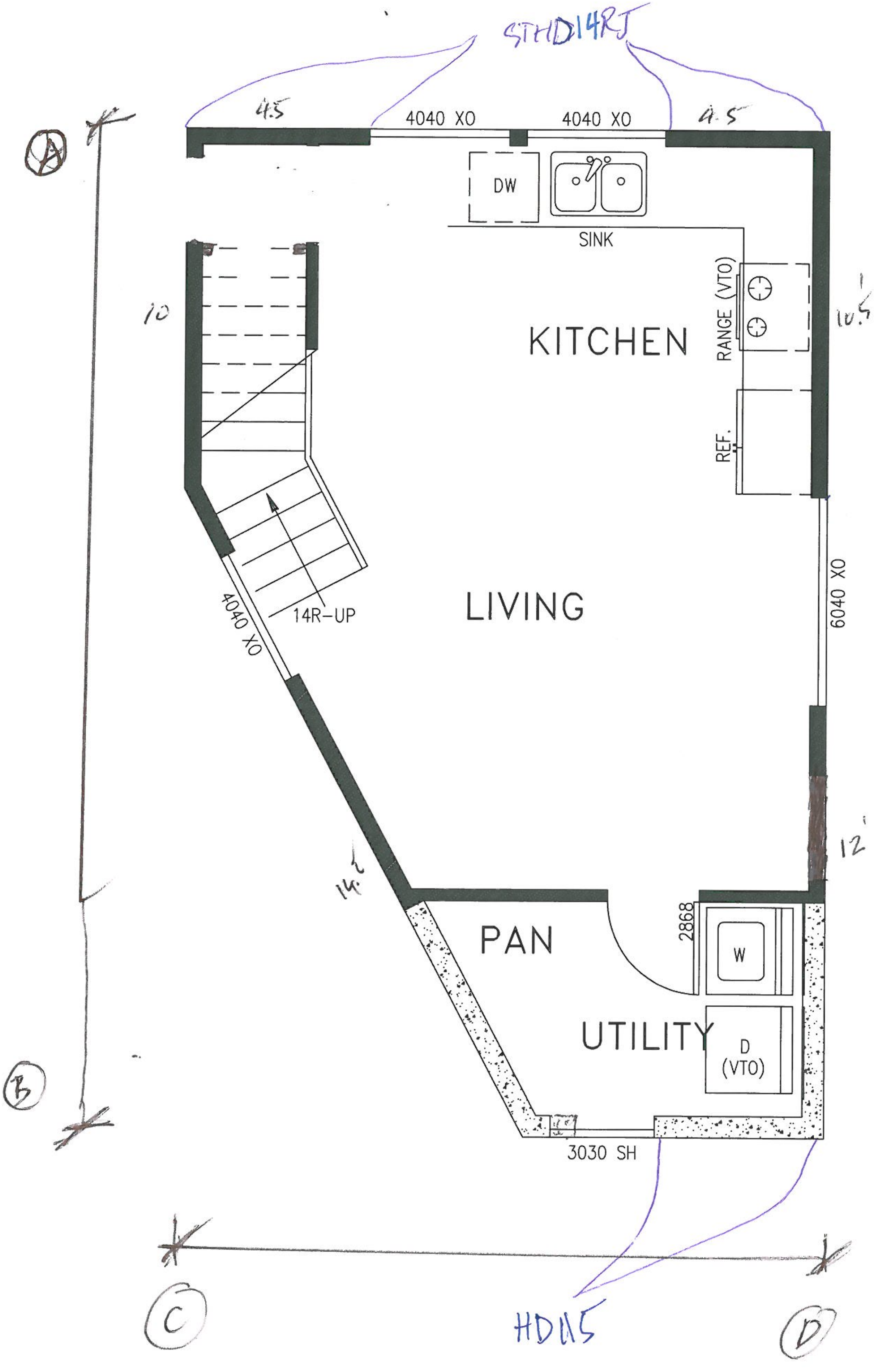
$A_s := 830\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1328\,lb$

$As := \dfrac{Z_B\cdot C_o}{vdd}$ $As = 29.76\,ft$ Per Bolt

Step 4. Shear Wall Summary:

Wind Force:	Seismic Force:	B.P. Nailing Spacing	A.B. Spacing	Holdown Force:	Holdown Types:
$\dfrac{vdd}{C_o} = 44.63\,lb\,ft^{-1}$	$\dfrac{E_{dd}}{C_o} = 39.93\,lb\,ft^{-1}$	$B_p = 3.64\,ft$	$As = 29.76\,ft$	$HDFdd = -362.99\,lb$	NO HOLDOWN
		16d @ 16" o.c.	**5/8" A.B. @ 60" o.c.**		

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf



MAIN FLOOR PLAN 1/4"=1'-0"

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Shear wall line A:

Step 1. Wind analysis on wall

Wind loads per foot: $W_1 = 187.32\,lb\,ft^{-1}$ $W_{1E} = 235.66\,lb\,ft^{-1}$

Distance between shear wall: $L1 := 29 \cdot ft$

Shear wall panels: $La := (4.5 \cdot 2)ft$ $La = 9\,ft$

Percent full height sheathing: $\% := \left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per IBC Table 2305.3.8.2

Wind Force: $va := \dfrac{vaa \cdot Laa + \frac{(W_1 + W_{1E})}{2} \cdot \frac{L1}{2}}{La}$ $va = 538.94\,lb\,ft^{-1}$ $\dfrac{va}{C_o} = 538.94\,lb\,ft^{-1}$ Wind loads per foot

Step 2. Seismic analysis on wall

Seismic Weight per level: $V_{1E} = 2321.24\,lb$ Assumed $\rho := 1.0$ Bldg Width in direction of Load: $Lt := 29 \cdot ft$

Seismic Force: $E_a := \dfrac{E_{aa} \cdot Laa + 0.7\rho \cdot \frac{V_{1E}}{Lt} \cdot \frac{L1}{2}}{La}$ $E_a = 203.4\,lb\,ft^{-1}$ $\dfrac{E_a}{C_o} = 203.4\,lb\,ft^{-1}$ Seismic loads per foot

Step 3. Checking Overturning Moment & Holdown on Wall

Overturning Moment on Wall: Plate Height: $Pt := 8.0 \cdot ft$

$L_a := 4.5 \cdot ft$ $OTM := va \cdot L_a \cdot Pt$ $OTM = 19402.02\,lb\,ft$

Dead Load Resisting Overturning:

$W_R := 0.6(15 \cdot psf) \cdot 2 \cdot ft \cdot L_a + 0.6 \cdot (10 \cdot psf) \cdot 2Pt \cdot L_a + [0.6(12 \cdot psf) \cdot 1 \cdot ft \cdot L_a]$

$DLRM := W_R \cdot \dfrac{L_a}{2}$ $DLRM = 1227.15\,lb\,ft$

Holdown Force & Net Uplift:

$HDFa := \dfrac{OTM - DLRM}{C_o \cdot L_a}$ $HDFa = 4038.86\,lb$

Base Plate Nail Spacing (2003 NDS Table11N)
16d Common Nails & 1-1/2" Plate Hem-Fir

$Z_N := 122 \cdot lb$ $C_D := 1.33$ $Z'_N := Z_N \cdot C_D$ $Z'_N = 162.26\,lb$

$B_p := \dfrac{Z'_N}{va}$ $B_p = 0.3\,ft$ Per Nail

Anchor Bolt Spacing (2003 NDS Table 11E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 830 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1328\,lb$

$As := \dfrac{Z_B \cdot C_o}{va}$ $As = 2.46\,ft$ Per Bolt

Step 4. Shear Wall Summary:

Wind Force:	Seismic Force:	B.P. Nailing Spacing	A.B. Spacing	Holdown Force:	Holdown Types:
$\dfrac{va}{C_o} = 538.94\,lb\,ft^{-1}$	$\dfrac{E_a}{C_o} = 203.4\,lb\,ft^{-1}$	$B_p = 0.3\,ft$	$As = 2.46\,ft$	$HDFa = 4038.86\,lb$	STHD14RJ
		16d @ 3" o.c.	**5/8" AB. @ 29" o.c.**		

P1-3: 7/16" Sheathing w/ 8d nails @ 3" O.C.
Wind Capacity = 638 plf
Seismic Capacity = 456 plf

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Shear wall line B:

Step 1. Wind analysis on wall

Wind loads per foot: $W_1 = 187.32\,lb\,ft^{-1}$ $W_{1E} = 235.66\,lb\,ft^{-1}$

Distance between shear wall: $L1 := 29 \cdot ft$

Shear wall panels: $Lb := (5)ft$ $Lb = 5\,ft$

Percent full height sheathing: $\% := \left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per IBC Table 2305.3.8.2

Wind Force: $vb := \dfrac{vbb \cdot Lbb + \frac{(W_1 + W_{1E})}{2} \cdot \frac{L1}{2}}{Lb}$ $vb = 970.1\,lb\,ft^{-1}$ $\frac{vb}{C_o} = 970.1\,lb\,ft^{-1}$ Wind loads per foot

Step 2. Seismic analysis on wall

Seismic Weight per level: $V_{1E} = 2321.24\,lb$ Assumed $\rho := 1.0$ Bldg Width in direction of Load: $Lt := 29 \cdot ft$

Seismic Force: $E_b := \dfrac{E_{bb} \cdot Lbb + 0.7\rho \cdot \frac{V_{1E}}{Lt} \cdot \frac{L1}{2}}{Lb}$ $E_b = 366.13\,lb\,ft^{-1}$ $\frac{E_b}{C_o} = 366.13\,lb\,ft^{-1}$ Seismic loads per foot

Step 3. Checking Overturning Moment & Holdown on Wall

Overturning Moment on Wall: Plate Height: $Pt := 5.0 \cdot ft$

$L_b := 5 \cdot ft$ $OTM := vb \cdot L_b \cdot Pt$ $OTM = 24252.52\,lb\,ft$

Dead Load Resisting Overturning:

$W_R := 0.6(15 \cdot psf) \cdot 2 \cdot ft \cdot L_b + 0.6 \cdot (10 \cdot psf) \cdot 2Pt \cdot L_b + \left[0.6(12 \cdot psf) \cdot 1 \cdot ft \cdot L_b\right]$

$DLRM := W_R \cdot \frac{L_b}{2}$ $DLRM = 1065\,lb\,ft$

Holdown Force & Net Uplift:

$HDFb := \dfrac{OTM - DLRM}{C_o \cdot L_b}$ $HDFb = 4637.5\,lb$

Base Plate Nail Spacing (2003 NDS Table11N)
16d Common Nails & 2-1/2" Plate Hem-Fir

$Z_N := 122 \cdot lb$ $C_D := 1.6$ $Z'_N := Z_N \cdot C_D$ $Z'_N = 195.2\,lb$

$B_p := \frac{Z'_N}{vb}$ $B_p = 0.2\,ft$ Per Nail

Anchor Bolt Spacing (2003 NDS Table 11E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 1030 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1648\,lb$

$As := \frac{Z_B \cdot C_o}{vb}$ $As = 1.7\,ft$ Per Bolt

Step 4. Shear Wall Summary:

Wind Force:	Seismic Force:	B.P. Nailing Spacing	A.B. Spacing	Holdown Force:	Holdown Types:
$\frac{vb}{C_o} = 970.1\,lb\,ft^{-1}$	$\frac{E_b}{C_o} = 366.13\,lb\,ft^{-1}$	$B_p = 0.2\,ft$ **16d @ 2.3" o.c. stagger**	$As = 1.7\,ft$ **5/8" AB. @ 20" o.c.**	$HDFb = 4637.5\,lb$	HDU5

P2-3: 7/16" Sheathing w/ 8d nails @ 3" O.C.
Wind Capacity = 1276 plf
Seismic Capacity = 988 plf

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Shear wall line C:

Step 1. Wind analysis on wall

Wind loads per foot: $W_1 = 187.32\,lb\,ft^{-1}$ $W_{1E} = 235.66\,lb\,ft^{-1}$

Distance between shear wall: $L1 := 18.5 \cdot ft$

Shear wall panels: $Lc := (8 + 14)ft$ $Lc = 22\,ft$

Percent full height sheathing: $\% := \left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100$ $\% = 100$ Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per IBC Table 2305.3.8.2

Wind Force: $vc := \dfrac{vcc \cdot Lcc + \frac{(W_1 + W_{1E})}{2} \cdot \frac{L1}{2}}{Lc}$ $vc = 140.65\,lb\,ft^{-1}$ $\frac{vc}{C_o} = 140.65\,lb\,ft^{-1}$ Wind loads per foot

Step 2. Seismic analysis on wall

Seismic Weight per level: $V_{1E} = 2321.24\,lb$ Assumed $\rho := 1.0$ Bldg Width in direction of Load: $Lt := 18.5 \cdot ft$

Seismic Force: $E_c := \dfrac{E_{cc} \cdot Lcc + 0.7\rho \cdot \frac{V_{1E}}{Lt} \cdot \frac{L1}{2}}{Lc}$ $E_c = 83.21\,lb\,ft^{-1}$ $\frac{E_c}{C_o} = 83.21\,lb\,ft^{-1}$ Seismic loads per foot

Step 3. Checking Overturning Moment & Holdown on Wall

Overturning Moment on Wall: Plate Height: $Pt := 8.0 \cdot ft$

$L_c := 8 \cdot ft$ $OTM := vc \cdot L_c \cdot Pt$ $OTM = 9001.56\,lb\,ft$

Dead Load Resisting Overturning:

$W_R := 0.6(15 \cdot psf) \cdot 7 \cdot ft \cdot L_c + 0.6 \cdot (10 \cdot psf) \cdot Pt \cdot 2L_c + [0.6(12 \cdot psf) \cdot 6 \cdot ft \cdot L_c]$

$DLRM := W_R \cdot \frac{L_c}{2}$ $DLRM = 6470.4\,lb\,ft$

Holdown Force & Net Uplift:

$HDFc := \dfrac{OTM - DLRM}{C_o \cdot L_c}$ $HDFc = 316.4\,lb$

Base Plate Nail Spacing (2003 NDS Table11N)
16d Common Nails & 1-1/2" Plate Hem-Fir

$Z_N := 122 \cdot lb$ $C_D := 1.33$ $Z'_N := Z_N \cdot C_D$ $Z'_N = 162.26\,lb$

$B_p := \frac{Z'_N}{vc}$ $B_p = 1.15\,ft$ Per Nail

Anchor Bolt Spacing (2003 NDS Table 11E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 830 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1328\,lb$

$As := \frac{Z_B \cdot C_o}{vc}$ $As = 9.44\,ft$ Per Bolt

Step 4. Shear Wall Summary:

Wind Force:	Seismic Force:	B.P. Nailing Spacing	A.B. Spacing	Holdown Force:	Holdown Types:
$\frac{vc}{C_o} = 140.65\,lb\,ft^{-1}$	$\frac{E_c}{C_o} = 83.21\,lb\,ft^{-1}$	$B_p = 1.15\,ft$	$As = 9.44\,ft$	$HDFc = 316.4\,lb$	no holdown
		16d @ 13" o.c.	**5/8" A.B. @ 60" o.c.**		

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

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Shear wall line D:

Step 1. Wind analysis on wall

Wind loads per foot: $W_1 = 187.32\,lb\,ft^{-1}$ $W_{1E} = 235.66\,lb\,ft^{-1}$

Distance between shear wall: $L1 := 18.5\cdot ft$

Shear wall panels: $Ld := (10.5 + 12)ft$ $Ld = 22.5\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$ Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per IBC Table 2305.3.8.2

Wind Force: $vd := \dfrac{vdd\cdot Ldd + \frac{(W_1 + W_{1E})}{2}\cdot\frac{L1}{2}}{Ld}$ $vd = 137.52\,lb\,ft^{-1}$ $\frac{vd}{C_o} = 137.52\,lb\,ft^{-1}$ Wind loads per foot

Step 2. Seismic analysis on wall

Seismic Weight per level: $V_{1E} = 2321.24\,lb$ Assumed $\rho := 1.0$ Bldg Width in direction of Load: $Lt := 18.5\cdot ft$

Seismic Force: $E_d := \dfrac{E_{dd}\cdot Ldd + 0.7\rho\cdot\frac{V_{1E}}{Lt}\cdot\frac{L1}{2}}{Ld}$ $E_d = 81.36\,lb\,ft^{-1}$ $\frac{E_d}{C_o} = 81.36\,lb\,ft^{-1}$ Seismic loads per foot

Step 3. Checking Overturning Moment & Holdown on Wall

Overturning Moment on Wall: Plate Height: $Pt := 8.0\cdot ft$

$L_d := 10.5\cdot ft$ $OTM := vd\cdot L_d\cdot Pt$ $OTM = 11552.01\,lb\,ft$

Dead Load Resisting Overturning:

$W_R := 0.6(15\cdot psf)\cdot 7\cdot ft\cdot L_d + 0.6\cdot(10\cdot psf)\cdot Pt\cdot 2L_d + [0.6(12\cdot psf)\cdot 6\cdot ft\cdot L_d]$

$DLRM := W_R\cdot\frac{L_d}{2}$ $DLRM = 11146.27\,lb\,ft$

Holdown Force & Net Uplift:

$HDFd := \dfrac{OTM - DLRM}{C_o\cdot L_d}$ $HDFd = 38.64\,lb$

Base Plate Nail Spacing (2003 NDS Table11N)
16d Common Nails & 1-1/2" Plate Hem-Fir

$Z_N := 122\cdot lb$ $C_D := 1.33$ $Z'_N := Z_N\cdot C_D$ $Z'_N = 162.26\,lb$

$B_p := \frac{Z'_N}{vd}$ $B_p = 1.18\,ft$ Per Nail

Anchor Bolt Spacing (2003 NDS Table 11E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 830\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1328\,lb$

$As := \frac{Z_B\cdot C_o}{vd}$ $As = 9.66\,ft$ Per Bolt

Step 4. Shear Wall Summary:

Wind Force:	Seismic Force:	B.P. Nailing Spacing	A.B. Spacing	Holdown Force:	Holdown Types:
$\frac{vd}{C_o} = 137.52\,lb\,ft^{-1}$	$\frac{E_d}{C_o} = 81.36\,lb\,ft^{-1}$	$B_p = 1.18\,ft$	$As = 9.66\,ft$	$HDFd = 38.64\,lb$	no holdown
		16d @ 12" o.c.	**5/8" AB. @ 60" o.c.**		

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf