STRUCTURAL CALCULATIONS

FOR:

Cynthia Mosby Design

REGARDING:

Structural Design For Addition

LOCATION:

Beck / Armstrong Residence
1601 N. Humbolt St., Portland, OR

AE1362

December 9, 2013

REGISTERED PROFESSIONAL ENGINEER
OREGON
APR 14 1992
MICHAEL D. ARCHER

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13-23613 6-RS

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PROJECT DESCRIPTION: STRUCTURAL DESIGN FOR A 14'x18' ONE STORY ADDITION TO AN EXISTING 1.5 STORY WOOD FRAMED SFR.

DESIGN CRITERIA: 2010 OREGON STRUCTURAL SPECIALTY CODE

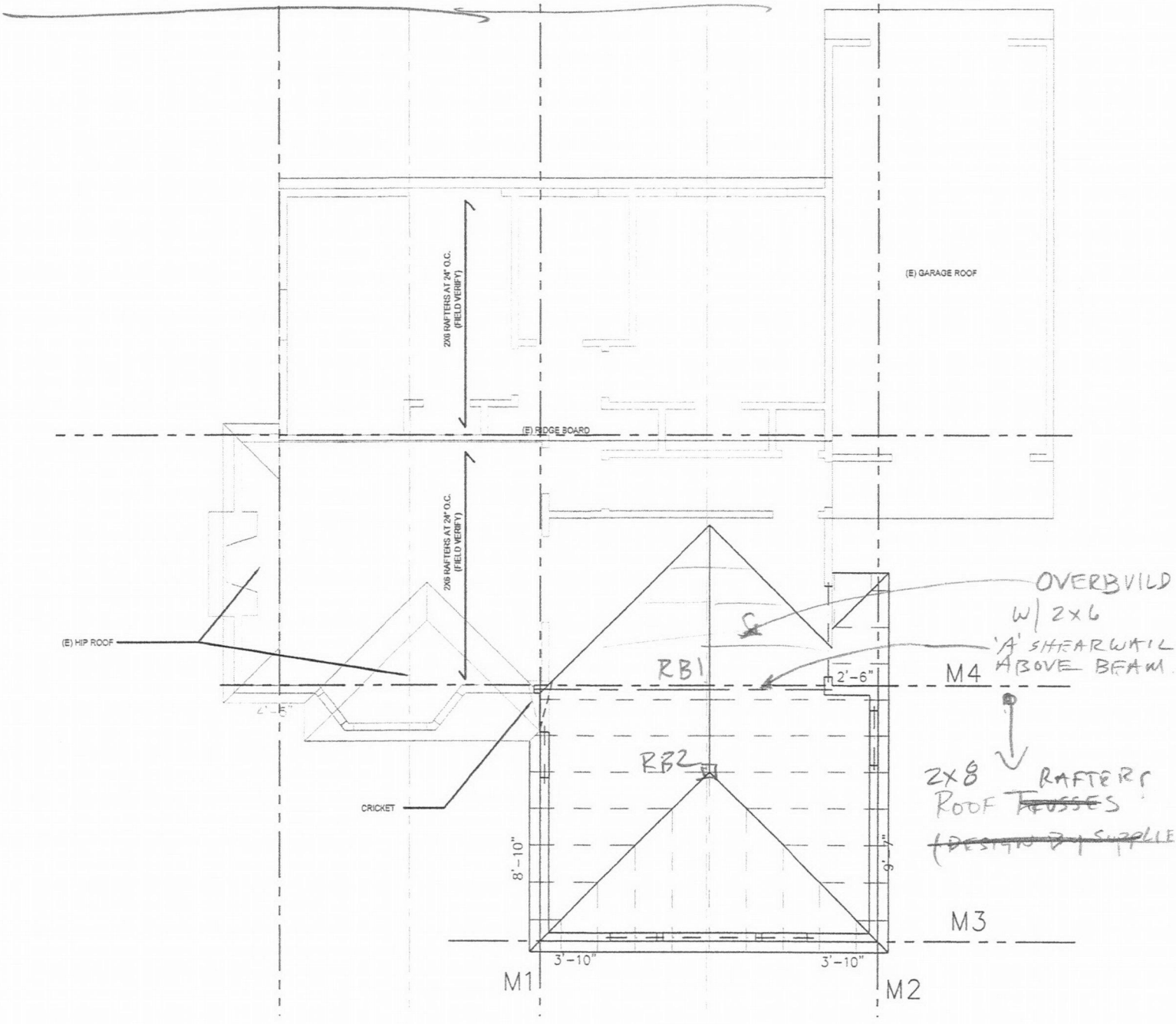
DESIGN LOADS: FLOOR: 40 PSF LIVE 15 PSF DEAD
ROOF: 25 PSF LIVE 15 PSF DEAD
WIND: 95 MPH EXP. B.
SEISMIC: $S_s = .974g$ $S_1 = .342g$ (SITE SPECIFIC)

SEISMIC DESIGN:

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 45.55867
Longitude = -122.683655
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.05 deg grid spacing

Period (sec)	Sa (g)
0.2	0.974 (Ss, Site Class B)
1.0	0.342 (S1, Site Class B)

SHEARWALL LINES & ROOF FRAMING



SEISMIC DESIGN :

1. **Lateral loads as follows:**
   a. **Seismic** Design Per 2010 Oregon Structural Specialty Code
   Use V = 0.080 W Per IBC Earthquake Loads Worksheet P. 4 (includes reduction for ASD)
2. **Vertical loads as follows:**

	DL (psf)	LL (psf)	TL (psf)
Roof =	15	25	40
Floor =	15	40	55
Int Wood Walls =	10		10
Ext Wood Walls =	12		12

IV. CALCULATIONS:

1. **Lateral Loading Calculations**

Main Floor Seismic Forces Front-to-Back:

Wall Line: **M1**

Seismic areas:				Seismic Wts:		Seismic Forces:	
Roof diaphragm area =	600	sq ft		Dead wt =	9,000 lbs	Seismic force =	720 lbs
Floor diaphragm area =	0	sq ft		Dead wt =	0 lbs	Seismic force =	0 lbs
Wall area to floor=	240	sq ft	Int	Dead wt =	2,400 lbs	Seismic force =	192 lbs
Wall area to floor =	184	sq ft	Ext	Dead wt =	2,208 lbs	Seismic force =	177 lbs
				Total wt =	13,608 lbs	Total force =	1,089 lbs

Wall Line: **M2**

Seismic areas:	0			Seismic Wts:		Seismic Forces:	
Roof diaphragm area =	270	sq ft		Dead wt =	4,050 lbs	Seismic force =	324 lbs
Floor diaphragm area =	0	sq ft		Dead wt =	0 lbs	Seismic force =	0 lbs
Wall area to floor=	0	sq ft	Int	Dead wt =	0 lbs	Seismic force =	0 lbs
Wall area to floor =	120	sq ft	Ext	Dead wt =	1,440 lbs	Seismic force =	115 lbs
				Total wt =	5,490 lbs	Total force =	439 lbs

Wall Line: **M3**

Seismic areas:				Seismic Wts:		Seismic Forces:	
Roof diaphragm area =	144	sq ft		Dead wt =	2,160 lbs	Seismic force =	173 lbs
Floor diaphragm area =	0	sq ft		Dead wt =	0 lbs	Seismic force =	0 lbs
Wall area to floor=	0	sq ft	Int	Dead wt =	0 lbs	Seismic force =	0 lbs
Wall area to floor =	100	sq ft	Ext	Dead wt =	1,200 lbs	Seismic force =	96 lbs
				Total wt =	3,360 lbs	Total force =	269 lbs

Wall Line: **M4**

Seismic areas:				Seismic Wts:		Seismic Forces:	
Roof diaphragm area =	364	sq ft		Dead wt =	5,460 lbs	Seismic force =	437 lbs
Floor diaphragm area =	0	sq ft		Dead wt =	0 lbs	Seismic force =	0 lbs
Wall area to floor=	64	sq ft	Int	Dead wt =	640 lbs	Seismic force =	51 lbs
Wall area to floor =	192	sq ft	Ext	Dead wt =	2,304 lbs	Seismic force =	184 lbs
				Total wt =	8,404 lbs	Total force =	672 lbs

IBC2006 (1613), ASCE 7-05 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Soil Site Class	D	Table 20-3-1, Default = D	
Response Spectral Acc. (0.2 sec) S_s =	97.40%g	= 0.974g	Figure 22-1 through 22-14
Response Spectral Acc.(1.0 sec) S_1 =	34.20%g	= 0.342g	Figure 22-1 through 22-14
Site Coefficient F_a =	1.110		Table 11.4-1
Site Coefficient F_v =	1.716		Table 11.4-2
Max Considered Earthquake Acc. S_{MS} =	$F_a.S_s$	= 1.082	(11.4-1)
Max Considered Earthquake Acc. S_{M1} =	$F_v.S_1$	= 0.587	(11.4-2)
@ 5% Damped Design S_{DS} =	$2/3(S_{MS})$	= 0.721	(11.4-3)
S_{D1} =	$2/3(S_{M1})$	= 0.391	(11.4-4)
Building Occupancy Categories	II, Standard		Table 1-1
Design Category Consideration:	Flexible Diaphragm		with dist. between seismic resisting system >40ft
Seismic Design Category for 0.1sec	D		Table 11.6-1
Seismic Design Category for 1.0sec	D		Table 11.6-2
S1 < .75g	NA		Section 11.6
Since Ta < .8Ts (see below), SDC =	D	Control (exception of Section 11.6 does not apply)	
Comply with Seismic Design Category D		IRC, Seismic Design Category= D1	T-R301.2.2.1.1

12.8 Equivalent lateral force procedure

B. BUILDING FRAME SYSTEMS T-12.2-

Seismic Force Resisting Systems 23. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets

C_t =	0.02	x = 0.75		T-12.8-2
Building ht. H_n =	30	ft	Limited Building Height (ft) = 65	
C_u =	1.400	for S_{D1} of 0.391g	Table 12.8-1	
Approx Fundamental period, T_a =	$C_t(h_n)^x$	= 0.256	12.8-7	T_L = 8 Sec
Calculated T shall not exceed ≤Cu.Ta		= 0.359		Use T = 0.256 sec.
0.8Ts =	$0.8(S_{D1}/S_{DS})$	= 0.434	Control (exception of Section 11.6 does not apply)	
Is structure Regular & ≤ 5 stories ?	Yes			12.8.1.3
Response Spectral Acc.(0.2 sec) S_s =	0.974g			*Max Ss ≤ 0.15g*
F_a =	1.11			
@ 5% Damped Design S_{DS} =	$\frac{2}{3}(F_a.S_s)$	= 0.721g		(11.4-3)
Response Modification Coef. R =	7			Table-12.2-1
Over Strength Factor Ω_o =	2			foot note g
Importance factor I =	1			Table 11.5-1
Seismic Base Shear V =	$C_s W$			

$$C_s = \frac{S_{DS}}{R/I} = 0.103 \quad (12.8\text{-}2)$$

$$\text{or need not to exceed, } C_s = \frac{S_{D1}}{(R/I).T} = 0.218 \quad \text{For } T \le T_L \quad (12.8\text{-}3)$$

$$\text{or } C_s = \frac{S_{D1}T_L}{T^2(R/I)} \quad \text{N/A} \quad \text{For } T > T_L \quad (12.8\text{-}4)$$

C_s shall not be less than = 0.01 (12.8-5)

Min C_s = $0.5S_1I/R$ N/A For $S_1 \ge 0.6g$ (12.8-6)

Use C_s = 0.103

Design base shear V = 0.103 W **Control**

T-12.14

12.14 Simplified Seismic base shear 13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets

@ 5% Damped Design S_{DS} = 0.721 SDC = D Limitations: P

F = 1 For one-story buuilding R = 6.5

$$V = \frac{FS_{DS}(W)}{R} = 0.111\ W \quad \times .7 = .08\ W$$

ASD

Wind Design - Alternate All-Heights method

Main Wind Force Resisting Systems

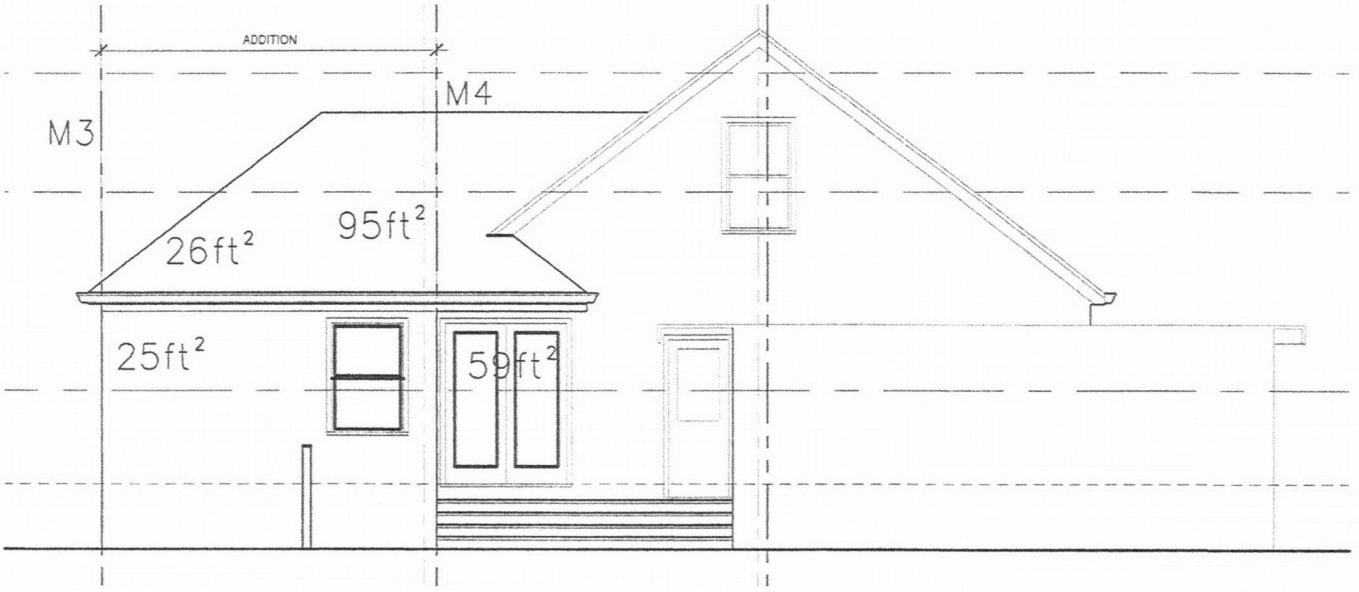
95	mph	Basic Wind Speed	23.15	psf	qs	Wind Pressure
B		Exposure Factor	0.85		G	Gust Factor
1	I	Importance Factor	0.85		Kd	Directionality Factor
1.0	Kzt	Topographic Factor	14	ft	h	Mean roof height
			9	/12		Roof pitch

Wind Pressures:

+ internal Pressure	qs	Kz	Cnet	I	Pw (psf)		
Horizontal Forces							
W-Wall							
15	23.15	0.57	0.43	1.00	5.7		
20	23.15	0.62	0.43	1.00	6.2		
25	23.15	0.67	0.43	1.00	6.6		
30	23.15	0.70	0.43	1.00	7.0		
L-Wall	23.15	0.56	-0.51	1.00	-6.7		
		Normal Forces				Horizontal	Vertical
W-Roof	23.15	0.56	0.14	1.00	1.8	1.1	1.1
L-Roof	23.15	0.56	-0.66	1.00	-8.6	-5.1	-6.9

- internal Pressure							
Horizontal Forces							
W-Wall							
0-15'	23.15	0.57	0.73	1.00	9.7		
15-20'	23.15	0.62	0.73	1.00	10.5		
20-25'	23.15	0.67	0.73	1.00	11.2		
25-30'	23.15	0.70	0.73	1.00	11.8		
L-Wall	23.15	0.56	-0.21	1.00	-2.7		
		Normal Forces				Horiz (psf)	Vert (psf)
W-Roof	23.15	0.56	0.44	1.00	5.7	3.4	4.6
L-Roof	23.15	0.56	-0.35	1.00	-4.6	-2.7	-3.7

WIND TRIBUTARY AREAS



Wind Loads at each Shear Wall Line:

M1

		Area	WW	LW	Total
+ internal Pressure					
Walls:	0-15'	47	5.7	-6.7	582
	15-20'	0	6.2	-6.7	0
	20-25'	0	6.6	-6.7	0
Roof:		184	1.1	-5.1	1148
					1730
- internal Pressure					
Walls:	0-15'	47	9.7	-2.7	585
	15-20'	0	10.5	-2.7	0
	20-25'	0	11.2	-2.7	0
Roof:	20'-29'	184	3.4	-2.7	1134
					1719

1730

M2

		Area (ft^2)	WW (psf)	LW (psf)	Total (lbs)
+ internal Pressure					
Walls:	0-15'	27	5.7	-6.7	334
	15-20'	0	6.2	-6.7	0
	20-25'	0	6.6	-6.7	0
Roof:		84	1.1	-5.1	524
					858
- internal Pressure					
Walls:	0-15'	27	9.7	-2.7	336
	15-20'	0	10.5	-2.7	0
	20-25'	0	11.2	-2.7	0
Roof:	20'-29'	84	3.4	-2.7	518
					854

858

M3

		Area (ft^2)	WW (psf)	LW (psf)	Total (lbs)
+ internal Pressure					
Walls:	0-15'	25	5.7	-6.7	309
	15-20'	0	6.2	-6.7	0
	20-25'	0	6.6	-6.7	0
Roof:		26	1.1	-5.1	162
					472
- internal Pressure					
Walls:	0-15'	25	9.7	-2.7	311
	15-20'	0	10.5	-2.7	0
	20-25'	0	11.2	-2.7	0
Roof:	20'-29'	26	3.4	-2.7	160
					472

472

M4

		Area (ft^2)	WW (psf)	LW (psf)	Total (lbs)
+ internal Pressure					
Walls:	0-15'	59	5.7	-6.7	730
	15-20'	0	6.2	-6.7	0
	20-25'	0	6.6	-6.7	0
Roof:		95	1.1	-5.1	593
					1323
- internal Pressure					
Walls:	0-15'	59	9.7	-2.7	735
	15-20'	0	10.5	-2.7	0
	20-25'	0	11.2	-2.7	0
Roof:	20'-29'	95	3.4	-2.7	585
					1320

1323

Shear Loads in each Shearwall Line:

Shear Wall Mark	Total Force to wall (lbs)	Length of Shear Wall (ft)	Unit Shear in wall (lbs/ft)	Shear wall type	
M1	1730	8.8	197	A	Wind controls
M2	858	9.5	90	A	Wind controls
M3	472	7.8	60	A	Wind controls
M4	1323	2.0	661	S	Wind controls

Overturn calculations:

Shear Wall Mark	Wall Length (L) (feet)	Wall Height (H) (feet)	Force to Wall (F) (lbs)	Moment Overturn (M_O=F*H) (ft - lbs)	Dead Load to Wall (W) (lbs/ft)	Dead Load Mom Resist (M_{R1}=2/3* W* L^2/2) (ft - lbs)	Wall End Point Load P (lbs)	Additional Moment Resisting (M_{R2}=2/3*P* L) (ft - lbs)	Net Moment (M_N=M_{R1+} M_{R2}-M_O) (ft - lbs)	Holdown Force Required * (=M_N/L-10") (lbs)	Holdown type
M1	8.8	8	1,730	13,839	150	3,872	500	2,933	-7,034	883	1
M2	9.5	8	858	6,867	150	4,513	500	3,167	813	Not Req'd	-
M3	3.8	8	230	1,838	150	722	1,000	2,533	1,417	Not Req'd	-
M4	2.0	8	1,323	10,584	150	200	1,000	1,333	-9,050	7,735	2

* Note: Holddown forces of up to 500lb are assumed to be resisted by conventional anchor bolts at the bottom plate.

Shearwall & Holddown Key:

'A' = 1/2" (nom) APA rated sheathing w/ 8d common nails at 6" O.C. at panel edges; 12" O.C. at 'field' studs. All panel edges blocked w/ 2x material. 1/2" dia. X 12" J-bolts with Simp BP1/2-3 plate washer at 4' O.C. at 2x6 PT sill plate. Capacity= 260 PLF
'S' = Simpson Wood Strong-Wall SW24x9 with Capacity= 360 PLF

1= Simpson HDU2-SDS2.5 Holddown w/ 5/8" dia. Simpson SSTB16 anchor bolt at double stud. Capacity = 3075 lbs.
2= Simpson SSTB 7/8" dia. anchor bolt at Wood Strong-Wall. Capacity = 9045 lbs.

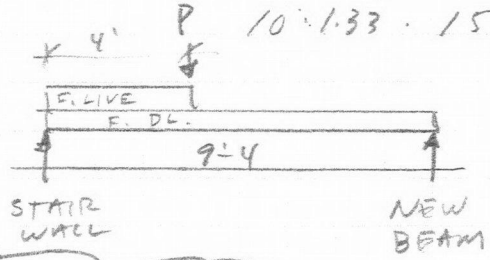
NOTE: ROOF BEAM @ M4 WALL LINE LOADS WALL:
USE (6) SIMP SDWC SCREWS - CAPACITY = 385# · 6 = 2310#
OK.

2FJ-1

EXISTING 2nd FLOOR JOISTS w/ ROOF LOAD SPAN = 9'-4"

LOADING 0-4' 40psf / 15psf @ 1.33' TRIB. FLOOR + DEAD
4'-9'-4 15 psf @ 1.33' TRIB. DEAD

POINT LOAD @ 4' 10' 1.33' · 25 psf = # SNOW
P 10' 1.33 · 15 psf # DEAD



EXISTING 2x8 @ 16" O.C. OK. P. 11

D 179#
L 46#
S 143#
} @ 1.33 O.C.

RB-1

NEW BEAM @ KIT. SPAN = 15'-6"

W = 179/1.33' = 135 PLF DL
46/1.33' = 35 PLF LL
143/1.33' = 108 PLF SL

P @ RIDGE → 65 ft² TRB. @ 7'
65 ft² 25 psf = 1625 SL
65 ft² 15 psf = 1000 DL

USE (3) 1 3/4 x 11 7/8 LVL P. 13

ARCHER ENGINEERING L.L.C.

Project Title: Beck Armstrong Addition
Engineer: mda
Project Descr:

Project ID: AE1362

11

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Wood Beam

Lic. # : KW-06007596 **Licensee : ARCHER ENGINEEIRNG LLC**

Description : Existing 2nd Floor Joist 2FJ

CODE REFERENCES

Calculations per NDS 2005, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-05

Wood Species : Douglas Fir - Larch
Wood Grade : No.1 & Better

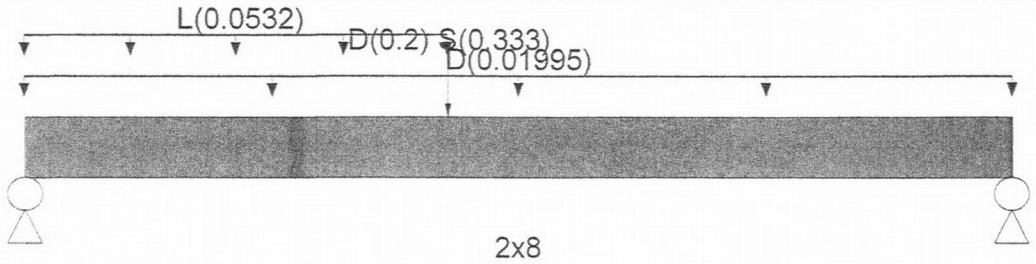
Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	1200 psi
Fb - Compr	1200 psi
Fc - Prll	1550 psi
Fc - Perp	625 psi
Fv	180 psi
Ft	800 psi

E : Modulus of Elasticity

Ebend- xx	1800 ksi
Eminbend - xx	660 ksi
Density	32.21 pcf

Repetitive Member Stress Increase



Span = 9.330 ft

Applied Loads

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

Uniform Load : D = 0.0150 ksf, Tributary Width = 1.330 ft, (floor dead)
Uniform Load : L = 0.040 ksf, Extent = 0.0 –>> 4.0 ft, Tributary Width = 1.330 ft, (Floor Live)
Point Load : D = 0.20, S = 0.3330 k @ 4.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.787 : 1**	Maximum Shear Stress Ratio	=	**0.338 : 1**
Section used for this span		**2x8**	Section used for this span		**2x8**
fb : Actual	=	1,302.90 psi	fv : Actual	=	60.79 psi
FB : Allowable	=	1,656.00 psi	Fv : Allowable	=	180.00 psi
Load Combination		+D+S+H	Load Combination		+D+0.750L+0.750S+H
Location of maximum on span	=	4.018 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.153 in	Ratio =	732
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.259 in	Ratio =	431
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+S	1	0.2593	4.461		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.565	0.367
D Only	0.207	0.179
L Only	0.167	0.046
S Only	0.190	0.143
L+S	0.357	0.188
D+L	0.375	0.224
D+S	0.398	0.322

Wood Beam

Lic. # : KW-06007596 **Licensee : ARCHER ENGINEEIRNG LLC**

Description : Typ ceiling Jst

CODE REFERENCES

Calculations per NDS 2005, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-05

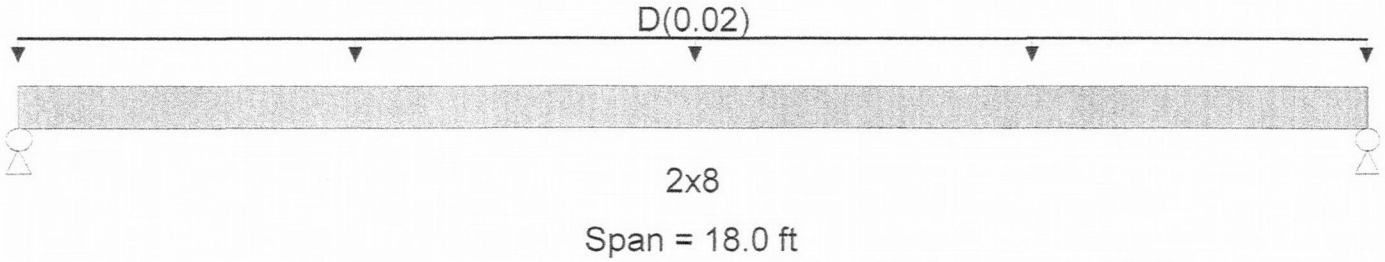
Wood Species :
Wood Grade :

Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	1,000.0 psi
Fb - Compr	1,000.0 psi
Fc - Prll	1,000.0 psi
Fc - Perp	1,000.0 psi
Fv	65.0 psi
Ft	65.0 psi

E : Modulus of Elasticity

Ebend- xx	1,300.0 ksi
Eminbend - xx	1,300.0 ksi
Density	34.0 pcf



Applied Loads

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

Uniform Load : D = 0.010 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.740 : 1**	Maximum Shear Stress Ratio	=	**0.357 : 1**
Section used for this span		**2x8**	Section used for this span		**2x8**
fb : Actual	=	739.69 psi	fv : Actual	=	23.20 psi
FB : Allowable	=	1,000.00 psi	Fv : Allowable	=	65.00 psi
Load Combination		D Only	Load Combination		D Only
Location of maximum on span	=	9.000 ft	Location of maximum on span	=	17.409 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.767 in	Ratio =	281
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.7673	9.066		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.180	0.180
D Only	0.180	0.180

Wood Beam

Lic. # : KW-06007596 **Licensee : ARCHER ENGINEEIRNG LLC**

Description : B1 - Roof Beam

CODE REFERENCES

Calculations per NDS 2005, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-05

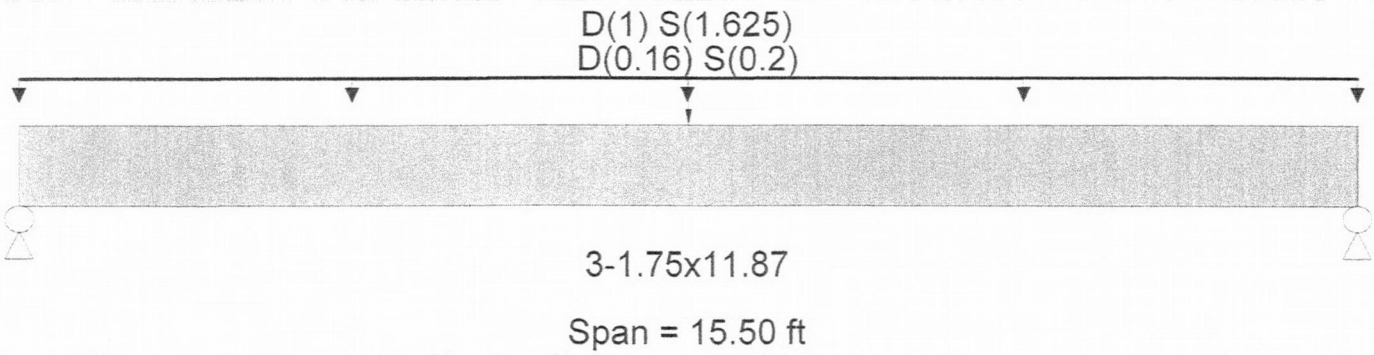
Wood Species : Boise Cascade
Wood Grade : Versa Lam 3100

Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	3,100.0 psi
Fb - Compr	3,100.0 psi
Fc - Prll	3,000.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	2,100.0 psi

E : Modulus of Elasticity

Ebend- xx	2,000.0 ksi
Eminbend - xx	2,000.0 ksi
Density	32.210 pcf



Applied Loads

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

Uniform Load : D = 0.020, S = 0.0250 ksf, Tributary Width = 8.0 ft, (Existing Roof)
Point Load : D = 1.0, S = 1.625 k @ 7.750 ft, (New ridge)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.658 : 1	Maximum Shear Stress Ratio	=	0.317 : 1
Section used for this span		3-1.75x11.87	Section used for this span		3-1.75x11.87
fb : Actual	=	2,040.69 psi	fv : Actual	=	90.38 psi
FB : Allowable	=	3,100.00 psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	7.750 ft	Location of maximum on span	=	14.538 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.328 in	Ratio =	567
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.562 in	Ratio =	330
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+S	1	0.5624	7.807		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.103	4.103
D Only	1.740	1.740
S Only	2.363	2.363
D+S	4.103	4.103

Project: 1362

Location: Roof Beam 1 HIPS
Roof Beam
[2009 International Building Code(2005 NDS)]
1.75 IN x 9.5 IN x 12.5 FT (Actual 14.1 FT)
1.9E Microllam - iLevel Trus Joist
Section Adequate By: 112.7%
Controlling Factor: Deflection

Michael Archer
Archer Engineeering

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DEFLECTIONS Center

Live Load	0.23	IN L/736
Dead Load	0.21	in
Total Load	0.44	IN L/383

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

REACTIONS

	A	B
Live Load	648 lb	324 lb
Dead Load	586 lb	312 lb
Total Load	1235 lb	636 lb
Bearing Length	0.94 in	0.48 in

BEAM DATA

Span Length	12.5 ft
Unbraced Length-Top	0 ft
Unbraced Length-Bottom	0 ft
Roof Pitch Side One:	9 :12
Roof Pitch Side Two:	9 :12
Roof Duration Factor	1.15
Beam End Elevation Difference	6.6 ft

MATERIAL PROPERTIES

1.9E Microllam - iLevel Trus Joist

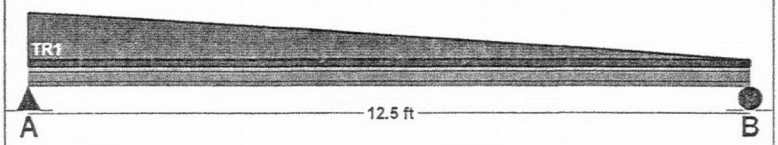
	Base Values	Adjusted
Bending Stress:	Fb = 2600 psi	Fb' = 3087 psi
	Cd=1.15 CF=1.03	
Shear Stress:	Fv = 285 psi	Fv' = 328 psi
	Cd=1.15	
Modulus of Elasticity:	E = 1900 ksi	E' = 1900 ksi
Comp. ⊥ to Grain:	Fc - ⊥ = 750 psi	Fc - ⊥' = 750 psi

Controlling Moment: 2994 ft-lb
6.25 ft from left support
Created by combining all dead and live loads.

Controlling Shear: 934 lb
At a distance d from support.
Created by combining all dead and live loads.

Comparisons with required sections:	Req'd	Provided
Section Modulus:	11.64 in3	26.32 in3
Area (Shear):	4.27 in2	16.63 in2
Moment of Inertia (deflection):	58.79 in4	125.03 in4
Moment:	2994 ft-lb	6771 ft-lb
Shear:	934 lb	3633 lb

LOADING DIAGRAM



ROOF LOADING

Side One:

Roof Live Load:	LL =	25	psf
Roof Dead Load:	DL =	15	psf
Rafter Length (Hip/Valley):	RL =	8.8	ft

Tributary Width based on half span of rafters.

Side Two:

Roof Live Load:	LL =	25	psf
Roof Dead Load:	DL =	15	psf
Rafter Length (Hip/Valley):	RL =	8.8	ft

Tributary Width based on half span of rafters.

Wall Load:	WALL =	0	plf

SLOPE/PITCH ADJUSTED LENGTHS AND LOADS

Adjusted Beam Length:	Ladj =	14.14	ft
Beam Self Weight:	BSW =	5	plf
Beam Triangular Live Load Adjusted For Slope:	TRL =	156	plf
Beam Triangular Dead Load Adjusted For Slope:	TRD =	117	plf
Beam Uniform Dead Load Adjusted For Slope:	wD-adj =	0	plf

Project Title: Beck Armstrong Addition
Engineer: mda
Project Descr:
Project ID: AE1362

ARCHER ENGINEERING L.L.C.

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ENERCALC, INC. 1983-2013, Build:6.13.8.31, Ver:6.13.8.31

Wood Beam

Lic. # : KW-06007596 **Licensee : ARCHER ENGINEEIRNG LLC**

Description : B2 - Ceiling jsts @ Hip support

CODE REFERENCES

Calculations per NDS 2005, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-05

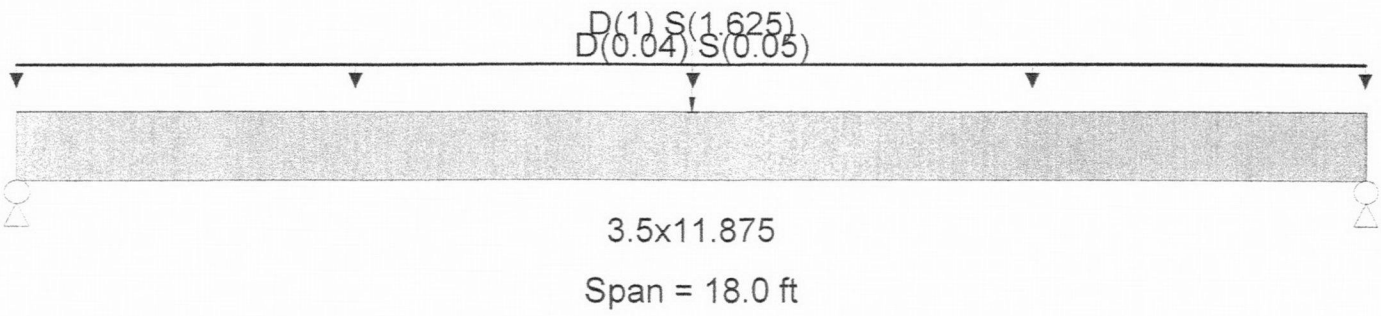
Wood Species : Boise Cascade
Wood Grade : Versa Lam 3100

Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	3,100.0 psi
Fb - Compr	3,100.0 psi
Fc - Prll	3,000.0 psi
Fc - Perp	750.0 psi
Fv	285.0 psi
Ft	2,100.0 psi

E : Modulus of Elasticity

Ebend- xx	2,000.0 ksi
Eminbend - xx	2,000.0 ksi
Density	32.210 pcf



Applied Loads

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

Uniform Load : D = 0.020, S = 0.0250 ksf, Tributary Width = 2.0 ft, (Typ Roof Load)
Point Load : D = 1.0, S = 1.625 k @ 9.0 ft, (Hip support)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.727** : 1	Maximum Shear Stress Ratio	=	**0.257** : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
fb : Actual	=	2,253.05 psi	fv : Actual	=	73.37 psi
FB : Allowable	=	3,100.00 psi	Fv : Allowable	=	285.00 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	9.000 ft	Location of maximum on span	=	17.015 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.472 in	Ratio =	457
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.785 in	Ratio =	275
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+S	1	0.7852	9.066		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.123	2.123
D Only	0.860	0.860
S Only	1.263	1.263
D+S	2.123	2.123

ARCHER ENGINEERING L.L.C.

Project Title: Beck Armstrong Addition
Engineer: mda
Project Descr:

Project ID: AE1362

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ENERCALC, INC. 1983-2013, Build:6.13.8.31, Ver:6.13.8.31

Wood Beam

Lic. # : KW-06007596

Licensee : ARCHER ENGINEEIRNG LLC

Description : TYP FLOOR JST - 1st Floor

CODE REFERENCES

Calculations per NDS 2005, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method : Allowable Stress Design
Load Combination :ASCE 7-05

Wood Species : Douglas Fir - Larch
Wood Grade : No.2

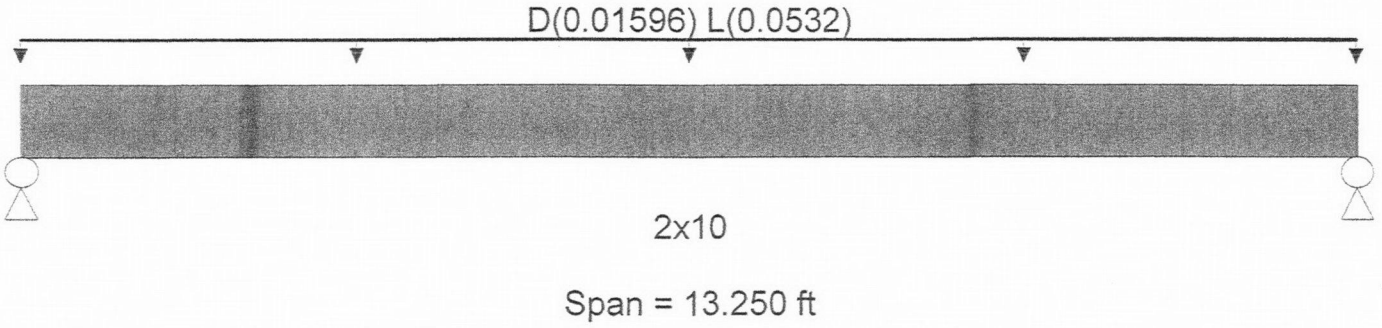
Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	900 psi
Fb - Compr	900 psi
Fc - Prll	1350 psi
Fc - Perp	625 psi
Fv	180 psi
Ft	575 psi

E : Modulus of Elasticity

Ebend- xx	1600 ksi
Eminbend - xx	580 ksi
Density	32.21 pcf

Repetitive Member Stress Increase



Applied Loads

Service loads entered Load Factors will be applied for calculations.

Uniform Load : D = 0.0120, L = 0.040 ksf, Tributary Width = 1.330 ft, (FLOOR)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.748 : 1**	Maximum Shear Stress Ratio	=	**0.245 : 1**
Section used for this span		**2x10**	Section used for this span		**2x10**
fb : Actual	=	851.44 psi	fv : Actual	=	44.11 psi
FB : Allowable	=	1,138.50 psi	Fv : Allowable	=	180.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	6.625 ft	Location of maximum on span	=	12.525 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.234 in	Ratio =	678
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.305 in	Ratio =	521
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.3048	6.673		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.458	0.458
D Only	0.106	0.106
L Only	0.352	0.352
D+L	0.458	0.458