STRUCTURAL CALCULATIONS

FOR

SIERRA B

LOT 437 @ WILLAMETTE LANDING

LEGEND HOMES
CORVALLIS, OR

STRUCTURAL REGISTERED PROFESSIONAL ENGINEER 15472 OREGON JULY 25, 1991 FARAMARZ DEAN AZIMI

EXPIRES: 12/31/2013

BVD13-00028

THESE CALCULATIONS ARE VOID IF SEAL AND SIGNATURE ARE NOT ORIGINAL

JANUARY 10, 2013
JOB NUMBER: 13-T008

FILE COPY

APPROVED JAN 14 2013 DEVELOPMENT SERVICES CITY OF CORVALLIS

TO BE SCANNED

FROELICH ENGINEERS INC

RECEIVED JAN 11 2013 DEVELOPMENT SERVICES

* * * LIMITATIONS * * *

ENGINEER WAS RETAINED IN A LIMITED CAPACITY FOR THIS PROJECT. DESIGN IS BASED UPON INFORMATION PROVIDED BY THE CLIENT, WHO IS SOLELY RESPONSIBLE FOR ACCURACY OF SAME. NO RESPONSIBILITY AND / OR LIABILITY IS ASSUMED BY, OR IS TO BE ASSIGNED TO THE ENGINEER FOR ITEMS BEYOND THAT SHOWN ON THESE SHEETS.

▲ Main Office
6969 SW Hampton St.
Portland, Oregon 97223
503-624-7005

www.froelich-engineers.com

▲ Central Oregon
745 NW Mt. Washington Dr. #205
Bend, Oregon 97701
541-383-1828

STRUCTURAL CALCULATIONS

FOR

SIERRA B

FOR

LEGEND HOMES

THESE CALCULATIONS ARE VOID IF SEAL AND SIGNATURE ARE NOT ORIGINAL

* * * LIMITATIONS * * *
ENGINEER WAS RETAINED IN A LIMITED CAPACITY FOR THIS PROJECT.
DESIGN IS BASED UPON INFORMATION PROVIDED BY THE CLIENT, WHO IS SOLELY RESPONSIBLE FOR ACCURACY OF SAME. NO RESPONSIBILITY AND / OR LIABILITY IS ASSUMED BY, OR IS TO BE ASSIGNED TO THE ENGINEER FOR ITEMS BEYOND THAT SHOWN ON THESE SHEETS

JANUARY 8, 2013
JOB NUMBER : 13 --T008

FROELICH
ENGINEERS

☑ MAIN OFFICE
6969 SW Hampton Street
Tigard, Oregon 97223
503 624-7005/503 624-9770 FAX

CENTRAL OREGON
745 N.W. Mt. Washington Dr., Suite 205
Bend, Oregon 97701
541 383-1828/541 383-7696 FAX

FROELICH
ENGINEERS

Client: Legend Homes
Project: Sierra B
Project Number: 13-T008
Date January 8, 2013
By: YSP

Scope of Work

Froelich Engineers has provided full structural lateral and gravity design of the project per the 2010 OSSC with the exception of continuous footings that fall within the parameters set forth by the 2011 ORSC. The roof trusses are designed by others.

Froelich Engineers has provided details only to the areas pertaining to our design. Froelich Engineers did not design or review the details for the entire project.

FROELICH
ENGINEERS

MAIN OFFICE
6969 SW Hampton Street
Tigard, Oregon 97223
503 624-7005/503 624-9770 FAX

CENTRAL OREGON
745 NW Mt Washington Dr Suite 205
Bend, Oregon 97701
541 383-1828/541 383-7696 FAX

-Statement of Code Compliance-

All engineering calculations in this packet that reference outdated codes, including but not limited to: hand written calculations, computer outputs, and spreadsheets, have been reviewed by FCE and meet or exceed the minimum requirements for the following code standards;

2010 OSSC
2011 ORSC
2005 NDS
2008 AF&PA SDPWS
ASCE 7-05
ACI 318-08

SIERRA B

FROELICH
ENGINEERS

Client: Legend Homes
Project: Sierra B
Proj. #: 13-T008
Date: 1/8/2013
By: YSP

Design Criteria:

General:

Building Code(s):	2009 IBC 2010 OSSC 2011 ORSC

Roof Live Load:

Snow =	25	psf
Deflection Criteria:	L/240	
Load Duration:	1.15	

Floor Live Loads:

Floor Live =	40	psf
Deflection Criteria:	L/360	

Wind Load*:

Speed:	80	mph
Exposure:	B	
Importance Factor:	1.0	
Special Req's:	no	

Seismic Load:

Design Category	D
Site Class	D
Response Coeff	6.5
Importance Factor:	1.0

Soils Data:

Allowable Bearing (assumed):	1500	psf
Frost Depth:	18	in
Special Soils Req's:	---	

FROELICH
ENGINEERS

Client: Legend Homes
Project: Sierra B
Project Number: 13-T008
Date: Jan. 8, 13
By: YSP

DEAD LOAD TAKE OFF

ROOF DEAD LOAD

Framing (RFR)	= 5.0 PSF
Sheathing ½"(RPL)	= 1.5 PSF
Roofing (RRF)	= 3.0 PSF
Mech/Electrical (RME)	= 1.5 PSF
Ceiling (RCG)	= 2.2 PSF
Insulation (RIN)	= 1.5 PSF
Miscellaneous (MIS)	= 0.3 PSF

ROOF DEAD LOAD RDL = RFR + RPL + RRF + RME + RCG + RIN + MIS

RDL = 15 PSF

FLOOR DEAD LOAD

Framing (FFR)	= 3.0 PSF
Sheathing/Floor (FPL)	= 3.5 PSF
Mech/Electrical (FME)	= 2.0 PSF
Lower Ceiling (FCG)	= 2.0 PSF
Miscellaneous (FMS)	= 1.5 PSF

FLOOR DEAD LOAD FDL = FFR + FPL + FME + FCG + FMS

FDL = 12 PSF

BALCONY DEAD LOAD

BDL = 10 PSF

FROELICH
ENGINEERS

Client:	Legend Homes
Project:	Sierra B
Proj. #:	13-T008
Date:	1/8/2013
By:	YSP

Snow drift Calculations

According to ASCE 7

D (psf) =	17.25
Wb (ft) =	38
Pg (psf) =	25
Pf (psf) =	25
W (ft) =	12

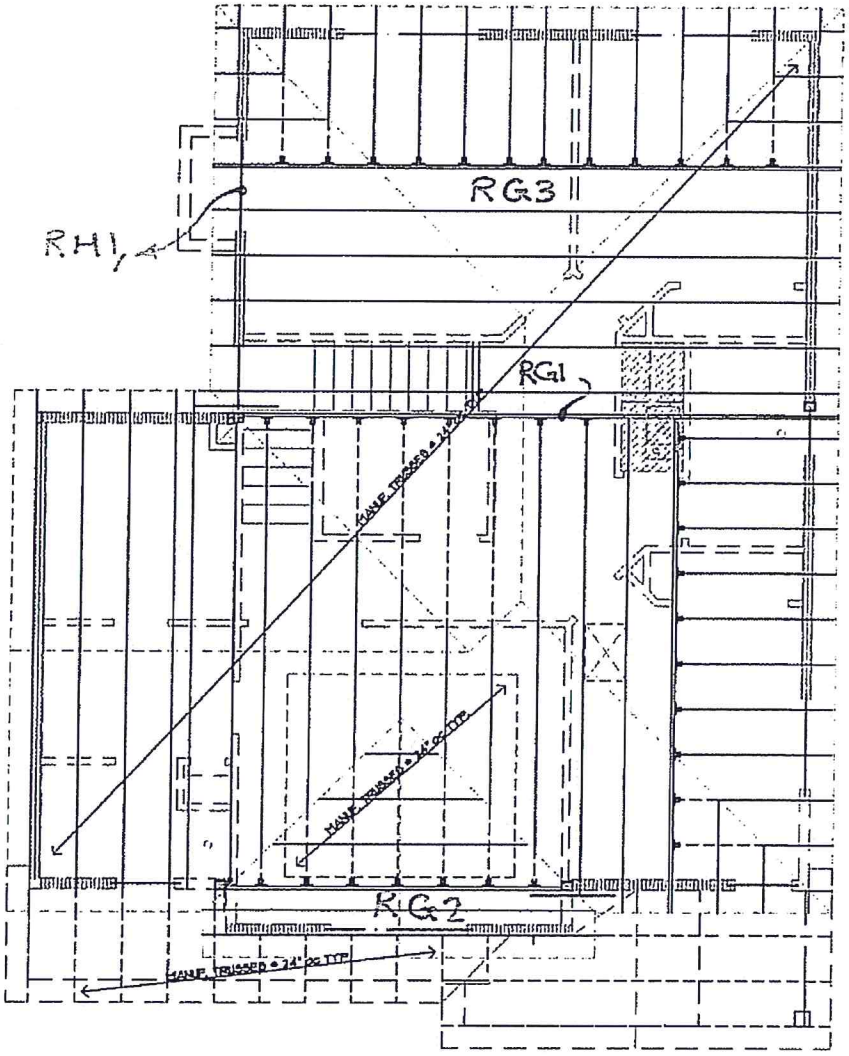
Drift Snow Loads	
hd =	2.02
Pm =	35
Wd =	8.07

Sliding Snow	
Pss =	8
Wd =	15

D = Density of Snow (pcf)
W_b = Width of Building (feet)
P_g = Ground Snow Load (psf)
P_f = Snow load on flat roofs (psf)
h_d = Height of drifted snow (feet)
P_m = Drifted snow pressure (psf)
W_d = Width of drift (feet)
W = Eave to Ridge (feet)
P_{ss} = Sliding snow pressure (psf)

$h_d = 0.43 * l_u^{1/3} * (p_g+10)^{1/4} - 1.5$
$P_m = D * h_d$
$W_d = 4 * h_d$

$P_{ss} = 0.4 * P_f * W / W_s$
W_s = Width of lower roof (feet)


RG3
RH1
RG1
RG2

FCE FROELICH CONSULTING ENGINEERS, INC
6969 SW Hampton Street
Portland, Oregon 97223
FAX 503.624.9770
503.624.7005
745 NW Mt. Washington Drive
Suite 204
Bend, Oregon 97701
FAX 541.383.7696
541.383.1828
www.froelich-engineers.com

CLIENT:
PROJECT:
NUMBER:
DATE:
BY:

ROOF FRAMING PLAN

ROOF DEAD LOAD: 15 PSF

ROOF LIVE LOAD: 25 PSF

USE PRE. MANUF. TRUSSES @ 24" O.C.

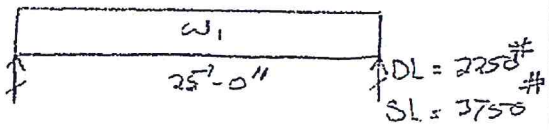
ROOF GIRDER TRUSS: (For Reaction Only)

RG1:

w_1:

DL = (12')(15) = 180 PLF

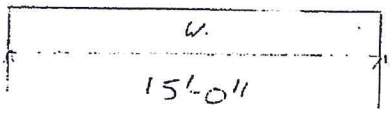
SL = (12')(25) = 300 PLF



RG2:

w_1 DL = 180 PLF

SL = 300 PLF



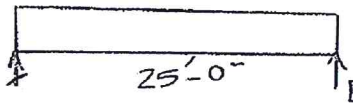
DL = 1,350#

SL = 2,250#

RG3:

w_1: DL = (4')(15) = 60 PLF

SL = (4')(25) = 100 PLF



DL = 750#

SL = 1250#

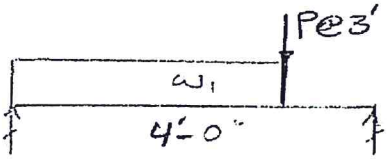
ROOF HEADER:

RH1:

w_1:

DL = (13')(15) = 200 PLF

SL = (13')(25) = 325 PLF



P (FROM RG3)

DL = 750#

SL = 1250#

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Jan 8, 2013 10:33	RH1 wwb

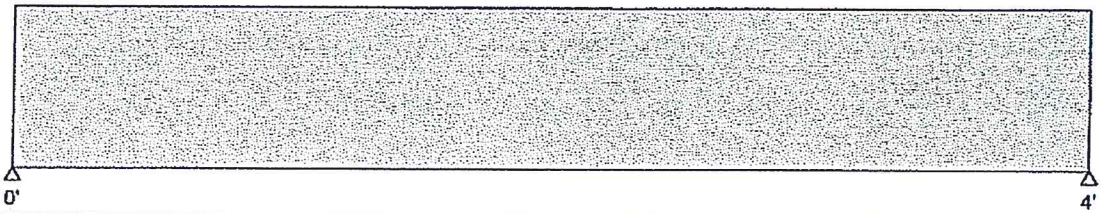
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Partial UDL	200.0	200.0	0.00	3.00	No
Load2	Snow	Partial UDL	325.0	325.0	0.00	3.00	No
Load3	Dead	Point	750		3.00		No
Load4	Snow	Point	1250		3.00		No

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



Dead	575		800
Live	922		1303
Total	1496		2103
Bearing:			
LC number	2		2
Length	1.00		1.00

Lumber-soft, D.Fir-L, No.2, 4x8"

Self Weight of 6.03 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 123	Fv' = 207	fv/Fv' = 0.59
Bending(+)	fb = 825	Fb' = 1337	fb/Fb' = 0.62
Live Defl'n	0.02 = <L/999	0.13 = L/360	0.15
Total Defl'n	0.03 = <L/999	0.20 = L/240	0.16

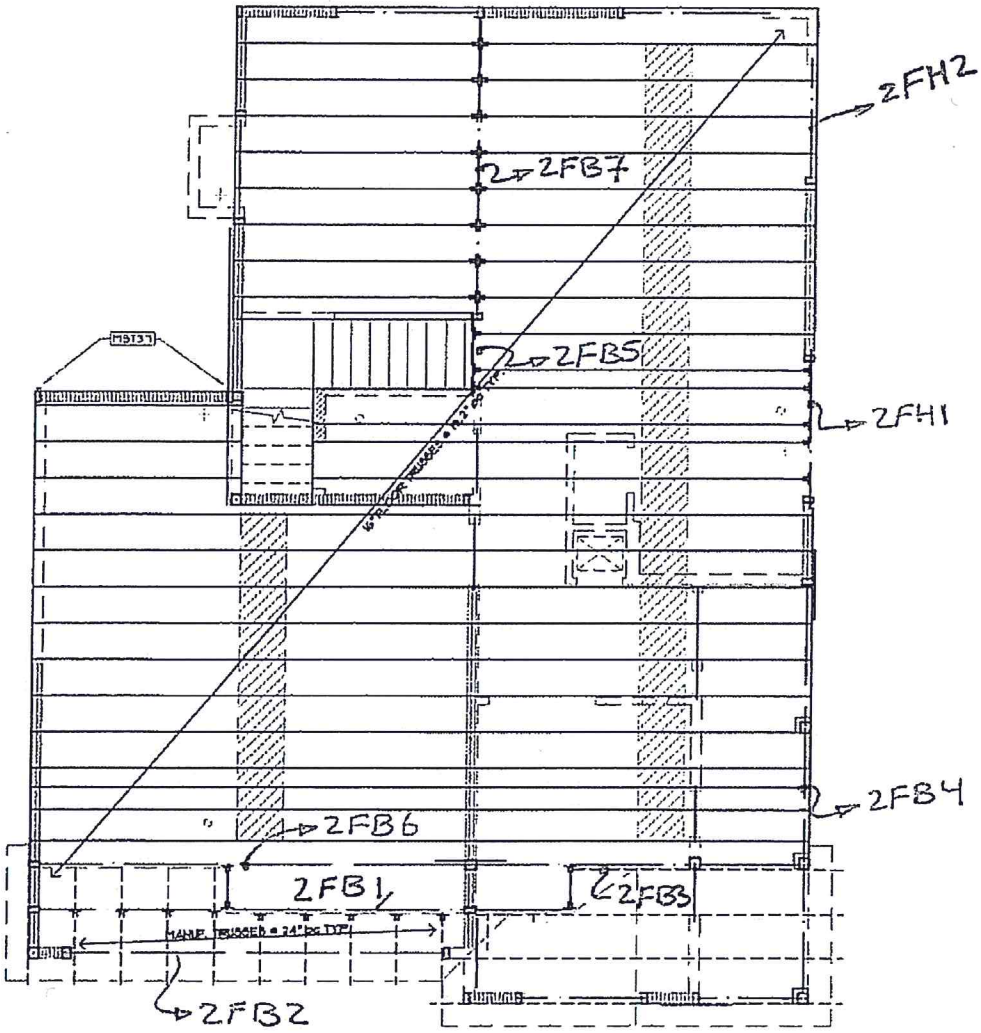
ADDITIONAL DATA:

FACTORS:	F	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fb'+	900	1.15	1.00	1.00	0.994	1.300	1.00	1.00	1.00	1.00	-	2
Fv'	180	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fcp'	625	-	1.00	1.00	-	-	-	-	1.00	1.00	-	-
E'	1.6 million		1.00	1.00	-	-	-	-	1.00	1.00	-	2

```
Bending(+): LC# 2 = D+S, M =   2108 lbs-ft
Shear     : LC# 2 = D+S, V =   2103, V design =   2075 lbs
Deflection: LC# 2 = D+S   EI=    178e06 lb-in2
Total Deflection = 1.00(Dead Load Deflection) + Live Load Deflection.
  (D=dead  L=live  S=snow  W=wind  I=impact  C=construction  CLd=concentrated)
  (All LC's are listed in the Analysis output)
```

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application
2. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4 4 1


2FH2
2FB7
2FB5
2FH1
2FB4
2FB6
2FB1
2FB3
2FB2

FCE
FROELICH CONSULTING ENGINEERS, INC

6969 SW Hampton Street
Portland, Oregon 97223
FAX 503 624 9770
503 624 7605

745 NW Mt. Washington Drive
Suite 204
Bend, Oregon 97701
FAX 541 383 7696
541 383 1898

www.froelich-engineers.com

CLIENT:

PROJECT: SIERRA

NUMBER:

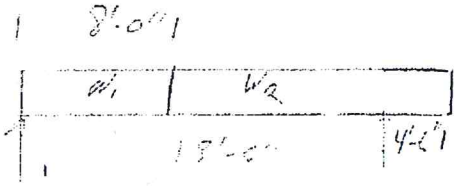
DATE:

BY:

2ND FLOOR FRAMING

USE 16" FLOOR TRUSSES BY OTHERS

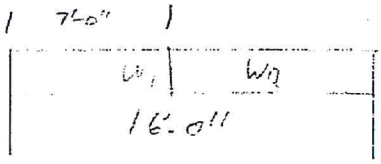
2ND FLOOR BEAMS



2FB1

W_1: $DL = (2')(15) = 30$ PLF
$SL = (2')(25) = 50$ PLF

W_2: $DL = (2')(15) + (9')(10) + 12 = 162$ PLF
$SL = 50$ PLF
$LL = 40$ PLF

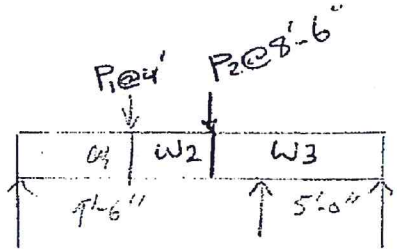


2FB2

W_1: $DL = (3')(15) = 45$ PLF
$SL = (3')(25) = 75$ PLF

W_2: $DL = (2')(15) = 30$ PLF
$SL = (2')(25) = 50$ PLF

2FB3



W_1: $DL = (2)(12) = 24$ PLF
$LL = (2)(40) = 80$ PLF

W_2: $DL = 12 + (8')(10) + (11')(15) = 257$ PLF
$SL = (11')(25) = 275$ PLF

W_3: $DL = (2')(12) + 90 + (2')(15) = 145$ PLF
$SL = (2')(25) = 50$ PLF
$LL = (2')(40) = 80$ PLF

P_1: (FROM RGI)
$DL = 1350^{\#}$
$SL = 2250^{\#}$

P_2 (FROM GIRDER TRUSS)
$DL = (21\frac{1}{2})(4')(15) = 630^{\#}$
$SL = (21\frac{1}{2})(4')(25) = 1050^{\#}$

FCE
FROELICH CONSULTING ENGINEERS, INC

6969 SW Hampton Street
Portland, Oregon 97223
FAX 503 624.9770
503 624 7005

745 NW Mt. Washington Drive
Suite 204
Bend, Oregon 97701
FAX 541 383.7696
541.383.1828

www.froelich-engineers.com

CLIENT:

PROJECT:

NUMBER:

DATE:

BY:

2FB4

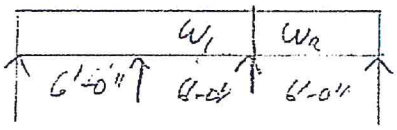
$DL = (3')(12) + (8')(10) + (4')(15) = 176\ PLF$

W_1 $LL = (3')(40) = 120\ PLF$

$SL = (4')(25) = 100\ PLF$

W_2 $DL = (2.5')(15) = 60$

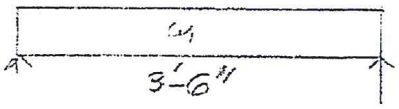
$SL = (2.5')(25 + 24 + 25) = 436$



2FB5

W_1 $DL = (15/2)(12) = 90\ PLF$

$LL = (15/2)(40) = 300\ PLF$



2FB6

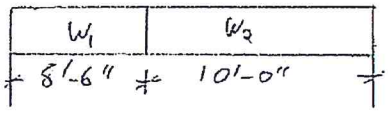
W_1 $DL = (8')(10) + (12')(15) + 12 = 272\ PLF$

$SL = (12')(25) = 300\ PLF$

$LL = 40\ PLF$

W_2 $DL = (2')(12) = 24\ PLF$

$LL = (2')(40) = 80\ PLF$



2FB7

SPAN = 13'-6"

$DL = (12.5')(12) = 150\ PLF$

$LL = (12.5')(40) = 500\ PLF$

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Seth Davis Froelich Consulting Engineers Feb. 8, 2012 13:21	2FB1.wwb

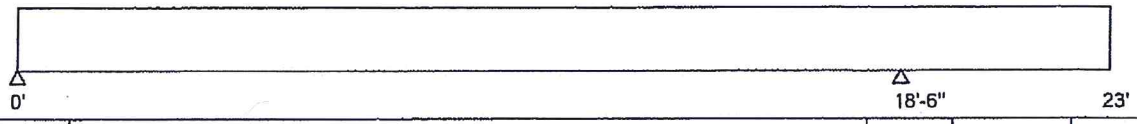
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Partial UDL	30.0	30.0	0.00	8.00	No
Load2	Snow	Partial UDL	50.0	50.0	0.00	8.00	No
Load3	Dead	Partial UDL	162.0	162.0	8.00	23.00	No
Load4	Snow	Partial UDL	50.0	50.0	8.00	23.00	No
Load5	Live	Partial UDL	40.0	40.0	8.00	23.00	No

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



	0'	18'-6"	23'
Dead	621	2152	
Live	435	913	
Total	1056	3065	
Bearing:			
LC number	4	3	0
Length	1.00	1.00	0.00
Cb	1.00	1.52	0.00

LSL, 1.55E, 2300Fb, 3-1/2x16"

Self Weight of 4.47 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 46	Fv' = 356	fv/Fv' = 0.13
Bending(+)	fb = 505	Fb' = 1243	fb/Fb' = 0.41
Bending(-)	fb = 168	Fb' = 2215	fb/Fb' = 0.08
Deflection:			
Interior Live	0.07 = <L/999	0.62 = L/360	0.11
Total	0.20 = <L/999	0.92 = L/240	0.21
Cantil. Live	0.04 = <L/999	0.30 = L/180	0.14
Total	0.13 = L/429	0.45 = L/120	0.28

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application.
2. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
3. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
4. The critical deflection value has been determined using maximum back-span deflection. Cantilever deflections do not govern design.

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Seth Davis Froelich Consulting Engineers Feb 8, 2012 13:23	2FB2 wwb

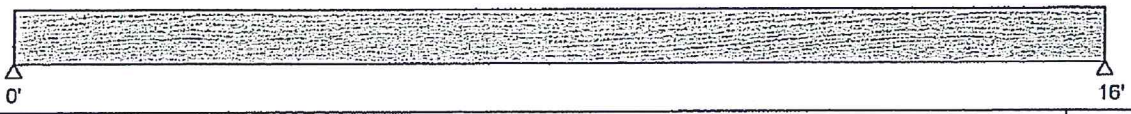
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Partial UDL	45.0	45.0	0.00	7.00	No
Load2	Snow	Partial UDL	75.0	75.0	0.00	7.00	No
Load3	Dead	Partial UDL	30.0	30.0	7.00	16.00	No
Load4	Snow	Partial UDL	50.0	50.0	7.00	16.00	No

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



Dead	421	362
Live	537	438
Total	958	801
Bearing:		
LC number	2	2
Length	1.00	1.00

Timber-soft, D.Fir-L, No.2, 6x10"

Self Weight of 12 41 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 25	Fv' = 195	fv/Fv' = 0.13
Bending(+)	fb = 502	Fb' = 1006	fb/Fb' = 0.50
Live Defl'n	0.17 = <L/999	0.53 = L/360	0.32
Total Defl'n	0.31 = L/613	0.80 = L/240	0.39

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application.
2. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4 4.1

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Jan. 8, 2013 10:48	2FB3 wwb

Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Partial UDL	24.0	24.0	0.00	4.00	No
Load2	Live	Partial UDL	80.0	80.0	0.00	4.00	Yes
Load3	Dead	Partial UDL	257.0	257.0	4.00	8.50	No
Load4	Snow	Partial UDL	275.0	275.0	4.00	8.50	Yes
Load5	Live	Partial UDL	80.0	80.0	4.00	8.50	Yes
Load6	Dead	Point	1350		4.00		No
Load7	Snow	Point	2250		4.00		Yes
Load8	Dead	Partial UDL	145.0	145.0	8.50	14.50	No
Load9	Live	Partial UDL	80.0	80.0	8.50	14.50	Yes
Load10	Snow	Partial UDL	50.0	50.0	8.50	14.50	Yes
Load11	Dead	Point	630		8.50		No
Load12	Snow	Point	1050		8.50		Yes

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :

Dead	1032		3539		
Live	1382		4208		
Uplift					1088
Total	2414		7747		10
Bearing:					
LC number	8		4		0
Length	1.06		3.03		1.00
Cb	1.00		1.12		1.00

Glulam-Unbal., West Species, 24F-1.8E WS, 3-1/2x16"

Self Weight of 12.89 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv* = 132	Fv' = 276	fv*/Fv' = 0.48
Bending(+)	fb = 751	Fb' = 2518	fb/Fb' = 0.30
Bending(-)	fb = 623	Fb' = 1637	fb/Fb' = 0.38
Live Defl'n	0.03 = <L/999	0.32 = L/360	0.10
Total Defl'n	0.05 = <L/999	0.48 = L/240	0.11

*The effect of point loads within a distance d of the support has been included as per NDS 3.4.3.1

ADDITIONAL DATA:

FACTORS:	F	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Notes	Cn	LC#
Fb'+	2400	1.15	1.00	1.00	0.912	1.000	1.00	1.00	1.00	1.00	-	4
Fb'-	1450	1.15	1.00	1.00	0.982	1.000	1.00	1.00	1.00	1.00	-	4
Fv'	240	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	4
Fcp'	650	-	1.00	1.00	-	-	-	-	1.00	-	-	-
E'	1.8 million		1.00	1.00	-	-	-	-	1.00	-	-	4

Bending(+): LC# 4 = D+S, M = 9350 lbs-ft
Bending(-): LC# 4 = D+S, M = 7753 lbs-ft
Shear : LC# 4 = D+S, V = 5676, V design* = 4941 lbs
Deflection: LC# 4 = D+S EI= 2150e06 lb-in2
Total Deflection = 1.00(Dead Load Deflection) + Live Load Deflection.
(D=dead L=live S=snow W=wind I=impact C=construction CLd=concentrated)
(All LC's are listed in the Analysis output)
(Load Pattern: s=S/2, X=L+S or L+C, _=no pattern load in this span)

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Feb 8, 2012 15:56	2FB4.wwb

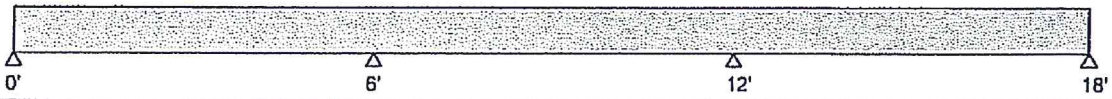
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Partial UDL	176.0	176.0	0.00	12.00	No
Load2	Live	Partial UDL	120.0	120.0	0.00	12.00	Yes
Load3	Snow	Partial UDL	100.0	100.0	0.00	12.00	Yes
Load4	Dead	Partial UDL	60.0	60.0	12.00	18.00	No
Load5	Snow	Partial UDL	435.0	435.0	12.00	18.00	Yes

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



Dead	429		1282		760		139
Live	450		1090		1997		1126
Total	880		2372		2756		1265
Bearing:							
LC number	23		21		17		16
Length	1.00		1.00		1.00		1.00
Cb	1.00		1.53		1.42		1.00

Lumber-soft, D.Fir-L, No.2, 4x10"

Self Weight of 7.69 plf automatically included in loads;
Lateral support: top= at supports. bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 64	Fv' = 207	fv/Fv' = 0.31
Bending(+)	fb = 383	Fb' = 1230	fb/Fb' = 0.31
Bending(-)	fb = 379	Fb' = 1236	fb/Fb' = 0.31
Live Defl'n	0.02 = <L/999	0.20 = L/360	0.12
Total Defl'n	0.02 = <L/999	0.30 = L/240	0.08

ADDITIONAL DATA:

FACTORS:	F	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fb'+	900	1.15	1.00	1.00	0.990	1.200	1.00	1.00	1.00	1.00	-	16
Fb'-	900	1.15	1.00	1.00	0.995	1.200	1.00	1.00	1.00	1.00	-	17
Fv'	180	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	17
Fcp'	625	-	1.00	1.00	-	-	-	-	1.00	1.00	-	-
E'	1.6 million		1.00	1.00	-	-	-	-	1.00	1.00	-	16

```
Bending(+): LC#16 = D+S (pattern: SsS), M =   1592 lbs-ft
Bending(-): LC#17 = D+S (pattern: sSS), M =   1577 lbs-ft
Shear     : LC#17 = D+S (pattern: sSS), V =   1771, V design =   1383 lbs
Deflection: LC#16 = D+S (pattern: SsS)   EI=    369e06 lb-in2
Total Deflection = 1.00(Dead Load Deflection) + Live Load Deflection.
  (D=dead  L=live  S=snow  W=wind  I=impact  C=construction  CLd=concentrated)
  (All LC's are listed in the Analysis output)
  (Load Pattern: s=S/2,  X=L+S or L+C,  _=no pattern load in this span)
```

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application
2. Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
3. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Seth Davis Froelich Consulting Engineers Feb. 8, 2012 14:04	2FB5.wwb

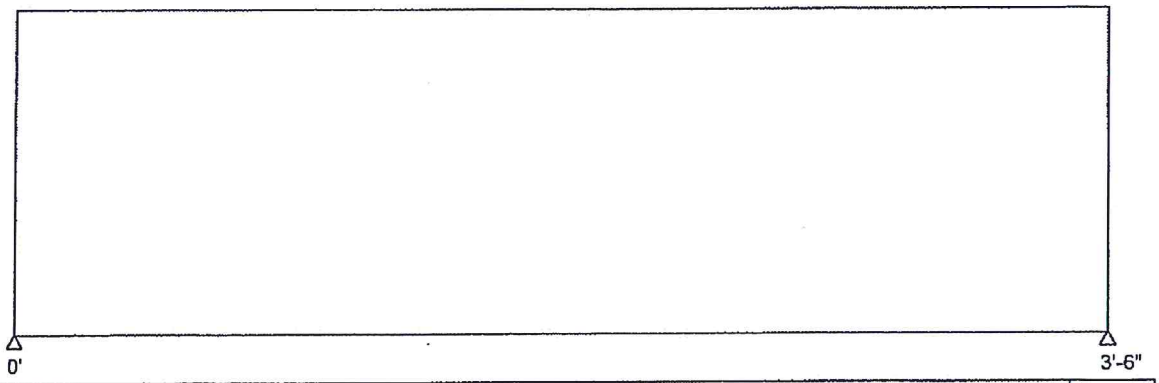
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Full UDL	90.0				No
Load2	Live	Full UDL	300.0				No

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



Dead	161		161
Live	525		525
Total	686		686
Bearing:			
LC number	2		2
Length	1.00		1.00

LSL, 1.55E, 2300Fb, 1-3/4x16"

Self Weight of 2.24 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 9	Fv' = 310	fv/Fv' = 0.03
Bending(+)	fb = 97	Fb' = 1226	fb/Fb' = 0.08
Live Defl'n	0.00 = <L/999	0.12 = L/360	0.01
Total Defl'n	0.00 = <L/999	0.18 = L/240	0.01

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application.
2. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer
3. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY
Seth Davis
Froelich Consulting Engineers
Mar 6, 2012 14:06

PROJECT

2FB6 wwb

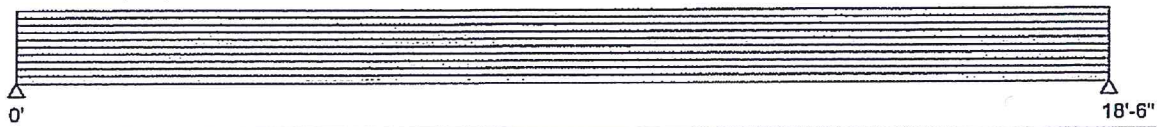
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load3	Dead	Partial UDL	272.0	272.0	0.00	8.50	No
Load2	Live	Partial UDL	40.0	40.0	0.00	8.50	No
Load4	Snow	Partial UDL	300.0	300.0	0.00	8.50	No
Load5	Dead	Partial UDL	24.0	24.0	8.50	18.50	No
Load6	Live	Partial UDL	80.0	80.0	8.50	18.50	No

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



Dead	1958	818
Live	1964	936
Total	3922	1754
Bearing:		
LC number	4	3
Length	1.72	1.00

Glulam-Unbal., West Species, 24F-1.8E WS, 3-1/2x15"

Self Weight of 12.09 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 92	Fv' = 276	fv/Fv' = 0.33
Bending(+)	fb = 1217	Fb' = 1787	fb/Fb' = 0.68
Live Defl'n	0.22 = <L/999	0.62 = L/360	0.35
Total Defl'n	0.43 = L/516	0.92 = L/240	0.46

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application.
2. Glulam design values are for materials conforming to AITC 117-2001 and manufactured in accordance with ANSI/AITC A190.1-1992
3. GLULAM: bxd = actual breadth x actual depth.
4. Glulam Beams shall be laterally supported according to the provisions of NDS Clause 3 3 3
5. GLULAM: bearing length based on smaller of Fcp(tension), Fcp(comp'n)

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Jan. 9, 2013 13:24	2FB7.wwb

Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Full UDL	150.0				No
Load2	Live	Full UDL	500.0				No

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :

Dead	1115		1115
Live	3375		3375
Total	4490		4490
Bearing:			
LC number	2		2
Length	1.26		1.26

Glulam-Unbal., West Species, 24F-1.8E WS, 5-1/2x12"

Self Weight of 15.2 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 87	Fv' = 240	fv/Fv' = 0.36
Bending(+)	fb = 1378	Fb' = 2353	fb/Fb' = 0.59
Live Defl'n	0.26 = L/618	0.45 = L/360	0.58
Total Defl'n	0.35 = L/464	0.67 = L/240	0.52

ADDITIONAL DATA:

FACTORS:	F	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Notes	Cn	LC#
Fb'+	2400	1.00	1.00	1.00	0.980	1.000	1.00	1.00	1.00	1.00	-	2
Fv'	240	1.00	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fcp'	650	-	1.00	1.00	-	-	-	-	1.00	-	-	-
E'	1.8 million		1.00	1.00	-	-	-	-	1.00	-	-	2

```
Bending(+): LC# 2 = D+L, M =  15154 lbs-ft
Shear     : LC# 2 = D+L, V =   4490, V design =   3825 lbs
Deflection: LC# 2 = D+L   EI=   1426e06 lb-in2
Total Deflection = 1.00(Dead Load Deflection) + Live Load Deflection.
  (D=dead  L=live  S=snow  W=wind  I=impact  C=construction  CLd=concentrated)
  (All LC's are listed in the Analysis output)
```

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application.
2. Glulam design values are for materials conforming to AITC 117-2001 and manufactured in accordance with ANSI/AITC A190.1-1992
3. GLULAM: bxd = actual breadth x actual depth
4. Glulam Beams shall be laterally supported according to the provisions of NDS Clause 3.3.3.
5. GLULAM: bearing length based on smaller of Fcp(tension), Fcp(comp'n).

FCE
FROELICH CONSULTING ENGINEERS, INC

6969 SW Hampton Street
Portland, Oregon 97223
FAX 503 624-9770
503 624 7005

745 NW Mt. Washington Drive
Suite 204
Bend, Oregon 97701
FAX 541 383.7696
541 383 1828

www.froelich-engineers.com

CLIENT:

PROJECT:

NUMBER:

DATE:

BY:

2ND FLOOR HEADERS

2FH1

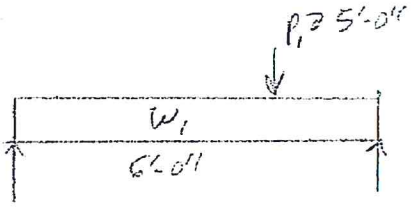
W_1: DL = (7.5')(12) + (8')(10) = 170 PLF

LL = (7.5')(40) = 300 PLF

P_1 (FROM RG2)

DL = 2250#

SL = 3750#



2FH2

DL = (7.5')(12) + (8')(10) + (13')(15) = 365 PLF

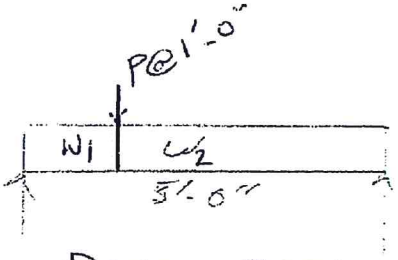
LL = (7.5')(40) = 300 PLF

SL = (13')(25) = 325 PLF

W_2: DL = (7.5')(12) + (8')(10) + (2')(15) = 200 PLF

LL = (7.5')(40) = 300 PLF

SL = (2')(25) = 50 PLF



P (FROM RG3)

DL = 750#

SL = 1250#

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY Seth Davis Froelich Consulting Engineers Feb 15. 2012 10:07	PROJECT 2FH1 wwb

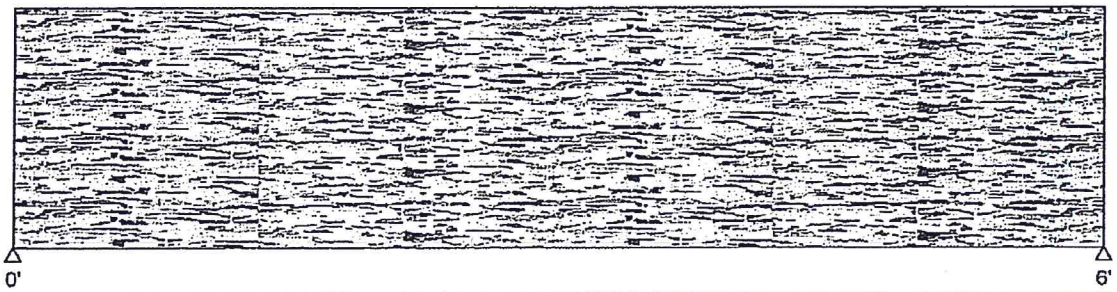
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Full UDL	170.0				No
Load2	Live	Full UDL	300.0				No
Load3	Dead	Point	2250		5.00		No
Load4	Snow	Point	3750		5.00		No

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



Dead	898		2398
Live	1144		3125
Total	2042		5523
Bearing:			
LC number	3		4
Length	1.00		1.97

LSL, 1.55E, 2300Fb, 3-1/2x16"

Self Weight of 4 47 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv* = 108	Fv' = 356	fv*/Fv' = 0.30
Bending(+)	fb = 437	Fb' = 2304	fb/Fb' = 0.19
Live Defl'n	0.01 = <L/999	0.20 = L/360	0.05
Total Defl'n	0.02 = <L/999	0.30 = L/240	0.06

*The effect of point loads within a distance d of the support has been included as per NDS 3.4.3.1

DESIGN NOTES:

1 Please verify that the default deflection limits are appropriate for your application.
2. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only For final member design contact your local SCL manufacturer.
3. Size factors vary from one manufacturer to another for SCL materials They can be changed in the database editor.

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Feb. 8, 2012 14:53	2FH2 wwb

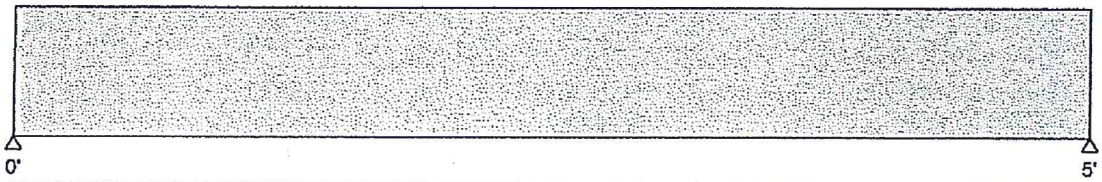
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Partial UDL	365.0	365.0	0.00	1.00	No
Load2	Live	Partial UDL	300.0	300.0	0.00	1.00	No
Load3	Snow	Partial UDL	325.0	325.0	0.00	1.00	No
Load4	Dead	Partial UDL	200.0	200.0	1.00	5.00	No
Load5	Live	Partial UDL	300.0	300.0	1.00	5.00	No
Load6	Snow	Partial UDL	50.0	50.0	1.00	5.00	No
Load7	Dead	Point	750		1.00		No
Load8	Snow	Point	1250		1.00		No

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



Dead	1264		682
Live	1592		864
Total	2855		1546
Bearing:			
LC number	3		3
Length	1.31		1.00

Lumber-soft, D.Fir-L, No.2, 4x8"

Self Weight of 6 03 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 140	Fv' = 207	fv/Fv' = 0.67
Bending(+)	fb = 998	Fb' = 1335	fb/Fb' = 0.75
Live Defl'n	0.04 = <L/999	0.17 = L/360	0.21
Total Defl'n	0.06 = L/936	0.25 = L/240	0.26

ADDITIONAL DATA:

FACTORS:	F	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fb'+	900	1.15	1.00	1.00	0.992	1.300	1.00	1.00	1.00	1.00	-	3
Fv'	180	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	3
Fcp'	625	-	1.00	1.00	-	-	-	-	1.00	1.00	-	-
E'	1.6 million		1.00	1.00	-	-	-	-	1.00	1.00	-	3

```
Bending(+): LC# 3 = D+.75(L+S), M =   2550 lbs-ft
Shear     : LC# 3 = D+.75(L+S), V =   2855, V design =   2362 lbs
Deflection: LC# 3 = D+.75(L+S)   EI=    178e06 lb-in2
Total Deflection = 1.00(Dead Load Deflection) + Live Load Deflection.
  (D=dead  L=live  S=snow  W=wind  I=impact  C=construction  CLd=concentrated)
  (All LC's are listed in the Analysis output)
```

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application.
2. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4 4 1

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Seth Davis Froelich Consulting Engineers Jan 4, 2011 11:38	(1) 2x6 DF#2 Cripple Stud wwc

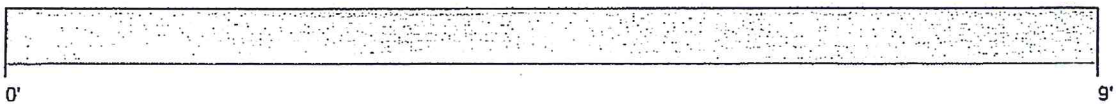
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
live	Live	Axial	7676	(Eccentricity = 0.00 in)			

MAXIMUM REACTIONS (lbs):



Lumber n-ply, D.Fir-L, No.2, 2x6", 1-ply

Self Weight of 1.96 plf automatically included in loads;
Pinned base; Loadface = width(b); Built-up fastener: nails; Ke x Lb: 1.00 x 0 00= 0.00 [ft]; Ke x Ld: 1.00 x 9.00= 9 00 [ft]; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Axial	fc = 933	Fc' = 932	fc/Fc' = 1.00
Axial Bearing	fc = 933	Fc* = 1485	fc/Fc* = 0.63

DESIGN NOTES:

1 Please verify that the default deflection limits are appropriate for your application.
2 BUILT-UP COLUMNS: nailed or bolted built-up columns shall conform to the provisions of NDS Clause 15 3.

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Seth Davis Froelich Consulting Engineers Oct. 26, 2005 10:01	(2) 2x6 DF#2 Cripple Stud.wwc

Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
live	Live	Axial	15352	(Eccentricity = 0.00 in)			

MAXIMUM REACTIONS (lbs):

Lumber n-ply, D.Fir-L, No.2, 2x6", 2-Plys

Self Weight of 3.92 plf automatically included in loads;

Pinned base; Loadface = width(b); Built-up fastener: nails; Ke x Lb: 1.00 x 0.00= 0.00 [ft]; Ke x Ld: 1.00 x 9.00= 9.00 [ft]; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Axial	fc = 933	Fc' = 932	fc/Fc' = 1.00
Axial Bearing	fc = 933	Fc* = 1485	fc/Fc* = 0.63

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application.
2. BUILT-UP COLUMNS: nailed or bolted built-up columns shall conform to the provisions of NDS Clause 15.3

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Seth Davis Froelich Consulting Engineers June 16, 2003 12:22	King Stud 2x6 wwc

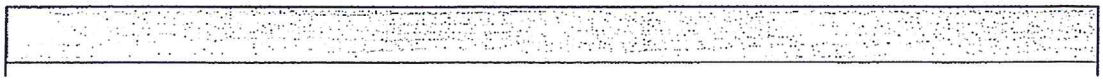
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Wind	Point	450		7.00		No

MAXIMUM REACTIONS (lbs):



0' 9'

Dead			
Live	100		350
Total	100		350

Lumber n-ply, D.Fir-L, Stud, 2x6", 1-ply

Pinned base; Loadface = width(b); Built-up fastener: nails; Ke x Lb: 1.00 x 0.00= 0.00 [ft]; Ke x Ld: 1.00 x 9.00= 9.00 [ft]; Lateral support: top = Lb, bottom = Lb; Load combinations: ICBO-UBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 64	Fv' = 288	fv/Fv' = 0.22
Bending(+)	fb = 1111	Fb' = 1120	fb/Fb' = 0.99
Live Defl'n	0.25 = L/427	0.60 = L/180	0.42
Total Defl'n	0.25 = L/427	0.60 = L/180	0.42

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application
2. BUILT-UP COLUMNS: nailed or bolted built-up columns shall conform to the provisions of NDS Clause 15.3.

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY Seth Davis Froelich Consulting Engineers June 16, 2003 10:53	PROJECT 4x6 Cripple Stud wwc

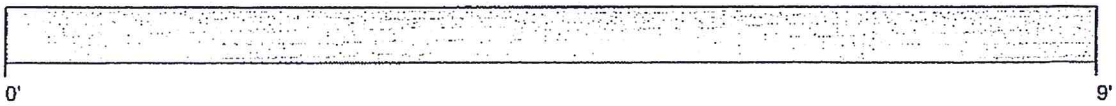
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Axial	7000	(Eccentricity =		0.00	in)
Load2	Snow	Axial	10000	(Eccentricity =		0.00	in)

MAXIMUM REACTIONS (lbs):



Lumber Post, D.Fir-L, No.2, 4x6"

Self Weight of 4.57 plf automatically included in loads;

Pinned base; Loadface = width(b); Ke x Lb: 1.00 x 0.00= 0.00 [ft]; Ke x Ld: 1.00 x 9.00= 9.00 [ft]; Load combinations: ICBO-UBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Axial	fc = 885	Fc' = 982	fc/Fc' = 0.90
Axial Bearing	fc = 885	Fc* = 1708	fc/Fc* = 0.52

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY
Seth Davis
Froelich Consulting Engineers
Sep 12, 2005 06:55

PROJECT

6x6 H F. #2.wwc

Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Axial	3500	(Eccentricity =	0.00		in)
Load2	Live	Axial	4500	(Eccentricity =	0.00		in)

MAXIMUM REACTIONS (lbs):

Timber-soft, Hem-Fir, No.2, 6x6"

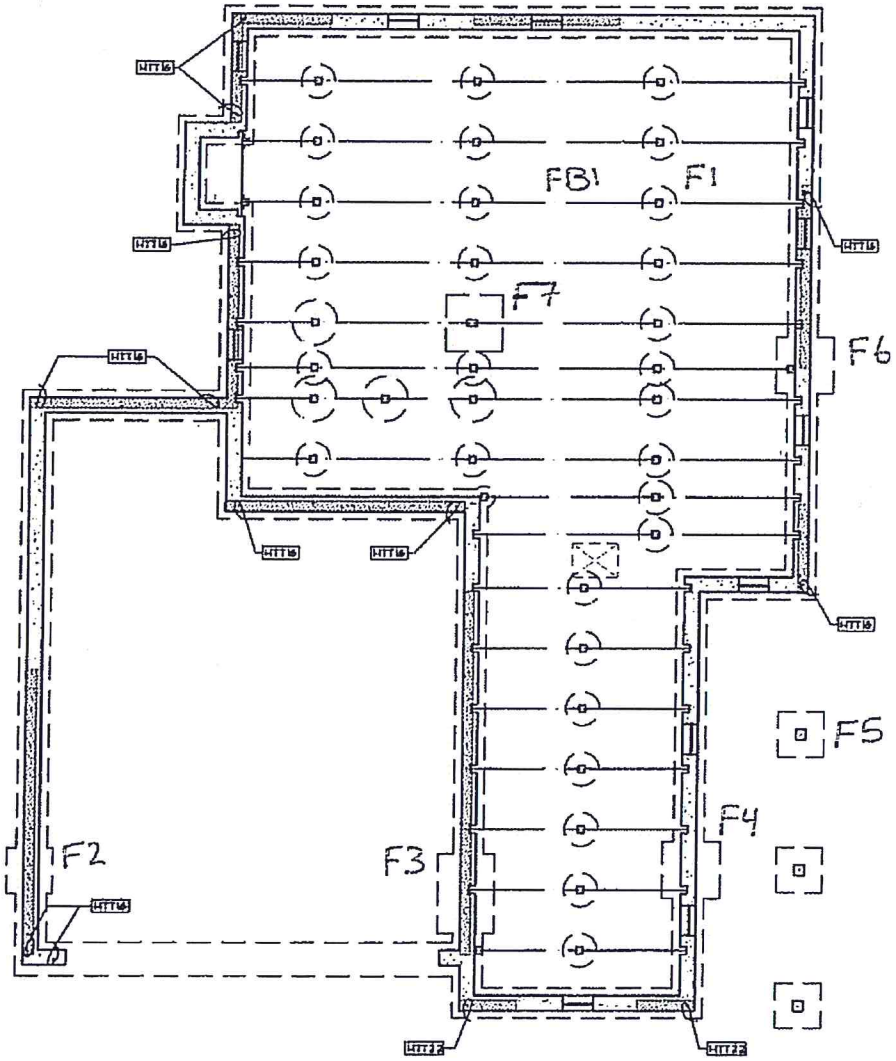
Self Weight of 6 25 plf automatically included in loads;
Pinned base; Loadface = width(b); Ke x Lb: 1 00 x 9 00= 9 00 [ft]; Ke x Ld: 1 00 x 9 00= 9.00 [ft]; Load combinations: ICBO-UBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Axial	fc = 266	Fc' = 465	fc/Fc' = 0.57
Axial Bearing	fc = 266	Fc* = 575	fc/Fc* = 0.46

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application.


HTT16
HTT16
HTT16
HTT16
HTT16
HTT16
HTT16
HTT22
HTT22
FB1
F1
F7
F6
F5
F4
F3
F2

FCE
FROELICH CONSULTING ENGINEERS, INC

6969 SW Hampton Street
Portland, Oregon 97223
FAX 503.684.9770
503 684 7005

745 NW Mt. Washington Drive
Suite 204
Bend, Oregon 97701
FAX 541.383.7696
541 383 1828

www.froelich-engineers.com

CLIENT:

PROJECT:

NUMBER:

DATE:

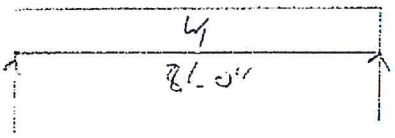
BY:

FIRST FLOOR FRAMING

FB 1

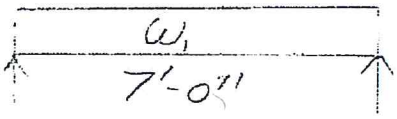
w_1 $DL = (32/12)(12) = 32 PLF$

$LL = (32/12)(40) = 107 PLF$



FB2 $DL = (3')(12) = 36 PLF$

$LL = (3')(100) + (1')(40) = 340 PLF$



WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Seth Davis Froelich Consulting Engineers Feb 8, 2012 15:14	FB1 wwb

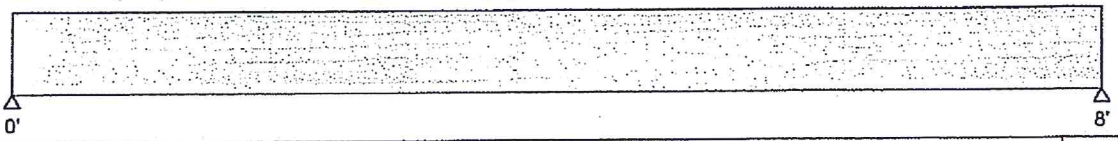
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Full UDL	32.0				No
Load2	Live	Full UDL	107.0				No

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



Dead	152		152
Live	428		428
Total	580		580
Bearing:			
LC number	2		2
Length	1.00		1.00

Lumber-soft, D.Fir-L, No.2, 4x8"

Self Weight of 6.03 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 29	Fv' = 180	fv/Fv' = 0.16
Bending(+)	fb = 454	Fb' = 1158	fb/Fb' = 0.39
Live Defl'n	0.06 = <L/999	0.27 = L/360	0.21
Total Defl'n	0.08 = <L/999	0.40 = L/240	0.19

DESIGN NOTES:

1. Please verify that the default deflection limits are appropriate for your application
2. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4 1.

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY Seth Davis Froelich Consulting Engineers Feb 8, 2012 15:15	PROJECT FB2 wwb

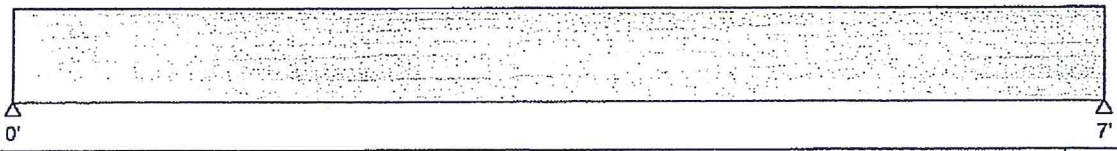
Design Check Calculation Sheet

Sizer 2004a

LOADS (lbs, psf, or plf) :

Load	Type	Distribution	Magnitude Start	Magnitude End	Location [ft] Start	Location [ft] End	Pat-tern
Load1	Dead	Full UDL	36.0				No
Load2	Live	Full UDL	240.0				No

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



Dead	147		147
Live	840		840
Total	987		987
Bearing:			
LC number	2		2
Length	1.00		1.00

Lumber-soft, D.Fir-L, No.2, 4x8"

Self Weight of 6 03 plf automatically included in loads;
Lateral support: top= at supports, bottom= at supports; Load combinations: ICC-IBC;

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2001 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	fv = 48	Fv' = 180	fv/Fv' = 0.27
Bending(+)	fb = 676	Fb' = 1159	fb/Fb' = 0.58
Live Defl'n	0.07 = <L/999	0.23 = L/360	0.31
Total Defl'n	0.09 = L/980	0.35 = L/240	0.24

DESIGN NOTES:

1 Please verify that the default deflection limits are appropriate for your application
2 Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4 4 1

FROELICH
ENGINEERS

Client: Legend Homes
Project:
Proj. #:
Date:
By:

FOOTING DESIGN

Footing		Member(s)	Load		Footing
F1	Supports:	(2) FB1	1200	Use:	18"dia x 8"
F2	Supports:	2FB6	5000	Use:	2'-0"x 2' 0" x 10"
F3	Supports:	2FB1 &2FB3	6500	Use:	2'-6"x2'-6" x 10"
F4	Supports:	2FB3 C	7300	Use:	2'-6"x2'-6" x 10"
F5	Supports:	2FB4	2600	Use:	2'-0"x 2' 0" x 10"
F6	Supports:	2FH1	5600	Use:	2'-6"x2'-6" x 10"
F7	Supports:	FB1, FB2 & Girder	6500	Use:	2'-6"x2'-6" x 10"

FROELICH
ENGINEERS

FROELICH CONSULTING ENGINEERS INC.,

Client: Legend Homes
Project:
Project #:
By:

Footings

Footing Size	Maximum Allowable Load (P_{tot})	q_{allow}	Required Area	Required Dimensions		Used Dimensions			Footing Weight	Bearing Pressure
				W (ft)	L (ft)	W (ft)	L (ft)	D (in)		
18"ϕ x 8"	2450	1500	1.63	1.28	1.28	1.5	1.5	8	177	1486
24"ϕ x 10"	4300	1500	2.87	1.69	1.69	2	2	10	393	1494
2'-0" x 2'-0" x 10"	5400	1500	3.60	1.90	1.90	2	2	10	500	1475
2'-6" x 2'-6" x 10"	8500	1500	5.67	2.38	2.38	2.5	2.5	10	781	1485
3'-0" x 3'-0" x 12"	12000	1500	8.00	2.83	2.83	3	3	12	1350	1483
3'-6" x 3'-6" x 12"	16500	1500	11.00	3.32	3.32	3.5	3.5	12	1838	1497
4'-0" x 4'-0" x 12"	21000	1500	14.00	3.74	3.74	4	4	12	2400	1463

** Indicate maximum load at header supports for continuous footing
*** Indicates maximum pt load for beams on continuous footing

Required Area: $= (P_{tot}/q_{allow})^{0.5}$

Bearing Pressure $= (P_{tot}+W_{ftg})/(W*L)$
$= (P_{tot}+W_{ftg})/(W*L*3.1415/4)$

FOOTING SCHEDULE:		
NO.	SIZE	REINFORCING
1	24" DIA. x 8"	
2	24" x 24" x 10"	(2) #4s EA. WAY
3	30" x 30" x 10"	(2) #4s EA. WAY
4	36" x 36" x 10"	(3) #4s EA. WAY
5	42" x 42" x 10"	(4) #4s EA. WAY

FROELICH
ENGINEERS

Client: Legend Homes
Project:
Proj. #:
Date:
By:

ASCE 7-02 Earthquake Load Cs Factor

Importance Factor (Seismic)
I = 1.0

Basic Seismic Force Resisting System
Light Framed Wood Shear Wall
R = 6.5

Development	Latitude	Longitude	Max Ground Motion		Site Classification	Site Coefficients		Maximum Considered Earthquake Acceleration		Design Spectral Acceleration		Seismic Design Category	Response Coefficient
			S_s	S_1		F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}		Cs
Edgewater	45.406	-122.798	0.914	0.332	D	1.135	1.74	1.037	0.576	0.692	0.384	D	**0.1064**
Maxfield	45.343	-122.664	0.902	0.324	D	1.139	1.75	1.027	0.567	0.685	0.378	D	**0.1054**
Legend at Taralon	45.445	-122.534	0.978	0.329	D	1.109	1.74	1.085	0.573	0.723	0.382	D	**0.1112**
Victoria Gardens	45.367	-122.769	0.904	0.329	D	1.138	1.742	1.029	0.573	0.686	0.382	D	**0.1055**
Village at Orenco	45.584	-122.955	0.956	0.363	D	1.118	1.674	1.069	0.608	0.713	0.405	D	**0.1096**
Legend at Villebois	45.308	-122.780	0.873	0.322	D	1.151	1.756	1.005	0.565	0.670	0.377	D	**0.1031**
Willamette Landing	44.500	-123.250	0.906	0.34	D	1.18	1.86	1.069	0.632	0.622	0.422	D	**0.0957**
Walnut Creek	45.445	-122.798	0.938	0.339	D	1.125	1.723	1.055	0.584	0.704	0.389	D	**0.1082**

- Information in table was found by USGS maps
- Conservatively design all structures in all developments for the Cs design value specified below

Controlling Cs Value = 0.1112

Use Cs = 0.12 for Design in all Developments

Equations:

$S_{MS} = F_a * S_s$	Maximum Considered Earthquake Acceleration
$S_{M1} = F_v * S_1$	
$S_{DS} = S_{MS} * 2/3$	Design Spectral Acceleration
$S_{D1} = S_{M1} * 2/3$	
$Cs = S_{DS} * 1/R$	Response Coefficient

FROELICH
ENGINEERS

Client:	Legend Homes
Project:	Sierra B
Proj. #:	13-T008
Date:	1/8/2013
By:	YSP

Lateral Design

SEISMIC:

R = 6.5 I = 1.0
Site Classification = D
Design Category = D
Seismic Design Coefficient (C_s) = 0.12

Working Stress Design: 0.7E
C_s = 0.084

Seismic Dead Loads

Level	diaph area (ft^2)	Load (psf)	Wall L (ft)	Trib Wall height (ft)	Wall Wt (psf)	Extra DL (lbs)	Total DL (lbs)
Roof	1100	15	155	4.5	10		23475
Upper Flr	1250	12	165	5	10		23250

Seismic Base Shear

$V = Cs(DL_{rf}+DL_{flr})$
V = 3925

Vertical Distribuition

Level	Weight	Height	Wt * Ht	$Wt(Ht)/_{Total}$	V	Vi = (Wt(Ht)/Total)*V	
Roof	23475	18	422550	0.669	3925	**2625**	$= V_{rf}$
Floor	23250	9	209250	0.331	3925	**1300**	$= V_{flr}$
		Total =	631800				

V_{rf} = 2625 lbs

V_{flr} = 1300 lbs

FROELICH
ENGINEERS

Client: Legend Homes
Project: Sierra B
Project #: 13-T008
Date: 1/8/2013
By: YSP

Reliability / Redundancy Factor ρ (per ASCE7-05 12.3.4.2)

Seismic Base Shear (V) = 3925

Percent base shear taken by story:		
Upper Story	2625	**67%**
Main Floor	3925	**100%**

Calculation of # of bays per story resisting more than 35% of the base shear:

Upper Story:

Story height (H) = 9
Critical wall length (L) = 10.5

Bays = 2L/H* = 2.33
*for light-framed construction

The Number of Bays > 2, Therefore, Rho = 1.0

Per Table 12.3-3:

By observation, the removal of any single shear wall with a height-to-width ratio greater than 1.0 does not result in a 33% reduction in story strength for any story resisting more than 35% of the total base shear. The resulting system does not have an extreme torsional irreguarity.

Therefore, ρ = 1.0

FROELICH
ENGINEERS

Client: Legend Homes
Project: Sierra B
Project #: 13-T008
Date: 1/8/2013
By: YSP

Front - Back Event

WIND FORCE CALCULATION - MWFRS
ASCE 7-05 SECTION 6.5
METHOD 2 - ANALYTICAL PROCEDURE

Basic Wind Speeds	**Input**	
Fastest Mile V_{fm} =	80 mph	
3 Second Gust V_{3s} =	95 mph	
Wind Directionality Factor K_d =	0.85	Table 6-4 (page 80)
Wind Importance Factor I_w =	1.00	Table 6-1 (page 77)
Wind Exposure Category =	B	

Building Parameters		
Longitudinal Dimension of Bldg B =	34 ft	
Transverse Dimension of Bldg L =	40 ft	
Mean Roof Height h =	27.5 ft	
Highest Roof Level h_n =	27.5 ft	
Approximate Fundamental Period T_a =	0.24 sec	Eq. 12.8-7 (page 129)

Output - Fundamental Frequency f = 4.2 Hz > 1 Hz Therefore Rigid

Topographic Effects	**Input**	
Hill Height H =	0 ft	Figure 6-4
Length of 1/2 hill height L_h =	0 ft	Figure 6-4
Dist. From Crest to Bldg. x =	0 ft	Figure 6-4
Height Above Local Grade z =	0 ft	Figure 6-4
Horizontal Attenuation Factor m =	1	Figure 6-4
Height Attenuation Factor g =	1	Figure 6-4
Shape Factor K1/(H/Lh) =	1	Figure 6-4

Output - Topographic Multipliers K_1 = 1.00
K_2 = 1.00
K_3 = 1.00
Topographic Factor K_{zt} = 1.00

Gust Effects	**Input**		
Integral Length Scale Factor **l** =	320 ft		Table 6-2
Integral Length Scale			
nominal height of boundary z_g =	1200		Table 6-2
3-s gust exponent α =	7.00		Table 6-2
Turbulence Intensity Factor c =	0.30		Table 6-2
Power Law Exponent **e** =	0.33		Table 6-2
Minimum Height z_{min} =	30 ft		Table 6-2
Integral Length Scale of Turbulence L_z =		310	ft
Output - Background Response Factor Q =		0.90	
Intensity of Turbulence I_z =		0.30	
Gust Effect Factor G =		0.87	

Pressure Coefficients	**Input**			
Length to Width Ratio L/B =		1.18		
Height to Length Ratio h/L =		0.69		
Roof Pitch =	8 : 12	=	33.69	deg
Velocity Pressure Exposure Coefficients K_h	(see below)		Table 6-3 (page 79)	
External Pressure Coefficients C_p	(see below)		Figure 6-6 (page 49)	

Direction	C_p
Windward	0.8
Leeward	-0.5
Roof Windward	0.30
Roof Leeward	-0.6

	Height (ft)	K_h	q_z (psf)	Velocity
	15	0.57	11.3	Pressure
	20	0.62	12.3	Output q_z
	25	0.67	13.1	
	30	0.70	13.8	
	40	0.76	14.9	
	50	0.81	15.9	
	60	0.85	16.8	
	70	0.89	17.5	
	80	0.93	18.2	
	90	0.96	18.8	
	100	0.99	19.4	
	120	1.04	20.4	
h =	27.5	0.68	13.4	q_h

Design Wind Pressures p (psf) - GC_{pi} = (-) 10 psf min per 6.1.4.1

Internal Pressure Coefficient GC_{pi} = -0.18 Figure 6-5 (page 47)

	Direction -	Windward	Leeward	Horizontal Effects Roof WW	Roof LW	Wall WW+LW	Roof Horiz. RWW+RLW
Height	15	10.2	-3.4			13.7	
ft	20	10.9	-3.4			14.3	
	25	11.5	-3.4			14.9	
	30	12.0	-3.4			15.4	
	40	12.8	-3.4			16.2	
	50	13.5	-3.4			16.9	
	60	14.1	-3.4			17.5	
	70	14.6	-3.4			18.0	
	80	15.1	-3.4			18.5	
	90	15.5	-3.4			18.9	
	100	15.9	-3.4			19.3	
	120	16.6	-3.4			20.0	
	27.5	11.7	-3.4	3.3	-2.2	15.1	5.55

Design Load Case 1 Controls - By Inspection Figure 6-9 (page 52)

Design Wind Pressures p (psf) - GC_{pi} = (+) 10 psf min per 6.1.4.1

Internal Pressure Coefficient GC_{pi} = 0.18 Figure 6-5 (page 47)

	Direction -	Windward	Leeward	Horizontal Effects Roof WW	Roof LW	Wall WW+LW	Roof Horiz. RWW+RLW
Height	15	5.4	-8.2			13.7	
ft	20	6.1	-8.2			14.3	
	25	6.6	-8.2			14.9	
	30	7.1	-8.2			15.4	
	40	8.0	-8.2			16.2	
	50	8.6	-8.2			16.9	
	60	9.2	-8.2			17.5	
	70	9.7	-8.2			18.0	
	80	10.2	-8.2			18.5	
	90	10.7	-8.2			18.9	
	100	11.1	-8.2			19.3	
	120	11.8	-8.2			20.0	
	27.5	6.9	-8.2	0.6	-4.9	15.1	5.55

Design Load Case 1 Controls - By Inspection Figure 6-9 (page 52)

FROELICH
ENGINEERS

Client: Legend Homes
Project: Sierra B
Project #: 13-T008
Date: 1/8/2013
By: YSP

Side - Side Event

WIND FORCE CALCULATION - MWFRS

ASCE 7-05 SECTION 6.5
METHOD 2 - ANALYTICAL PROCEDURE

Basic Wind Speeds	**Input**	
Fastest Mile V_{fm} =	80 mph	
3 Second Gust V_{3s} =	94.5 mph	
Wind Directionality Factor K_d =	0.85	Table 6-4 (page 80)
Wind Importance Factor I_w =	1.00	Table 6-1 (page 77)
Wind Exposure Category =	B	

Building Parameters		
Longitudinal Dimension of Bldg B =	40 ft	
Transverse Dimension of Bldg L =	34 ft	
Mean Roof Height h =	25.5 ft	
Highest Roof Level h_n =	27.5 ft	
Approximate Fundamental Period T_a =	0.24 sec	Eq. 12.8-7 (page 129)

Output - Fundamental Frequency f = 4.2 Hz > 1 Hz Therefore Rigid

Topographic Effects	**Input**	
Hill Height H =	0 ft	Figure 6-4
Length of 1/2 hill height L_h =	0 ft	Figure 6-4
Dist. From Crest to Bldg. x =	0 ft	Figure 6-4
Height Above Local Grade z =	0 ft	Figure 6-4
Horizontal Attenuation Factor m =	1	Figure 6-4
Height Attenuation Factor g =	1	Figure 6-4
Shape Factor K1/(H/Lh) =	1	Figure 6-4

Output - Topographic Multipliers K_1 = 1.00
K_2 = 1.00
K_3 = 1.00
Topographic Factor K_{zt} = 1.00

Gust Effects	**Input**		
Integral Length Scale Factor **l** =	320 ft		Table 6-2
Integral Length Scale			
nominal height of boundary z_g =	1200		Table 6-2
3-s gust exponent α =	7.00		Table 6-2
Turbulence Intensity Factor c =	0.30		Table 6-2
Power Law Exponent **e** =	0.33		Table 6-2
Minimum Height z_{min} =	30 ft		Table 6-2
Integral Length Scale of Turbulence L_z =		310	ft
Output - Background Response Factor Q =		0.90	
Intensity of Turbulence I_z =		0.30	
Gust Effect Factor G =		0.87	

Pressure Coefficients	**Input**		
Length to Width Ratio L/B =		0.85	
Height to Length Ratio h/L =		0.75	
Roof Pitch =	5 : 12 =	22.62	deg
Velocity Pressure Exposure Coefficients K_h	(see below)	Table 6-3 (page 79)	
External Pressure Coefficients C_p	(see below)	Figure 6-6 (page 49)	

Direction	C_p
Windward	0.8
Leeward	-0.5
Roof Windward	0.10
Roof Leeward	-0.6

	Height (ft)	K_h	q_z (psf)	Velocity
	15	0.57	11.2	Pressure
	20	0.62	12.1	Output q_z
	25	0.66	12.8	
	30	0.70	13.6	
	40	0.76	14.8	
	50	0.81	15.8	
	60	0.85	16.6	
	70	0.89	17.3	
	80	0.93	18.0	
	90	0.96	18.6	
	100	0.99	19.2	
	120	1.04	20.2	
h =	25.5	0.67	13.0	q_h

Design Wind Pressures p (psf) - GC_{pi} = (-) 10 psf min per 6.1.4.1

Internal Pressure Coefficient GC_{pi} = -0.18 Figure 6-5 (page 47)

	Direction -	Windward	Leeward	Horizontal Effects Roof WW	Roof LW	Wall WW+LW	Roof Horiz. RWW+RLW
Height	15	10.1	-3.3			13.4	
ft	20	10.7	-3.3			14.0	
	25	11.2	-3.3			14.5	
	30	11.8	-3.3			15.1	
	40	12.6	-3.3			15.9	
	50	13.2	-3.3			16.5	
	60	13.8	-3.3			17.1	
	70	14.3	-3.3			17.6	
	80	14.8	-3.3			18.1	
	90	15.2	-3.3			18.5	
	100	15.6	-3.3			18.9	
	120	16.3	-3.3			19.6	
	25.5	11.3	-3.3	1.3	-1.5	14.6	3.85

Design Load Case 1 Controls - By Inspection Figure 6-9 (page 52)

Design Wind Pressures p (psf) - GC_{pi} = (+) 10 psf min per 6.1.4.1

Internal Pressure Coefficient GC_{pi} = 0.18 Figure 6-5 (page 47)

	Direction -	Windward	Leeward	Horizontal Effects Roof WW	Roof LW	Wall WW+LW	Roof Horiz. RWW+RLW
Height	15	5.4	-8.0			13.4	
ft	20	6.1	-8.0			14.0	
	25	6.5	-8.0			14.5	
	30	7.1	-8.0			15.1	
	40	7.9	-8.0			15.9	
	50	8.6	-8.0			16.5	
	60	9.2	-8.0			17.1	
	70	9.7	-8.0			17.6	
	80	10.1	-8.0			18.1	
	90	10.6	-8.0			18.5	
	100	11.0	-8.0			18.9	
	120	11.7	-8.0			19.6	
	25.5	6.7	-8.0	-0.5	-3.3	14.6	3.85

Design Load Case 1 Controls - By Inspection Figure 6-9 (page 52)

FCE
FROELICH CONSULTING ENGINEERS, INC

6969 SW Hampton Street
Portland, Oregon 97223
FAX 503 624.9770
503.624.7005

745 NW Mt. Washington Drive
Suite 204
Bend, Oregon 97701
FAX 541.383.7696
541.383.1828

www.froelich-engineers.com

CLIENT:

PROJECT:

NUMBER:

DATE:

BY:

SIDE - SIDE EVENT

ROOF LINE

1. WIND = $(21\frac{1}{2})(136) = 1428^{\#}$
   EQ = $656^{\#}$
2. WIND $(21\frac{1}{2})(136) + (17\frac{1}{2})(93) = 4219^{\#}$
   EQ = $1312^{\#}$
3. WIND $(17\frac{1}{2})(93) = 791^{\#}$
   EQ = $656^{\#}$

GABLE

WIND. $(4')(14.5) + (2')(3.55) + (5')(14) = 136$ PLF

NO GABLE

WIND $(6')(3.55) + (5')(14) = 93$ PLF

FLOOR LINE

1. WIND. $(23\frac{1}{2})(127) = 1397^{\#}$
   EQ = $325^{\#}$
2. WIND = $(47\frac{1}{2})(127) = 2467^{\#}$
   EQ = $650^{\#}$
3. WIND = $(26\frac{1}{2})(127) = 1270^{\#}$
   EQ = $325^{\#}$

WIND

$(9.5')(13.4) = 127$ PLF

WINDWARD

$(9.5')(10.1) = 96$ PLF

FCE FROELICH CONSULTING ENGINEERS, INC

6969 SW Hampton Street
Portland, Oregon 97223
FAX 503 624.9770
503 624.7005

745 NW Mt. Washington Drive
Suite 204
Bend, Oregon 97701
FAX 541.383 7696
541 383 1828

www.froelich-engineers.com

CLIENT: LEGEND

PROJECT: SIERRA

NUMBER:

DATE:

BY:

LATERAL DISTRIBUTION

FRONT - BACK EVENT

ROOF LINE

A.1 WIND = (54/2)(97) + (4')(135.97) = 1801#

EQ = 2025#/4 = 656#

A.2 WIND = (89.5)(97) = 1649#

EQ = 656#

C WIND = (15')(135) = 2025#

EQ = 2025/3 = 1315#

WIND:

(6)(55.55) + (2')(44.9) + (5')(145)

= 135 PLF

WIND WALL

(6')(33.3) + 2'(11.5 [illegible])

= 97 PLF

FLOOR LINE

A WIND (4')(170) + (6')(97) = 1102#

EQ = 1200#/4 = 300#

A.2 WIND (17)(97) = 1649#

EQ = 825#

B WIND = (17)(97) = 1649

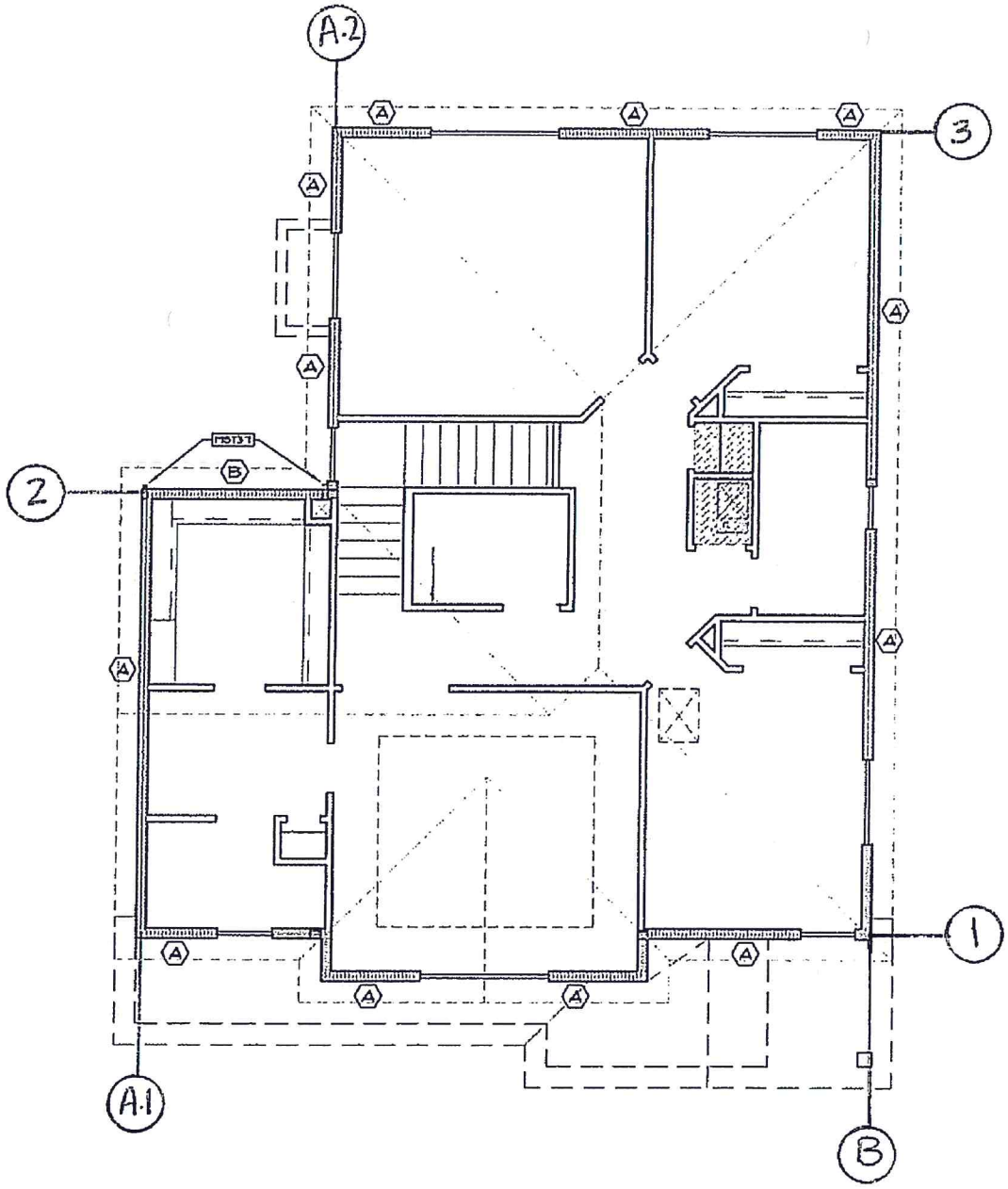
EQ = 550#

C WIND = (7')(97) + (8')(170) = 1459#

EQ = 365#

SIERRA B

1,130 SQ. FT.
34'-0" W x 44'-0" D



FCE FROELICH CONSULTING ENGINEERS, INC

Client:	Legend Homes
Project:	Sierra B
Proj. #:	13-T008
Date:	1/8/2013
By:	YSP

Upper Level Shear Walls and Holdowns

Roof dl:	15	psf
Wall dl:	10	psf
Floor dl:	12	psf

L = Length of individual wall
Lt = Total length of wall along gridline
La = Length of moment arm in wall (if different than wall length)
hu = Height of upper wall
hl = Height of lower wall
Hf = Horizontal force at gridline
Hs = Horizontal force at gridline from upper level
V = Unit shear in wall
Ms = Overturning moment when upper wall is stacked above lower wall
Mu = Overturning moment when upper wall is not stacked or does not exist
Rtrib, Wtrib, Ftrib = Roof, wall, and floor tributary area, used for calculating dead load
Mr = Resisting moment due to dead load
Tu = Tension if walls not stacked
Ts = Tension if walls stacked

$$V = \frac{Hf + Hs}{Lt}$$

$$Ms = [Hs(hl + hu + l) + Hf(hl)] \times \frac{L}{Lt}$$

$$Mu = [Hs(hl) + Hf(hl)] \times \frac{L}{Lt}$$

$$Mr = \frac{2}{3}[(Rtrib \times RoofDL)(Wtrib \times WallDL)(Ftrib \times FloorDL)]\frac{L^2}{2}$$

$$Tu = \frac{Mu - Mr}{La} \qquad Ts = \frac{Ms - Mr}{L}$$

Wall Grid	L (ft)	Lt (ft)	La (ft)	hu (ft)	Hf (lb)	V (plf)	Seismic Factor	Mu (lb*ft)	Rtrib (ft)	Wtrib (ft)	Ftrib (ft)	Mr (lb*ft)	Tu (lb)	Comments	Holdowns	Shearwall	
Front - Back Event																	
A.1	21	21	21	8	1801	86		14408	2	8	0	14553	-7		—	6/12	W
A.2	14	14	14	8	1649	118		13192	Perforated Shear Wall								W
C	10	26	10	8	2025	78		6231	2	8	0	3300	293		—	6/12	W
	16	26	16	8	2025	78		9969	13	8	0	21120	-697		—	6/12	W
Side - Side Event																	
1	3.5	19.5	3.5	8	1428	73		2050	11	8	0	900	329		—	6/12	W
	4.5	19.5	4.5	8	1428	73		2636	2	8	0	668	437		—	6/12	W
	4.5	19.5	4.5	8	1428	73		2636	2	8	0	668	437		—	6/12	W
	7	19.5	7	8	1428	73		4101	11	8	0	3602	71		—	6/12	W
2	8.5	8.5	8.5	9	2219	261		19971	11	8	0	5310	1725		MST37	4/12	W
3	4.5	14.5	4.5	9	791	55		2209	2	8	0	668	342		—	6/12	W
	7	14.5	7	9	791	55		3437	2	8	0	1617	260		—	6/12	W
	3	14.5	3	9	791	55		1473	2	8	0	297	392		—	6/12	W

FCE FROELICH CONSULTING ENGINEERS, INC

Client:	Legend Homes
Project:	Sierra B
Proj. #:	13-T008
Date:	January-13
By:	YSP
Wall:	Upper A.2

Perforated Shear Wall Design

per 2009 IBC 2306.3 & AF&PA SDPWS Section 4.3.3.5

Seismic Control (Y/N)?	n
If Seismic, Sds	0.731

V =	Shear force in Perforated Shear Wall (lbs.)	1649 (lbs.)
L =	Perforated Shear wall Length (feet)	14.00 (feet)
L_1 =	Width of perforated shear wall segment (feet)	5.00 (feet)
L_2 =	Width of perforated shear wall segment (feet)	5.00 (feet)
L_3 =	Width of perforated shear wall segment (feet)	0.00 (feet)
L_4 =	Width of perforated shear wall segment (feet)	0.00 (feet)
H =	Shear wall height (feet)	8 (feet)
h =	Opening height (feet)	2 (feet)
$\sum L_i$ =	Sum of widths of perforated shear wall segments (feet)	
C_o =	Shear resistance adjustment factor from Table 4.3.3.5	
T =	Tension Chord uplift force (lbs.)	
v =	Unit Shear Force (plf)	

Unit Shear in Entire wall V/L = 118 plf — Unit Shear OK

Perforated Shear Wall Height = 8 (feet) — Perforated Shear Wall Height OK

% full Height Shth = $\sum L_i/L$ = 0.71

Max opening Height = h/H = 0.250

From AF&PA SDPWS Table 4.3.3.5 and Interpolating

	Max Opening Height		
% Full Ht. Shth	0.33	0.25	0.33
70	1	1.00	1
71	1	1.00	1.00
80	1	1.00	1

C_o = **1.00**

Eqn. 23-4			**Shear Wall Nailing**
v (plf) =	$V/(Co*\sum L_i)$ =	165	6/12

Eqn. 23-3		
T (lbs.) =	$VH/(Co*\sum L_i)$:	1319

Before Resisting Moment

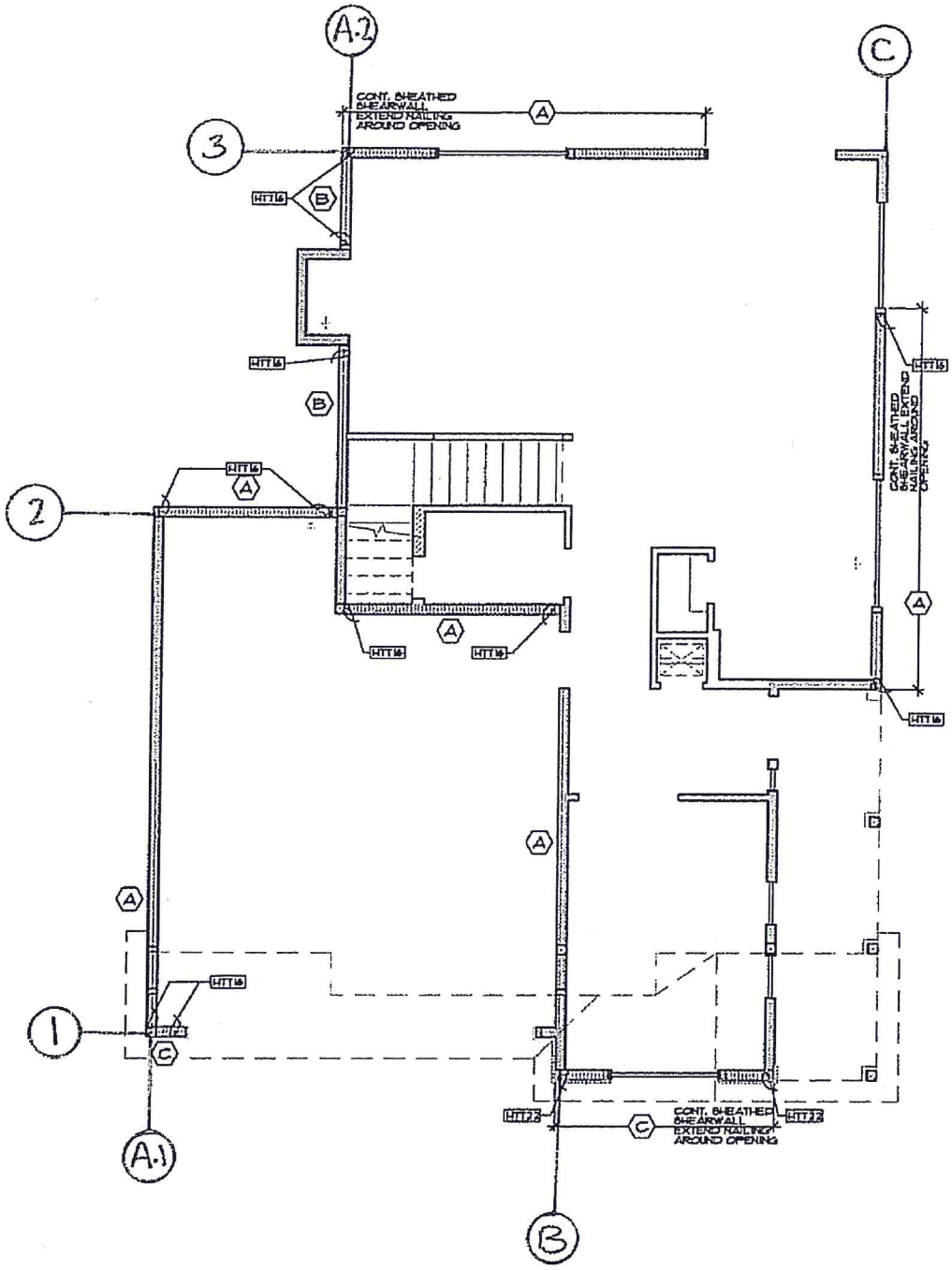
	DL (psf)	Trib (feet)
Roof	15	12
Wall	10	8
Floor	12	0

		Holdown Requirement
=>T (lbs)	227	Not Req'd

SIERRA B

812 SQ. FT.

34'-0" W x 44'-0" D



FROELICH ENGINEERS

Client:	Legend Homes
Project:	Sierra B
Proj. #:	13-T008
Date:	1/8/2013
By:	YSP

Main Level Shear Walls and Holdowns

Roof dl:	15	psf
Wall dl:	10	psf
Floor dl:	12	psf

L = Length of individual wall
Lt = Total length of wall along gridline
La = Length of moment arm in wall (if different than wall length)
hu = Height of upper wall
hl = Height of lower wall
Hf = Horizontal force at gridline
Hs = Horizontal force at gridline from upper level
V = Unit shear in wall
Ms = Overturning moment when upper wall is stacked above lower wall
Mu = Overturning moment when upper wall is not stacked or does not exist
Rtrib, Wtrib, Ftrib = Roof, wall, and floor tributary area, used for calculating dead load
Mr = Resisting moment due to dead load
Tu = Tension if walls not stacked
Ts = Tension if walls stacked

$$v = \frac{Hf + Hs}{Lt}$$

$$Ms = [Hs(hl + hu + 1) + Hf(hl)] \times \frac{L}{Lt}$$

$$Mu = [Hs(hl) + Hf(hl)] \times \frac{L}{Lt}$$

$$Mr = \frac{2}{3}\left[(Rtrib \times RoofDL)(Wtrib \times WallDL)(Ftrib \times FloorDL)\right]\frac{L^2}{2}$$

$$Tu = \frac{Mu - Mr}{La} \qquad Ts = \frac{Ms - Mr}{L}$$

Wall Grid	L (ft)	Lt (ft)	La (ft)	hu (ft)	hl (lb)	Hf (lb)	Hs (lb)	V (plf)	Seismic Factor	Ms (lb*ft)	Mu (lb*ft)	Rtrib (ft)	Wtrib (ft)	Ftrib (ft)	Mr (lb*ft)	Tu (lb)	Ts (lb)	Comments	Holdowns	Shearwall	
Front - Back Event																					
A.1	13	13	12.5	8	9	1102	1801	223		42336	26127	2	17	10	18027	648	---		---	6/12	W
A.2	7.5	12.5	7	8	9	1649	1649	264		26714	17809	13	17	6	8194	1374	---		HTT16	4/12	W
	5	12.5	4.5	8	9	1649	1649	264		17809	11873	13	17	6	3642	1829	---		HTT16	4/12	W
EQ	3	12.5	2.5	8	9		1649	198	1.50											6/12	
B	10	10	9.5	8	9	1649	0	165		14841	14841	0	9	17	9800	531	---		---	6/12	W
C	4	12	3.5	8	9	1459	2025	290		16527	10452	13	17	7	2395	2302	---	Perforated shear wall			W
	8	12	7.5	8	9	1459	2025	290		33054	20904	13	17	7	9579	1510	---				W
Side - Side Event																					
1	2	7.2	1.5	8	6.5	1397	1428	392		8671	5101	2	6.5	0	127	3316	---		HTT16	3/12	W
	2.6	7.2	2.1	8	9	1397	1428	392		13822	9181	2	9	0	270	4243	---	Perforated shear wall			W
	2.6	7.2	2.1	8	9	1397	1428	392		13822	9181	2	9	0	270	4243	---				W
EQ	1.875	7.2	1.375	8	6.5	325	656	236	1.73											6/12	EQ
	2	7.2	1.5	8	9	325	656	307	2.25											4/12	EQ
2	8.5	19	8	8	9	2667	2219	257		28607	19673	0	9	2	2746	2116	3233		HTT16	6/12	W
	10.5	19	10	8	9	2667	2219	257		35338	24301	0	9	2	4190	2011	---		HTT16	6/12	W
3	4.5	11	4	8	9	1270	791	187		10501	7588	2	18	2	1580	1502	2230	Perforated shear wall			W
	6.5	11	6	8	9	1270	791	187		15167	10961	2	18	2	3296	1278	1979				W

FCE FROELICH CONSULTING ENGINEERS, INC

Client:	Legend homes
Project:	Sierra B
Proj. #:	13-T008
Date:	January-13
By:	YSP
Wall:	Line C Main Floor

Perforated Shear Wall Design

per 2009 IBC 2306.3 & AF&PA SDPWS Section 4.3.3.5

Seismic Control (Y/N)?	y
If Seismic, Sds	0.731

V =	Shear force in Perforated Shear Wall (lbs.)	3625 (lbs.)
L =	Perforated Shear wall Length (feet)	18.00 (feet)
L_1 =	Width of perforated shear wall segment (feet)	8.00 (feet)
L_2 =	Width of perforated shear wall segment (feet)	4.00 (feet)
L_3 =	Width of perforated shear wall segment (feet)	0.00 (feet)
L_4 =	Width of perforated shear wall segment (feet)	0.00 (feet)
H =	Shear wall height (feet)	9 (feet)
h =	Opening height (feet)	2 (feet)
$\sum L_i$ =	Sum of widths of perforated shear wall segments (feet)	
C_o =	Shear resistance adjustment factor from Table 4.3.3.5	
T =	Tension Chord uplift force (lbs.)	
v =	Unit Shear Force (plf)	

Unit Shear in Entire wall V/L =		201 plf	Unit Shear OK
Perforated Shear Wall Height =		9 (feet)	Perforated Shear Wall Height OK
% full Height Shth =	$\sum L_i/L$ =	0.67	
Max opening Height =	h/H =	0 222	

From AF&PA SDPWS Table 4.3.3.5 and Interpolating

	Max Opening Height		
% Full Ht. Shth	0.33	0.22	0.33
60	1	1.00	1
67	1.000	1.00	1.00
70	1	1.00	1

C_o =	1.00

Eqn. 23-4		**Shear Wall Nailing**
v (plf) =	$V/(Co*\sum L_i)$ = 302	4/12

Eqn. 23-3	
T (lbs.) =	$VH/(Co*\sum L_i)$ = 2719

Before Resisting Moment

	DL (psf)	Trib (feet)
Roof	15	6
Wall	10	18
Floor	12	9

		Holdown Requirement
=>T (lbs)	1026	HTT16

FCE FROELICH CONSULTING ENGINEERS, INC

Client:	Legend homes
Project:	Sierra B
Proj. #:	13-T008
Date:	January-13
By:	YSP
Wall:	Line 1 Main Floor

Perforated Shear Wall Design

per 2009 IBC 2306.3 & AF&PA SDPWS Section 4.3.3.5

Seismic Control (Y/N)?	n
If Seismic, Sds	0.731

V =	Shear force in Perforated Shear Wall (lbs.)	2040 (lbs.)
L =	Perforated Shear wall Length (feet)	10.25 (feet)
L_1 =	Width of perforated shear wall segment (feet)	2.60 (feet)
L_2 =	Width of perforated shear wall segment (feet)	2.60 (feet)
L_3 =	Width of perforated shear wall segment (feet)	0.00 (feet)
L_4 =	Width of perforated shear wall segment (feet)	0.00 (feet)
H =	Shear wall height (feet)	9 (feet)
h =	Opening height (feet)	6 (feet)
$\sum L_i$ =	Sum of widths of perforated shear wall segments (feet)	
C_o =	Shear resistance adjustment factor from Table 4.3.3.5	
T =	Tension Chord uplift force (lbs.)	
v =	Unit Shear Force (plf)	

Unit Shear in Entire wall V/L =		199 plf	Unit Shear OK
Perforated Shear Wall Height =		9 (feet)	Perforated Shear Wall Height OK
% full Height Shth =	$\sum L_i/L$ =	0.51	
Max opening Height =	h/H =	0.667	

From AF&PA SDPWS Table 4.3.3.5 and Interpolating

	Max Opening Height		
% Full Ht. Shth	0.66	0.67	0.83
50	0.67	0.67	0.57
51	0.673	0.67	0.57
60	0.71	0.71	0.63

C_o =	0.67

Eqn. 23-4		
v (plf) =	$V/(Co*\sum L_i)$ =	586

Shear Wall Nailing
3/12

Eqn. 23-3		
T (lbs.) =	$VH/(Co*\sum L_i)$ =	5277

Before Resisting Moment

	DL (psf)	Trib (feet)
Roof	15	6
Wall	10	18
Floor	12	9

=>T (lbs) 4115

Holdown Requirement
HTT22

FCE FROELICH CONSULTING ENGINEERS, INC

Client:	Legend homes
Project:	Sierra B
Proj. #:	13-T008
Date:	January-13
By:	YSP
Wall:	Line 3 Main Floor

Perforated Shear Wall Design

per 2009 IBC 2306.3 & AF&PA SDPWS Section 4.3.3.5

Seismic Control (Y/N)?	n
If Seismic, Sds	0.731

V =	Shear force in Perforated Shear Wall (lbs.)	2060 (lbs.)
L =	Perforated Shear wall Length (feet)	17.00 (feet)
L_1 =	Width of perforated shear wall segment (feet)	4.50 (feet)
L_2 =	Width of perforated shear wall segment (feet)	6.50 (feet)
L_3 =	Width of perforated shear wall segment (feet)	0.00 (feet)
L_4 =	Width of perforated shear wall segment (feet)	0.00 (feet)
H =	Shear wall height (feet)	9 (feet)
h =	Opening height (feet)	5 (feet)
$\sum L_i$ =	Sum of widths of perforated shear wall segments (feet)	
C_o =	Shear resistance adjustment factor from Table 4.3.3.5	
T =	Tension Chord uplift force (lbs.)	
v =	Unit Shear Force (plf)	

Unit Shear in Entire wall V/L = 121 plf — Unit Shear OK

Perforated Shear Wall Height = 9 (feet) — Perforated Shear Wall Height OK

% full Height Shth = $\sum L_i/L$ = 0.65

Max opening Height = h/H = 0.556

From AF&PA SDPWS Table 4.3.3.5 and Interpolating

	Max Opening Height		
% Full Ht. Shth	0.5	0.56	0.66
60	0.83	0.79	0.71
65	0.849	0.81	0.74
70	0.87	0.84	0.77

C_o = **0.81**

Eqn. 23-4		**Shear Wall Nailing**
v (plf) =	$V/(Co*\sum L_i)$ = 231	6/12

Eqn. 23-3	
T (lbs.) =	$VH/(Co*\sum L_i)$ = 2080

Before Resisting Moment

	DL (psf)	Trib (feet)
Roof	15	6
Wall	10	18
Floor	12	3

		Holdown Requirement
=>T (lbs)	519	Not Req'd

SHEARWALL SCHEDULE

⬡	WALL INFORMATION	FASTENERS			
		SILL PLATE TO RIM	RIM OR BLKG TO DBL. TOP PLATE	MUD SILL/A.B.'s	COMMENTS
A	1/2" APA RATED SHEATHING (1) SIDE w/ 8d NAILS @ 6"o.c. EDGES & 12"o.c. FIELD	16d COMMON @ 6"o.c. INTO SOLID FRAMING	16d COMMON @ 8"o.c. (TOENAIL) & SIMP. LTP4 @ 48"oc w/ 8d COMMON	LAP SHT'G ONTO 2x MUDSILL. 1/2" DIA. x 10" A.B. @ 48" o.c. (EMBED 7")	
B	1/2" APA RATED SHEATHING (1) SIDE w/ 8d NAILS @ 4"o.c. EDGES & 12"o.c. FIELD	16d COMMON @ 4"o.c. INTO SOLID FRAMING	SIMPSON LTP4 @ 18"oc w/ 8d COMMON	LAP SHT'G ONTO 2x MUDSILL. 1/2" DIA. x 10" A.B. @ 36" o.c. (EMBED 7")	SEE NOTES #9, #11 & #12 BELOW
C	1/2" APA RATED SHEATHING (1) SIDE w/ 8d NAILS @ 3"o.c EDGES & 12"o.c. FIELD	SIMPSON LTP4 @ 12"o.c. INTO SOLID FRAMING	SIMPSON LTP4 @ 12"oc w/ 8d COMMON	LAP SHT'G ONTO 2x MUDSILL. 1/2" DIA. x 10" A.B. @ 24" o.c. (EMBED 7")	SEE NOTES #9, #11, & #12 BELOW
D	1/2" APA RATED SHEATHING (2) SIDES w/ 8d NAILS @ 4"o.c. EDGES & 12"o.c. FIELD	SIMPSON LTP4 @ 10"o.c. INTO SOLID FRAMING	SIMPSON LTP4 @ 9"oc w/ 8d COMMON	LAP SHT'G ONTO 2x MUDSILL. 1/2" DIA. x 10" A.B. @ 18" o.c. (EMBED 7")	SEE NOTES #10, #11, #13, #14 #15 & #16 BELOW
E	1/2" APA RATED SHEATHING (2) SIDES w/ 8d NAILS @ 3"o.c. EDGES & 12"o.c. FIELD	SIMPSON LTP4 @ 6"o.c. INTO SOLID FRAMING	SIMPSON LTP4 @ 6"oc w/ 8d COMMON	LAP SHT'G ONTO 2x MUDSILL. 1/2" DIA. x 10" A.B. 12" o.c. (EMBED 7")	SEE NOTES #10, #11, #13, #14 #15 & #16 BELOW

NOTES:
1. IF ANCHOR BOLT SPACING IS GREATER THAN SHEARWALL LENGTH, INSTALL (1) A.B. WITHIN 12" OF EA. END
2. SHEARWALL FRAMING TO BE 16"oc U.N.O. OR DETAILED OTHERWISE.
3. SHEARWALLS TO BE BLOCKED AT ALL PANEL EDGES U.N.O. OR DETAILED OTHERWISE.
4. ALL NAILS TO BE COMMON NAILS U.N.O. 8d GALVANIZED BOX NAILS MAY BE SUBSTITUTED FOR 8d COMMONS.
5. LTP4 PLATES SHALL BE INSTALLED w/ (12) 8d COMMONS.
6. ONLY THE MUD SILL IN CONTACT w/ CONCRETE SHALL BE P.T.
7. 3"x 3"x 1/4" PLATE WASHERS ARE TO BE USED AT ALL SHEARWALL ANCHOR BOLTS.
8. ALL HOLDOWNS, STRAPS & CONNECTORS TO BE SIMPSON OR EQUAL.

SHEARWALL SPECIFIC NOTES:
9. FRAMING AT ADJOINING PANEL EDGES SHALL BE 3" NOMINAL OR GREATER & NAILS SHALL BE STAGGERED. (DBL 2x ARE ACCEPTABLE)
10. FRAMING AT ADJOINING PANEL EDGES, SOLE PLATE & SILL PLATE SHALL BE 3" NOMINAL OR GREATER & NAILS SHALL BE STAGGERED. (DBL 2x ARE ACCEPTABLE)
11. SIMPSON LTP4 CLIPS @ SOLE PLATE CONNECTION TO RIM JOIST ARE NOT REQUIRED IF UPPER WALL SHEATHING IS CONTINUOUS & DIRECTLY EDGE NAILED TO RIM JOIST.
12. SIMPSON LTP4 CLIPS @ RIM JOIST CONNECTION TO DBL. TOP PLATE ARE NOT REQUIRED IF LOWER WALL SHEATHING IS CONTINUOUS & DIRECTLY EDGE NAILED TO RIM JOIST.
13. SIMPSON LTP4 CLIPS @ RIM JOIST CONNECTION TO DBL. TOP PLATE SHALL BE PLACED @ 12"oc IF LOWER WALL SHEATHING IS CONTINUOUS & DIRECTLY EDGE NAILED TO RIM JOIST.
14. 4 1/2"x 4 1/2"x 1/4" PLATE WASHERS ARE TO BE USED AT ALL ANCHOR BOLTS SHEARWALL TYPES "D" & "E".
15. AT GARAGE PIERS USE 3x MUDSILLS.
16. AT INTERIOR SHEARWALLS USE 3x SOLE PLATES (DBL. 2x ARE ACCEPTABLE, STAGGER NAILING). SHEARWALL 2011

HOLDOWN SCHEDULE

HOLDOWN	ATTACHMENT	FRAMING MEMBER SIZE (MIN)	COMMENTS
SIMP MST37	(11)-16d NAILS at TOP (11)-16d NAILS at BOTTOM CENTER STRAP ON FLOOR CAVITY. (22) TOTAL NAILS.	(2) 2x STUDS	WRAP & NAIL STRAP TO BEAM/HEADER BELOW IF APPLICABLE.
USP. HTT16	USP. STB24 INTO STEMWALL & (18)-16d NAILS TO DBL. STUDS.	(2) 2x STUDS	PROVIDE #4 DOWEL w/ STANDARD HOOK @ EA. HOLDOWN LOCATION.
USP HTT22	USP. STB24 INTO STEMWALL & (32)-16d SINKER NAILS	(2) 2x STUDS	PROVIDE #4 DOWEL w/ STANDARD HOOK @ EA. HOLDOWN LOCATION.

NOTES:
1. MULTIPLE STUDS SHALL BE LAMINATED TOGETHER WITH (2) ROWS 16d NAILS at 10"oc FULL HEIGHT (TYPICAL)
2. AT STUDS BELOW MST OR MSTC STRAPS, EDGE NAIL PLYWOOD TO STUDS.
3. ALL HOLDOWNS, STRAPS & CONNECTORS TO BE SIMPSON OR EQUAL. INSTALL PER MANUF. INSTRUCTIONS.
4. ADD 1" OFFSET FOR DOOR LOCATIONS.

FOOTING SCHEDULE:

NO.	SIZE	REINFORCING
1	24" DIA. x 8"	
2	24" x 24" x 15"	(2) #4s EA. WAY
3	36" x 36" x 15"	(2) #4s EA. WAY
4	36" x 36" x 15"	(3) #4s EA. WAY
5	42" x 42" x 15"	(4) #4s EA. WAY

FROELICH
ENGINEERS!

Client: Legend Homes
Project:
Proj. #:
Date:
By:

ACI 318-05 Appendix D - Shear Failures (Page 1 of 3)

Anchor description: **ASTM A36 Hex Head**

	1	**Number of Anchors**	$s_1 = 0$	in. (see Fig.
	0.625	**Inch Diameter**	$s_2 = 0$	RD.5.2.1)
	8	**Inch Embed**		
	2500	**psi Concrete Footing**		
V_{Design} =	**5**	**(kips) Allowable Design Shear**		

D.3 - General Requirements (ACI 318-05 Section D.3.3.3)

Are seismic loads induced into the anchor? N S_F = 1

D.4 - General Requirements for Anchor Strength (ACI 318-05 Section D.4.4)

Strength reduction factor Φ for anchors using load combinations from ACI 318-05 section 9.2

Will anchor be governed by brittle steel failure? N Anchor Φ = 0.65

Brittle failure: 0.60 *(brittle defined by tensile test elongation less than 14%)*

Ductile failure: 0.65

Is rebar present around anchor to resist blowout? N Reinforcing Φ = 0.70

If rebar is present around anchor: 0.75

Otherwise, 0.70

Summary

Summary From Below	ΦV_n	$\Phi V_{n\ Wind}$ $S_W = 1.0$	$\Phi V_{n\ Seismic}$ $S_F = 0.75$	
ΦV_s =	5.11	5.11	5.11	kips
ΦV_{cbg} =	24.56	24.56	24.56	kips
ΦV_{cp} =	38.01	38.01	38.01	kips
Minimum ΦV_n =	5.11	5.11	5.11	kips

Converting To Allowable Stress Design

	Wind	**Seismic**	
Conversion Factor	1.0	1.4	
$\Phi V_{Allowable}$ =	**N/A**	**N/A**	**kips**

V_{design}	<	$\Phi V_{Allowable}$
5.000	<	**5.11**

Therefore, Anchor Design OK

FROELICH
ENGINEERS

Client: Legend Homes
Project:
Proj. #:
Date:
By:

ACI 318-05 Appendix D - Shear Failures Cont. (Page 2 of 3)

Shear Design Calculations

D.6.1 - Steel Strength for Anchor in Shear

d_o (Anchor Diameter) =	0.625	inches
n =	1	# of anchors
n_t =	11	Number of Threads per inch
A_{se} =	0.23	in.^{2}- (effective cross-sectional area of anchor)
f_{ut} =	58.00	ksi - (tensile strength of anchor material (not the yield strength) not exceed 1.9f_y or 125 ksi)
Will anchor be used with a built-up grout pad?	N	(ACI 318-02 D.6.1.3)
V_s =	7.86	ksi - (Eqn. D-19)
Anchor Φ =	0.65	
ΦV_s =	**5.11**	**kips**

$$V_{sa} = n0.6A_{se}f_{uta}$$

D.6.2 - Concrete Breakout Strength of Anchor in Shear

c_{a1} =	**15**	inches (see Fig. RD.6.2.1)
c_{a2} =	**15**	inches (see Fig RD.6.2.1)
A_{Vc} =	**1012.5**	in.^{2} (see Figure RD.6.2.1)
A_{Vco} =	1012.5	in.^{2} (Eqn. D-23)
$\Psi_{ec,V}$ =	**1**	Eqn. D-26 (Anchors not Eccentrically Loaded, $\Psi_{ec,V}$ = 1.0)
Ψ_{ed} =	0.9	Eqn. D-27 & D-28
$\Psi_{c,V}$ =	**1.4**	Section D.6.2.7
f'c =	2500	psi
l_e =	5	inches
V_b =	27.85	kips - (Eqn. D-25)
V_{cb} =	35.09	kips - (Eqn. D-21)
V_{cbg} =	35.09	kips - (Eqn. D-22)
Reinforcing Φ =	0.70	
ΦV_{cbg} =	**24.56**	**kips**

$$V_b = 8\left(\frac{l_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}(c_{a1})^{1.5}$$

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}V_b$$

FROELICH
ENGINEERS

Client: Legend Homes
Project:
Proj. #:
Date:
By:

ACI 318-05 Appendix D - Shear Failures Cont. (Page 3 of 3)

D.6.3 - Concrete Pryout Strength of Anchor in Shear

N_{cb} =	27.15	kips - (Eqn. D-4)
h_{ef} =	8.00	inches
k_{cp} =	2.0	Section D.6.3.1
V_{cp} =	54.31	kips - (Eqn. D-28)
Reinforcing Φ =	0.70	
ΦV_{cp} =	**38.01**	**kips**

$$V_{cp} = k_{cp} N_{cb}$$