CBC STEEL BUILDINGS

CITY
REVISED
FEB 14 2008

CITY OF BRENTWOOD
COMMUNITY DEVELOPMENT DEPT.

STRUCTURAL CALCULATIONS

CUSTOMER: Senior Adult Center

JOB NO: 27335

PLAN REVIEW APPROVAL
FEB 07 2008
SHUMS CODA ASSOCIATES

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C. SHEETING DESIGN
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E. ENDWALL DESIGN
F. RIGID FRAME DESIGN
G. LONGITUDNAL BRACING
H. SPECIAL FRAMING DESIGN

REGISTERED PROFESSIONAL ENGINEER
WEI WANG
S 4834
EXP. 9-30-09
STRUCTURAL
STATE OF CALIFORNIA

February 13, 2008

CBC STEEL BUILDINGS IS AN APPROVED FABRICATOR OF STEEL BUILDINGS

ICBOES FA- 322

AISC CERTIFIED FABRICATOR CATEGORY MB

CLARK COUNTY, NV - NO. 404

CITY OF LAS VEGAS, NV - NO. GR90068

CITY OF LOS ANGELES, CA - TYPE 1 FABRICATOR NO.1436

1700 EAST LOUISE AVENUE • LATHROP, CA 95330 • PHONE (209) 983-0910 • FAX (209) 858-2354

RECEIVED
FEB 19 2008

CBC STEEL BUILDINGS
A NUCOR Company

To: City of Brentwood

From: CBC Steel Buildings

Date: 2/11/2008

Re: Plan Check Reply, CBC J27335

Your plan check questions to CBC's job J27335 have been reviewed. The following is CBC's response to those questions related to CBC.

Structural Comments:

S1. Alternate basic load combinations(CBC 1612.3.2) are used for the metal building structure design. The roof live load does not need to be combined with the seismic load. When using these alternate basic load combinations, a one-third increase shall be permitted in allowable stress for all combinations including W or E.

S2. The length of building is 85'4 for building part #3 from line 5 to line 8 (28'+28'+29'4).

S3. See revised inspection note on drawing sheet 1/8 for detail.

We appreciate your work in this project. If you have more questions, please let us know.

Wei Wang, S.E.
Senior Design Engineer
CBC Steel Buildings

P.O. Box 1009 • 1700 E. Louise Avenue, Lathrop, CA 95330 • Ph: (209) 983-0910 • Fax: (209) 858-2354
www.cbcsteelbuildings.com

CBC STEEL BUILDINGS

1700 E. Louise Avenue, Lathrop, Ca. 95330
Tel: (209) 983-0910 • Fax: (209) 858-2354

DESIGN PARAMETERS

Job No. : 27335
Customer : Senior Adult Center
Design by : WW
Date : 10-Sep-2007

Sheet : A - 1

STRUCTURE DESCRIPTION

Frame Type : Clear Span
Building Width : 50.00 ft.
Building Length : 174.00 ft.
Eave Height : 21.25 ft.
Max. Bay Spacing : 28.00 ft. (Tributary Width)
Roof Slope : 5 in. / ft

BASIC LOADS

Building Code : CBC 2001
Roof Live Load : 20 psf Tributary Reduction (Y/N) : Y
Frame Live Load : 12 psf

Wind Load
- Speed : 70 mph (fastest mile) Enclosure Condition : Enclosed
- Exposure : C
- Importance : 1

Seismic Load
- Zone : 4 Ca : 0.44 Na : 1.0
- Importance : 1 R_{trans}: 4.50 / Ω_o : 2.80
- Soil Profile : SD R_{long}: 4.50 / Ω_o : 2.80

Snow Load
- Roof Snow : 0 psf Ce : n/a
- Ground Snow : 0 psf
- Importance : n/a

Collateral Load : 5.0/12.0 psf

Dead Load : 5 psf (Total)
- Frame Wt: 2.0 psf
- Purlins: 1.0 psf
- Panels: 1.5 psf
- Misc.: 0.5 psf

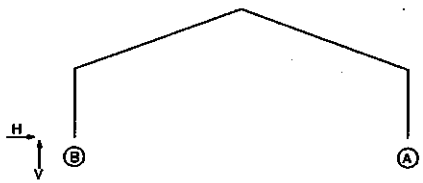
OTHER LOADS

Collateral load between lines 1-2 and 5-8 is 12.0 psf. Collateral load between lines 2-5 is 5.0 psf
(5)340 # fan coil unit at part #3. 20.0 plf pipe load at part #2.

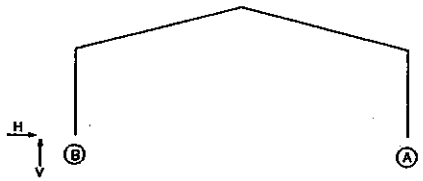
NOTES

*** This structure is designed in compliance with CBC specifications and standards utilizing the pertinent provisions and recommendations of the American Institute of Steel Construction (AISC), International Conference of Building Officials (ICBO), American Iron and Steel Institute (AISI), the Metal Building Manufacturer's Association (MBMA) and their publications. ***

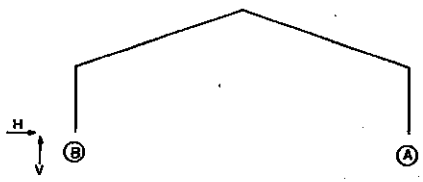
REACTION SCHEMATICS



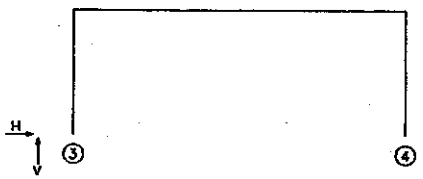
LOADING CONDITION	FRAME LINE 1			
	B		A	
	H (kips)	V (kips)	H (kips)	V (kips)
D	+0.8	+2.1	−0.8	+2.1
C	+2.4	+5.1	−2.4	+5.1
L	+2.1	+6.2	−2.1	+6.2
W	−2.7 to +1.8	−7.0	−1.8 to +2.7	−7.0
E	−2.7 to +2.7	−1.3 to +1.3	−2.7 to +2.7	−1.3 to +1.3



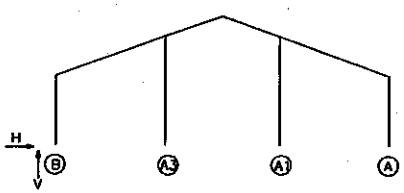
LOADING CONDITION	FRAME LINES 2 – 5			
	B		A	
	H (kips)	V (kips)	H (kips)	V (kips)
D	+0.6	+3.1	−0.6	+3.1
C	+1.0	+3.4	−1.0	+3.4
L	+1.7	+8.8	−1.7	+8.8
W	−6.4 to +5.0	−12.3	−5.0 to +6.4	−12.3
E	−3.2 to +3.2	−2.1 to +2.1	−2.4 to +2.4	−2.1 to +2.1



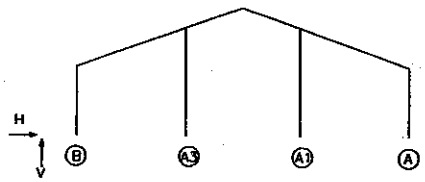
LOADING CONDITION	FRAME LINES 6 – 8			
	B		A	
	H (kips)	V (kips)	H (kips)	V (kips)
D	+1.3	+3.5	−1.3	+3.5
C	+6.2	+11.6	−6.2	+12.1
L	+4.8	+12.2	−4.8	+11.3
W	−5.0 to +2.5	−12.3	−2.5 to +5.0	−12.0
E	−4.9 to +4.9	−2.1 to +2.1	−4.3 to +4.3	−2.1 to +2.1



LOADING CONDITION	PORTAL FRAME LINE A, B			
	3		4	
	H (kips)	V (kips)	H (kips)	V (kips)
D		+1.0		+1.0
W	−3.3 to +3.3	−6.0 to +6.0	−3.3 to +3.3	−6.0 to +6.0
E	−5.8 to +5.8	−9.0 to +9.0	−5.0 to +5.0	−9.0 to +9.0

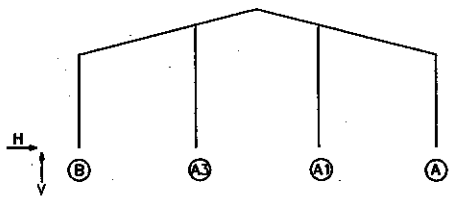


LOADING CONDITION	CURTAIN WALL AT FRAME LINE 1
	A1, A3
	H (kips)
W	−2.0 to +1.9

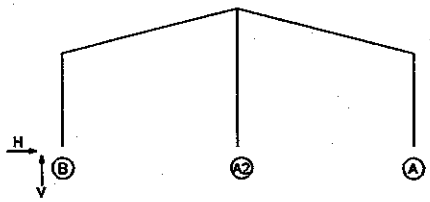


LOADING CONDITION	CURTAIN WALL AT FRAME LINE 8
	A1, A3
	H (kips)
W	−2.0 to +1.9

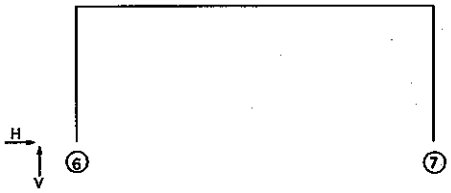
A-2



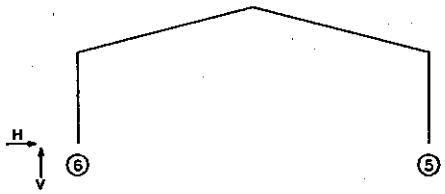
LOADING CONDITION	CURTAIN WALL AT LINE 2	
	A1 A2	
	H (kips)	V (kips)
W	−3.4 to +3.1	−2.0
D		+0.5
L		+2.9



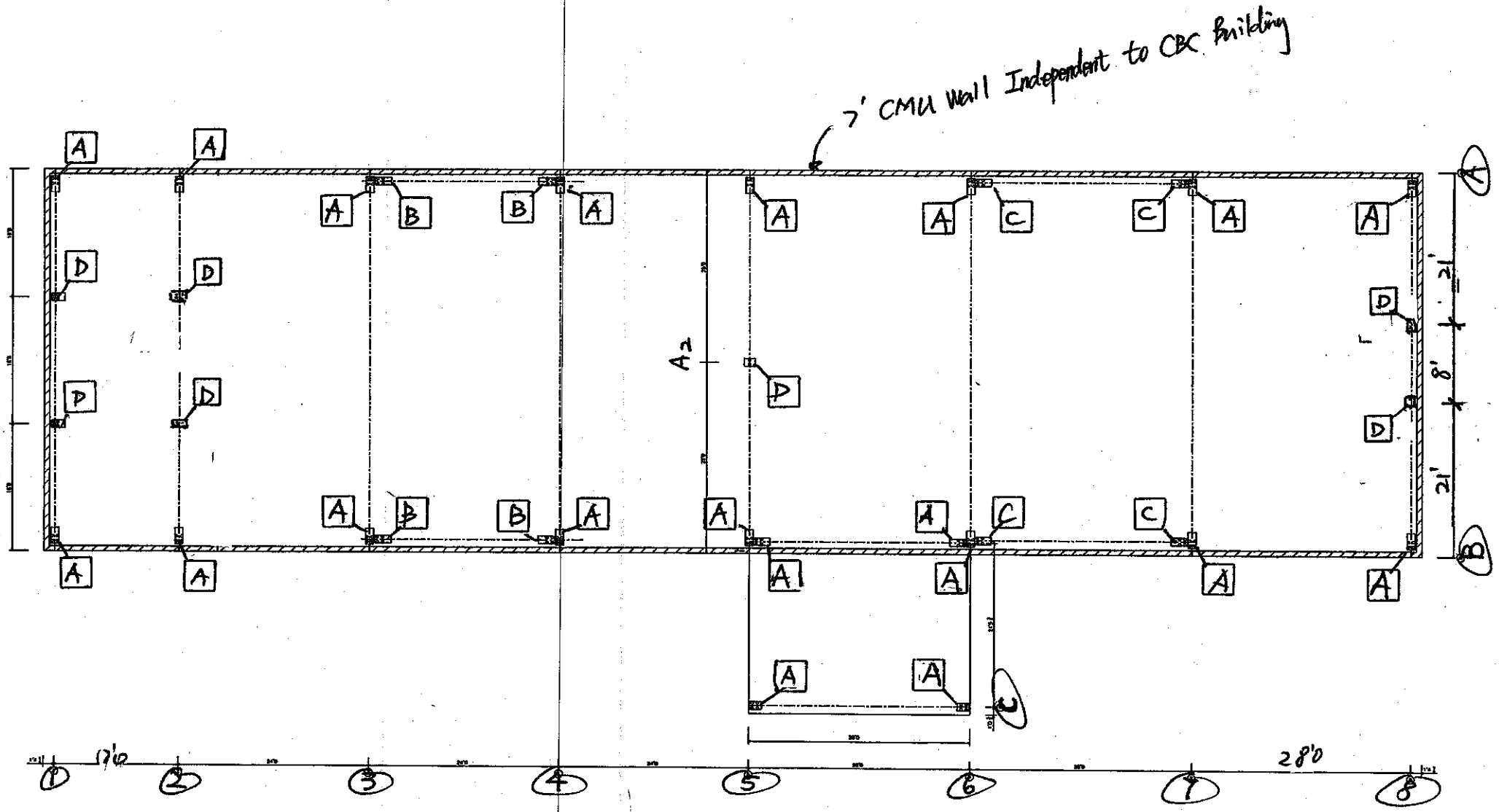
LOADING CONDITION	CURTAIN WALL AT FRAME LINE 5	
	A2	
	V (kips)	H (kips)
D	+1.8	
C	+4.2	
L	+7.0	
W	−7.3	−5.7 to +5.2



LOADING CONDITION	PORTAL FRAME LINE A, B			
	6		7	
	H (kips)	V (kips)	H (kips)	V (kips)
D	+0.1	+0.6	−0.1	+0.6
W	−2.0 to +2.0	−1.6 to +1.6	−2.0 to +2.0	−1.6 to +1.6
E	−8.3 to +8.2	−6.8 to +6.8	−8.2 to +8.3	−6.8 to +6.8



LOADING CONDITION	FRAME LINE B, C			
	6		5	
	H (kips)	V (kips)	H (kips)	V (kips)
D	+0.3	+1.5	−0.3	+1.5
C	+1.0	+4.2	−1.0	+4.2
L	+1.4	+5.6	−1.4	+5.6
W	−3.1 to +1.6	−7.3	−1.6 to +3.1	−7.3
E	−1.0 to +1.0	−0.9 to +0.9	−1.0 to +1.0	−0.9 to +0.9



ANCHOR BOLT PLAN AB01F-B

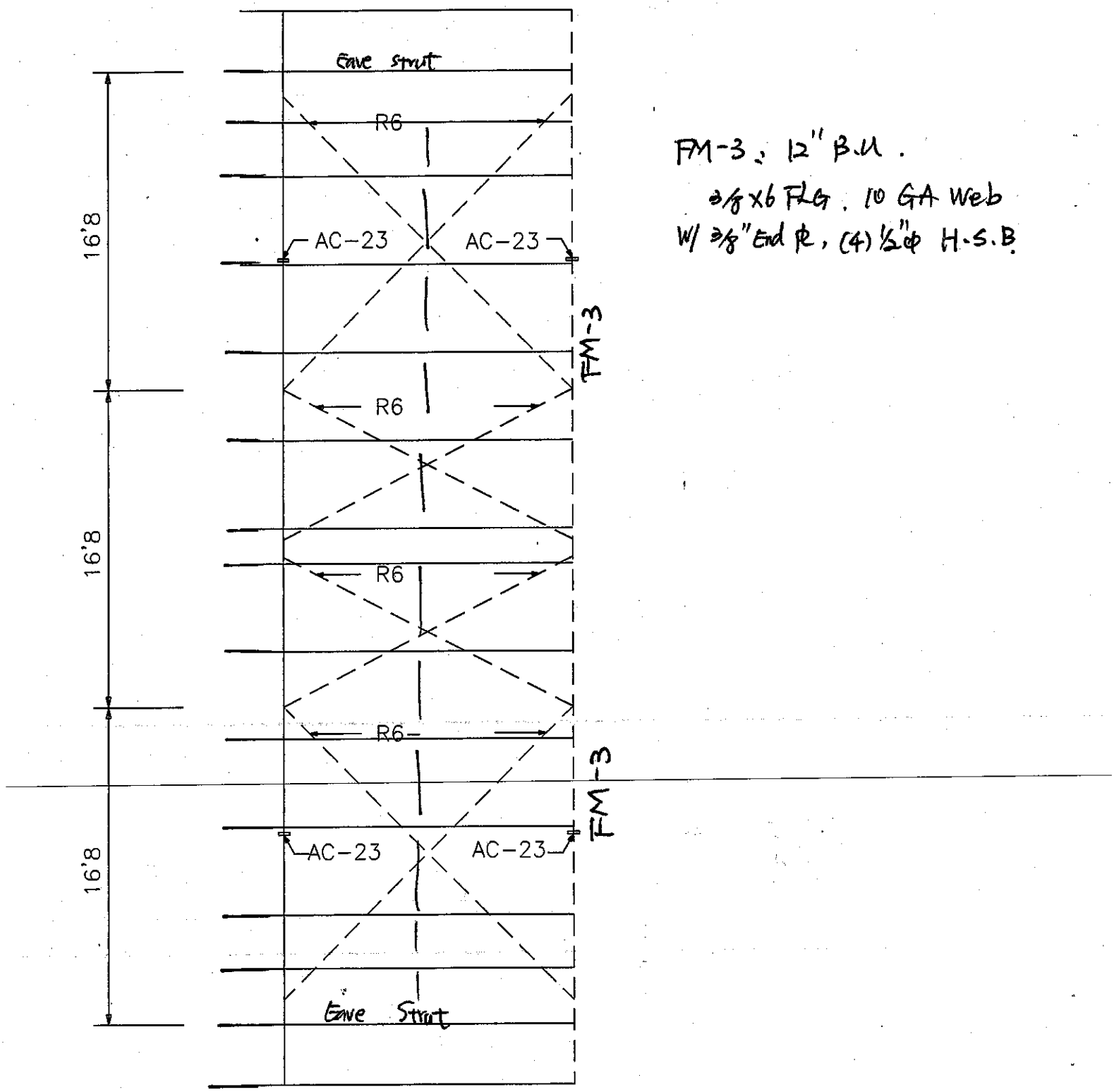
A 3/8 X 8 X 13" PL W/(4) 3/4"φ A36

B 3/8 X 8 X 19" PL W/(4) 3/4"φ A36

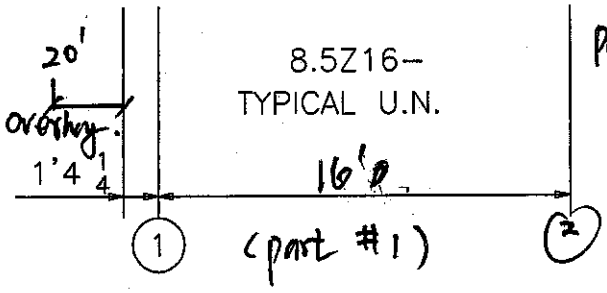
C 3/8 X 8 X 16 PL W/(4) 3/4"φ A36

D 3/8 X 8 X 9 1/2" PL W/(2) 3/4"φ A36

B-1



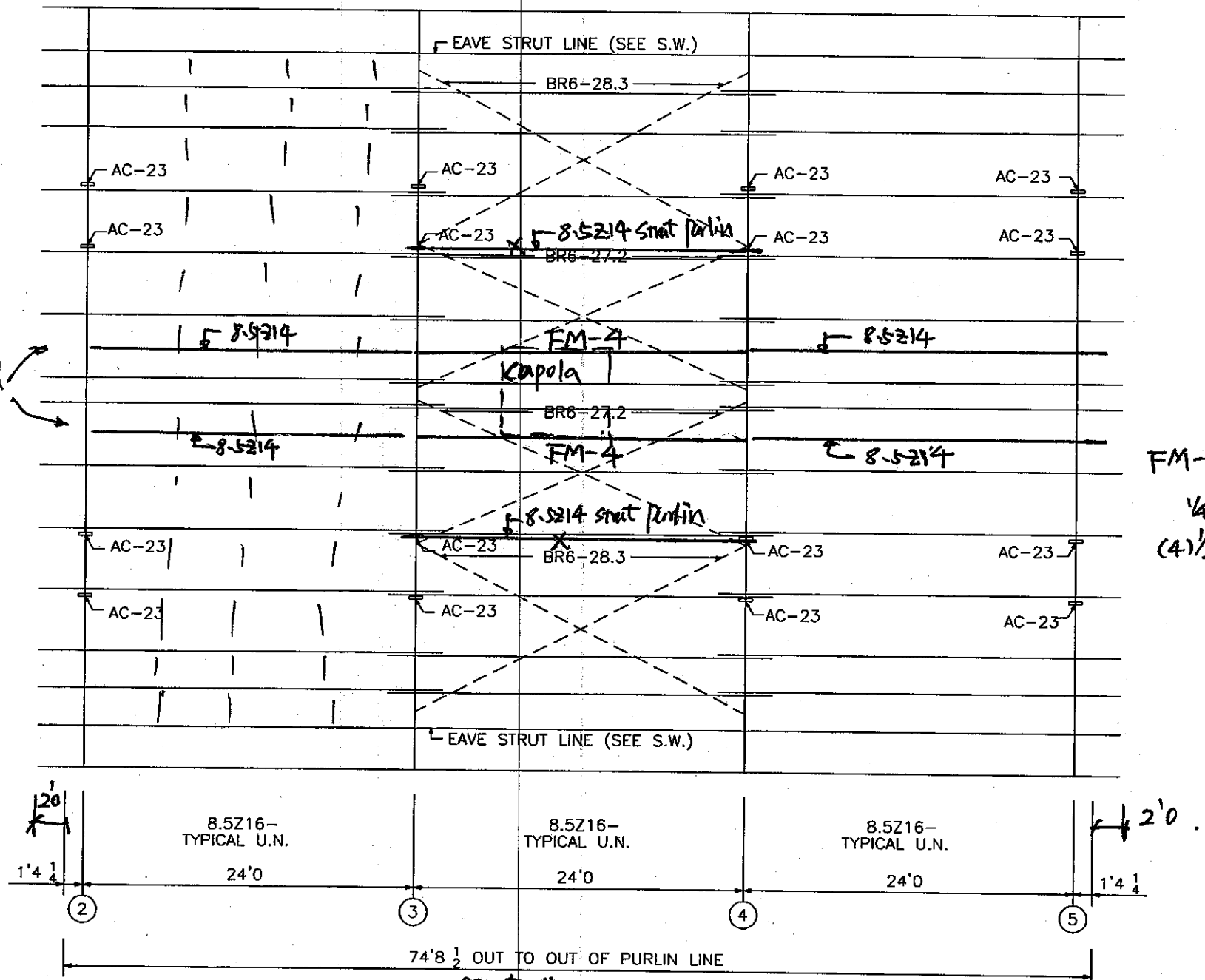
FM-3 : 12" B.U.
3/8 x 6 FLG. 10 GA Web
W/ 3/8" End PL, (4) 1/2"φ H.S.B.



(part #1)

Purlin space = 1'0, 4@5'0, 2@3'1/2

Sag Rodge @ 1/2 point.



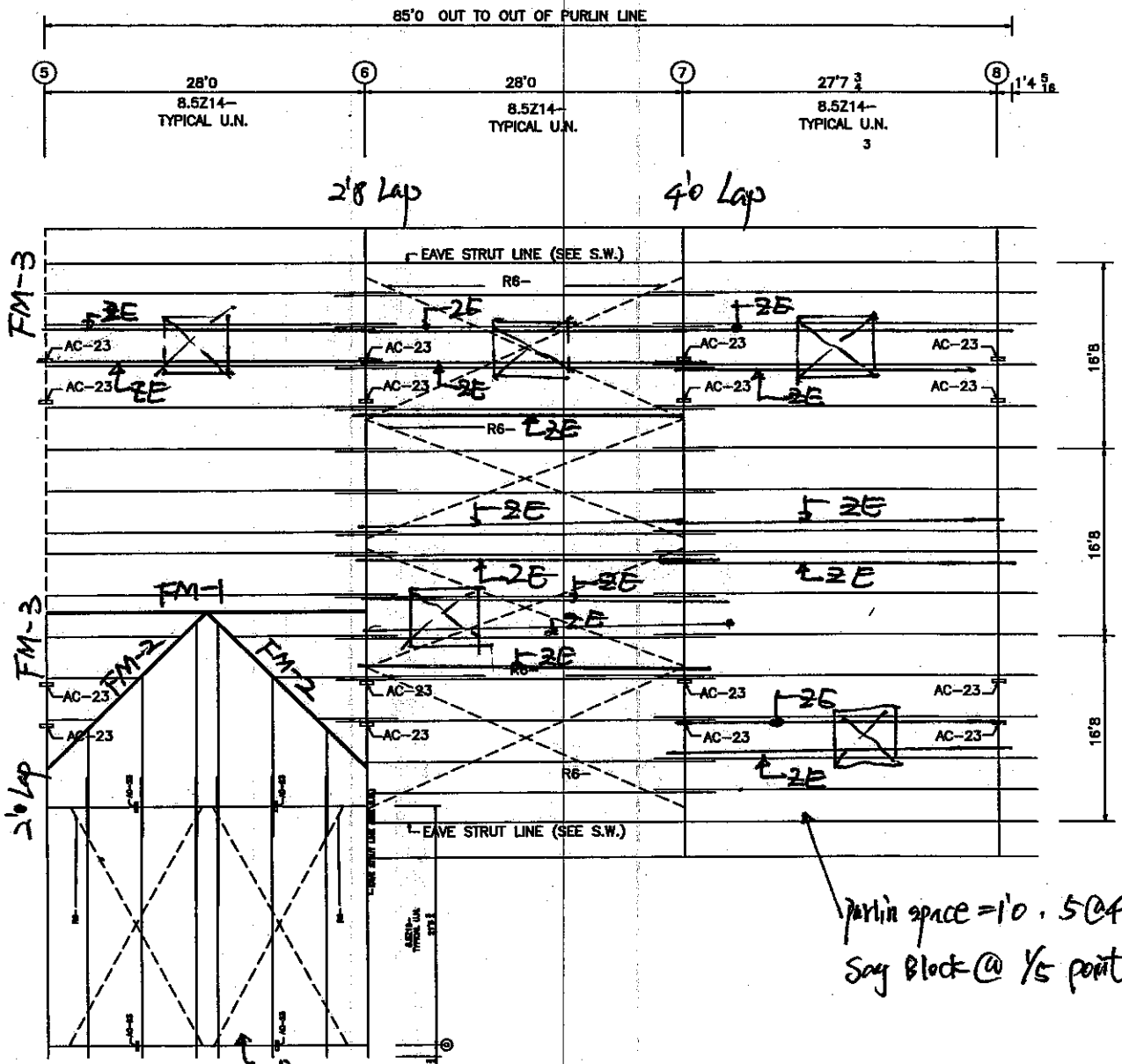
FM-4: 8 1/2" PU.
1/4x6 FLG, 10 GA Web.
(4) 1/2" ϕ MB Each End.

(part #2)

Purlin Space = 1'0, 4 @ 5'0. 2 @ 3' 1/2. 2'0 Lap
Sag-Angle @ 1/4 point.

ROOF FRAMING PLAN

B-3



FM-1 : 8½ B.U.
3/8x8 FLG, 3/16 Web
W/3/8" End PL (4) 1/2" ϕ HSB

FM-2 : 8½ B.U.
1/4x8 FLG, 10 GA Web
W/(2) 3/16 Bent-clip W/(4) 1/2" ϕ HSB.

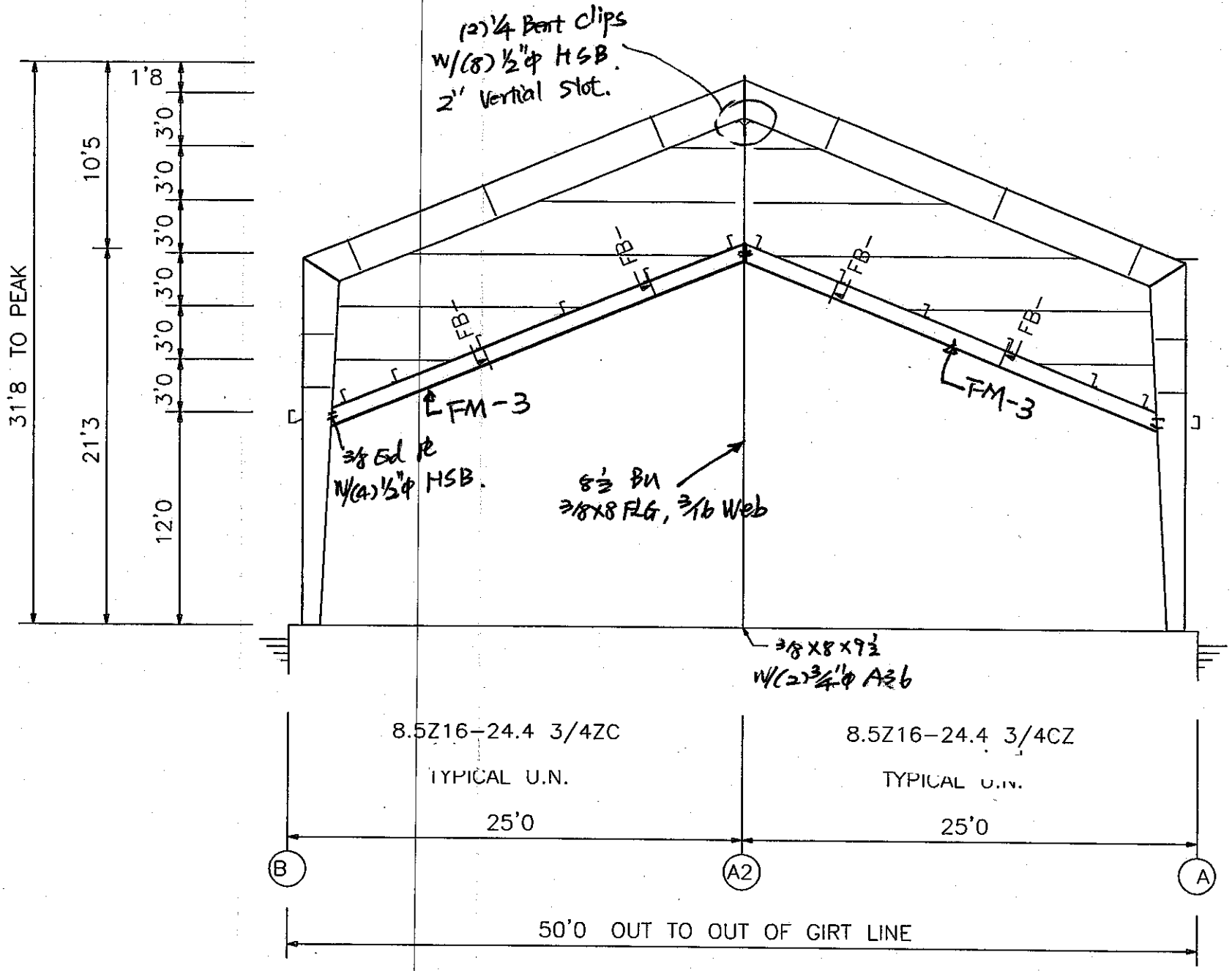
Purlin space = 1'0, 5@4'0, 2@3' 1/2
Sag Block @ 1/5 point.

Purlin space = 1'0, 2@5'0, 3'8,
Sag Block @ 1/3 point.

ROOF FRAMING PLAN RF01F–C

Part #3.

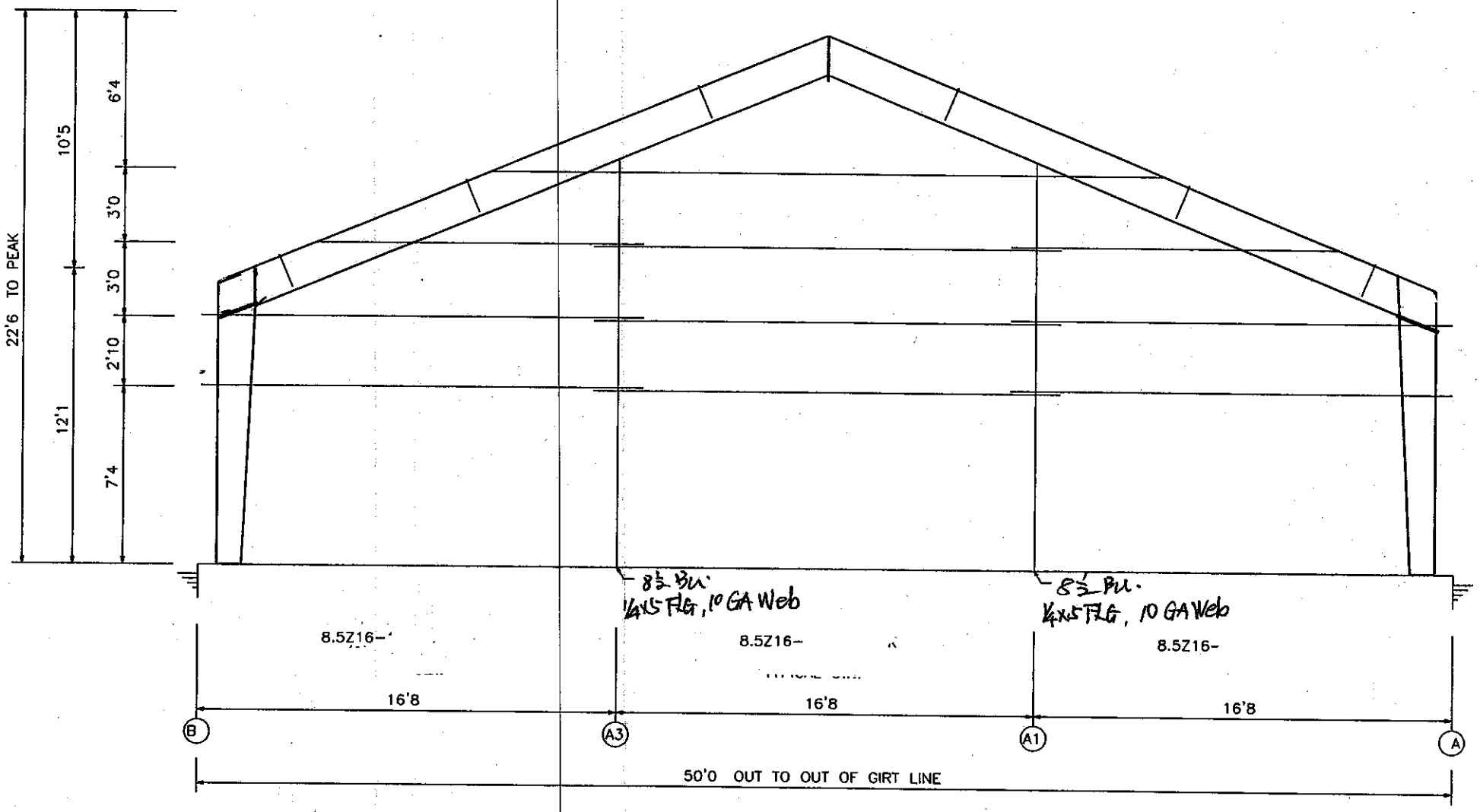
F-4



GABLE ANGLE GA-1 & RSA-1 TYPICAL

ENDWALL FRAMING AT FRAME LINE 5.

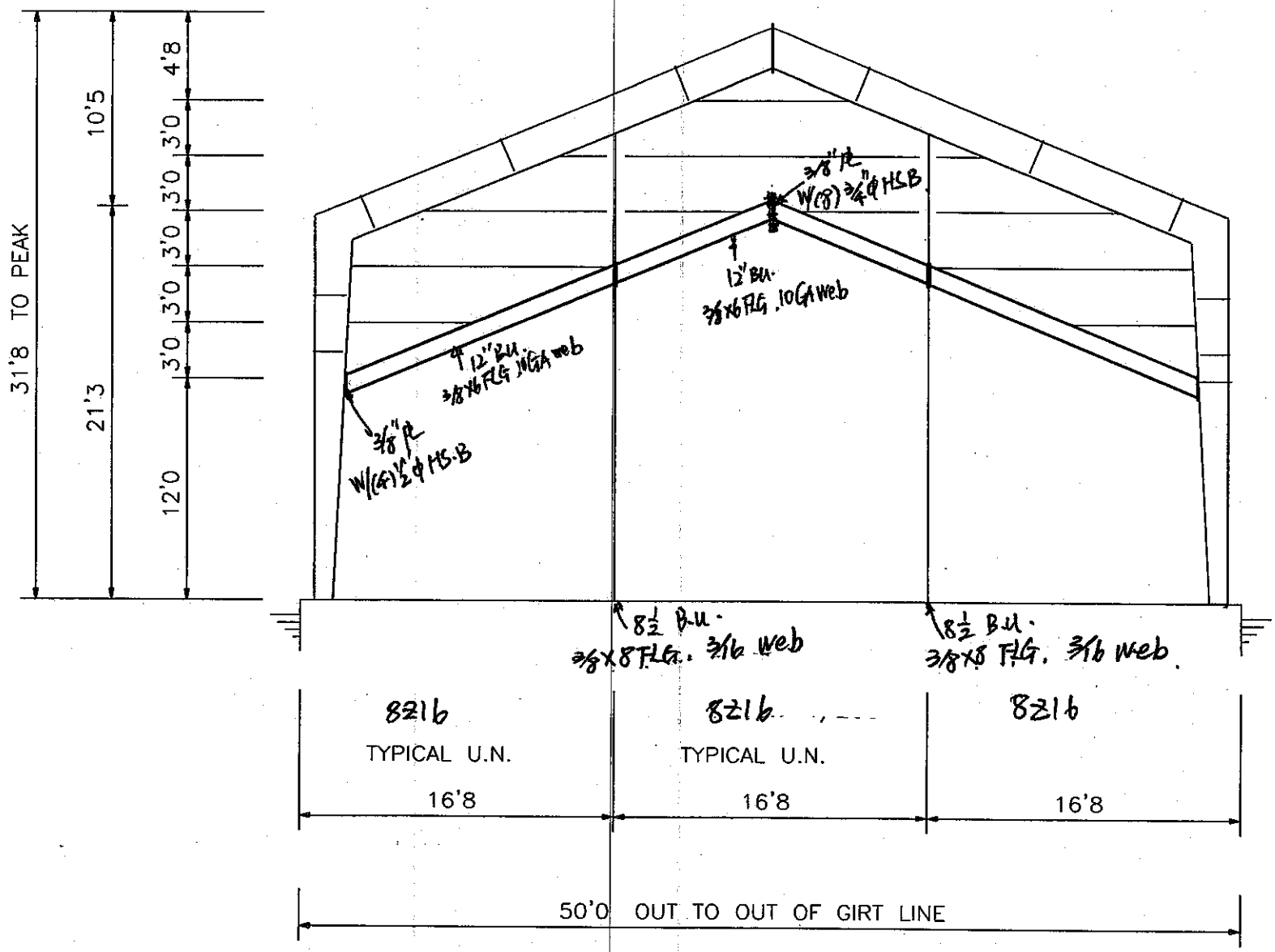
B-5



ENDWALL FRAMING ATLINES 1

Purlin Lap = 1'0

8-1



ENDWALL FRAMING AT FRAME LINE 2

L-8

C B C S T E E L B U I L D I N G S
F r o n t R o o f D e s i g n
(typical)

Part #1

Designer: WW
Job Number: J27335
Version Number: Ver. 22.1
Date/Time: 10/15/07 01:46 PM

Type	Width	Length	Ridge Dist	R.Col.Elev	Slope(F)	Slope(R)	No.BAYS
RF	50.000 ft	18.354 ft	25.000 ft	0.000 ft	5.000:12	5.000:12	1

S.Wall	Eave Ht.	Lean-To Width	E.Wall	Type	Col_Spc.	Girt Type	Overhang
Front:	12.083 ft	0.000 ft	Left	4	S	B	2.000 ft
Rear:	12.083 ft	0.000 ft	Right	7	S	I	0.000 ft

Building Code: 2001 California State Building Code

Category	Seis_Z	Expo_S	Expo_W	Coastal	DL	COLL.	LL
4.	4	3	C	N/A	2.000psf	5.000psf	20.000psf

G_Snow	D_Snow	WL_mph	WL_psf	S.Imp.Fac	S.Fac	W.Imp.Fac
0.000psf	0.000psf	70.000mph	13.760psf	1.000	0.700	1.000

Purlin locations on slope from peak to eave.

Line No.	Dist. (ft)	Design Spacing	Interesting Line	AR Clip	L.edge zone pkg	R.edge zone pkg	Weight (lbs)
1	1.000	3.500	Yes				59.928
2	6.000	5.000					59.928
3	11.000	5.000					59.928
4	16.000	5.000	Yes	Yes(DnHill)			59.928
5	21.000	4.021					59.928
6	24.042	3.042					59.928

purlin space = 1'0, 4@5'0, 2@3'½

Say Angle @ ½ point.

P U R L I N

Roof interesting purlin line 4 (Typical) analysis details.

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
5.000 ft	B	1.354 ft	0.000 ft	1.700 ft	-1.180	1.180	-2.281	0.000

Roof interesting purlin line 4 (Typical) analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	D+C + L+
	Check By ASD; No Deflection Limit
2	D- + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
4	D+C + W+
	Check By ASD With 1/3 Increase; No Deflection Limit
5	L+
	No Stress Check; L/150 Deflection Limit

Roof interesting purlin line 4 (Typical) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1L	3.354	8.5Z16	0.000	0.000	0	1	No	1	0.712	web crippling
								4	L/ 104	deflection
1	17.000	8.5Z16	0.000	0.000	1	1	No	1	0.900	bending
								5	L/ 464	deflection

Total design weight per run = 59.928 lbs. Maximum stress ratio = 0.900.

Roof interesting purlin line 4 (Typical) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	9.231	0.000	9.231	0.000
2	UNIF	D+C	ALL	30.533	0.000	30.533	0.000
3	UNIF	L+	ALL	68.166	0.000	68.166	0.000
4	UNIF	W-	1L	-123.842	0.000	-123.842	1.700
5	UNIF	W-	1L	-68.801	1.700	-68.801	3.354
6	UNIF	W-	1	-68.801	0.000	-68.801	15.300
7	UNIF	W-	1	-123.842	15.300	-123.842	17.000
8	UNIF	W+	ALL	74.970	0.000	74.970	0.000

Roof interesting purlin line 1 (At peak edge) analysis details.

Design	Z	Left	Right	Edge	-Int. Zone Coef-	-Ext. Zone Coef-

Spacing	Cond	Inset	Inset	Strip	Suction	Pressure	Suction	Pressure
3.500 ft	B	1.354 ft	0.000 ft	1.700 ft	-2.403	0.000	-2.403	0.000

Roof interesting purlin line 1 (At peak edge) analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	D+C + L+
	Check By ASD; No Deflection Limit
2	D- + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
4	L+
	No Stress Check; L/150 Deflection Limit

Roof interesting purlin line 1 (At peak edge) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1L	3.354	8.5Z16	0.000	0.000	0	1	No	1	0.504	web crippling
								2	L/ 126	deflection
1	17.000	8.5Z16	0.000	0.000	1	1	No	1	0.637	bending
								4	L/ 653	deflection

Total design weight per run = 59.928 lbs. Maximum stress ratio = 0.637.

Roof interesting purlin line 1 (At peak edge) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	6.462	0.000	6.462	0.000
2	UNIF	D+C	ALL	21.373	0.000	21.373	0.000
3	UNIF	L+	ALL	47.716	0.000	47.716	0.000
4	UNIF	W-	ALL	-99.002	0.000	-99.002	0.000

C B C S T E E L B U I L D I N G S

Front Side Wall Girt Design
(typical)

Designer: WW
Job Number: J27335

Version Number: Ver. 22.1
Date/Time: 10/15/07 01:57 PM

Type	Width	Length	Ridge Dist	R.Col.Elev	Slope(F)	Slope(R)	No.BAYS
RF	50.000 ft	18.354 ft	25.000 ft	0.000 ft	5.000:12	5.000:12	1

S.Wall	Eave Ht.	Lean-To Width	E.Wall	Type	Col_Spc.	Girt Type	Overhang
Front:	12.083 ft	0.000 ft	Left	4	S	B	2.000 ft
Rear:	12.083 ft	0.000 ft	Right	7	S	I	0.000 ft

Building Code: 2001 California State Building Code

Category	Seis_Z	Expo_S	Expo_W	Coastal	DL	COLL.	LL
4.	4	3	C	N/A	2.000psf	5.000psf	20.000psf

G_Snow	D_Snow	WL_mph	WL_psf	S.Imp.Fac	S.Fac	W.Imp.Fac
0.000psf	0.000psf	70.000mph	13.760psf	1.000	0.700	1.000

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
5.167 ft	B	1.354 ft	0.000 ft	1.700 ft	-0.941	0.913	-1.241	0.913

Standalone wall girt line analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	W-
	Check By ASD With 1/3 Increase; No Deflection Limit
2	W+
	Check By ASD With 1/3 Increase; No Deflection Limit
3	3/4W-
	No Stress Check; L/120 Deflection Limit
4	3/4W+
	No Stress Check; L/120 Deflection Limit

Standalone wall girt line analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1L	1.354	8.5Z16	0.000	0.000	0	1	No	2	0.276	web crippling
								1	L/ 143	deflection
1	17.000	8.5Z16	0.000	0.000	0	1	No	1	0.540	bending
								3	L/ 604	deflection

Total design weight per run = 54.039 lbs. Maximum stress ratio = 0.540.

Standalone wall girt line analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity	From	Intensity	To

			lb/ft(kips)	feet	lb/ft	feet
1	UNIF W-	1L	-88.197	0.000	-88.197	1.354
2	UNIF W-	1	-88.197	0.000	-88.197	0.346
3	UNIF W-	1	-66.868	0.346	-66.868	13.946
4	UNIF W-	1	-88.197	13.946	-88.197	15.646
5	UNIF W+	1L	64.904	0.000	64.904	1.354
6	UNIF W+	1	64.904	0.000	64.904	15.646

C B C S T E E L B U I L D I N G S
Left End Wall Girt Design
(typical)

Designer: WW
Job Number: J27335
Version Number: Ver. 22.0
Date/Time: 09/10/07 02:32 PM

Type	Width	Length	Ridge Dist	R.Col.Elev	Slope(F)	Slope(R)	No.BAYS
RF	50.000 ft	17.000 ft	25.000 ft	0.000 ft	5.000:12	5.000:12	1

S.Wall	Eave Ht.	Lean-To Width	E.Wall	Type	Col_Spc.	Girt Type	Overhang
Front:	12.083 ft	0.000 ft	Left	4	S	B	2.000 ft
Rear:	12.083 ft	0.000 ft	Right	7	S	I	0.000 ft

Building Code: 2001 California State Building Code

Category	Seis_Z	Expo_S	Expo_W	Coastal	DL	COLL.	LL
4.	4	3	C	N/A	2.000psf	5.000psf	20.000psf

G_Snow	D_Snow	WL_mph	WL_psf	S.Imp.Fac	S.Fac	W.Imp.Fac
0.000psf	0.000psf	70.000mph	13.760psf	1.000	0.700	1.000

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
5.083 ft	B	0.000 ft	0.000 ft	1.700 ft	-0.951	1.036	-1.251	1.036

Standalone wall girt line analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	W-
	Check By ASD With 1/3 Increase; No Deflection Limit
2	W+
	Check By ASD With 1/3 Increase; No Deflection Limit
3	3/4W-
	No Stress Check; L/120 Deflection Limit
4	3/4W+
	No Stress Check; L/120 Deflection Limit

Standalone wall girt line analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1	16.667	8.5Z16	0.000	0.906	0	1	No	2	0.419	web crippling
								4	L/1156	deflection
2	16.667	8.5Z16	0.906	0.906	0	1	No	2	0.331	web crippling
								4	L/10853	deflection
3	16.667	8.5Z16	0.906	0.000	0	1	No	2	0.419	web crippling
								4	L/1156	deflection

Total design weight per run = 158.867 lbs. Maximum stress ratio = 0.419.

Standalone wall girt line analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	W-	1	-87.500	0.000	-87.500	1.700
2	UNIF	W-	1	-66.515	1.700	-66.515	16.667
3	UNIF	W-	2	-66.515	0.000	-66.515	16.667
4	UNIF	W-	3	-66.515	0.000	-66.515	14.967
5	UNIF	W-	3	-87.500	14.967	-87.500	16.667
6	UNIF	W+	ALL	72.479	0.000	72.479	0.000

CBC STEEL BUILDINGS
Front Roof Design
(typical)

part #2

Designer: WW
Job Number: J27335
Version Number: Ver. 22.0
Date/Time: 09/06/07 01:40 PM

Type	Width	Length	Ridge Dist	R.Col.Elev	Slope(F)	Slope(R)	No.BAYS
RF	50.000 ft	74.708 ft	25.000 ft	0.000 ft	5.000:12	5.000:12	3

S.Wall	Eave Ht.	Lean-To Width	E.Wall	Type	Col_Spc.	Girt Type	Overhang
Front:	21.250 ft	0.000 ft	Left	4	C	I	2.000 ft
Rear:	21.250 ft	0.000 ft	Right	4	C	I	2.000 ft

Building Code: 2001 California State Building Code

Category	Seis_Z	Expo_S	Expo_W	Coastal	DL	COLL.	LL
4.	4	3	C	N/A	2.000psf	5.000psf	20.000psf

G_Snow	D_Snow	WL_mph	WL_psf	S.Imp.Fac	S.Fac	W.Imp.Fac
0.000psf	0.000psf	70.000mph	15.141psf	1.000	0.700	1.000

Purlin locations on slope from peak to eave.

Line No.	Dist. (ft)	Design Spacing	Interesting Line	AR Clip	L.edge zone pkg	R.edge zone pkg	Weight (lbs)
1	1.000	3.500	Yes				255.169
2	6.000	5.000					255.169
3	11.000	5.000		Yes(DnHill)			255.169
4	16.000	5.000	Yes	Yes(DnHill)			255.169
5	21.000	4.021					255.169
6	24.042	3.042	Yes				255.169

purlin space = 1'0 4@ 5'0 2@ 3' 1/2 , 2'0 Lap

Sag Angle @ 1/4 point.

P U R L I N

Roof interesting purlin line 4 (Typical) analysis details.

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
5.000 ft	B	1.354 ft	1.354 ft	5.000 ft	-1.000	1.000	-1.800	0.000

Roof interesting purlin line 4 (Typical) analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	D+C + L+
	Check By ASD; No Deflection Limit
2	D- + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
4	D+C + W+
	Check By ASD With 1/3 Increase; No Deflection Limit
5	L+
	No Stress Check; L/150 Deflection Limit

Roof interesting purlin line 4 (Typical) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1L	3.354	8.5Z16	0.000	0.000	0	1	No	1	0.762	web crippling
								4	L/ 58	deflection
1	24.000	8.5Z16	0.000	1.906	3	1	No	1	0.916	bending+shear
								5	L/ 327	deflection
2	24.000	8.5Z16	1.906	1.906	3	1	No	1	0.926	bending+shear
								5	L/3012	deflection
3	24.000	8.5Z16	1.906	0.000	3	1	No	1	0.916	bending+shear
								5	L/ 327	deflection
3R	3.354	8.5Z16	0.000	0.000	0	1	No	1	0.762	web crippling
								4	L/ 58	deflection

Total design weight per run = 255.169 lbs. Maximum stress ratio = 0.926.

Roof interesting purlin line 4 (Typical) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	9.231	0.000	9.231	0.000
2	UNIF	D+C	ALL	30.533	0.000	30.533	0.000
3	UNIF	L+	ALL	68.166	0.000	68.166	0.000
4	UNIF	W-	1L	-136.269	0.000	-136.269	3.354
5	UNIF	W-	1	-136.269	0.000	-136.269	1.646
6	UNIF	W-	1	-75.705	1.646	-75.705	24.000

7	UNIF W-	2	-75.705	0.000	-75.705	24.000
8	UNIF W-	3	-75.705	0.000	-75.705	22.354
9	UNIF W-	3	-136.269	22.354	-136.269	24.000
10	UNIF W-	3R	-136.269	0.000	-136.269	3.354
11	UNIF W+	ALL	82.530	0.000	82.530	0.000

Roof interesting purlin line 6 (At eave edge) analysis details.

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
3.042 ft	B	1.354 ft	1.354 ft	5.000 ft	-2.022	0.000	-2.022	0.000

Roof interesting purlin line 6 (At eave edge) analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	D+C + L+
	Check By ASD; No Deflection Limit
2	D- + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
4	L+
	No Stress Check; L/150 Deflection Limit

Roof interesting purlin line 6 (At eave edge) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1L	3.354	8.5Z16	0.000	0.000	0	1	No	1	0.472	web crippling
								2	L/ 81	deflection
1	24.000	8.5Z16	0.000	1.906	3	1	No	1	0.567	bending+shear
								4	L/ 525	deflection
2	24.000	8.5Z16	1.906	1.906	3	1	No	1	0.573	bending+shear
								4	L/4832	deflection
3	24.000	8.5Z16	1.906	0.000	3	1	No	1	0.567	bending+shear
								4	L/ 525	deflection
3R	3.354	8.5Z16	0.000	0.000	0	1	No	1	0.472	web crippling
								2	L/ 81	deflection

Total design weight per run = 255.169 lbs. Maximum stress ratio = 0.573.

Roof interesting purlin line 6 (At eave edge) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	5.615	0.000	5.615	0.000
2	UNIF	D+C	ALL	18.574	0.000	18.574	0.000
3	UNIF	L+	ALL	41.467	0.000	41.467	0.000
4	UNIF	W-	ALL	-91.166	0.000	-91.166	0.000

Roof interesting purlin line 1 (At peak edge) analysis details.

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
3.500 ft	B	1.354 ft	1.354 ft	5.000 ft	-1.921	0.000	-1.921	0.000

Roof interesting purlin line 1 (At peak edge) analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	D+C + L+
	Check By ASD; No Deflection Limit
2	D- + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
4	L+
	No Stress Check; L/150 Deflection Limit

Roof interesting purlin line 1 (At peak edge) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1L	3.354	8.5Z16	0.000	0.000	0	1	No	1	0.540	web crippling
								2	L/ 74	deflection
1	24.000	8.5Z16	0.000	1.906	3	1	No	1	0.649	bending+shear
								4	L/ 460	deflection
2	24.000	8.5Z16	1.906	1.906	3	1	No	1	0.656	bending+shear
								4	L/4233	deflection
3	24.000	8.5Z16	1.906	0.000	3	1	No	1	0.649	bending+shear
								4	L/ 460	deflection
3R	3.354	8.5Z16	0.000	0.000	0	1	No	1	0.540	web crippling
								2	L/ 74	deflection

Total design weight per run = 255.169 lbs. Maximum stress ratio = 0.656.

Roof interesting purlin line 1 (At peak edge) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	6.462	0.000	6.462	0.000
2	UNIF	D+C	ALL	21.373	0.000	21.373	0.000
3	UNIF	L+	ALL	47.716	0.000	47.716	0.000
4	UNIF	W-	ALL	-99.239	0.000	-99.239	0.000

D-12

C B C S T E E L B U I L D I N G S
Front Side Wall Girt Design
& RSW (typical)

Designer: WW
Job Number: J27335
Version Number: Ver. 22.0
Date/Time: 09/10/07 09:55 AM

Type	Width	Length	Ridge Dist	R.Col.Elev	Slope(F)	Slope(R)	No.BAYS
RF	50.000 ft	74.708 ft	25.000 ft	0.000 ft	5.000:12	5.000:12	3

S.Wall	Eave Ht.	Lean-To Width	E.Wall	Type	Col_Spc.	Girt Type	Overhang
Front:	21.250 ft	0.000 ft	Left	4	C	I	2.000 ft
Rear:	21.250 ft	0.000 ft	Right	4	C	I	2.000 ft

Building Code: 2001 California State Building Code

Category	Seis_Z	Expo_S	Expo_W	Coastal	DL	COLL.	LL
4.	4	3	C	N/A	2.000psf	5.000psf	20.000psf

G_Snow	D_Snow	WL_mph	WL_psf	S.Imp.Fac	S.Fac	W.Imp.Fac
0.000psf	0.000psf	70.000mph	15.141psf	1.000	0.700	1.000

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
5.167 ft	B	1.354 ft	1.354 ft	5.000 ft	-0.900	0.891	-1.200	0.891

Standalone wall girt line analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	W-
	Check By ASD With 1/3 Increase; No Deflection Limit
2	W+
	Check By ASD With 1/3 Increase; No Deflection Limit
3	3/4W-
	No Stress Check; L/120 Deflection Limit
4	3/4W+
	No Stress Check; L/120 Deflection Limit

Standalone wall girt line analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1L	1.354	8.5Z14	0.000	0.000	0	1	No	2	0.191	web crippling
								1	L/ 102	deflection
1	24.000	8.5Z14	0.000	0.906	0	1	No	1	0.487	bending
								3	L/ 497	deflection
2	24.000	8.5Z14	0.906	0.906	0	1	No	1	0.400	bearing at bolt
								3	L/5186	deflection
3	24.000	8.5Z14	0.906	0.000	0	1	No	1	0.487	bending
								3	L/ 497	deflection
3R	1.354	8.5Z14	0.000	0.000	0	1	No	2	0.191	web crippling
								1	L/ 102	deflection

Total design weight per run = 298.334 lbs. Maximum stress ratio = 0.487.

Standalone wall girt line analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	W-	1L	-93.874	0.000	-93.874	1.354
2	UNIF	W-	1	-93.874	0.000	-93.874	3.646
3	UNIF	W-	1	-70.406	3.646	-70.406	24.000
4	UNIF	W-	2	-70.406	0.000	-70.406	24.000
5	UNIF	W-	3	-70.406	0.000	-70.406	20.354
6	UNIF	W-	3	-93.874	20.354	-93.874	24.000
7	UNIF	W-	3R	-93.874	0.000	-93.874	1.354
8	UNIF	W+	ALL	69.722	0.000	69.722	0.000

D-14

C B C S T E E L B U I L D I N G S
Left End Wall Girt Design
& REW (typical)

Designer: WW
Job Number: J27335
Version Number: Ver. 22.0
Date/Time: 09/06/07 10:46 AM

Type	Width	Length	Ridge Dist	R.Col.Elev	Slope(F)	Slope(R)	No.BAYS
RF	50.000 ft	74.708 ft	25.000 ft	0.000 ft	5.000:12	5.000:12	3

S.Wall	Eave Ht.	Lean-To Width	E.Wall	Type	Col_Spc.	Girt Type	Overhang
Front:	21.250 ft	0.000 ft	Left	4	C	I	2.000 ft
Rear:	21.250 ft	0.000 ft	Right	4	C	I	2.000 ft

Building Code: 2001 California State Building Code

Category	Seis_Z	Expo_S	Expo_W	Coastal	DL	COLL.	LL
4.	4	3	C	N/A	2.000psf	12.000psf	20.000psf

G_Snow	D_Snow	WL_mph	WL_psf	S.Imp.Fac	S.Fac	W.Imp.Fac
0.000psf	0.000psf	70.000mph	15.141psf	1.000	0.700	1.000

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
3.000 ft	I	0.000 ft	0.000 ft	5.000 ft	-0.983	1.072	-1.283	1.072

Standalone wall girt line analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	W-
	Check By ASD With 1/3 Increase; No Deflection Limit
2	W+
	Check By ASD With 1/3 Increase; No Deflection Limit
3	3/4W-
	No Stress Check; L/120 Deflection Limit
4	3/4W+
	No Stress Check; L/120 Deflection Limit

Standalone wall girt line analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1	25.000	8.5Z16	0.000	0.000	0	1	No	1	0.810	bending
								4	L/ 257	deflection
2	25.000	8.5Z16	0.000	0.000	0	1	No	1	0.810	bending
								4	L/ 257	deflection

Total design weight per run = 147.213 lbs. Maximum stress ratio = 0.810.

Standalone wall girt line analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity	From	Intensity	To

D-15

			lb/ft(kips)	feet	lb/ft	feet
1	UNIF W-	1	-58.293	0.000	-58.293	5.000
2	UNIF W-	1	-44.666	5.000	-44.666	25.000
3	UNIF W-	2	-44.666	0.000	-44.666	20.000
4	UNIF W-	2	-58.293	20.000	-58.293	25.000
5	UNIF W+	ALL	48.693	0.000	48.693	0.000

D-16

C B C S T E E L B U I L D I N G S
R o o f D e s i g n
(typical)

part #3

Designer: WW
Job Number: J27335

Version Number: Ver. 22.0
Date/Time: 01/23/08 10:15 AM

Type	Width	Length	Ridge Dist	R.Col.Elev	Slope(F)	Slope(R)	No.BAYS
RF	50.000 ft	85.354 ft	25.000 ft	0.000 ft	5.000:12	5.000:12	3

S.Wall	Eave Ht.	Lean-To Width	E.Wall	Type	Col_Spc.	Girt Type	Overhang
Front:	12.083 ft	0.000 ft	Left	7	S	I	0.000 ft
Rear:	12.083 ft	0.000 ft	Right	4	S	B	0.000 ft

Building Code: 2001 California State Building Code

Category	Seis_Z	Expo_S	Expo_W	Coastal	DL	COLL.	LL
4.	4	3	C	N/A	2.000psf	5.000psf	20.000psf

G_Snow	D_Snow	WL_mph	WL_psf	S.Imp.Fac	S.Fac	W.Imp.Fac
0.000psf	0.000psf	70.000mph	13.760psf	1.000	0.700	1.000

Purlin locations on slope from peak to eave.

Line No.	Dist. (ft)	Design Spacing	Interesting Line	AR Clip	L.edge zone pkg	R.edge zone pkg	Weight (lbs)
1	1.000	3.000					374.101
2	5.000	4.000	Yes				374.101
3	9.000	4.000					374.101
4	13.000	4.000					374.101
5	17.000	4.000	Yes				374.101
6	21.000	3.521	Yes				374.101
7	24.042	3.042	Yes				374.101

Purlin space = 1'0, 5@4'0, 2@3'1/2, 2'8 Lap @Lin 6, ~~4'0 Lap @Line~~

Sag Bocking @ 1/5 point.

P U R L I N

Roof interesting purlin line 5 (Typical) analysis details.

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
4.000 ft	B	0.000 ft	1.354 ft	5.000 ft	-1.000	1.000	-1.800	0.000

Roof interesting purlin line 5 (Typical) analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	D+C + L+
	Check By ASD; No Deflection Limit
2	D- + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
4	L+
	No Stress Check; L/150 Deflection Limit

Roof interesting purlin line 5 (Typical) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1	28.000	8.5Z14	0.000	2.573	4	1	No	1	0.959	bending
								4	L/ 297	deflection
2	28.000	8.5Z14	2.573	3.906	4	1	No	1	0.830	bending+shear
								4	L/2168	deflection
3	28.000	8.5Z14	3.906	0.000	4	1	No	1	0.927	bending
								4	L/ 311	deflection
3R	1.354	8.5Z14	0.000	0.000	0	1	No	1	0.462	web crippling
								1	L/ 46	deflection

Total design weight per run = 374.101 lbs. Maximum stress ratio = 0.959.

Roof interesting purlin line 5 (Typical) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	7.385	0.000	7.385	0.000
2	UNIF	L+	ALL	54.533	0.000	54.533	0.000
3	UNIF	W-	1	-99.074	0.000	-99.074	5.000
4	UNIF	W-	1	-55.041	5.000	-55.041	28.000
5	UNIF	W-	2	-55.041	0.000	-55.041	28.000
6	UNIF	W-	3	-55.041	0.000	-55.041	24.354
7	UNIF	W-	3	-99.074	24.354	-99.074	28.000
8	UNIF	W-	3R	-99.074	0.000	-99.074	1.354
9	UNIF	D+C	ALL	48.284	0.000	48.284	0.000

Roof interesting purlin line 7 (At eave edge) analysis details.

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
3.042 ft	B	0.000 ft	1.354 ft	5.000 ft	-1.914	0.000	-1.914	0.000

Roof interesting purlin line 7 (At eave edge) analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	D+C + L+
	Check By ASD; No Deflection Limit
2	D- + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
4	L+
	No Stress Check; L/150 Deflection Limit

Roof interesting purlin line 7 (At eave edge) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1	28.000	8.5Z14	0.000	2.573	4	1	No	1	0.737	bending
								4	L/ 384	deflection
2	28.000	8.5Z14	2.573	3.906	4	1	No	1	0.638	bending+shear
								4	L/2799	deflection
3	28.000	8.5Z14	3.906	0.000	4	1	No	1	0.713	bending
								4	L/ 402	deflection
3R	1.354	8.5Z14	0.000	0.000	0	1	No	1	0.355	web crippling
								1	L/ 60	deflection

Total design weight per run = 374.101 lbs. Maximum stress ratio = 0.737.

Roof interesting purlin line 7 (At eave edge) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	5.615	0.000	5.615	0.000
2	UNIF	L+	ALL	41.467	0.000	41.467	0.000
3	UNIF	W-	ALL	-80.345	0.000	-80.345	0.000
4	UNIF	D+C	ALL	36.716	0.000	36.716	0.000

Roof interesting purlin line 6 (Designed Line) analysis details.

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
3.521 ft	B	0.000 ft	1.354 ft	5.000 ft	-1.000	1.000	-1.800	0.000

Roof interesting purlin line 6 (Designed Line) analysis details.

Design Data - Load Combinations

No.	Load Case Description	
1	D+C + L+	Check By ASD; No Deflection Limit
2	D- + W-	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-	Check By ASD With 1/3 Increase; No Deflection Limit
4	D+C + W+	Check By ASD With 1/3 Increase; No Deflection Limit
5	L+	No Stress Check; L/150 Deflection Limit

Roof interesting purlin line 6 (Designed Line) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1	28.000	8.5Z14	0.000	2.573	4	1	No	1	0.774	bending
								5	L/ 335	deflection
2	28.000	8.5Z14	2.573	3.906	4	1	No	1	0.666	bending+shear
								5	L/2444	deflection
3	28.000	8.5Z14	3.906	0.000	4	1	No	1	0.699	bending
								5	L/ 351	deflection
3R	1.354	8.5Z14	0.000	0.000	0	1	No	1	0.330	web crippling
								4	L/ 57	deflection

Total design weight per run = 374.101 lbs. Maximum stress ratio = 0.774.

Roof interesting purlin line 6 (Designed Line) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	6.500	0.000	6.500	0.000
2	UNIF	L+	ALL	48.000	0.000	48.000	0.000
3	UNIF	W-	1	-87.206	0.000	-87.206	5.000
4	UNIF	W-	1	-48.448	5.000	-48.448	28.000
5	UNIF	W-	2	-48.448	0.000	-48.448	28.000
6	UNIF	W-	3	-48.448	0.000	-48.448	24.354
7	UNIF	W-	3	-87.206	24.354	-87.206	28.000
8	UNIF	W-	3R	-87.206	0.000	-87.206	1.354
9	UNIF	W+	ALL	52.791	0.000	52.791	0.000
10	UNIF	D+C	ALL	21.500	0.000	21.500	0.000
11	CONC	D+C	1	0.200	14.000	0.000	0.000
12	CONC	D+C	2	0.200	14.000	0.000	0.000
13	CONC	D+C	3	0.200	10.000	0.000	0.000

Roof interesting purlin line 2 (At peak edge) analysis details.

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure

D-19-1

4.000 ft B 0.000 ft 1.354 ft 5.000 ft -1.800 0.000 -1.800 0.000

Roof interesting purlin line 2 (At peak edge) analysis details.
Design Data - Load Combinations

No.	Load Case Description	
1	D+C + L+	Check By ASD; No Deflection Limit
2	D- + W-	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-	Check By ASD With 1/3 Increase; No Deflection Limit
4	L+	No Stress Check; L/150 Deflection Limit

Roof interesting purlin line 2 (At peak edge) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1	28.000	8.5Z14	0.000	2.573	4	1	No	1	0.959	bending
								4	L/ 297	deflection
2	28.000	8.5Z14	2.573	3.906	4	1	No	1	0.830	bending+shear
								4	L/2168	deflection
3	28.000	8.5Z14	3.906	0.000	4	1	No	1	0.927	bending
								4	L/ 311	deflection
3R	1.354	8.5Z14	0.000	0.000	0	1	No	1	0.462	web crippling
								1	L/ 46	deflection

Total design weight per run = 374.101 lbs. Maximum stress ratio = 0.959.

Roof interesting purlin line 2 (At peak edge) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	7.385	0.000	7.385	0.000
2	UNIF	L+	ALL	54.533	0.000	54.533	0.000
3	UNIF	W-	ALL	-99.074	0.000	-99.074	0.000
4	UNIF	D+C	ALL	48.284	0.000	48.284	0.000

C B C S T E E L B U I L D I N G S
Front Side Wall Girt Design
(typical)

Designer: WW
Job Number: J27335

Version Number: Ver. 22.1
Date/Time: 10/15/07 02:47 PM

Type	Width	Length	Ridge Dist	R.Col.Elev	Slope(F)	Slope(R)	No.BAYS
RF	50.000 ft	85.354 ft	25.000 ft	0.000 ft	5.000:12	5.000:12	3

S.Wall	Eave Ht.	Lean-To Width	E.Wall	Type	Col_Spc.	Girt Type	Overhang
Front:	12.083 ft	0.000 ft	Left	7	S	I	0.000 ft
Rear:	12.083 ft	0.000 ft	Right	4	S	B	0.000 ft

Building Code: 2001 California State Building Code

Category	Seis_Z	Expo_S	Expo_W	Coastal	DL	COLL.	LL
4.	4	3	C	N/A	2.000psf	5.000psf	20.000psf

G_Snow	D_Snow	WL_mph	WL_psf	S.Imp.Fac	S.Fac	W.Imp.Fac
0.000psf	0.000psf	70.000mph	13.760psf	1.000	0.700	1.000

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
5.167 ft	B	0.000 ft	1.354 ft	5.000 ft	-0.900	0.874	-1.200	0.874

Standalone wall girt line analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	W- Check By ASD With 1/3 Increase; No Deflection Limit
2	W+ Check By ASD With 1/3 Increase; No Deflection Limit
3	3/4W- No Stress Check; L/120 Deflection Limit
4	3/4W+ No Stress Check; L/120 Deflection Limit

Standalone wall girt line analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1	28.000	8.5Z16	0.000	0.906	0	1	No	1	0.858	bending
								3	L/ 261	deflection
2	28.000	8.5Z16	0.906	0.906	0	1	No	2	0.743	bending
								3	L/2766	deflection
3	28.000	8.5Z16	0.906	0.000	0	1	No	1	0.844	bending
								3	L/ 265	deflection
3R	1.354	8.5Z16	0.000	0.000	0	1	No	2	0.325	web crippling
								1	L/ 54	deflection

Total design weight per run = 262.959 lbs. Maximum stress ratio = 0.858.

Standalone wall girt line analysis details.

Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	W-	1	-85.314	0.000	-85.314	5.000
2	UNIF	W-	1	-63.985	5.000	-63.985	28.000
3	UNIF	W-	2	-63.985	0.000	-63.985	28.000
4	UNIF	W-	3	-63.985	0.000	-63.985	24.000
5	UNIF	W-	3	-85.314	24.000	-85.314	28.000
6	UNIF	W-	3R	-85.314	0.000	-85.314	1.000
7	UNIF	W+	1	62.105	0.000	62.105	28.000
8	UNIF	W+	2	62.105	0.000	62.105	28.000
9	UNIF	W+	3	62.105	0.000	62.105	28.000
10	UNIF	W+	3R	62.105	0.000	62.105	1.000

D-22

C B C S T E E L B U I L D I N G S
Right End Wall Girt Design
(typical)

Designer: WW
Job Number: J27335
Version Number: Ver. 22.1
Date/Time: 10/15/07 02:29 PM

Type	Width	Length	Ridge Dist	R.Col.Elev	Slope(F)	Slope(R)	No.BAYS
RF	50.000 ft	85.000 ft	25.000 ft	0.000 ft	5.000:12	5.000:12	3

S.Wall	Eave Ht.	Lean-To Width	E.Wall	Type	Col_Spc.	Girt Type	Overhang
Front:	12.083 ft	0.000 ft	Left	7	S	I	0.000 ft
Rear:	12.083 ft	0.000 ft	Right	4	S	B	0.000 ft

Building Code: 2001 California State Building Code

Category	Seis_Z	Expo_S	Expo_W	Coastal	DL	COLL.	LL
4.	4	3	C	N/A	2.000psf	5.000psf	20.000psf

G_Snow	D_Snow	WL_mph	WL_psf	S.Imp.Fac	S.Fac	W.Imp.Fac
0.000psf	0.000psf	70.000mph	13.760psf	1.000	0.700	1.000

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
5.083 ft	B	0.000 ft	0.000 ft	5.000 ft	-0.951	1.036	-1.251	1.036

Standalone wall girt line design details.
Design Data - Load Combinations

No.	Load Case Description
1	W-
	Check By ASD With 1/3 Increase; No Deflection Limit
2	W+
	Check By ASD With 1/3 Increase; No Deflection Limit
3	3/4W-
	No Stress Check; L/120 Deflection Limit
4	3/4W+
	No Stress Check; L/120 Deflection Limit

Standalone wall girt line design details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest. Mem #	Web Clip	Load Case	Check Ratio	Control Check
1	21.000	8.5Z16	0.000	0.906	0	1	No	2	0.549	web crippling
								4	L/ 508	deflection
2	8.000	8.5Z16	0.906	0.906	0	1	No	2	0.417	bending
								4	L/1085	deflection
3	21.000	8.5Z16	0.906	0.000	0	1	No	2	0.549	web crippling
								4	L/ 508	deflection

Total design weight per run = 158.867 lbs. Maximum stress ratio = 0.549.

Standalone wall girt line design details.
Design Data - Applied loads

D-23

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	W-	1	-87.500	0.000	-87.500	5.000
2	UNIF	W-	1	-66.515	5.000	-66.515	21.000
3	UNIF	W-	2	-66.515	0.000	-66.515	8.000
4	UNIF	W-	3	-66.515	0.000	-66.515	16.000
5	UNIF	W-	3	-87.500	16.000	-87.500	21.000
6	UNIF	W+	ALL	72.479	0.000	72.479	0.000

CBC Steel BUILDINGS
Front Roof Design
(typical)

Designer: WW
Job Number: J27335
Version Number: Ver. 22.0
Date/Time: 09/08/07 02:17 PM

Type	Width	Length	Ridge Dist	R.Col.Elev	Slope(F)	Slope(R)	No.BAYS
RF	28.000 ft	40.000 ft	14.000 ft	0.000 ft	5.000:12	5.000:12	2

S.Wall	Eave Ht.	Lean-To Width	E.Wall	Type	Col_Spc.	Girt Type	Overhang
Front:	12.083 ft	0.000 ft	Left	6	C	I	0.000 ft
Rear:	12.083 ft	0.000 ft	Right	7	C	I	0.000 ft

Building Code: 2001 California State Building Code

Category	Seis_Z	Expo_S	Expo_W	Coastal	DL	COLL.	LL
4.	4	3	C	N/A	2.000psf	5.000psf	20.000psf

G_Snow	D_Snow	WL_mph	WL_psf	S.Imp.Fac	S.Fac	W.Imp.Fac
0.000psf	0.000psf	70.000mph	13.356psf	1.000	0.700	1.000

Purlin locations on slope from peak to eave.

Line No.	Dist. (ft)	Design Spacing	Interesting Line	AR Clip	L.edge zone pkg	R.edge zone pkg	Weight (lbs)
1	1.500	4.000	Yes				129.486
2	6.500	5.000	Yes	Yes(DnHill)			129.486
3	11.500	4.333					129.486

Purlin space = 1'0 2@5'0 , 3'8

2'0 Lap

Sag Angle @ 1/3 point

P U R L I N

Roof interesting purlin line 2 (Typical) analysis details.

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
5.000 ft	B	1.063 ft	0.000 ft	2.800 ft	-1.300	0.500	-1.800	0.000

Roof interesting purlin line 2 (Typical) analysis details.
Design Data - Load Combinations

No.	Load Case Description	
1	D+C + L+	Check By ASD; No Deflection Limit
2	D- + W-	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-	Check By ASD With 1/3 Increase; No Deflection Limit
4	D+C + W+	Check By ASD With 1/3 Increase; No Deflection Limit
5	L+	No Stress Check; L/150 Deflection Limit

Roof interesting purlin line 2 (Typical) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1L	1.063	8.5Z16	0.000	0.000	0	1	No	1	0.914	web crippling
								1	L/ 87	deflection
1	21.479	8.5Z16	0.000	1.906	2	1	No	1	0.914	web crippling
								5	L/ 471	deflection
2	17.458	8.5Z16	1.906	0.000	0	1	No	1	0.822	bending+shear
								5	L/1974	deflection

Total design weight per run = 129.486 lbs. Maximum stress ratio = 0.914.

Roof interesting purlin line 2 (Typical) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	9.231	0.000	9.231	0.000
2	UNIF	D+C	ALL	30.533	0.000	30.533	0.000
3	UNIF	L+	ALL	68.166	0.000	68.166	0.000
4	UNIF	W-	1L	-120.204	0.000	-120.204	1.063
5	UNIF	W-	1	-120.204	0.000	-120.204	1.737
6	UNIF	W-	1	-86.814	1.737	-86.814	21.479
7	UNIF	W-	2	-86.814	0.000	-86.814	14.658
8	UNIF	W-	2	-120.204	14.658	-120.204	17.458
9	UNIF	W+	ALL	35.595	0.000	35.595	0.000

Roof interesting purlin line 1 (At peak edge) analysis details.

Design Spacing	Z Cond	Left Inset	Right Inset	Edge Strip	-Int. Zone Coef- Suction	Pressure	-Ext. Zone Coef- Suction	Pressure
4.000 ft	B	1.063 ft	0.000 ft	2.800 ft	-1.978	0.000	-1.978	0.000

Roof interesting purlin line 1 (At peak edge) analysis details.
Design Data - Load Combinations

No.	Load Case Description
1	D+C + L+
	Check By ASD; No Deflection Limit
2	D- + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
3	D+C + W-
	Check By ASD With 1/3 Increase; No Deflection Limit
4	L+
	No Stress Check; L/150 Deflection Limit

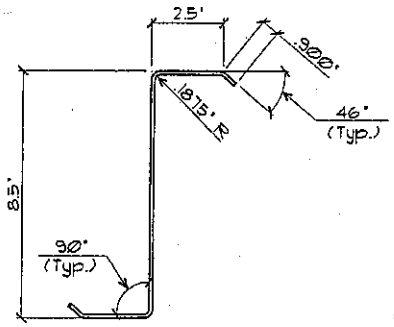
Roof interesting purlin line 1 (At peak edge) analysis details.
Design Data - Summary

Span ID	Length (ft)	Mark No.	Left Lap (ft)	Right Lap (ft)	Brace Pts #	Nest Mem #	Web Clip	Load Case	Check Ratio	Control Check
1L	1.063	8.5Z16	0.000	0.000	0	1	No	1	0.736	web crippling
								2	L/ 93	deflection
1	21.479	8.5Z16	0.000	1.906	2	1	No	1	0.736	web crippling
								4	L/ 584	deflection
2	17.458	8.5Z16	1.906	0.000	0	1	No	1	0.662	bending+shear
								4	L/2444	deflection

Total design weight per run = 129.486 lbs. Maximum stress ratio = 0.736.

Roof interesting purlin line 1 (At peak edge) analysis details.
Design Data - Applied loads

No.	Load Type	Load Group	Span #	Intensity lb/ft(kips)	From feet	Intensity lb/ft	To feet
1	UNIF	D-	ALL	7.385	0.000	7.385	0.000
2	UNIF	D+C	ALL	24.426	0.000	24.426	0.000
3	UNIF	L+	ALL	54.533	0.000	54.533	0.000
4	UNIF	W-	ALL	-105.661	0.000	-105.661	0.000



ZEE SECTION PROPERTIES

ZEE SECTION PHYSICAL PROPERTIES						
SIZE	D (IN)	B (IN)	L (IN)	T (IN)	WGT (LB/FT)	A (IN^2)
8.5 Z 16	8.5	2.5	0.900	0.058	2.944	0.866
8.5 Z 14	8.5	2.5	0.900	0.075	3.793	1.115
8.5 Z 12	8.5	2.5	0.900	0.102	5.127	1.508

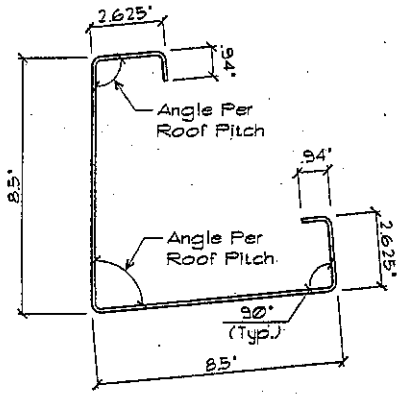
ZEE SECTION FULL PROPERTIES							
SIZE	I_x (IN^4)	r_x (IN)	I_y (IN^4)	r_y (IN)	J (IN^4)	C_w (IN^6)	r_o (IN)
8.5 Z 16	9.344	3.285	1.359	1.253	0.000971	17.445	3.516
8.5 Z 14	11.977	3.277	1.733	1.246	0.002091	22.146	3.506
8.5 Z 12	16.060	3.264	2.303	1.236	0.00523	29.243	3.490

ZEE SECTION EFFECTIVE PROPERTIES								
SIZE	I_x (IN^4)	S_{xc} (IN^3)	S_{xt} (IN^3)	M_n (K IN)	M_a (K IN)	V_X (KIPS)	DEFL. I_x (IN^4)	A_e (IN^2)
8.5 Z 16	8.358	1.828	2.129	100.519	60.191	2.077	9.344	0.450
8.5 Z 14	11.211	2.534	2.751	139.341	83.438	4.510	11.977	0.644
8.5 Z 12	16.060	3.779	3.779	207.831	124.450	11.42	16.060	1.101

NOTES:

1. SECTION PROPERTIES ARE CALCULATED IN ACCORDANCE WITH THE 1996 AISI SPECIFICATION FOR THE DESIGN OF COLD FORMED STEEL MEMBERS.

2. BENDING ALLOWABLES SHOWN MAY BE UTILIZED FOR MEMBERS HAVING THE COMPRESSION FLANGE CONTINUOUSLY FASTENED TO DECK OR SHEATHING.



EAVE STRUT SECTION PROPERTIES

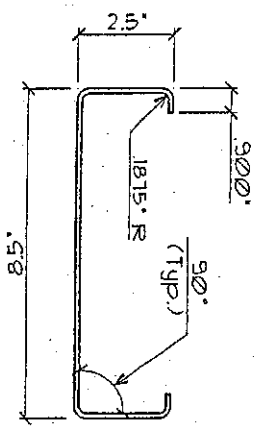
EAVE STRUT SECTION PHYSICAL PROPERTIES						
SIZE	D (IN)	B (IN)	L (IN)	T (IN)	WGT (LB/FT)	A ($1N^2$)
ES 1:12	8.5	2.625/8.5	0.940	0.058	4.733	1.392
ES 4:12	8.5	2.625/8.5	0.940	0.058	4.733	1.392

(16 GA)

EAVE STRUT SECTION FULL PROPERTIES							
SIZE	I_x (IN^4)	r_x (IN)	I_y (IN^4)	r_y (IN)	J (IN^4)	C_w (IN^6)	r_o (IN)
ES 1:12	12.836	3.081	13.541	3.164	0.001517	158.317	8.856
ES 4:12	12.579	3.023	11.705	2.916	0.001543	98.303	8.455

EAVE STRUT SECTION EFFECTIVE PROPERTIES								
SIZE	I_x (IN^4)	S_{xc} (IN^3)	S_{xt} (IN^3)	M_n (K IN)	M_a (K IN)	V_a (KIPS)	DEFL. I_x (IN^4)	A_e (IN^2)
ES 1:12 TOP	10.899	1.791	4.570	98.505	59.103	2.050	12.551	0.640
BOTTOM	10.210	2.669	2.198	120.890	72.534	2.050	10.538	0.640
ES 4:12 TOP	9.769	1.690	4.281	92.950	55.770	2.069	11.181	0.641
BOTTOM	9.204	2.504	2.097	115.335	69.201	2.069	9.520	0.641

NOTES:
1. SECTION PROPERTIES ARE CALCULATED IN ACCORDANCE WITH THE 1996 AISI SPECIFICATION FOR THE DESIGN OF COLD FORMED STEEL MEMBERS.
2. BENDING ALLOWABLES SHOWN MAY BE UTILIZED FOR MEMBERS HAVING THE COMPRESSION FLANGE CONTINUOUSLY FASTENED TO DECK OR SHEATHING.



CEE SECTION PROPERTIES

CEE SECTION PHYSICAL PROPERTIES						
SIZE	D (IN)	B (IN)	L (IN)	T (IN)	WGT (LB/FT)	A ($1N^2$)
8.5 C 16	8.5	2.5	0.900	0.058	2.898	0.852
8.5 C 14	8.5	2.5	0.900	0.075	3.727	1.096
8.5 C 12	8.5	2.5	0.900	0.102	5.022	1.477

CEE SECTION FULL PROPERTIES							
SIZE	I_x (IN^4)	r_x (IN)	I_y (IN^4)	r_y (IN)	J (IN^4)	C_w (IN^6)	r_o (IN)
8.5 C 16	9.019	3.253	0.724	0.921	0.000956	10.766	3.817
8.5 C 14	11.528	3.243	0.914	0.913	0.002055	13.515	3.798
8.5 C 12	15.391	3.228	1.198	0.901	0.00512	17.527	3.768

CEE SECTION EFFECTIVE PROPERTIES								
SIZE	I_x (IN^4)	S_{xc} (IN^3)	S_{xt} (IN^3)	M_n (K IN)	M_a (K IN)	V_a (KIPS)	DEFL. I_x (IN^4)	A_e (IN^2)
8.5 C 16	8.401	1.886	2.077	103.717	62.106	2.077	9.019	0.468
8.5 C 14	11.256	2.608	2.690	143.426	85.884	4.510	11.528	0.682
8.5 C 12	15.391	3.621	3.621	199.181	119.270	11.42	15.391	1.070

NOTES:

1. SECTION PROPERTIES ARE CALCULATED IN ACCORDANCE WITH THE 1996 AISI SPECIFICATION FOR THE DESIGN OF COLD FORMED STEEL MEMBERS.

2. BENDING ALLOWABLES SHOWN MAY BE UTILIZED FOR MEMBERS HAVING THE COMPRESSION FLANGE CONTINUOUSLY FASTENED TO DECK OR SHEATHING.

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Column and Beam Endwall Design  Ver. 22.0                    Page   1
CBC Steel Buildings                          Mon Sep 10 14:34:32 2007
Job Name: J27335   Job Part: 1   LEW
```

BUILDING TYPE IS SYMMETRICAL SINGLE GABLE

BUILDING WIDTH	=	50.000	ft	BUILDING LENGTH	=	17.000	ft
LEFT HEIGHT	=	12.083	ft	RIGHT HEIGHT	=	12.083	ft
LEFT SLOPE	=	5.000	:12	RIGHT SLOPE	=	5.000	:12
BAY SPACING	=	17.000	ft	ROOF OVERHANG	=	2.000	ft

BUILDING CODE:
2001 California State Building Code
BUILDING USE CATEGORY:
1. Essential facilities
2. Hazardous facilities
3. Special occupancy structures
■4. Standard occupancy Structures
5. Miscellaneous structures
SNOW EXPOSURE CATEGORY:
1. Roofs located in generally open terrain extending for one-half mile or more from the structure
2. Structures located in densely forested or sheltered areas
■3. All other structures
ENCLOSURE CONDITION:
■Enclosed
Partially enclosed
Unenclosed
WIND EXPOSURE CATEGORY:
B. Terrain with buildings, forest, or surface irregularities covering at least 20% of the ground area extending 1 mile or more from the site
■C. Terrain which is flat and generally open, extending to 1/2 mile or more from the site in any full quadrant
D. Areas with basic wind speeds (fastest mile) of 80 mph or greater and terrain that is flat and unobstructed facing large bodies of water

DESIGN ROOF LIVE LOAD	=	16.000	psf
COLLATERAL LOAD	=	5.000	psf
SNOW EXPOSURE FACTOR	=	0.700	
SNOW IMPORTANCE FACTOR	=	1.000	
SLOPE ROOF SNOW LOAD	=	0.000	psf
DESIGN WIND VELOCITY	=	70.000	mph
WIND IMPORTANCE FACTOR	=	1.000	

*** DESIGN LOAD COMBINATIONS

CASE	LOAD FACT	GROUP =>
1	1.00 W+	
2	1.00 W-	

*** LOADS

GROUP	TYPE	M	FM	TO	CL	D	START ft	HORIZ psf/ kips	VERT psf/ kips	MOMT kip-ft	END ft	HORIZ psf	VERT psf
---	---	-	--	--	--	-	---	---	---	---	---	---	---

E-3

BUILDING TYPE IS SYMMETRICAL SINGLE GABLE

BUILDING WIDTH	=	50.000	ft	BUILDING LENGTH	=	74.708	ft
LEFT HEIGHT	=	21.250	ft	RIGHT HEIGHT	=	21.250	ft
LEFT SLOPE	=	5.000	:12	RIGHT SLOPE	=	5.000	:12
BAY SPACING	=	25.354	ft	ROOF OVERHANG	=	2.000	ft

BUILDING CODE:
2001 California State Building Code
BUILDING USE CATEGORY:
1. Essential facilities
2. Hazardous facilities
3. Special occupancy structures
■4. Standard occupancy Structures
5. Miscellaneous structures
SNOW EXPOSURE CATEGORY:
1. Roofs located in generally open terrain extending for one-half mile or more from the structure
2. Structures located in densely forested or sheltered areas
■3. All other structures
ENCLOSURE CONDITION:
■Enclosed
Partially enclosed
Unenclosed
WIND EXPOSURE CATEGORY:
B. Terrain with buildings, forest, or surface irregularities covering at least 20% of the ground area extending 1 mile or more from the site
■C. Terrain which is flat and generally open, extending to 1/2 mile or more from the site in any full quadrant
D. Areas with basic wind speeds (fastest mile) of 80 mph or greater and terrain that is flat and unobstructed facing large bodies of water

DESIGN ROOF LIVE LOAD	=	16.000	psf
COLLATERAL LOAD	=	5.000	psf
SNOW EXPOSURE FACTOR	=	0.700	
SNOW IMPORTANCE FACTOR	=	1.000	
SLOPE ROOF SNOW LOAD	=	0.000	psf
DESIGN WIND VELOCITY	=	70.000	mph
WIND IMPORTANCE FACTOR	=	1.000	

*** DESIGN LOAD COMBINATIONS

CASE	LOAD FACT	GROUP =>
1	1.00 D+C	1.00 L
2	1.00 D	1.00 W+
3	1.00 D	1.00 W-

*** LOADS

GROUP	TYPE	M	FM	TO	CL	D	START ft	HORIZ psf/ kips	VERT psf/ kips	MOMT kip-ft	END ft	HORIZ psf	VERT psf
W+	UNIF	C	1	1	0		0.00	14.86	0.00	0.00	0.00	14.86	0.00
W-	UNIF	C	1	1	0		0.00	-18.17	0.00	0.00	0.00	-18.17	0.00
W+	UNIF	C	2	3	0		0.00	14.86	0.00	0.00	0.00	14.86	0.00
W-	UNIF	C	2	3	0		0.00	-13.63	0.00	0.00	0.00	-13.63	0.00
W+	UNIF	C	4	4	0		0.00	14.86	0.00	0.00	0.00	14.86	0.00
W-	UNIF	C	4	4	0		0.00	-18.17	0.00	0.00	0.00	-18.17	0.00
D	CONC	C	2	3	0		18.00	0.00	-0.50	0.00			
D+C	CONC	C	2	3	0		18.00	0.00	-1.20	0.00			
L	CONC	C	2	3	0		18.00	0.00	-2.90	0.00			
W+	CONC	C	2	3	0		18.00	0.00	2.00	0.00			
W-	CONC	C	2	3	0		18.00	0.00	2.00	0.00			

FY COLD FORMED = 55.0 ksi
FY HOT ROLLED = 50.0 ksi
FY BUILT UP = 55.0 ksi

*** ENDWALL COLUMNS

MEM	DESCRIPTION	LOCATION ft	BASE PLATES in	in	in	A BLT ROW in		BEND RT	SHR RT	CT
2	BUILT-UP	16.667	8	0.500	9.00	2	0.75	0.614	0.106	
	8.0 X 0.375 in FLG,		0.188 in WEB			7.750 in O/O WEB				
3	BUILT-UP	33.333	8	0.500	9.00	2	0.75	0.614	0.106	
	8.0 X 0.375 in FLG,		0.188 in WEB			7.750 in O/O WEB				

*** MAXIMUM ENDWALL REACTIONS AND DESIGN LOAD COMBINATIONS

CASE	M VERT kips	M HORZ kips	LOAD FACTOR / LOAD GROUP =>
1	4.1	0.0	1.0 D+C 1.0 L
2	-1.5	-3.4	1.0 D 1.0 W+
3	-1.5	3.1	1.0 D 1.0 W-
4	-2.0	-3.4	1.0 W+
5	-2.0	3.1	1.0 W-
6	0.5	0.0	1.0 D
7	2.9	0.0	1.0 L

E-4.1

BUILDING TYPE IS SYMMETRICAL SINGLE GABLE

BUILDING WIDTH	=	50.000	ft	BUILDING LENGTH	=	85.354	ft
LEFT HEIGHT	=	12.083	ft	RIGHT HEIGHT	=	12.083	ft
LEFT SLOPE	=	5.000	:12	RIGHT SLOPE	=	5.000	:12
BAY SPACING	=	29.354	ft	ROOF OVERHANG	=	0.000	ft

BUILDING CODE:
2001 California State Building Code
BUILDING USE CATEGORY:
1. Essential facilities
2. Hazardous facilities
3. Special occupancy structures
■4. Standard occupancy Structures
5. Miscellaneous structures
SNOW EXPOSURE CATEGORY:
1. Roofs located in generally open terrain extending for one-half mile or more from the structure
2. Structures located in densely forested or sheltered areas
■3. All other structures
ENCLOSURE CONDITION:
■Enclosed
Partially enclosed
Unenclosed
WIND EXPOSURE CATEGORY:
B. Terrain with buildings, forest, or surface irregularities covering at least 20% of the ground area extending 1 mile or more from the site
■C. Terrain which is flat and generally open, extending to 1/2 mile or more from the site in any full quadrant
D. Areas with basic wind speeds (fastest mile) of 80 mph or greater and terrain that is flat and unobstructed facing large bodies of water

DESIGN ROOF LIVE LOAD	=	16.000	psf
COLLATERAL LOAD	=	5.000	psf
SNOW EXPOSURE FACTOR	=	0.700	
SNOW IMPORTANCE FACTOR	=	1.000	
SLOPE ROOF SNOW LOAD	=	0.000	psf
DESIGN WIND VELOCITY	=	70.000	mph
WIND IMPORTANCE FACTOR	=	1.000	

*** DESIGN LOAD COMBINATIONS

CASE	LOAD FACT	GROUP =>
1	1.00 W+	
2	1.00 W-	

*** LOADS

GROUP	TYPE	M FM TO CL D	START ft	HORIZ psf/ kips	VERT psf/ kips	MOMT kip-ft	END ft	HORIZ psf	VERT psf
---	---	- -- -- -- -	---	---	---	---	---	---	---

E-5

W+	UNIF	C	1	1	0	0.00	13.49	0.00	0.00	0.00	13.49	0.00
W-	UNIF	C	1	1	0	0.00	-16.51	0.00	0.00	0.00	-16.51	0.00
W+	UNIF	C	2	3	0	0.00	13.49	0.00	0.00	0.00	13.49	0.00
W-	UNIF	C	2	3	0	0.00	-12.38	0.00	0.00	0.00	-12.38	0.00
W+	UNIF	C	4	4	0	0.00	13.49	0.00	0.00	0.00	13.49	0.00
W-	UNIF	C	4	4	0	0.00	-16.51	0.00	0.00	0.00	-16.51	0.00

FY COLD FORMED = 55.0 ksi
FY HOT ROLLED = 50.0 ksi
FY BUILT UP = 55.0 ksi

*** ENDWALL COLUMNS

MEM	DESCRIPTION	LOCATION ft	BASE PLATES in	in	in	A BLT ROW in	BEND RT	SHR RT	CT
2	BUILT-UP	21.000	6	0.375	10.00	2	0.75	0.576	0.095
	5.0 X 0.250 in FLG,			0.135 in WEB		8.000 in O/O WEB			
3	BUILT-UP	29.000	6	0.375	10.00	2	0.75	0.576	0.095
	5.0 X 0.250 in FLG,			0.135 in WEB		8.000 in O/O WEB			

INT COL 2
7.333
INT COL 3
7.333

*** MAXIMUM ENDWALL REACTIONS AND DESIGN LOAD COMBINATIONS

CASE	M VERT kips	M HORZ kips	LOAD FACTOR / LOAD GROUP =>
1	0.0	-2.0	1.0 W+
2	0.0	1.8	1.0 W-
3	0.0	-2.0	1.0 W+
4	0.0	1.8	1.0 W-

F-1

CBC Steel Buildings

STEEL FRAME ANALYSIS AND DESIGN BY THE DIRECT STIFFNESS METHOD
ALL DESIGN IN ACCORDANCE WITH 89 AISC AND MBMA AS APPLICABLE

BUILDING DESCRIPTION - - RF@FL-1 J27335

FRAME WIDTH	BAY SPACING	ROOF SLOPES	INT. COLUMNS	MEMBERS	NODES
50.000 ft.	17.000 ft.	2	0	6	7

LEFT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
----------	0.000/ 12.0	8.50 in.	145.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
BASE	1	0.000 ft.	11.250 in.	PINNED
EAVE	2	12.083 ft.	17.250 in.	RIGID

ROOF SLOPE 1	SLOPE W/HORIZ.	PURLIN DEPTH	TYP. PURLIN SPACE
----------	5.000/ 12.0	8.50 in.	33.69 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
LEFT END	2	0.000 ft.	17.500 in.	RIGID
SPLICE	3	10.708 ft.	17.500 in.	RIGID
RIGHT END	4	25.000 ft.	17.500 in.	RIGID

ROOF SLOPE 2	SLOPE W/HORIZ.	PURLIN DEPTH	TYP. PURLIN SPACE
----------	-5.000/ 12.0	8.50 in.	33.69 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
LEFT END	4	25.000 ft.	17.500 in.	RIGID
SPLICE	5	39.292 ft.	17.500 in.	RIGID
RIGHT END	6	50.000 ft.	17.500 in.	RIGID

RIGHT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
----------	0.000/ 12.0	8.50 in.	145.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
EAVE	6	12.083 ft.	17.250 in.	RIGID
BASE	7	0.000 ft.	11.250 in.	PINNED

LOAD CASE 1 : D+C
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	15.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	-0.18	0.76
2	4	12.08	0.00	-0.18	-0.76

LOAD CASE 2 : D
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	3.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	-0.18	0.76
2	4	12.08	0.00	-0.18	-0.76

LOAD CASE 3 : L
No Stress Check; L/60 H Deflection Limit; L/180 V Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
12.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	-1.13	4.86
2	4	12.08	0.00	-1.13	-4.86

LOAD CASE 4 : WL^
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS * WIND COEFFICIENTS

LIVE (psf)	WIND (psf)	DEAD (psf)	C1	C2	C3	C4
0.00	13.76	0.00	-0.70	-0.70	-0.70	-0.70

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	0.68	-2.93
2	4	12.08	0.00	0.68	2.93

LOAD CASE 5 : W1->
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	13.76	0.00	0.80	-0.90	-0.70	-0.50

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	1.66	-7.11
2	4	12.08	0.00	0.68	2.93

LOAD CASE 6 : W1<-
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	13.76	0.00	-0.50	-0.70	-0.90	0.80

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	0.68	-2.93
2	4	12.08	0.00	1.66	7.11

LOAD CASE 7 : W2->
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	13.76	0.00	0.80	0.30	-0.70	-0.50

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	0.49	-2.09
2	4	12.08	0.00	0.68	2.93

LOAD CASE 8 : W2<-
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	13.76	0.00	-0.50	-0.70	0.30	0.80

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	0.68	-2.93
2	4	12.08	0.00	0.49	2.09

LOAD CASE 9 : E->
No Stress Check; L/120 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	0.00	0.00	0.00	0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	2.70	0.00	0.00
2	4	12.08	2.70	0.00	0.00

LOAD CASE 10 : E<-
No Stress Check; L/120 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	0.00	0.00	0.00	0.00	0.00	0.00

CONCENTRATED LOADS LOAD COMPONENTS

LOAD NO.	LINE NO.	LOCATION (ft.)	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	-2.70	0.00	0.00
2	4	12.08	-2.70	0.00	0.00

LOAD CASE 11 : ΩE->
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	5.80	0.00	0.00
2	4	12.08	5.80	0.00	0.00

LOAD CASE 12 : ΩE<-
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	-5.80	0.00	0.00
2	4	12.08	-5.80	0.00	0.00

LOAD CASE 13 : D+C + L
Check By ASD; No Deflection Limits

Highest check ratio achieved in this load case = 0.851

LOAD CASE 14 : D + WL^
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.089

LOAD CASE 15 : D + W1->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.237

LOAD CASE 16 : D + W1<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.237

LOAD CASE 17 : D + W2->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.339

LOAD CASE 18 : D + W2<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.339

LOAD CASE 19 : D+C + 0.71E->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.570

LOAD CASE 20 : D+C + 0.71E<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.570

LOAD CASE 21 : 0.71D+C + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

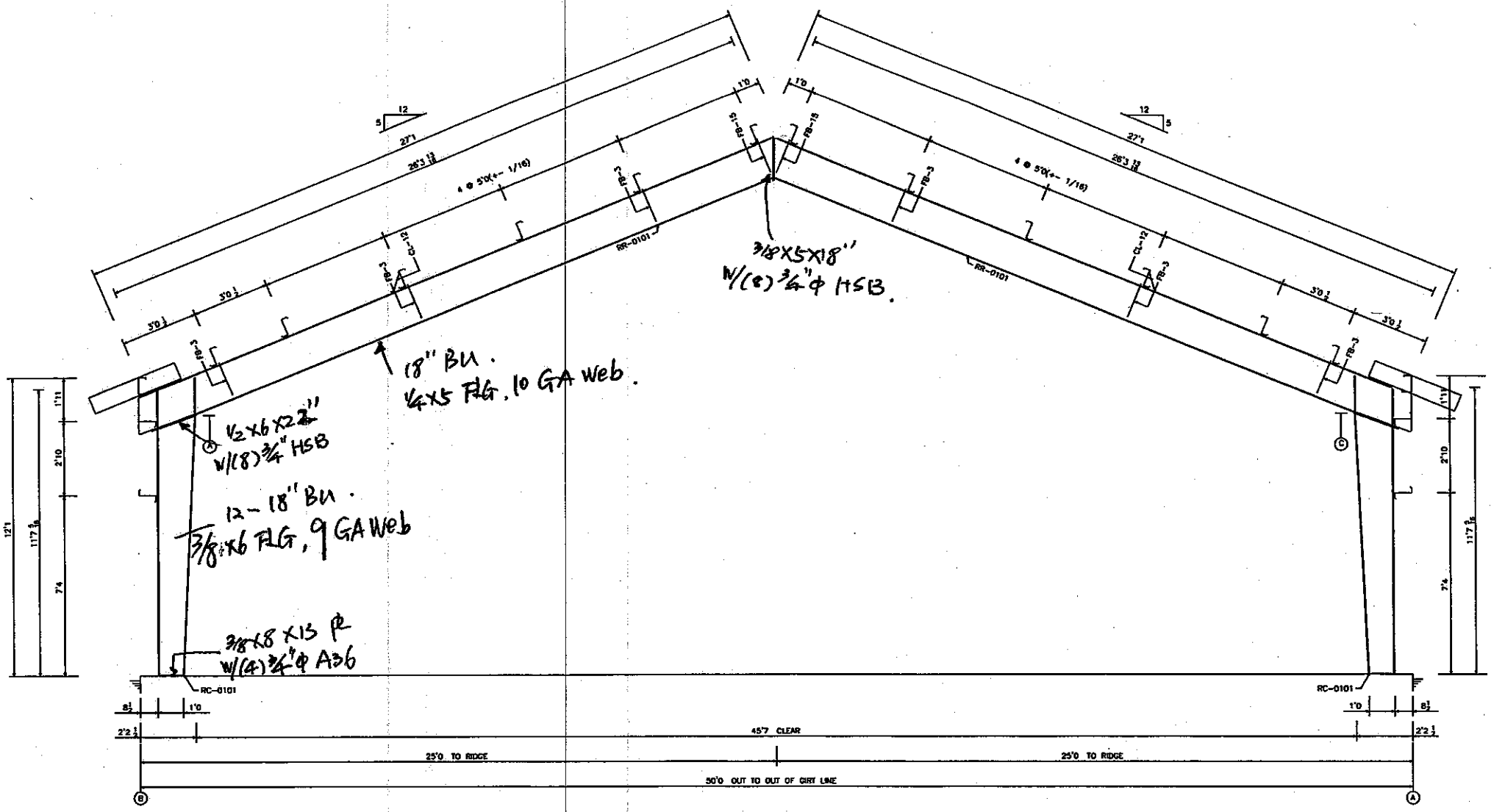
LOAD CASE 22 : 0.71D+C + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

LOAD CASE 23 : 0.53D+C + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

LOAD CASE 24 : 0.53D+C + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

MAXIMUM CHECK RATIOS PER 1989 AISC

MEM	OUTER WIDTH (in)	FLANGE * THICK * (in)	WEB * THICK * (in)	INNER WIDTH (in)	FLANGE * THICK * (in)	OUTER FLG RATIO	LOAD	WEB SHEAR RATIO	LOAD	INNER FLG RATIO	LOAD
1	6.00	0.3750	0.1500	6.00	0.3750	0.372	13	0.224	13	0.716	13
2	5.00	0.2500	0.1350	5.00	0.2500	0.688	13	0.684	13	0.851	13
3	5.00	0.2500	0.1350	5.00	0.2500	0.379	13	0.286	13	0.321	18
4	5.00	0.2500	0.1350	5.00	0.2500	0.379	13	0.286	13	0.321	18
5	5.00	0.2500	0.1350	5.00	0.2500	0.688	13	0.684	13	0.851	13
6	6.00	0.3750	0.1500	6.00	0.3750	0.372	13	0.224	13	0.716	13



RIGID FRAME CROSS SECTION AT LINE 1

FRAME ID: J27335.01A

T-8.

RIGID FRAME CONNECTION DESIGN

By

Sheet F-_____

Date 09-08-2007 Cust

Job #

KNEE CONNECTION

USE : Plate 6 x .5 x 21 in. (Fy = 55 ksi)
w/ 3/4 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 15.732 k/bolt
Plate Allow bolt tension w/o stiffener = 7.866 k/bolt
Bolt Allowable tension = 19.400 k/bolt

Actual Dead + Live moment = 65.00 k-ft (Outside bolts in tension)
Allow Dead + Live moment = 89.15 k-ft w/ 2 rows of bolts w/ 0 stiff
1 row of bolts outside flange

Actual Dead + Live moment = 19.00 k-ft (Inside bolts in tension)
Allow Dead + Live moment = 55.06 k-ft w/ 2 rows of bolts w/ 0 stiff

RAFTER RIDGE CONNECTION

USE : Plate 5 x .375 x 18 in. (Fy = 55 ksi)
w/ 3/4 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 8.856 k/bolt
Plate Allow bolt tension w/o stiffener = 4.428 k/bolt
Bolt Allowable tension = 19.400 k/bolt

Actual Dead + Wind moment = 6.00 k-ft (Upper bolts in tension)
Allow Dead + Wind moment = 42.80 k-ft w/ 2 rows of bolts w/ 0 stiff

Actual Dead + Live moment = 22.00 k-ft (Lower bolts in tension)
Allow Dead + Live moment = 32.10 k-ft w/ 2 rows of bolts w/ 0 stiff
0 row of bolts below flange

COLUMN BASE PLATE

Vertical load = 13.50 kips
Column flg width = 6.00 in.

*** USE: BASE PLATE 8 x .375 x 13 in. (55 ksi)

Min Req'd Thk : 1.6 x SQR [13.5 / (8 x 13 x .25 x 55)] = .155 in

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SUPPORT REACTIONS

CASE 1 : D+C

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	3.24	7.30	0.00
2	7	-3.24	7.30	0.00

CASE 2 : D

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	0.79	2.20	0.00
2	7	-0.79	2.20	-0.00

CASE 3 : L

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	2.18	6.23	0.00
2	7	-2.18	6.23	-0.00

CASE 4 : WL^

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-1.32	-4.78	-0.00
2	7	1.32	-4.78	0.00

CASE 5 : W1->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-2.57	-6.99	0.00
2	7	1.14	-4.70	0.00

CASE 6 : W1<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-1.14	-4.70	-0.00
2	7	2.57	-6.99	0.00

CASE 7 : W2->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-2.62	-1.44	0.00
2	7	-1.73	-2.07	0.00

CASE 8 : W2<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	1.73	-2.07	0.00
2	7	2.62	-1.44	-0.00

CASE 9 : E->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-2.70	-1.26	0.00
2	7	-2.70	1.26	0.00

CASE 10 : E<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	2.70	1.26	-0.00
2	7	2.70	-1.26	-0.00

CASE 11 : ΩE->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-5.80	-2.71	-0.00
2	7	-5.80	2.71	0.00

CASE 12 : ΩE<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	5.80	2.71	0.00
2	7	5.80	-2.71	-0.00

F-11

FLANGE BRACE LOCATIONS

SEQ#	P/G#	REQD?
LEFT COLUMN BASE		
1	1	No (Knee)
LEFT EAVE		
2	6	Yes
3	5	No
4	4	Yes
5	3	No
6	2	Yes
7	1	Yes (Ridge)
ROOF SLOPE CHANGE		
8	1	Yes (Ridge)
9	2	Yes
10	3	No
11	4	Yes
12	5	No
13	6	Yes
RIGHT EAVE		
14	1	No (Knee)
RIGHT COLUMN BASE		

F-12

STEEL FRAME ANALYSIS AND DESIGN BY THE DIRECT STIFFNESS METHOD
ALL DESIGN IN ACCORDANCE WITH 89 AISC AND MBMA AS APPLICABLE

BUILDING DESCRIPTION - - RF@FL-2,5 J27335

FRAME WIDTH	BAY SPACING	ROOF SLOPES	INT. COLUMNS	MEMBERS	NODES
50.000 ft.	14.000 ft.	2	0	6	7

LEFT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
----------	0.000/ 12.0	8.50 in.	255.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
BASE	1	0.000 ft.	11.500 in.	PINNED
EAVE	2	21.250 ft.	23.500 in.	RIGID

ROOF SLOPE 1	SLOPE W/HORIZ.	PURLIN DEPTH	TYP. PURLIN SPACE
----------	5.000/ 12.0	8.50 in.	33.69 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
LEFT END	2	0.000 ft.	23.500 in.	RIGID
SPLICE	3	10.708 ft.	23.500 in.	RIGID
RIGHT END	4	25.000 ft.	23.500 in.	RIGID

ROOF SLOPE 2	SLOPE W/HORIZ.	PURLIN DEPTH	TYP. PURLIN SPACE
----------	-5.000/ 12.0	8.50 in.	33.69 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
LEFT END	4	25.000 ft.	23.500 in.	RIGID
SPLICE	5	39.292 ft.	23.500 in.	RIGID
RIGHT END	6	50.000 ft.	23.500 in.	RIGID

RIGHT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
----------	0.000/ 12.0	8.50 in.	255.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
EAVE	6	21.250 ft.	23.500 in.	RIGID
BASE	7	0.000 ft.	11.500 in.	PINNED

LOAD CASE 1 : D+C
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	8.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	-0.25	1.07
2	4	21.25	0.00	-0.25	-1.07
3	1	12.00	0.00	-1.50	0.00
4	4	12.00	0.00	-1.50	0.00

LOAD CASE 2 : D
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	3.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	-0.25	1.07
2	4	21.25	0.00	-0.25	-1.07
3	1	12.00	0.00	-0.60	0.00
4	4	12.00	0.00	-0.60	0.00

LOAD CASE 3 : L
No Stress Check; L/60 H Deflection Limit; L/180 V Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
12.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	-1.60	6.87
2	4	21.25	0.00	-1.60	-6.87
3	1	12.00	0.00	-2.10	0.00
4	4	12.00	0.00	-2.10	0.00

LOAD CASE 4 : WL^
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	15.14	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
-0.70	-0.70	-0.70	-0.70

DISTRIBUTED LOADS

LOAD NO.	LINE NO.	START LOCATION (ft.)	END LOCATION (ft.)	HORIZONTAL LOAD INTENSITY AT START (psf)	HORIZONTAL LOAD INTENSITY AT END (psf)	VERTICAL LOAD INTENSITY AT START (psf)	VERTICAL LOAD INTENSITY AT END (psf)
1	1	0.00	12.10	-10.60	-10.60	0.00	0.00
2	4	0.00	12.10	10.60	10.60	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	1.06	-4.55
2	4	21.25	0.00	1.06	4.55
3	1	12.00	0.00	2.00	0.00
4	4	12.00	0.00	2.00	0.00

LOAD CASE 5 : W1->
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	15.14	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.80	-0.90	-0.70	-0.50

DISTRIBUTED LOADS

LOAD NO.	LINE NO.	START LOCATION (ft.)	END LOCATION (ft.)	HORIZONTAL LOAD INTENSITY AT START (psf)	HORIZONTAL LOAD INTENSITY AT END (psf)	VERTICAL LOAD INTENSITY AT START (psf)	VERTICAL LOAD INTENSITY AT END (psf)
1	1	0.00	12.10	12.10	12.10	0.00	0.00
2	4	0.00	12.10	7.60	7.60	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	2.57	-11.05
2	4	21.25	0.00	1.06	4.55
3	1	12.00	0.00	2.00	0.00
4	4	12.00	0.00	2.00	0.00

LOAD CASE 6 : W1<-
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	15.14	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
-0.50	-0.70	-0.90	0.80

DISTRIBUTED LOADS

LOAD NO.	LINE NO.	START LOCATION (ft.)	END LOCATION (ft.)	HORIZONTAL LOAD INTENSITY AT START (psf)	HORIZONTAL LOAD INTENSITY AT END (psf)	VERTICAL LOAD INTENSITY AT START (psf)	VERTICAL LOAD INTENSITY AT END (psf)
1	1	0.00	12.10	-7.60	-7.60	0.00	0.00
2	4	0.00	12.10	-12.10	-12.10	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	1.06	-4.55
2	4	21.25	0.00	2.57	11.05
3	1	12.00	0.00	2.00	0.00
4	4	12.00	0.00	2.00	0.00

LOAD CASE 7 : W2->
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	15.14	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.80	0.30	-0.70	-0.50

DISTRIBUTED LOADS

LOAD NO.	LINE NO.	START LOCATION (ft.)	END LOCATION (ft.)	HORIZONTAL LOAD INTENSITY AT START (psf)	HORIZONTAL LOAD INTENSITY AT END (psf)	VERTICAL LOAD INTENSITY AT START (psf)	VERTICAL LOAD INTENSITY AT END (psf)
1	1	0.00	12.10	12.10	12.10	0.00	0.00
2	4	0.00	12.10	7.60	7.60	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	0.76	-3.25
2	4	21.25	0.00	1.06	4.55
3	1	12.00	0.00	2.00	0.00
4	4	12.00	0.00	2.00	0.00

LOAD CASE 8 : W2<-
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

WIND COEFFICIENTS

LIVE	WIND	DEAD	C1	C2	C3	C4

(psf).	(psf)	(psf)	*				
0.00	15.14	0.00	*	-0.50	-0.70	0.30	0.80

DISTRIBUTED LOADS LOAD NO.	LINE NO.	START LOCATION (ft.)	END LOCATION (ft.)	HORIZONTAL LOAD INTENSITY AT START (psf)	HORIZONTAL LOAD INTENSITY AT END (psf)	VERTICAL LOAD INTENSITY AT START (psf)	VERTICAL LOAD INTENSITY AT END (psf)
1	1	0.00	12.10	-7.60	-7.60	0.00	0.00
2	4	0.00	12.10	-12.10	-12.10	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	1.06	-4.55
2	4	21.25	0.00	0.76	3.25
3	1	12.00	0.00	2.00	0.00
4	4	12.00	0.00	2.00	0.00

LOAD CASE 9 : E->
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	0.00	0.00	0.00	0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	1.10	0.00	0.00
2	4	21.25	1.10	0.00	0.00
3	1	12.00	3.50	0.00	0.00
4	4	12.00	1.90	0.00	0.00

LOAD CASE 10 : E<-
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	0.00	0.00	0.00	0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	-1.10	0.00	0.00
2	4	21.25	-1.10	0.00	0.00
3	1	12.00	-3.50	0.00	0.00

4	4	12.00	-1.90	0.00	0.00

LOAD CASE 11 : ΩE->
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	3.00	0.00	0.00
2	4	21.25	3.00	0.00	0.00
3	1	12.00	9.50	0.00	0.00
4	4	12.00	5.00	0.00	0.00

LOAD CASE 12 : ΩE<-
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	-3.00	0.00	0.00
2	4	21.25	-3.00	0.00	0.00
3	1	12.00	-9.50	0.00	0.00
4	4	12.00	-5.00	0.00	0.00

LOAD CASE 13 : D+C + L
Check By ASD; No Deflection Limits

Highest check ratio achieved in this load case = 0.673

LOAD CASE 14 : D + WL^
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.117

LOAD CASE 15 : D + W1->
Check By ASD With 1/3 Increase; No Deflection Limits

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Highest check ratio achieved in this load case = 0.340

LOAD CASE 16 : D + W1<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.340

LOAD CASE 17 : D + W2->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.773

LOAD CASE 18 : D + W2<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.773

LOAD CASE 19 : D+C + 0.71E->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.714

LOAD CASE 20 : D+C + 0.71E<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.769

LOAD CASE 21 : 0.71D+C + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

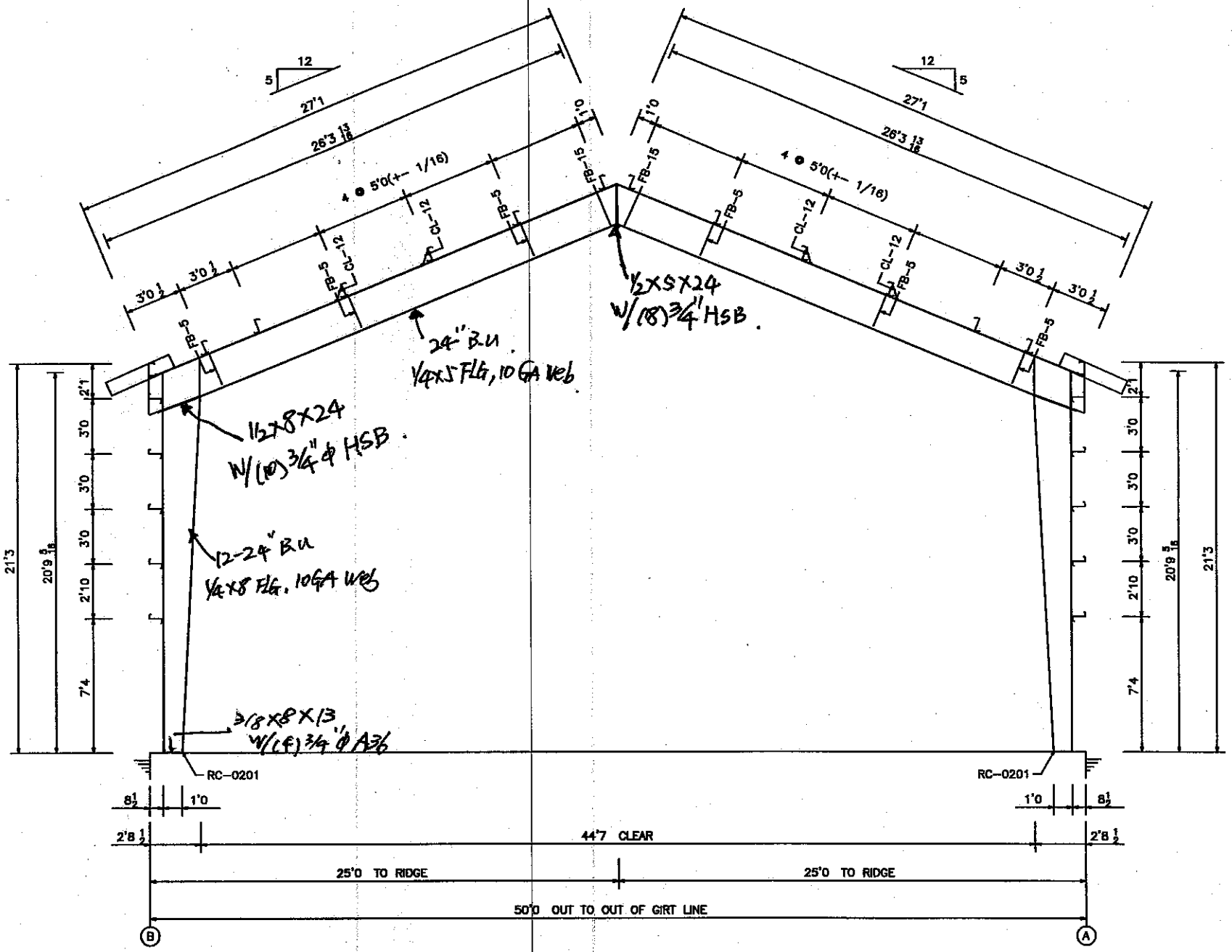
LOAD CASE 22 : 0.71D+C + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

LOAD CASE 23 : 0.53D+C + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

LOAD CASE 24 : 0.53D+C + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

MAXIMUM CHECK RATIOS PER 1989 AISC

MEM	OUTER FLANGE WIDTH (in)	* THICK *	WEB THICK (in)	* INNER FLANGE WIDTH (in)	* THICK *	OUTER FLG RATIO	LOAD	WEB SHEAR RATIO	LOAD	INNER FLG RATIO	LOAD
1	8.00	0.2500	0.1345	8.00	0.2500	0.439	19	0.178	20	0.773	18
2	5.00	0.2500	0.1345	5.00	0.2500	0.408	18	0.656	13	0.500	17
3	5.00	0.2500	0.1345	5.00	0.2500	0.317	17	0.314	13	0.406	17
4	5.00	0.2500	0.1345	5.00	0.2500	0.317	17	0.314	13	0.406	17
5	5.00	0.2500	0.1345	5.00	0.2500	0.408	18	0.656	13	0.500	17
6	8.00	0.2500	0.1345	8.00	0.2500	0.439	19	0.178	20	0.773	18



RIGID FRAME CROSS SECTION AT LINE 2, 5

FRAME ID: J27335.02B

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RIGID FRAME CONNECTION DESIGN

By

Sheet F-

Date 01-23-2008 Cust

Job #

KNEE CONNECTION

USE : Plate 8 x .5 x 27 in. (Fy = 55 ksi)
w/ 3/4 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 15.732 k/bolt
Plate Allow bolt tension w/o stiffener = 7.866 k/bolt
Bolt Allowable tension = 19.400 k/bolt

Actual Dead + Live moment = 100.00 k-ft (Outside bolts in tension)
Allow Dead + Live moment = 144.87 k-ft w/ 3 rows of bolts w/ 0 stiff
1 row of bolts outside flange

Actual Dead + Live moment = 77.00 k-ft (Inside bolts in tension)
Allow Dead + Live moment = 102.26 k-ft w/ 2 rows of bolts w/ 1 stiff

RAFTER RIDGE CONNECTION

USE : Plate 5 x .5 x 24 in. (Fy = 55 ksi)
w/ 3/4 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 15.732 k/bolt
Plate Allow bolt tension w/o stiffener = 7.866 k/bolt
Bolt Allowable tension = 19.400 k/bolt

Actual Dead + Live moment = 21.00 k-ft (Upper bolts in tension)
Allow Dead + Live moment = 80.63 k-ft w/ 2 rows of bolts w/ 0 stiff

Actual Dead + Live moment = 24.00 k-ft (Lower bolts in tension)
Allow Dead + Live moment = 80.63 k-ft w/ 2 rows of bolts w/ 0 stiff
0 row of bolts below flange

COLUMN BASE PLATE

Vertical load = 13.00 kips
Column flg width = 8.00 in.

*** USE: BASE PLATE 8 x .375 x 13 in. (55 ksi)

Min Req'd Thk : .8 x SQR [13 / (8 x 13 x .25 x 55)] = .076 in

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SUPPORT REACTIONS

CASE 1 : D+C

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	0.93	5.56	-0.00
2	7	-0.93	5.56	0.00

CASE 2 : D

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	0.43	2.91	-0.00
2	7	-0.43	2.91	-0.00

CASE 3 : L

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	1.00	7.90	-0.00
2	7	-1.00	7.90	-0.00

CASE 4 : WL^

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-0.53	-6.77	0.00
2	7	0.53	-6.77	0.00

CASE 5 : W1->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-2.49	-10.43	-0.00
2	7	-0.54	-5.68	0.00

CASE 6 : W1<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	0.54	-5.68	0.00
2	7	2.49	-10.43	0.00

CASE 7 : W2->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-3.10	-5.10	0.00
2	7	-2.58	-2.84	-0.00

CASE 8 : W2<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	2.58	-2.84	0.00
2	7	3.10	-5.10	-0.00

CASE 9 : E->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-4.18	-2.29	0.00
2	7	-3.42	2.29	0.00

CASE 10 : E<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	4.18	2.29	-0.00
2	7	3.42	-2.29	-0.00

CASE 11 : ΩE->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-11.32	-6.20	-0.00
2	7	-9.18	6.20	0.00

CASE 12 : ΩE<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	11.32	6.20	0.00
2	7	9.18	-6.20	-0.00

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FLANGE BRACE LOCATIONS

SEQ#	P/G#	REQD?
LEFT COLUMN BASE		
1	1	No (Knee)
LEFT EAVE		
2	6	Yes
3	5	No
4	4	Yes
5	3	No
6	2	Yes
7	1	Yes (Ridge)
ROOF SLOPE CHANGE		
8	1	Yes (Ridge)
9	2	Yes
10	3	No
11	4	Yes
12	5	No
13	6	Yes
RIGHT EAVE		
14	1	No (Knee)
RIGHT COLUMN BASE		

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STEEL FRAME ANALYSIS AND DESIGN BY THE DIRECT STIFFNESS METHOD
ALL DESIGN IN ACCORDANCE WITH 89 AISC AND MBMA AS APPLICABLE

BUILDING DESCRIPTION - - RF@FL-3,4 J27335

FRAME WIDTH	BAY SPACING	ROOF SLOPES	INT. COLUMNS	MEMBERS	NODES
50.000 ft.	24.000 ft.	2	0	6	7

LEFT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
----------	0.000/ 12.0	8.50 in.	255.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
BASE	1	0.000 ft.	11.250 in.	PINNED
EAVE	2	21.250 ft.	23.250 in.	RIGID

ROOF SLOPE 1	SLOPE W/HORIZ.	PURLIN DEPTH	TYP. PURLIN SPACE
----------	5.000/ 12.0	8.50 in.	300.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
LEFT END	2	0.000 ft.	19.000 in.	RIGID
SPLICE	3	14.292 ft.	19.000 in.	RIGID
RIGHT END	4	25.000 ft.	19.000 in.	RIGID

ROOF SLOPE 2	SLOPE W/HORIZ.	PURLIN DEPTH	TYP. PURLIN SPACE
----------	-5.000/ 12.0	8.50 in.	300.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
LEFT END	4	25.000 ft.	19.000 in.	RIGID
SPLICE	5	35.708 ft.	19.000 in.	RIGID
RIGHT END	6	50.000 ft.	19.000 in.	RIGID

RIGHT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
----------	0.000/ 12.0	8.50 in.	255.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
EAVE	6	21.250 ft.	23.250 in.	RIGID
BASE	7	0.000 ft.	11.250 in.	PINNED

F-23

LOAD CASE 1 : D+C
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	8.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	-0.25	1.07
2	4	21.25	0.00	-0.25	-1.07
3	3	0.00	0.00	-0.50	0.00
4	3	0.00	0.00	-0.30	0.00

LOAD CASE 2 : D
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	3.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	-0.25	1.07
2	4	21.25	0.00	-0.25	-1.07

LOAD CASE 3 : L
No Stress Check; L/60 H Deflection Limit; L/180 V Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
12.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	-1.60	6.87
2	4	21.25	0.00	-1.60	-6.87

LOAD CASE 4 : WL^
No Stress Check; L/45 H Deflection Limit

```
UNIFORM LOADS                          *       WIND COEFFICIENTS
                                       *
  LIVE    WIND    DEAD                 *   C1     C2     C3     C4
 (psf)   (psf)   (psf)                 *
  0.00   15.14    0.00                  -0.70  -0.70  -0.70  -0.70


                     *                 *
CONCENTRATED LOADS   *                 *        LOAD COMPONENTS
LOAD NO.  LINE NO.   *     LOCATION    * HORIZONTAL   VERTICAL     MOMENT
                     *      (ft.)      *     (kips)     (kips)    (kip-ft)
    1         1              21.25          0.00       1.06       -4.55
    2         4              21.25          0.00       1.06        4.55
-------------------------------------------------------------------------------

LOAD CASE 5 :  W1->
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS                          *       WIND COEFFICIENTS
                                       *
  LIVE    WIND    DEAD                 *   C1     C2     C3     C4
 (psf)   (psf)   (psf)                 *
  0.00   15.14    0.00                   0.80  -0.90  -0.70  -0.50


                     *                 *
CONCENTRATED LOADS   *                 *        LOAD COMPONENTS
LOAD NO.  LINE NO.   *     LOCATION    * HORIZONTAL   VERTICAL     MOMENT
                     *      (ft.)      *     (kips)     (kips)    (kip-ft)
    1         1              21.25          0.00       2.57      -11.05
    2         4              21.25          0.00       1.06        4.55
-------------------------------------------------------------------------------

LOAD CASE 6 :  W1<-
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS                          *       WIND COEFFICIENTS
                                       *
  LIVE    WIND    DEAD                 *   C1     C2     C3     C4
 (psf)   (psf)   (psf)                 *
  0.00   15.14    0.00                  -0.50  -0.70  -0.90   0.80


                     *                 *
CONCENTRATED LOADS   *                 *        LOAD COMPONENTS
LOAD NO.  LINE NO.   *     LOCATION    * HORIZONTAL   VERTICAL     MOMENT
                     *      (ft.)      *     (kips)     (kips)    (kip-ft)
    1         1              21.25          0.00       1.06       -4.55
    2         4              21.25          0.00       2.57       11.05
-------------------------------------------------------------------------------

LOAD CASE 7 :  W2->
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS                          *       WIND COEFFICIENTS
                                       *
  LIVE    WIND    DEAD                 *   C1     C2     C3     C4
```

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(psf)	(psf)	(psf)	*			
0.00	15.14	0.00	0.80	0.30	-0.70	-0.50

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	0.76	-3.25
2	4	21.25	0.00	1.06	4.55

LOAD CASE 8 : W2<-
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	15.14	0.00	-0.50	-0.70	0.30	0.80

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	0.00	1.06	-4.55
2	4	21.25	0.00	0.76	3.25

LOAD CASE 9 : E->
No Stress Check; L/120 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	0.00	0.00	0.00	0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	2.00	0.00	0.00
2	4	21.25	2.00	0.00	0.00
3	1	12.00	1.60	0.00	0.00

LOAD CASE 10 : E<-
No Stress Check; L/120 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3	C4
0.00	0.00	0.00	0.00	0.00	0.00	0.00

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CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	-2.00	0.00	0.00
2	4	21.25	-2.00	0.00	0.00
3	1	12.00	-1.60	0.00	0.00

LOAD CASE 11 : ΩE->
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	5.50	0.00	0.00
2	4	21.25	5.50	0.00	0.00
3	1	12.00	4.50	0.00	0.00

LOAD CASE 12 : ΩE<-
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	-5.50	0.00	0.00
2	4	21.25	-5.50	0.00	0.00
3	1	12.00	-4.50	0.00	0.00

LOAD CASE 13 : D+C + L
Check By ASD; No Deflection Limits

Highest check ratio achieved in this load case = 0.204

LOAD CASE 14 : D + WL^
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.399

LOAD CASE 15 : D + W1->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.300

LOAD CASE 16 : D + W1<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.300

LOAD CASE 17 : D + W2->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.499

LOAD CASE 18 : D + W2<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.499

LOAD CASE 19 : D+C + 0.71E->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.233

LOAD CASE 20 : D+C + 0.71E<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.237

LOAD CASE 21 : 0.71D+C + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

LOAD CASE 22 : 0.71D+C + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

LOAD CASE 23 : 0.53D+C + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

LOAD CASE 24 : 0.53D+C + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

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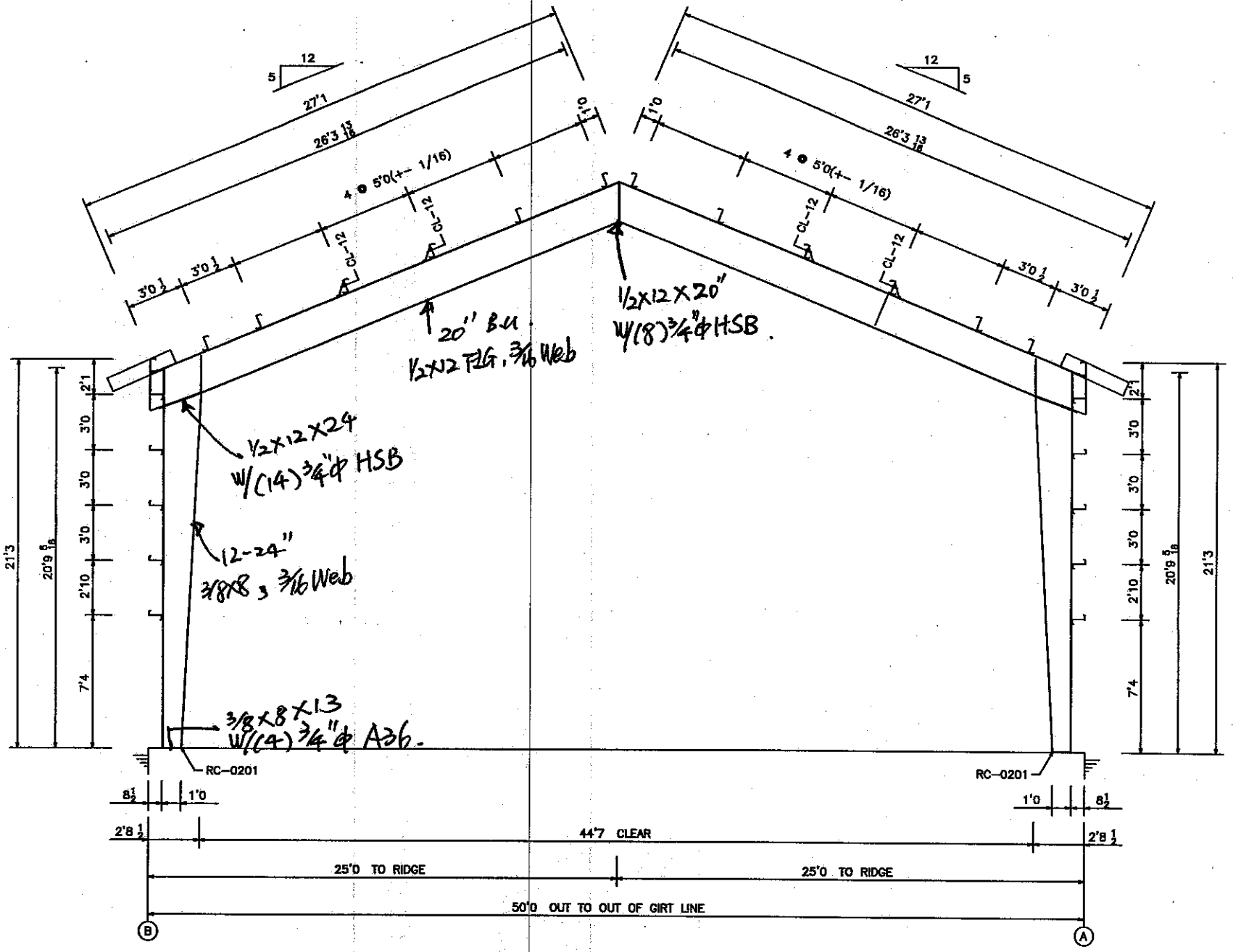
MAXIMUM CHECK RATIOS PER 1989 AISC

MEM	OUTER WIDTH (in)	FLANGE * THICK * (in)	WEB * THICK * (in)	INNER WIDTH (in)	FLANGE * THICK * (in)	OUTER FLG RATIO	LOAD	WEB SHEAR RATIO	LOAD	INNER FLG RATIO	LOAD
1	8.00	0.3750	0.1875	8.00	0.3750	0.401	17	0.093	17	0.487	18
2	12.00	0.5000	0.1875	12.00	0.5000	0.431	17	0.118	18	0.498	18
3	12.00	0.5000	0.1875	12.00	0.5000	0.412	14	0.101	17	0.190	17
4	12.00	0.5000	0.1875	12.00	0.5000	0.412	14	0.101	17	0.190	17
5	12.00	0.5000	0.1875	12.00	0.5000	0.431	17	0.118	18	0.498	18
6	8.00	0.3750	0.1875	8.00	0.3750	0.401	17	0.093	17	0.487	18



RIGID FRAME CROSS SECTION AT LINE 3, 4

FRAME ID: J27335.02A

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RIGID FRAME CONNECTION DESIGN

By

Sheet F-

Date 01-23-2008 Cust

Job #

KNEE CONNECTION

USE : Plate 12 x .5 x 24 in. (Fy = 55 ksi)
w/ 3/4 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 15.732 k/bolt
Plate Allow bolt tension w/o stiffener = 7.866 k/bolt
Bolt Allowable tension = 19.400 k/bolt

Actual Dead + Live moment = 100.00 k-ft (Outside bolts in tension)
Allow Dead + Live moment = 173.71 k-ft w/ 3 rows of bolts w/ 1 stiff
1 of the rows are 4 bolt rows
0 row of bolts outside flange

Actual Dead + Live moment = 85.00 k-ft (Inside bolts in tension)
Allow Dead + Live moment = 133.07 k-ft w/ 2 rows of bolts w/ 1 stiff
1 of the rows are 4 bolt rows

RAFTER RIDGE CONNECTION

USE : Plate 12 x .5 x 20 in. (Fy = 55 ksi)
w/ 3/4 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 15.732 k/bolt
Plate Allow bolt tension w/o stiffener = 7.866 k/bolt
Bolt Allowable tension = 19.400 k/bolt

Actual Dead + Live moment = 20.00 k-ft (Upper bolts in tension)
Allow Dead + Live moment = 64.89 k-ft w/ 2 rows of bolts w/ 0 stiff

Actual Dead + Live moment = 33.00 k-ft (Lower bolts in tension)
Allow Dead + Live moment = 64.89 k-ft w/ 2 rows of bolts w/ 0 stiff
0 row of bolts below flange

COLUMN BASE PLATE

Vertical load = 17.00 kips
Column flg width = 8.00 in.

*** USE: BASE PLATE 8 x .375 x 13 in. (55 ksi)

Min Req'd Thk : .8 x SQR [17 / (8 x 13 x .25 x 55)] = .087 in

SUPPORT REACTIONS

CASE 1 : D+C

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	0.60	7.76	0.00
2	7	-0.60	7.76	-0.00

CASE 2 : D

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	0.40	4.36	-0.00
2	7	-0.40	4.36	-0.00

CASE 3 : L

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-0.10	8.80	0.00
2	7	0.10	8.80	-0.00

CASE 4 : WL^

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	0.87	-7.42	-0.00
2	7	-0.87	-7.42	-0.00

CASE 5 : W1->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-1.74	-12.72	0.00
2	7	-2.64	-5.45	-0.00

CASE 6 : W1<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	2.64	-5.45	0.00
2	7	1.74	-12.72	-0.00

CASE 7 : W2->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-4.37	-1.91	0.00
2	7	-4.55	-3.54	0.00

CASE 8 : W2<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	4.55	-3.54	-0.00
2	7	4.37	-1.91	-0.00

CASE 9 : E->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-3.17	-2.11	-0.00
2	7	-2.43	2.11	0.00

CASE 10 : E<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	3.17	2.11	0.00
2	7	2.43	-2.11	-0.00

CASE 11 : ΩE->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-8.78	-5.83	-0.00
2	7	-6.72	5.83	-0.00

CASE 12 : ΩE<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	8.78	5.83	0.00
2	7	6.72	-5.83	0.00

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FLANGE BRACE LOCATIONS

SEQ#	P/G#	REQD?
LEFT COLUMN BASE		
1	1	Yes (Ridge)
RIGHT EAVE		
2	1	Yes (Ridge)
RIGHT COLUMN BASE		

F-33

STEEL FRAME ANALYSIS AND DESIGN BY THE DIRECT STIFFNESS METHOD
ALL DESIGN IN ACCORDANCE WITH 89 AISC AND MBMA AS APPLICABLE

BUILDING DESCRIPTION - - RF@FL-6,7,8 J27335

FRAME WIDTH	BAY SPACING	ROOF SLOPES	INT. COLUMNS	MEMBERS	NODES
50.000 ft.	28.000 ft.	2	0	6	7

LEFT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
----------	0.000/ 12.0	8.50 in.	145.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
BASE	1	0.000 ft.	11.000 in.	PINNED
EAVE	2	12.083 ft.	23.000 in.	RIGID

ROOF SLOPE 1	SLOPE W/HORIZ.	PURLIN DEPTH	TYP. PURLIN SPACE
----------	5.000/ 12.0	8.50 in.	33.69 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
LEFT END	2	0.000 ft.	23.250 in.	RIGID
SPLICE	3	10.708 ft.	23.250 in.	RIGID
RIGHT END	4	25.000 ft.	23.250 in.	RIGID

ROOF SLOPE 2	SLOPE W/HORIZ.	PURLIN DEPTH	TYP. PURLIN SPACE
----------	-5.000/ 12.0	8.50 in.	33.69 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
LEFT END	4	25.000 ft.	23.250 in.	RIGID
SPLICE	5	39.292 ft.	23.250 in.	RIGID
RIGHT END	6	50.000 ft.	23.250 in.	RIGID

RIGHT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
----------	0.000/ 12.0	8.50 in.	145.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
EAVE	6	12.083 ft.	23.000 in.	RIGID
BASE	7	0.000 ft.	11.000 in.	PINNED

LOAD CASE 1 : D+C
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	15.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

DISTRIBUTED LOADS

LOAD NO.	LINE NO.	START LOCATION (ft.)	END LOCATION (ft.)	HORIZONTAL LOAD INTENSITY AT START (psf)	HORIZONTAL LOAD INTENSITY AT END (psf)	VERTICAL LOAD INTENSITY AT START (psf)	VERTICAL LOAD INTENSITY AT END (psf)
1	3	0.00	25.00	0.00	0.00	-3.20	-3.20

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	-0.29	1.25
2	4	12.08	0.00	-0.29	-1.25
3	2	18.00	0.00	-2.70	0.00
4	3	0.00	0.00	-1.20	0.00
5	2	20.00	0.00	-0.40	0.00
6	3	15.00	0.00	-0.40	0.00

LOAD CASE 2 : D
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	3.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	-0.29	1.25
2	4	12.08	0.00	-0.29	-1.25

LOAD CASE 3 : L
No Stress Check; L/60 H Deflection Limit; L/180 V Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
12.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD COMPONENTS

LOAD NO.	LINE NO.	LOCATION (ft.)	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	-1.87	8.01
2	4	12.08	0.00	-1.87	-8.01
3	2	18.00	0.00	-3.00	0.00

LOAD CASE 4 : WL^
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	13.76	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
-0.70	-0.70	-0.70	-0.70

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	1.12	-4.82
2	4	12.08	0.00	1.12	4.82
3	2	18.00	0.00	1.20	0.00

LOAD CASE 5 : W1->
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	13.76	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.80	-0.90	-0.70	-0.50

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	2.73	-11.71
2	4	12.08	0.00	1.12	4.82
3	2	18.00	0.00	1.20	0.00

LOAD CASE 6 : W1<-
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	13.76	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
-0.50	-0.70	-0.90	0.80

CONCENTRATED LOADS

LOAD NO. LINE NO. LOCATION LOAD COMPONENTS HORIZONTAL VERTICAL MOMENT

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		* (ft.)	* (kips)	(kips)	(kip-ft)
1	1	12.08	0.00	1.12	-4.82
2	4	12.08	0.00	2.73	11.71
3	2	18.00	0.00	1.20	0.00

LOAD CASE 7 : W2->
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	13.76	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.80	0.30	-0.70	-0.50

CONCENTRATED LOADS

LOAD NO.	LINE NO.	* LOCATION (ft.)	* LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	0.80	-3.44
2	4	12.08	0.00	1.12	4.82
3	2	18.00	0.00	1.20	0.00

LOAD CASE 8 : W2<-
No Stress Check; L/45 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	13.76	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
-0.50	-0.70	0.30	0.80

CONCENTRATED LOADS

LOAD NO.	LINE NO.	* LOCATION (ft.)	* LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	0.00	1.12	-4.82
2	4	12.08	0.00	0.80	3.44
3	2	18.00	0.00	1.20	0.00

LOAD CASE 9 : E->
No Stress Check; L/120 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	* LOCATION (ft.)	* LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)

1	1	12.08	3.80	0.00	0.00
2	4	12.08	3.80	0.00	0.00
3	1	12.08	1.60	0.00	0.00

LOAD CASE 10 : E<-
No Stress Check; L/120 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	-3.80	0.00	0.00
2	4	12.08	-3.80	0.00	0.00
3	1	12.08	-1.60	0.00	0.00

LOAD CASE 11 : ΩE->
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	9.60	0.00	0.00
2	4	12.08	9.60	0.00	0.00
3	1	12.08	4.50	0.00	0.00

LOAD CASE 12 : ΩE<-
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3	C4
0.00	0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.08	-9.60	0.00	0.00

2	4	12.08	-9.60	0.00	0.00
3	1	12.08	-4.50	0.00	0.00

LOAD CASE 13 : D+C + L
Check By ASD; No Deflection Limits

Highest check ratio achieved in this load case = 0.866

LOAD CASE 14 : D + WL^
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.088

LOAD CASE 15 : D + W1->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.167

LOAD CASE 16 : D + W1<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.168

LOAD CASE 17 : D + W2->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.199

LOAD CASE 18 : D + W2<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.200

LOAD CASE 19 : D+C + 0.71E->
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.558

LOAD CASE 20 : D+C + 0.71E<-
Check By ASD With 1/3 Increase; No Deflection Limits

Highest check ratio achieved in this load case = 0.577

LOAD CASE 21 : 0.71D+C + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

LOAD CASE 22 : 0.71D+C + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

LOAD CASE 23 : 0.53D+C + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

LOAD CASE 24 : 0.53D+C + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

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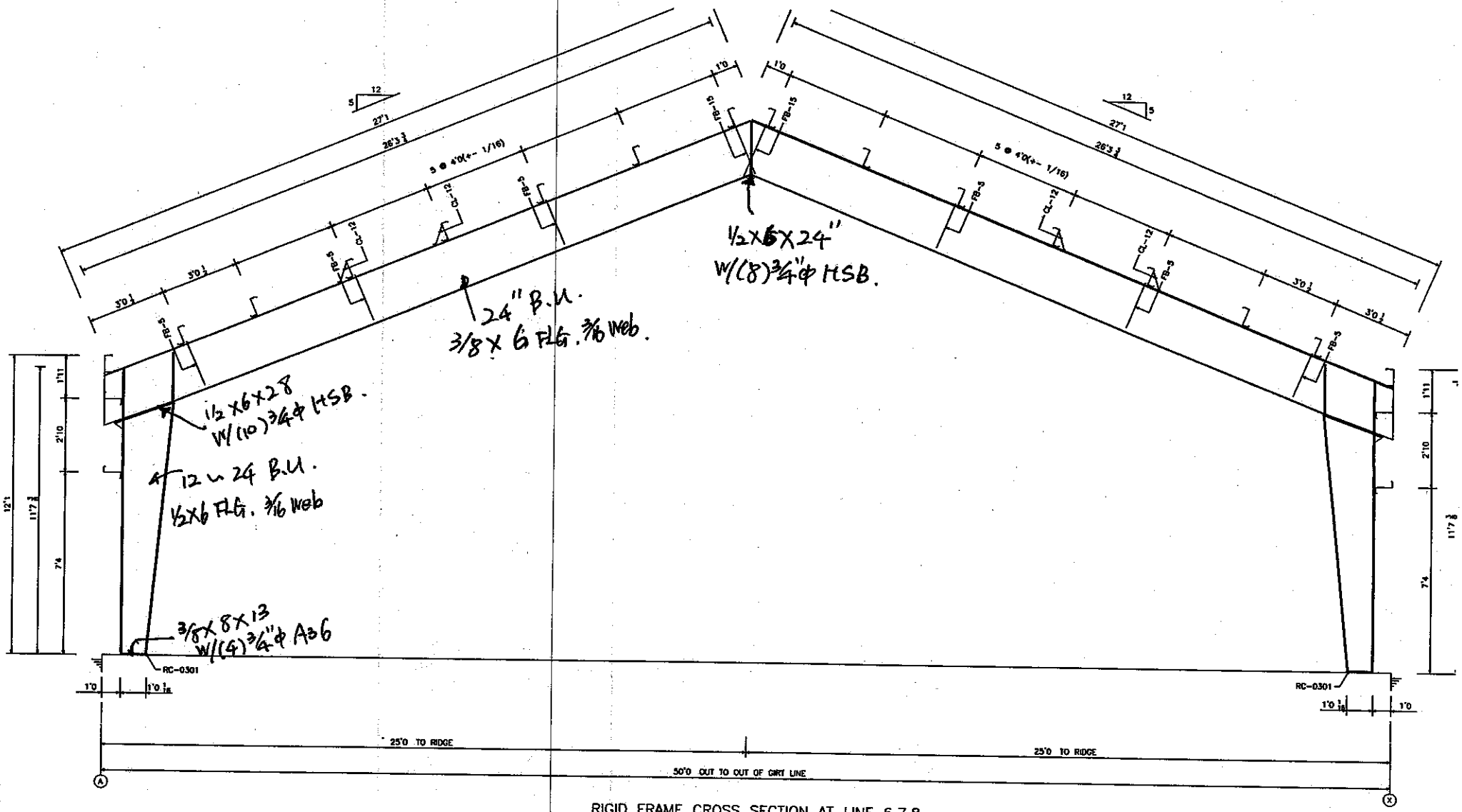
MAXIMUM CHECK RATIOS PER 1989 AISC

MEM	OUTER WIDTH (in)	FLANGE * THICK * (in)	WEB * THICK * (in)	INNER WIDTH (in)	FLANGE * THICK * (in)	OUTER RATIO	FLG LOAD	WEB SHEAR RATIO	LOAD	INNER RATIO	FLG LOAD
1	6.00	0.5000	0.1875	6.00	0.5000	0.408	13	0.278	13	0.866	13
2	6.00	0.3750	0.1875	6.00	0.3750	0.566	13	0.702	13	0.704	13
3	6.00	0.3750	0.1875	6.00	0.3750	0.415	13	0.417	13	0.318	13
4	6.00	0.3750	0.1875	6.00	0.3750	0.415	13	0.417	13	0.318	13
5	6.00	0.3750	0.1875	6.00	0.3750	0.566	13	0.702	13	0.704	13
6	6.00	0.5000	0.1875	6.00	0.5000	0.408	13	0.278	13	0.866	13



RIGID FRAME CROSS SECTION AT LINE 6,7,8

FRAME ID: J27335.03A

RIGID FRAME CONNECTION DESIGN

By

Sheet F-

Date 09-08-2007 Cust

Job #

KNEE CONNECTION

USE : Plate 6 x .5 x 27 in. (Fy = 55 ksi)
w/ 3/4 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 15.732 k/bolt
Plate Allow bolt tension w/o stiffener = 7.866 k/bolt
Bolt Allowable tension = 19.400 k/bolt

Actual Dead + Live moment = 124.60 k-ft (Outside bolts in tension)
Allow Dead + Live moment = 169.12 k-ft w/ 3 rows of bolts w/ 1 stiff
1 row of bolts outside flange

Actual Dead + Live moment = 44.20 k-ft (Inside bolts in tension)
Allow Dead + Live moment = 78.66 k-ft w/ 2 rows of bolts w/ 0 stiff

RAFTER RIDGE CONNECTION

USE : Plate 6 x .5 x 24 in. (Fy = 55 ksi)
w/ 3/4 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 15.732 k/bolt
Plate Allow bolt tension w/o stiffener = 7.866 k/bolt
Bolt Allowable tension = 19.400 k/bolt

Actual Dead + Wind moment = 10.00 k-ft (Upper bolts in tension)
Allow Dead + Wind moment = 107.50 k-ft w/ 2 rows of bolts w/ 0 stiff

Actual Dead + Live moment = 36.00 k-ft (Lower bolts in tension)
Allow Dead + Live moment = 80.63 k-ft w/ 2 rows of bolts w/ 0 stiff
0 row of bolts below flange

COLUMN BASE PLATE

Vertical load = 25.80 kips

Column flg width = 6.0

*** USE: BASE PLATE 8 x .375 x 13 in. (55 ksi)

Min Req'd Thk : 1.6 x SQR [25.8 / (8 x 13 x .25 x 55)] = .215 in

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SUPPORT REACTIONS

CASE 1 : D+C

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	7.59	15.25	-0.00
2	7	-7.59	15.77	-0.00

CASE 2 : D

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	1.37	3.64	-0.00
2	7	-1.37	3.64	0.00

CASE 3 : L

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	4.69	12.21	-0.00
2	7	-4.69	11.33	0.00

CASE 4 : WL^

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-2.67	-8.64	-0.00
2	7	2.67	-8.29	-0.00

CASE 5 : W1->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-4.73	-12.28	-0.00
2	7	2.38	-8.19	0.00

CASE 6 : W1<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-2.38	-8.54	0.00
2	7	4.73	-11.92	0.00

CASE 7 : W2->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-4.76	-3.14	-0.00
2	7	-2.40	-3.84	-0.00

CASE 8 : W2<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	2.40	-4.19	0.00
2	7	4.76	-2.79	-0.00

CASE 9 : E->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-4.85	-2.11	0.00
2	7	-4.35	2.11	0.00

CASE 10 : E<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	4.85	2.11	-0.00
2	7	4.35	-2.11	-0.00

CASE 11 : ΩE->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-12.54	-5.45	0.00
2	7	-11.16	5.45	0.00

CASE 12 : ΩE<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.54	5.45	-0.00
2	7	11.16	-5.45	-0.00

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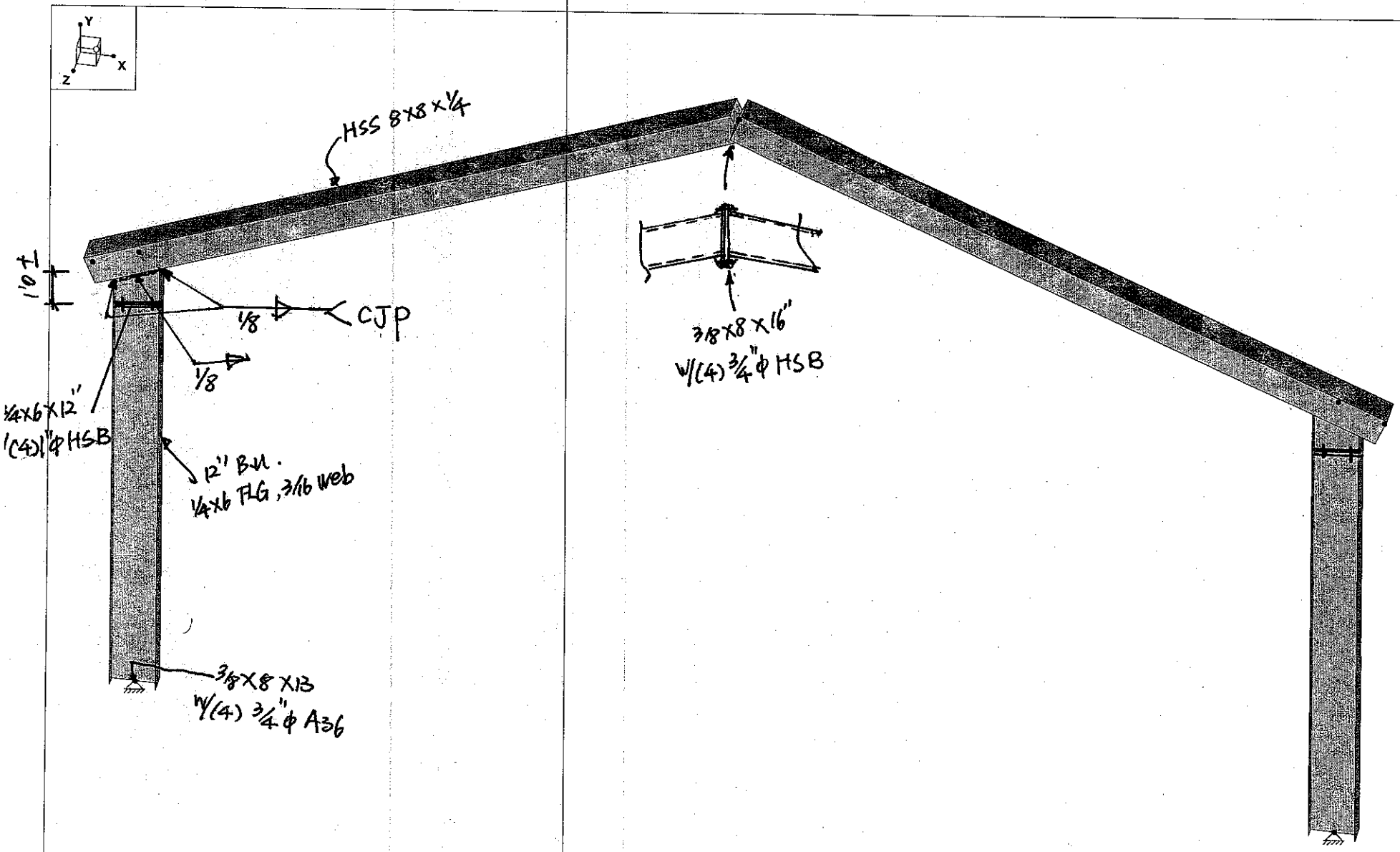
FLANGE BRACE LOCATIONS

SEQ#	P/G#	REQD?
LEFT COLUMN BASE		
1	1	No (Knee)
LEFT EAVE		
2	7	Yes
3	6	No
4	5	Yes
5	4	No
6	3	Yes
7	2	No
8	1	Yes (Ridge)
ROOF SLOPE CHANGE		
9	1	Yes (Ridge)
10	2	No
11	3	Yes
12	4	No
13	5	Yes
14	6	No
15	7	Yes
RIGHT EAVE		
16	1	No (Knee)
RIGHT COLUMN BASE		



Solution: Envelope

CBC Steel Buildings		Entry Frame	
ww			Oct 16, 2007 at 8:16 AM
J27335		ENTRY RIGID FRAME	ENTRY.R2D

RIGID FRAME CONNECTION DESIGN

By

Sheet F-

Date 10-16-2007 Cust

Job #

RAFTER SPLICE CONNECTION Column Splice

USE : Plate 6 x .75 x 12 in. (Fy = 55 ksi)
w/ 1 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 37.142 k/bolt
Plate Allow bolt tension w/o stiffener = 18.571 k/bolt
Bolt Allowable tension = 34.600 k/bolt

Actual Dead + Live moment = 33.00 k-ft (Upper bolts in tension)
Allow Dead + Live moment = 54.78 k-ft w/ 1 rows of bolts w/ 0 stiff

Actual Dead + Live moment = 33.00 k-ft (Lower bolts in tension)
Allow Dead + Live moment = 54.78 k-ft w/ 1 rows of bolts w/ 0 stiff
0 row of bolts below flange

RAFTER RIDGE CONNECTION Connectoin at ridge

USE : Plate 8 x .5 x 14 in. (Fy = 55 ksi)
w/ 3/4 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 15.732 k/bolt
Plate Allow bolt tension w/o stiffener = 7.866 k/bolt
Bolt Allowable tension = 19.400 k/bolt

Actual Dead + Live moment = 2.70 k-ft (Upper bolts in tension)
Allow Dead + Live moment = 20.98 k-ft w/ 1 rows of bolts w/ 0 stiff

Actual Dead + Live moment = 16.60 k-ft (Lower bolts in tension)
Allow Dead + Live moment = 20.98 k-ft w/ 1 rows of bolts w/ 0 stiff
1 row of bolts below flange

COLUMN BASE PLATE

Vertical load = 11.20 kips
Column flg width = 6.00 in.

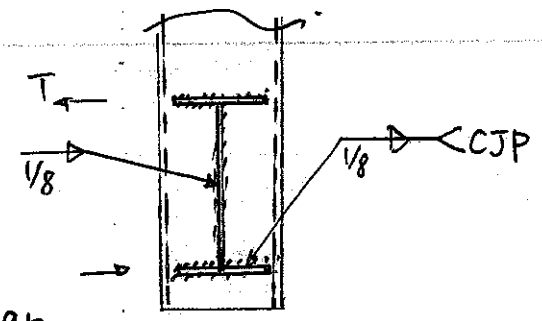
~~*** USE: BASE PLATE 8 x .375 x 13 in. (55 ksi)~~

Min Req'd Thk : 1.6 x SQR [11.2 / (8 x 13 x .25 x 55)] = .142 in

check Beam-Column Connection :

$M_{max} = 33.4$ K-ft

$T = \frac{33.4 \times 12}{12'' - 0.5''} = 34.9$ kips



$T_a = (0.25 + 2 \times 0.125 \times 0.707) \times 6'' \times 21$ ksi $= 53.8$ kips > 34.9 K

OK

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Temp [F]
1	N1	143.470278	111.654009	0
2	N2	144.470278	100.699148	0
3	N3	144.470278	112.070676	0
4	N4	157.470278	117.487342	0
5	N5	170.470278	100.699148	0
6	N6	170.470278	112.070676	0
7	N7	171.470278	111.654009	0

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Rotation[k-ft/rad]
1	N2	Reaction	Reaction	
2	N5	Reaction	Reaction	
3	N1			
4	N7			

Hot Rolled Steel Section Sets

	Label	Shape	Design List	Type	Material	Design Rules	A [in2]	I (90,270) [i...	I (0,180) [in4]
1	TR_01	HSS8X8X4	Tube	Beam	A500_46	Typical	7.085	70.527	70.527
2	WF_COL	COL	Wide Flange	Column	A572Grade50	Typical	5.162	9.006	127.39

Hot Rolled Steel Design Parameters

	Label	Shape	Length[ft]	Lb-out[ft]	Lb-in[ft]	Lcomp top[ft]	Lcomp bot[ft]	K-out	K-in	Cm	Cb	Out s...	In sway
1	M1	WF_COL	11.372										
2	M2	TR_01	15.167										
3	M3	TR_01	15.167										
4	M4	WF_COL	11.372										

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm (\1E5 F)	Density[k/ft^3]	Yield[ksi]
1	A36	29000	11154	.3	.65	.49	36
2	A572Grade50	29000	11154	.3	.65	.49	50
3	A992	29000	11154	.3	.65	.49	50
4	A500_42	29000	11154	.3	.65	.49	42
5	A500_46	29000	11154	.3	.65	.49	46

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Joint	Point	Distributed
1	DL	DL		-1			2
2	DL + CL	DL		-1			2
3	L	RLL					2
4	W1->	WL					2
5	W1<-	WL					2
6	W2->	WL					2
7	W2<-	WL					2
8	E->	EL			2		
9	Em->	EL			2		

Member Distributed Loads (BLC 1 : DL)

	Member Label	Direction	Start Magnitude[k/ft,d...	End Magnitude[k/ft,d...	Start Location[ft,%]	End Location[ft,%]
1	M2	Y	-.075	-.075	0	0
2	M3	Y	-.075	-.075	0	0

Member Distributed Loads (BLC 2 : DL + CL)

	Member Label	Direction	Start Magnitude[k/ft,d...	End Magnitude[k/ft,d...	Start Location[ft,%]	End Location[ft,%]
1	M2	Y	-.2	-.2	0	0
2	M3	Y	-.2	-.2	0	0

Member Distributed Loads (BLC 3 : L)

	Member Label	Direction	Start Magnitude[k/ft,d...	End Magnitude[k/ft,d...	Start Location[ft,%]	End Location[ft,%]
1	M2	Y	-.5	-.5	0	0
2	M3	Y	-.5	-.5	0	0

Member Distributed Loads (BLC 4 : W1->)

	Member Label	Direction	Start Magnitude[k/ft,d...	End Magnitude[k/ft,d...	Start Location[ft,%]	End Location[ft,%]
1	M2	y	.07	.07	0	0
2	M3	y	-.4	-.4	0	0

Member Distributed Loads (BLC 5 : W1<-)

	Member Label	Direction	Start Magnitude[k/ft,d...	End Magnitude[k/ft,d...	Start Location[ft,%]	End Location[ft,%]
1	M2	y	.4	.4	0	0
2	M3	y	-.07	-.07	0	0

Member Distributed Loads (BLC 6 : W2->)

	Member Label	Direction	Start Magnitude[k/ft,d...	End Magnitude[k/ft,d...	Start Location[ft,%]	End Location[ft,%]
1	M2	y	.469	.469	0	0
2	M3	y	-.402	-.402	0	0

Member Distributed Loads (BLC 7 : W2<-)

	Member Label	Direction	Start Magnitude[k/ft,d...	End Magnitude[k/ft,d...	Start Location[ft,%]	End Location[ft,%]
1	M2	y	.402	.402	0	0
2	M3	y	-.469	-.469	0	0

Joint Loads and Enforced Displacements (BLC 8 : E->)

	Joint Label	L,D,M	Direction	Magnitude[k,k-ft in,rad k*s^2/ft]
1	N3	L	X	1
2	N6	L	X	1

Joint Loads and Enforced Displacements (BLC 9 : Em->)

	Joint Label	L,D,M	Direction	Magnitude[k,k-ft in,rad k*s^2/ft]
1	N3	L	X	2.6
2	N6	L	X	2.6

Load Combinations

	Description	Solve	PD...	SR...	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	D+C + L	Yes			2	1	3	1												
2	D+W1->	Yes			1	1	4	1												
3	D+W1<-	Yes			1	1	5	1												
4	D+W2->	Yes			1	1	6	1												
5	D+W2<-	Yes			1	1	7	1												

Load Combinations (Continued)

	Description	Solve	PD...	SR...	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
6	D+C+E->	Yes			2	1	8	.714												
7	D+C + E<-	Yes			2	1	8	-.714												
8	D+C + Em->				2	1	9	.59												
9	D+C + Em<-				2	1	9	-.59												

Envelope Member Section Forces

	Member	Sec		Axial[k]	lc	Shear[k]	lc	Moment[k-ft]	lc
1	M1	1	max	11.182	1	1.341	2	0	1
2			min	-4.438	4	-2.941	1	0	1
3		2	max	11.132	1	1.341	2	8.36	1
4			min	-4.488	4	-2.941	1	-3.811	2
5		3	max	11.082	1	1.341	2	16.72	1
6			min	-4.538	4	-2.941	1	-7.622	2
7		4	max	11.032	1	1.341	2	25.081	1
8			min	-4.588	4	-2.941	1	-11.434	2
9		5	max	10.982	1	1.341	2	33.441	1
10			min	-4.638	4	-2.941	1	-15.245	2
11	M2	1	max	0	1	0	1	0	1
12			min	0	1	0	1	0	1
13		2	max	5.883	1	6.472	1	13.855	1
14			min	-3.039	5	-2.546	5	-12.562	2
15		3	max	4.827	1	3.938	1	11.257	3
16			min	-3.183	5	-1.368	5	-8.559	2
17		4	max	3.771	1	1.435	3	8.033	3
18			min	-3.328	5	-1.178	2	-16.007	1
19		5	max	2.715	1	2.605	3	2.624	5
20			min	-3.472	5	-1.259	2	-16.524	1
21	M3	1	max	0	1	0	1	0	1
22			min	0	1	0	1	0	1
23		2	max	5.883	1	2.546	4	12.562	3
24			min	-3.039	4	-6.472	1	-13.855	1
25		3	max	4.827	1	1.368	4	8.559	3
26			min	-3.183	4	-3.938	1	-11.257	2
27		4	max	3.771	1	1.178	3	16.007	1
28			min	-3.328	4	-1.435	2	-8.033	2
29		5	max	2.715	1	1.259	3	16.524	1
30			min	-3.472	4	-2.605	2	-2.624	5
31	M4	1	max	11.182	1	2.941	1	0	1
32			min	-4.438	5	-1.341	3	0	1
33		2	max	11.132	1	2.941	1	3.811	3
34			min	-4.488	5	-1.341	3	-8.36	1
35		3	max	11.082	1	2.941	1	7.622	3
36			min	-4.538	5	-1.341	3	-16.72	1
37		4	max	11.032	1	2.941	1	11.434	3
38			min	-4.588	5	-1.341	3	-25.081	1
39		5	max	10.982	1	2.941	1	15.245	3
40			min	-4.638	5	-1.341	3	-33.441	1

Envelope Member Section Stresses

	Member	Sec		Axial[ksi]	lc	Shear[ksi]	lc	Top Bending[k...	lc	Bottom Bendin...	lc
1	M1	1	max	2.166	1	.594	2	0	1	0	1
2			min	-.86	4	-1.304	1	0	1	0	1
3		2	max	2.157	1	.594	2	2.154	2	4.725	1
4			min	-.869	4	-1.304	1	-4.725	1	-2.154	2

Envelope Member Section Stresses (Continued)

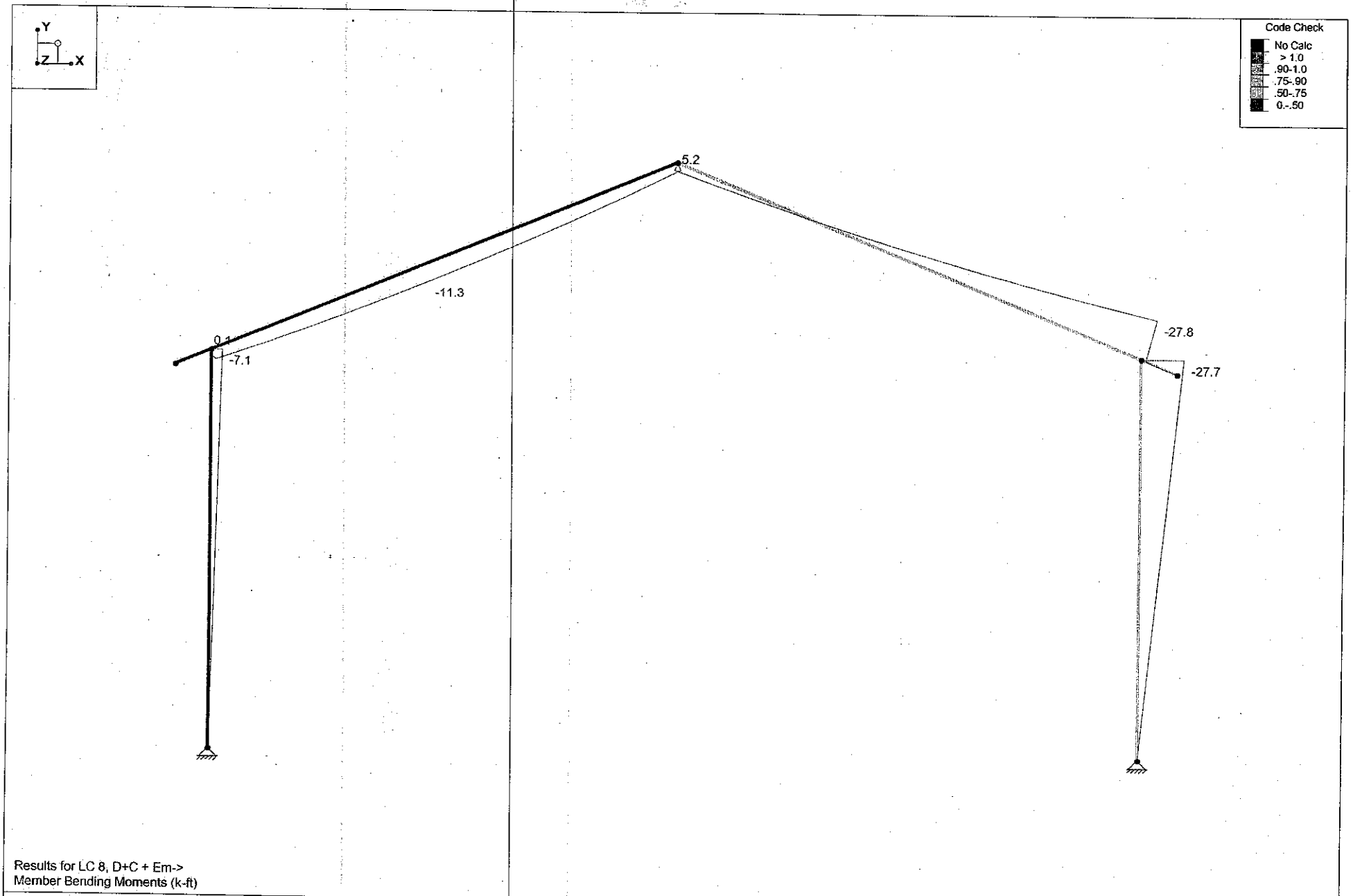
	Member	Sec		Axial[ksi]	lc	Shear[ksi]	lc	Top Bending[k...	lc	Bottom Bendin...	lc
5		3	max	2.147	1	.594	2	4.308	2	9.45	1
6			min	-.879	4	-1.304	1	-9.45	1	-4.308	2
7		4	max	2.137	1	.594	2	6.462	2	14.175	1
8			min	-.889	4	-1.304	1	-14.175	1	-6.462	2
9		5	max	2.128	1	.594	2	8.616	2	18.901	1
10			min	-.898	4	-1.304	1	-18.901	1	-8.616	2
11	M2	1	max	0	1	0	1	0	1	0	1
12			min	0	1	0	1	0	1	0	1
13		2	max	.83	1	1.744	1	8.549	2	9.43	1
14			min	-.429	5	-.686	5	-9.43	1	-8.549	2
15		3	max	.681	1	1.061	1	5.825	2	7.661	3
16			min	-.449	5	-.369	5	-7.661	3	-5.825	2
17		4	max	.532	1	.387	3	10.894	1	5.467	3
18			min	-.47	5	-.317	2	-5.467	3	-10.894	1
19		5	max	.383	1	.702	3	11.246	1	1.786	5
20			min	-.49	5	-.339	2	-1.786	5	-11.246	1
21	M3	1	max	0	1	0	1	0	1	0	1
22			min	0	1	0	1	0	1	0	1
23		2	max	.83	1	.686	4	9.43	1	8.549	3
24			min	-.429	4	-1.744	1	-8.549	3	-9.43	1
25		3	max	.681	1	.369	4	7.661	2	5.825	3
26			min	-.449	4	-1.061	1	-5.825	3	-7.661	2
27		4	max	.532	1	.317	3	5.467	2	10.894	1
28			min	-.47	4	-.387	2	-10.894	1	-5.467	2
29		5	max	.383	1	.339	3	1.786	5	11.246	1
30			min	-.49	4	-.702	2	-11.246	1	-1.786	5
31	M4	1	max	2.166	1	1.304	1	0	1	0	1
32			min	-.86	5	-.594	3	0	1	0	1
33		2	max	2.157	1	1.304	1	4.725	1	2.154	3
34			min	-.869	5	-.594	3	-2.154	3	-4.725	1
35		3	max	2.147	1	1.304	1	9.45	1	4.308	3
36			min	-.879	5	-.594	3	-4.308	3	-9.45	1
37		4	max	2.137	1	1.304	1	14.175	1	6.462	3
38			min	-.889	5	-.594	3	-6.462	3	-14.175	1
39		5	max	2.128	1	1.304	1	18.901	1	8.616	3
40			min	-.898	5	-.594	3	-8.616	3	-18.901	1

Envelope Joint Reactions

	Joint		X [k]	lc	Y [k]	lc	Moment [k-ft]	lc
1	N2	max	2.941	1	11.182	1	0	1
2		min	-1.341	2	-4.438	4	0	1
3	N5	max	1.341	3	11.182	1	0	1
4		min	-2.941	1	-4.438	5	0	1
5	Totals:	max	1.925	3	22.364	1		
6		min	-1.925	2	-8.788	5		

Envelope ASD Steel Code Checks

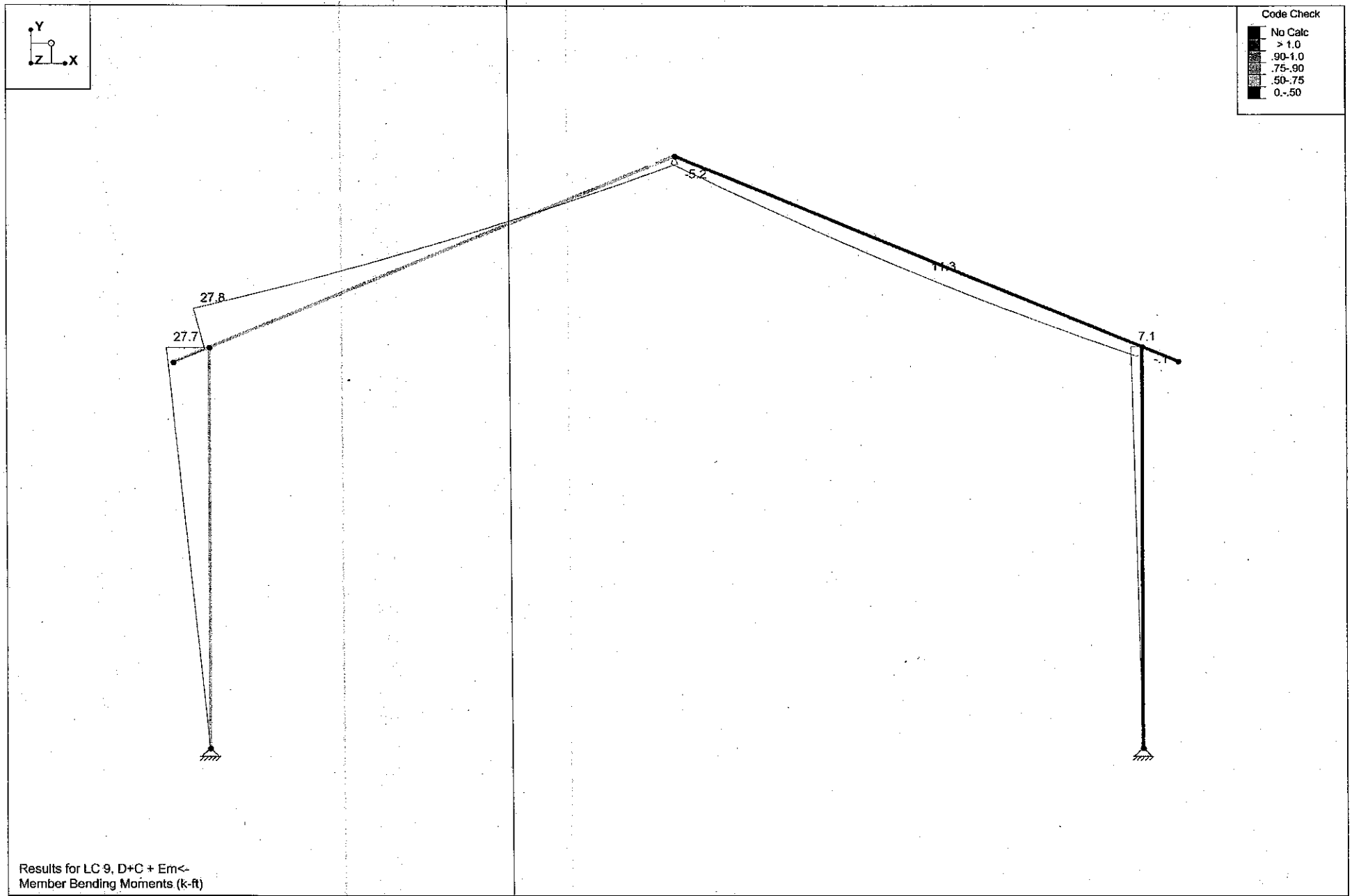
	Member	Shape	Code Check	Loc[ft]	lc	Shear ...	Loc[ft]	lc	Fa [ksi]	Ft [ksi]	Fb [ksi]	Cb	Cm	ASD E...
1	M1	COL	.794	11.372	1	.074	0	1	13.931	30	26.147	1.75	.6	H1-2
2	M2	HSS8X8...	.864	1.106	1	.121	1.106	1	21.619	27.6	27.6	1	.85	H1-2
3	M3	HSS8X8...	.864	1.106	1	.121	1.106	1	21.619	27.6	27.6	1	.85	H1-2
4	M4	COL	.794	11.372	1	.074	0	1	13.931	30	26.147	1.75	.6	H1-2



Results for LC 8, D+C + Em->
Member Bending Moments (k-ft)

CBC Steel Buildings	Entry Frame	
ww		Oct 16, 2007 at 8:26 AM
J27335	D+C + Em->	ENTRY.R2D

T-59



Results for LC 9, D+C + Em<-
Member Bending Moments (k-ft)

CBC Steel Buildings		Entry Frame	
ww			Oct 16, 2007 at 8:27 AM
J27335		D+C + Em<-	ENTRY.R2D

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G-1

CBC STEEL BUILDINGS

Minimum Seismic And Wind Forces Calculation

(UBC97/CBC2001)

Job Number: J27335(part#1) Engineer: WW

Building Geometry Information

Building Width =	50.00	ft.	Roof Weight W/ 0 % Snow =	17.00
Building Length =	19.00	ft.	Weight of Sidewall =	3.00
FSW Eave Height =	12.10	ft.	Weight of Endwall =	3.00
Ridge From FSW =	25.00	ft.	Longitudinal Partition WT. =	0.00
Roof Pitch =	5	/12	Quantity of Longitudinal Part. =	0
Canopy Width @ FSW =	3.20	ft.	Transverse Partition WT. =	0.00
Canopy Width @ RSW =	3.20	ft.	Quantity of Transverse Part. =	0
Max. Interior Bay Trib. =	17.00	ft.	Building End Bay Trib. =	10.00
Longitudinal Special Weight =	0.00	kips	Transverse Special Weight =	0.00
Torsional Bracing: ○ Yes	◉ No		Least Number of Braced Bay Per Sidewall =	1

Rigid Frame Interior Column Information

Number of Interior Column = 0 Top Connection: ◉ Pinned ○ Fixed

Seismic Information

Transverse Direction(Interior):	R =	4.50	Ω_o =	2.80	r_{max} = 0.89
Transverse Direction (End):	R =	4.50	Ω_o =	2.80	
Longitudinal Direction:	R =	4.50	Ω_o =	2.80	r_{max} = 0.50
Soil = SD	N_a =	1.00	N_v =	1.00	C_a = 0.44
Zone = 4			Seismic I =	1.00	

Wind Information

$C_e \cdot q_s \cdot I_W$ = 13.80 psf I_w = 1.00 C_q = 1.3 (Total)

$P_w = C_e \cdot C_q \cdot q_s \cdot I_w$ = 17.9 psf

Wind/Seismic Forces in Transverse Direction

Interior Bay Tributary Width = 17 ft

1. Wind Load

Total Load = P_w* B* H/2 = *3.4 Kips*

2. Seismic Load

ρ =2-20/(r_{max}*A_B^0.5) = 1.32

W = 16.9 Kips

V = 2.5*C_a*I*W/R = 4.1 Kips

C_S = 0.24 E = ρ*V = *5.4 Kips*

E_m = Ω_o*V = *11.6 Kips*

End Bay Tributary Width = 10 ft

1. Wind Load

Total Load = P_w* B* H/2 = *2.0 Kips*

2. Seismic Load

ρ =2-20/(r_{max}*A_B^0.5) = 1.32

W = 10.0 Kips

V = 2.5C_a*I*W/R = 2.4 Kips

C_S = 0.24 E = ρ*V = *3.2 Kips*

E_m = Ω_o*V = *6.8 Kips*

Wind/Seismic Forces in Longitudinal Direction

1. Wind Load

Total Load = P_w*B* H/2 = *7.8* *Kips*

2. Seismic Load (Accidental Torsion Included)

ρ =2-20/(r_{max}*A_B^0.5) = 1.00

W = 20.8 Kips

V = 2.5*C_a*I*W/R = 5.6 Kips

C_S = 0.24 E = ρ*V = *5.6* *Kips*

E_m = Ω_o*V = *15.7* *Kips*

G-2

CBC STEEL BUILDINGS

Minimum Seismic And Wind Forces Calculation

(UBC97/CBC2001)

Job Number: J27335(part#2) Engineer: WW

Building Geometry Information

Building Width =	50.00	ft.	Roof Weight W_i 0 % Snow =	10.00
Building Length =	76.00	ft.	Weight of Sidewall =	3.00
FSW Eave Height =	21.25	ft.	Weight of Endwall =	3.00
Ridge From FSW =	25.00	ft.	Longitudinal Partition WT. =	0.00
Roof Pitch =	5	/12	Quantity of Longitudinal Part. =	0
Canopy Width @ FSW =	3.20	ft.	Transverse Partition WT. =	0.00
Canopy Width @ RSW =	3.20	ft.	Quantity of Transverse Part. =	0
Max. Interior Bay Trib. =	24.00	ft.	Building End Bay Trib. =	14.00
Longitudinal Special Weight =	0.00	kips	Transverse Special Weight =	0.00
Torsional Bracing: ○ Yes	◉ No		Least Number of Braced Bay Per Sidewall =	1

Rigid Frame Interior Column Information

Number of Interior Column = 0 Top Connection: ◉ Pinned ○ Fixed

Seismic Information

Transverse Direction(Interior):	R =	4.50	Ω_o =	2.80	r_{max} =	0.32
Transverse Direction (End):	R =	4.50	Ω_o =	2.80		
Longitudinal Direction:	R =	4.50	Ω_o =	2.80	r_{max} =	0.50
Soil = SD	N_a =	1.00	N_v =	1.00	C_a =	0.44
Zone = 4			Seismic I =	1.00		

Wind Information

$C_e \cdot q_s \cdot I_W$ = 15.20 psf I_w = 1.00 C_q = 1.3 (Total)

$P_w = C_e \cdot C_q \cdot q_s \cdot I_w$ = 19.8 psf

Wind/Seismic Forces in Transverse Direction

Interior Bay Tributary Width = 24 ft

1. Wind Load

Total Load = P_w* B* H/2 = ***7.5 Kips***

2. Seismic Load

ρ =2-20/(r_{max}*A_B^0.5) = 1.03

W = 15.1 Kips

V = 2.5*C_a*I*W/R = 3.7 Kips

C_S = 0.24 E = ρ*V = ***3.8 Kips***

E_m = Ω_o*V = ***10.3 Kips***

End Bay Tributary Width = 14 ft

1. Wind Load

Total Load = P_w* B* H/2 = ***4.4 Kips***

2. Seismic Load

ρ =2-20/(r_{max}*A_B^0.5) = 1.03

W = 8.8 Kips

V = 2.5C_a*I*W/R = 2.1 Kips

C_S = 0.24 E = ρ*V = ***2.2 Kips***

E_m = Ω_o*V = ***6.0 Kips***

Wind/Seismic Forces in Longitudinal Direction

1. Wind Load

Total Load = P_w*B* H/2 = ***13.1*** ***Kips***

2. Seismic Load (Accidental Torsion Included)

ρ =2-20/(r_{max}*A_B^0.5) = 1.39

W = 46.8 Kips

V = 2.5*C_a*I*W/R = 12.6 Kips

C_S = 0.24 E = ρ*V = ***17.5*** ***Kips***

E_m = Ω_o*V = ***35.3*** ***Kips***

CBC STEEL BUILDINGS

Minimum Seismic And Wind Forces Calculation

(UBC97/CBC2001)

Job Number: J27335(part#3) Engineer: WW

Building Geometry Information

Building Width =	50.00	ft.	Roof Weight W/ 0 % Snow =	17.00
Building Length =	87.00	ft.	Weight of Sidewall =	3.00
FSW Eave Height =	12.10	ft.	Weight of Endwall =	3.00
Ridge From FSW =	25.00	ft.	Longitudinal Partition WT. =	0.00
Roof Pitch =	5	/12	Quantity of Longitudinal Part. =	0
Canopy Width @ FSW =	3.20	ft.	Transverse Partition WT. =	0.00
Canopy Width @ RSW =	3.20	ft.	Quantity of Transverse Part. =	0
Max. Interior Bay Trib. =	28.00	ft.	Building End Bay Trib. =	28.00
Longitudinal Special Weight =	0.00	kips	Transverse Special Weight =	0.00
Torsional Bracing: ○ Yes	◉ No		Least Number of Braced Bay Per Sidewall =	1

Rigid Frame Interior Column Information

Number of Interior Column = 0 Top Connection: ◉ Pinned ○ Fixed

Seismic Information

Transverse Direction(Interior):	R =	4.50	Ω_o =	2.80	r_{max} = 0.32
Transverse Direction (End):	R =	4.50	Ω_o =	2.80	
Longitudinal Direction:	R =	4.50	Ω_o =	2.80	r_{max} = 0.50
Soil = SD	N_a =	1.00	N_v =	1.00	C_a = 0.44
Zone = 4			Seismic I =	1.00	

Wind Information

$C_e \cdot q_s \cdot I_W$ = 13.80 psf I_w = 1.00 C_q = 1.3 (Total)

$P_w = C_e \cdot C_q \cdot q_s \cdot I_w$ = 17.9 psf

Wind/Seismic Forces in Transverse Direction

Interior Bay Tributary Width = 28 ft

1. Wind Load

Total Load = P_w* B* H/2 = *5.7 Kips*

2. Seismic Load

ρ =2-20/(r_{max}*A_B^0.5) = 1.11

W = 27.9 Kips

V = 2.5*C_a*I*W/R = 6.8 Kips

C_S = 0.24 E = ρ*V = *7.6 Kips*

$E_m = \Omega_o$*V = *19.1 Kips*

End Bay Tributary Width = 28 ft

1. Wind Load

Total Load = P_w* B* H/2 = *5.7 Kips*

2. Seismic Load

ρ =2-20/(r_{max}*A_B^0.5) = 1.11

W = 27.9 Kips

V = 2.5C_a*I*W/R = 6.8 Kips

C_S = 0.24 E = ρ*V = *7.6 Kips*

$E_m = \Omega_o$*V = *19.1 Kips*

Wind/Seismic Forces in Longitudinal Direction

1. Wind Load

Total Load = P_w*B* H/2 = *7.8* *Kips*

2. Seismic Load (Accidental Torsion Included)

ρ =2-20/(r_{max}*A_B^0.5) = 1.43

W = 86.0 Kips

V = 2.5*C_a*I*W/R = 23.1 Kips

C_S = 0.24 E = ρ*V = *33.0* *Kips*

$E_m = \Omega_o$*V = *64.8* *Kips*

G-4

CBC STEEL BUILDINGS

Minimum Seismic And Wind Forces Calculation

(UBC97/CBC2001)

Job Number: J27335(entry) Engineer: WW

Building Geometry Information

Building Width =	28.00	ft.	Roof Weight W/ 0 % Snow =	10.00
Building Length =	40.00	ft.	Weight of Sidewall =	0.00
FSW Eave Height =	12.10	ft.	Weight of Endwall =	0.00
Ridge From FSW =	14.00	ft.	Longitudinal Partition WT. =	0.00
Roof Pitch =	5	/12	Quantity of Longitudinal Part. =	0
Canopy Width @ FSW =	1.00	ft.	Transverse Partition WT. =	0.00
Canopy Width @ RSW =	1.00	ft.	Quantity of Transverse Part. =	0
Max. Interior Bay Trib. =	25.00	ft.	Building End Bay Trib. =	12.00
Longitudinal Special Weight =	0.00	kips	Transverse Special Weight =	0.00
Torsional Bracing: ○ Yes	◉ No		Least Number of Braced Bay Per Sidewall =	1

Rigid Frame Interior Column Information

Number of Interior Column = 0 Top Connection: ◉ Pinned ○ Fixed

Seismic Information

Transverse Direction(Interior):	R =	4.50	Ω_o =	2.80	r_{max} = 0.63
Transverse Direction (End):	R =	4.50	Ω_o =	2.80	
Longitudinal Direction:	R =	4.50	Ω_o =	2.80	r_{max} = 0.50
Soil = SD	N_a =	1.00	N_v =	1.00	C_a = 0.44
Zone = 4			Seismic I =	1.00	

Wind Information

$C_e \cdot q_s \cdot I_W$ = 13.80 psf I_w = 1.00 C_q = 1.3 (Total)

$P_w = C_e \cdot C_q \cdot q_s \cdot I_w$ = 17.9 psf

Wind/Seismic Forces in Transverse Direction

Interior Bay Tributary Width = 25 ft

1. Wind Load

Total Load = P_w* B* H/2 = *4.0 Kips*

2. Seismic Load

ρ =2-20/(r_{max}*A_B^0.5) = 1.08

W = 7.5 Kips

V = 2.5*C_a*I*W/R = 1.8 Kips

C_S = 0.24 E = ρ*V = *2.0 Kips*

E_m = Ω_o*V = *5.1 Kips*

End Bay Tributary Width = 12 ft

1. Wind Load

Total Load = P_w* B* H/2 = *1.9 Kips*

2. Seismic Load

ρ =2-20/(r_{max}*A_B^0.5) = 1.08

W = 3.6 Kips

V = 2.5C_a*I*W/R = 0.9 Kips

C_S = 0.24 E = ρ*V = *0.9 Kips*

E_m = Ω_o*V = *2.5 Kips*

Wind/Seismic Forces in Longitudinal Direction

1. Wind Load

Total Load = P_w*B* H/2 = *3.8* *Kips*

2. Seismic Load (Accidental Torsion Included)

ρ =2-20/(r_{max}*A_B^0.5) = 1.00

W = 12.0 Kips

V = 2.5*C_a*I*W/R = 3.2 Kips

C_S = 0.24 E = ρ*V = *3.2* *Kips*

E_m = Ω_o*V = *9.0* *Kips*

CBC STEEL BUILDINGS
1700 E. Louise Avenue • Lathrop, CA 95330
Tel: (209) 983-0910 • Fax: (209) 858-2354

Bracing Design
(part #1)

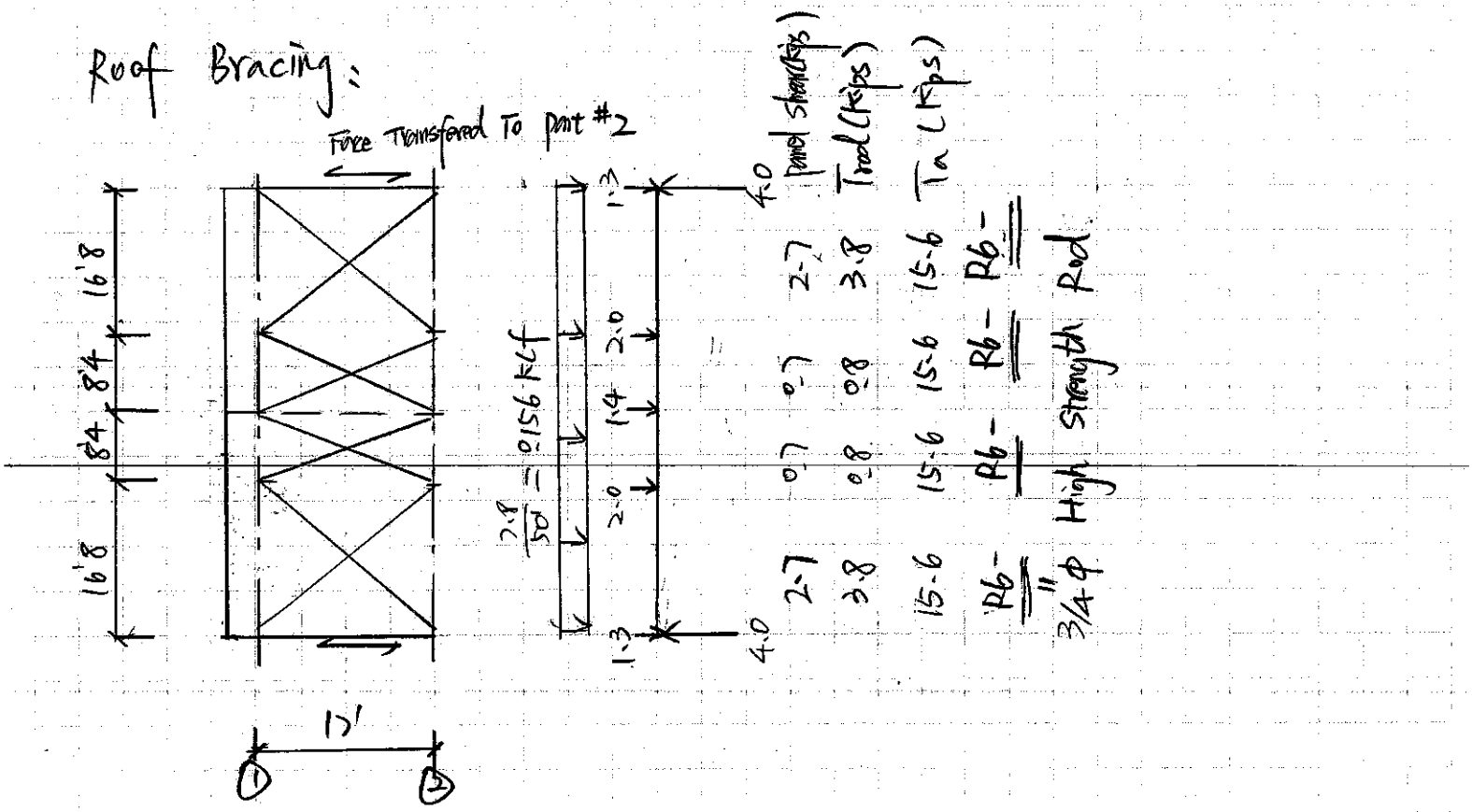
Longitudinal Direction

Wind = 7.8 kips

Seismic $\frac{E}{1.4} = \frac{5.6}{1.4} = 4.0$ kips

$\frac{Em}{1.7} = \frac{15.7}{1.7} = 9.3$ kips

Roof Bracing:



CBC STEEL BUILDINGS
1700 E. Louise Avenue • Lathrop, CA 95330
Tel: (209) 983-0910 • Fax: (209) 858-2354

Bracing Design
(part #2)

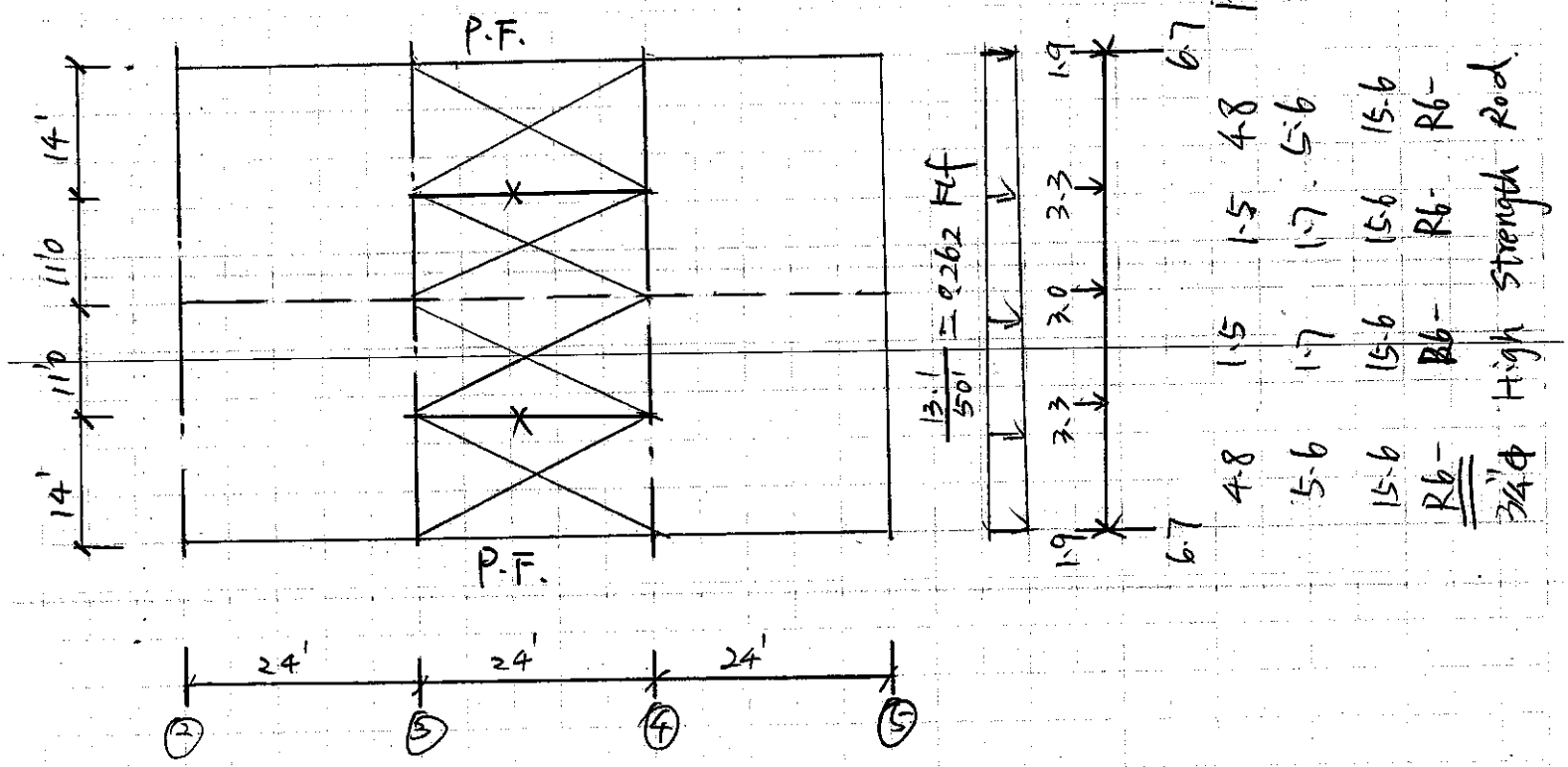
1. Longitudinal Direction

Wind Force = 13.1 kips

Seismic $\frac{E}{1.4} = \frac{17.5}{1.4} = 12.5$ kips

$\frac{E_m}{1.7} = \frac{35.3}{1.7} = 20.8$ kips

Roof bracing: —X— 8.5Z14 strut purlin



G-8

STEEL FRAME ANALYSIS AND DESIGN BY THE DIRECT STIFFNESS METHOD
ALL DESIGN IN ACCORDANCE WITH 89 AISC AND MBMA AS APPLICABLE

BUILDING DESCRIPTION - - POTAL FRAME@BAY 3 J27335

FRAME WIDTH	BAY SPACING	ROOF SLOPES	INT. COLUMNS	MEMBERS	NODES
24.000 ft.	0.100 ft.	1	0	4	5

LEFT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
---	0.000/ 12.0	6.50 in.	255.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
BASE	1	0.000 ft.	17.250 in.	PINNED
EAVE	2	21.250 ft.	17.250 in.	RIGID

ROOF SLOPE 1	SLOPE W/HORIZ.	PURLIN DEPTH	TYP. PURLIN SPACE
---	0.000/ 12.0	8.50 in.	288.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
LEFT END	2	0.000 ft.	21.500 in.	RIGID
SPLICE	3	11.600 ft.	21.500 in.	RIGID
RIGHT END	4	24.000 ft.	21.500 in.	RIGID

RIGHT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
---	0.000/ 12.0	6.50 in.	255.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
EAVE	4	21.250 ft.	17.250 in.	RIGID
BASE	5	0.000 ft.	17.250 in.	PINNED

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LOAD CASE 1 : D
No Stress Check; No Deflection Limits

UNIFORM LOADS			*	WIND COEFFICIENTS		
LIVE (psf)	WIND (psf)	DEAD (psf)	*	C1	C2	C3
0.00	0.00	0.10		0.00	0.00	0.00

LOAD CASE 2 : W->
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS			*	WIND COEFFICIENTS		
LIVE (psf)	WIND (psf)	DEAD (psf)	*	C1	C2	C3
0.00	0.00	0.00		0.00	0.00	0.00

CONCENTRATED LOADS		*		*	LOAD COMPONENTS		
LOAD NO.	LINE NO.	*	LOCATION (ft.)	*	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	*	21.25	*	6.60	0.00	0.00

LOAD CASE 3 : W<-
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS			*	WIND COEFFICIENTS		
LIVE (psf)	WIND (psf)	DEAD (psf)	*	C1	C2	C3
0.00	0.00	0.00		0.00	0.00	0.00

CONCENTRATED LOADS		*		*	LOAD COMPONENTS		
LOAD NO.	LINE NO.	*	LOCATION (ft.)	*	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	*	21.25	*	-6.60	0.00	0.00

LOAD CASE 4 : E->
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS			*	WIND COEFFICIENTS		
LIVE (psf)	WIND (psf)	DEAD (psf)	*	C1	C2	C3
0.00	0.00	0.00		0.00	0.00	0.00

CONCENTRATED LOADS		*		*	LOAD COMPONENTS		
LOAD NO.	LINE NO.	*	LOCATION	*	HORIZONTAL	VERTICAL	MOMENT

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		(ft.)	(kips)	(kips)	(kip-ft)
1	1	21.25	8.00	0.00	0.00
2	1	13.00	2.80	0.00	0.00

LOAD CASE 5 : E<-
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	21.25	-8.00	0.00	0.00
2	1	13.00	-2.80	0.00	0.00

LOAD CASE 6 : ΩE->
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	18.00	0.00	0.00
2	1	13.00	8.00	0.00	0.00

LOAD CASE 7 : ΩE<-
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	21.25	-18.00	0.00	0.00
2	1	13.00	-8.00	0.00	0.00

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LOAD CASE 8 : D + W->
Check By ASD With 1/3 Increase; No Deflection Limits
ALLOWABLE STRESS FACTOR = 1.33 1989 AISC/ASD

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.10

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	6.60	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	-5.07	3.27	0.00	5.70	-3.27	64.19
2	2	3	3.33	-5.79	-64.19	-3.33	6.04	3.29
3	3	4	3.33	-6.04	-3.29	-3.33	6.30	-65.19
4	4	5	6.40	3.33	65.19	-7.02	-3.33	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-3.27	-5.07	0.00
2	5	-3.33	7.02	0.00

Highest check ratio achieved in this load case = 0.531

LOAD CASE 9 : D + W<-
Check By ASD With 1/3 Increase; No Deflection Limits
ALLOWABLE STRESS FACTOR = 1.33 1989 AISC/ASD

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.10

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	21.25	-6.60	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	7.02	-3.33	-0.00	-6.40	3.33	-65.19
2	2	3	3.33	6.31	65.19	-3.33	-6.06	-1.55
3	3	4	3.33	6.06	1.55	-3.33	-5.79	64.19
4	4	5	-5.70	-3.27	-64.19	5.07	3.27	-0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	3.33	7.02	-0.00
2	5	3.27	-5.07	-0.00

Highest check ratio achieved in this load case = 0.531

LOAD CASE 10 : D + 0.71E->
Check By ASD With 1/3 Increase; No Deflection Limits
ALLOWABLE STRESS FACTOR = 1.33 1989 AISC/ASD

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.10

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	5.71	0.00	0.00
2	1	13.00	2.00	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	-5.48	4.10	0.00	6.10	-2.10	67.07
2	2	3	3.62	-6.19	-67.07	-3.62	6.44	2.02
3	3	4	3.62	-6.44	-2.02	-3.62	6.71	-70.94
4	4	5	6.80	3.62	70.94	-7.42	-3.62	-0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-4.10	-5.48	0.00
2	5	-3.62	7.42	-0.00

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Highest check ratio achieved in this load case = 0.578

LOAD CASE 11 : D + 0.71E<-
Check By ASD With 1/3 Increase; No Deflection Limits
ALLOWABLE STRESS FACTOR = 1.33 1989 AISC/ASD

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.10

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	21.25	-5.71	0.00	0.00
2	1	13.00	-2.00	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	7.42	-4.15	0.00	-6.80	2.15	-68.09
2	2	3	2.15	6.71	68.09	-2.15	-6.46	-0.29
3	3	4	2.15	6.46	0.29	-2.15	-6.19	69.93
4	4	5	-6.10	-3.57	-69.93	5.48	3.57	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	4.15	7.42	-0.00
2	5	3.57	-5.48	0.00

Highest check ratio achieved in this load case = 0.570

LOAD CASE 12 : 0.71D + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.07

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	10.59	0.00	0.00

2	1	13.00	4.71	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	-11.59	8.24	0.00	12.21	-3.54	130.55
2	2	3	7.05	-12.30	-130.55	-7.05	12.55	2.57
3	3	4	7.05	-12.55	-2.57	-7.05	12.82	-138.20
4	4	5	12.91	7.05	138.20	-13.53	-7.05	-0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-8.24	-11.59	0.00
2	5	-7.05	13.53	-0.00

LOAD CASE 13 : 0.71D + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.07

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	21.25	-10.59	0.00	0.00
2	1	13.00	-4.71	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	13.53	-8.29	0.00	-12.91	3.59	-131.48
2	2	3	3.59	12.82	131.48	-3.59	-12.57	-0.76
3	3	4	3.59	12.57	0.76	-3.59	-12.30	137.27
4	4	5	-12.21	-7.00	-137.27	11.59	7.00	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	8.29	13.53	-0.00
2	5	7.00	-11.59	0.00

LOAD CASE 14 : 0.53D + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

UNIFORM LOADS			*	WIND COEFFICIENTS		
LIVE (psf)	WIND (psf)	DEAD (psf)	*	C1	C2	C3
0.00	0.00	0.05		0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	21.25	10.59	0.00	0.00
2	1	13.00	4.71	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	-11.59	8.24	0.00	12.21	-3.54	130.55
2	2	3	7.05	-12.30	-130.55	-7.05	12.55	2.57
3	3	4	7.05	-12.55	-2.57	-7.05	12.82	-138.20
4	4	5	12.91	7.05	138.20	-13.53	-7.05	-0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-8.24	-11.59	0.00
2	5	-7.05	13.53	-0.00

LOAD CASE 15 : 0.53D + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

UNIFORM LOADS			*	WIND COEFFICIENTS		
LIVE (psf)	WIND (psf)	DEAD (psf)	*	C1	C2	C3
0.00	0.00	0.05		0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	21.25	-10.59	0.00	0.00
2	1	13.00	-4.71	0.00	0.00

MEMBER END ACTIONS

NODE I * NODE J

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MEMBER	NODES I	NODES J	* AXIAL (kips)	SHEAR (kips)	MOMENT (kip-ft)	* AXIAL (kips)	SHEAR (kips)	MOMENT (kip-ft)
1	1	2	13.53	-8.29	0.00	-12.91	3.59	-131.48
2	2	3	3.59	12.82	131.48	-3.59	-12.57	-0.76
3	3	4	3.59	12.57	0.76	-3.59	-12.30	137.27
4	4	5	-12.21	-7.00	-137.27	11.59	7.00	0.00

SUPPORT	NODE	* SUPPORT REACTIONS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	8.29	13.53	-0.00
2	5	7.00	-11.59	0.00

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MAXIMUM CHECK RATIOS PER 1989 AISC

MEM	OUTER WIDTH (in)	FLANGE * THICK * (in)	WEB * THICK * (in)	INNER WIDTH (in)	FLANGE * THICK * (in)	OUTER FLG RATIO	LOAD	WEB SHEAR RATIO	LOAD	INNER FLG RATIO	LOAD
1	8.00	0.3750	0.1875	8.00	0.3750	0.423	10	0.565	11	0.447	11
2	8.00	0.2500	0.1350	8.00	0.2500	0.471	10	0.517	11	0.469	11
3	8.00	0.2500	0.1350	8.00	0.2500	0.485	11	0.517	10	0.496	10
4	8.00	0.3750	0.1875	8.00	0.3750	0.433	11	0.578	10	0.457	10

CBC STEEL BUILDINGS
1700 E. Louise Avenue • Lathrop, CA 95330
Tel: (209) 983-0910 • Fax: (209) 858-2354

Bracing Design
(part #3)

1. Longitudinal Direction

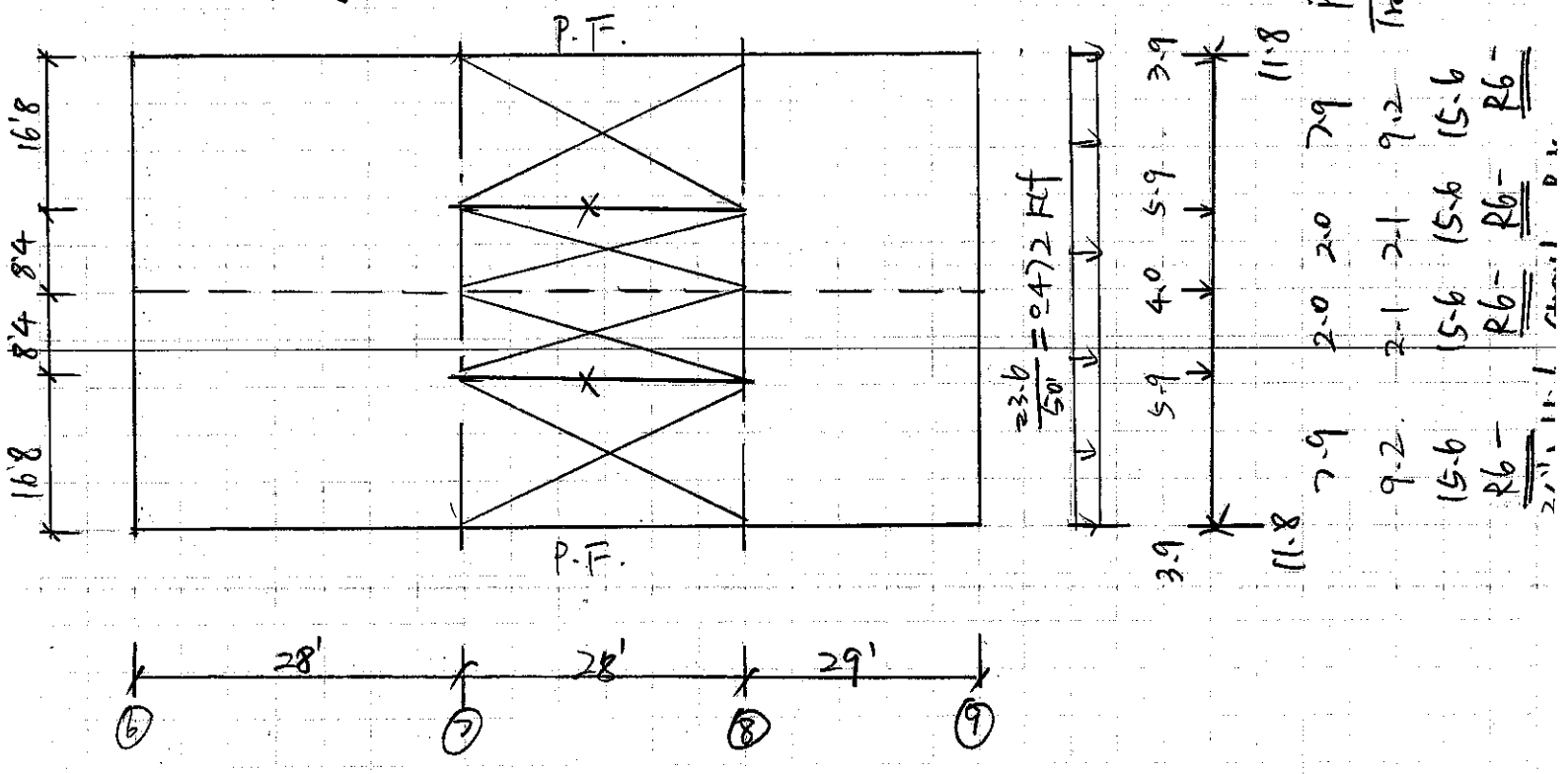
Wind Force = 7.8 Kips

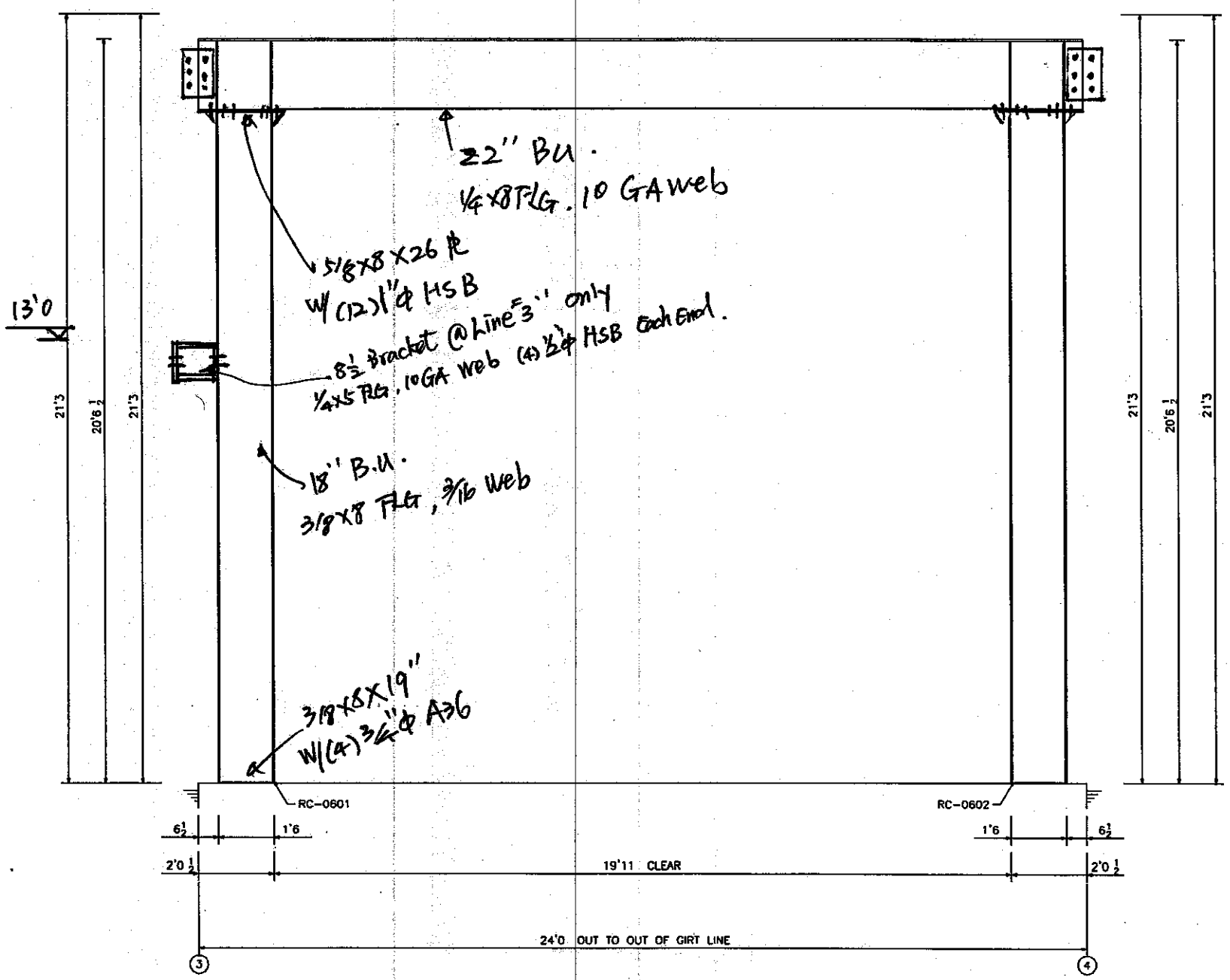
Seismic $\frac{E}{1.4} = \frac{33}{1.4} = 23.6$ Kips

$\frac{Em}{1.7} = \frac{648}{1.7} = 38.2$ Kips

Roof Bracing

— × — 8.5214 strut purlin




13'0
21'3
20'6 1/2
21'3
22" BU.
1/4x8 FLG. 10 GA web
5/8x8x26 PL
W/ (12) 1" ϕ HSB
8 1/2 Bracket @ Line "3" only
1/4x5 FLG, 10GA web (4) 1/2" ϕ HSB Each End.
18" B.U.
3/8x8 FLG, 3/16 web
3/8x8x19"
W/ (4) 3/4" ϕ A36
RC-0601
RC-0602
6 1/2
1'6
2'0 1/2
19'11 CLEAR
24'0 OUT TO OUT OF GIRT LINE
3
4

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SUPPORT REACTIONS

CASE 1 : D

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	0.03	0.97	-0.00
2	5	-0.03	0.97	0.00

CASE 2 : W->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-3.30	-6.05	-0.00
2	5	-3.30	6.05	-0.00

CASE 3 : W<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	3.30	6.05	-0.00
2	5	3.30	-6.05	0.00

CASE 4 : E->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-5.77	-9.03	0.00
2	5	-5.03	9.03	-0.00

CASE 5 : E<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	5.77	9.03	-0.00
2	5	5.03	-9.03	-0.00

CASE 6 : ΩE->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-14.06	-21.35	-0.00
2	5	-11.94	21.35	-0.00

CASE 7 : ΩE<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	14.05	21.35	0.00
2	5	11.95	-21.35	-0.00

CASE 8 : D + W->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-3.27	-5.07	0.00
2	5	-3.33	7.02	0.00

CASE 9 : D + W<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	3.33	7.02	-0.00
2	5	3.27	-5.07	-0.00

CASE 10 : D + 0.71E->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-4.10	-5.48	0.00
2	5	-3.62	7.42	-0.00

CASE 11 : D + 0.71E<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	4.15	7.42	-0.00
2	5	3.57	-5.48	0.00

CASE 12 : 0.71D + 0.59ΩE->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-8.24	-11.59	0.00
2	5	-7.05	13.53	-0.00

CASE 13 : 0.71D + 0.59ΩE<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	8.29	13.53	-0.00
2	5	7.00	-11.59	0.00

CASE 14 : 0.53D + 0.59ΩE->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-8.24	-11.59	0.00
2	5	-7.05	13.53	-0.00

CASE 15 : 0.53D + 0.59ΩE<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	8.29	13.53	-0.00
2	5	7.00	-11.59	0.00

G-21

FLANGE BRACE LOCATIONS

SEQ#	P/G#	REQD?
LEFT COLUMN BASE		
1	1	No (Knee)
RIGHT EAVE		
2	1	No (Knee)
RIGHT COLUMN BASE		

G-22

STEEL FRAME ANALYSIS AND DESIGN BY THE DIRECT STIFFNESS METHOD
ALL DESIGN IN ACCORDANCE WITH 89 AISC AND MBMA AS APPLICABLE

BUILDING DESCRIPTION - - PORTAL FRAME@BAY 6 J27335

FRAME WIDTH	BAY SPACING	ROOF SLOPES	INT. COLUMNS	MEMBERS	NODES
28.000 ft.	0.100 ft.	1	0	4	5

LEFT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
----------	0.000/ 12.0	6.50 in.	145.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
BASE	1	0.000 ft.	14.500 in.	PINNED
EAVE	2	12.083 ft.	14.500 in.	RIGID

ROOF SLOPE 1	SLOPE W/HORIZ.	PURLIN DEPTH	TYP. PURLIN SPACE
----------	0.000/ 12.0	8.50 in.	336.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
LEFT END	2	0.000 ft.	19.500 in.	RIGID
SPLICE	3	11.600 ft.	19.500 in.	RIGID
RIGHT END	4	28.000 ft.	19.500 in.	RIGID

RIGHT WALL	SLOPE W/VERT.	GIRT DEPTH	TYP. GIRT SPACE
----------	0.000/ 12.0	6.50 in.	145.00 in.

	NODE	LOCATION	WEB DEPTH	CONNECTION
EAVE	4	12.083 ft.	14.500 in.	RIGID
BASE	5	0.000 ft.	14.500 in.	PINNED

LOAD CASE 1 : D
No Stress Check; No Deflection Limits

UNIFORM LOADS			*	WIND COEFFICIENTS		
LIVE (psf)	WIND (psf)	DEAD (psf)	*	C1	C2	C3
0.00	0.00	0.10		0.00	0.00	0.00

LOAD CASE 2 : W->
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS			*	WIND COEFFICIENTS		
LIVE (psf)	WIND (psf)	DEAD (psf)	*	C1	C2	C3
0.00	0.00	0.00		0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.00	4.00	0.00	0.00

LOAD CASE 3 : W<-
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS			*	WIND COEFFICIENTS		
LIVE (psf)	WIND (psf)	DEAD (psf)	*	C1	C2	C3
0.00	0.00	0.00		0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	12.00	-4.00	0.00	0.00

LOAD CASE 4 : E->
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS			*	WIND COEFFICIENTS		
LIVE (psf)	WIND (psf)	DEAD (psf)	*	C1	C2	C3
0.00	0.00	0.00		0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION	LOAD COMPONENTS HORIZONTAL	VERTICAL	MOMENT

		(ft.)	(kips)	(kips)	(kip-ft)
1	1	12.00	16.50	0.00	0.00

LOAD CASE 5 : E<-
No Stress Check; L/100 H Deflection Limit

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	12.00	-16.50	0.00	0.00

LOAD CASE 6 : ΩE->
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.00	32.40	0.00	0.00

LOAD CASE 7 : ΩE<-
No Stress Check; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.00

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	12.00	-32.40	0.00	0.00

LOAD CASE 8 : D + W->
Check By ASD With 1/3 Increase; No Deflection Limits
ALLOWABLE STRESS FACTOR = 1.33 1989 AISC/ASD

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UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.10

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.00	4.00	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	-1.07	1.93	-0.00	1.29	-1.93	20.27
2	2	3	2.07	-1.35	-20.27	-2.07	1.59	4.98
3	3	4	2.07	-1.59	-4.98	-2.07	1.94	-21.82
4	4	5	1.99	2.07	21.82	-2.22	-2.07	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-1.93	-1.07	-0.00
2	5	-2.07	2.22	0.00

Highest check ratio achieved in this load case = 0.328

LOAD CASE 9 : D + W<-
Check By ASD With 1/3 Increase; No Deflection Limits
ALLOWABLE STRESS FACTOR = 1.33 1989 AISC/ASD

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.10

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	12.00	-4.00	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)

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1	1	2	2.22	-2.07	-0.00	-1.99	2.07	-21.82
2	2	3	2.07	1.94	21.82	-2.07	-1.70	-2.90
3	3	4	2.07	1.70	2.90	-2.07	-1.35	20.27
4	4	5	-1.29	-1.93	-20.27	1.07	1.93	-0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	2.07	2.22	-0.00
2	5	1.93	-1.07	-0.00

Highest check ratio achieved in this load case = 0.328

LOAD CASE 10 : D + 0.71E->
Check By ASD With 1/3 Increase; No Deflection Limits
ALLOWABLE STRESS FACTOR = 1.33 1989 AISC/ASD

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.10

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.00	11.79	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	-4.26	5.83	-0.00	4.49	-5.83	61.34
2	2	3	5.96	-4.54	-61.34	-5.96	4.78	12.77
3	3	4	5.96	-4.78	-12.77	-5.96	5.13	-62.65
4	4	5	5.19	5.96	62.65	-5.41	-5.96	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-5.83	-4.26	-0.00
2	5	-5.96	5.41	0.00

Highest check ratio achieved in this load case = 0.943

LOAD CASE 11 : D + 0.71E<-
Check By ASD With 1/3 Increase; No Deflection Limits

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ALLOWABLE STRESS FACTOR = 1.33 1989 AISC/ASD

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.10

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	12.00	-11.79	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	5.41	-5.96	-0.00	-5.19	5.96	-62.65
2	2	3	5.96	5.13	62.65	-5.96	-4.89	-10.45
3	3	4	5.96	4.89	10.45	-5.96	-4.55	61.34
4	4	5	-4.49	-5.83	-61.34	4.27	5.83	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	5.96	5.41	-0.00
2	5	5.83	-4.27	0.00

~~Highest check ratio achieved in this load case = 0.943~~

LOAD CASE 12 : 0.71D + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.07

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.00	19.06	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)

1	1	2	-7.25	9.48	0.00	7.47	-9.48	99.72
2	2	3	9.58	-7.53	-99.72	-9.58	7.77	20.05
3	3	4	9.58	-7.77	-20.05	-9.58	8.12	-100.80
4	4	5	8.17	9.58	100.80	-8.40	-9.58	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-9.48	-7.25	0.00
2	5	-9.58	8.40	0.00

LOAD CASE 13 : 0.71D + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)
0.00	0.00	0.07

WIND COEFFICIENTS

C1	C2	C3
0.00	0.00	0.00

CONCENTRATED LOADS

LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	12.00	-19.06	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	8.40	-9.58	0.00	-8.17	9.58	-100.80
2	2	3	9.58	8.12	100.80	-9.58	-7.88	-17.50
3	3	4	9.58	7.88	17.50	-9.58	-7.53	99.72
4	4	5	-7.48	-9.48	-99.72	7.25	9.48	-0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	9.58	8.40	0.00
2	5	9.48	-7.25	-0.00

LOAD CASE 14 : 0.53D + 0.59ΩE->
Check By ASD Special Seismic; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)

WIND COEFFICIENTS

C1	C2	C3

0.00	0.00	0.05	0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	12.00	19.06	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	-7.25	9.48	0.00	7.47	-9.48	99.72
2	2	3	9.58	-7.53	-99.72	-9.58	7.77	20.05
3	3	4	9.58	-7.77	-20.05	-9.58	8.12	-100.80
4	4	5	8.17	9.58	100.80	-8.40	-9.58	0.00

SUPPORT REACTIONS

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-9.48	-7.25	0.00
2	5	-9.58	8.40	0.00

LOAD CASE 15 : 0.53D + 0.59ΩE<-
Check By ASD Special Seismic; No Deflection Limits

UNIFORM LOADS

LIVE (psf)	WIND (psf)	DEAD (psf)	WIND COEFFICIENTS C1	C2	C3
0.00	0.00	0.05	0.00	0.00	0.00

CONCENTRATED LOADS LOAD NO.	LINE NO.	LOCATION (ft.)	LOAD COMPONENTS HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	3	12.00	-19.06	0.00	0.00

MEMBER END ACTIONS

MEMBER	NODES I	NODES J	NODE I AXIAL (kips)	NODE I SHEAR (kips)	NODE I MOMENT (kip-ft)	NODE J AXIAL (kips)	NODE J SHEAR (kips)	NODE J MOMENT (kip-ft)
1	1	2	8.40	-9.58	0.00	-8.17	9.58	-100.80
2	2	3	9.58	8.12	100.80	-9.58	-7.88	-17.50
3	3	4	9.58	7.88	17.50	-9.58	-7.53	99.72
4	4	5	-7.48	-9.48	-99.72	7.25	9.48	-0.00

SUPPORT REACTIONS

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SUPPORT	NODE	* *	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1		9.58	8.40	0.00
2	5		9.48	-7.25	-0.00

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MAXIMUM CHECK RATIOS PER 1989 AISC

MEM	OUTER WIDTH (in)	FLANGE * THICK * (in)	WEB * THICK * (in)	INNER WIDTH (in)	FLANGE * THICK * (in)	OUTER RATIO	FLG LOAD	WEB SHEAR RATIO	LOAD	INNER RATIO	FLG LOAD
1	8.00	0.2500	0.1500	8.00	0.2500	0.587	10	0.943	11	0.624	11
2	8.00	0.2500	0.1350	8.00	0.2500	0.658	10	0.358	11	0.666	11
3	8.00	0.2500	0.1350	8.00	0.2500	0.658	11	0.358	10	0.666	10
4	8.00	0.2500	0.1500	8.00	0.2500	0.587	11	0.943	10	0.624	10

RIGID FRAME CONNECTION DESIGN

By Sheet F-

Date 09-10-2007 Cust Job #

KNEE CONNECTION

USE : Plate 8 x .625 x 24 in. (Fy = 55 ksi)
w/ 1 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 25.798 k/bolt
Plate Allow bolt tension w/o stiffener = 12.899 k/bolt
Bolt Allowable tension = 34.600 k/bolt

Actual Dead + Live moment = 128.50 k-ft (Outside bolts in tension)
Allow Dead + Live moment = 167.69 k-ft w/ 3 rows of bolts w/ 0 stiff
1 row of bolts outside flange

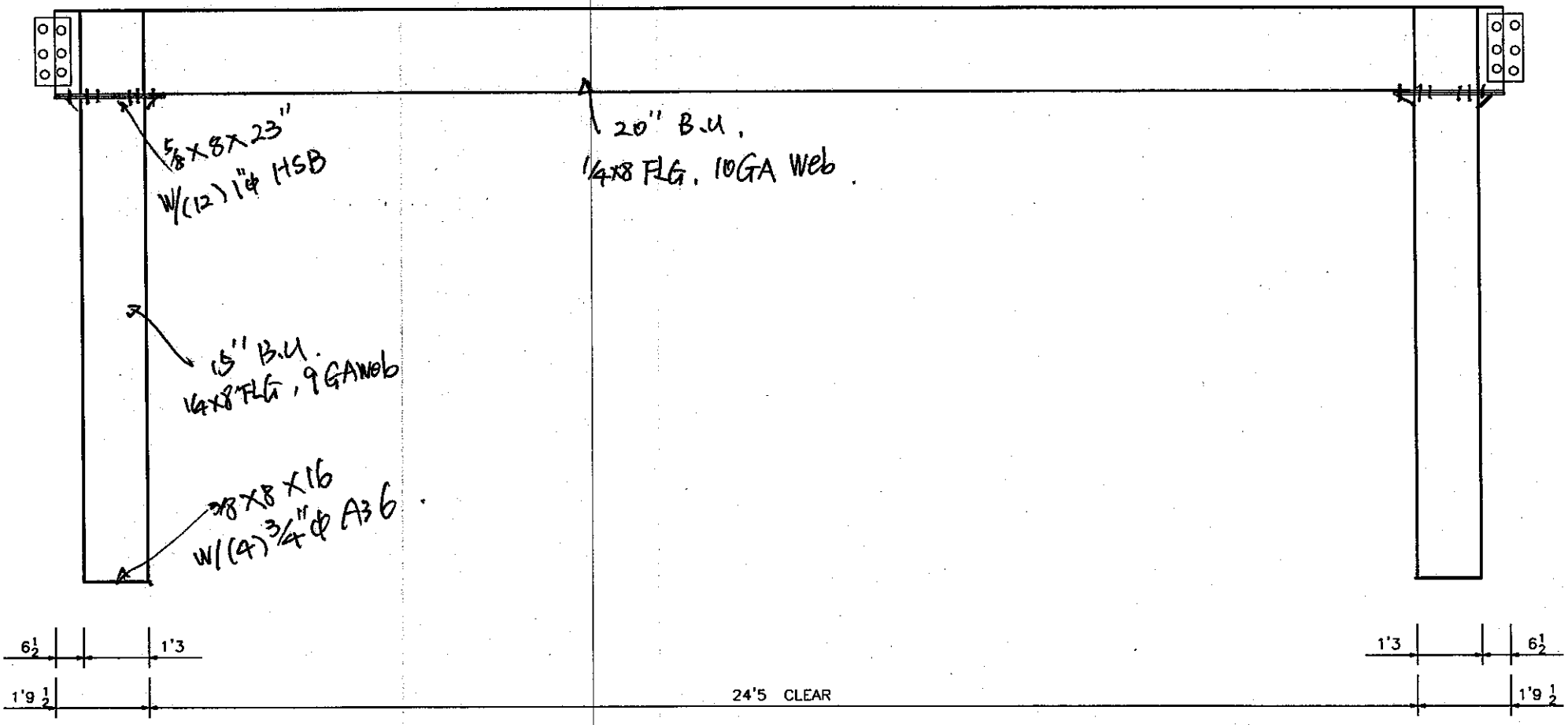
Actual Dead + Live moment = 128.50 k-ft (Inside bolts in tension)
Allow Dead + Live moment = 167.69 k-ft w/ 3 rows of bolts w/ 0 stiff
1 row of Bolts outside flange

COLUMN BASE PLATE

Vertical load = 11.00 kips
Column flg width = 8.00 in.

*** USE: BASE PLATE 8 x .375 x 19 in. (55 ksi)

Min Req'd Thk : .8 x SQR [11 / (8 x 19 x .25 x 55)] = .058 in


5/8 X 8 X 23"
W/(12) 1"φ HSB
20" B.U.
1/4 X 8 FLG. 10GA Web.
15" B.U.
1/4 X 8" FLG, 9GA Web
3/8 X 8 X 16
W/(4) 3/4"φ A36.
6 1/2
1'3
1'3
6 1/2
1'9 1/2
24'5 CLEAR
1'9 1/2

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G-33

RIGID FRAME CONNECTION DESIGN

By

Sheet F-_____

Date 09-10-2007 Cust

Job #

KNEE CONNECTION

USE : Plate 8 x .625 x 21 in. (Fy = 55 ksi)
w/ 1 in. dia. ASTM (A325-N) High Strength Bolts
Plate Allow bolt tension w/ stiffener = 25.798 k/bolt
Plate Allow bolt tension w/o stiffener = 12.899 k/bolt
Bolt Allowable tension = 34.600 k/bolt

Actual Dead + Live moment = 100.80 k-ft (Outside bolts in tension)
Allow Dead + Live moment = 135.44 k-ft w/ 3 rows of bolts w/ 0 stiff
1 row of bolts outside flange

Actual Dead + Live moment = 100.80 k-ft (Inside bolts in tension)
Allow Dead + Live moment = 135.44 k-ft w/ 3 rows of bolts w/ 0 stiff
1 row of Bolts outside flange

COLUMN BASE PLATE

Vertical load = 6.00 kips
Column flg width = 8.00 in.

*** USE: BASE PLATE 8 x .375 x 16 in. (55 ksi)

Min Req'd Thk : .8 x SQR [6 / (8 x 16 x .25 x 55)] = .047 in

SUPPORT REACTIONS

CASE 1 : D

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	0.08	0.57	0.00
2	5	-0.08	0.57	0.00

CASE 2 : W->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-2.01	-1.64	-0.00
2	5	-1.99	1.64	0.00

CASE 3 : W<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	1.99	1.64	-0.00
2	5	2.01	-1.64	0.00

CASE 4 : E->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-8.27	-6.77	0.00
2	5	-8.23	6.77	-0.00

CASE 5 : E<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	8.23	6.77	0.00
2	5	8.27	-6.77	0.00

CASE 6 : ΩE->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-16.25	-13.30	-0.00
2	5	-16.15	13.30	-0.00

CASE 7 : ΩE<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	16.15	13.30	-0.00
2	5	16.25	-13.30	-0.00

CASE 8 : D + W->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-1.93	-1.07	-0.00
2	5	-2.07	2.22	0.00

CASE 9 : D + W<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	2.07	2.22	-0.00
2	5	1.93	-1.07	-0.00

CASE 10 : D + 0.71E->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-5.83	-4.26	-0.00
2	5	-5.96	5.41	0.00

CASE 11 : D + 0.71E<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	5.96	5.41	-0.00
2	5	5.83	-4.27	0.00

CASE 12 : 0.71D + 0.59ΩE->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-9.48	-7.25	0.00
2	5	-9.58	8.40	0.00

CASE 13 : 0.71D + 0.59ΩE<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	9.58	8.40	0.00
2	5	9.48	-7.25	-0.00

CASE 14 : 0.53D + 0.59ΩE->

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	-9.48	-7.25	0.00
2	5	-9.58	8.40	0.00

CASE 15 : 0.53D + 0.59ΩE<-

SUPPORT	NODE	HORIZONTAL (kips)	VERTICAL (kips)	MOMENT (kip-ft)
1	1	9.58	8.40	0.00
2	5	9.48	-7.25	-0.00

FLANGE BRACE LOCATIONS

SEQ#	P/G#	REQD?
LEFT COLUMN BASE		
1	1	No (Knee)
RIGHT EAVE		
2	1	No (Knee)
RIGHT COLUMN BASE		

SINGLE-SPAN BEAM LOADING ANALYSIS

Job Number : J27335

Description : Valley Rafter Support Beam FM-1

Designer : WW

Span Length

L = 28.00 ft.

Max. Shear = 5.53 kips

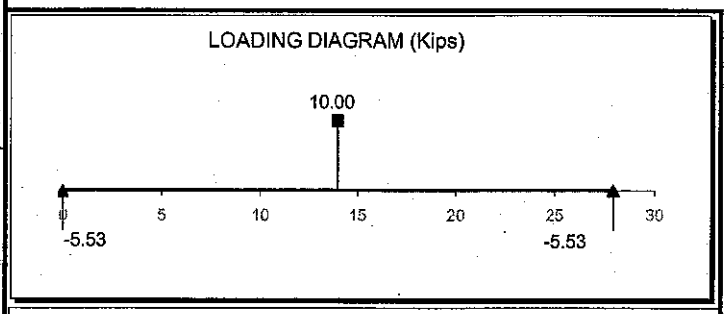
Max. Moment = 73.68 kip-ft at X = 14.00 ft

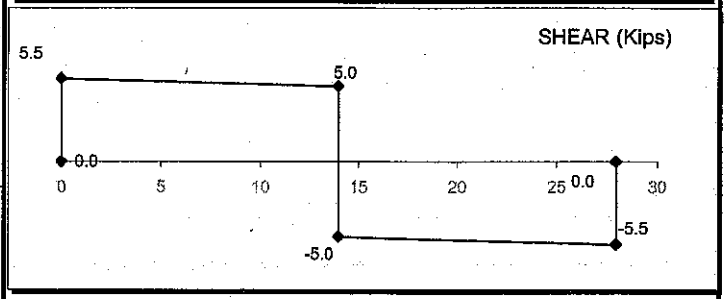
End Moments (Kip-ft)		*Dist. X, ft.*
M_1 =	0.00	0.00
M_2 =	0.00	0.00
Distributed Loading		
w_1 =	0.010	kips/ft.
w_2 =	0.010	kips/ft.
Partial Distributed Loading		
w, kips/ft	*Dist. A*	*Dist. B*
0.00	0.00	0.00
0.00	0.00	0.00
0.00	0.00	0.00

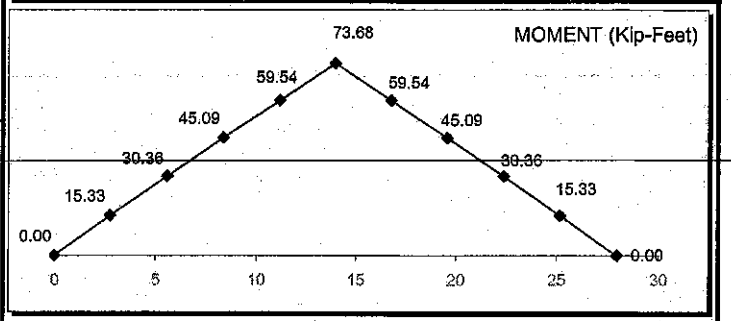
Point Loads, kips		*Dist. X, ft.*
P_1 =	10.00	14.00
P_2 =	0.00	0.00
P_3 =	0.00	0.00
P_4 =	0.00	0.00
P_5 =	0.00	0.00
P_6 =	0.00	0.00
P_7 =	0.00	0.00
P_8 =	0.00	0.00
P_9 =	0.00	0.00
P_{10} =	0.00	0.00

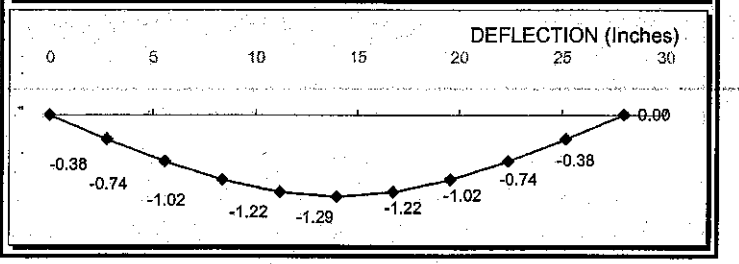
Define Unsupported Segments for C_b Calculation

X_1	X_2	M_1	M_{max}	M_2	C_b
0.00	14.00	0.00	73.68	73.68	1.75
14.00	28.00	73.68	73.68	0.00	1.75









Final Design Requirements *Load includes dead weight of beam.*

Axial Load, P = 0.00 kips

X Axis Moment, Mx = 73.68 kip-ft

Y Axis Moment, My = 0.00 kip-ft

Shear, V = 5.53 kips

L_y = 14.00 ft C_b = 1.75

L_b = 14.00 ft C_{mx} = 1.00

K_x = 1.00 C_{my} = 1.00

K_y = 1.00

Selected Member Size

3-Plate Built-Up Section

F_y = 55 ksi F_u = 70 ksi

Top Flange 0.375 in x 8 in

Web 0.1875 in x 11.25 in

Bottom Flange 0.375 in x 8 in

Section Properties

Area = 8.11 in^2 Full I_x = 225.03 in^4

Weight = 27.59 lb Eff. I_x = 225.03 in^4

Depth = 12.00 in Eff. S_x = 37.50 in^3

I_y = 32.01 in^4 r_t = 2.18

S_y = 8.00 in^3 r_{tequiv} = 2.41

r_x = 5.27 Q_s = 1.00 (Eq. A-B5-3)

r_y = 1.99 Q_a = 1.00 (Eq. A-B5-8 & 10)

Deflection

Maximum Deflection is -1.291 in

Location, X = 14.00 ft

Deflection Ratio = L / 260

Calculated Stresses (AISC ASD 9th Edition)

Axial Stress	f_a = 0 ksi	F_a = 18.94 ksi	(E2-1)
Strong Axis Bending Stress	f_{bx} = 23.58 ksi	F_{bx} = 31.25 ksi	(F1-8)
Tension Flange Bending Stress	f_{bxt} = 23.58 ksi	F_{bxt} = 33 ksi	(F1-8)
Weak Axis Bending Stress	f_{by} = 0 ksi	F_{by} = 37.37 ksi	(F2-3)
Shear Stress	f_v = 2.62 ksi	F_v = 18.79 ksi	(F4-2)

Combined Axial and Bending Stress Ratio

0.75 (Chap. H, Eq. (H1-3))

Shear Stress Ratio = **0.14**

Web to Flange Welds

Standard web to flange weld checks OK.

American Buildings Company

9/7/2007, 11:38 AM

H-2

SINGLE-SPAN BEAM LOADING ANALYSIS

Description : Valley Rafter FM-2

Job Number : J27335

Designer : WW

Span Length

L = 20.00 ft.

Max. Shear = 4.82 kips

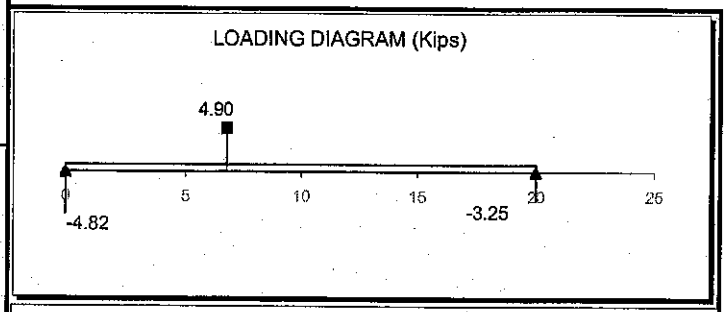
Max. Moment = 29.12 kip-ft at X = 6.80 ft

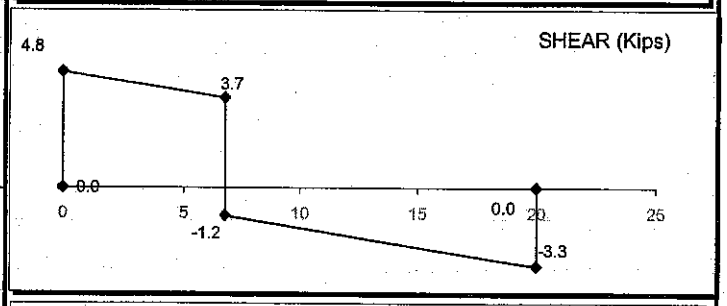
End Moments (Kip-ft)		*Dist. X, ft.*
M_1 =	0.00	0.00
M_2 =	0.00	0.00
Distributed Loading		
w_1 =	0.140	kips/ft.
w_2 =	0.140	kips/ft.
Partial Distributed Loading		
w, kips/ft	*Dist. A*	*Dist. B*
0.00	0.00	0.00
0.00	0.00	0.00
0.00	0.00	0.00

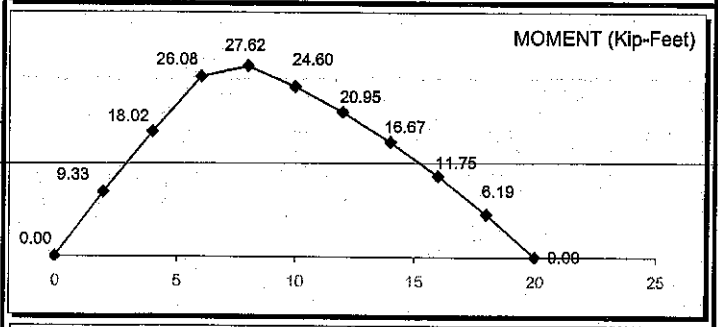
Point Loads, kips		*Dist. X, ft.*
P_1 =	4.90	6.80
P_2 =	0.00	0.00
P_3 =	0.00	0.00
P_4 =	0.00	0.00
P_5 =	0.00	0.00
P_6 =	0.00	0.00
P_7 =	0.00	0.00
P_8 =	0.00	0.00
P_9 =	0.00	0.00
P_{10} =	0.00	0.00

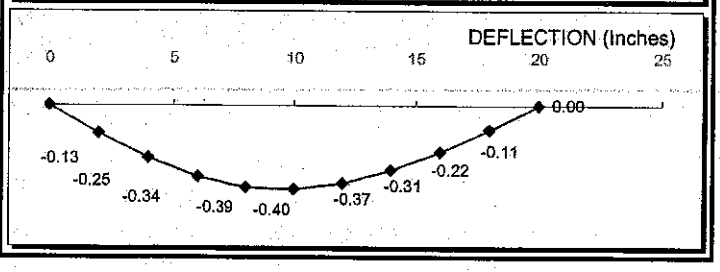
Define Unsupported Segments for C_b Calculation

X_1	X_2	M_1	M_{max}	M_2	C_b
0.00	20.00	0.00	27.62	0.00	1.00
0.00	20.00	0.00	27.62	0.00	1.00









Final Design Requirements — *Load includes dead weight of beam.*

Axial Load, P = 0.00 kips

X Axis Moment, Mx = 29.12 kip-ft

Y Axis Moment, My = 0.00 kip-ft

Shear, V = 4.82 kips

L_y = 20.00 ft — C_b = 1.00

L_b = 20.00 ft — C_{mx} = 1.00

K_x = 1.00 — C_{my} = 1.00

K_y = 1.00

Selected Member Size

3-Plate Built-Up Section

F_y = 55 ksi — F_u = 70 ksi

Top Flange 0.25 in x 8 in

Web 0.1345 in x 11.5 in

Bottom Flange 0.25 in x 8 in

Section Properties

Area = 5.55 in^2 — Full I_x = 155.13 in^4

Weight = 18.87 lb — Eff. I_x = 155.13 in^4

Depth = 12.00 in — Eff. S_x = 25.85 in^3

I_y = 21.34 in^4 — r_t = 2.17

S_y = 5.33 in^3 — r_{tequiv} = 2.36

r_x = 5.29 — Q_s = 0.79 (Eq. A-B5-3)

r_y = 1.96 — Q_a = 1.00 (Eq. A-B5-8 & 10)

Deflection

Maximum Deflection is -0.399 in

Location, X = 9.60 ft

Deflection Ratio = L / 601

Calculated Stresses (AISC ASD 9th Edition)

Axial Stress	f_a = 0 ksi	F_a = 9.97 ksi	(A-B5-12)
Strong Axis Bending Stress	f_{bx} = 13.52 ksi	F_{bx} = 16.48 ksi	(F1-7)
Tension Flange Bending Stress	f_{bxt} = 13.52 ksi	F_{bxt} = 33 ksi	(F1-7)
Weak Axis Bending Stress	f_{by} = 0 ksi	F_{by} = 33 ksi	(F2-2)
Shear Stress	f_v = 3.12 ksi	F_v = 11.39 ksi	(F4-2)

Combined Axial and Bending Stress Ratio

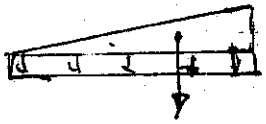
0.82 (Chap. H, Eq. (H1-3))

Shear Stress Ratio = **0.27**

Web to Flange Welds

Standard web to flange weld checks OK.

Bearing stiffeners are required. See Summary.



SINGLE-SPAN BEAM LOADING ANALYSIS

Description : False Rafter FM - 3

Job Number : J27335

Designer : WW

Span Length

L = **25.00** **ft.**

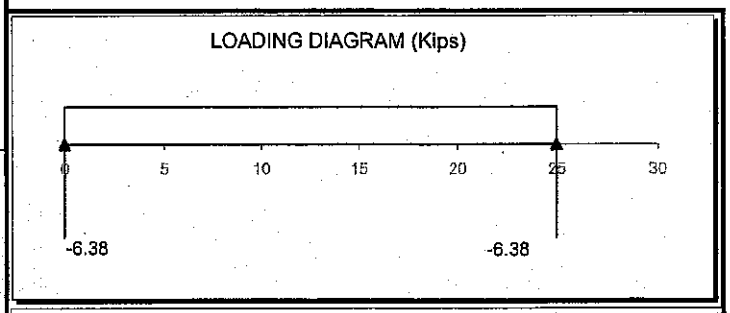
Max. Shear = **6.38** kips

Max. Moment = **39.88** kip-ft at X = **12.50** ft

End Moments (Kip-ft)		*Dist. X, ft.*	*Point Loads, kips*		*Dist. X, ft.*
M_1 =	0.00	0.00	P_1 =	0.00	0.00
M_2 =	0.00	0.00	P_2 =	0.00	0.00
Distributed Loading			P_3 =	0.00	0.00
w_1 =	0.490	kips/ft.	P_4 =	0.00	0.00
w_2 =	0.490	kips/ft.	P_5 =	0.00	0.00
Partial Distributed Loading			P_6 =	0.00	0.00
w, kips/ft	*Dist. A*	*Dist. B*	P_7 =	0.00	0.00
0.00	0.00	0.00	P_8 =	0.00	0.00
0.00	0.00	0.00	P_9 =	0.00	0.00
0.00	0.00	0.00	P_{10} =	0.00	0.00

Define Unsupported Segments for C_b Calculation

X_1	X_2	M_1	M_{max}	M_2	C_b
0.00	25.00	0.00	39.88	0.00	1.00
0.00	25.00	0.00	39.88	0.00	1.00



Final Design Requirements *Load includes dead weight of beam.*

Axial Load, P = **0.00** **kips**

X Axis Moment, Mx = **41.48** **kip-ft**

Y Axis Moment, My = **0.00** **kip-ft**

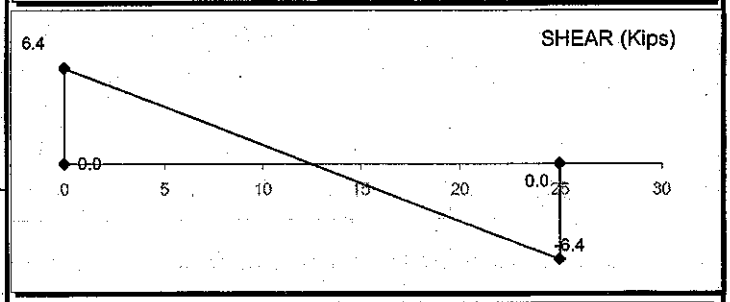
Shear, V = **7.98** **kips**

L_y = **10.00** **ft** C_b = **1.00**

L_b = **10.00** **ft** C_{mx} = **1.00**

K_x = **1.00** C_{my} = **1.00**

K_y = **1.00**



Selected Member Size

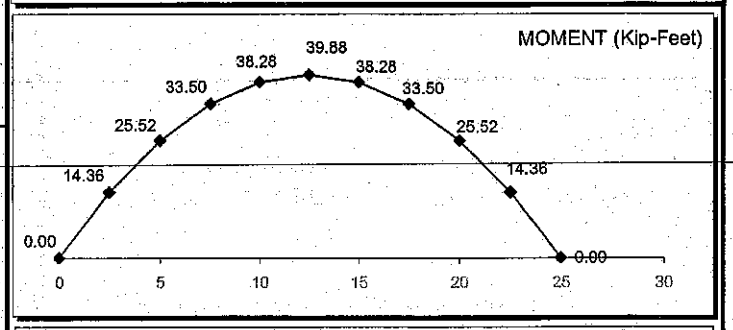
3-Plate Built-Up Section

F_y = **55 ksi** F_u = **70 ksi**

Top Flange **0.375 in x 6 in**

Web **0.1345 in x 11.25 in**

Bottom Flange **0.375 in x 6 in**



Section Properties

Area = **6.01** **in^2** Full I_x = **168.04** **in^4**

Weight = **20.46** **lb** Eff. I_x = **168.04** **in^4**

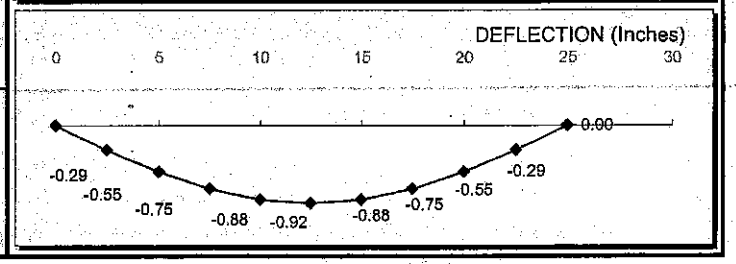
Depth = **12.00** **in** Eff. S_x = **28.01** **in^3**

I_y = **13.50** **in^4** r_t = **1.64**

S_y = **4.50** **in^3** r_{tequiv} = **1.80**

r_x = **5.29** Q_s = **1.00** (Eq. A-B5-3)

r_y = **1.50** Q_a = **1.00** (Eq. A-B5-8 & 10)



Deflection

Maximum Deflection is **-0.921** **in**

Location, X = **12.50** **ft**

Deflection Ratio = **L / 326**

Calculated Stresses (AISC ASD 9th Edition)

Axial Stress	f_a = **0 ksi**	F_a = **20.02 ksi**	(E2-1)
Strong Axis Bending Stress	f_{bx} = **17.77 ksi**	F_{bx} = **27.88 ksi**	(F1-6)
Tension Flange Bending Stress	f_{bxt} = **17.77 ksi**	F_{bxt} = **33 ksi**	(F1-6)
Weak Axis Bending Stress	f_{by} = **0 ksi**	F_{by} = **41.25 ksi**	(F2-1)
Shear Stress	f_v = **5.27 ksi**	F_v = **11.9 ksi**	(F4-2)

Combined Axial and Bending Stress Ratio

0.64 (Chap. H, Eq. (H1-3))

Shear Stress Ratio = **0.44**

Web to Flange Welds

Standard web to flange weld checks OK.

H-4

SINGLE-SPAN BEAM LOADING ANALYSIS

Job Number : J27335

Description : Cupola Support Beam FM-4.

Designer : WW

Span Length

L = 24.00 ft.

Max. Shear = 0.79 kips

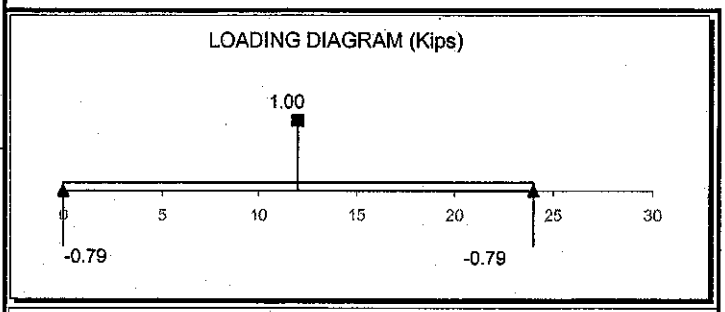
Max. Moment = 7.72 kip-ft at X = 12.00 ft

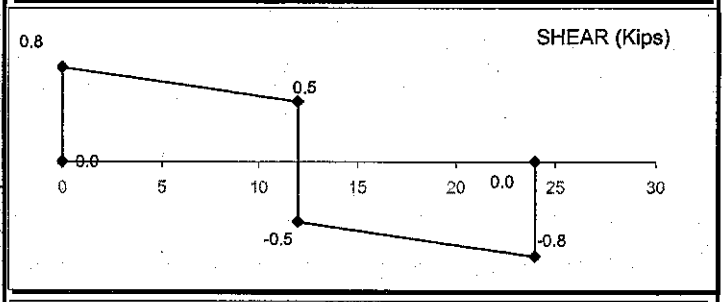
End Moments (Kip-ft)		*Dist. X, ft.*
M_1 =	0.00	0.00
M_2 =	0.00	0.00
Distributed Loading		
w_1 =	0.010	kips/ft.
w_2 =	0.010	kips/ft.
Partial Distributed Loading		
w, kips/ft	*Dist. A*	*Dist. B*
0.00	0.00	0.00
0.00	0.00	0.00
0.00	0.00	0.00

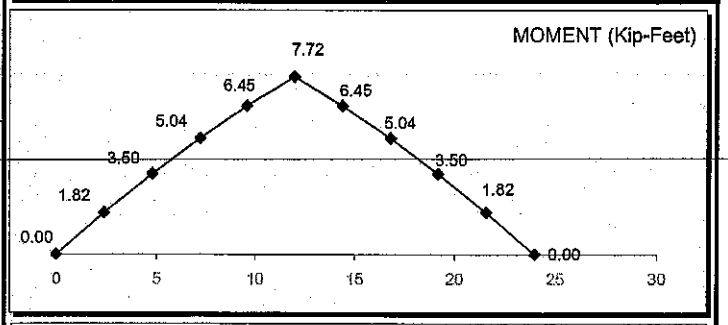
Point Loads, kips		*Dist. X, ft.*
P_1 =	1.00	12.00
P_2 =	0.00	0.00
P_3 =	0.00	0.00
P_4 =	0.00	0.00
P_5 =	0.00	0.00
P_6 =	0.00	0.00
P_7 =	0.00	0.00
P_8 =	0.00	0.00
P_9 =	0.00	0.00
P_{10} =	0.00	0.00

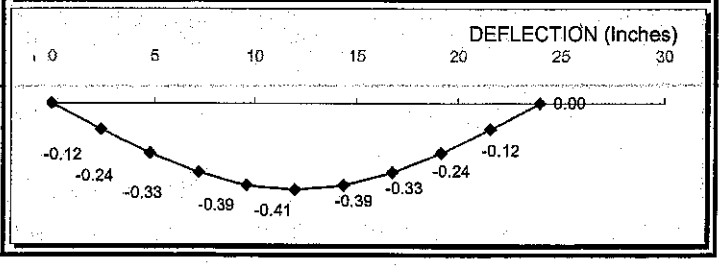
Define Unsupported Segments for C_b Calculation

X_1	X_2	M_1	M_{max}	M_2	C_b
0.00	24.00	0.00	7.72	0.00	1.00
0.00	24.00	0.00	7.72	0.00	1.00









Final Design Requirements *Load includes dead weight of beam.*

Axial Load, P = 0.00 kips

X Axis Moment, Mx = 7.72 kip-ft

Y Axis Moment, My = 0.00 kip-ft

Shear, V = 0.79 kips

L_y =	24.00 ft	C_b =	1.00
L_b =	24.00 ft	C_{mx} =	1.00
K_x =	1.00	C_{my} =	1.00
K_y =	1.00		

Selected Member Size

3-Plate Built-Up Section

F_y = 55 ksi F_u = 70 ksi

Top Flange 0.25 in x 6 in

Web 0.1345 in x 8 in

Bottom Flange 0.25 in x 6 in

Section Properties

Area =	4.08 in^2	Full I_x =	56.80 in^4
Weight =	13.87 lb	Eff. I_x =	56.80 in^4
Depth =	8.50 in	Eff. S_x =	13.36 in^3
I_y =	9.00 in^4	r_t =	1.64
S_y =	3.00 in^3	r_{tequiv} =	2.10
r_x =	3.73	Q_s =	1.00 (Eq. A-B5-3)
r_y =	1.49	Q_a =	1.00 (Eq. A-B5-8 & 10)

Deflection

Maximum Deflection is -0.410 in

Location, X = 12.00 ft

Deflection Ratio = L / 702

Calculated Stresses (AISC ASD 9th Edition)

Axial Stress	f_a = 0 ksi	F_a = 3.98 ksi	(E2-2)
Strong Axis Bending Stress	f_{bx} = 6.93 ksi	F_{bx} = 9.05 ksi	(F1-7)
Tension Flange Bending Stress	f_{bxt} = 6.93 ksi	F_{bxt} = 33 ksi	(F1-7)
Weak Axis Bending Stress	f_{by} = 0 ksi	F_{by} = 34.65 ksi	(F2-3)
Shear Stress	f_v = 0.73 ksi	F_v = 18.95 ksi	(F4-2)

Combined Axial and Bending Stress Ratio

0.77 (Chap. H, Eq. (H1-3))

Shear Stress Ratio = 0.04

Web to Flange Welds

Standard web to flange weld checks OK.

H-5

SINGLE-SPAN BEAM LOADING ANALYSIS

Job Number : J27335

Description : False Rafter Support Column

Designer : WW

Span Length

L = **30.00** ft.

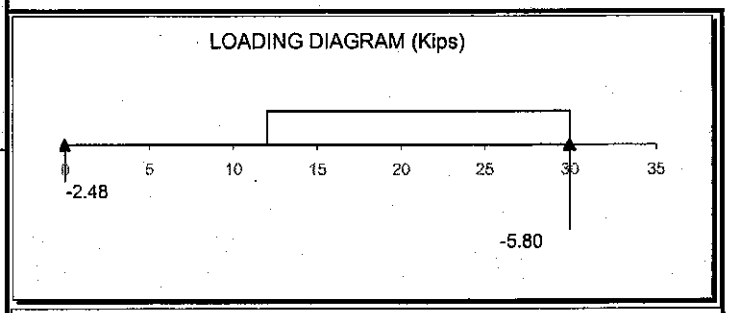
Max. Shear = **5.80** kips

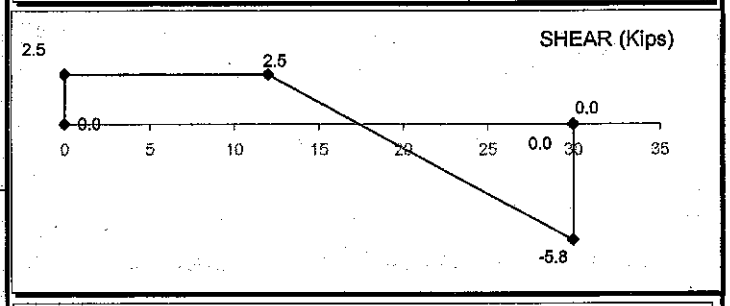
Max. Moment = **36.51** kip-ft at X = **17.40** ft

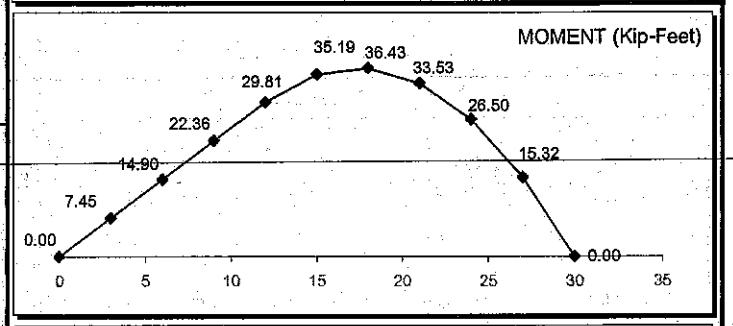
End Moments (Kip-ft)		*Dist. X, ft.*	*Point Loads, kips*		*Dist. X, ft.*
M_1 =	0.00	0.00	P_1 =	0.00	0.00
M_2 =	0.00	0.00	P_2 =	0.00	0.00
Distributed Loading			P_3 =	0.00	0.00
w_1 =	0.000	kips/ft.	P_4 =	0.00	0.00
w_2 =	0.000	kips/ft.	P_5 =	0.00	0.00
Partial Distributed Loading			P_6 =	0.00	0.00
w, kips/ft	*Dist. A*	*Dist. B*	P_7 =	0.00	0.00
0.46	12.00	30.00	P_8 =	0.00	0.00
0.00	0.00	0.00	P_9 =	0.00	0.00
0.00	0.00	0.00	P_{10} =	0.00	0.00

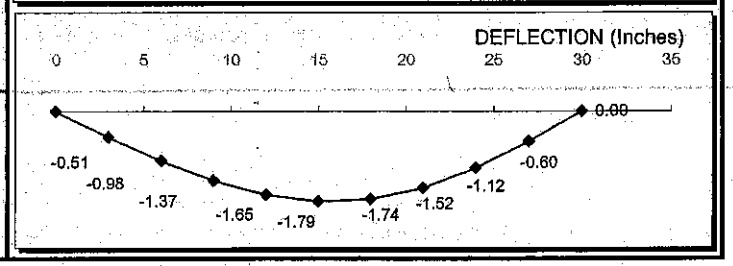
Define Unsupported Segments for C_b Calculation

X_1	X_2	M_1	M_{max}	M_2	C_b
0.00	22.00	0.00	36.51	31.65	1.00
22.00	30.00	31.65	31.65	0.00	1.75









Final Design Requirements *Dead weight of beam not included.*

Axial Load, P = **6.06 kips**

X Axis Moment, Mx = **36.51 kip-ft**

Y Axis Moment, My = **0.00 kip-ft**

Shear, V = **5.80 kips**

L_y = **22.00** ft C_b = **1.00**

L_b = **22.00** ft C_{mx} = **1.00**

K_x = **1.00** C_{my} = **1.00**

K_y = **1.00**

Selected Member Size

3-Plate Built-Up Section

F_y = **55 ksi** F_u = **70 ksi**

Top Flange **0.375 in x 8 in**

Web **0.1875 in x 7.75 in**

Bottom Flange **0.375 in x 8 in**

Section Properties

Area = **7.45 in^2** Full I_x = **106.37 in^4**

Weight = **25.36 lb** Eff. I_x = **106.37 in^4**

Depth = **8.50 in** Eff. S_x = **25.03 in^3**

I_y = **32.00 in^4** r_t = **2.22**

S_y = **8.00 in^3** r_{tequiv} = **2.90**

r_x = **3.78** Q_s = **1.00** (Eq. A-B5-3)

r_y = **2.07** Q_a = **1.00** (Eq. A-B5-8 & 10)

Deflection

Maximum Deflection is **-1.791 in**

Location, X = **15.60 ft**

Deflection Ratio = **L / 201**

Calculated Stresses (AISC ASD 9th Edition)

(Allowable stresses are increased by 1.33 due to wind/seismic load)

Axial Stress	f_a = **0.81 ksi**	F_a = **12.27 ksi**	(E2-2)
Strong Axis Bending Stress	f_{bx} = **17.51 ksi**	F_{bx} = **27.11 ksi**	(F1-6)
Tension Flange Bending Stress	f_{bxt} = **17.51 ksi**	F_{bxt} = **44 ksi**	(F1-6)
Weak Axis Bending Stress	f_{by} = **0 ksi**	F_{by} = **49.83 ksi**	(F2-3)
Shear Stress	f_v = **3.64 ksi**	F_v = **29.33 ksi**	(F4-1)

Combined Axial and Bending Stress Ratio

0.71 (Chap. H, Eq. (H1-3))

Shear Stress Ratio = **0.12**

Web to Flange Welds

Standard web to flange weld checks OK.

H-6

CBC STEEL BUILDINGS

Framed Openings Calculation

Job Number J27335 Engineer WW

Module 1 ☑ FSW BAY ☑ RSW BAY ☐ LEW BAY ☐ REW BAY

DIMENSIONS ☐ No Stress Increase	
Span length (column to column)	24.00 ft
Door width	12.50 ft
Door Height	13.17 ft
Distance from left column to 1st jamb	5.75 ft
Distance from header to jamb support	0.00 ft
Ht. of the girt/eave after jamb support	16.17 ft
Deflection (standard is L/90 for 50 yr. wind)	L / 90

MSA SECONDARY FRAME OUTPUT		ω
Wind pressure (50 yr. wind)	13.80 psf	1.0
Suction coefficient	-0.90	
Pressure coefficient	0.90	
Suction	-12.42 psf	
Pressure	12.42 psf	
Design spacing, jamb supp.	18.00 in	
Allowable Stress Ratio	1.03	

Wall Girt Depth ◉ 8.5" ○ 9.5" ○ 12"

Nested (2) Girts ? ☐ No

Stress Increase = 1.33

Use Hot-Rolled Channels? ○ Yes ◉ No

Distance Between Lateral Supports (in) N/A in

Channel Depth Selection ○ C8 ○ C9 ○ C10

Use Hot Rolled Jambs? ☐ No

Use Different Depth Jambs? ☐ No

PANEL CONDITION

Jamb Support(s) R = 0.70

See comment window for R values

Header R = 0.62 *Jambs R = 0.62

See comment windows for R values

Use Different Depth Jamb Support? ☐ No

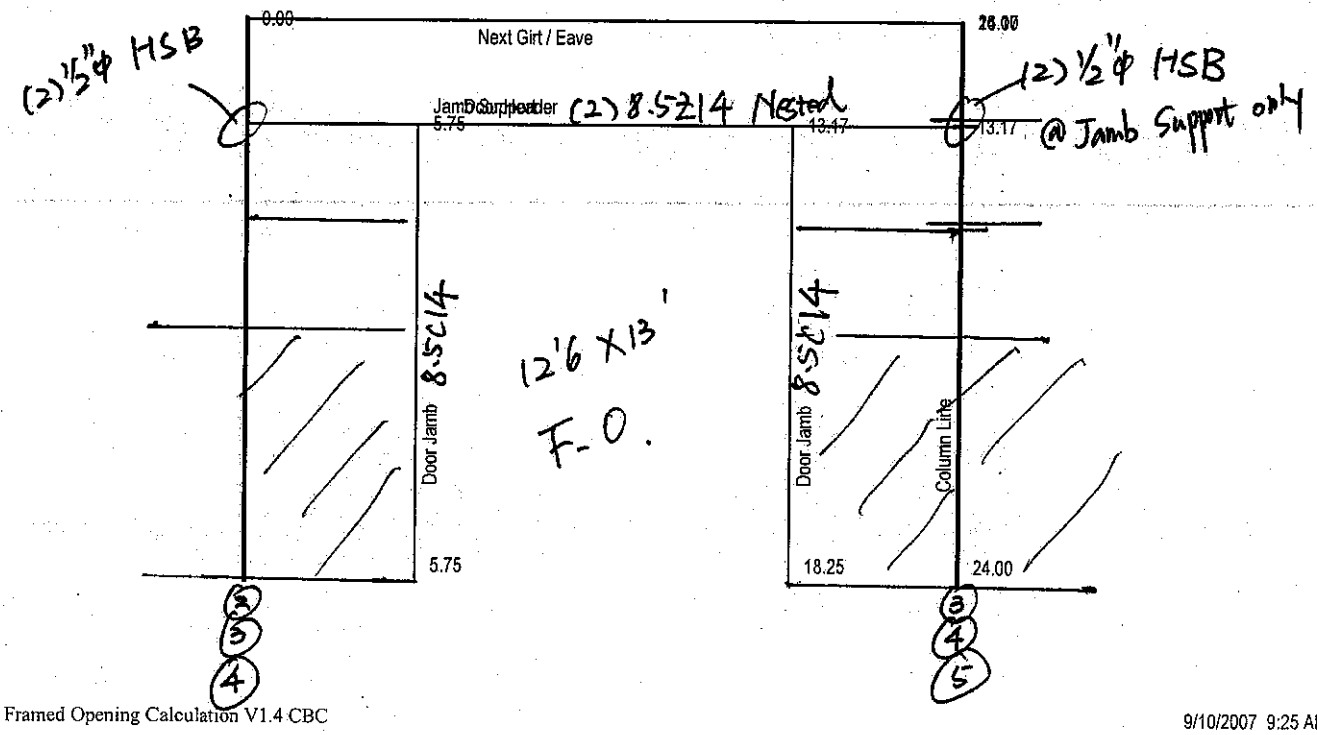
Maximum Girt Spacing = 7.33 ft

Recommended Member For *Jamb Support(s)*	~~8.5Z14~~ (2) 8.5Z14	Stress Ratio= 0.87	Δ_{max} = L / 161
Recommended Minimum Member Size For *Jambs*	~~8.5C16~~ 8.5C14	Stress Ratio= 0.38	Δ_{max} = L / 548
Recommended Minimum Member Size For *Header*	#####	Stress Ratio= ######	Δ_{max} = L / #VALUE!

Minimum 16 gage for jamb and header

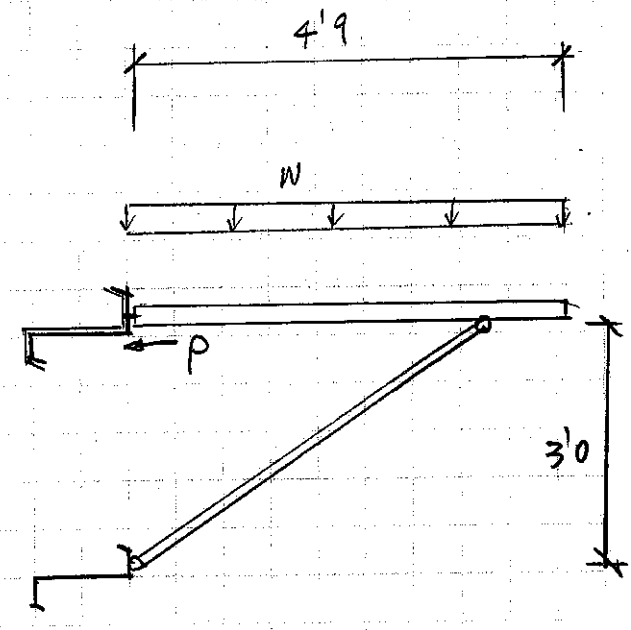
All members are designed as simple span.

The reduced sectional properties were used for cold formed members.



CBC STEEL BUILDINGS
1700 E. Louise Avenue • Lathrop, CA 95330
Tel: (209) 983-0910 • Fax: (209) 858-2354

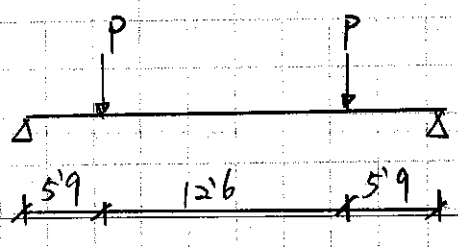
Shade Canopy Support



$W = 2.5 \times 13.8 \times 7' = 242$ plf

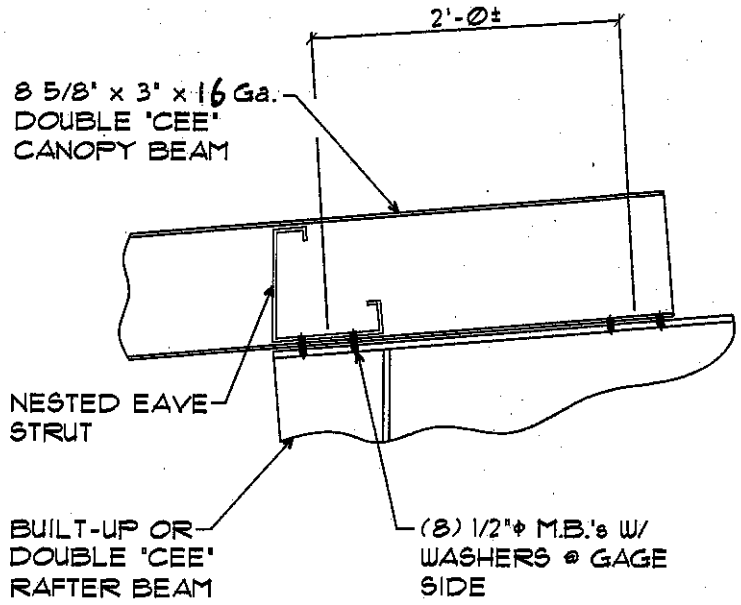
$P = \dfrac{242 \times \frac{4.8^2}{2}}{3'} = 928$ #

$M_{max} = 928 \times 5.75 = 5.4$ k-ft

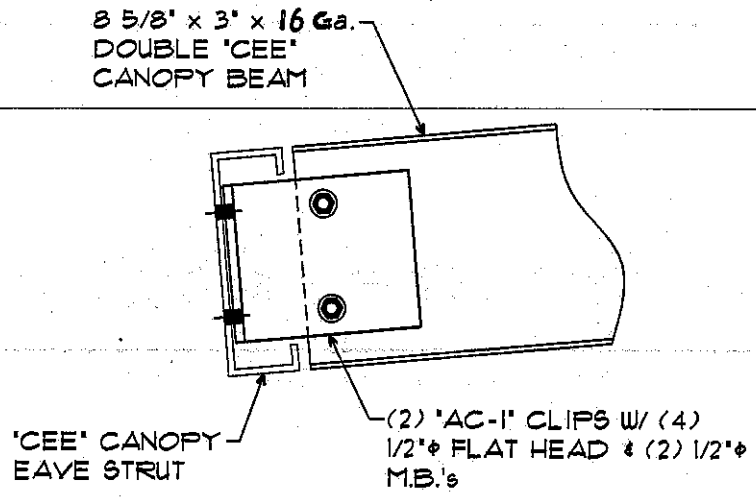


use 8.5Z14 $\quad M_a = 1.33 \times 4.52 = 6.1$ k-ft

Add (1) 8.5Z14 @ Jamb Support. 13'2



PIGGYBACK CANOPY



CANOPY STRUT