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Structural and Civil Engineering Consultants
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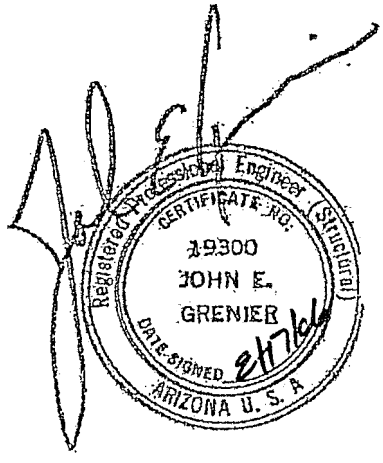
Structural Calculations For Arizona National Guard Silverbell Fire Station No. 361 Marana, Arizona



PREPARED FOR: **Hunt & Caraway Architects, Ltd.**
71450 North 57Th Drive
Glendale, Arizona 85301

PREPARED BY: **Grenier Engineering, Inc.**
4515 South McClintock Drive, Suite 211
Tempe, Arizona 85282

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PROJECT: Silverbell N.G.F.S.
JOB NO.: 06202 DATE: 8/06
BY: AE

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Registered Professional Engineer (Structural)
CERTIFICATE NO. 19300
JOHN E. GRENIER
DATE SIGNED 8/17/06
ARIZONA U. S. A.

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STRUCTURAL DESIGN CRITERIA

DESCRIPTION OF STRUCTURE:

The building superstructure consists of a single story fire station with the following materials of construction: wood I-joist and wood chord/steel web roof systems, concrete masonry walls and light gage steel stud walls; all bearing on a convention foundation. Grenier Engineering, Inc. has been retained to perform the framing and foundation designs and perform a lateral analysis only for this project.

DESIGN CODES: INTERNATIONAL BUILDING CODE (I.B.C.) 2003 EDITION

DESIGN LOADS SUMMARY:

GRAVITY:	Roof Dead Load	= 17 PSF
	Roof Live Load	= 20 PSF

WIND: Basic Wind Speed = 90 mph (3 Second Gust), Exposure C, I = 1.15

SEISMIC: See Page 4

SOIL: Allowable soil bearing pressure of 2000 PSF at 1'-6" below lowest adjacent finished grade per soils report No. 26557 dated August 8, 2006 performed by LMT Engineering, Inc.

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PROJECT:	National Guard F.S.	SHEET NO:	3
JOB NO:	06202	DATE:	Aug-06
		BY:	AE

WOOD ROOF DEAD LOADS

Built-Up Roof	=	4	psf
5/8" Plywood	=	2	psf
Wood Joist	=	3.5	psf
Insulation	=	2.7	psf
Ceiling - ½ Gyp Board	=	2.2	psf
M.P.E.	=	1.5	psf
MISC.	=	1.1	psf
DL	**=**	**17**	**psf**
LL (Reducible)	**=**	**20**	**psf**

EXTERIOR CMU WALL LOADS:

8" CMU	=	66	psf
		66	**psf**
12" CMU	=	103	psf
		103	**psf**

INTERIOR WALL LOADS:

6" Studs at 16" o.c.	=	1.5	psf
Insulation	=	1.8	psf
1/2" Gyp. Board	=	4.4	psf
Misc.	=	2.3	psf
		10	**psf**

WALL DESIGN

IBC 2003

Wind: 90 MPH, Exposure C, I = 1.15

Height (feet)	Coeff.	q (psf)	Load (psf)
0' 0" to 15'-0"	1.21	15.1	21.0
15' 1" to 20'-0"	1.29	15.1	22.4
20' 1" to 25'-0"	1.35	15.1	23.4

Seismic:

$0.7 \times F_P = 0.40 \times I_e \times S_{DS} \times W_W =$ **0.118 W**

$I_e =$ 1.25

$S_{DS} =$ 0.338

LATERAL DESIGN

Wind: 90 MPH, Exposure C,

Flat Roof I = 1.15

Height (feet)	Coeff.	q (psf)	Load (psf)
0' 0" to 15'-0"	1.21	12.8	17.8
15' 1" to 20'-0"	1.29	12.8	19.0
20' 1" to 25'-0"	1.35	12.8	19.9

3:12 Pitch Roof I = 1.15

Height (feet)	Coeff.	q (psf)	Load (psf)
0' 0" to 15'-0"	1.21	16.1	22.4
15' 1" to 20'-0"	1.29	16.1	23.9
20' 1" to 25'-0"	1.35	16.1	25.0

Grenier Structural Engineering, Inc.
Tempe * Tucson * Yuma

PROJECT:	Silverbell National Guard F.S.	SHEET NO:	4
JOB NO:	06202	DATE:	Aug-06
		BY:	AE

2003 IBC SEISMIC DESIGN

S_{MS} = FaSs =	0.507	S_{DS} =	0.338	SEISMIC DESIGN CATEGORY =	C
S_{M1} = FvS1 =	0.211	S_{D1} =	0.141	SEISMIC DESIGN CATEGORY =	C
Seismic use Group	II	I_E =	1.250	**SEISMIC DESIGN CATEGORY =**	**C**
R=	3.5	ρ =	1		
Building Height (ft)	26	Cs=	0.121	(ASCE 7-02 Eq 9.5.5.2.1-1)	
C_T =	0.02	Cs=	0.218	(ASCE 7-02 Eq 9.5.5.2.1-2)	
T=	0.23 sec	Cs=	0.019	(ASCE 7-02 Eq 9.5.5.2.1-3	
		EQ 9.5.5.2.1-1 controls			
E =	ρ Qe + 0.2S_{DS} D	Qe =	Cs W	;(W=D)	

E= 0.121 + 0.2SDS D
.7E= 0.085 + 0.2SDS D

Silverbell National Guard Fire Station

5

MCE Parameters - Conterminous 48 States
Zip Code - 85653 Central Latitude = 32.421721
Central Longitude = -111.391974
Data are based on the 0.01 deg grid set

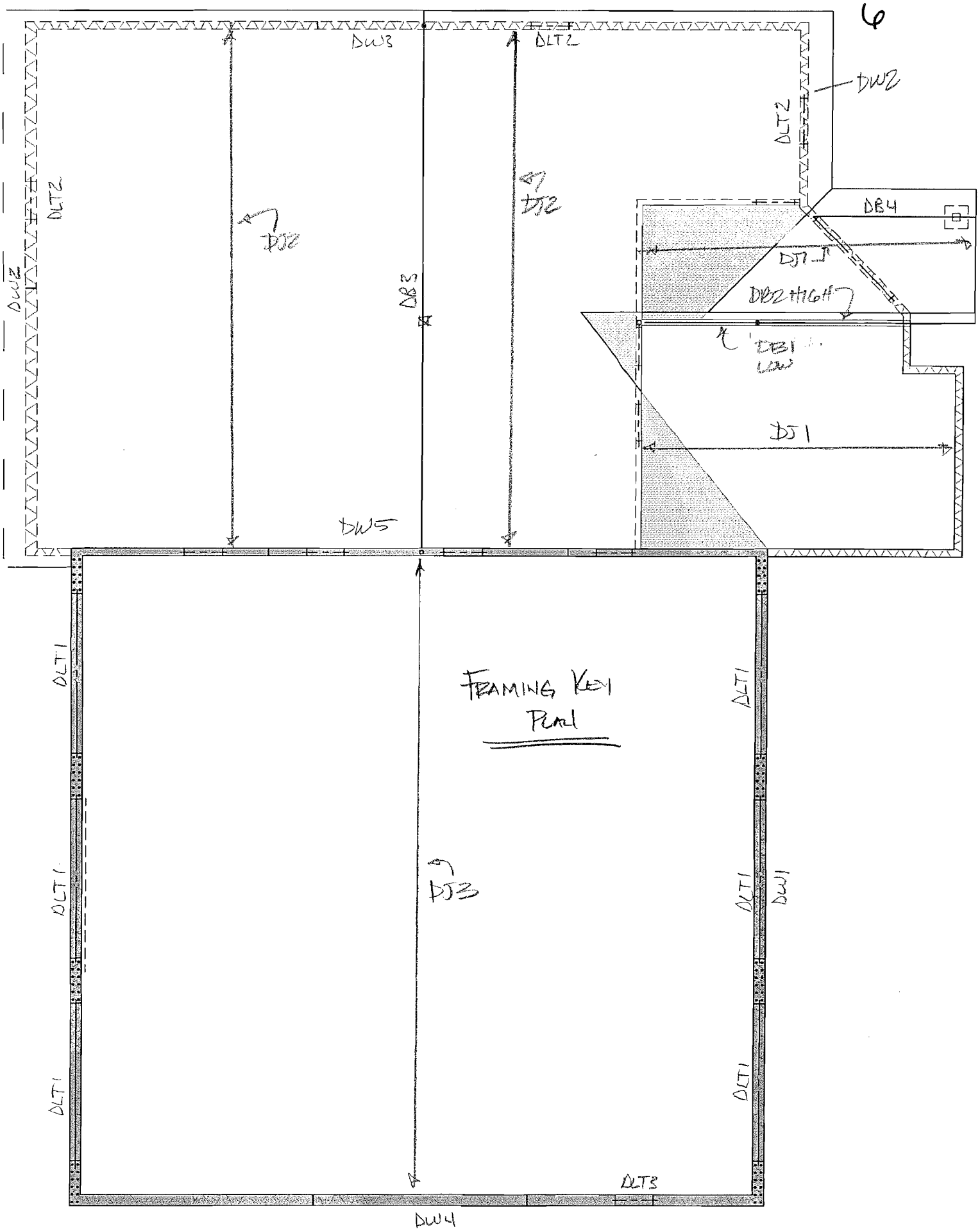
Period (sec)	SA (%g)	
0.2	032.9	Map Value, Soil Factor of 1.0
1.0	008.8	Map Value, Soil Factor of 1.0
MCE Parameters x Specified Soil Factors		
0.2	050.7	Soil Factor of 1.54
1.0	021.1	Soil Factor of 2.40

MCE Parameters - Conterminous 48 States
Zip Code - 85653 Central Latitude = 32.421721
Central Longitude = -111.391974
Data are based on the 0.01 deg grid set

Period (sec)	SA (%g)	
0.2	032.9	Map Value, Soil Factor of 1.0
1.0	008.8	Map Value, Soil Factor of 1.0

MCE SPECTRUM x SOIL FACTORS
Fa = 1.54
Fv = 2.40

Period (sec)	SA (%g)	
0.000	020.3	0.4FaSs
0.083	050.7	To
0.200	050.7	T=0.2, FaSs
0.416	050.7	Ts
0.500	042.2	
0.600	035.2	
0.700	030.1	
0.800	026.4	
0.900	023.4	
1.000	021.1	T=1.0, FvS1
1.100	019.2	
1.200	017.6	
1.300	016.2	
1.400	015.1	
1.500	014.1	
1.600	013.2	
1.700	012.4	
1.800	011.7	
1.900	011.1	
2.000	010.5	


DW3
DLT2
DW2
DLT2
DLT2
DJ2
DJ2
DB4
DJ1
DB2 HIGH
DB3
DW2
DB1
LOW
DJ1
DW5
DLT1
DLT1
FRAMING KEY
PLAN
DLT1
DLT1
DJ3
DW1
DLT1
DLT1
DLT3
DW4

GRENIER ENGINEERING, INC.
ENGINEERS ▪ TUCSON ▪ YUMA ▪ TEMPE ▪ FARMINGTON

PROJECT: SILVERBELL NATIONAL GUARD SHEET NO. 7
JOB NO. 04202 DATE: 8/06
BY: MAM

BEAMS

DB1 Span1 = 23'-6" CANT = 6'-0"

DL = 405#/' } 0 - 10'-4"
LL = 315#/'

DL = 90#/' } 10'-4" to 20'-10"
LL = 105#/'

DL = 230#/' } 20'-10" to 23'-8"
LL = 105#/'

DL = 90#/' } 23'-8" to 29'-8"
LL = 105#/'

POINT LOAD

AT 10'-4" AT = 23'-8"
DL = 1040# DL = 2330#
LL = 1330# LL = 2560#

SEE PRINTOUT

USE W14x36

DB2 Span1 = 13'-0" CANT = 5'-0"

DL = 175#/'
LL = 205#/'

SEE PRINTOUT

USE HSS6x5x3/16

DB3 Span = 3'-0" to 15'-8"
w_{DL} = (67.33/2)(17)
= 573 plf
w_{LL} = (67.33/2)(12)
= 404 plf
Span = 15'-8" to 44'-0"
w_{DL} = (53.67/2)(17)
= 457 plf
w_{LL} = (53.67/2)(12)
= 322
SEE PRINTOUT
USE 6 3/4" x 24" GLB

DB4 Span = 15'-0"
w_{DL} = (9.34/2 + 2.5)(17)
= 122 plf
w_{LL} = (9.34/2 + 2.5)(20)
= 144 plf
SEE PRINTOUT
USE 5 1/8 x 12 GLB

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Title : Silverbell National Guard
Dsgnr: Mark Medina
Description :
Scope :

Job # 06202
Date: 3:16PM, 17 AUG 06

8

Steel Beam Design

Description DB1

General Information

Code Ref: AISC 9th ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Steel Section : W16X36

		Pinned-Pinned	Fy	50.00 ksi
Center Span	23.67 ft	Bm Wt. Added to Loads	Load Duration Factor	1.00
Left Cant.	0.00 ft	LL & ST Act Together	Elastic Modulus	29,000.0 ksi
Right Cant	6.00 ft			
Lu : Unbraced Length	6.00 ft			

Distributed Loads

Note! Short Term Loads Are WIND Loads.

	# 1	# 2	# 3	# 4	# 5	# 6	# 7	
DL	0.405	0.090	0.230	0.090				k/ft
LL	0.315	0.105	0.105	0.105				k/ft
ST								k/ft
Start Location		10.330	20.833	23.670				ft
End Location	10.333	20.830	23.670	29.670				ft

Point Loads

Note! Short Term Loads Are WIND Loads.

	# 1	# 2	# 3	# 4	# 5	# 6	# 7	
Dead Load	1.040	2.330						k
Live Load	1.330	2.560						k
Short Term								k
Location	10.330	23.670						ft

Summary

Beam OK
Static Load Case Governs Stress

Using: W16X36 section, Span = 23.67ft, Fy = 50.0ksi, Left Cant. = 0.00ft, Right Cant. = 6.00ft
End Fixity = Pinned-Pinned, Lu = 6.00ft, LDF = 1.000

	Actual	Allowable
Moment	44.754 k-ft	155.375 k-ft
fb : Bending Stress	9.505 ksi	33.000 ksi
fb / Fb	**0.288 : 1**	
Shear	8.239 k	93.574 k
fv : Shear Stress	1.761 ksi	20.000 ksi
fv / Fv	**0.088 : 1**	

Max. Deflection	-0.318 in
Length/DL Defl	1,194.6 : 1
Length/(DL+LL Defl)	611.6 : 1

Force & Stress Summary

<<– These columns are Dead + Live Load placed as noted –>>

	Maximum	DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max. M +	44.75 k-ft	23.47	44.75		22.65		k-ft
Max. M -		-2.27	-2.27		-4.16		k-ft
Max. M @ Left							k-ft
Max. M @ Right		-2.27	-2.27		-4.16		k-ft
Shear @ Left	8.24 k	4.55	8.24		4.47		k
Shear @ Right	5.42 k	3.12	5.42		3.20		k
Center Defl.	-0.318 in	-0.167	-0.318	-0.318	-0.158	-0.158	in
Left Cant Defl	0.000 in	0.000	0.000	0.000	0.000	0.000	in
Right Cant Defl	0.235 in	0.121	0.235	0.235	0.104	0.104	in
...Query Defl @	0.000 ft	0.000	0.000	0.000	0.000	0.000	in
Reaction @ Left	8.24	4.55	8.24	8.24	4.47	4.47	k
Reaction @ Rt	11.07	6.21	11.07	11.07	9.48	9.48	k

Fa calc'd per Eq. E2-1, K*L/r < Cc
I Beam Passes Table B5.1, Fb per Eq. F1-1, Fb = 0.66 Fy

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Scope :

Job # 06202
Date: 3:16PM, 17 AUG 06

9

Steel Beam Design

Description DB1

Section Properties W16X36

Depth	15.860 in	Weight	36.00 #/ft
Web Thick	0.295 in	Ixx	448.000 in4
Width	6.985 in	Iyy	24.500 in4
Flange Thick	0.430 in	Sxx	56.500 in3
Area	10.60 in2	Syy	7.000 in3
Rt	1.790 in	R-xx	6.510 in
Values for LRFD Design....		R-yy	1.520 in
J	0.540 in4	Zx	64.000 in3
Cw	1,460.00 in6	Zy	10.800 in3
		K	0.832 in

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Scope :

Job # 06202
Date: 3:16PM, 17 AUG 06

10

Steel Beam Design

Description DB2

General Information

Code Ref: AISC 9th ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Steel Section : HSS6X5X3/16

		Pinned-Pinned	Fy	46.00 ksi
Center Span	13.00 ft	Bm Wt. Added to Loads	Load Duration Factor	1.00
Left Cant.	0.00 ft	LL & ST Act Together	Elastic Modulus	29,000.0 ksi
Right Cant	5.00 ft			
Lu : Unbraced Length	5.00 ft			

Distributed Loads

Note! Short Term Loads Are WIND Loads.

	# 1	# 2	# 3	# 4	# 5	# 6	# 7	
DL	0.175							k/ft
LL	0.205							k/ft
ST								k/ft
Start Location								ft
End Location								ft

Summary

Beam OK
Static Load Case Governs Stress

Using: HSS6X5X3/16 section, Span = 13.00ft, Fy = 46.0ksi, Left Cant. = 0.00ft, Right Cant. = 5.00ft
End Fixity = Pinned-Pinned, Lu = 5.00ft, LDF = 1.000

	Actual	Allowable		
Moment	7.158 k-ft	14.812 k-ft	Max. Deflection	-0.374 in
fb : Bending Stress	13.339 ksi	27.600 ksi	Length/DL Defl	1,117.5 : 1
fb / Fb	**0.483 : 1**		Length/(DL+LL Defl)	333.7 : 1
Shear	2.730 k	38.419 k		
fv : Shear Stress	1.308 ksi	18.400 ksi		
fv / Fv	**0.071 : 1**			

Force & Stress Summary

<<-- These columns are Dead + Live Load placed as noted -->>

	Maximum	DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max. M +	7.16 k-ft	2.87	7.16		1.89		k-ft
Max. M -		-2.34	-2.34		-4.90		k-ft
Max. M @ Left							k-ft
Max. M @ Right		-2.34	-2.34		-4.90		k-ft
Shear @ Left	2.37 k	1.04	2.37		0.84		k
Shear @ Right	2.73 k	1.40	2.73		1.93		k
Center Defl.	-0.374 in	-0.140	-0.374	-0.374	-0.062	-0.062	in
Left Cant Defl	0.000 in	0.000	0.000	0.000	0.000	0.000	in
Right Cant Defl	0.360 in	0.066	0.360	0.360	-0.157	-0.157	in
...Query Defl @	0.000 ft	0.000	0.000	0.000	0.000	0.000	in
Reaction @ Left	2.37	1.04	2.37	2.37	0.84	0.84	k
Reaction @ Rt	4.89	2.33	3.67	3.67	3.56	3.56	k

Fa calc'd per Eq. E2-1, K*L/r < Cc

Section Properties HSS6X5X3/16

Depth	6.000 in	Weight	12.33 #/ft
Web Thick	0.174 in	Ixx	19.300 in4
Width	5.000 in	Iyy	14.600 in4
Flange Thick	0.174 in	Sxx	6.440 in3
Area	3.63 in2	Syy	5.840 in3
Rt	2.500 in	R-xx	2.310 in
Values for LRFD Design....		R-yy	2.010 in
J	26.300 in4	Zx	7.620 in3
Cw	9.76 in6	Zy	6.730 in3

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Scope :

Job # 06202
Date: 3:53PM, 16 AUG 06

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Rev: 580004
User: KW-0605362, Ver 5.8.0, 1-Dec-2003
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Multi-Span Timber Beam

Description DB3

General Information

Code Ref: 1997/2001 NDS, 2000/2003 IBC, 2003 NFPA 5000. Base allowables are user defined

Douglas Fir, 24F - V8	Fb : Basic Allow	2,400.0 psi	Elastic Modulus	1,700.0 ksi
Spans Considered Continuous Over Support	Fv : Basic Allow	240.0 psi	Load Duration Factor	1.250

Timber Member Information

Description		Span 1	Span 2
Span	ft	20.50	25.67
Timber Section		6.75x24	6.75x24
Beam Width	in	6.750	6.750
Beam Depth	in	24.000	24.000
End Fixity		Pin - Pin	Pin - Pin
Le: Unbraced Length	ft	2.00	2.00
Member Type		GluLam	GluLam

Loads

Live Load Used This Span ?		Yes	Yes
Dead Load	#/ft	457.00	457.00
Live Load	#/ft	322.00	322.00
Dead Load	#/ft		115.00
Live Load	#/ft		85.00
Start	ft		10.500
End	ft	20.500	25.670

Results

Mmax @ Cntr	in-k	200.4	592.5
@ X =	ft	6.56	15.57
Max @ Left End	in-k	0.0	-709.5
Max @ Right End	in-k	-709.5	0.0
fb : Actual	psi	1,094.9	1,094.9
Fb : Allowable	psi	2,719.6	2,659.4
		Bending OK	**Bending OK**
Shear @ Left	k	5.10	13.20
Shear @ Right	k	10.87	9.83
fv : Actual	psi	85.9	108.6
Fv : Allowable	psi	300.0	300.0
		Shear OK	**Shear OK**

Reactions & Deflection

DL @ Left	k	3.00	14.10
LL @ Left	k	2.11	9.96
Total @ Left	k	**5.10**	**24.07**
DL @ Right	k	14.10	5.75
LL @ Right	k	9.96	4.09
Total @ Right	k	**24.07**	**9.83**
Max. Deflection	in	-0.043	-0.360
@ X =	ft	6.83	14.38

Query Values

Location	ft	0.00	0.00
Moment	in-k	0.0	-709.5
Shear	k	5.1	13.2
Deflection	in	0.0000	0.0000

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Scope :

Job # 06202
Date: 9:37AM, 16 AUG 06

12

General Timber Beam

Description DB4

General Information

Code Ref: 1997/2001 NDS, 2000/2003 IBC, 2003 NFPA 5000. Base allowables are user defined

Section Name	**5.125x12**	Center Span	13.00 ft	.....Lu	2.00 ft
Beam Width	5.125 in	Left Cantilever	ft	.....Lu	0.00 ft
Beam Depth	12.000 in	Right Cantilever	ft	.....Lu	0.00 ft
Member Type	GluLam	Douglas Fir, 24F - V8			
Bm Wt. Added to Loads		Fb Base Allow	2,400.0 psi		
Load Dur. Factor	1.250	Fv Allow	240.0 psi		
Beam End Fixity	Pin-Pin	Fc Allow	650.0 psi		
Wood Density	35.000 pcf	E	1,700.0 ksi		

Full Length Uniform Loads

Center	DL	122.00 #/ft	LL	144.00 #/ft
Left Cantilever	DL	#/ft	LL	#/ft
Right Cantilever	DL	#/ft	LL	#/ft

Summary

Beam Design OK

Span= 13.00ft, Beam Width = 5.125in x Depth = 12.in, Ends are Pin-Pin

Max Stress Ratio 0.194 : 1

Maximum Moment	5.9 k-ft	Maximum Shear * 1.5	2.3 k
Allowable	30.6 k-ft	Allowable	18.5 k

Max. Positive Moment	5.94 k-ft	at	6.500 ft	Shear:	@ Left	1.83 k
Max. Negative Moment	0.00 k-ft	at	0.000 ft		@ Right	1.83 k
Max @ Left Support	0.00 k-ft			Camber:	@ Left	0.000 in
Max @ Right Support	0.00 k-ft				@ Center	0.105 in
Max. M allow	30.64				@ Right	0.000 in

				Reactions...			
fb	579.03 psi	fv	37.77 psi	Left DL	0.89 k	Max	1.83 k
Fb	2,989.57 psi	Fv	300.00 psi	Right DL	0.89 k	Max	1.83 k

Deflections

Center Span...	Dead Load	Total Load
Deflection	-0.070 in	-0.144 in
...Location	6.500 ft	6.500 ft
...Length/Defl	2,224.0	1,084.08
Camber (using 1.5 * D.L. Defl) ...		
@ Center	0.105 in	
@ Left	0.000 in	
@ Right	0.000 in	

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

Stress Calcs

Bending Analysis

Ck	19.306	Le	4.118 ft	Sxx	123.000 in3	Area	61.500 in2
Cv	1.000	Rb	4.753	Cl	0.997		

	Max Moment	Sxx Req'd	Allowable fb
@ Center	5.94 k-ft	23.82 in3	2,989.57 psi
@ Left Support	0.00 k-ft	0.00 in3	3,000.00 psi
@ Right Support	0.00 k-ft	0.00 in3	3,000.00 psi

Shear Analysis	@ Left Support	@ Right Support
Design Shear	2.32 k	2.32 k
Area Required	7.743 in2	7.743 in2
Fv: Allowable	300.00 psi	300.00 psi

Bearing @ Supports

Max. Left Reaction	1.83 k	Bearing Length Req'd	0.548 in
Max. Right Reaction	1.83 k	Bearing Length Req'd	0.548 in

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Description :
Scope :

Job # 06202
Date: 3:16PM, 17 AUG 06

13

Steel Column

Description DC1 At DB1

General Information

Code Ref: AISC 9th ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Steel Section	HSS5X5X1/4	Fy	46.00 ksi	X-X Sidesway :	Restrained
		Duration Factor	1.000	Y-Y Sidesway :	Restrained
Column Height	12.500 ft	Elastic Modulus	29,000.00 ksi		
End Fixity	Pin-Pin	X-X Unbraced	12.500 ft	Kxx	1.000
Live & Short Term Loads Combined		Y-Y Unbraced	12.500 ft	Kyy	1.000

Loads

Axial Load...

Dead Load	4.55 k	Ecc. for X-X Axis Moments	0.000 in
Live Load	8.24 k	Ecc. for Y-Y Axis Moments	0.000 in
Short Term Load	k		

Summary

Column Design OK

Section : HSS5X5X1/4, Height = 12.50ft, Axial Loads: DL = 4.55, LL = 8.24, ST = 0.00k, Ecc. = 0.000in
Unbraced Lengths: X-X = 12.50ft, Y-Y = 12.50ft

Combined Stress Ratios	Dead	Live	DL + LL	DL + ST + (LL if Chosen)
AISC Formula H1 - 1			0.1611	0.1611
AISC Formula H1 - 2			0.1078	0.1078
AISC Formula H1 - 3	0.0573	0.1038		

XX Axis : Fa calc'd per Eq. E2-1, K*L/r < Cc
YY Axis : Fa calc'd per Eq. E2-1, K*L/r < Cc

Stresses

Allowable & Actual Stresses	Dead	Live	DL + LL	DL + Short
Fa : Allowable	18.47 ksi	18.47 ksi	18.47 ksi	18.47 ksi
fa : Actual	1.06 ksi	1.92 ksi	2.97 ksi	2.97 ksi
Fb:xx : Allow [F3.1]	27.60 ksi	27.60 ksi	27.60 ksi	27.60 ksi
fb : xx Actual	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi
Fb:yy : Allow [F3.1]	27.60 ksi	27.60 ksi	27.60 ksi	27.60 ksi
fb : yy Actual	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi

Analysis Values

F'ex : DL+LL	24,696 psi	Cm:x DL+LL	0.60	Cb:x DL+LL	1.00
F'ey : DL+LL	24,696 psi	Cm:y DL+LL	0.60	Cb:y DL+LL	1.00
F'ex : DL+LL+ST	24,696 psi	Cm:x DL+LL+ST	0.60	Cb:x DL+LL+ST	1.00
F'ey : DL+LL+ST	24,696 psi	Cm:y DL+LL+ST	0.60	Cb:y DL+LL+ST	1.00

Max X-X Axis Deflection 0.000 in at 0.000 ft Max Y-Y Axis Deflection 0.000 in at 0.000 ft

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Scope :

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Description DC1 At DB1

Section Properties HSS5X5X1/4

Depth	5.000 in	Weight	14.61 #/ft	Values for LRFD Design....	
Web Thick	0.233 in	Ixx	16.000 in4	J	25.800 in4
Width	5.000 in	Iyy	16.000 in4	Cw	10.50 in6
Flange Thick	0.233 in	Sxx	6.410 in3	Zx	7.610 in3
Area	4.30 in2	Syy	6.410 in3	Zy	7.610 in3
Rt	0.000 in	Rxx	1.930 in		0.000
		Ryy	1.930 in		

Section Type = HSS-Square

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Rev: 580001
User: KW-0605362, Ver 5.8.0, 1-Dec-2003
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Steel Column Base Plate

06202.ecw:Calculations

Description Base Plate at DC1

General Information

Code Ref : AISC 9th Ed ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Loads

Axial Load	8.24 k
X-X Axis Moment	0.00 k-ft

Plate Dimensions

Plate Length	12.000 in
Plate Width	12.000 in
Plate Thickness	0.000 in

Support Pier Size

Pier Length	48.000 in
Pier Width	48.000 in

Steel Section HSS5x5x1/4

Section Length	5.000 in
Section Width	5.000 in
Flange Thickness	0.233 in
Web Thickness	5.000 in

Allowable Stresses

Concrete f'c	2,500.0 psi
Base Plate Fy	36.00 ksi
Load Duration Factor	1.000

Anchor Bolt Data

Dist. from Plate Edge	2.000 in
Bolt Count per Side	2
Tension Capacity	5.500 k
Bolt Area	0.442 in2

Summary

Baseplate OK

Concrete Bearing Stress **Bearing Stress OK**

Actual Bearing Stress	57.2 psi
Allow per ACI318-95, A3.1 = 0.3 * f'c * Sqrt(A2/A1) * LDF	1,500.0 psi
Allow per AISC J9	1,750.0 psi

Full Bearing : No Bolt Tension

Plate Bending Stress **Thickness OK**

Actual fb	19,531.9 psi
Max Allow Plate Fb	27,000.0 psi

Tension Bolt Force **Bolt Tension OK**

Actual Tension	0.000 k
Allowable	5.500 k

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Rev: 580002
User: KW-0605362, Ver 5.8.0, 1-Dec-2003
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General Footing Analysis & Design

Description Footing At DC1

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Allowable Soil Bearing	2,000.0 psf	**Dimensions...**	
Short Term Increase	1.000	Width along X-X Axis	3.000 ft
Seismic Zone	2	Length along Y-Y Axis	3.000 ft
		Footing Thickness	12.00 in
Live & Short Term Combined		Col Dim. Along X-X Axis	5.00 in
f'c	2,500.0 psi	Col Dim. Along Y-Y Axis	5.00 in
Fy	40,000.0 psi	Base Pedestal Height	0.000 in
Concrete Weight	145.00 pcf		
Overburden Weight	0.00 psf	Min Steel %	0.0014
		Rebar Center To Edge Distance	3.50 in

Loads

Applied Vertical Load...			
Dead Load	4.550 k	...ecc along X-X Axis	0.000 in
Live Load	3.690 k	...ecc along Y-Y Axis	0.000 in
Short Term Load	k		

Applied Moments...	Creates Rotation about Y-Y Axis (pressures @ left & right)	Creates Rotation about X-X Axis (pressures @ top & bot)
Dead Load	k-ft	k-ft
Live Load	k-ft	k-ft
Short Term	k-ft	k-ft

Applied Shears...	Creates Rotation about Y-Y Axis (pressures @ left & right)	Creates Rotation about X-X Axis (pressures @ top & bot)
Dead Load	k	k
Live Load	k	k
Short Term	k	k

Summary

Footing Design OK

3.00ft x 3.00ft Footing, 12.0in Thick, w/ Column Support 5.00 x 5.00in x 0.0in high

	DL+LL	DL+LL+ST
Max Soil Pressure	1,060.6	1,060.6 psf
Allowable	2,000.0	2,000.0 psf
"X' Ecc, of Resultant	0.000 in	0.000 in
"Y' Ecc, of Resultant	0.000 in	0.000 in
X-X Min. Stability Ratio	No Overturning	1.500 :1
Y-Y Min. Stability Ratio	No Overturning	

	Actual	Allowable
Max Mu	1.172	
Required Steel Area		0.259
Shear Stresses....	Vu	Vn * Phi
1-Way	7.846	85.000 psi
2-Way	23.797	170.000 psi

Footing Design

Shear Forces	ACI C-1	ACI C-2	ACI C-3	Vn * Phi
Two-Way Shear	23.80 psi	17.85 psi	7.71 psi	170.00 psi
One-Way Shears...				
Vu @ Left	7.85 psi	5.88 psi	2.53 psi	85.00 psi
Vu @ Right	7.85 psi	5.88 psi	2.53 psi	85.00 psi
Vu @ Top	7.85 psi	5.88 psi	2.53 psi	85.00 psi
Vu @ Bottom	7.85 psi	5.88 psi	2.53 psi	85.00 psi

Moments	ACI C-1	ACI C-2	ACI C-3	Ru / Phi	As Req'd	
Mu @ Left	1.17 k-ft	0.88 k-ft	0.38 k-ft	18.0 psi	0.26 in2	per ft
Mu @ Right	1.17 k-ft	0.88 k-ft	0.38 k-ft	18.0 psi	0.26 in2	per ft
Mu @ Top	1.17 k-ft	0.88 k-ft	0.38 k-ft	18.0 psi	0.26 in2	per ft
Mu @ Bottom	1.17 k-ft	0.88 k-ft	0.38 k-ft	18.0 psi	0.26 in2	per ft

Description Footing At DC1

Soil Pressure Summary

Service Load Soil Pressures	Left	Right	Top	Bottom
DL + LL	1,060.56	1,060.56	1,060.56	1,060.56 psf
DL + LL + ST	1,060.56	1,060.56	1,060.56	1,060.56 psf
Factored Load Soil Pressures				
ACI Eq. C-1	1,607.78	1,607.78	1,607.78	1,607.78 psf
ACI Eq. C-2	1,205.83	1,205.83	1,205.83	1,205.83 psf
ACI Eq. C-3	585.50	585.50	585.50	585.50 psf

ACI Factors (per ACI 318-02, applied internally to entered loads)

ACI C-1 & C-2 DL	1.400	ACI C-2 Group Factor	0.750	Add"l "1.4" Factor for Seismic	1.400
ACI C-1 & C-2 LL	1.700	ACI C-3 Dead Load Factor	0.900	Add"l "0.9" Factor for Seismic	0.900
ACI C-1 & C-2 ST	1.700	ACI C-3 Short Term Factor	1.300		
....seismic = ST * :	1.100 Used in ACI C-2 & C-3				

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Description :
Scope :

Job # 06202
Date: 3:16PM, 17 AUG 06

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Steel Column

Description DC2 at DB3

General Information

Code Ref: AISC 9th ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Steel Section	HSS5X5X1/4	Fy	46.00 ksi	X-X Sidesway :	Restrained
		Duration Factor	1.000	Y-Y Sidesway :	Restrained
Column Height	21.000 ft	Elastic Modulus	29,000.00 ksi		
End Fixity	Pin-Pin	X-X Unbraced	21.000 ft	Kxx	1.000
Live & Short Term Loads Combined		Y-Y Unbraced	21.000 ft	Kyy	1.000

Loads

Axial Load...

Dead Load	14.10 k	Ecc. for X-X Axis Moments	0.000 in
Live Load	10.00 k	Ecc. for Y-Y Axis Moments	0.000 in
Short Term Load	k		

Summary

Column Design OK

Section : HSS5X5X1/4, Height = 21.00ft, Axial Loads: DL = 14.10, LL = 10.00, ST = 0.00k, Ecc. = 0.000in
Unbraced Lengths: X-X = 21.00ft, Y-Y = 21.00ft

Combined Stress Ratios	Dead	Live	DL + LL	DL + ST + (LL if Chosen)
AISC Formula H1 - 1	0.3748	0.2658	0.6406	0.6406
AISC Formula H1 - 2	0.1188	0.0843	0.2031	0.2031
AISC Formula H1 - 3				

XX Axis : Fa calc'd per Eq. E2-2, K*L/r > Cc
YY Axis : Fa calc'd per Eq. E2-2, K*L/r > Cc

Stresses

Allowable & Actual Stresses	Dead	Live	DL + LL	DL + Short
Fa : Allowable	8.75 ksi	8.75 ksi	8.75 ksi	8.75 ksi
fa : Actual	3.28 ksi	2.33 ksi	5.61 ksi	5.61 ksi
Fb:xx : Allow [F3.1]	30.36 ksi	30.36 ksi	30.36 ksi	30.36 ksi
fb : xx Actual	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi
Fb:yy : Allow [F3.1]	30.36 ksi	30.36 ksi	30.36 ksi	30.36 ksi
fb : yy Actual	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi

Analysis Values

F'ex : DL+LL	8,750 psi	Cm:x DL+LL	0.60	Cb:x DL+LL	1.00
F'ey : DL+LL	8,750 psi	Cm:y DL+LL	0.60	Cb:y DL+LL	1.00
F'ex : DL+LL+ST	8,750 psi	Cm:x DL+LL+ST	0.60	Cb:x DL+LL+ST	1.00
F'ey : DL+LL+ST	8,750 psi	Cm:y DL+LL+ST	0.60	Cb:y DL+LL+ST	1.00

Max X-X Axis Deflection 0.000 in at 0.000 ft Max Y-Y Axis Deflection 0.000 in at 0.000 ft

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Scope :

Job # 06202
Date: 3:16PM, 17 AUG 06

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Description DC2 at DB3

Section Properties HSS5X5X1/4

Depth	5.000 in	Weight	14.61 #/ft	Values for LRFD Design....	
Web Thick	0.233 in	Ixx	16.000 in4	J	25.800 in4
Width	5.000 in	Iyy	16.000 in4	Cw	10.50 in6
Flange Thick	0.233 in	Sxx	6.410 in3	Zx	7.610 in3
Area	4.30 in2	Syy	6.410 in3	Zy	7.610 in3
Rt	0.000 in	Rxx	1.930 in		0.000
		Ryy	1.930 in		

Section Type = HSS-Square

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Rev: 580001
User: KW-0605362, Ver 5.8.0, 1-Dec-2003
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Steel Column Base Plate

Description Base plate for DC2

General Information

Code Ref : AISC 9th Ed ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Loads

Axial Load	29.20 k
X-X Axis Moment	0.00 k-ft

Plate Dimensions

Plate Length	12.000 in
Plate Width	12.000 in
Plate Thickness	0.000 in

Support Pier Size

Pier Length	48.000 in
Pier Width	48.000 in

Steel Section HSS5x5x1/4

Section Length	5.000 in
Section Width	5.000 in
Flange Thickness	0.233 in
Web Thickness	5.000 in

Allowable Stresses

Concrete f'c	2,500.0 psi
Base Plate Fy	36.00 ksi
Load Duration Factor	1.000

Anchor Bolt Data

Dist. from Plate Edge	2.000 in
Bolt Count per Side	2
Tension Capacity	5.500 k
Bolt Area	0.442 in2

Summary

Baseplate OK

Concrete Bearing Stress **Bearing Stress OK**

Actual Bearing Stress	202.8 psi
Allow per ACI318-95, A3.1 = 0.3 * f'c * Sqrt(A2/A1) * LDF	1,500.0 psi
Allow per AISC J9	1,750.0 psi

Full Bearing : No Bolt Tension

Plate Bending Stress **Thickness OK**

Actual fb	24,917.3 psi
Max Allow Plate Fb	27,000.0 psi

Tension Bolt Force **Bolt Tension OK**

Actual Tension	0.000 k
Allowable	5.500 k

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Scope :

Job # 06202
Date: 3:16PM, 17 AUG 06

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General Footing Analysis & Design

Description Footing at DC2

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Allowable Soil Bearing	2,000.0 psf	**Dimensions...**	
Short Term Increase	1.000	Width along X-X Axis	4.000 ft
Seismic Zone	2	Length along Y-Y Axis	4.000 ft
		Footing Thickness	12.00 in
Live & Short Term Combined		Col Dim. Along X-X Axis	5.00 in
f'c	2,500.0 psi	Col Dim. Along Y-Y Axis	5.00 in
Fy	40,000.0 psi	Base Pedestal Height	0.000 in
Concrete Weight	145.00 pcf		
Overburden Weight	0.00 psf	Min Steel %	0.0014
		Rebar Center To Edge Distance	3.50 in

Loads

Applied Vertical Load...

Dead Load	20.720 k	...ecc along X-X Axis	0.000 in
Live Load	8.500 k	...ecc along Y-Y Axis	0.000 in
Short Term Load	k		

Applied Moments...	Creates Rotation about Y-Y Axis (pressures @ left & right)	Creates Rotation about X-X Axis (pressures @ top & bot)
Dead Load	k-ft	k-ft
Live Load	k-ft	k-ft
Short Term	k-ft	k-ft

Applied Shears...	Creates Rotation about Y-Y Axis (pressures @ left & right)	Creates Rotation about X-X Axis (pressures @ top & bot)
Dead Load	k	k
Live Load	k	k
Short Term	k	k

Summary

Footing Design OK

4.00ft x 4.00ft Footing, 12.0in Thick, w/ Column Support 5.00 x 5.00in x 0.0in high

	DL+LL	DL+LL+ST
Max Soil Pressure	1,971.2	1,971.2 psf
Allowable	2,000.0	2,000.0 psf
"X' Ecc, of Resultant	0.000 in	0.000 in
"Y' Ecc, of Resultant	0.000 in	0.000 in
X-X Min. Stability Ratio	No Overturning	1.500 :1
Y-Y Min. Stability Ratio	No Overturning	

	Actual	Allowable
Max Mu	4.359	
Required Steel Area		0.259
Shear Stresses....	Vu	Vn * Phi
1-Way	27.826	85.000 psi
2-Way	87.595	170.000 psi

Footing Design

Shear Forces	ACI C-1	ACI C-2	ACI C-3	Vn * Phi
Two-Way Shear	87.59 psi	65.70 psi	37.59 psi	170.00 psi
One-Way Shears...				
Vu @ Left	27.83 psi	20.87 psi	11.92 psi	85.00 psi
Vu @ Right	27.83 psi	20.87 psi	11.92 psi	85.00 psi
Vu @ Top	27.83 psi	20.87 psi	11.92 psi	85.00 psi
Vu @ Bottom	27.83 psi	20.87 psi	11.92 psi	85.00 psi

Moments	ACI C-1	ACI C-2	ACI C-3	Ru / Phi	As Req'd	
Mu @ Left	4.36 k-ft	3.27 k-ft	1.87 k-ft	67.0 psi	0.26 in2	per ft
Mu @ Right	4.36 k-ft	3.27 k-ft	1.87 k-ft	67.0 psi	0.26 in2	per ft
Mu @ Top	4.36 k-ft	3.27 k-ft	1.87 k-ft	67.0 psi	0.26 in2	per ft
Mu @ Bottom	4.36 k-ft	3.27 k-ft	1.87 k-ft	67.0 psi	0.26 in2	per ft

Description Footing at DC2

Soil Pressure Summary

Service Load Soil Pressures	Left	Right	Top	Bottom
DL + LL	1,971.25	1,971.25	1,971.25	1,971.25 psf
DL + LL + ST	1,971.25	1,971.25	1,971.25	1,971.25 psf
Factored Load Soil Pressures				
ACI Eq. C-1	2,919.12	2,919.12	2,919.12	2,919.12 psf
ACI Eq. C-2	2,189.34	2,189.34	2,189.34	2,189.34 psf
ACI Eq. C-3	1,296.00	1,296.00	1,296.00	1,296.00 psf

ACI Factors (per ACI 318-02, applied internally to entered loads)

ACI C-1 & C-2 DL	1.400	ACI C-2 Group Factor	0.750	Add"l "1.4" Factor for Seismic	1.400
ACI C-1 & C-2 LL	1.700	ACI C-3 Dead Load Factor	0.900	Add"l "0.9" Factor for Seismic	0.900
ACI C-1 & C-2 ST	1.700	ACI C-3 Short Term Factor	1.300		
....seismic = ST * :	1.100 Used in ACI C-2 & C-3				

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PROJECT: SILVERBELL NATIONAL GUARD SHEET NO. 23
JOB NO. CX0202 DATE: 8/06
BY: MAM

LINTELS

ALT1 Span = 14'-0"

$w_{DL} = (59.67/2)(17) + 12(108)$

$= 1744$ plf

$w_{LL} = (59.67/2)(20)$

$= 597$ plf

SEE PRINTOUT

USE W16 X 36

ALT2 Span = 4'-0"

$w_{DL} = (33.5/2 + 2.67)(17) + 4(66)$

$= 595$ plf

$w_{LL} = (33.5/2 + 2.67)(20)$

$= 389$ plf

$w_{TOTAL} = 984$ plf

USE 8" X 24" DEEP CMU LINTEL
W/(2) #5 BOTTOM + TOP REINFORCING

ALT3 Span = 8'-4"

$w_{DL} = (2)(17) + 3.34(108)$

$= 379$ plf

$w_{LL} = (2)(20)$

$= 40$

$w_{TOTAL} = 419$ plf

USE 8" X 24" DEEP CMU LINTEL
W/(2) #5 BOTTOM + TOP REINFORCING

STEEL STUD LINTEL

SPAN = 40' MAX

$M_{MAX} = 17psf(\frac{22}{2}) + 17psf(8) = 323$ #'

$M_{MAX} = 20psf(\frac{22}{2}) = 220$ #'

$M = 543$ #'

USE (2) 600S200-43 unpunched
16 gage BOXED HEADERS

$W_{ALLOW} =$

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Job # 06202
Date: 9:34AM, 16 AUG 06

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Steel Beam Design

Description DLT1

General Information

Code Ref: AISC 9th ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Steel Section : W16X36

		Pinned-Pinned	Fy	36.00 ksi
Center Span	14.00 ft	Bm Wt. Added to Loads	Load Duration Factor	1.00
Left Cant.	0.00 ft	LL & ST Act Together	Elastic Modulus	29,000.0 ksi
Right Cant	0.00 ft			
Lu : Unbraced Length	14.00 ft			

Distributed Loads

Note! Short Term Loads Are WIND Loads.

	# 1	# 2	# 3	# 4	# 5	# 6	# 7	
DL	1.744							k/ft
LL	0.597							k/ft
ST								k/ft
Start Location								ft
End Location								ft

Summary

Beam OK
Static Load Case Governs Stress

Using: W16X36 section, Span = 14.00ft, Fy = 36.0ksi
End Fixity = Pinned-Pinned, Lu = 14.00ft, LDF = 1.000

	Actual	Allowable
Moment	58.237 k-ft	77.869 k-ft
fb : Bending Stress	12.369 ksi	16.539 ksi
fb / Fb	**0.748 : 1**	
Shear	16.639 k	67.373 k
fv : Shear Stress	3.556 ksi	14.400 ksi
fv / Fv	**0.247 : 1**	

Max. Deflection	-0.158 in
Length/DL Defl	1,418.7 : 1
Length/(DL+LL Defl)	1,062.4 : 1

Force & Stress Summary

<<-- These columns are Dead + Live Load placed as noted -->>

	Maximum	DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max. M +	58.24 k-ft	43.61	58.24				k-ft
Max. M -							k-ft
Max. M @ Left							k-ft
Max. M @ Right							k-ft
Shear @ Left	16.64 k	12.46	16.64				k
Shear @ Right	16.64 k	12.46	16.64				k
Center Defl.	-0.158 in	-0.118	-0.158	-0.158	0.000	0.000	in
Left Cant Defl	0.000 in	0.000	0.000	0.000	0.000	0.000	in
Right Cant Defl	0.000 in	0.000	0.000	0.000	0.000	0.000	in
...Query Defl @	0.000 ft	0.000	0.000	0.000	0.000	0.000	in
Reaction @ Left	16.64	12.46	16.64	16.64			k
Reaction @ Rt	16.64	12.46	16.64	16.64			k

Fa calc'd per Eq. E2-2, K*L/r > Cc
I Beam, Major Axis, (102,000 * Cb / Fy)^.5 <= L/rT <= (510,000 * Cb / Fy)^.5 , Fb per Eq. F1-6

Description DLT1

Section Properties W16X36

Depth	15.860 in	Weight	36.00 #/ft
Web Thick	0.295 in	Ixx	448.000 in4
Width	6.985 in	Iyy	24.500 in4
Flange Thick	0.430 in	Sxx	56.500 in3
Area	10.60 in2	Syy	7.000 in3
Rt	1.790 in	R-xx	6.510 in
Values for LRFD Design....		R-yy	1.520 in
J	0.540 in4	Zx	64.000 in3
Cw	1,460.00 in6	Zy	10.800 in3
		K	0.832 in

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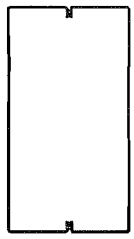
Header Load Table Notes

1. Deflection limit is L/360.
2. Allowable loads have not been modified for wind or earthquake loading.
3. Headers are made from two "boxed" or back to back members.
4. Allowable moment, shear and web crippling are based on twice the capacity of a single member. The moment of inertia is based on twice the value of the single member.
5. Bearing length for web crippling = 1" minimum.
6. Values are for unpunched members.
7. Members are assumed adequately braced for bending.
8. Allowable loads are for simply supported headers with uniform bending loads only.



Boxed Header

Back-to-Back Header

Header Allowable Uniform Loads (lb/ft)

Section	Yield Strength (ksi)	SPAN 3 (ft)	4 (ft)	5 (ft)	6 (ft)	8 (ft)	10 (ft)	12 (ft)
550S162-33	33	893 e	670 e	536 e	374 e	210 e	127 e	73 e
550S162-43	33	1982 e	1232 e	789 e	547 e	308 e	164 e	95
550S162-54	33	2779 e	1563 e	1000 e	694 e	390 e	203	117
550S162-54	50	3643 e	2049 e	1311 e	910 e	396 e	203	117
550S162-68	33	3514 e	1976 e	1265 e	878 e	488 e	250	144
550S162-68	50	5176 e	2911 e	1863 e	1157 e	488	250	144
600S137-33	33	816 e	612 e	489 e	373 e	209 e	134 e	80 e
600S162-33	33	816 e	612 e	489 e	408 e	237 e	152 e	90 e
600S200-33	33	816 e	612 e	489 e	408 e	254 e	162 e	104 e
600S137-43	33	1810 e	1233 e	789 e	548 e	308 e	178 e	103
600S162-43	33	1810 e	1357 e	889 e	617 e	347 e	202 e	117 e
600S200-43	33	1810 e	1357 e	919 e	638 e	359 e	229 e	135 e
600S250-43	33	1810 e	1357 e	967 e	671 e	377 e	241 e	155 e
600S137-54	33	2812 e	1581 e	1012 e	703 e	395 e	220 e	127
600S162-54	33	3135 e	1763 e	1128 e	783 e	440 e	250 e	144
600S200-54	33	3566 e	2006 e	1283 e	891 e	501 e	290 e	167
600S250-54	33	3392 e	1908 e	1221 e	848 e	477 e	305 e	193 e
600S137-54	50	3610 e	2269 e	1452 e	1008 e	429 e	220	127
600S162-54	50	3610 e	2313 e	1480 e	1028 e	488 e	250	144
600S200-54	50	3610 e	2500 e	1600 e	1111 e	566 e	290 e	167
600S250-54	50	3610 e	2666 e	1706 e	1185 e	641 e	328 e	190
600S137-68	33	3562 e	2004 e	1282 e	890 e	501 e	270	156
600S162-68	33	3968 e	2232 e	1428 e	992 e	558 e	308	178
600S200-68	33	4506 e	2534 e	1622 e	1126 e	633 e	358 e	207
600S250-68	33	4456 e	2506 e	1604 e	1114 e	626 e	401 e	239
600S137-68	50	5274 e	2966 e	1898 e	1252 e	528 e	270	156
600S162-68	50	5846 e	3288 e	2104 e	1426 e	601 e	308	178
600S200-68	50	6475 e	3642 e	2331 e	1618 e	700 e	358	207
600S250-68	50	5954 e	3349 e	2143 e	1488 e	807 e	413 e	239
600S137-97	33	5108 e	2873 e	1839 e	1277 e	715	366	211
600S162-97	33	5685 e	3197 e	2046 e	1421 e	799	419	242
600S200-97	33	6443 e	3624 e	2319 e	1610 e	906 e	490	283
600S250-97	33	7229 e	4066 e	2602 e	1807 e	1016 e	567	328
600S137-97	50	7526 e	4233 e	2709 e	1694 e	715	366	211
600S162-97	50	8403 e	4727 e	3025 e	1941 e	819	419	242
600S200-97	50	9560 e	5377 e	3441 e	2270 e	958	490	283
600S250-97	50	10277 e	5781 e	3700 e	2569 e	1109 e	567	328

"e" Web stiffeners required at each support

WALLS

DW1 H = 21'-3" Parapet = 4'-9"

w_{DL} = (59.67/2)(17)
= 508 plf

w_{LL} = (59.67/2)(20)
= 597 plf

SEE PRINTOUT

Use 12" CMU with #6 bars @ 16" o.c.

DW2 H = 10'-8" Parapet = 0'-0"

w_{DL} = (33.75/2 + 2.33)(17)
= 327 plf

w_{LL} = (33.75/2 + 2.33)(20)
= 385 plf

SEE PRINTOUT

Use 8" CMU with #5 bars @ 16" o.c.

DW3 H = 25'-0" Parapet = 0'-0"

w_{DL} = (2 + 1.33)17
= 57 plf

w_{LL} = (2 + 1.33)20
= 67 plf

SEE PRINTOUT

Use 8" CMU with (2) #5 bars @ 16" o.c.

DW4 H = 22'-8" Parapet = 3'-6"

w_{DL} = (2)(17)
= 34 plf

w_{LL} = (2)(20)
= 40 plf

SEE PRINTOUT

Use 12" CMU with #6 bars @ 16" o.c.

DW5 H = 22'-8" Parapet = 3'-6"

w_{DL} = (4)(17)
= 68 plf

w_{LL} = (4)(20)
= 80 plf

SEE PRINTOUT

Use 8" CMU with (2) #5 bars @ 16" o.c.

DW6 H = 15'-0"

DL = 17psf $(\frac{22}{2})\frac{16}{12}$ = 250 #/stud

LL = 20psf $(\frac{22}{2})\frac{16}{12}$ = 293 #/stud

WLAT = 10psf

Use 600S162-43 studs @ 16" o.c.

LEDGER FOR DW6

P_{Total} = 543#

Use 2x10 DF #2 w/ (3) #10 Screws & Simpson L70 EACH STUD

VALUES = 555# OK

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Description :
Scope :

Job # 06202
Date: 5:29PM, 16 AUG 06

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Rev: 580002
User: KW-0605362, Ver 5.8.0, 1-Dec-2003
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Masonry Wall Design

Description DW1

General Information

Code Ref: ACI 530-02

Wall Height	21.25 ft	Seismic Factor	0.1420	f'm	1,500.0 psi
Parapet Height	4.75 ft	Calc of Em = f'm *	900.00	Fs	24,000.0 psi
		Duration Factor	1.000	Special Inspection	
Thickness	12.0 in	Wall Wt Mult.	1.000	Grout @ Rebar Only	
Rebar Size	6			Normal Weight Block	
Rebar Spacing	16 in			Equivalent	
Depth to Rebar	5.750 in	@ Center		Solid Thickness	8.500 in

Loads

Uniform Load		Concentric Axial Load		Wind Load	23.400 psf
Dead Load	508.000 #/ft	Dead Load	0.000 #/ft		
Live Load	597.000 #/ft	Live Load	0.000 #/ft		
Load Eccentricity	9.310 in	Roof Load			
Roof Load					

Design Values

E	1,350,000 psi	Rebar Area	0.330 in2	np	0.10274	j	0.87932
n : Es / Em	21.481	Radius of Gyration	3.605 in	k	0.36205	2 / kj	6.28222
Wall Weight	103.000 psf	Moment of Inertia	1,325.390 in4				

Max Allow Axial Stress = 0.25 f'm (1-(h/140r)^2) * SpInsp 279.26 psi
Allow Masonry Bending Stress = 0.33 f'm * SpInsp = 495.00 psi
Allow Steel Bending Stress = 24,000.00 psi

Load Combination & Stress Details Summary

	Moment	Axial Load	Bending Stresses Steel	Bending Stresses Masonry	Axial Compression
Top of Wall	in-#	lbs	psi	psi	psi
DL + LL	10,287.6	1,594.3	6,165.7	162.9	15.63
DL + LL + Wind	7,897.3	997.3	4,733.1	125.0	9.78
DL + LL + Seismic	6,709.5	997.3	4,021.3	106.2	9.78
Between Base & Top of Wall					
DL + LL	5,143.8	2,688.6	3,082.9	81.4	26.36
DL + LL + Wind	16,640.2	2,091.6	9,973.2	263.5	20.51
DL + LL + Seismic	11,329.2	2,091.6	6,790.0	179.4	20.51

Summary

21.25ft high wall with 4.75ft parapet, Normal Block w/ 12.00in wall w/ #6 bars at 16.00ino.c. at center

Max. Bending Compressive Stress . . . . . 283.99 OK
Allowable . . . . . 495.00 **OK**

Max. Axial Only Compressive Stress . . . . . 26.36 psi
Allowable . . . . . 279.26 **OK**

Max Steel Bending Stress . . . . . 9,973.17 psi
Allowable . . . . . 24,000.00 **OK**

Description DW1

Final Loads & Moments

Wall Weight moment @ Mid Ht	1,583.63	lbs	Wind Moment @ Mid Ht	14,305.52	in-#
			Seismic Moment @ Mid Ht	8,941.57	in-#
Dead Load Moment @ Top of Wall	4,729.48	in-#			
Dead Load Moment @ Mid Ht	2,364.74	in-#	Total Dead Load	508.00	lbs
			Total Live Load	597.00	lbs
Live Load Moment @ Top of Wall	5,558.07	in-#			
LiveLoad Moment @ Mid Ht	2,779.04	in-#			

Maximum Allow Moment for Applied Axial Load = 31,261.43 in-#
Maximum Allow Axial Load for Applied Moment = 28,484.10 lbs

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Description :
Scope :

Job # 06202
Date: 5:29PM, 16 AUG 06

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Masonry Wall Design

Description DW2

General Information

Code Ref: ACI 530-02

Wall Height	10.67 ft	Seismic Factor	0.1420	f'm	1,500.0 psi
Parapet Height	0.00 ft	Calc of Em = f'm *	900.00	Fs	24,000.0 psi
		Duration Factor	1.000	Special Inspection	
Thickness	8.0 in	Wall Wt Mult.	1.000	Grout @ Rebar Only	
Rebar Size	5			Normal Weight Block	
Rebar Spacing	48 in			Equivalent	
Depth to Rebar	3.810 in @ Center			Solid Thickness	4.600 in

Loads

Uniform Load		Concentric Axial Load		Wind Load	21.000 psf
Dead Load	327.000 #/ft	Dead Load	0.000 #/ft		
Live Load	385.000 #/ft	Live Load	0.000 #/ft		
Load Eccentricity	0.000 in	Roof Load			
Roof Load					

Design Values

E	1,350,000 psi	Rebar Area	0.078 in2	np	0.03641	j	0.92137
n : Es / Em	21.481	Radius of Gyration	2.535 in	k	0.23590	2 / kj	9.20187
Wall Weight	55.000 psf	Moment of Inertia	354.810 in4				

Max Allow Axial Stress = 0.25 f'm (1-(h/140r)^2) * SpInsp 326.20 psi
Allow Masonry Bending Stress = 0.33 f'm * SpInsp = 495.00 psi
Allow Steel Bending Stress = 24,000.00 psi

Load Combination & Stress Details Summary

	Moment	Axial Load	Bending Stresses Steel	Bending Stresses Masonry	Axial Compression
Top of Wall	in-#	lbs	psi	psi	psi
DL + LL	0.0	712.0	0.0	0.0	12.90
DL + LL + Wind	0.0	327.0	0.0	0.0	5.92
DL + LL + Seismic	0.0	327.0	0.0	0.0	5.92
Between Base & Top of Wall					
DL + LL	0.0	1,005.4	0.0	0.0	18.21
DL + LL + Wind	3,586.2	620.4	13,181.9	189.4	11.24
DL + LL + Seismic	1,333.7	620.4	4,902.4	70.5	11.24

Summary

10.67ft high wall with 0.00ft parapet, Normal Block w/ 8.00in wall w/ #5 bars at 48.00ino.c. at center

Max. Bending Compressive Stress . . . . . 200.68 OK
Allowable . . . . . 495.00 **OK**

Max. Axial Only Compressive Stress . . . . . 18.21 psi
Allowable . . . . . 326.20 **OK**

Max Steel Bending Stress . . . . . 13,181.94 psi
Allowable . . . . . 24,000.00 **OK**

Description DW2

Final Loads & Moments

Wall Weight moment @ Mid Ht	293.43	lbs	Wind Moment @ Mid Ht	3,586.24	in-#
			Seismic Moment @ Mid Ht	1,333.74	in-#
Dead Load Moment @ Top of Wall	0.00	in-#			
Dead Load Moment @ Mid Ht	0.00	in-#	Total Dead Load	327.00	lbs
			Total Live Load	385.00	lbs
Live Load Moment @ Top of Wall	0.00	in-#			
LiveLoad Moment @ Mid Ht	0.00	in-#			

Maximum Allow Moment for Applied Axial Load = 6,529.37 in-#
Maximum Allow Axial Load for Applied Moment = 18,006.30 lbs

Masonry Wall Design

Description DW3

General Information

Code Ref: ACI 530-02

Wall Height	23.00 ft	Seismic Factor	0.1420	f'm	1,500.0 psi
Parapet Height	0.00 ft	Calc of Em = f'm *	900.00	Fs	24,000.0 psi
		Duration Factor	1.000	Special Inspection	
Thickness	8.0 in	Wall Wt Mult.	1.000	Grout @ Rebar Only	
Rebar Size	5			Normal Weight Block	
Rebar Spacing	16 in			Equivalent	
Depth to Rebar	5.250 in @ Edge			Solid Thickness	5.800 in

Loads

Uniform Load		Concentric Axial Load		Wind Load	23.400 psf
Dead Load	57.000 #/ft	Dead Load	0.000 #/ft		
Live Load	67.000 #/ft	Live Load	0.000 #/ft		
Load Eccentricity	0.000 in	Roof Load			
Roof Load					

Design Values

E	1,350,000 psi	Rebar Area	0.233 in2	np	0.07928	j	0.89109
n : Es / Em	21.481	Radius of Gyration	2.368 in	k	0.32673	2 / kj	6.86948
Wall Weight	66.000 psf	Moment of Inertia	390.210 in4				

Max Allow Axial Stress = 0.25 f'm (1-(h/140r)^2) * SpInsp 135.24 psi
Allow Masonry Bending Stress = 0.33 f'm * SpInsp = 495.00 psi
Allow Steel Bending Stress = 24,000.00 psi

Load Combination & Stress Details Summary

	Moment in-#	Axial Load lbs	Bending Stresses Steel psi	Bending Stresses Masonry psi	Axial Compression psi
Top of Wall					
DL + LL	0.0	124.0	0.0	0.0	1.78
DL + LL + Wind	0.0	57.0	0.0	0.0	0.82
DL + LL + Seismic	0.0	57.0	0.0	0.0	0.82
Between Base & Top of Wall					
DL + LL	0.0	883.0	0.0	0.0	12.69
DL + LL + Wind	18,567.9	816.0	17,071.0	385.6	11.72
DL + LL + Seismic	7,436.7	816.0	6,837.1	154.5	11.72

Summary

23.00ft high wall with 0.00ft parapet, Normal Block w/ 8.00in wall w/ #5 bars at 16.00ino.c. at edge

Max. Bending Compressive Stress . . . . . 397.37 OK
Allowable . . . . . 495.00 **OK**

Max. Axial Only Compressive Stress . . . . . 12.69 psi
Allowable . . . . . 135.24 **OK**

Max Steel Bending Stress . . . . . 17,070.97 psi
Allowable . . . . . 24,000.00 **OK**

Description DW3

Final Loads & Moments

Wall Weight moment @ Mid Ht	759.00	lbs	Wind Moment @ Mid Ht	18,567.89	in-#
			Seismic Moment @ Mid Ht	7,436.68	in-#
Dead Load Moment @ Top of Wall	0.00	in-#			
Dead Load Moment @ Mid Ht	0.00	in-#	Total Dead Load	57.00	lbs
			Total Live Load	67.00	lbs
Live Load Moment @ Top of Wall	0.00	in-#			
LiveLoad Moment @ Mid Ht	0.00	in-#			

Maximum Allow Moment for Applied Axial Load = 23,833.14 in-#
Maximum Allow Axial Load for Applied Moment = 9,412.56 lbs

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Description :
Scope :

Job # 06202
Date: 5:30PM, 16 AUG 06

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Masonry Wall Design

Description DW4

General Information

Code Ref: ACI 530-02

Wall Height	22.67 ft	Seismic Factor	0.1420	f'm	1,500.0 psi
Parapet Height	3.50 ft	Calc of Em = f'm *	900.00	Fs	24,000.0 psi
		Duration Factor	1.000	Special Inspection	
Thickness	12.0 in	Wall Wt Mult.	1.000	Grout @ Rebar Only	
Rebar Size	6			Normal Weight Block	
Rebar Spacing	16 in			Equivalent	
Depth to Rebar	5.750 in @ Center			Solid Thickness	8.500 in

Loads

Uniform Load		Concentric Axial Load		Wind Load	23.400 psf
Dead Load	34.000 #/ft	Dead Load	0.000 #/ft		
Live Load	40.000 #/ft	Live Load	0.000 #/ft		
Load Eccentricity	8.310 in	Roof Load			
Roof Load					

Design Values

E	1,350,000 psi	Rebar Area	0.330 in2	np	0.10274	j	0.87932
n : Es / Em	21.481	Radius of Gyration	3.605 in	k	0.36205	2 / kj	6.28222
Wall Weight	103.000 psf	Moment of Inertia	1,325.390 in4				

Max Allow Axial Stress = 0.25 f'm (1-(h/140r)^2) * SpInsp	266.03 psi
Allow Masonry Bending Stress = 0.33 f'm * SpInsp =	495.00 psi
Allow Steel Bending Stress =	24,000.00 psi

Load Combination & Stress Details Summary

	Moment in-#	Axial Load lbs	Bending Stresses Steel psi	Bending Stresses Masonry psi	Axial Compression psi
Top of Wall					
DL + LL	614.9	434.5	368.6	9.7	4.26
DL + LL + Wind	2,002.4	394.5	1,200.1	31.7	3.87
DL + LL + Seismic	1,357.6	394.5	813.6	21.5	3.87
Between Base & Top of Wall					
DL + LL	307.5	1,602.0	184.3	4.9	15.71
DL + LL + Wind	17,326.6	1,562.0	10,384.5	274.4	15.31
DL + LL + Seismic	10,882.1	1,562.0	6,522.1	172.3	15.31

Summary

22.67ft high wall with 3.50ft parapet, Normal Block w/ 12.00in wall w/ #6 bars at 16.00ino.c. at center

Max. Bending Compressive Stress	289.67	OK
Allowable	495.00	**OK**
Max. Axial Only Compressive Stress	15.71	psi
Allowable	266.03	**OK**
Max Steel Bending Stress	10,384.52	psi
Allowable	24,000.00	**OK**

Description DW4

Final Loads & Moments

Wall Weight moment @ Mid Ht	1,528.01	lbs	Wind Moment @ Mid Ht	17,189.20	in-#
			Seismic Moment @ Mid Ht	10,743.99	in-#
Dead Load Moment @ Top of Wall	282.54	in-#			
Dead Load Moment @ Mid Ht	141.27	in-#	Total Dead Load	34.00	lbs
			Total Live Load	40.00	lbs
Live Load Moment @ Top of Wall	332.40	in-#			
LiveLoad Moment @ Mid Ht	166.20	in-#			

Maximum Allow Moment for Applied Axial Load = 31,261.43 in-#
Maximum Allow Axial Load for Applied Moment = 27,135.31 lbs

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Description :
Scope :

Job # 06202
Date: 5:52PM, 16 AUG 06

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Masonry Wall Design

Description DW5

General Information

Code Ref: ACI 530-02

Wall Height	22.67 ft	Seismic Factor	0.1420	f'm	1,500.0 psi
Parapet Height	3.50 ft	Calc of Em = f'm *	900.00	Fs	24,000.0 psi
		Duration Factor	1.000	Special Inspection	
Thickness	8.0 in	Wall Wt Mult.	1.000	Grout @ Rebar Only	
Rebar Size	6			Normal Weight Block	
Rebar Spacing	16 in			Equivalent	
Depth to Rebar	5.250 in @ Edge			Solid Thickness	5.800 in

Loads

Uniform Load		Concentric Axial Load		Wind Load	23.400 psf
Dead Load	68.000 #/ft	Dead Load	0.000 #/ft		
Live Load	80.000 #/ft	Live Load	0.000 #/ft		
Load Eccentricity	6.440 in	Roof Load			
Roof Load					

Design Values

E	1,350,000 psi	Rebar Area	0.330 in2	np	0.11252	j	0.87499
n : Es / Em	21.481	Radius of Gyration	2.368 in	k	0.37503	2 / kj	6.09484
Wall Weight	66.000 psf	Moment of Inertia	390.210 in4				

Max Allow Axial Stress = 0.25 f'm (1-(h/140r)^2) * SpInsp 139.20 psi
Allow Masonry Bending Stress = 0.33 f'm * SpInsp = 495.00 psi
Allow Steel Bending Stress = 24,000.00 psi

Load Combination & Stress Details Summary

	Moment	Axial Load	Bending Stresses Steel	Bending Stresses Masonry	Axial Compression
Top of Wall	in-#	lbs	psi	psi	psi
DL + LL	953.1	379.0	628.7	17.6	5.45
DL + LL + Wind	2,157.8	299.0	1,423.4	39.8	4.30
DL + LL + Seismic	1,126.8	299.0	743.3	20.8	4.30
Between Base & Top of Wall					
DL + LL	476.6	1,127.1	314.4	8.8	16.19
DL + LL + Wind	17,403.2	1,047.1	11,480.3	320.7	15.04
DL + LL + Seismic	7,099.7	1,047.1	4,683.4	130.8	15.04

Summary

22.67ft high wall with 3.50ft parapet, Normal Block w/ 8.00in wall w/ #6 bars at 16.00ino.c. at edge

Max. Bending Compressive Stress . . . . . 335.74 OK
Allowable . . . . . 495.00 **OK**

Max. Axial Only Compressive Stress . . . . . 16.19 psi
Allowable . . . . . 139.20 **OK**

Max Steel Bending Stress . . . . . 11,480.31 psi
Allowable . . . . . 24,000.00 **OK**

Description DW5

Final Loads & Moments

Wall Weight moment @ Mid Ht	979.11	lbs	Wind Moment @ Mid Ht	17,189.20	in-#
			Seismic Moment @ Mid Ht	6,884.50	in-#
Dead Load Moment @ Top of Wall	437.92	in-#			
Dead Load Moment @ Mid Ht	218.96	in-#	Total Dead Load	68.00	lbs
			Total Live Load	80.00	lbs
Live Load Moment @ Top of Wall	515.20	in-#			
LiveLoad Moment @ Mid Ht	257.60	in-#			

Maximum Allow Moment for Applied Axial Load = 26,862.25 in-#
Maximum Allow Axial Load for Applied Moment = 9,688.58 lbs

Combined Axial and Lateral Load Tables

15 psf Lateral Load

Wall Height (ft)	Spacing (in.) o.c.	400S137-(mils) 33 ksi 33	43	50 ksi 54	68	400S162-(mils) 33 ksi 33	43	50 ksi 54	68	400S200-(mils) 33 ksi 33	43	50 ksi 54	68	550S162-(mils) 33 ksi 33	43	50 ksi 54	68
8	12	1.79	2.49	3.77	5.08	2.22	3.05	4.73	6.25	2.63	3.76	5.92	7.72	2.53	3.49	5.74	7.58
	16	1.70	2.49	3.77	5.08	2.21	3.05	4.73	6.25	2.63	3.76	5.92	7.72	2.53	3.49	5.74	7.58
	24	1.39 [7]	2.34	3.77	5.08	1.86 [7]	3.00	4.73	6.25	2.30	3.74	5.92	7.72	2.53	3.49	5.74	7.58
9	12	1.68	2.43	3.65	4.90	2.16	2.97	4.54	6.00	2.57	3.65	5.65	7.36	2.51	3.47	5.67	7.58
	16	1.48 [7]	2.40	3.65	4.90	1.94	2.97	4.54	6.00	2.38	3.65	5.65	7.36	2.51	3.47	5.67	7.58
	24	1.10 [6]	2.02 [6]	3.65 [7]	4.90	1.52 [6]	2.63 [7]	4.54	6.00	1.92 [6]	3.27	5.65	7.36	2.35	3.47	5.67	7.58
10	12	1.47 [7]	2.37	3.50	4.67	1.91	2.88	4.32	5.69	2.35	3.52	5.34	6.95	2.48	3.43	5.58	7.47
	16	1.24 [6]	2.13 [7]	3.50	4.67	1.66 [6]	2.71 [7]	4.32	5.69	2.06 [7]	3.35	5.34	6.95	2.48	3.43	5.58	7.47
	24	0.80 [3]	1.68 [6]	3.34 [6]	4.67 [7]	1.17 [6]	2.22 [6]	4.14 [6]	5.69 [7]	1.53 [6]	2.78 [6]	5.23 [7]	6.95	2.08	3.39	5.58	7.47
12	12	1.04 [6]	1.84 [6]	3.13 [6]	4.08 [7]	1.39 [6]	2.34 [6]	3.80 [7]	4.92	1.76 [6]	2.88 [7]	4.66	5.99	2.30	3.32	5.32	7.13
	16	0.75 [3]	1.54 [3]	2.92 [6]	4.08 [6]	1.08 [3]	2.01 [6]	3.57 [6]	4.92 [7]	1.41 [6]	2.49 [6]	4.47 [6]	5.99	2.02 [7]	3.27	5.32	7.13
	24	0.25 [2]	1.00 [2]	2.43 [3]	3.57 [3]	0.52 [2]	1.42 [3]	2.98 [3]	4.48 [6]	0.77 [3]	1.80 [3]	3.79 [6]	5.66 [6]	1.49 [6]	2.74 [6]	5.30 [7]	7.13
14	12	0.63 [2]	1.32 [3]	2.44 [6]	3.36 [6]	0.90 [3]	1.71 [3]	2.93 [6]	4.07 [6]	1.20 [3]	2.12 [6]	3.64 [6]	4.93 [7]	1.86 [6]	3.03	4.96	6.64
	16	0.33 [2]	0.99 [2]	2.14 [3]	3.05 [3]	0.57 [2]	1.36 [3]	2.58 [3]	3.76 [6]	0.81 [2]	1.69 [3]	3.24 [6]	4.71 [6]	1.51 [6]	2.68 [6]	4.96 [7]	6.64
	24		0.41 [2]	1.61 [2]	2.48 [2]		0.73 [2]	1.97 [2]	3.14 [3]	0.15 [2]	0.96 [2]	2.54 [3]	4.02 [3]	0.88 [3]	2.03 [6]	4.27 [6]	6.47 [6]
16	12	0.30 [2]	0.86 [2]	1.80 [3]	2.51 [3]	0.50 [2]	1.17 [2]	2.15 [3]	3.08 [3]	0.71 [2]	1.46 [3]	2.68 [3]	3.84 [6]	1.40 [6]	2.48 [6]	4.44 [6]	6.02
	16		0.53 [2]	1.50 [2]	2.19 [2]	0.17 [2]	0.81 [2]	1.80 [2]	2.73 [3]	0.33 [2]	1.04 [2]	2.28 [3]	3.45 [3]	1.01 [3]	2.07 [6]	4.01 [6]	5.97 [6]
	24			0.98 [2]	1.64 [2]		0.19 [1]	1.21 [2]	2.13 [2]		0.31 [2]	1.60 [2]	2.78 [2]	0.32 [2]	1.33 [3]	3.24 [3]	5.16 [6]

15 psf Lateral Load

Wall Height (ft)	Spacing (in.) o.c.	600S137-(mils) 33 ksi 33	43	50 ksi 54	68	97	600S162-(mils) 33 ksi 33	43	50 ksi 54	68	97	600S200-(mils) 33 ksi 33	43	50 ksi 54	68	97
8	12	1.88	2.62	3.90	5.14	7.62	2.55	3.52	5.72	7.56	11.50	2.99	4.44	7.56	10.02	15.60
	16	1.88	2.62	3.90	5.14	7.62	2.55	3.52	5.72	7.56	11.50	2.99	4.44	7.56	10.02	15.60
	24	1.88	2.62	3.90	5.14	7.62	2.55	3.52	5.72	7.56	11.50	2.99	4.44	7.56	10.02	15.60
9	12	1.88	2.62	3.90	5.14	7.62	2.55	3.52	5.72	7.56	11.50	2.97	4.40	7.46	9.89	15.38
	16	1.88	2.62	3.90	5.14	7.62	2.55	3.52	5.72	7.56	11.50	2.97	4.40	7.46	9.89	15.38
	24	1.78	2.62	3.90	5.14	7.62	2.52	3.52	5.72	7.56	11.50	2.97	4.40	7.46	9.89	15.38
10	12	1.88	2.62	3.90	5.14	7.62	2.54	3.52	5.72	7.56	11.50	2.95	4.35	7.33	9.72	15.11
	16	1.88	2.62	3.90	5.14	7.62	2.54	3.52	5.72	7.56	11.50	2.95	4.35	7.33	9.72	15.11
	24	1.61	2.62	3.90	5.14	7.62	2.29	3.52	5.72	7.56	11.50	2.74	4.35	7.33	9.72	15.11
12	12	1.83	2.62	3.90	5.14	7.62	2.49	3.44	5.60	7.52	11.50	2.88	4.22	6.99	9.27	14.40
	16	1.61	2.62	3.90	5.14	7.62	2.24	3.44	5.60	7.52	11.50	2.67	4.22	6.99	9.27	14.40
	24	1.20 [6]	2.21 [7]	3.90	5.14	7.62	1.75 [6]	3.06 [7]	5.60	7.52	11.50	2.14 [7]	3.79	6.99	9.27	14.40
14	12	1.56 [7]	2.56	3.90	5.14	7.62	2.10 [7]	3.33	5.32	7.13	11.28	2.51	4.04	6.55	8.68	13.45
	16	1.28 [6]	2.27 [6]	3.90	5.14	7.62	1.78 [6]	3.02 [7]	5.32	7.13	11.28	2.15 [6]	3.70	6.55	8.68	13.45
	24	0.75 [3]	1.73 [6]	3.64 [6]	5.14 [6]	7.62	1.17 [3]	2.40 [6]	4.91 [6]	7.13 [7]	11.28	1.49 [6]	2.97 [6]	6.18 [7]	8.68	13.45
16	12	1.26 [6]	2.24 [6]	3.90 [7]	5.14	7.62	1.67 [6]	2.84 [6]	4.94	6.62	10.42	2.03 [6]	3.46 [7]	6.01	7.97	12.32
	16	0.91 [3]	1.88 [6]	3.73 [6]	5.14 [6]	7.62	1.29 [3]	2.44 [6]	4.69 [6]	6.62 [7]	10.42	1.61 [6]	2.99 [6]	5.82 [7]	7.97	12.32
	24	0.28 [2]	1.22 [3]	3.11 [3]	4.62 [6]	7.62 [6]	0.60 [2]	1.72 [3]	3.91 [6]	6.08 [6]	10.42 [7]	0.84 [3]	2.14 [3]	4.92 [6]	7.56 [6]	12.32 [7]

15 psf Lateral Load

Wall Height (ft)	Spacing (in.) o.c.	800S137-(mils) 33 ksi 33	43	50 ksi 54	68	97	800S162-(mils) 33 ksi 33	43	50 ksi 54	68	97	800S200-(mils) 33 ksi 33	43	50 ksi 54	68	97
8	12	1.79	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
	16	1.79	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
	24	1.79	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
9	12	1.79	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
	16	1.79	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
	24	1.79	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
10	12	1.79	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
	16	1.79	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
	24	1.75	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
12	12	1.79	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
	16	1.76	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.07	4.58	7.85	10.38	16.07
	24	1.46	2.49	3.62	4.80	7.27	2.17	3.44	5.51	7.32	11.33	2.76	4.58	7.85	10.38	16.07
14	12	1.74	2.50	3.62	4.80	7.27	2.49	3.44	5.51	7.32	11.33	3.03	4.51	7.70	10.25	16.07
	16	1.53	2.50	3.62	4.80	7.27	2.25	3.44	5.51	7.32	11.33	2.82	4.51	7.70	10.25	16.07
	24	1.12 [7]	2.17	3.62	4.80	7.27	1.75 [7]	3.13	5.51	7.32	11.33	2.24	4.27	7.70	10.25	16.07
16	12	1.53	2.50	3.62	4.80	7.27	2.24	3.44	5.51	7.32	11.33	2.75	4.40	7.41	9.87	15.52
	16	1.26 [7]	2.29	3.62	4.80	7.27	1.91 [7]	3.28	5.51	7.32	11.33	2.38	4.33	7.41	9.87	15.52
	24	0.74 [6]	1.81 [6]	3.56 [7]	4.80	7.27	1.27 [6]	2.67 [6]	5.51	7.32	11.33	1.66 [6]	3.62 [7]	7.41	9.87	15.52

1 Deflection exceeds L/120
2 Deflection exceeds L/240
3 Deflection exceeds L/360
6 Deflection exceeds L/600
7 Deflection exceeds L/720
-- If not noted, deflection is less than L/720

See Combined Axial and Lateral Load Table Notes on page 20.

Trus Joist
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Beam® 6.20 Serial Number: 7005122663
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Page 1 Engine Version: 6.20.16

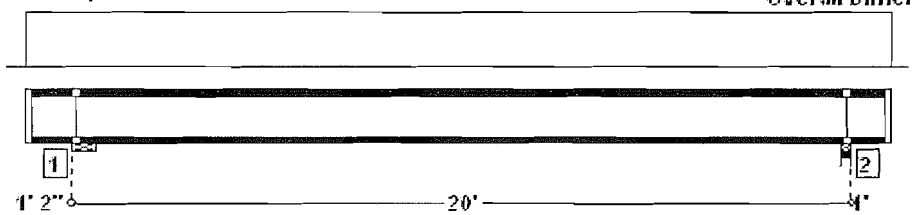
DJ1

14" TJI® 110 @ 24" o/c

39

THIS PRODUCT MEETS OR EXCEEDS THE SET DESIGN CONTROLS FOR THE APPLICATION AND LOADS LISTED

Member Slope: 0/12 Roof Slope0/12 **Overall Dimension: 22' 2"**



All dimensions are horizontal. **Product Diagram is Conceptual.**

)ADS:

alysis is for a Joist Member.

Primary Load Group - Roof (psf): 20.0 Live at 125 % duration, 17.0 Dead

^'JPPORTS:

		Input Width	Bearing Length	Vertical Reactions (lbs) Live/Dead/Uplift/Total	Detail	Other
1	Plate on masonry wall	7.63"	7.63"	452 / 383 / 0 / 835	Overhang	1 Ply 14" TJI® 110
	Stud wall	3.50"	3.50"	437 / 370 / 0 / 807	Overhang	1 Ply 14" TJI® 110

DESIGN CONTROLS:

	Maximum	Design	Control	Control	Location
ıear (lbs)	725	-678	2325	Passed (29%)	Rt. end Span 1 under Roof ADJACENT span loading
ertical Reaction (lbs)	807	807	2419	Passed (33%)	Bearing 2 under Roof ADJACENT span loading
Moment (Ft-Lbs)	3483	3483	4456	Passed (78%)	MID Span 1 under Roof ALTERNATE span loading
've Load Defl (in)		0.398	0.651	Passed (L/589)	MID Span 1 under Roof ALTERNATE span loading
ıtal Load Defl (in)		0.733	0.977	Passed (L/320)	MID Span 1 under Roof ALTERNATE span loading

-Deflection Criteria: STANDARD(LL:L/360,TL:L/240).

-TJ maximum bearing length controls reaction capacity. Limits: End supports, 3 1/2". Intermediate supports, 5 1/4".

- acing(Lu): All compression edges (top and bottom) must be braced at 2' 9" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve ı mber stability.

- ıne load conditions considered in this design analysis include alternate and adjacent member pattern loading.

^DDITIONAL NOTES:

- PORTANT! The analysis presented is output from software developed by Trus Joist (TJ). TJ warrants the sizing of its products by this software will be accomplished in accordance ı ... ı TJ product design criteria and code accepted design values. The specific product application, input design loads, and stated dimensions have been provided by the software user. This output has not been reviewed by a TJ Associate.

-Not all products are readily available. Check with your supplier or TJ technical representative for product availability.

- IIS ANALYSIS FOR TRUS JOIST PRODUCTS ONLY! PRODUCT SUBSTITUTION VOIDS THIS ANALYSIS.

- owable Stress Design methodology was used for Building Code IBC analyzing the TJ Custom product listed above.

PROJECT INFORMATION:

Silverbell National Guard

ɔ No: 06202

OPERATOR INFORMATION:

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Beam® 6.20 Serial Number: 7005116794
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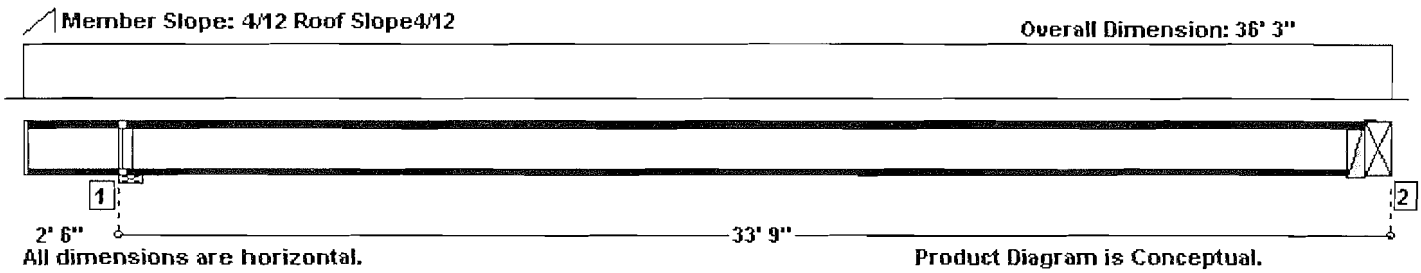
Page 1 Engine Version: 6.20.16

DJ2

20" TJI®/L65 @ 24" o/c

40

THIS PRODUCT MEETS OR EXCEEDS THE SET DESIGN CONTROLS FOR THE APPLICATION AND LOADS LISTED



)ADS:

alysis is for a Joist Member.

Primary Load Group - Roof (psf): 20.0 Live at 125 % duration, 17.0 Dead

)PPORTS:

		Input Width	Bearing Length	Vertical Reactions (lbs) Live/Dead/Uplift/Total	Ply	Depth	Nailing Depth	Detail	Other
	Plate on masonry wall	8.00"	8.00"	772 / 692 / 0 / 1464	N/A	N/A	N/A	Overhang	1 Ply 20" TJI®/L65
2	Glulam or solid sawn lumber beam	8.75"	Hanger	680 / 607 / 0 / 1288	N/A	N/A	N/A	Top Mount Hanger	None

-Web stiffeners are required at support(s): 1, 2. See TJI JOIST INSTALLATION INFORMATION for nailing requirements.

NGERS: No Manufacturer Selected

ıpport		Model	Slope	Skew	Reverse Flanges	Top Flange Offset	Top Flange Slope	Support Wood Species
2	Top Mount Hanger	NONE FOUND	4/12	0	N/A	N/A	N/A	Douglas Fir

DESIGN CONTROLS:

	Maximum	Design	Control	Control	Location
ıear (lbs)	1249	-1233	3425	Passed (36%)	Rt. end Span 1 under Roof ALTERNATE span loading
ertical Reaction (lbs)	1233	1233	3594	Passed (34%)	Bearing 2 under Roof ALTERNATE span loading
Moment (Ft-Lbs)	10017	10017	14425	Passed (69%)	MID Span 1 under Roof ALTERNATE span loading
ve Load Defl (in)		0.781	1.723	Passed (L/530)	MID Span 1 under Roof ALTERNATE span loading
ɔtal Load Defl (in)		1.474	2.297	Passed (L/281)	MID Span 1 under Roof ALTERNATE span loading

-Deflection Criteria: STANDARD(LL:L/240,TL:L/180).

- ! maximum bearing length controls reaction capacity. Limits: End supports, 3 1/2". Intermediate supports, 5 1/4".
- acing(Lu): All compression edges (top and bottom) must be braced at 4' o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.

-The load conditions considered in this design analysis include alternate member pattern loading.

OJECT INFORMATION:

Silverbell National Guard

b# - 06202

OPERATOR INFORMATION:

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TJ-Beam® 6.20 Serial Number: 7005122663
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Page 1 Engine Version: 6.20.16

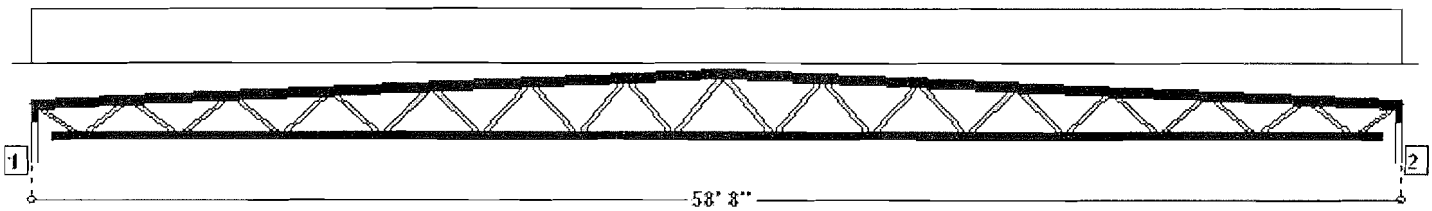
DJ3

28"(brg)-42 5/8"(ridge) TJL Open Web Truss @ 24" o/c

41

THIS PRODUCT MEETS OR EXCEEDS THE SET DESIGN CONTROLS FOR THE APPLICATION AND LOADS LISTED

Left Slope.501/12 Right Slope-.501/12



Product Diagram is Conceptual.

LOADS:

Analysis is for a Joist Member.
Primary Load Group - Roof (psf): 20.0 Live at 125 % duration, 17.0 Dead

SUPPORTS:

		Input Width	Vertical Reactions (lbs) Live/Dead/Uplift/Total	Detail	Other
1	Stud wall	3.50"	1173 / 997 / 0 / 2171	Wall	2x_ Blocking
2	Stud wall	3.50"	1173 / 997 / 0 / 2171	Wall	2x_ Blocking

-Left Support: Top-All, Approx. clip height: 1 5/8", Approx. clip width: 7 3/16", Allowed choice(s): TOP (NO-NOTCH),TOP (U-CLIP)
-Right Support: Top-All, Approx. clip height: 1 5/8", Approx. clip width: 7 3/16", Allowed choice(s): TOP (NO-NOTCH),TOP (U-CLIP)

DESIGN CONTROLS:

	Maximum	Design	Control	Control	Location
Shear (lbs)	2158	2158	2395	Passed (90%)	Rt. end Span 1 under Roof loading
Moment (Ft-Lbs)	31724	30622	30911	Passed (99%)	MID Span 1 under Roof loading
Live Load Defl (in)		1.641	2.917	Passed (L/427)	MID Span 1 under Roof loading
Total Load Defl (in)		3.036	3.889	Passed (L/231)	MID Span 1 under Roof loading

-Deflection Criteria: STANDARD(LL:L/240,TL:L/180).
-Allowable moment was increased for repetitive member usage.
-Bracing(Lu): All compression edges (top and bottom) must be braced at 5' 3" o/c unless detailed otherwise. Proper attachment and positioning of lateral bracing is required to achieve member stability.

ADDITIONAL NOTES:

-IMPORTANT! The analysis presented is output from software developed by Trus Joist (TJ). TJ warrants the sizing of its products by this software will be accomplished in accordance with TJ product design criteria and code accepted design values. The specific product application, input design loads, and stated dimensions have been provided by the software user. This output has not been reviewed by a TJ Associate.
-Not all products are readily available. Check with your supplier or TJ technical representative for product availability.
-THIS ANALYSIS FOR TRUS JOIST PRODUCTS ONLY! PRODUCT SUBSTITUTION VOIDS THIS ANALYSIS.
-Allowable Stress Design methodology was used for Building Code IBC analyzing the TJ Custom product listed above.
-The open web truss analysis presented is approximate. All open web trusses are custom designed to carry the specific design loads for each project. Actual truss capacity when fabricated is limited to that required to resist the specified loads. Do not use this analysis to verify the capacity of existing trusses.
-Pricing Load (plf) = 74

PROJECT INFORMATION:

Silverbell National Guard
Job No: 06202

OPERATOR INFORMATION:

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Description: Bearing

DL 1

Uniform Loads

Dead load	=	170	lb/ft
Live Load	=	200	lb/ft
Lateral Load	=	180	lb/ft

Wood Ledger

Width	=	2.5	in
Depth	=	7.25	in

Anchor bolt

Diameter	=	3/4	in
Spacing	=	24	in

Bolt values for (Douglas Fir-Larch)

Parallel	=	1480	lbs
Perpendicular	=	780	lbs

Duration factors

Dead + live load	=	1.25	
Dead + Short Term	=	1.33	
Dead + Live + Short Term	=	1.33	(Floor load)

Load Combinations

Dead + Live

Vertical Force	=	740	lbs	
Allowable Force	=	975	lbs	**OK**

Dead + Live + Short Term

Vertical Force	=	740	lbs	
Horizontal Force	=	360	lbs	
Resultant Force	=	822.9	lbs	
Angle of Resultant	=	64.06	Deg	
Allow Diagonal force	=	1141	lbs	**OK**

Dead + Short Term

Vertical Force	=	340	lbs	
Horizontal Force	=	360	lbs	
Resultant Force	=	495	lbs	
Angle of Resultant	=	43.4	Deg	
Allow Diagonal force	=	1383	lbs	**OK**

Description: Bearing at Apparatus Bay

DL 2

Uniform Loads

Dead load	=	510	lb/ft
Live Load	=	600	lb/ft
Lateral Load	=	180	lb/ft

Wood Ledger

Width	=	3.5	in
Depth	=	11.25	in

Anchor bolt

Diameter	=	1	in
Spacing	=	16	in

Bolt values for (Douglas Fir-Larch)

Parallel	=	2700	lbs
Perpendicular	=	1300	lbs

Duration factors

Dead + live load	=	1.25	
Dead + Short Term	=	1.33	
Dead + Live + Short Term	=	1.33	(Floor load)

Load Combinations

Dead + Live

Vertical Force	=	1480	lbs	
Allowable Force	=	1625	lbs	**OK**

Dead + Live + Short Term

Vertical Force	=	1480	lbs	
Horizontal Force	=	240	lbs	
Resultant Force	=	1499	lbs	
Angle of Resultant	=	80.79	Deg	
Allow Diagonal force	=	1752	lbs	**OK**

Dead + Short Term

Vertical Force	=	680	lbs	
Horizontal Force	=	240	lbs	
Resultant Force	=	721	lbs	
Angle of Resultant	=	70.6	Deg	
Allow Diagonal force	=	1834	lbs	**OK**

Description: Non-Bearing
DL 3

Uniform Loads

Dead load	=	34	lb/ft
Live Load	=	40	lb/ft
Lateral Load	=	180	lb/ft

Wood Ledger

Width	=	2.5	in
Depth	=	7.25	in

Anchor bolt

Diameter	=	3/4	in
Spacing	=	48	in

Bolt values for (Douglas Fir-Larch)

Parallel	=	1480	lbs
Perpendicular	=	780	lbs

Duration factors

Dead + live load	=	1.25	
Dead + Short Term	=	1.33	
Dead + Live + Short Term	=	1.33	(Floor load)

Load Combinations

Dead + Live

Vertical Force	=	296	lbs	
Allowable Force	=	975	lbs	**OK**

Dead + Live + Short Term

Vertical Force	=	296	lbs	
Horizontal Force	=	720	lbs	
Resultant Force	=	778.5	lbs	
Angle of Resultant	=	22.35	Deg	
Allow Diagonal force	=	1742	lbs	**OK**

Dead + Short Term

Vertical Force	=	136	lbs	
Horizontal Force	=	720	lbs	
Resultant Force	=	733	lbs	
Angle of Resultant	=	10.7	Deg	
Allow Diagonal force	=	1909	lbs	**OK**

FOOTINGS

DF1

$w_{DL} = 508 + 26(103)$
$= 3186$ plf

$w_{LL} = 597$

$w_{TOTAL} = 3783$ plf

$w = 3783/2000 = 1.89$ ft

USE 2'-0" WIDE X 10" DEEP FOOTING W/(3) #4 CONT.

DF2

$w_{DL} = 327 + 10.67(66)$
$= 1032$ plf

$w_{LL} = 385$ plf

$w_{TOTAL} = 1417$ plf

$w = 1417/2000 = 0.71$ ft

USE 1'-6" WIDE X 10" DEEP FOOTING W/(2) #4 CONT.

DF3

$w_{DL} = 57 + 23(66)$
$= 1575$ plf

$w_{LL} = 67$ plf

$w_{TOTAL} = 1642$ plf

$w = 1642/2000 = 0.82$ ft

USE 1'-6" WIDE X 10" DEEP FOOTING W/(2) #4 CONT.

DF4

$w_{DL} = 34 + 26(103)$
$= 2712$ plf

$w_{LL} = 40$ plf

$w_{TOTAL} = 2752$ plf

$w = 2752/2000 = 1.38$ ft

USE 2'-0" WIDE X 10" DEEP FOOTING W/(3) #4 CONT.

JAMBS

12" ① Span = 14'-0" H = 21'-3"

$w_{DL} = (59.67/2)(17)$ $P_{DL} = 508\,(14/2) = 3554^{\#}$

$= 508$ plf

$w_{LL} = (59.67/2)(20)$ $P_{LL} = 597\,(14/2) = 4179^{\#}$

$= 597$ plf

8" ② Span = 4'-0" H = 10'-9"

$w_{DL} = (33.5/2 + 2.5)(17)$ $P_{DL} = 328\,(4/2) = 656^{\#}$

$= 328$ plf

$w_{LL} = (33.5/2 + 2.5)(20)$ $P_{LL} = 385\,(4/2) = 770^{\#}$

$= 385$ plf

8" ③ Span = 3'-4" H = 10'-9"

$w_{DL} = (35.75/2 + 2.33)(17)$ $P_{DL} = 327\,(3.34/2) = 546^{\#}$

$= 327$ plf

$w_{LL} = (33.75/2 + 2.33)(20)$ $P_{LL} = 385\,(3.34/2) = 643^{\#}$

$= 385$ plf

12" ④ Span = 3'-4" H = 22'-8"

$w_{DL} = (2)(17)$ $P_{DL} = 34\,(3.34/2) = 57^{\#}$

$= 34$ plf

$w_{LL} = (2)(40)$ $P_{LL} = 40\,(3.34/2) = 67^{\#}$

$= 80$ plf

8" ⑤ Span = 3'-4" H = 23'-0"

$w_{DL} = 4(17)$ $P_{DL} = 68\,(3.34/2) = 114^{\#}$

$= 68$ plf

$w_{LL} = 4(20)$ $P_{LL} = 80\,(3.34/2) = 134^{\#}$

$= 80$ plf

8" ⑥ Span = 6'-10" H = 11'-8"

$w_{DL} = (15/2)(17)$ $P_{DL} = 128\,(10.84/2) = 694^{\#}$

$= 128$ plf

$w_{LL} = (15/2)(20)$ $P_{LL} = 150\,(10.84/2) = 813^{\#}$

$= 150$ plf

JAMBS (CONT.)

8" (2) Span = 3'-4"

$w_{DL} = (2 + 1.33)(17)$
$= 57$ plf

$w_{LL} = (2 + 1.33)(20)$
$= 67$ plf

$P_{DL} = 57\left(3.34/2\right) = 96^{\#}$

$P_{LL} = 67\left(3.34/2\right) = 112^{\#}$

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PROJECT:	Sopori Gymnasium	SHEET NO:	51
JOB NO:	06233	DATE:	Jul-06
		BY:	MAM

Jamb 7 (3'-4" Opening)

Width =	1.33	Ft	Roof PDL	96	lbs	p =	0.007316
Height=	20	ft	P LL	112	lbs	np=	0.157155
Opening Width	3.34	ft				k=	0.425089
Wall Rein.	1.33	ft				j=	0.858304
Wind psf=	22.4	psf	**Results :**			2/jk=	5.481625
wind load	82.096	lb/ft	M=	4104.8	ft-lb	F'm=	1500 psi
			fb=	600.0	psi	S.I. (y/n)	y
Wall thickness	7.625	in	fs=	17432	psi		1
Rebar area	0.31	in^2	fa dl=	0.8	psi	Fb=	500 psi
# each face	2		fa ll =	0.9	psi	r=	2.201148 in
total area	0.62	in^2				h/r=	109.034
d =	5.31	in	fb/Fb=	1.20003		Fa=	154.5622 psi
Fs=	24000	psi	fs/Fs=	0.72633		Em =	1350000 PSI
						Es =	2.90E+07 PSI
						n=	21.48

Combined loads

DL + LL =	0.011	<=	1.33	**OK**
0.6DL + W =	1.203	<=	1.33	**OK**
DL+LL+W	1.211	<=	1.33	**OK**

use 16" jamb with 2 #5 each cell = (4) bars Total (special inspection is required)

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Title : Silverbell National Guard
Dsgnr: Mark Medina
Description :
Scope :

Job # 06202
Date: 3:16PM, 17 AUG 06

S2

Steel Beam Design

Description Trellis Beam

General Information

Code Ref: AISC 9th ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Steel Section : HSS4X4X1/8

		Pinned-Pinned	Fy	46.00 ksi
Center Span	5.00 ft	Bm Wt. Added to Loads	Load Duration Factor	1.00
Left Cant.	0.00 ft	LL & ST Act Together	Elastic Modulus	29,000.0 ksi
Right Cant	1.00 ft			
Lu : Unbraced Length	5.00 ft			

Distributed Loads

Note! Short Term Loads Are WIND Loads.

	# 1	# 2	# 3	# 4	# 5	# 6	# 7	
DL	0.108							k/ft
LL	0.215							k/ft
ST								k/ft
Start Location								ft
End Location								ft

Summary

Beam OK
Static Load Case Governs Stress

Using: HSS4X4X1/8 section, Span = 5.00ft, Fy = 46.0ksi, Left Cant. = 0.00ft, Right Cant. = 1.00ft
End Fixity = Pinned-Pinned, Lu = 5.00ft, LDF = 1.000

	Actual	Allowable		
Moment	1.000 k-ft	5.060 k-ft	Max. Deflection	-0.035 in
fb : Bending Stress	5.454 ksi	27.600 ksi	Length/DL Defl	3,562.7 : 1
fb / Fb	**0.198 : 1**		Length/(DL+LL Defl)	1,077.1 : 1
Shear	0.834 k	17.075 k		
fv : Shear Stress	0.899 ksi	18.400 ksi		
fv / Fv	**0.049 : 1**			

Force & Stress Summary

<<-- These columns are Dead + Live Load placed as noted -->>

	Maximum	DL Only	LL @ Center	LL+ST @ Center	LL @ Cants	LL+ST @ Cants	
Max. M +	1.00 k-ft	0.33	1.00		0.28		k-ft
Max. M -		-0.06	-0.06		-0.16		k-ft
Max. M @ Left							k-ft
Max. M @ Right		-0.06	-0.06		-0.16		k-ft
Shear @ Left	0.81 k	0.27	0.81		0.25		k
Shear @ Right	0.83 k	0.30	0.83		0.32		k
Center Defl.	-0.035 in	-0.011	-0.035	-0.035	-0.009	-0.009	in
Left Cant Defl	0.000 in	0.000	0.000	0.000	0.000	0.000	in
Right Cant Defl	0.022 in	0.007	0.022	0.022	0.004	0.004	in
...Query Defl @	0.000 ft	0.000	0.000	0.000	0.000	0.000	in
Reaction @ Left	0.81	0.27	0.81	0.81	0.25	0.25	k
Reaction @ Rt	1.18	0.41	0.95	0.95	0.65	0.65	k

Fa calc'd per Eq. E2-1, K*L/r < Cc

Section Properties HSS4X4X1/8

Depth	4.000 in	Weight	6.01 #/ft
Web Thick	0.116 in	Ixx	4.400 in4
Width	4.000 in	Iyy	4.400 in4
Flange Thick	0.116 in	Sxx	2.200 in3
Area	1.77 in2	Syy	2.200 in3
Rt	0.000 in	R-xx	1.580 in
Values for LRFD Design....		R-yy	1.580 in
J	6.910 in4	Zx	2.560 in3
Cw	3.49 in6	Zy	2.560 in3

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Job # 06202
Date: 3:16PM, 17 AUG 06

53

Multi-Span Steel Beam

Description Trellis Joist

General Information

Code Ref: AISC 9th ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Fy - Yield Stress 46.00 ksi Load Duration Factor 1.00

Spans Considered Continuous Over Supports

Span Information

Description		Span 1	Span 2
Span	ft	9.00	8.00
Steel Section		HSS4X2X1/8	HSS4X2X1/8
End Fixity		Pin-Pin	Pin-Pin
Unbraced Length	ft	9.00	8.00

Loads

Live Load Used This Span ?		Yes	Yes
Dead Load	k/ft	0.010	0.010
Live Load	k/ft	0.020	0.020

Results

Mmax @ Cntr	k-ft	0.18	0.12
@ X =	ft	3.48	5.12
Max @ Left End	k-ft	0.00	-0.27
Max @ Right End	k-ft	-0.27	0.00
fb : Actual	psi	2,479.2	2,479.2
Fb : Allowable	psi	27,600.0	27,600.0
		Bending OK	Bending OK
fv : Actual	psi	178.3	166.2
Fv : Allowable	psi	18,400.0	18,400.0

Reactions & Deflections

Shear @ Left	k	0.10	0.15
Shear @ Right	k	0.17	0.09
Reactions...			
DL @ Left	k	0.03	0.11
LL @ Left	k	0.07	0.21
Total @ Left	k	0.10	0.32
DL @ Right	k	0.11	0.03
LL @ Right	k	0.21	0.06
Total @ Right	k	0.32	0.09
Max. Deflection	in	-0.027	-0.012
@ X =	ft	3.90	4.80
Span/Deflection Ratio		3,977.8	7,870.2

Query Values

Location	ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Shear	k	0.10	0.15	0.00	0.00	0.00	0.00	0.00	0.00
Moment	k-ft	0.00	-0.27	0.00	0.00	0.00	0.00	0.00	0.00
Max. Deflection	in	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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Scope :

Job # 06202
Date: 3:16PM, 17 AUG 06

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Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Rev: 580016
User: KW-0605362, Ver 5.8.0, 1-Dec-2003
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Cantilevered Retaining Wall Design

Description Site Wall (H = 6'-0")

Criteria

Retained Height	=	0.67 ft
Wall height above soil	=	6.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	6.00 in
Soil Density	=	110.00 pcf
Wind on Stem	=	13.0 psf

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	35.0
Toe Active Pressure	=	0.0
Passive Pressure	=	250.0
Water height over heel	=	0.0 ft
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

f'c = 2,500 psi Fy = 40,000 psi

Min. As %	=	0.0014
Toe Width	=	0.67 ft
Heel Width	=	1.33
Total Footing Width	=	2.00
Footing Thickness	=	10.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

Cover @ Top = 3.00 in @ Btm.= 3.00 in

Design Summary

Total Bearing Load	=	714 lbs
...resultant ecc.	=	5.91 in
Soil Pressure @ Toe	=	938 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	2,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	1,293 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	6.5 psi OK
Footing Shear @ Heel	=	2.6 psi OK
Allowable	=	85.0 psi

Wall Stability Ratios

Overturning	=	1.98 OK
Sliding	=	3.71 (Vertical Co

Sliding Calcs (Vertical Component Used)

Lateral Sliding Force	=	117.6 lbs
less 100% Passive Force	= -	222.2 lbs
less 100% Friction Force	= -	214.1 lbs
Added Force Req'd	=	0.0 lbs OK
...for 1.5:1 Stability	=	0.0 lbs OK

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,293	0 psf
Mu' : Upward	=	367	0 ft-#
Mu' : Downward	=	88	109 ft-#
Mu: Design	=	280	109 ft-#
Actual 1-Way Shear	=	6.48	2.62 psi
Allow 1-Way Shear	=	85.00	85.00 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Stem Construction

Top Stem

		Stem OK
Design height	ft =	0.00
Wall Material Above "Ht"	=	Masonry
Thickness	=	8.00
Rebar Size	=	# 4
Rebar Spacing	=	48.00
Rebar Placed at	=	Center
Design Data		
fb/FB + fa/Fa	=	0.976
Total Force @ Section	lbs =	85.9
Moment....Actual	ft-# =	288.0
Moment.....Allowable	=	295.2
Shear.....Actual	psi =	3.2
Shear.....Allowable	psi =	19.4
Bar Develop ABOVE Ht.	in =	20.00
Bar Lap/Hook BELOW Ht.	in =	6.00
Wall Weight	=	55.0
Rebar Depth 'd'	in =	3.81
Masonry Data		
f'm	psi =	1,500
Fs	psi =	20,000
Solid Grouting	=	No
Special Inspection	=	No
Modular Ratio 'n'	=	25.78
Short Term Factor	=	1.000
Equiv. Solid Thick.	in =	4.60
Masonry Block Type =		
Concrete Data		
f'c	psi =	
Fy	psi =	

Other Acceptable Sizes & Spacings

Toe:
Heel:
Key:

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Job # 06202
Date: 3:16PM, 17 AUG 06

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Cantilevered Retaining Wall Design

Description Site Wall (H = 6'-0")

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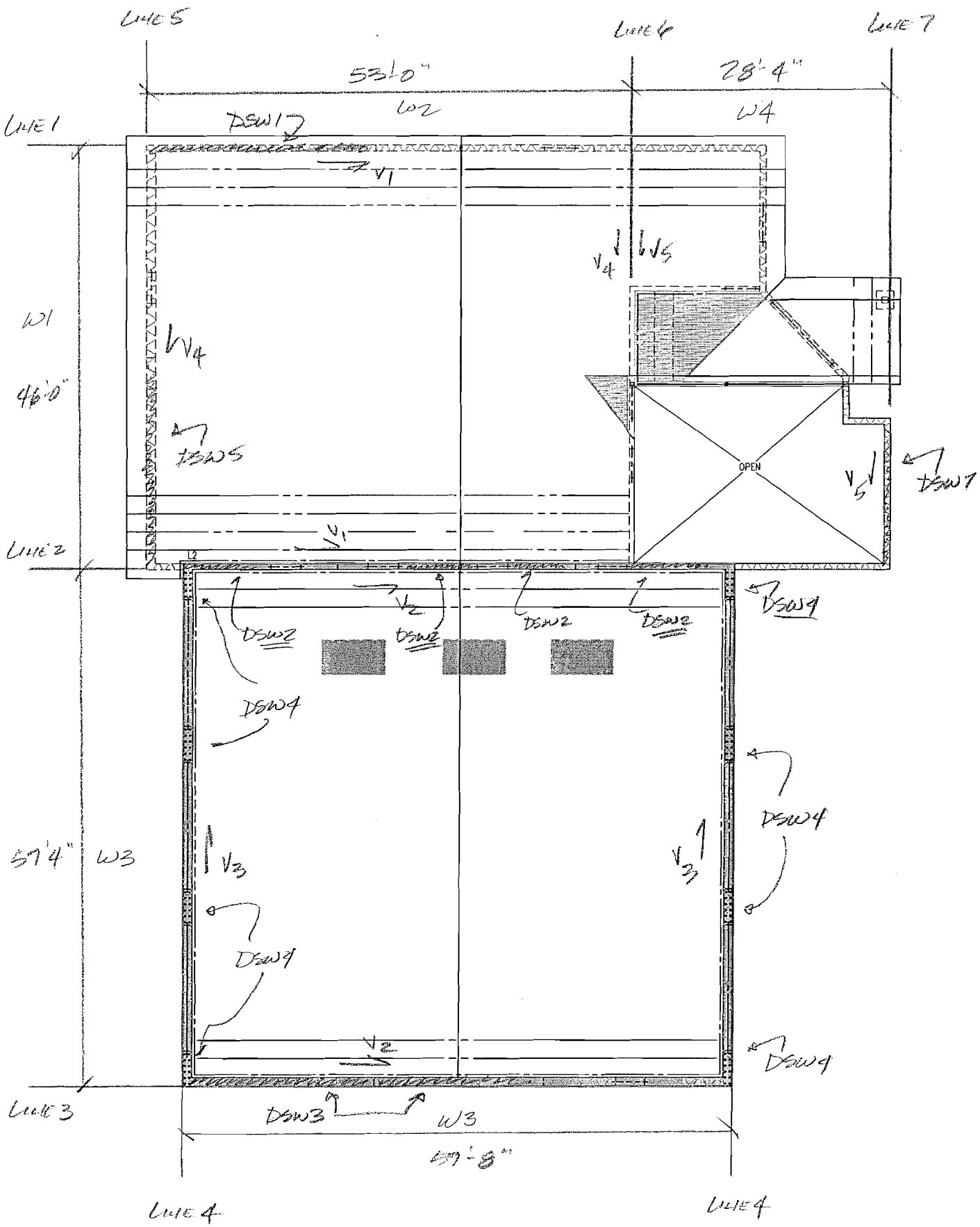
Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	39.6	0.50	19.8	Soil Over Heel	= 49.1	1.67	81.9
Toe Active Pressure	=				Sloped Soil Over Heel	=		
Surcharge Over Toe	=				Surcharge Over Heel	=		
Adjacent Footing Load	=				Adjacent Footing Load	=		
Added Lateral Load	=				Axial Dead Load on Stem	=	0.00	
Load @ Stem Above Soil	=	78.0	4.50	351.3	Soil Over Toe	= 36.7	0.33	12.2
SeismicLoad	=				Surcharge Over Toe	=		
					Stem Weight(s)	= 366.9	1.00	366.9
Total	=	117.6	**O.T.M.** =	371.1	Earth @ Stem Transitions	=		
Resisting/Overturning Ratio			=	**1.98**	Footing Weight	= 250.0	1.00	250.0
Vertical Loads used for Soil Pressure =		713.7 lbs			Key Weight	=		
					Vert. Component	= 11.0	2.00	22.0
					Total =	713.7 lbs	**R.M.=**	733.0

Vertical component of active pressure used for soil pressure


LINE 5
LINE 6
LINE 7
53'-0"
28'-4"
W2
W4
LINE 1
DSW1
V1
V4
V5
W1
46'-0"
DSW5
OPEN
DSW7
LINE 2
V2
DSW2
DSW4
57'-4"
W3
V3
DSW3
LINE 3
49'-8"
LINE 4

LATERAL ANALYSIS

WIND: 90 Mph, Exposure "C"

	Roof	Flat	3:12	
H = 0-15'-0"	P =	17.0	22.4	PSF
H = 20'-0"	P =	19.0	23.9	PSF
H = 25'-0"	P =	19.9	25.0	PSF

SEISMIC: DESIGN CATEGORY C (Detailing)

V = 0.085 W Controls

Loads to Roof Diaph.

$W1_{Wind}$ =	112	lb/ft	$W2_{Wind}$ =	187	lb/ft	$W3_{Wind}$ =	285	lb/ft
$W1_{Seismic}$ =	211	lb/ft	$W2_{Seismic}$ =	234	lb/ft	$W3_{Seismic}$ =	348	lb/ft

$W4_{Wind}$ = 187 lb/ft

$W4_{Seismic}$ = 198 lb/ft

Diap Stresses

V_1 =	211	x	46	÷	2	÷	53	=	91	lb/ft
V_2 =	348	x	57.33	÷	2	÷	59.67	=	167	lb/ft
V_3 =	348	x	59.66	÷	2	÷	57.67	=	180	lb/ft
V_4 =	234	x	53	÷	2	÷	46	=	135	lb/ft
V_5 =	198	x	28.33	÷	2	÷	16	=	175	lb/ft

Use 1/2" plywood sheathing unblocked with 8d at 6-6-12 at Roof Level Typical, U.N.O.

V_{ALLOW} = 180 lb/ft >= 180 lb/ft **OK**

BEAM SPLICE CONNECTION: Use 3/4" Bolt With 5-1/4" Member T_{ALLOW} = 4227 #/BOLT

Number of Bolts Required = 0.082

Use minimum (2) 3/4" bolts in double shear with 1/4" plate with 5 1/4" beam

Floor Shear Wall Line Forces

Grid Line 1 V =	4844	lbs.	Grid Line 5 V =	6205	lbs.
Grid Line 2 V =	14831	lbs.	Grid Line 6 V =	9012	lbs.
Grid Line 3 V =	9986	lbs.	Grid Line 7 V =	2807	lbs.
Grid Line 4 V =	10392	lbs.			

Chord Force $_{MAX}$: = (348#/ft x 59'-8"^{2}) / (8 x 57'-4") = 2701 lbs

Provide #5 minimum horizontal in 8" deep bond beam at roof line

T_{allow} = 6200 lbs

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PROJECT:	Silverbell N.G.F.S.	SHEET NO:	58
JOB NO:	06202	DATE:	Aug-06
		BY:	AE

Walls *"DSW1" - Line 1*

				Sds =	0.338		
Length =	25 ft	$Roof_{DL}$ =	17 psf	$Roof_{DL}$=	34 lb/ft	ρ =	0.0001
Height =	14.167 ft	$Roof_{LL}$ =	20 psf	$Roof_{LL}$=	40 lb/ft	ηρ =	0.0035
Roof Shear =	4844 lb	$Roof_{Trib}$ =	2 ft	$Floor_{DL}$=	0 lb/ft	k =	0.0807
Roof Height =	14.167 ft	$Floor_{DL}$ =	0 psf	$Floor_{LL}$=	0 lb/ft	j =	0.9731
Floor Shear =	0 lb	$Floor_{LL}$ =	0 psf	$Footing_{DL}$=	187.4 lb/ft	2/jk =	25.47
Flr Height =	0 ft	$Floor_{Trib}$ =	0 ft	$Wall_{DL}$=	821.7 lb/ft	Kern =	4.17 ft
Total shear =	4844 lb	Footing		fa_{DL} =	15.50 psi	S.I. (Y/N)	y 1
Wall thickness	7.625 in	dpth below grade	1.5 ft	fa_{LL} =	0.725 psi	W or E (0/1)	1
Rebar area =	0.31 in^2	Width =	1.5 ft	Fb =	500 psi	F_V =	43.44 psi
Number =	1	Depth =	0.833 ft	Fa =	375 psi	F_S =	24000 psi
total area	0.31 in^2	Length =	25 ft	M/V_d =	0.64		
Wall Weight=	58 psf	te =	4.6	M/V_d>1	f_V =	38.73 psi	
d =	24.67 ft	T_{Allow} =	7440 lb	M/V_d<1	f_V =	43.44 psi	

Load combination Seismic Controls - See Load Combinations 3 and 4

(1) D+W+L: $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 75891 ft-lb M_R = 3E+05 ft-lb OK

16.23	÷	375	+	Seismic	÷	500	=	N/A	≤	1	N/A
16.23	÷	375	+	Seismic	÷	24000	=	N/A	≤	1	N/A
e =	2.80 ft	m =	9.70 ft		q =	1208 psf			≤	2666.667	OK
fv =	4 psi	fb =	13.2 psi		fc =	29 psi			T =	195 lb	
Ptl =	27077.8 lbs	ft =	3.0 psi		a =	27.99			fs =	628.7622 psi	

(2) 0.6D + W: $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 75891 ft-lb M_R = 2E+05 ft-lb OK

9.30	÷	375	+	Seismic	÷	500	=	N/A	≤	1	N/A
9.30	÷	375	+	Seismic	÷	24000	=	N/A	≤	1	N/A
e =	4.85 ft	m =	7.65 ft		q =	909 psf			≤	2666.667	OK
fv =	4 psi	fb =	13 psi		fc =	29 psi			T =	118 lb	
Ptl =	15646.7 lbs	ft =	2.30 psi		a =	22.286			fs =	380.82 psi	

(3) D + 0.7E + L: $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 75891 ft-lb M_R = 3E+05 ft-lb **OK**

16.23	÷	**375**	+	**13**	÷	**500**	=	**0.07**	≤	**1**	**OK**
16.23	÷	**375**	+	**629**	÷	**24000**	=	**0.069**	≤	**1**	**OK**
e =	**2.80 ft**	**m =**	**9.70 ft**		**q =**	**1208 psf**			≤	**2666.667**	**OK**
fv =	4 psi	fb =	13 psi		fc =	29 psi			T =	195 lb	
Ptl =	27077.8 lbs	ft =	3.03 psi		a =	27.99			fs =	628.7622 psi	

(4) 0.6D + 0.7E: $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 75891 ft-lb M_R = 2E+05 ft-lb **OK**

9.30	÷	375	+	13	÷	500	=	**0.051**	≤	**1**	**OK**
9.30	÷	375	+	2062	÷	24000	=	**0.111**	≤	**1**	**OK**
e =	4.85 ft	m =	7.65 ft		q =	909 psf			≤	2666.667	OK
fv =	4 psi	fb =	13 psi		fc =	21 psi			T =	639 lb	
Ptl =	15646.7 lbs	ft =	-4.95 psi		a =	56.204			fs =	2062.232 psi	

8" cmu wall with grouted cells at 48" o.c. with (1) #5 each end

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PROJECT:	Silverbell N.G.F.S.	SHEET NO:	59
JOB NO:	06202	DATE:	Aug-06
		BY:	AE

Walls *"DSW2" - Line 2 (Four Panels)*

				Sds =	0.338				
Length =	7.67 ft	$Roof_{DL}$ =	17 psf	$Roof_{DL}$=	68 lb/ft	ρ =	0.0009		
Height =	26 ft	$Roof_{LL}$ =	20 psf	$Roof_{LL}$=	80 lb/ft	ηρ =	0.0238		
Roof Shear =	3708 lb	$Roof_{Trib}$ =	4 ft	$Floor_{DL}$=	0 lb/ft	k =	0.1957		
Roof Height =	21.33 ft	$Floor_{DL}$ =	0 psf	$Floor_{LL}$=	0 lb/ft	j =	0.9348		
Floor Shear =	0 lb	$Floor_{LL}$ =	0 psf	$Footing_{DL}$=	249.9 lb/ft	2/jk =	10.94		
Flr Height =	0 ft	$Floor_{Trib}$ =	0 ft	$Wall_{DL}$=	1237 lb/ft	Kern =	3.50 ft		
Total shear =	3708 lb	Footing		fa_{DL} =	23.64 psi	S.I. (Y/N)	y	1	
Wall thickness	7.625 in	dpth below grade	1.5 ft	fa_{LL} =	1.449 psi	W or E (0/1)	1		
Rebar area =	0.31 in^2	Width =	2 ft	Fb =	500 psi	F_V =	38.73 psi		
Number =	2	Depth =	0.833 ft	Fa =	375 psi	F_S =	24000 psi		
total area	0.62 in^2	Length =	21 ft	M/V_d =	3.11				
Wall Weight=	58 psf	te =	4.6	M/V_d>1	f_V =	38.73 psi			
d =	7.34 ft	T_{Allow} =	14880 lb	M/V_d<1	f_V =	11.49 psi			

Load combination Seismic Controls - See Load Combinations 3 and 4

(1) **D+W+L:** $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 84654 ft-lb M_R = 2E+05 ft-lb OK

25.09	÷	375	+	Seismic	÷	500	=	N/A	≤	1	N/A
25.09	÷	375	+	Seismic	÷	24000	=	N/A	≤	1	N/A
e =	5.33 ft	m =	5.17 ft		q =	1024 psf			≤	2666.667	OK
fv =	10 psi	fb =	156.4 psi		fc =	182 psi			T =	11670 lb	
Ptl =	15871.9 lbs	ft =	-131.3 psi		a =	38.637			fs =	18821.88 psi	

(2) **0.6D + W:** $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 84654 ft-lb M_R = 96127 ft-lb OK

14.19	÷	375	+	Seismic	÷	500	=	N/A	≤	1	N/A
14.19	÷	375	+	Seismic	÷	24000	=	N/A	≤	1	N/A
e =	9.25 ft	m =	1.25 ft		q =	2435 psf			≤	2666.667	OK
fv =	10 psi	fb =	156 psi		fc =	180 psi			T =	11929 lb	
Ptl =	9154.99 lbs	ft =	-132.77 psi		a =	39.063			fs =	19239.62 psi	

(3) **D + 0.7E + L:** $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 84654 ft-lb M_R = 2E+05 ft-lb OK

25.09	÷	375	+	156	÷	500	=	0.38	≤	1	OK
25.09	÷	375	+	18822	÷	24000	=	0.851	≤	1	OK
e =	5.33 ft	m =	5.17 ft		q =	1024 psf			≤	2666.667	OK
fv =	10 psi	fb =	156 psi		fc =	182 psi			T =	11670 lb	
Ptl =	15871.9 lbs	ft =	-131.32 psi		a =	38.637			fs =	18821.88 psi	

(4) **0.6D + 0.7E:** $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 84654 ft-lb M_R = 96127 ft-lb OK

14.19	÷	375	+	156	÷	500	=	0.351	≤	1	OK
14.19	÷	375	+	22577	÷	24000	=	0.979	≤	1	OK
e =	9.25 ft	m =	1.25 ft		q =	2435 psf			≤	2666.667	OK
fv =	10 psi	fb =	156 psi		fc =	169 psi			T =	13998 lb	
Ptl =	9154.99 lbs	ft =	-143.82 psi		a =	42.316			fs =	22577.3 psi	

8" cmu wall with grouted cells at 48" o.c. with (2) #5 each end

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PROJECT:	Silverbell N.G.F.S.	SHEET NO:	60
JOB NO:	06202	DATE:	Aug-06
		BY:	AE

Walls *"DSW3" - Line 3 (Two Panels)*

				Sds =	0.338				
Length =	18.67 ft	$Roof_{DL}$ =	17 psf	$Roof_{DL}$=	34 lb/ft	ρ =	0.0002		
Height =	26 ft	$Roof_{LL}$ =	20 psf	$Roof_{LL}$=	40 lb/ft	ηρ =	0.0044		
Roof Shear =	4993 lb	$Roof_{Trib}$ =	2 ft	$Floor_{DL}$=	0 lb/ft	k =	0.0898		
Roof Height =	21 ft	$Floor_{DL}$ =	0 psf	$Floor_{LL}$=	0 lb/ft	j =	0.9701		
Floor Shear =	0 lb	$Floor_{LL}$ =	0 psf	$Footing_{DL}$=	375 lb/ft	2/jk =	22.95		
Flr Height =	0 ft	$Floor_{Trib}$ =	0 ft	$Wall_{DL}$=	2163 lb/ft	Kern =	3.17 ft		
Total shear =	4993 lb	Footing		fa_{DL} =	39.80 psi	S.I. (Y/N)	y	1	
Wall thickness	11.625 in	dpth below grade	1.5 ft	fa_{LL} =	0.725 psi	W or E (0/1)	1		
Rebar area =	0.44 in^2	Width =	2.5 ft	Fb =	500 psi	F_V =	38.73 psi		
Number =	1	Depth =	1 ft	Fa =	375 psi	F_S =	24000 psi		
total area	0.44 in^2	Length =	19 ft	M/V_d =	1.23				
Wall Weight=	103 psf	te =	4.6	M/V_d>1	f_V =	38.73 psi			
d =	18.34 ft	T_{Allow} =	10560 lb	M/V_d<1	f_V =	35.80 psi			

Load combination Seismic Controls - See Load Combinations 3 and 4

(1) **D+W+L:**	$f_a / F_a + f_b / F_b \le 1.0$					M_{OTM} = 112343 ft-lb			M_R = 5E+05 ft-lb			OK
40.53	÷	375	+	Seismic	÷	500	=	N/A	≤	1		N/A
40.53	÷	375	+	Seismic	÷	24000	=	N/A	≤	1		N/A
e =	2.30 ft		m =	7.20 ft		q =	1776 psf		≤	2666.667		OK
fv =	5 psi		fb =	35.0 psi		fc =	76 psi		T =	192 lb		
Ptl =	48889.8 lbs		ft =	5.5 psi		a =	15.184		fs =	435.9995 psi		
(2) **0.6D + W:**	$f_a / F_a + f_b / F_b \le 1.0$					M_{OTM} = 112343 ft-lb			M_R = 3E+05 ft-lb			OK
23.88	÷	375	+	Seismic	÷	500	=	N/A	≤	1		N/A
23.88	÷	375	+	Seismic	÷	24000	=	N/A	≤	1		N/A
e =	3.89 ft		m =	5.61 ft		q =	1373 psf		≤	2666.667		OK
fv =	5 psi		fb =	35 psi		fc =	75 psi		T =	147 lb		
Ptl =	28885.8 lbs		ft =	4.77 psi		a =	13.421		fs =	334.5374 psi		
(3) **D + 0.7E + L:**	$f_a / F_a + f_b / F_b \le 1.0$					M_{OTM} = 112343 ft-lb			M_R = 5E+05 ft-lb			OK
40.53	÷	375	+	35	÷	500	=	**0.178**	≤	1		OK
40.53	÷	375	+	436	÷	24000	=	**0.126**	≤	1		OK
e =	2.30 ft		m =	7.20 ft		q =	1776 psf		≤	2666.667		OK
fv =	5 psi		fb =	35 psi		fc =	76 psi		T =	192 lb		
Ptl =	48889.8 lbs		ft =	5.49 psi		a =	15.184		fs =	435.9995 psi		
(4) **0.6D + 0.7E:**	$f_a / F_a + f_b / F_b \le 1.0$					M_{OTM} = 112343 ft-lb			M_R = 3E+05 ft-lb			OK
23.88	÷	375	+	35	÷	500	=	**0.134**	≤	1		OK
23.88	÷	375	+	3203	÷	24000	=	**0.197**	≤	1		OK
e =	3.89 ft		m =	5.61 ft		q =	1373 psf		≤	2666.667		OK
fv =	5 psi		fb =	35 psi		fc =	56 psi		T =	1409 lb		
Ptl =	28885.8 lbs		ft =	-13.84 psi		a =	44.263		fs =	3202.73 psi		

12" cmu wall with grouted cells at 48" o.c. with (1) #6 each end

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PROJECT:	Silverbell N.G.F.S.	SHEET NO:	61
JOB NO:	06202	DATE:	Aug-06
		BY:	AE

Walls ***"DSW4" - Line 4 (Four Panels)***

					Sds =	0.338			
Length =	4	ft	$Roof_{DL}$ =	17	psf	$Roof_{DL}$=	507.2 lb/ft	ρ =	0.0017
Height =	26	ft	$Roof_{LL}$ =	20	psf	$Roof_{LL}$=	596.7 lb/ft	ηρ =	0.0443
Roof Shear =	2598	lb	$Roof_{Trib}$ =	29.835	ft	$Floor_{DL}$=	0 lb/ft	k =	0.2567
Roof Height =	21	ft	$Floor_{DL}$ =	0	psf	$Floor_{LL}$=	0 lb/ft	j =	0.9144
Floor Shear =	0	lb	$Floor_{LL}$ =	0	psf	$Footing_{DL}$=	450 lb/ft	2/jk =	8.52
Flr Height =	0	ft	$Floor_{Trib}$ =	0	ft	$Wall_{DL}$=	2163 lb/ft	Kern =	2.33 ft
Total shear =	2598	lb	Footing			fa_{DL} =	48.37 psi	S.I. (Y/N)	y 1
Wall thickness	11.625	in	dpth below grade	1.5	ft	fa_{LL} =	10.81 psi	W or E (0/1)	1
Rebar area =	0.44	in^2	Width =	3	ft	Fb =	500 psi	F_V =	38.73 psi
Number =	2		Depth =	1	ft	Fa =	375 psi	F_S =	24000 psi
total area	0.88	in^2	Length =	14	ft	M/V_d =	6.13		
Wall Weight=	103	psf	te =	4.6		M/V_d>1	f_V = 38.73 psi		
d =	3.67	ft	T_{Allow} =	21120	lb	M/V_d<1	f_V = -27.51 psi		

Load combination Seismic Controls - See Load Combinations 3 and 4

(1) D+W+L:	$f_a / F_a + f_b / F_b \le 1.0$					M_{OTM} =	58455 ft-lb		M_R =	1E+05 ft-lb		OK
59.18	÷	375	+	Seismic	÷	500	=	N/A	≤	1		N/A
59.18	÷	375	+	Seismic	÷	24000	=	N/A	≤	1		N/A
e =	3.02 ft		m =	3.98 ft		q =	1081 psf		≤	2666.667		OK
fv =	14 psi		fb =	397.1 psi		fc =	456 psi		T =	15874 lb		
Ptl =	19367.6 lbs		ft =	-337.9 psi		a =	20.423		fs =	18038.31 psi		
(2) 0.6D + W:	$f_a / F_a + f_b / F_b \le 1.0$					M_{OTM} =	58455 ft-lb		M_R =	71319 ft-lb		OK
29.02	÷	375	+	Seismic	÷	500	=	N/A	≤	1		N/A
29.02	÷	375	+	Seismic	÷	24000	=	N/A	≤	1		N/A
e =	5.74 ft		m =	1.26 ft		q =	1793 psf		≤	2666.667		OK
fv =	14 psi		fb =	397 psi		fc =	445 psi		T =	16905 lb		
Ptl =	10188.5 lbs		ft =	-348.74 psi		a =	21.077		fs =	19210.79 psi		
(3) D + 0.7E + L:	$f_a / F_a + f_b / F_b \le 1.0$					M_{OTM} =	58455 ft-lb		M_R =	1E+05 ft-lb		**OK**
59.18	÷	375	+	397	÷	500	=	**0.952**	≤	**1**		**OK**
59.18	÷	375	+	18038	÷	24000	=	**0.909**	≤	**1**		**OK**
e =	**3.02 ft**		**m =**	**3.98 ft**		**q =**	**1081 psf**		≤	2666.667		**OK**
fv =	14 psi		fb =	397 psi		fc =	456 psi		T =	15874 lb		
Ptl =	19367.6 lbs		ft =	-337.93 psi		a =	20.423		fs =	18038.31 psi		
(4) 0.6D + 0.7E:	$f_a / F_a + f_b / F_b \le 1.0$					M_{OTM} =	58455 ft-lb		M_R =	71319 ft-lb		**OK**
29.02	÷	375	+	397	÷	500	=	**0.872**	≤	**1**		**OK**
29.02	÷	375	+	21784	÷	24000	=	**0.985**	≤	**1**		**OK**
e =	5.74 ft		m =	1.26 ft		q =	1793 psf		≤	2666.667		**OK**
fv =	14 psi		fb =	397 psi		fc =	423 psi		T =	19170 lb		
Ptl =	10188.5 lbs		ft =	-371.36 psi		a =	22.444		fs =	21783.63 psi		

12" cmu wall with grouted cells at 48" o.c. with (4) #6 each end

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PROJECT:	Silverbell N.G.F.S.	SHEET NO:	62
JOB NO:	06202	DATE:	Aug-06
		BY:	AE

Walls *"DSW5" - Line 5*

					Sds =	0.338			
Length =	23.33 ft	$Roof_{DL}$ =	17 psf	$Roof_{DL}$=	329.4	lb/ft	ρ =	0.0001	
Height =	10 ft	$Roof_{LL}$ =	20 psf	$Roof_{LL}$=	387.5	lb/ft	ηρ =	0.0038	
Roof Shear =	6205 lb	$Roof_{Trib}$ =	19.375 ft	$Floor_{DL}$=	0	lb/ft	k =	0.0834	
Roof Height =	10 ft	$Floor_{DL}$ =	0 psf	$Floor_{LL}$=	0	lb/ft	j =	0.9722	
Floor Shear =	0 lb	$Floor_{LL}$ =	0 psf	$Footing_{DL}$=	187.4	lb/ft	2/jk =	24.66	
Flr Height =	0 ft	$Floor_{Trib}$ =	0 ft	$Wall_{DL}$=	580	lb/ft	Kern =	4.17 ft	
Total shear =	6205 lb	Footing		fa_{DL} =	16.47	psi	S.I. (Y/N)	y	1
Wall thickness	7.625 in	dpth below grade	1.5 ft	fa_{LL} =	7.02	psi	W or E (0/1)	1	
Rebar area =	0.31 in^2	Width =	1.5 ft	Fb =	500	psi	F_V =	45.18 psi	
Number =	1	Depth =	0.833 ft	Fa =	375	psi	F_S =	24000 psi	
total area	0.31 in^2	Length =	25 ft	M/V_d =	0.50				
Wall Weight=	58 psf	te =	4.6	M/V_d>1	f_V =	38.73 psi			
d =	23 ft	T_{Allow} =	7440 lb	M/V_d<1	f_V =	45.18 psi			

Load combination Seismic Controls - See Load Combinations 3 and 4

(1) D+W+L: $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 71358 ft-lb M_R = 4E+05 ft-lb OK

23.49	÷	375	+	Seismic	÷	500	=	N/A	≤	1	N/A
23.49	÷	375	+	Seismic	÷	24000	=	N/A	≤	1	N/A
e =	2.04 ft	m =	10.46 ft		q =	1388 psf			≤	2666.667	OK
fv =	5 psi	fb =	14.3 psi		fc =	38 psi			T =	1171 lb	
Ptl =	34941.7 lbs	ft =	9.2 psi		a =	55.076			fs =	3777.292 psi	

(2) 0.6D + W: $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 71358 ft-lb M_R = 2E+05 ft-lb OK

9.88	÷	375	+	Seismic	÷	500	=	N/A	≤	1	N/A
9.88	÷	375	+	Seismic	÷	24000	=	N/A	≤	1	N/A
e =	4.59 ft	m =	7.91 ft		q =	873 psf			≤	2666.667	OK
fv =	5 psi	fb =	14 psi		fc =	31 psi			T =	97 lb	
Ptl =	15540.8 lbs	ft =	2.22 psi		a =	18.897			fs =	311.7988 psi	

(3) D + 0.7E + L: $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 71358 ft-lb M_R = 4E+05 ft-lb **OK**

23.49	÷	375	+	14	÷	500	=	**0.091**	≤	**1**	**OK**
23.49	÷	375	+	3777	÷	24000	=	**0.22**	≤	**1**	**OK**
e =	**2.04 ft**	**m =**	**10.46 ft**		**q =**	**1388 psf**			≤	2666.667	**OK**
fv =	5 psi	fb =	14 psi		fc =	38 psi			T =	1171 lb	
Ptl =	34941.7 lbs	ft =	9.24 psi		a =	55.076			fs =	3777.292 psi	

(4) 0.6D + 0.7E: $f_a / F_a + f_b / F_b \le 1.0$ M_{OTM} = 71358 ft-lb M_R = 2E+05 ft-lb **OK**

9.88	÷	375	+	14	÷	500	=	**0.055**	≤	**1**	**OK**
9.88	÷	375	+	2188	÷	24000	=	**0.118**	≤	**1**	**OK**
e =	4.59 ft	m =	7.91 ft		q =	873 psf			≤	2666.667	OK
fv =	5 psi	fb =	14 psi		fc =	23 psi			T =	678 lb	
Ptl =	15540.8 lbs	ft =	-5.48 psi		a =	53.824			fs =	2188.135 psi	

8" cmu wall with grouted cells at 48" o.c. with (1) #5 each end

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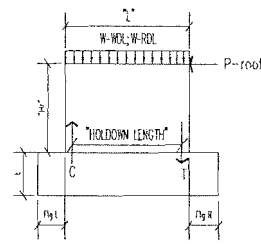
PROJECT:	Silverbell N.G.F.S.	SHEET NO:	63
JOB NO:	06202	DATE:	Aug-06
		BY:	AE

Shear Panel "*DSW6*"-- Line 6 (Two panels)

Roof W_{DL} =	187	lb/ft	$Footing_{Thickness}$ =	10	in
$Wall_{DL}$ =	10	PSF	$Footing_{Width}$ =	3	ft
Length =	9.5	ft	Footing L=	5.5	ft
Ht_{Roof} =	15	ft	Footing R=	2.67	ft
P_{Roof} =	4506	lbs	Depth below (t) =	1.5	ft
P_{Total} =	4506	lbs			
End stud Lgtl	1.5	in	Footing Wt_{DL} =	375	lb/ft
			Slab Weight =	1325.3	LB



RESULTS **1st Floor**

unit shear=	474.3	lb/ft	⇐ **Use SW1 = 1/2" plywood with**
H/L ratio=	1.579		**#8 screws at 3" o.c.. V_{Allow} = 608 #/Ft**
M for Hd=	67590	ft-lb	
Mr=	15207	ft-lb	
T/C =	5663	lbs	⇐ **Use Simpson S/HD8 with (24) #10 Screws and 7/8"ϕ Anchor Bolt**
			T_{Allow} = 7615#

Over Turning Seismic(0)/Wind (1) 0

M_{OTM} =	74349	lb-ft					
$M_{R\ LEFT}$ =	97659	lb-ft	$M_{R\ CR}$ =	88596	lb-ft	Safety $Factor_{Required}$ =	1.18
$M_{R\ RIGHT}$ =	88596	lb-ft				Safety $Factor_{Provided}$ =	1.19 **OK**

Footing pressure

P =	9828	lbs	M_L=	67113	lb-ft	e=		6.93 ft	outside kern
CL of Ftg=	8.835	ft	M_R=	68067	lb-ft	Lt/6=		2.95 ft	
Cl of Wall=	10.25	ft	M_{CR} =	68067	lb-ft		m=	1.91 ft	
q max =	1144	psf <	2667	psf	**OK**				

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PROJECT:	Silverbell N.G.F.S.	SHEET NO:	64
JOB NO:	06202	DATE:	Aug-06
		BY:	AE

Walls *"DSW7" - Line 7*

				Sds =	0.338				
Length =	16 ft	$Roof_{DL}$ =	17 psf	$Roof_{DL}$=	34 lb/ft	ρ =	0.0002		
Height =	13 ft	$Roof_{LL}$ =	20 psf	$Roof_{LL}$=	40 lb/ft	ηρ =	0.0056		
Roof Shear =	2807 lb	$Roof_{Trib}$ =	2 ft	$Floor_{DL}$=	0 lb/ft	k =	0.1002		
Roof Height =	13 ft	$Floor_{DL}$ =	0 psf	$Floor_{LL}$=	0 lb/ft	j =	0.9666		
Floor Shear =	0 lb	$Floor_{LL}$ =	0 psf	$Footing_{DL}$=	187.4 lb/ft	2/jk =	20.66		
Flr Height =	0 ft	$Floor_{Trib}$ =	0 ft	$Wall_{DL}$=	754 lb/ft	Kern =	2.83 ft		
Total shear =	2807 lb	Footing		fa_{DL} =	14.28 psi	S.I. (Y/N)	y	1	
Wall thickness	7.625 in	dpth below grade	1.5 ft	fa_{LL} =	0.725 psi	W or E (0/1)	1		
Rebar area =	0.31 in^2	Width =	1.5 ft	Fb =	500 psi	F_V =	39.69 psi		
Number =	1	Depth =	0.833 ft	Fa =	375 psi	F_S =	24000 psi		
total area	0.31 in^2	Length =	17 ft	M/V_d =	0.93				
Wall Weight=	58 psf	te =	4.6	M/V_d>1	f_V =	38.73 psi			
d =	15.67 ft	T_{Allow} =	7440 lb	M/V_d<1	f_V =	39.69 psi			

Load combination Seismic Controls - See Load Combinations 3 and 4

(1) D+W+L:	$f_a / F_a + f_b / F_b \le 1.0$				M_{OTM} =	40702 ft-lb		M_R =	1E+05 ft-lb		OK
15.00	÷	375	+	Seismic	÷	500	=	N/A	≤	1	N/A
15.00	÷	375	+	Seismic	÷	24000	=	N/A	≤	1	N/A
e =	2.48 ft	m =	6.02 ft			q =	1208 psf		≤	2666.667	OK
fv =	3 psi	fb =	17.3 psi			fc =	32 psi		T =	67 lb	
Ptl =	16434.2 lbs	ft =	-2.3 psi			a =	12.674		fs =	214.5427 psi	
(2) 0.6D + W:	$f_a / F_a + f_b / F_b \le 1.0$				M_{OTM} =	40702 ft-lb		M_R =	80551 ft-lb		OK
8.57	÷	375	+	Seismic	÷	500	=	N/A	≤	1	N/A
8.57	÷	375	+	Seismic	÷	24000	=	N/A	≤	1	N/A
e =	4.29 ft	m =	4.21 ft			q =	1002 psf		≤	2666.667	OK
fv =	3 psi	fb =	17 psi			fc =	32 psi		T =	115 lb	
Ptl =	9476.54 lbs	ft =	-3.01 psi			a =	16.7		fs =	372.4656 psi	
(3) D + 0.7E + L:	$f_a / F_a + f_b / F_b \le 1.0$				M_{OTM} =	40702 ft-lb		M_R =	1E+05 ft-lb		OK
15.00	÷	375	+	17	÷	500	=	0.075	≤	1	OK
15.00	÷	375	+	215	÷	24000	=	0.049	≤	1	OK
e =	2.48 ft	m =	6.02 ft			q =	1208 psf		≤	2666.667	OK
fv =	3 psi	fb =	17 psi			fc =	32 psi		T =	67 lb	
Ptl =	16434.2 lbs	ft =	-2.28 psi			a =	12.674		fs =	214.5427 psi	
(4) 0.6D + 0.7E:	$f_a / F_a + f_b / F_b \le 1.0$				M_{OTM} =	40702 ft-lb		M_R =	80551 ft-lb		OK
8.57	÷	375	+	17	÷	500	=	0.057	≤	1	OK
8.57	÷	375	+	3863	÷	24000	=	0.184	≤	1	OK
e =	4.29 ft	m =	4.21 ft			q =	1002 psf		≤	2666.667	OK
fv =	3 psi	fb =	17 psi			fc =	25 psi		T =	1198 lb	
Ptl =	9476.54 lbs	ft =	-9.68 psi			a =	53.78		fs =	3863.012 psi	

8" cmu wall with grouted cells at 48" o.c. with (1) #5 each end