STRUCTURAL CALCULATIONS

FOR

CFSD HIGH SCHOOL KITCHEN AND PERFORMING ARTS EXPANSION TUCSON, ARIZONA

Registered Professional Engineer (Structural)
CERTIFICATE NO.
15325
WARREN L.
WHITE
Date Signed 4-8-11
ARIZONA U.S.A.

Expires 12·31·13

Prepared For:

NTD Architecture
2940 N Swan Road, Suite 214
Tucson, AZ 85712

April 2011

Holben, Martin & White, Inc.
2950 North Country Club Road
Tucson, Arizona 85716
520-327-9491

INDEX
TO
STRUCTURAL CALCULATIONS

Holben, Martin & White, Inc.
2950 North Country Club Road
Tucson, Arizona 85716
520-327-9491

MARTIN
HOLBEN & WHITE

Code Search

Code: International Building Code 2006

Occupancy:

Occupancy Group = E Educational

Occupancy Category & Importance Factors:

Occupancy Category =	II
Wind factor =	1.00
Snow factor =	1.00
Seismic factor =	1.00

Type of Construction:

Fire Rating:

Roof =	2.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	0.00 / 12	0.0 deg
Building length (L)	32.0 ft	
Least width (B)	32.0 ft	
Mean Roof Ht (h)	18.0 ft	
Parapet ht above grd	21.0 ft	
Minimum parapet ht	3.0 ft	

Live Loads:

Roof

0 to 200 sf:	20 psf
200 to 600 sf:	24 - 0.02Area, but not less than 12 psf
over 600 sf:	12 psf

Awnings and canopys	5 psf

Floor

Typical Floor	50 psf
Partitions	15 psf
Stairs & Exitways	100 psf
Balconies (exterior)	100 psf
Mechanical	
Corridors above first floor	80 psf
Lobbies & first floor corridors	100 psf

Holben, Martin & White
2950 N. Country Club
Tucson, AZ
327-9491

JOB TITLE	CFSD HS Kitchen and Performing Arts Expansion		
JOB NO.	100048	SHEET NO.	2
CALCULATED BY		DATE	3/11/11
CHECKED BY	WW	DATE	4/11/11

Roof Design Loads

Items	Description	Multiple	psf (max)	psf (min)
Roofing	3 ply composite, no gravel		4.0	2.0
Decking	Metal Roof deck, 1.5, 20 ga.		2.5	2.0
Framing	Steel roof joists & girders		4.0	3.0
Insulation	Rigid insulation, per 1"	x 3.0	5.0	3.0
Ceiling	Suspended acoustical tile		2.0	1.0
Mech & Elec	Mech. & Elec.		3.0	1.0
Misc.	Misc.		1.5	0.0
		Actual Dead Load	● 22.0	● 12.0
		Use this DL instead	○ 20.0	○ 9.0
		Live Load	20.0	0.0
		Snow Load	0.0	0.0
		Wind (zone 2 - 100sf)	10.0	-19.9
ASD Loading		Dead + Live Load	42.0	-
		Dead + 0.75(Wind + Live) Load	44.5	-
		0.6*Dead + Wind Load	-	-12.7
LRFD Loading		1.2D + 1.6 Lr + 0.8W	66.4	-
		1.2D + 1.6W + 0.5Lr	52.4	-
		0.9D + 1.6W	-	-21.0

Roof Live Load Reduction   Roof angle   0.00 / 12   0.0 deg

0 to 200 sf: 20.0 psf
200 to 600 sf: 24 - 0.02Area, but not less than 12 psf
over 600 sf: 12.0 psf

	300 sf	18.0 psf
	400 sf	16.0 psf
	500 sf	14.0 psf
User Input:	450 psf	15.0 psf

CODE SUMMARY

Code: International Building Code 2006

Live Loads:

Roof	0 to 200 sf:	20 psf
	200 to 600 sf:	24 - 0.02Area, but not less than 12 psf
	over 600 sf:	12 psf
Awnings and canopys		5 psf
Typical Floor		50 psf
Corridors above first floor		80 psf
Lobbies & first floor corridors		100 psf
Mechanical		0 psf
Stairs & Exitways		100 psf
Balconies (exterior)		100 psf
Partitions		15 psf

Dead Loads:

~~Floor~~	~~86.1 psf~~
Roof	22.0 psf

Wind Design Data:

Basic Wind speed	90 mph
Mean Roof Ht (h)	18.0 ft
Occupancy Category	II
Importance Factor	1.00
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure Coef.	+/-0.18
Directionality (Kd)	0.85

Roof Snow Loads:

Design Roof Snow load	=	0.0 psf
Flat Roof Snow Load	Pf =	5.0 psf
Ground Snow Load	Pg =	5.0 psf
Rain on Snow Surcharge	=	3.5 psf
Importance Factor	I =	1.00
Snow Exposure Factor	Ce =	1.00
Thermal Factor	Ct =	1.00
Sloped-roof Factor	Cs =	1.00

Earthquake Design Data:

Occupancy Category:	=	II
Importance Factor	I =	1.00
Mapped spectral response	Ss =	28.30 %g
accelerations	S1 =	8.10 %g
Site Class	=	D
Spectral Response Coef.	Sds =	0.297
	Sd1 =	0.130
Seismic Design Category	=	B
Basic Structural System	=	Bearing Wall Systems
Seismic Resisting System	=	Intermediate reinforced masonry shear walls
Design Base Shear	V =	0.085W
Seismic Response Coef.	Cs =	0.085
Response Modification Factor	R =	3.5
Analysis Procedure	=	Equivalent Lateral-Force Analysis

Seismic Loads:

Occupancy Category: II
Importance Factor (I) : 1.00
Site Class : D

Ss (0.2 sec) = 28.30 %g
S1 (1.0 sec) = 8.10 %g

Fa =	1.574		Sms =	0.445		S_{DS} =	0.297		Design Category =	B
Fv =	2.400		Sm1 =	0.194		S_{D1} =	0.130		Design Category =	B

Seismic Design Category = **B**
Number of Stories: 1
Structure Type: Not applicable
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity
Flexible Diaphragms: Yes
Building System: **Bearing Wall Systems**
Seismic resisting system: **Intermediate reinforced masonry shear walls**
System Building Height Limit: **Height not limited**
Actual Building Height (hn) = 18.0 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Factor (R) = 3.5
System Over-Strength Factor (Ωo) = 2
Deflection Amplification Factor (Cd) = 2.25
S_{DS} = 0.297
S_{D1} = 0.130

Seismic Load Effect (E) = $\rho Q_E \pm 0.2S_{DS} D$ = ρQ_E +/- 0.059D
Special Seismic Load Effect (E) = $\Omega o\, Q_E \pm 0.2S_{DS} D$ = 2.0 Q_E +/- 0.059D

ρ = redundancy coefficient
Q_E = horizontal seismic force
D = dead load

PERMITTED ANALYTICAL PROCEDURES

Index Force Analysis (Seismic Category A only) Method Not Permitted

Simplified Analysis Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted

Building period coef. (C_T) = 0.020 Cu = 1.64
Approx fundamental period (Ta) = $C_T h_n^x$ = 0.175 sec x= 0.75 Tmax = CuTa = 0.287
User calculated fundamental period (T) = 0 sec Use T = 0.175
Long Period Transition Period (TL) = ASCE7 map = 6
Seismic response coef. (Cs) = SdsI/R = 0.085
need not exceed Cs = Sd1 I /RT = 0.212
but not less than Cs = 0.044Sds = 0.013
USE Cs = 0.085
Design Base Shear V = 0.085W

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

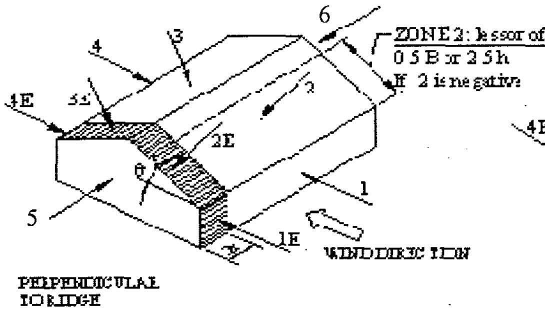
Structure Type: All other structures

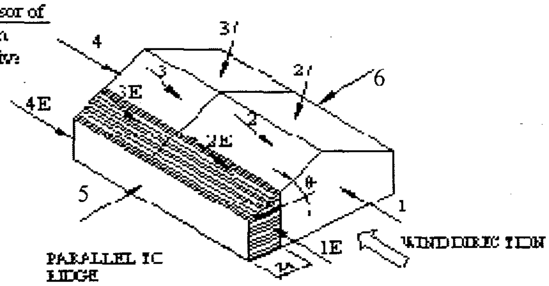
Allowable story drift = 0.020hsx where hsx is the story height below level x

Holben, Martin & White
2950 N. Country Club
Tucson, AZ
327-9491

JOB TITLE CFSD HS Kitchen and Performing Arts Expansion
JOB NO. 100048 SHEET NO. 5
CALCULATED BY DATE 3/11/11
CHECKED BY WW DATE 4/11/11

Wind Loads - MWFRS h≤60' (Low-rise Buildings) Enclosed/partially enclosed only





Torsional loads are 25% of zones 1 - 4. See code for loading diagram

Transverse Direction

Kz = Kh (case 1) = 0.88
Base pressure (qh) = **15.5 psf**
GCpi = +/-0.18

Longitudinal Direction

Edge Strip (a) 3.2 ft
End Zone (2a) 6.4 ft
Zone 2 length = 16.0 ft

	Transverse Direction			Longitudinal Direction			
	Perpendicular θ = 0.0 deg			Parallel θ = 0.0 deg			
Surface	GCpf	w/-GCpi	w/+GCpi	GCpf	w/-GCpi	w/+GCpi	
1	0.40	0.58	0.22	0.40	0.58	0.22	
2	-0.69	-0.51	-0.87	-0.69	-0.51	-0.87	
3	-0.37	-0.19	-0.55	-0.37	-0.19	-0.55	
4	-0.29	-0.11	-0.47	-0.29	-0.11	-0.47	
5	-0.45	-0.27	-0.63	-0.45	-0.27	-0.63	
6	-0.45	-0.27	-0.63	-0.45	-0.27	-0.63	
1E	0.61	0.79	0.43	0.61	0.79	0.43	
2E	-1.07	-0.89	-1.25	-1.07	-0.89	-1.25	
3E	-0.53	-0.35	-0.71	-0.53	-0.35	-0.71	
4E	-0.43	-0.25	-0.61	-0.43	-0.25	-0.61	

Wind Surface pressures (psf)

Surface		Transverse w/-GCpi	Transverse w/+GCpi		Longitudinal w/-GCpi	Longitudinal w/+GCpi	
1		9.0	3.4		9.0	3.4	
2		-7.9	-13.5		-7.9	-13.5	
3		-3.0	-8.6		-3.0	-8.6	
4		-1.7	-7.3		-1.7	-7.3	
5		-4.2	-9.8		-4.2	-9.8	
6		-4.2	-9.8		-4.2	-9.8	
1E		12.3	6.7		12.3	6.7	
2E		-13.8	-19.4		-13.8	-19.4	
3E		-5.4	-11.0		-5.4	-11.0	
4E		-3.9	-9.5		-3.9	-9.5	

Windward roof overhangs: 10.6 psf (upward) add to windward roof pressure

Parapet

Windward parapet: 24.1 psf (GCpn = +1.5)
Leeward parapet: -16.1 psf (GCpn = -1.0)

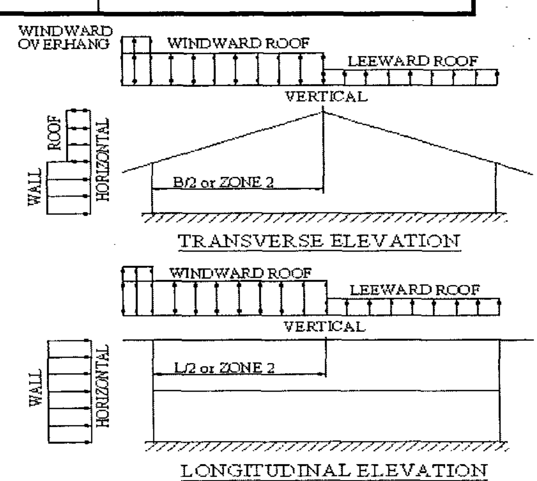
Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

Interior Zone: Wall 10.7 psf
Roof -5.0 psf
End Zone: Wall 16.2 psf
Roof -8.4 psf

Longitudinal direction (parallel to L)

Interior Zone: Wall 10.7 psf
End Zone: Wall 16.2 psf



Holben, Martin & White
2950 N. Country Club
Tucson, AZ
327-9491

JOB TITLE CFSD HS Kitchen and Performing Arts Expansion

JOB NO. 100048 SHEET NO. 7
CALCULATED BY DATE 3/11/11
CHECKED BY WW DATE 4/11/11

Wind Loads - Components & Cladding : h <= 60'

Kh (case 1) =	0.88	h =	18.0 ft
Base pressure (qh) =	**15.5 psf**	a =	3.2 ft
Minimum parapet ht =	3.0 ft	GCpi =	+/-0.18
Roof Angle =	0.0 deg		
Type of roof =	Monoslope		

Roof

	GCp +/- GCpi			Surface Pressure (psf)			**User input**	
Area	10 sf	50 sf	100 sf	10 sf	50 sf	100 sf	120 sf	500 sf
Negative Zone 1	-1.18	-1.11	-1.08	-18.3 psf	-17.3 psf	-16.8 psf	-16.8 psf	-16.8 psf
Negative Zone 2	-1.98	-1.49	-1.28	-30.8 psf	-23.2 psf	-19.9 psf	-19.9 psf	-19.9 psf
Negative Zone 3	-1.98	-1.49	-1.28	-30.8 psf	-23.2 psf	-19.9 psf	-19.9 psf	-19.9 psf
Positive Zone 1	0.48	0.41	0.38	10.0 psf	10.0 psf	10.0 psf	10.0 psf	10.0 psf
Positive Zones 2 & 3	1.08	0.97	1.05	16.8 psf	15.1 psf	16.4 psf	14.1 psf	12.6 psf
Overhang Zone 1&2	-1.70	-1.63	-1.60	-26.4 psf	-25.3 psf	-24.9 psf	-24.0 psf	-17.1 psf
Overhang Zone 3	-1.70	-1.63	-1.60	-26.4 psf	-25.3 psf	-24.9 psf	-24.0 psf	-17.1 psf

Negative zone 3 = zone 2, since parapet >= 3ft.
Overhang pressures in the table above assume an internal pressure coefficient (Gcpi) of 0.0

Parapet

qp = 16.1 psf

CASE A = pressure towards building
CASE B = pressure away from building

	Surface Pressure (psf)			**User input**
Solid Parapet Pressure	10 sf	100 sf	500 sf	40 sf
CASE A : Interior zone:	43.4 psf	29.6 psf	27.8 psf	35.1 psf
Corner zone:	43.4 psf	29.6 psf	27.8 psf	35.1 psf
CASE B : Interior zone:	-30.4 psf	-25.2 psf	-21.7 psf	-27.3 psf
Corner zone:	-34.7 psf	-27.0 psf	-21.7 psf	-30.1 psf

Walls

	GCp +/- GCpi			Surface Pressure at "h"			**User input**	
Area	10 sf	100 sf	500 sf	10 sf	100 sf	500 sf	50 sf	200 sf
Negative Zone 4	-1.17	-1.01	-0.90	-18.2 psf	-15.7 psf	-14.0 psf	-16.5 psf	-15.0 psf
Negative Zone 5	-1.44	-1.12	-0.90	-22.4 psf	-17.4 psf	-14.0 psf	-18.9 psf	-16.0 psf
Positive Zone 4 & 5	1.08	0.92	0.81	16.8 psf	14.3 psf	12.6 psf	15.1 psf	13.6 psf

Note: GCp reduced by 10% due to roof angle <= 10 deg.

MARTIN

HOLBEN & WHITE

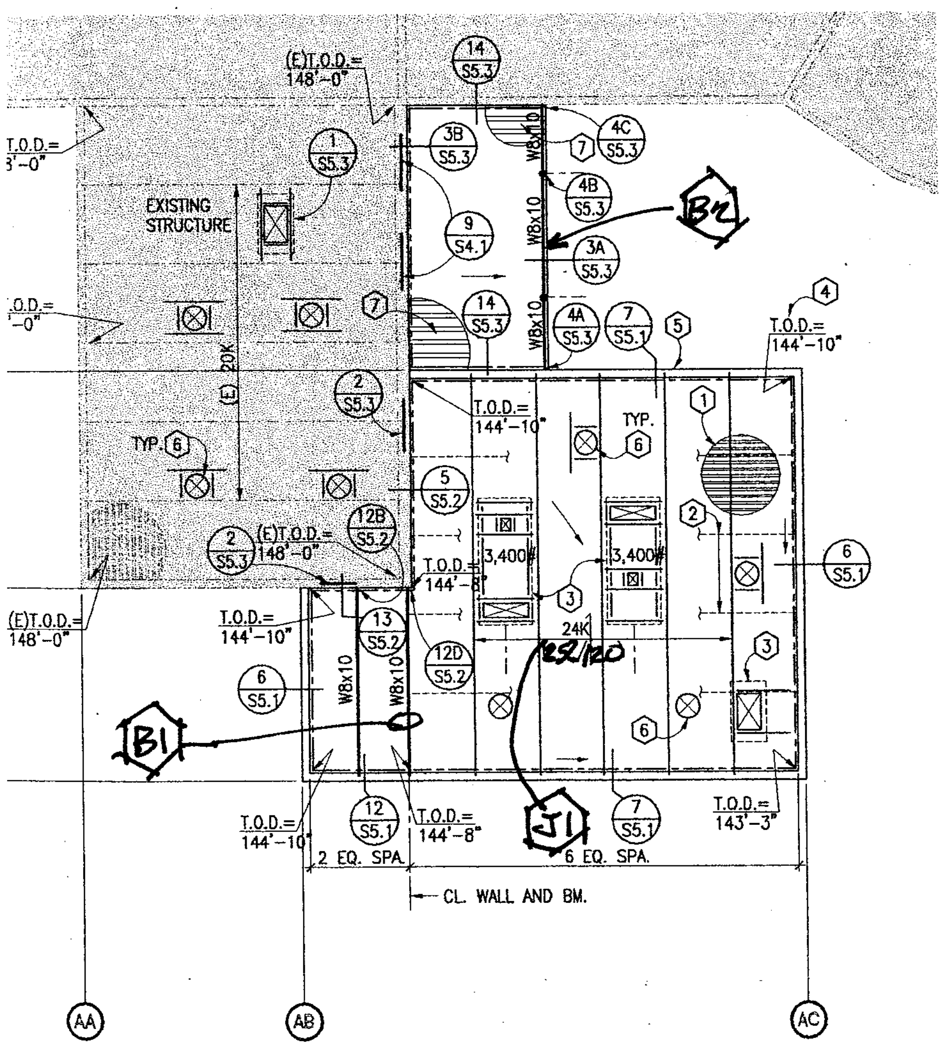
HOLBEN, MARTIN & WHITE ▫ CONSULTING STRUCTURAL ENGINEERS, INC.

Project CFSD Performing Arts - Bldg A   Sheet 8 of ____

Client NTD   Job no. ____

By WW

Roof Framing   Date 3/11



2 A BUILDING – ROOF FRAMING PLAN

1/8"=1'-0"

NOTE:
PRIOR TO FABRICATION OR CONSTRUCTION, FIELD VERIFY (FVD) EXISTING CONDITIONS.

NORTH

HOLBEN MARTIN & WHITE

HOLBEN, MARTIN & WHITE ▫ CONSULTING STRUCTURAL ENGINEERS, INC.

Project ~~GSD~~ Performing Arts - Bldg A Sheet 9 of ____

Client NTD Job no. ____

By ____

Roof framing Date ____

Roof Joists DL=22 LL=20

J1 max joist spacing = 6'-0" span = 33'

Total Load = 6(22+20) = 252 plf
Live Load = 6×20 = 120 plf

Use 24K joists (252/120)

Wind uplift = 20 psf

Use net uplift (20-) 6' = 12 plf use 80 plf

B1 span = 15' spacing = 5' D = 5×22 = .11 k/' L = 5×20 = 0.1 k/'

See Enercalc Output - use W8×10

B2 Canopy Bm span = 10

Gravity Loading w_D = 10 psf × 6' = 60 plf w_L = 20×6 = 120

Wind uplift = 6'× -20 psf = -0.12

Title Block Line 1
You can changes this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title :
Dsgnr:
Project Desc.:

Project Notes :

Job #
10

Printed: 30 MAR 2011, 11:44AM

Steel Beam

ENERCALC, INC. 1983-2011, Ver: 6.2.00

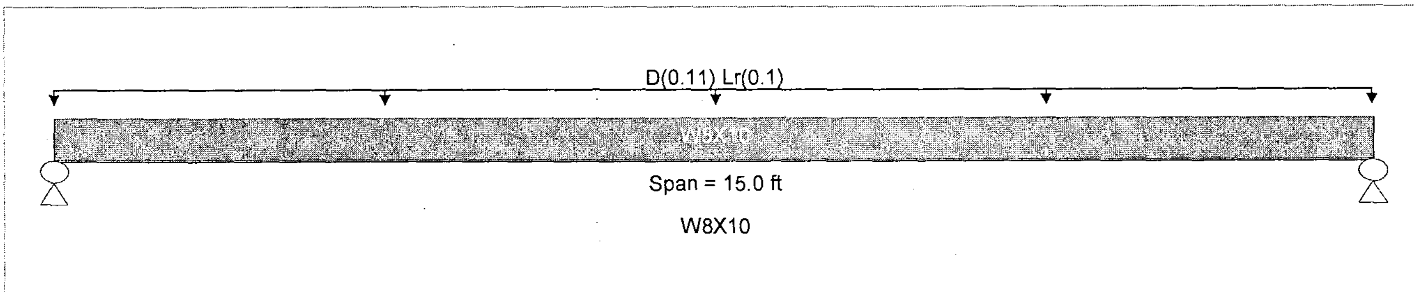
Lic. # : KW-06002641 **Licensee : HOLBEN, MARTIN, & WHITE**

Description : Performing Arts W8x10 roof beam B1

Material Properties

Calculations per AISC 360-05, ASCE 7-05

Analysis Method :	Allowable Stress Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsion buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		
Load Combination	2006 IBC & ASCE 7-05		



Applied Loads

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

Uniform Load : D = 0.110, Lr = 0.10 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.270** : 1	Maximum Shear Stress Ratio =	**0.059** : 1
Section used for this span	**W8X10**	Section used for this span	**W8X10**
Mu : Applied	5.906 k-ft	Vu : Applied	1.575 k
Mn / Omega : Allowable	21.870 k-ft	Vn/Omega : Allowable	26.826 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	7.500ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.129 in	Ratio =	1400
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.270 in	Ratio =	666
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 15.00 ft	1	0.270	0.059	5.91		5.91	36.52	21.87	1.00	1.00	1.58	40.24	26.83
+D													
Dsgn. L = 15.00 ft	1	0.141	0.031	3.09		3.09	36.52	21.87	1.00	1.00	0.83	40.24	26.83
+D+Lr+H													
Dsgn. L = 15.00 ft	1	0.270	0.059	5.91		5.91	36.52	21.87	1.00	1.00	1.58	40.24	26.83
+D+0.750Lr+0.750L+H													
Dsgn. L = 15.00 ft	1	0.238	0.052	5.20		5.20	36.52	21.87	1.00	1.00	1.39	40.24	26.83
+D+0.750Lr+0.750L+0.750W+H													
Dsgn. L = 15.00 ft	1	0.238	0.052	5.20		5.20	36.52	21.87	1.00	1.00	1.39	40.24	26.83
+D+0.750Lr+0.750L+0.5250E+H													
Dsgn. L = 15.00 ft	1	0.238	0.052	5.20		5.20	36.52	21.87	1.00	1.00	1.39	40.24	26.83

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+Lr	1	0.2699	7.575		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.575	1.575
D Only	0.825	0.825
Lr Only	0.750	0.750
D+Lr	1.575	1.575

Steel Beam

ENERCALC, INC. 1983-2011, Ver: 6.2.00

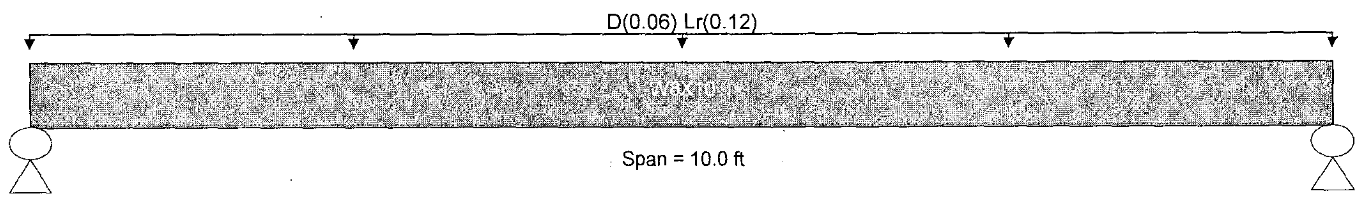
Lic. # : KW-06002641 **Licensee : HOLBEN, MARTIN, & WHITE**

Description : Performing Arts W8x10 Canopy Bm B2 Gravity Loads

Material Properties

Calculations per AISC 360-05, ASCE 7-05

Analysis Method :	Allowable Stress Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsion buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		
Load Combination	2006 IBC & ASCE 7-05		



Applied Loads

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

Uniform Load : D = 0.060, Lr = 0.120 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.103** : 1	Maximum Shear Stress Ratio =	**0.034** : 1
Section used for this span	**W8X10**	Section used for this span	**W8X10**
Mu : Applied	2.250 k-ft	Vu : Applied	0.90 k
Mn / Omega : Allowable	21.870 k-ft	Vn/Omega : Allowable	26.826 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	5.000 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.030 in	Ratio =	3938
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.046 in	Ratio =	2625
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 10.00 ft	1	0.103	0.034	2.25		2.25	36.52	21.87	1.00	1.00	0.90	40.24	26.83
+D													
Dsgn. L = 10.00 ft	1	0.034	0.011	0.75		0.75	36.52	21.87	1.00	1.00	0.30	40.24	26.83
+D+Lr+H													
Dsgn. L = 10.00 ft	1	0.103	0.034	2.25		2.25	36.52	21.87	1.00	1.00	0.90	40.24	26.83
+D+0.750Lr+0.750L+H													
Dsgn. L = 10.00 ft	1	0.086	0.028	1.88		1.88	36.52	21.87	1.00	1.00	0.75	40.24	26.83
+D+0.750Lr+0.750L+0.750W+H													
Dsgn. L = 10.00 ft	1	0.086	0.028	1.88		1.88	36.52	21.87	1.00	1.00	0.75	40.24	26.83
+D+0.750Lr+0.750L+0.5250E+H													
Dsgn. L = 10.00 ft	1	0.086	0.028	1.88		1.88	36.52	21.87	1.00	1.00	0.75	40.24	26.83

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+Lr	1	0.0457	5.050		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.900	0.900
D Only	0.300	0.300
Lr Only	0.600	0.600
D+Lr	0.900	0.900

Title Block Line 1
You can changes this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title :
Dsgnr:
Project Desc.:

Project Notes :

Job #
12

Printed: 30 MAR 2011, 1:09PM

Steel Beam

ENERCALC, INC. 1983-2011, Ver: 6.2.00

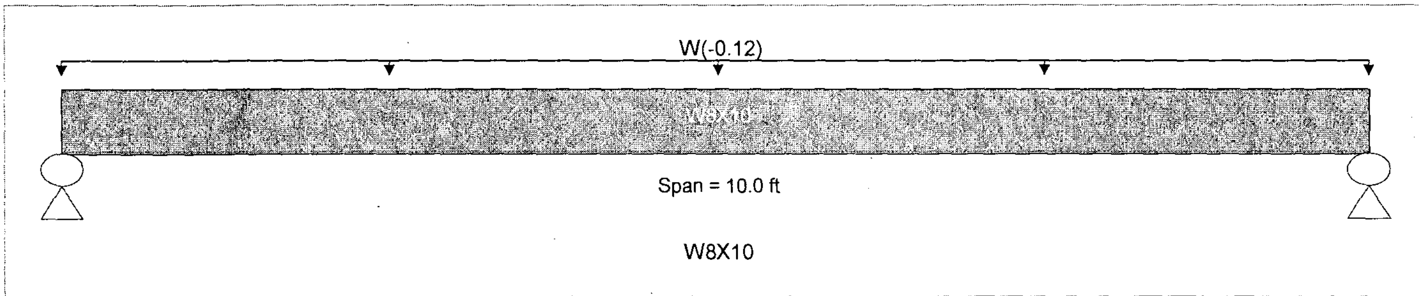
Lic. # : KW-06002641 **Licensee : HOLBEN, MARTIN, & WHITE**

Description : Performing Arts W8x10 Canopy Bm B2 Wind Uplift

Material Properties

Calculations per AISC 360-05, ASCE 7-05

Analysis Method :	Allowable Stress Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		
Load Combination	2006 IBC & ASCE 7-05		



Applied Loads

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

Uniform Load : W = -0.120 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.124** : 1	Maximum Shear Stress Ratio =	**0.022** : 1
Section used for this span	**W8X10**	Section used for this span	**W8X10**
Mu : Applied	1.500 k-ft	Vu : Applied	0.60 k
Mn / Omega : Allowable	12.063 k-ft	Vn/Omega : Allowable	26.826 k
Load Combination	+D+W+H	Load Combination	+D+W+H
Location of maximum on span	5.000ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.000 in	Ratio =	0 <240
Max Upward Total Deflection	-0.030 in	Ratio =	3938

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 10.00 ft	1	0.124	0.022		-1.50	1.50	20.14	12.06	1.14	1.00	0.60	40.24	26.83
+D+W+H													
Dsgn. L = 10.00 ft	1	0.124	0.022		-1.50	1.50	20.14	12.06	1.14	1.00	0.60	40.24	26.83
+D+0.750Lr+0.750L+0.750W+H													
Dsgn. L = 10.00 ft	1	0.093	0.017		-1.13	1.13	20.14	12.06	1.14	1.00	0.45	40.24	26.83
+D+0.750L+0.750S+0.750W+H													
Dsgn. L = 10.00 ft	1	0.093	0.017		-1.13	1.13	20.14	12.06	1.14	1.00	0.45	40.24	26.83
+0.60D+W+H													
Dsgn. L = 10.00 ft	1	0.124	0.022		-1.50	1.50	20.14	12.06	1.14	1.00	0.60	40.24	26.83

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.0305	5.050

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-0.600	-0.600
W Only	-0.600	-0.600

MARTIN HOLBEN & WHITE

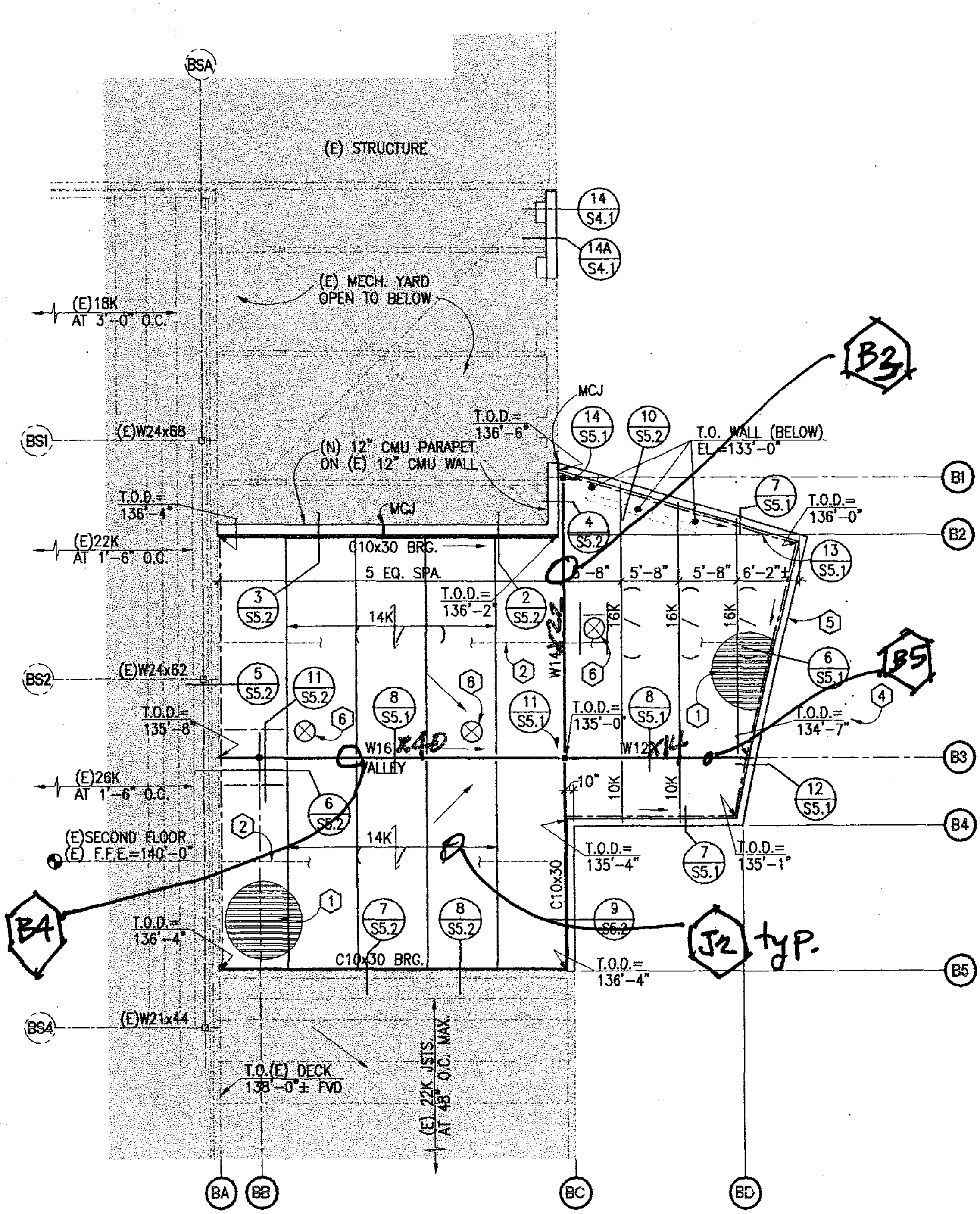
HOLBEN, MARTIN & WHITE ▫ CONSULTING STRUCTURAL ENGINEERS, INC.

Project CFSD Kitchen - Bldg B   Sheet 13 of

Client NTD   Job no.

By

Roof Frmg   Date



① B BUILDING – ROOF FRAMING PLAN
1/8"=1'-0"

NOTE:
PRIOR TO FABRICATION OR CONSTRUCTION, FIELD VERIFY (FVD) EXISTING CONDITIONS.

Roof Joists

J2 - Joists have same loading as J1

for spans 20' to 22' use 14K series (252/120)

for spans 26' use 16K (252/120)

spans ≤ 8' use 10K (252/120)

B3 - span = 27'

$w_D = 6' \times 22psf = .132^{k/l}$ $w_L = 6 \times 20 = 0.12^{k/l}$

See enercalc - USE W14x22

B4 span = 30' $w_D = 21' \times 22psf = 0.46^{k/l}$ $w_L = 20 \times 21' = 0.42^{k/l}$

See enercalc - USE W16x40

B5 - span = 18' $w_D = 16 \times 22psf = 0.35^{k/l}$ $w_L = 16 \times 20 = 0.32$

Title Block Line 1 You can changes this area using the "Settings" menu item and then using the "Printing & Title Block" selection. Title Block Line 6	Title : Dsgnr: Project Desc.: Project Notes :	Job # 15 Printed: 30 MAR 2011, 1:29PM

Steel Beam

ENERCALC, INC. 1983-2011, Ver: 6.2.00

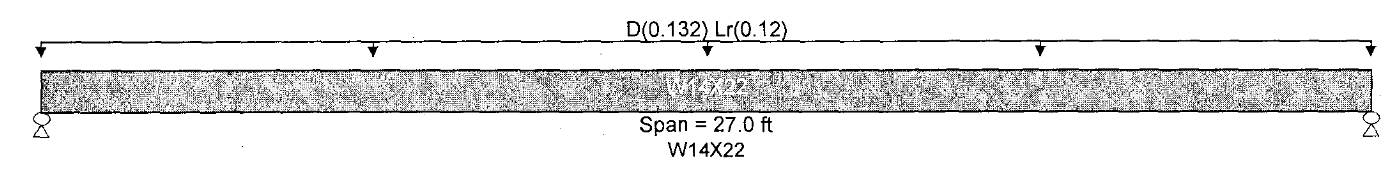
Lic. # : KW-06002641 **Licensee : HOLBEN, MARTIN, & WHITE**

Description : Kitchen W14x22 roof beam B3

Material Properties

Calculations per AISC 360-05, ASCE 7-05

Analysis Method :	Allowable Stress Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsion buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		
Load Combination	2006 IBC & ASCE 7-05		



Applied Loads

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

Uniform Load : D = 0.1320, Lr = 0.120 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.277** : 1	Maximum Shear Stress Ratio =	**0.054** : 1
Section used for this span	**W14X22**	Section used for this span	**W14X22**
Mu : Applied	22.964 k-ft	Vu : Applied	3.402 k
Mn / Omega : Allowable	82.834 k-ft	Vn/Omega : Allowable	63.020 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	13.500ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.251 in	Ratio =	1292
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.526 in	Ratio =	615
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 27.00 ft	1	0.277	0.054	22.96		22.96	138.33	82.83	1.00	1.00	3.40	94.53	63.02
+D													
Dsgn. L = 27.00 ft	1	0.145	0.028	12.03		12.03	138.33	82.83	1.00	1.00	1.78	94.53	63.02
+D+Lr+H													
Dsgn. L = 27.00 ft	1	0.277	0.054	22.96		22.96	138.33	82.83	1.00	1.00	3.40	94.53	63.02
+D+0.750Lr+0.750L+H													
Dsgn. L = 27.00 ft	1	0.244	0.048	20.23		20.23	138.33	82.83	1.00	1.00	3.00	94.53	63.02
+D+0.750Lr+0.750L+0.750W+H													
Dsgn. L = 27.00 ft	1	0.244	0.048	20.23		20.23	138.33	82.83	1.00	1.00	3.00	94.53	63.02
+D+0.750Lr+0.750L+0.5250E+H													
Dsgn. L = 27.00 ft	1	0.244	0.048	20.23		20.23	138.33	82.83	1.00	1.00	3.00	94.53	63.02

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+Lr	1	0.5263	13.635		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.402	3.402
D Only	1.782	1.782
Lr Only	1.620	1.620
D+Lr	3.402	3.402

Title Block Line 1
You can changes this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title :
Dsgnr:
Project Desc.:

Project Notes :

Job #
16

Printed: 30 MAR 2011, 1:51PM

Steel Beam

ENERCALC, INC. 1983-2011, Ver: 6.2.00

Lic. # : KW-06002641 **Licensee : HOLBEN, MARTIN, & WHITE**

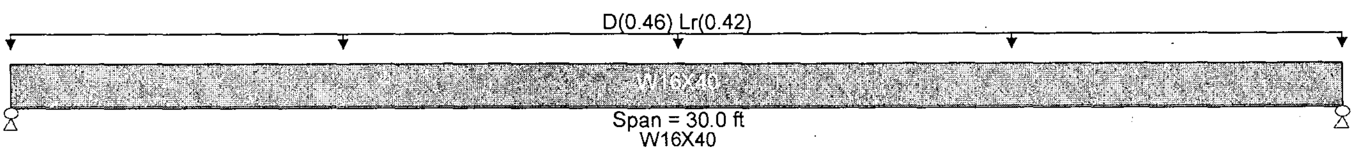
Description : Kitchen W16x40 roof beam B4

Material Properties

Calculations per AISC 360-05, ASCE 7-05

Analysis Method : Allowable Stress Design
Beam Bracing : Beam is Fully Braced against lateral-torsion buckling
Bending Axis : Major Axis Bending
Load Combination 2006 IBC & ASCE 7-05

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

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

Uniform Load : D = 0.460, Lr = 0.420 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.544** : 1	Maximum Shear Stress Ratio =	**0.135** : 1
Section used for this span	**W16X40**	Section used for this span	**W16X40**
Mu : Applied	99.000 k-ft	Vu : Applied	13.20 k
Mn / Omega : Allowable	182.136 k-ft	Vn/Omega : Allowable	97.60 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	15.000 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.514 in	Ratio =	700
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	1.076 in	Ratio =	334
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 30.00 ft	1	0.544	0.135	99.00		99.00	304.17	182.14	1.00	1.00	13.20	146.40	97.60
+D													
Dsgn. L = 30.00 ft	1	0.284	0.071	51.75		51.75	304.17	182.14	1.00	1.00	6.90	146.40	97.60
+D+Lr+H													
Dsgn. L = 30.00 ft	1	0.544	0.135	99.00		99.00	304.17	182.14	1.00	1.00	13.20	146.40	97.60
+D+0.750Lr+0.750L+H													
Dsgn. L = 30.00 ft	1	0.479	0.119	87.19		87.19	304.17	182.14	1.00	1.00	11.63	146.40	97.60
+D+0.750Lr+0.750L+0.750W+H													
Dsgn. L = 30.00 ft	1	0.479	0.119	87.19		87.19	304.17	182.14	1.00	1.00	11.63	146.40	97.60
+D+0.750Lr+0.750L+0.5250E+H													
Dsgn. L = 30.00 ft	1	0.479	0.119	87.19		87.19	304.17	182.14	1.00	1.00	11.63	146.40	97.60

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+Lr	1	1.0762	15.150		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	13.200	13.200
D Only	6.900	6.900
Lr Only	6.300	6.300
D+Lr	13.200	13.200

Steel Beam

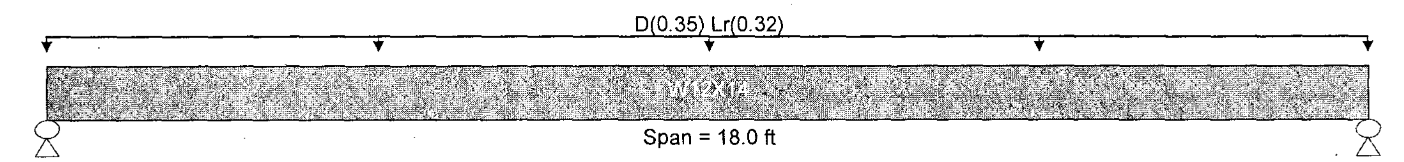
Lic. # : KW-06002641 **Licensee : HOLBEN, MARTIN, & WHITE**

Description : Kitchen W12x14 roof beam B5

Material Properties

Calculations per AISC 360-05, ASCE 7-05

Analysis Method :	Allowable Stress Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsion buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		
Load Combination	2006 IBC & ASCE 7-05		



Applied Loads

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

Uniform Load : D = 0.350, Lr = 0.320 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.625** : 1	Maximum Shear Stress Ratio =	**0.141** : 1
Section used for this span	**W12X14**	Section used for this span	**W12X14**
Mu : Applied	27.135 k-ft	Vu : Applied	6.030 k
Mn / Omega : Allowable	43.413 k-ft	Vn/Omega : Allowable	42.754 k
Load Combination	+D+Lr+H	Load Combination	+D+Lr+H
Location of maximum on span	9.000ft	Location of maximum on span	18.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.297 in	Ratio =	728
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.621 in	Ratio =	347
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 18.00 ft	1	0.625	0.141	27.14		27.14	72.50	43.41	1.00	1.00	6.03	71.40	42.75
+D													
Dsgn. L = 18.00 ft	1	0.327	0.074	14.18		14.18	72.50	43.41	1.00	1.00	3.15	71.40	42.75
+D+Lr+H													
Dsgn. L = 18.00 ft	1	0.625	0.141	27.14		27.14	72.50	43.41	1.00	1.00	6.03	71.40	42.75
+D+0.750Lr+0.750L+H													
Dsgn. L = 18.00 ft	1	0.550	0.124	23.90		23.90	72.50	43.41	1.00	1.00	5.31	71.40	42.75
+D+0.750Lr+0.750L+0.750W+H													
Dsgn. L = 18.00 ft	1	0.550	0.124	23.90		23.90	72.50	43.41	1.00	1.00	5.31	71.40	42.75
+D+0.750Lr+0.750L+0.5250E+H													
Dsgn. L = 18.00 ft	1	0.550	0.124	23.90		23.90	72.50	43.41	1.00	1.00	5.31	71.40	42.75

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+Lr	1	0.6208	9.090		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.030	6.030
D Only	3.150	3.150
Lr Only	2.880	2.880
D+Lr	6.030	6.030

MARTIN

HOLBEN & WHITE

HOLBEN, MARTIN & WHITE ▫ CONSULTING STRUCTURAL ENGINEERS, INC.

Project CFSD Kitchen - Bldg B Sheet 18 of

Client NTD Job no.

By

Col. Design Date

Col C1 $DL = 22 psf$ $LL = 20$ $l_u = 13'$

$P_D = 24' \times 18' \times 22 psf = 9.5^k$ $P_L = 24 \times 18 \times 20 = 8.6$

See Enercalc - Use HSS 4×4×1/4

Note: This column is the most heavily loaded column in the bldg and at the canopy, therefore use same column thru-out.

Steel Column

Description : Building B Column C1 HSS 4x4x1/4

General Information

Calculations per AISC 360-05, ASCE 7-05

Steel Section Name : **HSS4X4X1/4**
Analysis Method : 2006 IBC & ASCE 7-05
Steel Stress Grade
Fy : Steel Yield 46.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi
Load Combination : Allowable Stress

Overall Column Height 13.0 ft
Top & Bottom Fixity Top & Bottom Pinned

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 13 ft, K = 1.0
Y-Y (depth) axis :Fully braced against buckling along Y-Y Axis

Applied Loads

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

Column self weight included : 158.35 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 13.0 ft, Xecc = 3.500 in, D = 9.50, LR = 8.60 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.8320** : 1

Load Combination	+D+Lr+H
Location of max.above base	12.913 ft
At maximum location values are . . .	
Pu : Axial	18.258 k
Pn / Omega : Allowable	45.762 k
Mu-x : Applied	0.0 k-ft
Mn-x / Omega : Allowable	10.765 k-ft
Mu-y : Applied	5.244 k-ft
Mn-y / Omega : Allowable	10.765 k-ft

PASS Maximum Shear Stress Ratio = **0.01597** : 1

Load Combination	+D+Lr+H
Location of max.above base	0.0 ft
At maximum location values are . . .	
Vu : Applied	0.4061 k
Vn / Omega : Allowable	25.423 k

Maximum SERVICE Load Reactions . .

Top along X-X	0.4061 k
Bottom along X-X	0.4061 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum SERVICE Load Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination :

Along X-X 0.4410 in at 7.591 ft above base
for load combination : D+Lr

Load Combination Results

	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	Stress Ratio	Status	Location	Stress Ratio	Status	Location
+D	0.438	PASS	12.91 ft	0.008	PASS	0.00 ft
+D+Lr+H	0.832	PASS	12.91 ft	0.016	PASS	0.00 ft
+D+0.750Lr+0.750L+H	0.734	PASS	12.91 ft	0.014	PASS	0.00 ft
+D+0.750Lr+0.750L+0.750W+H	0.734	PASS	12.91 ft	0.014	PASS	0.00 ft
+D+0.750Lr+0.750L+0.5250E+H	0.734	PASS	12.91 ft	0.014	PASS	0.00 ft

Maximum Reactions - Unfactored

Note: Only non-zero reactions are listed.

	X-X Axis Reaction		Y-Y Axis Reaction		Axial Reaction
Load Combination	@ Base	@ Top	@ Base	@ Top	@ Base
D Only	-0.213	-0.213 k		k	9.658 k
Lr Only	-0.193	-0.193 k		k	8.600 k
D+Lr	-0.406	-0.406 k		k	18.258 k

Maximum Deflections for Load Combinations - Unfactored Loads

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.2315 in	7.591 ft	0.000 in	0.000 ft
Lr Only	0.2095 in	7.591 ft	0.000 in	0.000 ft
D+Lr	0.4410 in	7.591 ft	0.000 in	0.000 ft

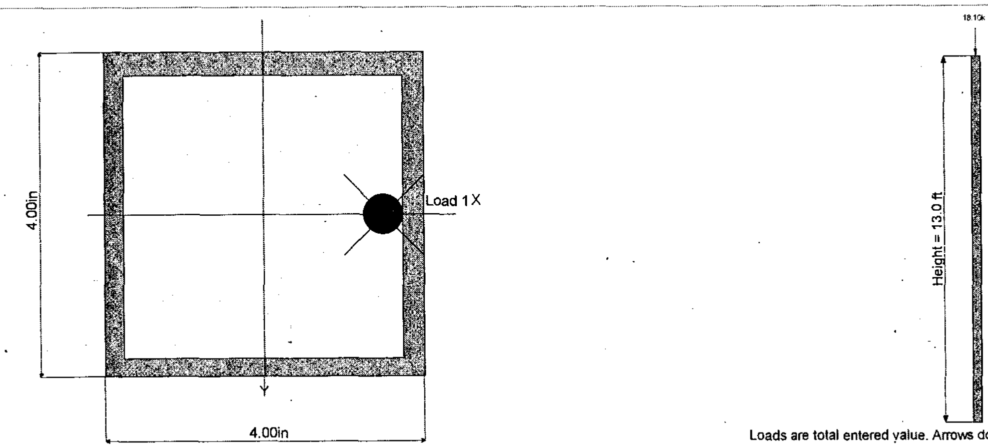
Steel Section Properties : HSS4X4X1/4

Steel Column

Description : Building B Column C1 HSS 4x4x1/4

Steel Section Properties : HSS4X4X1/4

Depth	=	4.000	in	I xx	=	7.80	in^4	J	=	12.800	in^4
Web Thick	=	0.000	in	S xx	=	3.90	in^3				
Flange Width	=	4.000	in	R xx	=	1.520	in				
Flange Thick	=	0.250	in								
Area	=	3.370	in^2	I yy	=	7.800	in^4				
Weight	=	12.181	plf	S yy	=	3.900	in^3				
				R yy	=	1.520	in				
Ycg	=	0.000	in								



Loads are total entered value. Arrows do not reflect absolute direction.

MARTIN HOLBEN & WHITE

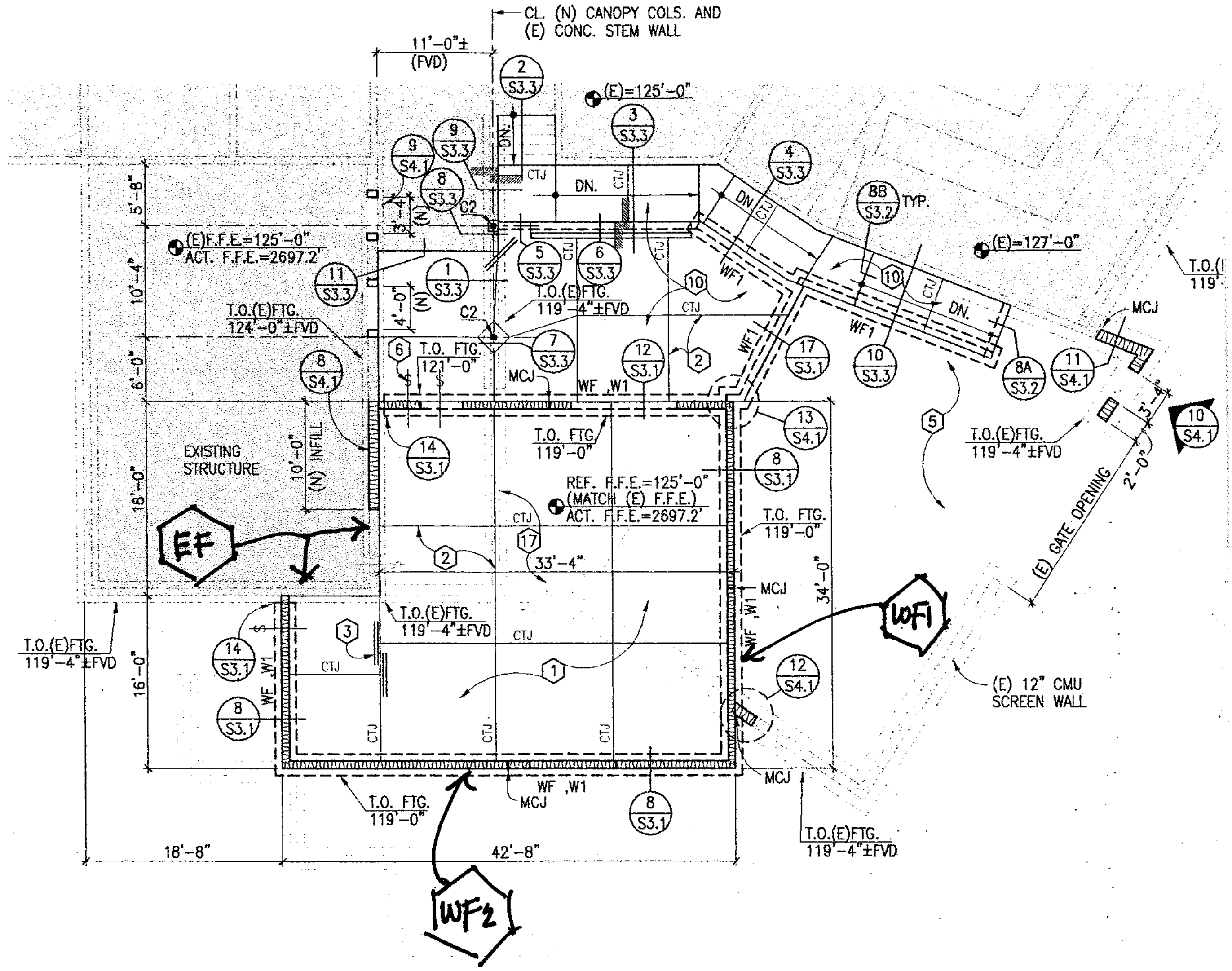
HOLBEN, MARTIN & WHITE ▫ CONSULTING STRUCTURAL ENGINEERS, INC.

Project CFSD HS Performing Arts - Bldg A Sheet 21 of

Client NTD Job no.

By WW

Foundations Date 3/11



2 A BUILDING – FOUNDATION PLAN

1/8"=1'-0"

NOTE:
PRIOR TO FABRICATION OR CONSTRUCTION, FIELD VERIFY (FVD) EXISTING CONDITIONS.

NORTH

Soils Report: Sergent, Hawkins & Beckwith
Report No. 591-6019, July 8, 1991

Existing site was prepared utilizing engineered fill with an allowable brg pressure of 3000 psf @ 1'-6" depth.

Since the new building connects to the existing structure, to minimize potential differential settlement, use an allowable bearing of 2000 psf.

WF2 cont. wall ftg

T.O. Parapet = 148'-4"
T.O. Ftg = 119-0"
} wall ht = 29.33' (8" cmu)

Wall DL = 29.33' x 85 psf = 2500 plf
Roof D+L = 42 psf x 17' = 700
Mech Units = (2x3500x1/2)/16' = 250
3450 plf

$W = \frac{3450}{2000} = 2'$ use 2'-6" wide cont ftg

WF1 check @ non-brg wall

Wall DL = 29.33' x 85 psf = 2500
D+L = 3' x 42 psf = 150
2650

$W = \frac{2650}{2000} = 1.3'$ use 2'-0" wide cont. ftg

EF check existing footing

T.O. parapet = 149'-4"
T.O. ftg = 119'-4"
} 30' (8" cmu)

Wall DL = 30' x 80psf = 2400 plf
(e) Roof D+L = 42psf x 13' = 550
(n) Roof D+L = 42psf x 3' = 120
3070 plf

existing footing width = 2'-0"

$q = \frac{3070}{2'-0"} = 1535 psf < 2000$ ok

MARTIN

HOLBEN & WHITE

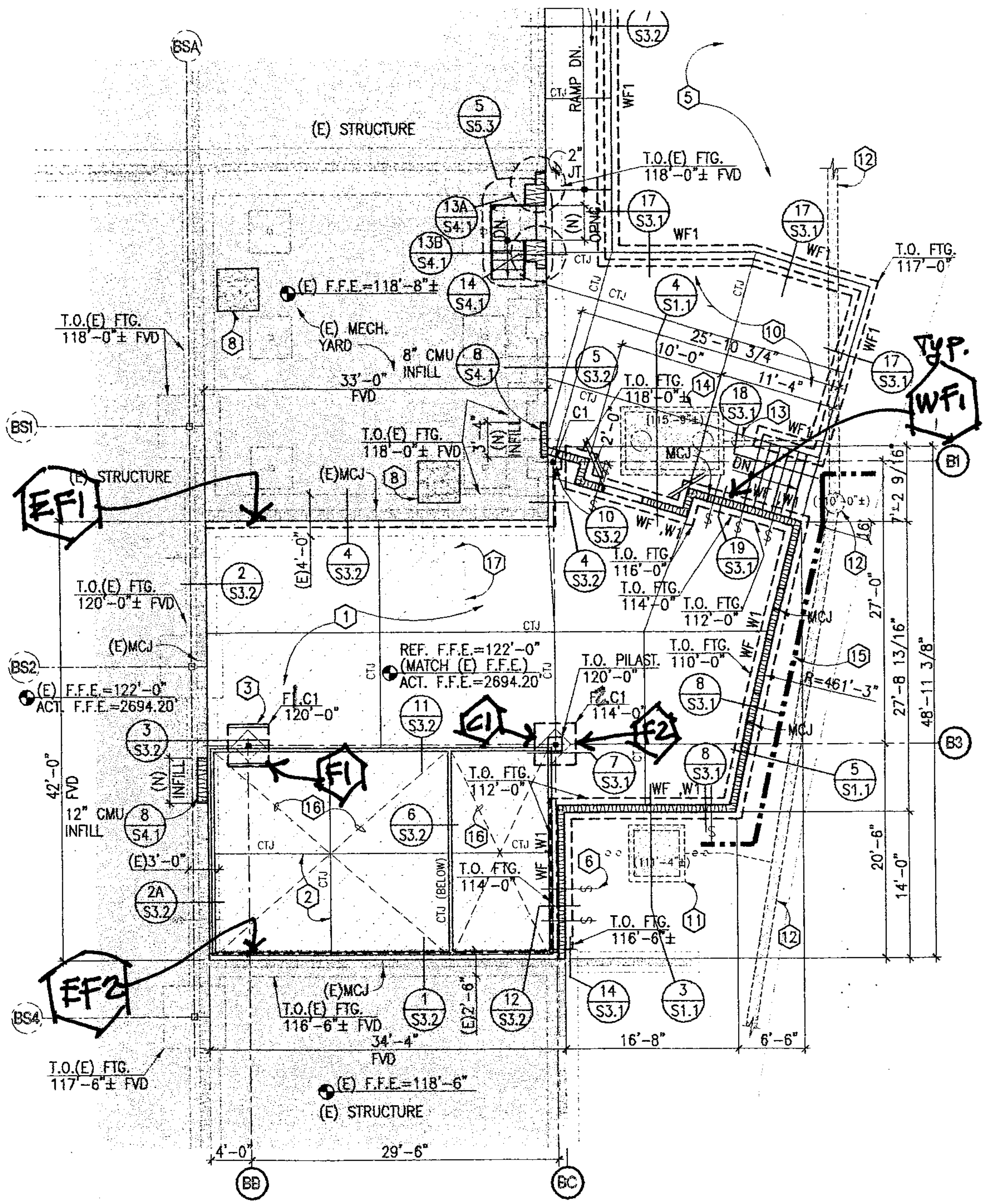
HOLBEN, MARTIN & WHITE ▫ CONSULTING STRUCTURAL ENGINEERS, INC.

Project Kitchen - Bldg B | Sheet 24 of

Client NTD | Job no.

By WW

Foundations & Column Plan | Date 3/11



1 B BUILDING – FOUNDATION PLAN
1/8"=1'-0"

NOTE:
PRIOR TO FABRICATION OR CONSTRUCTION, FIELD VERIFY (FVD) EXISTING CONDITIONS.

NORTH

Soils Report: Sergent, Hawkins & Beckwith
Report No. 591-6019, July 8, 1991

Existing site was prepared utilizing engineered fill with an allowable pressure $q_a = 3000$ psf @ 1'-6" depth.

Since the new buildings of this project connect to the existing structure, to minimize potential differential settlement use $q_a = 2000$ psf.

F1 $P = (20+22 \text{ psf}) \times 21' \times 20' = 17.6^k$

$W = \sqrt{\frac{17.6}{2}} = 3'$ use 4'-0" sq × 12" thick w/ 5#5

F2 $P = 42\text{psf}\left[(21\times15)+(9\times16)\right] = 19.3^k$

$W = \sqrt{\frac{19.3}{2}} = 3.2'$ Use 4'-0" sq × 12" thick w/ 5#5

WF1 wall ftg @ bug wall

T.O. Parapet = 139'-4"
T.O. Ftg = 116-0 } wall ht = 23.33' (8" cmu)

Wall DL = 23.33 × 80 psf avg = 1870 plf
Roof D+L = 42 psf × 12' = 510
2380 plf

$W = \frac{2380}{2000} = 1.19'$ use 2'-0" wide cont. Ftg

EF1 existing wall ftg check

T.O. parapet = 139'-4"
T.O. Ftg = 118'-0" } wall ht = 21.33' (12" cmu

Wall DL = 21.33' × 105 psf = 2250 plf
Roof D+L = 42 psf × 11' = 470
2720 plf

(e) ftg width = 3'-6"

$q = \frac{2720}{3.5'} = 780\,psf << 2\,ksf \therefore$ OK

EF2 existing wall ftg check

T.O. parapet = 143.33'
T.O. ftg = 116.50'
} 26.83' (8" cmu)

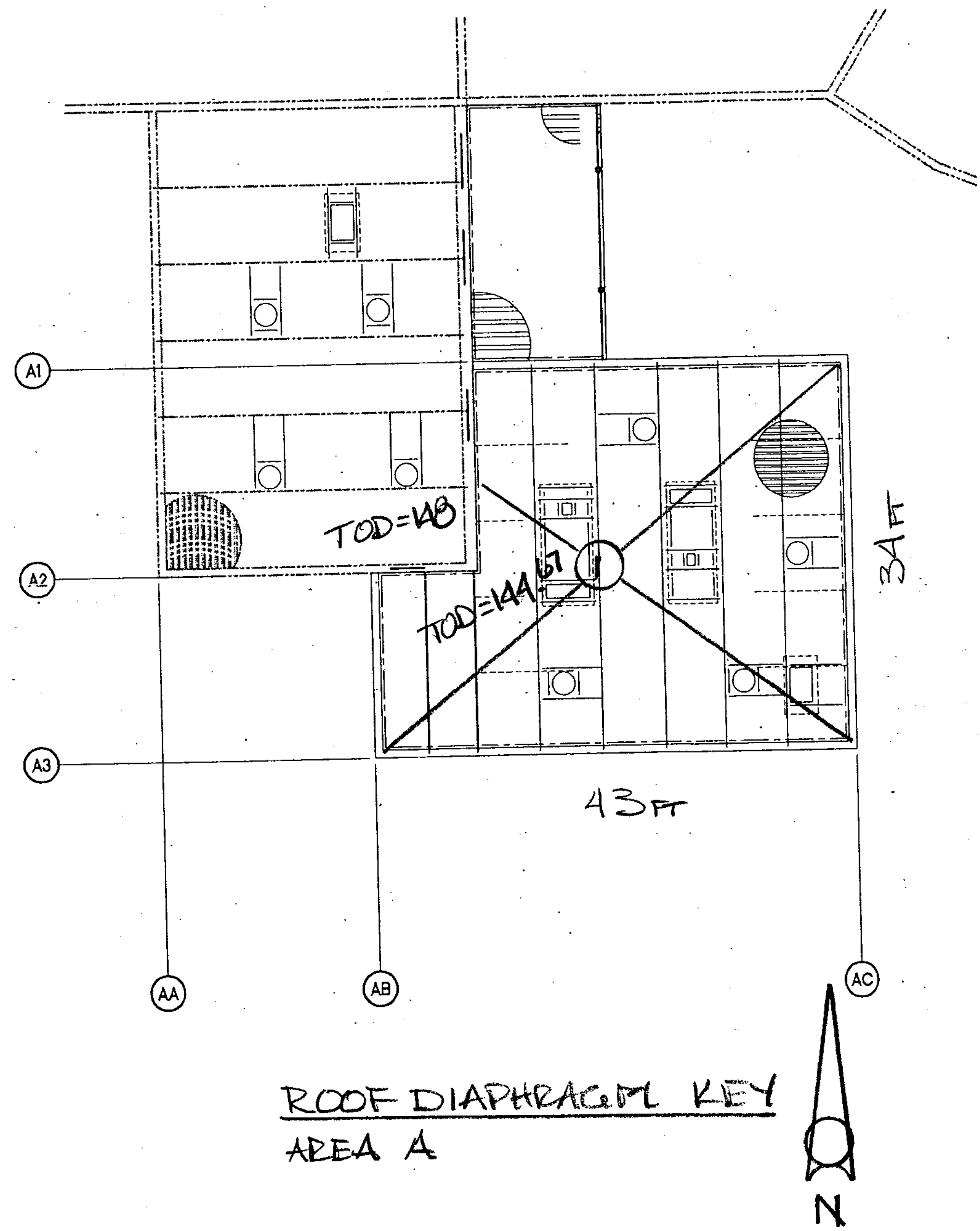
Wall DL = 26.83' × 75psf = 2000 plf
Roof D+L = 42psf × 12' = 500
2500 plf

(e) ftg width = 2'-6"

$q = \frac{2500}{2.5'} = 1000\,psf < 2000 \therefore \underline{OK}$

CFSD HS KIT. EXP
10048

28



AREA 'A' ROOF DIAPHRAGM

WIDTH = 34 FT
LENGTH = 43 FT

WALL HT. = 144'-10" − 125'-0" = 19.83 FT → SAY 20 FT
PAR. HT = 4 FT

N-S DIRECTION:

WIND - $A_T = (20/2 + 4)(43\text{ FT}) = 602\text{ FT}^2$

$A_{TEZa} = (20/2)(6.4\text{ FT}) = 64\text{ FT}^2$

$A_{TEPAR} = (4)(43) = 172\text{ FT}^2$

$F_W = (602 - 64 - 172)(11\text{ PSF}) = 4026$

$+ (64)(17\text{ PSF}) = 1088\text{ LB}$

$+ (172)(25\text{ PSF}) = 4300\text{ LB}$

$9414\text{ LB} = 9.41\text{ K}$

SEISMIC - ROOF = $(34)(43)(0.022) = 32.2\text{ K}$

S.WALL = $(20/2 + 4)(43)(0.075) = 45.2\text{ K}$

N.WALL = $(20/2 + 4)(43)(0.075) = 45.2\text{ K}$

122.6 K

$F_S = (0.085)(122.6) = 10.4\text{ K}$ (STRENGTH)

$= 7.29\text{ K}$ (SERVICE)

E-W DIRECTION:

WIND - $A_T = (20/2 + 4)(34) = 476\text{ FT}^2$

$A_{TEZa} = (20/2)(6.4) = 64\text{ FT}^2$

$A_{TEPAR} = (4)(34) = 136\text{ FT}^2$

$F_W = (476 - 64 - 136)(11\text{ PSF}) = 3036\text{ LB}$

$+ (64)(17\text{ PSF}) = 1088\text{ LB}$

$+ (136)(25\text{ PSF}) = 3400\text{ LB}$

$7524\text{ LB} = 7.52\text{ K}$

AREA 'A' ROOF DIAPHRAGM (CONT.)

SEISMIC: ROOF = $(34)(43)(0.022) = 32.2_k$
E.WALL = $(20/2+4)(34)(0.075) = 35.7_k$
W.WALL = $(20/2+4)(34)(0.075) = 35.7_k$
103.6_k

$F_s = (0.085)(103.6_k) = 8.81_k$ (STRENGTH)
6.16_k (SERVICE)

DIAPHRAGM SHEAR:

N-S DIR: $\omega_{NS} = \dfrac{9.41_k/2}{34} = 0.138_{KLF}$

ANCHOR @ RE-ENTRANT CORNER:
$F_A = 0.138_{KLF}(16_{FT}) = 2.2_k$
5/8"ϕ EPOXY ANCHOR HAS 1.7_k CAP, 4 PROV.

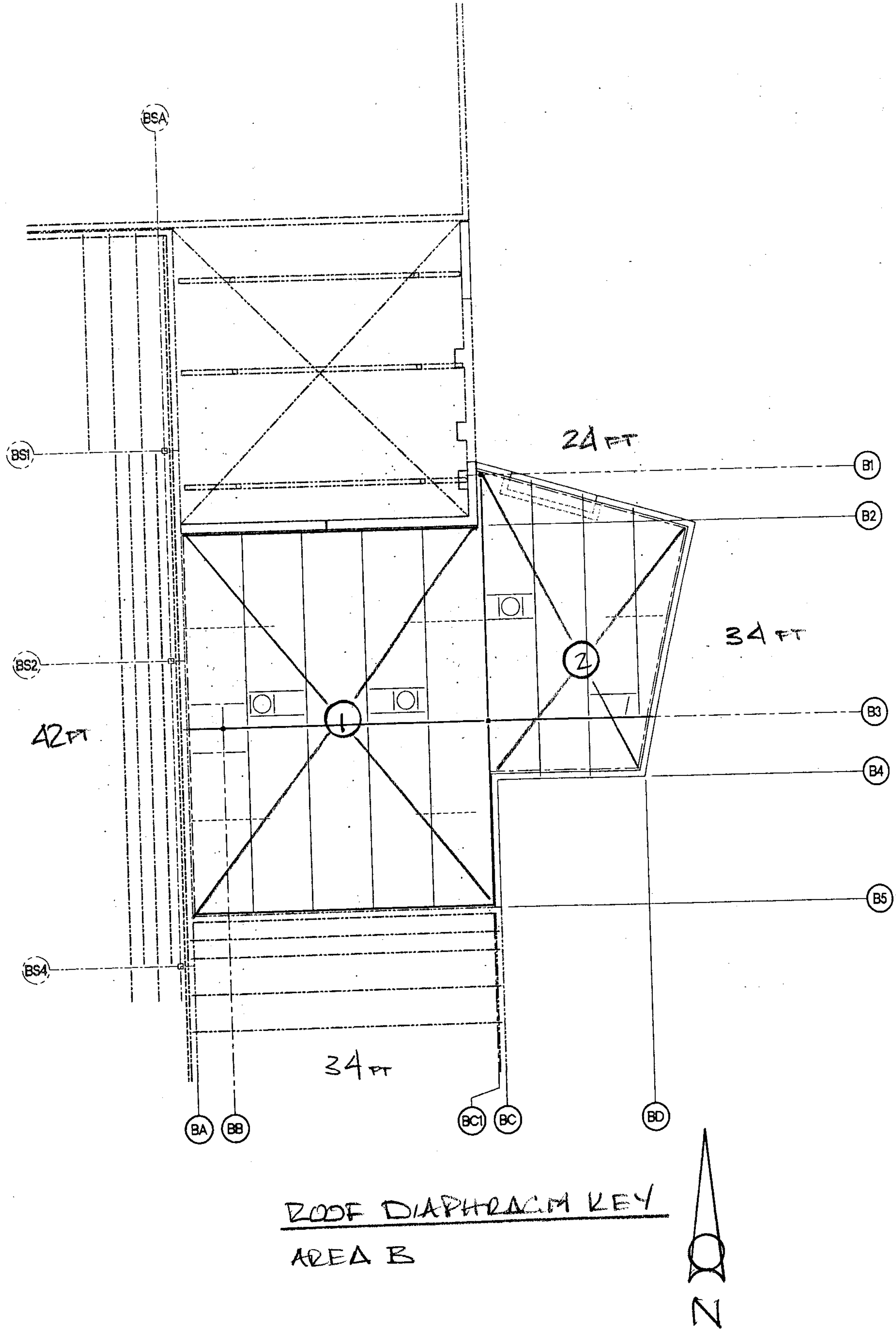
E-W DIR: $\omega_{EW} = \dfrac{7.52_k/2}{43\text{-}10} = 0.114_{KLF}$

1½" x 20GA DECK HAS 0.568_{KLF} CAP (5 WELD, BP@12")

DIAPHRAGM CHORD:

N-S DIR CONTROLS – $\omega_{NS} = 10.4_k/(43\text{-}10_{FT}) = 0.315_{KLF}$
$M_{DIA} = 0.315(43)^2/8 = 72.8_{k\text{-}FT}$
$T = C = 72.8/34 = 2.14_k$
$A_{SREQ} = \dfrac{2.14}{(0.9)(60_{KSI})} = 0.0397_{IN^2} < 0.61_{IN^2}$

→ USE (2) #5 BARS IN BOND BM @ ROOF



ROOF DIAPHRAGM KEY

AREA B

AREA 'B' ROOF DIAPHRAGM

PORTION ①: WIDTH = 34 FT, LENGTH = 42 FT

WALL HT = 14 FT
PAR HT = 4 FT
@ EXIST BLDG, WEST WALL
2 STORY, ∴ TRIB IS 18/2 = 9 FT
WT = 0.100 KSF
@ EXIST BLDG, NORTH WALL
HT = 15 FT, PAR = 7 FT
WT = 0.100 KSF

N-S DIRECTION:

WIND - $A_T = (14/2 + 4)(34_{FT}) = 374_{FT^2}$
$A_{TE2a} = (14/2)(6.4_{FT}) = 45_{FT^2}$
$A_{T@PAR} = (4)(34) = 136_{FT^2}$

$F_W = (374 - 45 - 136)(11_{PSF}) = 2123_{LB}$
$+ (45)(17_{PSF}) = 765_{LB}$
$+ (136)(25_{PSF}) = 3400_{LB}$
$6288_{LB} = 6.29_K$

SEISMIC - ROOF = $(34)(42)(0.022) = 31.4_K$
S.WALL = $(7_{FT})(34_{FT})(0.100) = 30.6_K$
N.WALL = $(15/2 + 7)(34_{FT})(0.100) = 47.6_K$
109.6_K

$F_S = (0.085)(109.6) = 9.32_K$ (STRENGTH)
6.52_K (SERVICE)

E-W DIRECTION:

SEISMIC - ROOF = $(34)(42)(0.022) = 31.4_K$
E.WALL = $(14/2 + 4)(14_{FT})(0.075) = 14.7_K$
W.WALL = $(9)(42_{FT})(0.100) = 37.8_K$
83.9_K

$F_S = (0.085)(83.9_K) = 7.13_K$ (STRENGTH) 5.0_K (SERVICE)

AREA 'B' ROOF DIAPHRAGM (CONT).

PORTION ②: WIDTH = 24 FT WALL HT = 14 FT
LENGTH = 34 FT PAR HT = 4 FT

N-S DIRECTION:

WIND - $A_T = (14/2 + 4)(24\,FT) = 264\,FT^2$
$A_{T@Z_a} = (14/2)(6.4) = 45\,FT^2$
$A_{T@PAR} = (4)(24) = 96\,FT^2$

$F_W = (264 - 45 - 96)(11\,PSF) = 1353\,LB$
$+ (45)(17\,PSF) = 765\,LB$
$+ (96)(25\,PSF) = 2400\,LB$
$4518\,LB = 4.52\,K$

SEISMIC - ROOF $= (24)(34)(0.022) = 18.0\,K$
S. WALL $= (14/2 + 4)(24)(0.075) = 19.8\,K$
N. WALL $= (14/2 + 4)(24)(0.075) = 19.8\,K$
$57.6\,K$

$F_S = (0.085)(57.6) = 4.90\,K$ (STRENGTH)
$3.43\,K$ (SERVICE)

E-W DIRECTION

WIND - $A_T = (14/2 + 4)(34\,FT) = 374\,FT^2$
$A_{T@Z_a} = (14/2)(6.4) = 45\,FT^2$
$A_{T@PAR} = (4)(34) = 136\,FT^2$

$F_W = (374 - 45 - 136)(11\,PSF) = 2123\,LB$
$+ (45)(17\,PSF) = 765\,LB$
$+ (136)(25\,PSF) = 3400\,LB$
$6288 = 6.29\,K$
← CONTROLS

AREA 'B' ROOF DIAPHRAGM (CONT)

DIAPHRAGM SHEAR

CONTROLLING LOAD WILL BE INTERSECTION OF ① & ②:

$$w_{NS} = \frac{6.52/2}{42_{FT}} + \frac{4.52/2}{34_{FT}} = 0.078_{KLF} + 0.066_{KLF} = 0.144_{KLF}$$

DIA. IS ADEQUATE, CHECK BEAM CONNECTION:

$F_A = 0.144_{KLF}(27_{FT}) = 3.90_{KIP}$

→ W14x22 w/ $L_U = 21$ FT IS ADEQUATE FOR 4K LOAD

NORTH CONN. HAS A.B. @ 16" O.C. w/ CAP OF 1.7K EA (5 TOTAL) ∴ ADEQUATE

SOUTH CONN. HAS C10 BOLTED TO WALL w/ (2) BOLTS @ 32" O.C. (10 TOTAL) ∴ ADEQUATE

→ USE 1½" x 20 GA. DECK w/ 5 WELD PATT. & B.P. @ 12" O.C.

DIAPHRAGM CHORD

AGAIN, CONTROLLING LOAD WILL BE INT. OF ① & ②:

$$w_{EW} = \frac{7.13_K}{42_{FT}} + \frac{6.29_K}{34_{FT}} = 0.170 + 0.185 = 0.355\ KLF$$

$M_{DIA} = 0.355(42)^2/8 = 78.3$ K-FT

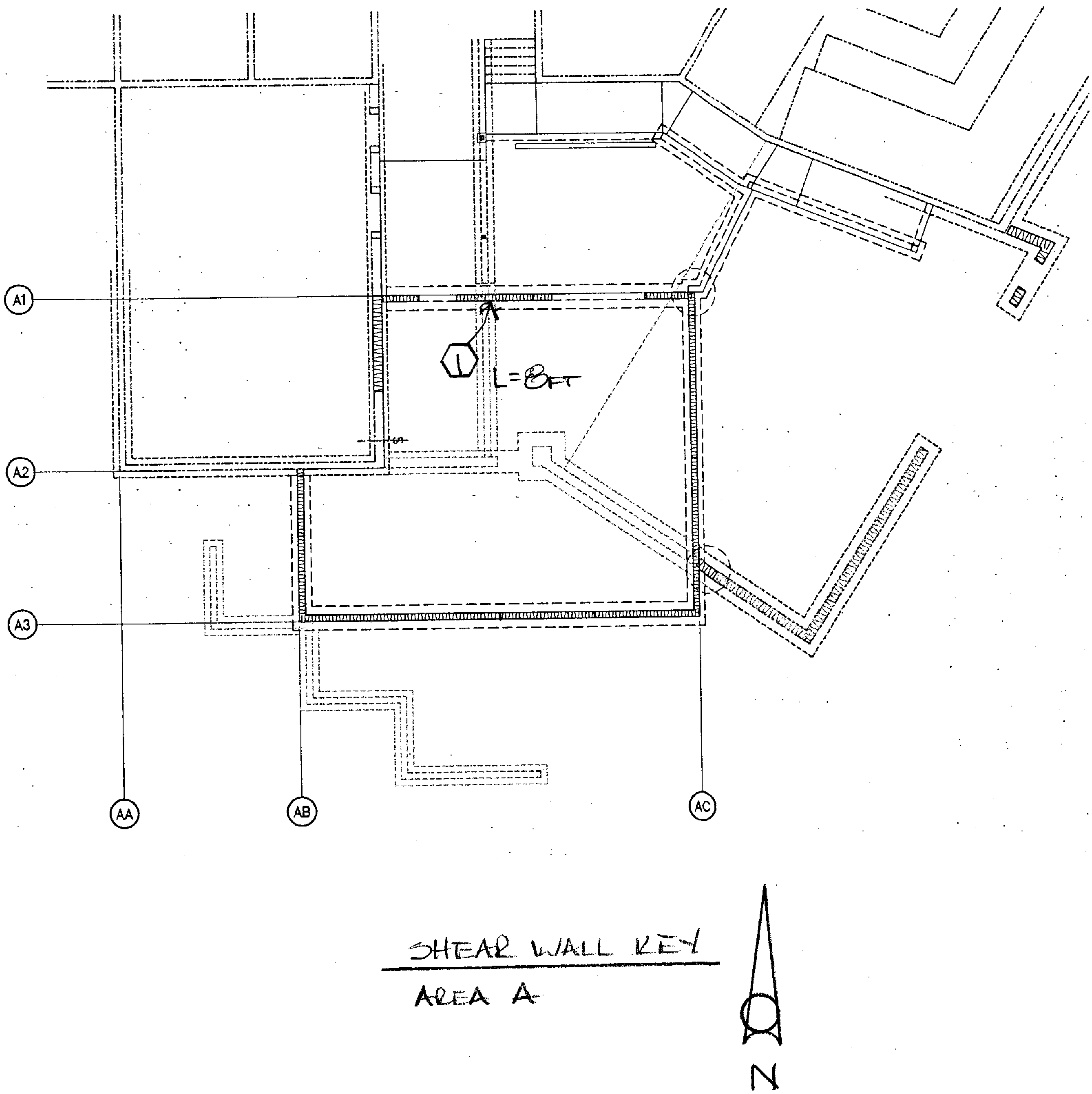
$T = C = 78.3/34 = 2.3_K$ ← W14x22 IS ADEQUATE FOR AXIAL LOAD

DIAPHRAGM OOP CONNECTION (ALL)

→ ALL DIA. ANCHORS CAN RESIST $0.280(4) = 1.12_K$ FORCE AT EA. ANCHOR POINT

CFSD HS KIT. EXP.
10048

35

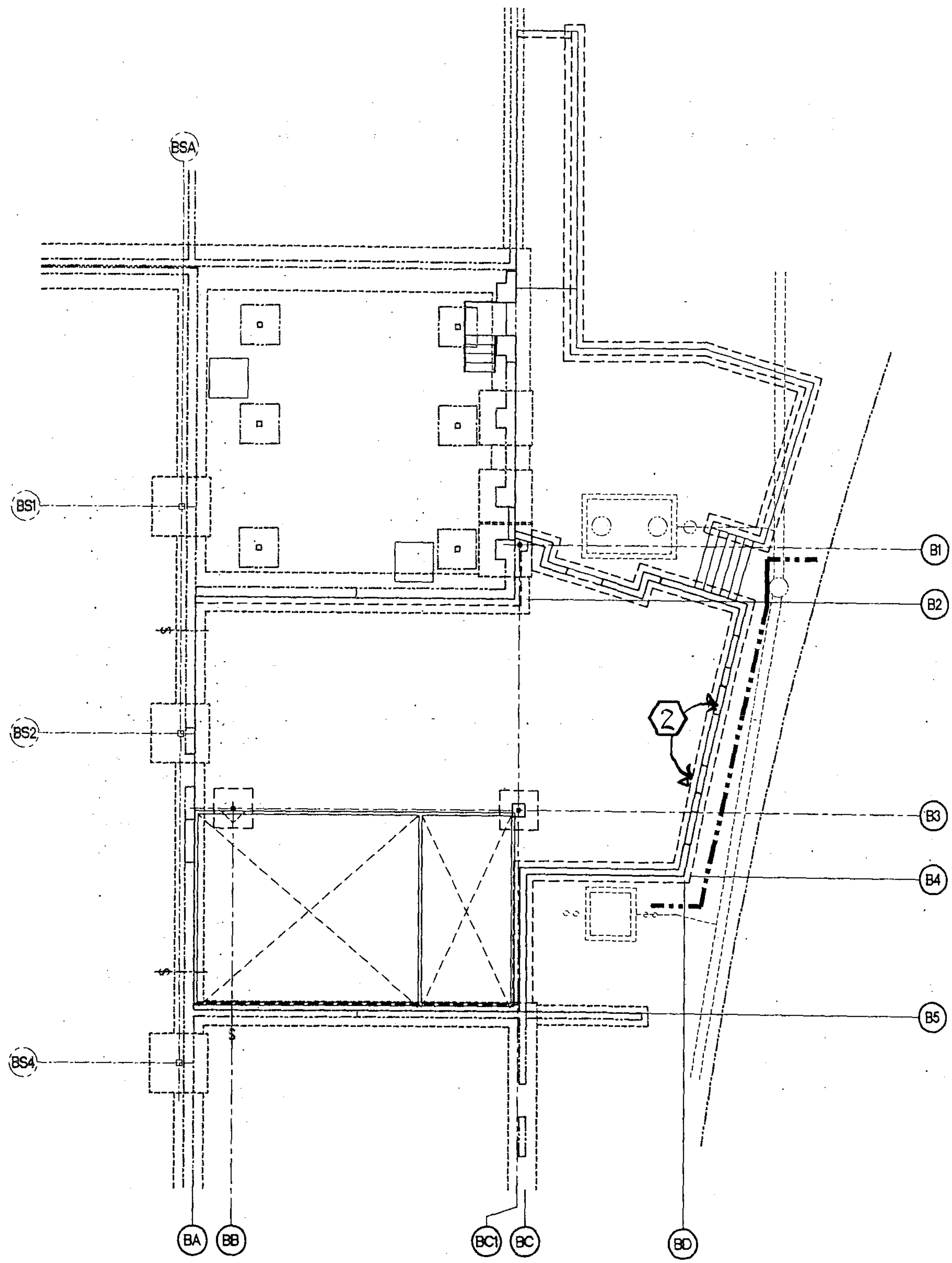


SHEAR WALL KEY

AREA A

CFSD HS KIT. EVP
10048

36



SHEAR WALL KEY

AREA B

N

① CONTROLLING SHEAR WALL - AREA A

8" CMU L = 8 FT ROOF = 20 FT PAR = 4 FT

$V_{DIA} = 7.52_K (\text{WIND})/2 = 3.76_K$ (CONS. ASSUME PIER TAKES TOTAL SHEAR LOAD)

$V_{WALL} = 0.085(0.075_{KSF})(8+4/2)(20+4) = 1.53_K$

$V_{TOT} = 3.76 + 1.53 = 5.29_K$

$P_{Dr} = (0.022)(34/2)(8+4/2) = 3.74_K$

$P_{Lr} = (0.020)(17)(10) = 3.40_K$

$P_{D\,LINTEL} = (0.075)(4/2)(24-7_{FT}) = 2.55_K$

→ 8" WALL w/ #5 AT 16" & STD JAMB BARS IS ADEQUATE (SEE CALC)

② CONTROLLING SHEAR WALL - AREA B (2 EACH)

8" CMU L = 2.67 FT ROOF = 14 FT PAR = 4 FT

$V_{DIA} = 4.52_K (\text{WIND})/2 = 2.26_K = 1.13_K$ EA. PIER (CONS. ASSUME PIERS TAKE TOTAL SHEAR)

$V_{WALL} = 0.085(0.075)(14+4)(2.67+5/2) = 0.60_K$

$V_{TOT} = 1.13 + 0.6 = 1.73_K$

$P_{Dr} = (0.022)(22/2)(2.67+5/2) = 1.25_K$

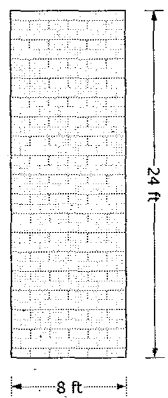
$P_{Lr} = (0.020)(11)(5.17) = 1.14_K$

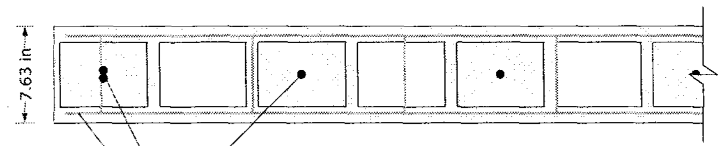
$P_{D\,LINTEL} = (0.075)(5/2)(8_{FT}) = 1.5_K$

→ 8" WALL W/ #5 AT 8" & HOR. PIER REINF. IS ADEQUATE (SEE CALC)

(000) 000-0000

Design Detail

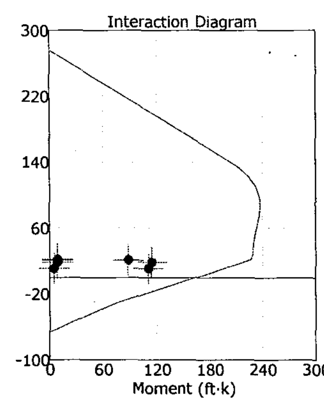




Check Summary

Ratio	Check	Provided	Required	Combination
----- Reinforcement Limits -----				
✓ 0.043	Vert Bar Area	0.02 in²	0 in²	1.0D + 1.0W
✓ 0.167	Vert Bar Spacing	16 in	96 in	1.0D + 1.0L
✓ 0.360	Shear Bar Spacing	16 in	44.5 in	1.0D + 1.0L
----- Strength Checks -----				
✓ 0.000	Axial Tension	24 k	0 k	1.0D + 1.0L
✓ 0.409	Shear	58.09 psi	23.78 psi	1.0D + 1.0W
✓ 0.320	Axial Compression	69.87 k	22.36 k	1.0D + 1.0L
✓ 0.566	Axial+Flexure	196.1 ft·k	110.9 ft·k	0.6D + 1.0W

Interaction Diagram



Criteria

Building Code	MSCJ-05 (ASD)
Load Combination	ASCE 7-05 (ASD)
Seismic R Value	3.50
Amplify Axial Stress For Slenderness	Yes
f'm	1500 psi
fy	60000 psi
Specify Wall Weight Manually	No
Block Weight	Normal weight
Design As Clay Masonry	No
Include Wall Self-Weight	Yes
End Bars Only For Flexural/Axial Analysis	No
Multiply Seismic Shear By 1.5	No

Load Combinations

ASCE 7-05 (ASD)

1.0D + 1.0L
1.0D + 1.0W
1.0D
1.0D + 0.75L + 0.75W
1.0D + 0.75L
0.6D + 1.0W
0.6D

Loads Summary

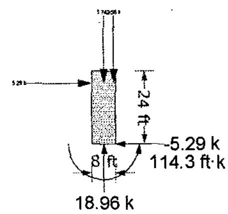
Load Set	Source	Axial Pt Load	Offset from C...	End 1 Axial D...	End 2 Axial D...	Shear Pt Load	Shear Distrib...	Shear Offset ...	Moment
Roof	Dead	3.74 k	0 ft	0 lb/ft	0 lb/ft	0 k	0 lb/ft	0 ft	0 ft·k
Roof	Live	3.4 k	0 ft	0 lb/ft	0 lb/ft	0 k	0 lb/ft	0 ft	0 ft·k
Lintel	Dead	2.55 k	3.33 ft	0 lb/ft	0 lb/ft	0 k	0 lb/ft	0 ft	0 ft·k
Wind	Wind	0 k	0 ft	0 lb/ft	0 lb/ft	5.29 k	0 lb/ft	0 ft	0 ft·k

Load Combination: 1.0D + 1.0W

Design Forces

Factored Loads

Wall Weight 12.67 k



Compression Check

Axial Compression [MSJC-05 2.3.3.2.1]

F_s = 24000 psi (Grade 60 reinf)

A_{st} = 0 in² / in (bars are not tied)

h / r = (24 ft) / (2.39 in) = 120.5534 > 99

$P_a = [0.25\, f'_m A_n + 0.65\, A_{st} F_s]\left[\frac{70\, r}{h}\right]$ ^ 2

$= [0.25\,(1500\text{ psi})\,(3.84\text{ ft}^2) + 0.65\,(0\text{ in}^2)\,(24000\text{ psi})]\left[\frac{70\,(2.39\text{ in})}{(24\text{ ft})}\right]$ ^ 2

= 69.87 k

P = 18.96 k ≤ P_a = 69.87 k ✓

Shear Check

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{b\,d} = \frac{(5.29\text{ k})}{(2.5\text{ in})(89\text{ in})} = 23.78$ psi

$A_v = V\,s / F_s\,d$ = (5.29 k) (16 in) / (24000 psi) (89 in) = 0.04 in²

A_{v_prov} = 0.05 in² ≥ A_v = 0.04 in²

Provided shear reinf. is sufficient; masonry stresses must still satisfy 2.3.5.2.3

$\frac{M}{V\,d} = \frac{(114.3\text{ ft·k})}{(5.29\text{ k})(89\text{ in})} = 2.9131 > 1$

$F_v = 1.5\sqrt{f'_m} = 1.5\sqrt{(1500\text{ psi})} = 58.09$ psi (≤ 75 psi)

f_v = 23.78 psi ≤ F_v = 58.09 psi ✓

Other Checks

Axial Tension [MSJC-05 2.3.4, 2.3.2.1]

$f_t = \frac{T}{A_s} = \frac{(0\text{ k})}{(2.76\text{ in}^2)} = 0$ psi

F_s = 24000 psi (Grade 60 reinf)

f_t = 0 psi ≤ F_s = 24000 psi ✓

Shear Reinforcement [MSJC-05 2.3.5.3.1, 2.3.5.3.2]

d / 2 = (89 in) / 2 = 44.50 ≤ 48

s_{reqd} = 44.5 in

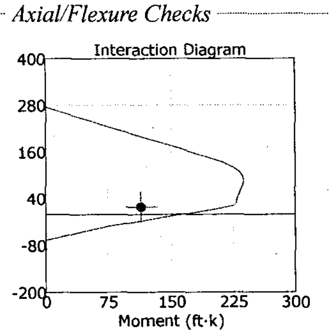
s = 16 in ≤ s_{reqd} = 44.5 in ✓

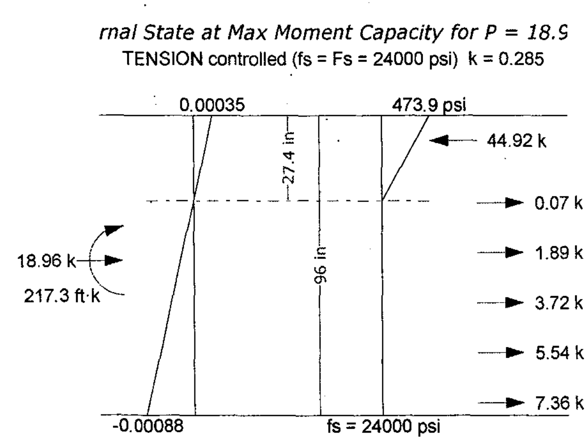
(1 / 3) A_v = [1 / 3] (0.0025) = 0 in² / in

$A_{v\_perp\_prov}$ = 0.02 in² / in ≥ $A_{v\_perp\_reqd}$ = 0 in² / in ✓

$s_{\_perp}$ = 16 in ≤ $s_{\_perp\_reqd}$ = 96 in ✓

Axial/Flexure Checks





Combined Axial Flexure [MSJC-05 2.3.3.2.2, 2.3.2.1]

P = 18.96 k

M_a = 217.3 ft·k (from interaction diagram given P)

M = 114.3 ft·k ≤ M_a = 217.3 ft·k ✓

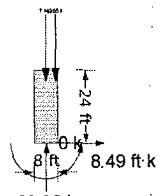
40

Load Combination: 1.0D + 1.0L

Design Forces

Factored Loads

Wall Weight 12.67 k



Compression Check

Axial Compression [MSJC-05 2.3.3.2.1]

F_s = 24000 psi (Grade 60 reinf)

A_{st} = 0 in² / in (bars are not tied)

h / r = (24 ft) / (2.39 in) = 120.5534 > 99

$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s] \left[\frac{70\,r}{h}\right]$ ^ 2

$= [0.25\,(1500\text{ psi})\,(3.84\text{ ft}^2) + 0.65\,(0\text{ in}^2)\,(24000\text{ psi})] \left[\frac{70\,(2.39\text{ in})}{(24\text{ ft})}\right]$ ^ 2

= 69.87 k

P = 22.36 k ≤ P_a = 69.87 k ✓

Shear Check

Shear [MSJC-05 2.3.5]

There is zero applied shear force in this load case✓

Other Checks

Axial Tension [MSJC-05 2.3.4, 2.3.2.1]

$f_t = \frac{T}{A_s} = \frac{(0\text{ k})}{(2.76\text{ in}^2)}$ = 0 psi

F_s = 24000 psi (Grade 60 reinf)

f_t = 0 psi ≤ F_s = 24000 psi ✓

Shear Reinforcement [MSJC-05 2.3.5.3.1, 2.3.5.3.2]

d / 2 = (89 in) / 2 = 44.50 ≤ 48

s_{reqd} = 44.5 in

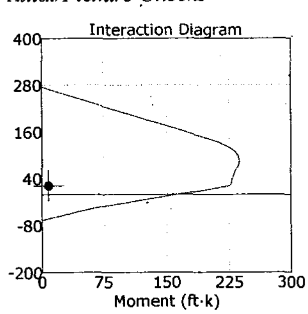
s = 16 in ≤ s_{reqd} = 44.5 in ✓

(1 / 3) A_v = [1 / 3] (0.0) = 0 in² / in

$A_{v\_perp\_prov}$ = 0.02 in² / in ≥ $A_{v\_perp\_reqd}$ = 0 in² / in✓

$s_{\_perp}$ = 16 in ≤ $s_{\_perp\_reqd}$ = 96 in✓

Axial/Flexure Checks



Combined Axial Flexure [MSJC-05 2.3.3.2.2, 2.3.2.1]

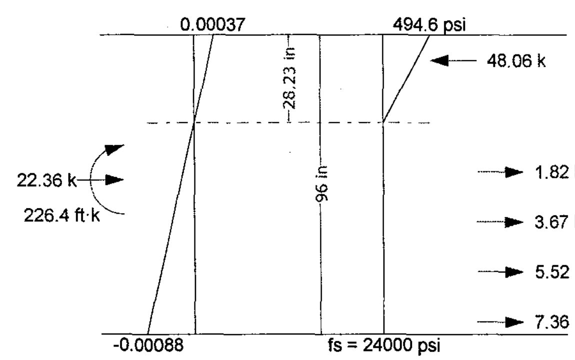
P = 22.36 k

M_a = 226.4 ft·k (from interaction diagram given P)

M = 8.49 ft·k ≤ M_a = 226.4 ft·k✓

rnal State at Max Moment Capacity for P = 22.3

TENSION controlled (fs = Fs = 24000 psi) k = 0.294

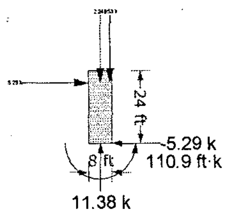


Load Combination: 0.6D + 1.0W

Design Forces

Factored Loads

Wall Weight 7.6 k



Compression Check

Axial Compression [MSJC-05 2.3.3.2.1]

F_s = 24000 psi (Grade 60 reinf)

A_{st} = 0 in² / in (bars are not tied)

h / r = (24 ft) / (2.39 in) = 120.5534 > 99

$P_a = [0.25\, f'_m A_n + 0.65\, A_{st} F_s]\left[\frac{70\,r}{h}\right]$ ^ 2

$= [0.25\,(1500\text{ psi})\,(3.84\text{ ft}^2) + 0.65\,(0\text{ in}^2)\,(24000\text{ psi})]\left[\frac{70\,(2.39\text{ in})}{(24\text{ ft})}\right]$ ^ 2

= 69.87 k

P = 11.38 k ≤ P_a = 69.87 k ✓

Shear Check

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{b\,d} = \frac{(5.29\text{ k})}{(2.5\text{ in})(89\text{ in})} = 23.78$ psi

$A_v = V s / F_s d$ = (5.29 k) (16 in) / (24000 psi) (89 in) = 0.04 in²

A_{v_prov} = 0.05 in² ≥ A_v = 0.04 in²

Provided shear reinf. is sufficient; masonry stresses must still satisfy 2.3.5.2.3

$\frac{M}{V\,d} = \frac{(110.9\text{ ft·k})}{(5.29\text{ k})(89\text{ in})} = 2.8265 > 1$

$F_v = 1.5\sqrt{f'_m} = 1.5\sqrt{(1500\text{ psi})} = 58.09$ psi (≤ 75 psi)

f_v = 23.78 psi ≤ F_v = 58.09 psi ✓

Other Checks

Axial Tension [MSJC-05 2.3.4, 2.3.2.1]

$f_t = \frac{T}{A_s} = \frac{(0\text{ k})}{(2.76\text{ in}^2)} = 0$ psi

F_s = 24000 psi (Grade 60 reinf)

f_t = 0 psi ≤ F_s = 24000 psi ✓

Shear Reinforcement [MSJC-05 2.3.5.3.1, 2.3.5.3.2]

d / 2 = (89 in) / 2 = 44.50 ≤ 48

s_{reqd} = 44.5 in

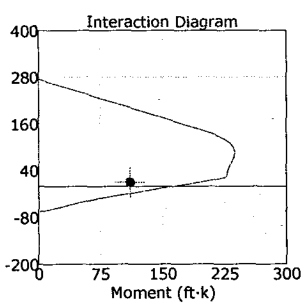
s = 16 in ≤ s_{reqd} = 44.5 in ✓

(1 / 3) A_v = [1 / 3] (0.0025) = 0 in² / in

$A_{v\_perp\_prov}$ = 0.02 in² / in ≥ $A_{v\_perp\_reqd}$ = 0 in² / in ✓

$s_{\_perp}$ = 16 in ≤ $s_{\_perp\_reqd}$ = 96 in ✓

Axial/Flexure Checks



Combined Axial Flexure [MSJC-05 2.3.3.2.2, 2.3.2.1]

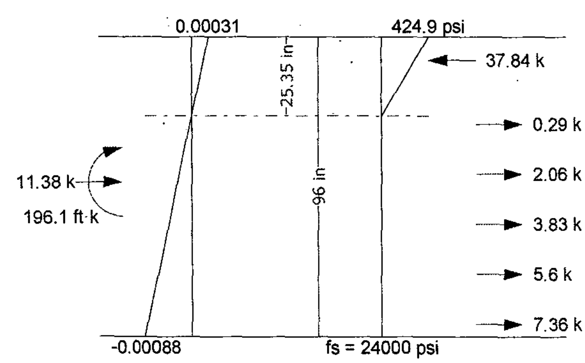
P = 11.38 k

M_a = 196.1 ft·k (from interaction diagram given P)

M = 110.9 ft·k ≤ M_a = 196.1 ft·k ✓

rnal State at Max Moment Capacity for P = 11.3

TENSION controlled (fs = Fs = 24000 psi) k = 0.264



Art Cantrell
Holben, Martin _White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

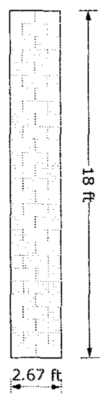
Shear Wall 2 - Area B

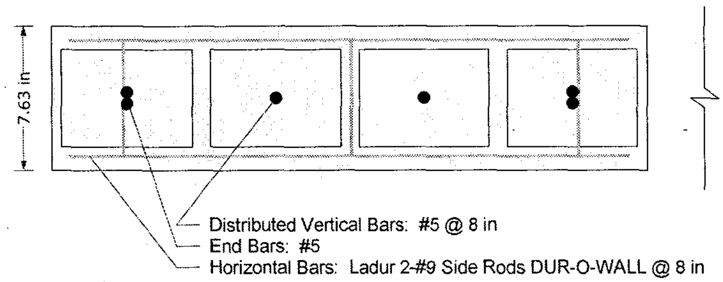
CFSD HS Kitchen Expansion

Job # 10048

42

Design Detail

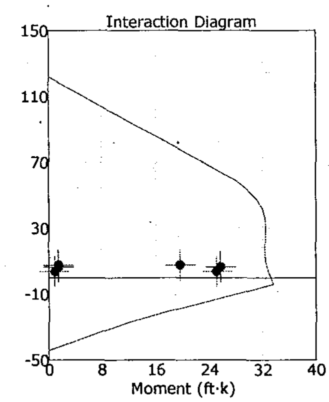




Check Summary

Ratio	Check	Provided	Required	Combination
----- Reinforcement Limits -----				
✓ 0.025	Vert Bar Area	0.04 in²	0 in²	1.0D + 1.0W
✓ 0.083	Vert Bar Spacing	8 in	96 in	1.0D + 1.0L
✓ 0.639	Shear Bar Spacing	8 in	12.52 in	1.0D + 1.0L
----- Strength Checks -----				
✓ 0.000	Axial Tension	24 k	0 k	1.0D + 1.0L
✓ 0.156	Shear	58.09 psi	9.06 psi	1.0D + 1.0W
✓ 0.170	Axial Compression	46.6 k	7.93 k	1.0D + 1.0L
✓ 0.787	Axial+Flexure	32.67 ft·k	25.72 ft·k	1.0D + 1.0W

Interaction Diagram



Criteria

Building Code	MSCJ-05 (ASD)
Load Combination	ASCE 7-05 (ASD)
Seismic R Value	3.50
Amplify Axial Stress For Slenderness	Yes
f'm	1500 psi
fy	60000 psi
Specify Wall Weight Manually	No
Block Weight	Normal weight
Design As Clay Masonry	No
Include Wall Self-Weight	Yes
End Bars Only For Flexural/Axial Analysis	No
Multiply Seismic Shear By 1.5	No

Load Combinations

ASCE 7-05 (ASD)

1.0D + 1.0L
1.0D + 1.0W
1.0D
1.0D + 0.75L + 0.75W
1.0D + 0.75L
0.6D + 1.0W
0.6D

Loads Summary

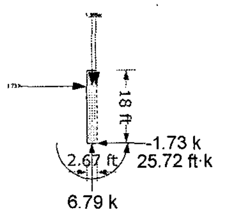
Load Set	Source	Axial Pt Load	Offset from C...	End 1 Axial D...	End 2 Axial D...	Shear Pt Load	Shear Distrib...	Shear Offset ...	Moment
Roof	Dead	1.25 k	0 ft	0 lb/ft	0 lb/ft	0 k	0 lb/ft	0 ft	0 ft·k
Roof	Live	1.14 k	0 ft	0 lb/ft	0 lb/ft	0 k	0 lb/ft	0 ft	0 ft·k
Lintel	Dead	1.5 k	1 ft	0 lb/ft	0 lb/ft	0 k	0 lb/ft	0 ft	0 ft·k
Wind	Wind	0 k	0 ft	0 lb/ft	0 lb/ft	1.73 k	0 lb/ft	0 ft	0 ft·k

Load Combination: 1.0D + 1.0W

Design Forces

Factored Loads

Wall Weight 4.04 k



Compression Check

Axial Compression [MSJC-05 2.3.3.2.1]

F_s = 24000 psi (Grade 60 reinf)

A_{st} = 0 in² / in (bars are not tied)

h / r = (18 ft) / (2.2 in) = 98.1306 ≤ 99

$P_a = [0.25\, f'_m A_n + 0.65\, A_{st} F_s] \left[1 - \left[\frac{h}{140\, r}\right] \wedge 2\right]$

$= [0.25\,(1500 \text{ psi})\,(1.7 \text{ ft}^2) + 0.65\,(0 \text{ in}^2)\,(24000 \text{ psi})] \left[1 - \left[\frac{(18 \text{ ft})}{140\,(2.2 \text{ in})}\right] \wedge 2\right]$

= 46.6 k

P = 6.79 k ≤ P_a = 46.6 k ✓

Shear Check

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{b\, d} = \frac{(1.73 \text{ k})}{(7.63 \text{ in})\,(25.04 \text{ in})} = 9.06 \text{ psi}$

$A_v = V\, s / F_s\, d$ = (1.73 k) (8 in) / (24000 psi) (25.04 in) = 0.02 in²

A_{v_prov} = 0.05 in² ≥ A_v = 0.02 in²

Provided shear reinf. is sufficient; masonry stresses must still satisfy 2.3.5.2.3

$\frac{M}{V\, d} = \frac{(25.72 \text{ ft·k})}{(1.73 \text{ k})\,(25.04 \text{ in})} = 7.1248 > 1$

$F_v = 1.5\sqrt{f'_m} = 1.5\sqrt{(1500 \text{ psi})} = 58.09 \text{ psi}$ (≤ 75 psi)

f_v = 9.06 psi ≤ F_v = 58.09 psi ✓

Other Checks

Axial Tension [MSJC-05 2.3.4, 2.3.2.1]

$f_t = \frac{T}{A_s} = \frac{(0 \text{ k})}{(1.84 \text{ in}^2)} = 0 \text{ psi}$

F_s = 24000 psi (Grade 60 reinf)

f_t = 0 psi ≤ F_s = 24000 psi ✓

Shear Reinforcement [MSJC-05 2.3.5.3.1, 2.3.5.3.2]

d / 2 = (25.04 in) / 2 = 12.520 ≤ 48

s_{reqd} = 12.52 in

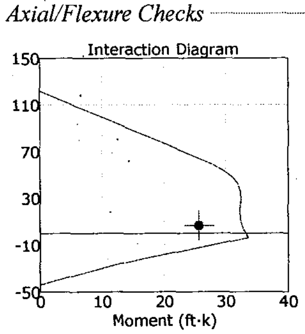
s = 8 in ≤ s_{reqd} = 12.52 in ✓

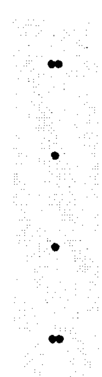
(1 / 3) A_v = [1 / 3] (0.0029) = 0 in² / in

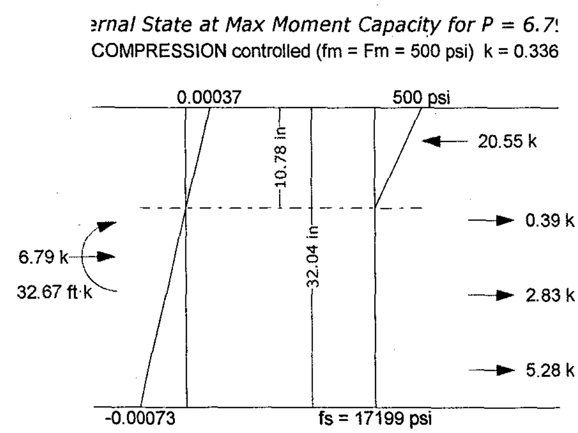
$A_{v\_perp\_prov}$ = 0.04 in² / in ≥ $A_{v\_perp\_reqd}$ = 0 in² / in ✓

$s_{\_perp}$ = 8 in ≤ $s_{\_perp\_reqd}$ = 96 in ✓

Axial/Flexure Checks







Combined Axial Flexure [MSJC-05 2.3.3.2.2, 2.3.2.1]

P = 6.79 k

M_a = 32.67 ft·k (from interaction diagram given P)

M = 25.72 ft·k ≤ M_a = 32.67 ft·k ✓

Art Cantrell
Holben, Martin _White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

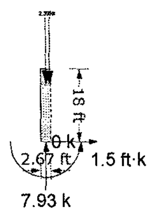
Shear Wall 2 - Area B
CFSD HS Kitchen Expansion

Load Combination: 1.0D + 1.0L

Design Forces

Factored Loads

Wall Weight 4.04 k



Compression Check

Axial Compression [MSJC-05 2.3.3.2.1]

F_s = 24000 psi (Grade 60 reinf)

A_{st} = 0 in² / in (bars are not tied)

h / r = (18 ft) / (2.2 in) = 98.1306 ≤ 99

$P_a = [0.25\, f'_m A_n + 0.65\, A_{st} F_s]\left[1 - \left[\frac{h}{140\, r}\right]^{\wedge} 2\right]$

$= [0.25\,(1500 \text{ psi})\,(1.7 \text{ ft}^2) + 0.65\,(0 \text{ in}^2)\,(24000 \text{ psi})]\left[1 - \left[\frac{(18 \text{ ft})}{140\,(2.2 \text{ in})}\right]^{\wedge} 2\right]$

= 46.6 k

P = 7.93 k ≤ P_a = 46.6 k ✓

Shear Check

Shear [MSJC-05 2.3.5]

There is zero applied shear force in this load case ✓

Other Checks

Axial Tension [MSJC-05 2.3.4, 2.3.2.1]

$f_t = \frac{T}{A_s} = \frac{(0 \text{ k})}{(1.84 \text{ in}^2)} = 0 \text{ psi}$

F_s = 24000 psi (Grade 60 reinf)

f_t = 0 psi ≤ F_s = 24000 psi ✓

Shear Reinforcement [MSJC-05 2.3.5.3.1, 2.3.5.3.2]

d / 2 = (25.04 in) / 2 = 12.520 ≤ 48

s_{reqd} = 12.52 in

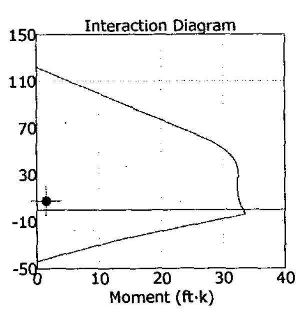
s = 8 in ≤ s_{reqd} = 12.52 in ✓

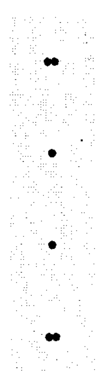
(1 / 3) A_v = [1 / 3] (0.0) = 0 in² / in

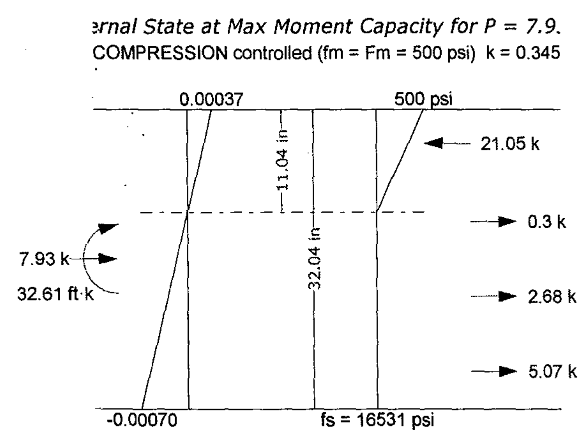
$A_{v\_perp\_prov}$ = 0.04 in² / in ≥ $A_{v\_perp\_reqd}$ = 0 in² / in ✓

$s_{\_perp}$ = 8 in ≤ $s_{\_perp\_reqd}$ = 96 in ✓

Axial/Flexure Checks







Combined Axial_Flexure [MSJC-05 2.3.3.2.2, 2.3.2.1]

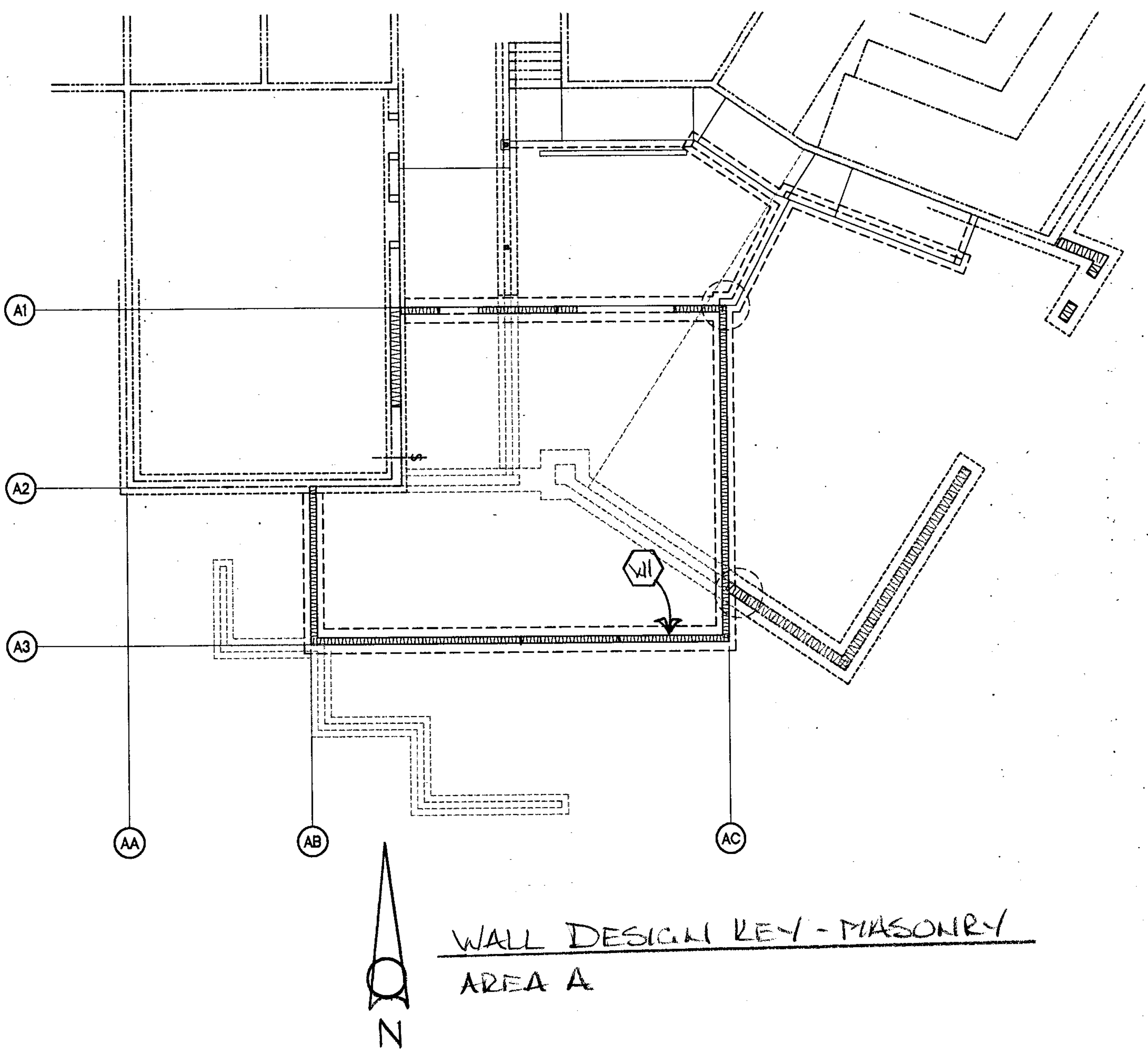
P = 7.93 k

M_a = 32.61 ft·k (from interaction diagram given P)

M = 1.5 ft·k ≤ M_a = 32.61 ft·k ✓

CFSD HS KIT EVP
10048

45

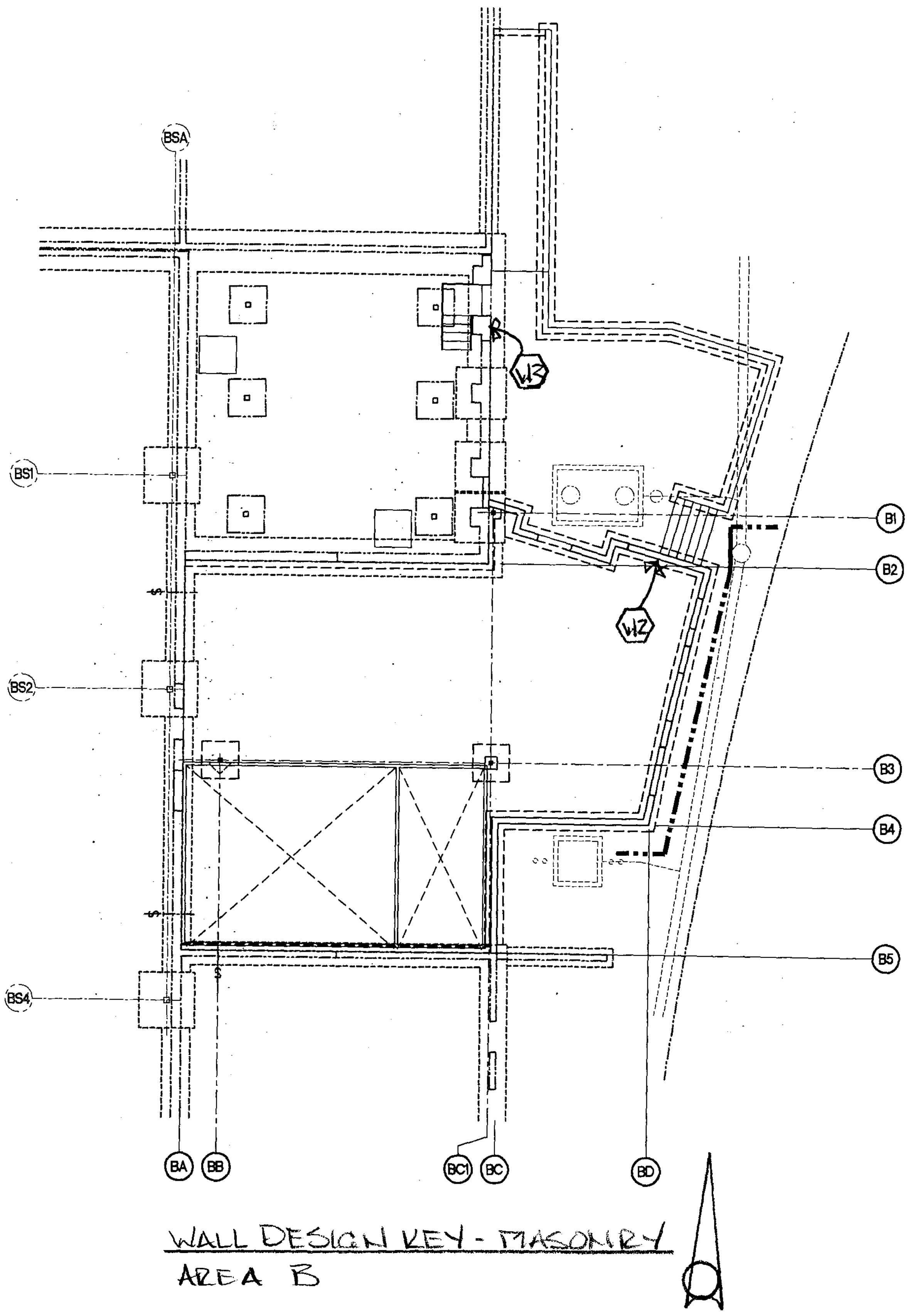


WALL DESIGN KEY - MASONRY

AREA A

CFSD HS KIT EXP
10048

46



WALL DESIGN KEY - MASONRY
AREA B

W1 MASONRY WALL AT AREA A 8" CMU

$L = 10$ FT $e = 2.5$ IN
$RF = 20$ FT
$PAR = 4$ FT

ROOF LOAD: $w_{Dr} = (22\text{PSF})(34/2) = 374$ PLF
$w_{Lr} = (20\text{PSF})(34/2) = 340$ PLF

LATERAL: $F_S = 0.4 S_{DS} I W_w$
$= 0.4(0.297)(1.0)(75\text{PSF})$
$= 8.91$ PSF
F_w: $A_T = 20(20/3) = 133$ FT² ← USE 100
$F_w = 18$ PSF (WIND)
$= 30$ PSF (PARAPET)

→ USE 8" CMU w/ #5 AT 16" O.C. (SEE CALC)

W2 MASONRY WALL AT AREA B 8" CMU

$L = 10$ FT $RF = 14$ FT $PAR = 4$ FT $e = 2.5$ IN

ROOF LOAD: $w_{Dr} = (22\text{PSF})(34/2) = 374$ PLF
$w_{Lr} = (20\text{PSF})(34/2) = 340$ PLF

LATERAL: FROM ABOVE, USE WIND (18PSF, 30PSF)

→ USE 8" CMU w/ #5 @ 32" O.C. (SEE CALC)

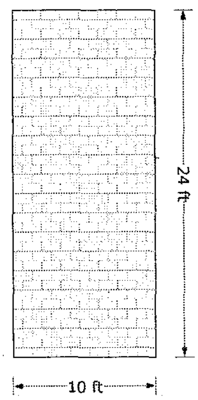
Art Cantrell
Holben, Martin _White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

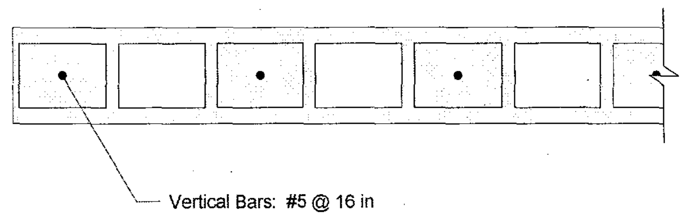
Bearing Wall 1 - Area A
CFSD HS Kitchen Expansion

Job # 10048

48

Design Detail

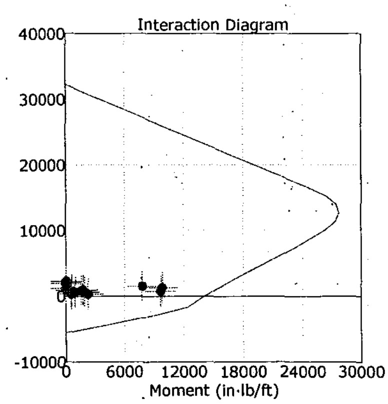




Check Summary

Ratio	Check	Provided	Required	Combination
----- Strength Checks -----				
✓ 0.185	Axial Compression	1.03 k	0.19 k	1.0D + 1.0L
✓ 0.000	Axial Tension	24 k	0 k	1.0D + 1.0L
✓ 0.107	Shear	38.73 psi	4.15 psi	0.6D + 1.0W
✓ 0.645	Axial+Flexure	14995 in·lb/ft	9672 in·lb/ft	0.6D + 1.0W

Interaction Diagram



Criteria

Building Code	MSCJ-05 (ASD)
Load Combination	ASCE 7-05 (ASD)
Seismic R Value	3.50
Amplify Axial Stress For Slenderness	Yes
f'm	1500 psi
fy	60000 psi
Specify Wall Weight Manually	No
Block Weight	Normal weight
Design As Clay Masonry	No
Include Wall Self-Weight	Yes
Neglect Lateral Load on Parapet	No
Include Wall Wt In Virtual Eccentricity	No
Always use I-cracked	No

Load Combinations

ASCE 7-05 (ASD)

1.0D + 1.0L
1.0D + 1.0W
1.0D
1.0D + 0.75L + 0.75W
1.0D + 0.75L
0.6D + 1.0W
0.6D

Loads Summary

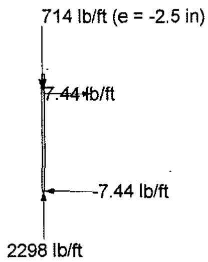
Load Set	Source	Axial Unifo...	Axial Pt Lo...	Pt Ld Eff W...	Eccentricity	Lateral Pre...	Top Lateral...	Parapet Pr...	Lateral Uni...	Lat Unif Ld...	Moment
Roof	Dead	374 lb/ft	0 k	1 ft	-2.5 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft
Roof	Live	340 lb/ft	0 k	1 ft	-2.5 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft
Wind	Wind	0 lb/ft	0 k	1 ft	0 in	18 psf	18 psf	30 psf	0 lb/ft	1 ft	0 in·lb/ft

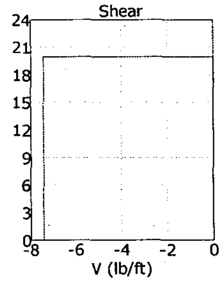
Art Cantrell
Holben, Martin _White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

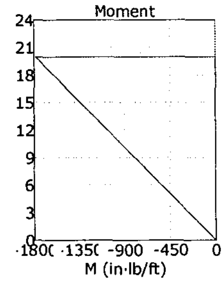
Bearing Wall 1 - Area A
CFSD HS Kitchen Expansion

Load Combination: 1.0D + 1.0L

Design Forces



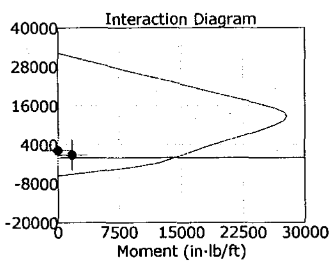




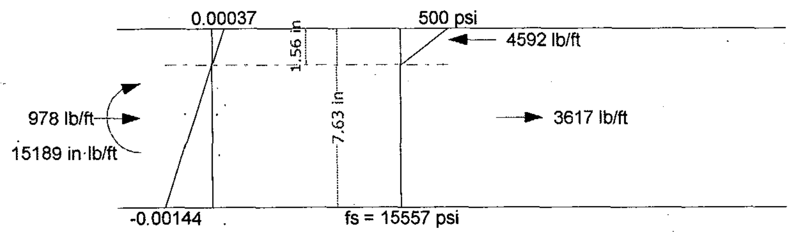
Factored Loads

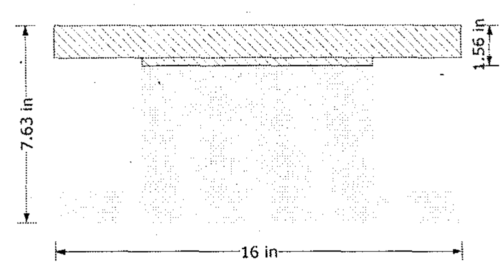
Wall Weight	1584 lb/ft
Effective Eccentricity	-2.5 in
Applied Eccentricity	-2.5 in

Axial/Flexure Checks



Internal State at Max Moment Capacity for P = 978 lb/ft
COMPRESSION controlled (fm = Fm = 500 psi) k = 0.204





Combined Axial_Flexure (@ base) [MSJC-05 2.3.3.2.2, 2.3.:

$P = 2298$ lb / ft
$M_a = 16713$ in·lb / ft (from interaction diagram given P)
$M = 0$ in·lb / ft $\leq$ $M_a = 16713$ in·lb / ft ✓

Combined Axial_Flexure (@ max M) [MSJC-05 2.3.3.2.2, 2.:

$P = 978$ lb / ft
$M_a = 15189$ in·lb / ft (from interaction diagram given P)
$M = 1785$ in·lb / ft $\leq$ $M_a = 15189$ in·lb / ft ✓

Combined Axial_Flexure (@ top) [MSJC-05 2.3.3.2.2, 2.3.2.

$P = 978$ lb / ft
$M_a = 15189$ in·lb / ft (from interaction diagram given P)
$M = 1785$ in·lb / ft $\leq$ $M_a = 15189$ in·lb / ft ✓

Other Checks

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{d} = \frac{(7.44 \text{ lb / ft})}{(3.81 \text{ in})} = 0.16$ psi

$F_v = \sqrt{f'_m} = \sqrt{(1500 \text{ psi})} = 38.73$ psi ($\leq$ 50 psi)

$f_v = 0.16$ psi $\leq$ $F_v = 38.73$ psi ✓

Axial Compression (@ base) [MSJC-05 2.3.3.2.1]

$F_s = 24000$ psi (Grade 60 reinf)
$A_{st} = 0$ in² / in (bars are not tied)
$h / r = (20 \text{ ft}) / (2.42 \text{ in}) = 99.3488 > 99$

$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s] \left[\frac{70 r}{h}\right]^2$

$= [0.25 (1500 \text{ psi}) (0.46 \text{ ft}^2 / \text{ft}) + 0.65 (0 \text{ in}^2 / \text{in}) (24000 \text{ psi})] \left[\frac{70 (2.42 \text{ in})}{(20 \text{ ft})}\right]^2$

$= 12407$ lb / ft

$P = 2298$ lb / ft $\leq$ $P_a = 12407$ lb / ft ✓

Art Cantrell
Holben, Martin White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

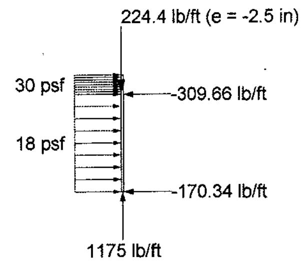
Bearing Wall 1 - Area A
CFSD HS Kitchen Expansion

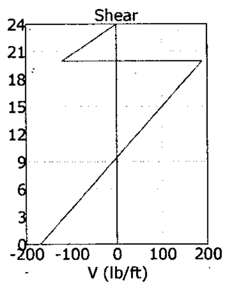
Job # 10048

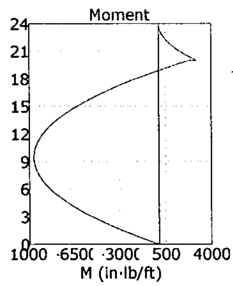
50

Load Combination: 0.6D + 1.0W

Design Forces



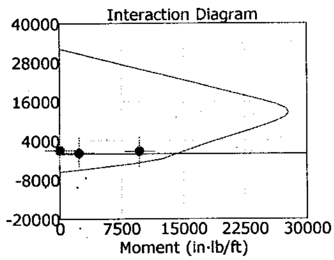




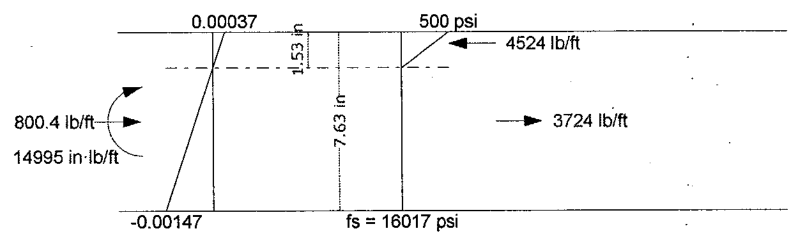
Factored Loads

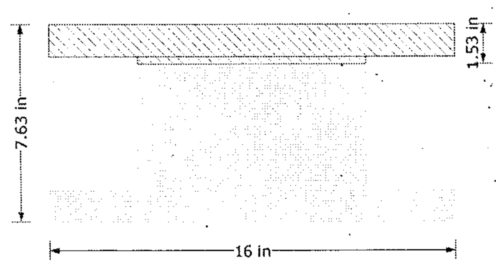
Wall Weight	950.4 lb/ft
Effective Eccentricity	-2.5 in
Applied Eccentricity	-2.5 in

Axial/Flexure Checks



Internal State at Max Moment Capacity for P = 800.4 lb/ft
COMPRESSION controlled (fm = Fm = 500 psi) k = 0.201





Combined Axial Flexure (@ base) [MSJC-05 2.3.3.2.2, 2.3.

$P = 1175$ lb / ft

$M_a = 15413$ in·lb / ft (from interaction diagram given P)

$M = 0$ in·lb / ft $\leq$ $M_a = 15413$ in·lb / ft ✓

Combined Axial Flexure (@ max M) [MSJC-05 2.3.3.2.2, 2.

$P = 800.4$ lb / ft

$M_a = 14995$ in·lb / ft (from interaction diagram given P)

$M = 9672$ in·lb / ft $\leq$ $M_a = 14995$ in·lb / ft ✓

Combined Axial Flexure (@ top) [MSJC-05 2.3.3.2.2, 2.3.2.

$P = 382.8$ lb / ft

$M_a = 14543$ in·lb / ft (from interaction diagram given P)

$M = 2319$ in·lb / ft $\leq$ $M_a = 14543$ in·lb / ft ✓

Other Checks

Shear [MSJC-05 2.3.5]

$$f_v = \frac{V}{d} = \frac{(189.7 \text{ lb / ft})}{(3.81 \text{ in})} = 4.15 \text{ psi}$$

$$F_v = \sqrt{f'_m} = \sqrt{(1500 \text{ psi})} = 38.73 \text{ psi} \quad (\leq 50 \text{ psi})$$

$f_v = 4.15$ psi $\leq$ $F_v = 38.73$ psi ✓

Axial Compression (@ base) [MSJC-05 2.3.3.2.1]

$F_s = 24000$ psi (Grade 60 reinf)

$A_{st} = 0$ in² / in (bars are not tied)

$h / r = (20 \text{ ft}) / (2.42 \text{ in}) = 99.3488 > 99$

$$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s]\left[\frac{70 r}{h}\right]^2$$

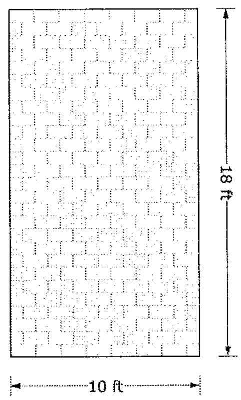
$$= [0.25 (1500 \text{ psi}) (0.46 \text{ ft}^2 / \text{ft}) + 0.65 (0 \text{ in}^2 / \text{in}) (24000 \text{ psi})]\left[\frac{70 (2.42 \text{ in})}{(20 \text{ ft})}\right]^2$$

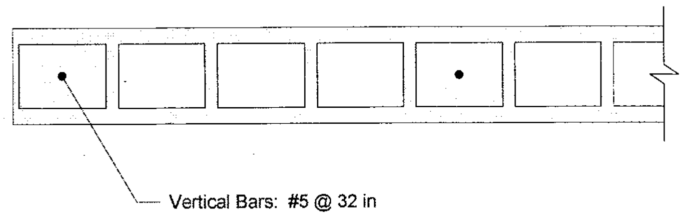
$= 12407$ lb / ft

$P = 1175$ lb / ft $\leq$ $P_a = 12407$ lb / ft ✓

(000) 000-0000

Design Detail

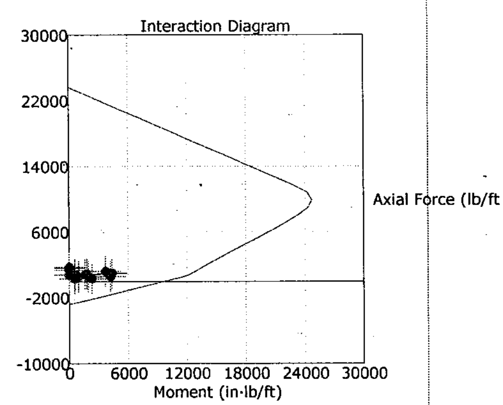




Check Summary

Ratio	Check	Provided	Required	Combination
----- Strength Checks -----				
✓ 0.121	Axial Compression	1.21 k	0.15 k	1.0D + 1.0L
✓ 0.000	Axial Tension	24 k	0 k	1.0D + 1.0L
✓ 0.079	Shear	38.73 psi	3.06 psi	0.6D + 1.0W
✓ 0.358	Axial+Flexure	11721 in·lb/ft	4196 in·lb/ft	0.6D + 1.0W

Interaction Diagram



Criteria

Building Code	MSCJ-05 (ASD)
Load Combination	ASCE 7-05 (ASD)
Seismic R Value	3.50
Amplify Axial Stress For Slenderness	Yes
f'm	1500 psi
fy	60000 psi
Specify Wall Weight Manually	No
Block Weight	Normal weight
Design As Clay Masonry	No
Include Wall Self-Weight	Yes
Neglect Lateral Load on Parapet	No
Include Wall Wt In Virtual Eccentricity	No
Always use I-cracked	No

Load Combinations

ASCE 7-05 (ASD)

1.0D + 1.0L
1.0D + 1.0W
1.0D
1.0D + 0.75L + 0.75W
1.0D + 0.75L
0.6D + 1.0W
0.6D

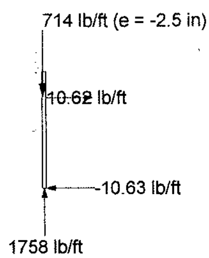
Loads Summary

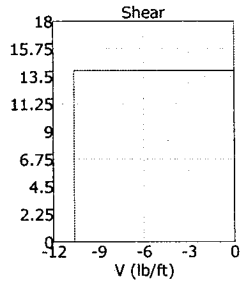
Load Set	Source	Axial Unifo...	Axial Pt Lo...	Pt Ld Eff W...	Eccentricity	Lateral Pre...	Top Lateral...	Parapet Pr...	Lateral Uni...	Lat Unif Ld...	Moment
Roof	Dead	374 lb/ft	0 k	1 ft	-2.5 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft
Roof	Live	340 lb/ft	0 k	1 ft	-2.5 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft
Wind	Wind	0 lb/ft	0 k	1 ft	0 in	18 psf	18 psf	30 psf	0 lb/ft	1 ft	0 in·lb/ft

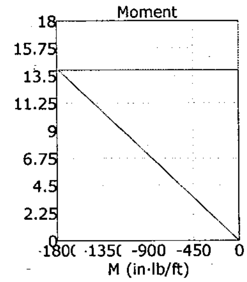
Art Cantrell
Holben, Martin _White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

Load Combination: 1.0D + 1.0L

Design Forces



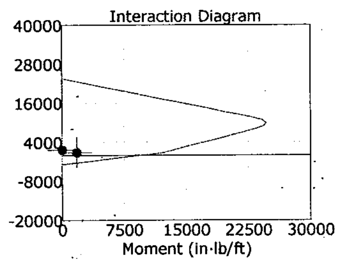




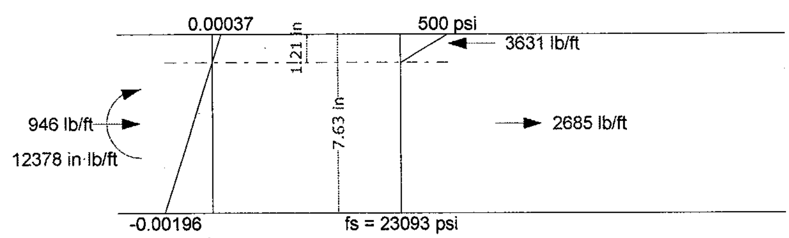
Factored Loads

Wall Weight	1044 lb/ft
Effective Eccentricity	-2.5 in
Applied Eccentricity	-2.5 in

Axial/Flexure Checks



Internal State at Max Moment Capacity for P = 946 lb/ft
COMPRESSION controlled (fm = Fm = 500 psi) k = 0.159



Combined Axial Flexure (@ base) [MSJC-05 2.3.3.2.2, 2.3.:

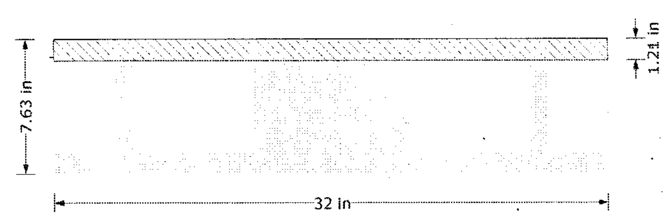
$P = 1758$ lb / ft
$M_a = 13586$ in·lb / ft (from interaction diagram given P)
$M = 0$ in·lb / ft $\leq$ $M_a = 13586$ in·lb / ft ✓

Combined Axial Flexure (@ max M) [MSJC-05 2.3.3.2.2, 2.:

$P = 946$ lb / ft
$M_a = 12378$ in·lb / ft (from interaction diagram given P)
$M = 1785$ in·lb / ft $\leq$ $M_a = 12378$ in·lb / ft ✓

Combined Axial Flexure (@ top) [MSJC-05 2.3.3.2.2, 2.3.2.

$P = 946$ lb / ft
$M_a = 12378$ in·lb / ft (from interaction diagram given P)
$M = 1785$ in·lb / ft $\leq$ $M_a = 12378$ in·lb / ft ✓



Other Checks

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{d} = \frac{(10.63 \text{ lb / ft})}{(3.81 \text{ in})} = 0.23$ psi

$F_v = \sqrt{f'_m} = \sqrt{(1500 \text{ psi})} = 38.73$ psi (≤ 50 psi)

$f_v = 0.23$ psi $\leq$ $F_v = 38.73$ psi ✓

Axial Compression (@ base) [MSJC-05 2.3.3.2.1]

$F_s = 24000$ psi (Grade 60 reinf)
$A_{st} = 0$ in² / in (bars are not tied)
$h / r = (14 \text{ ft}) / (2.68 \text{ in}) = 62.5871 \leq 99$

$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s]\left[1 - \left[\frac{h}{140 r}\right]^{\wedge}2\right]$

$= [0.25 (1500 \text{ psi}) (0.34 \text{ ft}^2 / \text{ft}) + 0.65 (0 \text{ in}^2 / \text{in}) (24000 \text{ psi})]\left[1 - \left[\frac{(14 \text{ ft})}{140 (2.68 \text{ in})}\right]^{\wedge}2\right]$

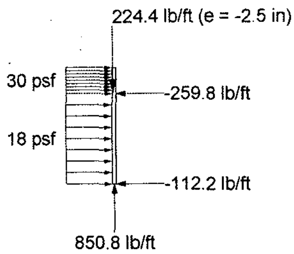
$= 14538$ lb / ft

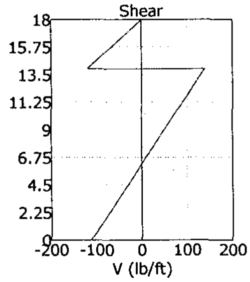
$P = 1758$ lb / ft $\leq$ $P_a = 14538$ lb / ft ✓

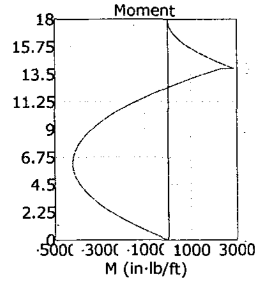
(000) 000-0000

Load Combination: 0.6D + 1.0W

Design Forces



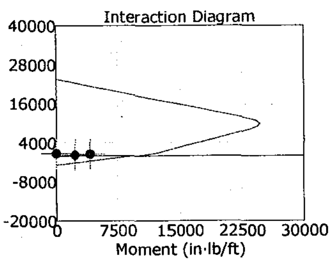




Factored Loads

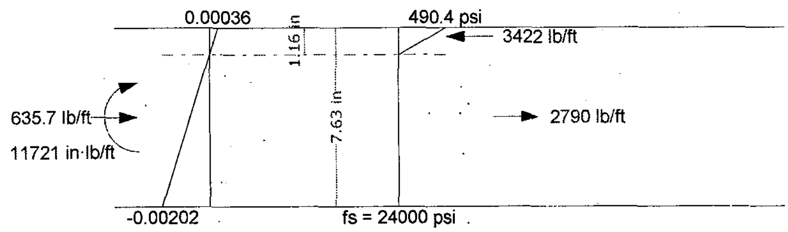
Wall Weight	626.4 lb/ft
Effective Eccentricity	-2.5 in
Applied Eccentricity	-2.5 in

Axial/Flexure Checks



Internal State at Max Moment Capacity for P = 635.7 lb/ft

TENSION controlled (fs = Fs = 24000 psi) k = 0.153



Combined Axial Flexure (@ base) [MSJC-05 2.3.3.2.2, 2.3.

$P = 850.8$ lb / ft

$M_a = 12235$ in·lb / ft (from interaction diagram given P)

$M = 0$ in·lb / ft $\leq$ $M_a = 12235$ in·lb / ft ✓

Combined Axial Flexure (@ max M) [MSJC-05 2.3.3.2.2, 2.

$P = 635.7$ lb / ft

$M_a = 11721$ in·lb / ft (from interaction diagram given P)

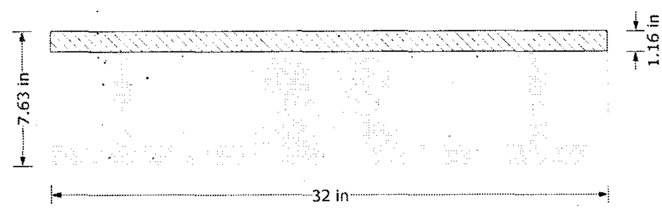
$M = 4196$ in·lb / ft $\leq$ $M_a = 11721$ in·lb / ft ✓

Combined Axial Flexure (@ top) [MSJC-05 2.3.3.2.2, 2.3.2.

$P = 363.6$ lb / ft

$M_a = 10845$ in·lb / ft (from interaction diagram given P)

$M = 2319$ in·lb / ft $\leq$ $M_a = 10845$ in·lb / ft ✓



Other Checks

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{d} = \frac{(139.8 \text{ lb / ft})}{(3.81 \text{ in})} = 3.06$ psi

$F_v = \sqrt{f'_m} = \sqrt{(1500 \text{ psi})} = 38.73$ psi ($\leq$ 50 psi)

$f_v = 3.06$ psi $\leq$ $F_v = 38.73$ psi ✓

Axial Compression (@ base) [MSJC-05 2.3.3.2.1]

$F_s = 24000$ psi (Grade 60 reinf)

$A_{st} = 0$ in² / in (bars are not tied)

$h / r = (14 \text{ ft}) / (2.68 \text{ in}) = 62.5871 \leq 99$

$$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s]\left[1 - \left[\frac{h}{140 r}\right]^\wedge 2\right]$$

$$= [0.25 (1500 \text{ psi}) (0.34 \text{ ft}^2 / \text{ft}) + 0.65 (0 \text{ in}^2 / \text{in}) (24000 \text{ psi})]\left[1 - \left[\frac{(14 \text{ ft})}{140 (2.68 \text{ in})}\right]^\wedge 2\right]$$

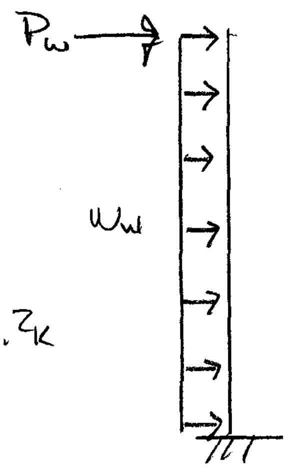
$= 14538$ lb / ft

$P = 850.8$ lb / ft $\leq$ $P_a = 14538$ lb / ft ✓

(W3) MECH YARD PILASTER

HT = 14 FT
HT WALL = 17 FT

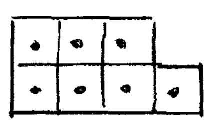
WIND = 25 PSF



$w_w = (25 \text{ PSF}) 2.67 \text{ FT} = 67 \text{ PLF}$
$P_w = 67 \text{ PLF} (3 \text{ FT}) = 200 \text{ LB}$

LINTEL LOAD:

$P_D = (0.100 \text{ KSF})(4 \text{ FT}/2)(11 \text{ FT}) = 2.2 \text{ K}$



12" CMU
7 CELLS W/ #5 BAR

PIER CONFIGURATION

→ 24" x 2'-8" PILASTER IS ADEQUATE W/ (1) #3 EA. CELL (SEE CALC)

Masonry Column

File: c:\Users\Art\Documents\HMW Projects\10050 Canyon View Elementary\Calculations\canyon view.ec6
ENERCALC, INC. 1983-2011, Ver: 6.2.00

Lic. # : KW-06002641 Licensee : HOLBEN, MARTIN, & WHITE

Description : Mechanical Yard Pilaster

General Information

Calculations per ACI 530-05, IBC 2006, CBC 2007, ASCE 7-05

Material Properties

F'm	=	1,500.0 psi
Fr - Rupture	=	75.0 psi
Em = f'm *	=	900.0
Column Density	=	130.0 pcf
Rebar Grade	=	Grade 60
Fy - Yield	=	60000 psi
Fs - Allowable	=	24000 psi
E - Rebar	=	29,000.0 ksi
Load Combination	=	2006 IBC & ASCE 7-05

Column Data

Column width along X-X	=	23.625 in
Column depth along Y-Y	=	23.625 in
Longitudinal Bar Size	=	# 5.0
Bars per side at +Y & -Y	=	3.0
Bars per side at +X & -X	=	3.0
Cover from ties	=	3.50 in
Actual Edge to Bar Center	=	4.1875 in

Analysis Settings

Analysis Method	=	**Working Stress Design**
End Fixity Condition	=	Top Free, Bottom Fixed
Overall Column Height	=	14 ft
Construction Type		Solid Grouted Hollow Concrete Masonry
Tie Bar Size	=	# 3
Tie Bar Spacing	=	8.0 in

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 10 ft, K = 2.1
Y-Y (depth) axis : Fully braced against buckling along Y-Y Axis

Applied Loads

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

Column self weight included : 7,054.28 lbs * Dead Load Factor
AXIAL LOADS . . .
Lintel: Axial Load at 14.0 ft, D = 2.20 k
BENDING LOADS . . .
Wind: Lat. Uniform Load creating My-y, W = 0.0670 k/ft
Wind: Lat. Point Load at 10.0 ft creating My-y, W = 0.20 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS **Maximum Bending Stress Ratio =** **0.329** : 1

Load Combination	+0.60D+W+H
Location of max.above base	0.000 ft
At maximum location values are	
Axial - Applied	1.320 k
Axial - Allowable	4.060 k
Moment - Applied	8.566 k-ft
Moment - Allowable	26.032 k-ft

PASS **Reinforcing Area Check** *(ACI 530-05, Sec 2.1.6.4)*

As : Actual Reinforcement	2.480
Min: 0.0025 * An	1.395
Max: 0.04 * An	22.326

Dimensional Checks

Min. Width/Depth >= 8" *(ACI 530-05, Sec 2.1.6.1)*

PASS Overall Height / Min Dim <= 25 *(ACI 530-05, Sec 2.1.6.2)*

Maximum SERVICE Load Reactions . .

Top along X-X	0.000 k
Bottom along X-X	1.138 k

Maximum SERVICE Load Deflections . . .

Along x-x 0.021 in at 14.000 ft above base
for load combination : W Only

Compressive Strength 229.889 k *(ACI 530-05, Sec 2.3.3.2.1)*
Pa = (0.25 f'm An + 0.65 Ast Fs)

FAIL **Check Column Ties** *(ACI 530-05, Sec 2.1.6.5)*
Min. Tie Dia. = 1/4", # 3 bar provided
Max Tie Spacing = 10.00 in, Provided = 8.00 in

Load Combination Results

Load Combination	Maximum Bending Stress Ratios			Maximum Axial Load		Maximum Moments	
	Stress Ratio	Status	Location	Actual	Allow	Actual	Allow
+D	0.03990	PASS	0.09396 ft	9.254 k	229.88 k	0.0 k-ft	30.938 k-ft
+D+W+H	0.3018	PASS	0.0 ft	2.20 k	7.321 k	8.566 k-ft	28.373 k-ft
+D+0.750Lr+0.750L+0.750W+H	0.2094	PASS	0.0 ft	2.20 k	10.573 k	6.425 k-ft	30.653 k-ft
+D+0.750L+0.750S+0.750W+H	0.2094	PASS	0.0 ft	2.20 k	10.573 k	6.425 k-ft	30.653 k-ft
+0.60D+W+H	0.3290	PASS	0.0 ft	1.320 k	4.060 k	8.566 k-ft	26.032 k-ft

Maximum Reactions - Unfactored

Note: Only non-zero reactions are listed.

Load Combination	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base
D Only	k	k	2.200 k
W Only	-1.138 k	k	k
D+W	-1.138 k	k	2.200 k

MARTIN
HOLBEN & WHITE

Holben, Martin & White
Consulting Structural Engineers
2950 N. Country Club Rd.
Tucson, Arizona 85716

Title : Canyon View Elementary MPR Expansion   Job # 10050
Dsgnr:
Project Desc.:

Project Notes :

56

Masonry Column

File: c:\Users\Art\Documents\HMW Projects\10050 Canyon View Elementary\Calculations\canyon view.ec6
ENERCALC, INC. 1983-2011, Ver: 6.2.00

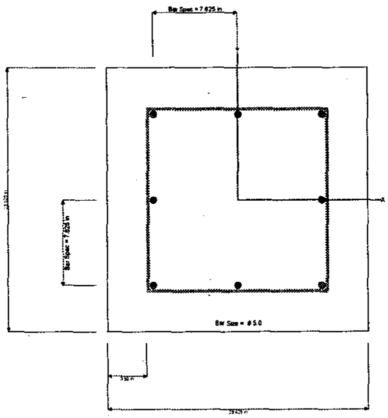
Lic. # : KW-06002641   **Licensee : HOLBEN, MARTIN, & WHITE**

Description : Mechanical Yard Pilaster

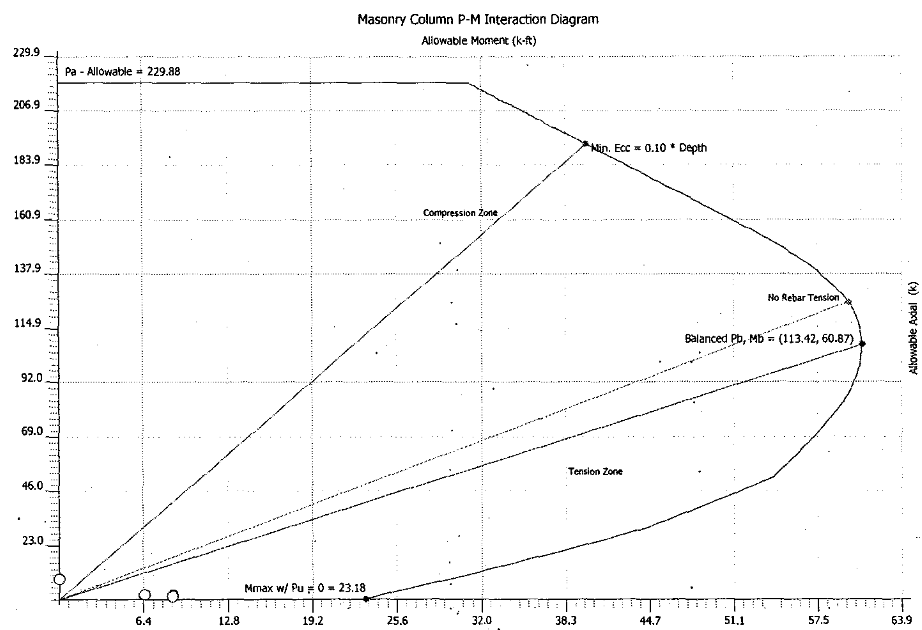
Maximum Deflections for Load Combinations - Unfactored Loads

Load Combination	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft
W Only	0.0210 in	14.000 ft
D+W	0.0208 in	13.906 ft

Cross Section

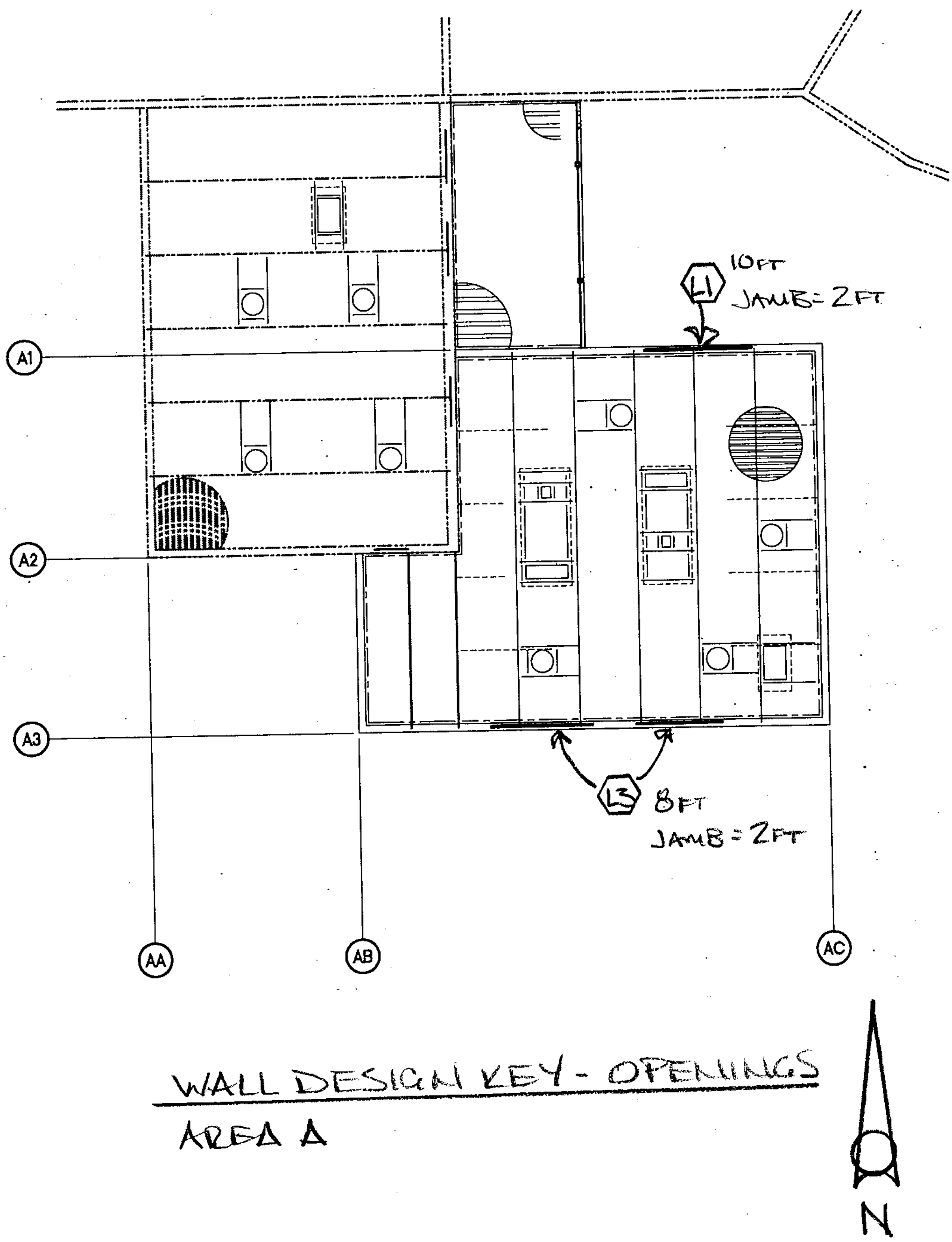


Interaction Diagram



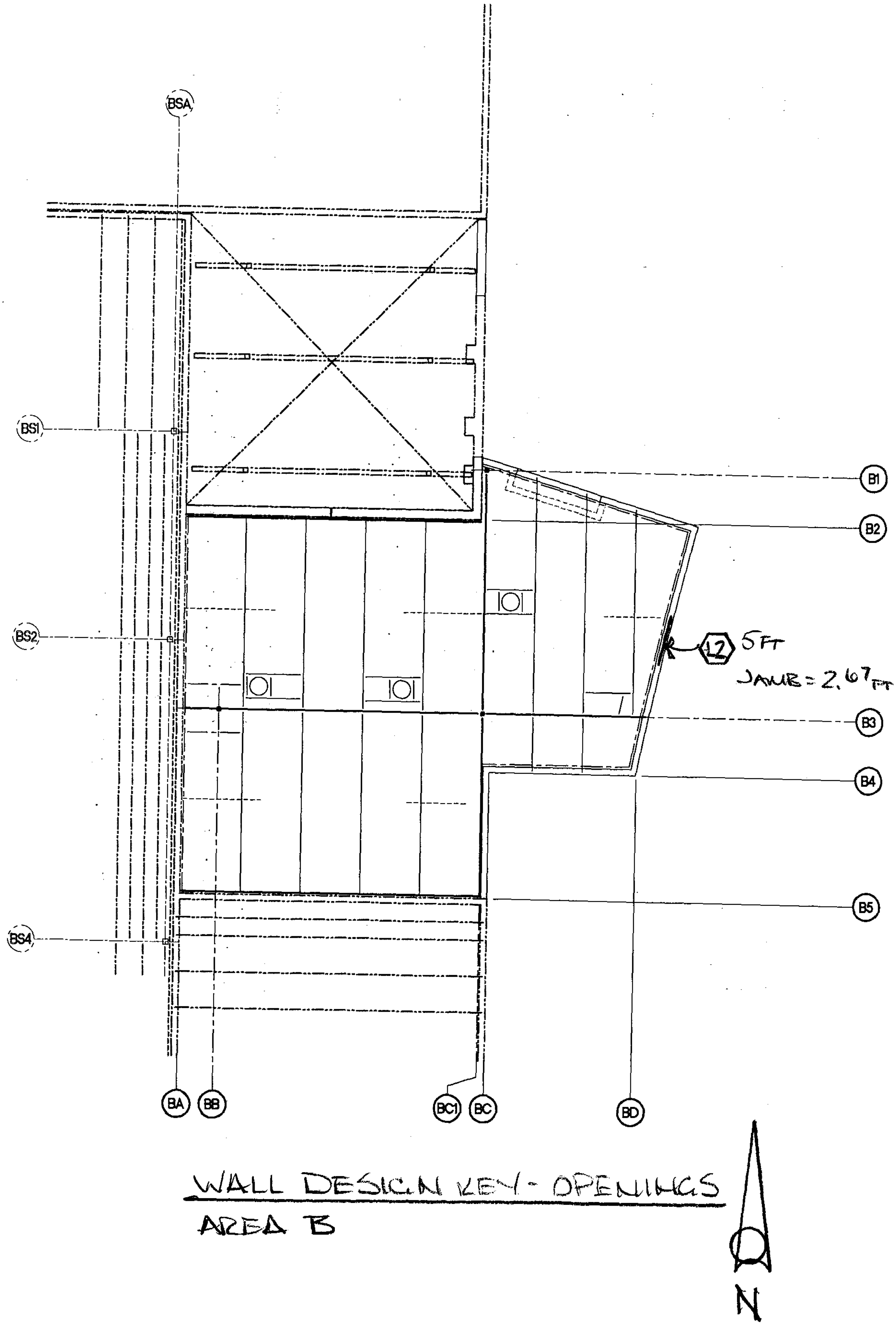
CFSD HS KIT EXP
10048

57



WALL DESIGN KEY - OPENINGS

AREA A



WALL DESIGN KEY - OPENINGS
AREA B

(4) MASONRY LINTEL $L = 10$ FT 8" CMU

$TH_{MAS} = 12$ FT

$TW_{RF} = 34/2 = 17$ FT

ROOF LOADS: $w_{D_r} = (22\,PSF)\,17\,FT = 374\,PLF$

$w_{L_r} = (20\,PSF)(17) = 340\,PLF$

WALL LOADS: $w_D = (75\,PSF)(12\,FT) = 900\,PLF$

→ USE 40" DP. LINTEL w/ (2) #5 BOTT & #4 STIR. @8"
(SEE CALC)

JAMB $L = 2.^{67}$ FT HT_{RF} 20 FT $HT_{LIN} = 8$ FT

ROOF LOADS: $P_{D_r} = 374\,(10/2) = 1870\,LB$

$P_{L_r} = 340(10/2) = 1700\,LB$

WALL LOADS: $P_D = 900(10/2) = 4500\,LB$

LATERAL: WIND = 18 PSF

$w_w = (18\,PSF)(8/2 + 12/2)(10/2)/2.^{0}\,FT = 450\,PLF$ @ 8 FT

→ USE 8" CMU w/ 8 #5 BARS, 2 IN EA. OF 4 CELLS
(SEE CALC)

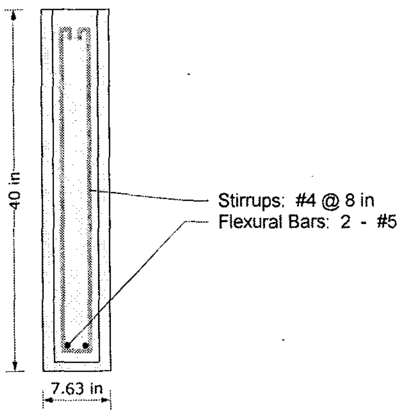
Art Cantrell
Holben, Martin _White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

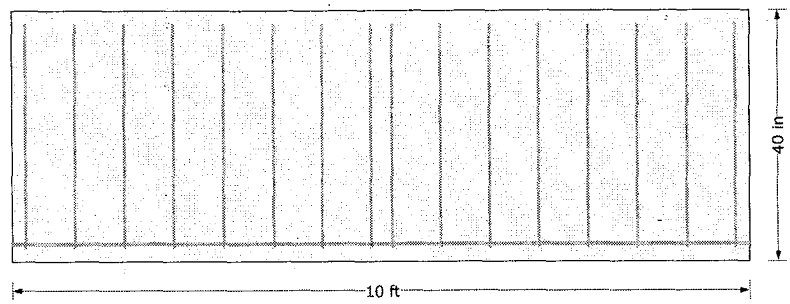
Lintel 1
CFSD HS Kitchen Expansion

Job # 10048

60

Design Detail





Check Summary

Ratio	Check	Provided	Required	Combination
----- Details -----				
✓ 0.455	Bar Clear Spacing	0.63 in	1.38 in	N/A
✓ 0.600	Bar Cover	2.5 in	1.5 in	N/A
----- Strength -----				
✓ 0.245	Shear	116.2 psi	28.46 psi	1.0D + 1.0L
✓ 0.484	Flexure	41.67 ft·k	20.17 ft·k	1.0D + 1.0L
✓ 0.109	Deflection	0.2 in	0.02 in	1.0D + 1.0L

Criteria

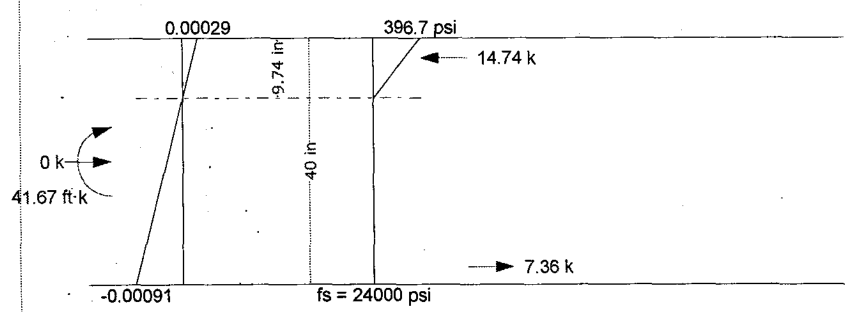
Building Code	MSCJ-05 (ASD)
Load Combination	ASCE 7-05 (ASD)
Seismic R Value	3.50
f'm	1500 psi
fy	60000 psi
Average Unit Weight	100 lb/ft³
Design As Clay Masonry	No
Exposed To Earth Or Weather	Yes
Take Shear at 'd/2' From Support	No
Enforce Deflection Check	Yes
Include Beam Self-Weight	No
Include Weight From Wall Above	No
Allow Arching Action	No
Point Load Dispersion Angle	30°
Uniform Load Dispersion Angle	45°

Load Combinations

ASCE 7-05 (ASD)
1.0D + 1.0L
1.0D
1.0D + 0.75L
0.6D

Internal State At Max Moment Capacity

Internal State at Max Moment Capacity
TENSION controlled (fs = Fs = 24000 psi) k = 0.244



Notes

Notes
➔ Lateral support requirements of MSJC-02 2.3.3.4.4 are not checked.
➔ Bearing length should be at least 4 inches in the direction of span (MSJC-02 2.3.3.4.3).
➔ Assumes bars are not epoxy coated.

Loads Summary

Load Set	Source	Uniform Load	Point Load	Pt Load Offset From Center
Roof	Dead	374 lb/ft	0 k	0 ft
Roof	Live	340 lb/ft	0 k	0 ft
Wall	Dead	900 lb/ft	0 k	0 ft

61

Detail Checks

Bar Clear Spacing [MSJC-05 1.12.3.1]

d_b = 0.63 in < 1.0

s_{clear} = 1.38 in ≥ s_{clear_min} = 0.63 in ✓

Bar Cover [MSJC-05 1.12.4.1]

cover = 2.5 in ≥ $cover_{min}$ = 1.5 in (Exposed to earth or weather, bars No. 5 or smaller)

Development/Splice Lengths

Bar Development/Splice Lengths [MSJC-05 2.1.10]

$$l_d = \frac{0.13\, d_b \,^\wedge 2\, f_y\, \gamma}{K\sqrt{f'_m}}$$

$$= \frac{0.13\,(0.63\text{ in})\,^\wedge 2\,(60000\text{ psi})\,(1.0)}{(1.38\text{ in})\sqrt{1500\text{ psi}}}$$

$$= 57.21\text{ in}$$

Stirrup Development/Splice Lengths [MSJC-05 2.1.10]

$$l_d = \frac{0.13\, d_b \,^\wedge 2\, f_y\, \gamma}{K\sqrt{f'_m}}$$

$$= \frac{0.13\,(0.5\text{ in})\,^\wedge 2\,(60000\text{ psi})\,(1.0)}{(2.5\text{ in})\sqrt{1500\text{ psi}}}$$

$$= 20.14\text{ in}$$

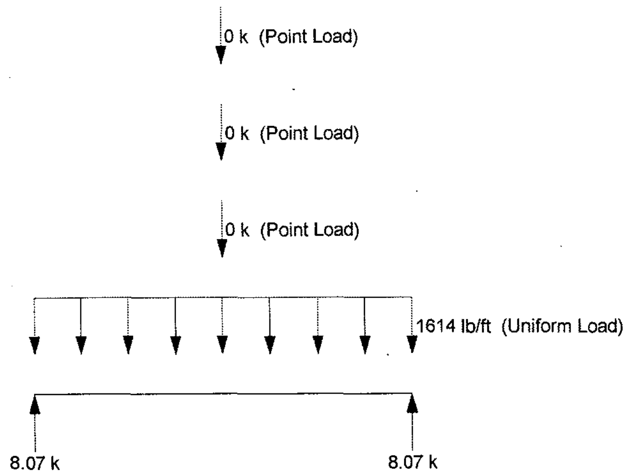
Eff_Embed = 0.5 l_d = 0.5 (20.14 in) = 10.0698 (when ended in a standard hook)

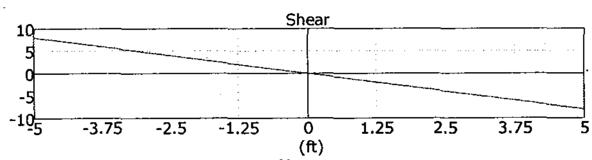
Eff_Embed = 0.33 l_d = 0.33 (20.14 in) = 6.6460 (when hooked 135 deg around a flexural bar, No. 5 & smaller only)

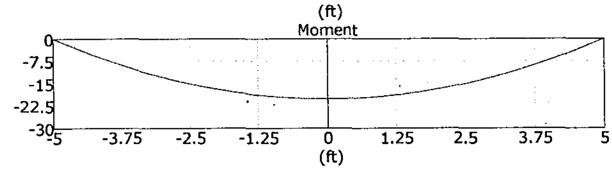
(000) 000-0000

Load Combination: 1.0D + 1.0L

Design Forces







Factored Loads	
Uniform Load	1614 lb/ft
Self Weight	0 lb/ft
Wall Above Peak Weight	0 lb/ft

Checks

Flexure [MSJC-05 2.3.3.2.2, 2.3.2.1]

M_a = 41.67 ft·k

M = 20.17 ft·k ≤ M_a = 41.67 ft·k ✓

Shear [MSJC-05 2.3.5]

$f_v = \dfrac{V}{b\,d} = \dfrac{(8.07\text{ k})}{(7.63\text{ in})(37.19\text{ in})}$ = 28.46 psi

A_v = V s / F_s d = (8.07 k)(8 in) / (24000 psi)(37.19 in) = 0.07 in²

A_{v_prov} = 0.4 in² ≥ A_v = 0.07 in²

Provided shear reinf. is sufficient; masonry stresses must still satisfy 2.3.5.2.3

$F_v = 3\sqrt{f'_m} = 3\sqrt{(1500\text{ psi})}$ = 116.2 psi (≤ 150 psi)

f_v = 28.46 psi ≤ F_v = 116.2 psi ✓

Deflection [MSJC-05 1.10.1]

n = E_s / E_m = (29000000 psi) / (1350000 psi) = 21.4815

$I_{cr} = \dfrac{b\,[k\,d]^3}{3} + n\,A_s\,[d - k\,d]^2$

$= \dfrac{(7.63\text{ in})\,[(0.2436)(37.19\text{ in})]^3}{3} + (21.4815)(0.61\text{ in}^2)\,[(37.19\text{ in}) - (0.2436)(37.19\text{ in})]^2$

= 12318 in^4

$I_g = \dfrac{b\,d_T^3}{12} = \dfrac{(7.63\text{ in})(40\text{ in})^3}{12}$ = 40667 in^4 (just for comparison)

$\delta = \dfrac{5\,M_{ser}\,L^2}{48\,E_m\,I_{cr}} = \dfrac{5\,(20.17\text{ ft·k})\,(10\text{ ft})^2}{48\,(1350000\text{ psi})\,(12318\text{ in}^4)}$ = 0.02 in

L / 600 = (10 ft) / 600 = 0.2 in ≤ 0.3

δ = 0.02 in ≤ δ_a = 0.2 in ✓

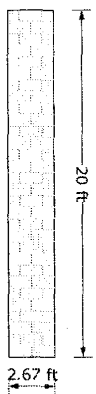
Art Cantrell
Holben, Martin _White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

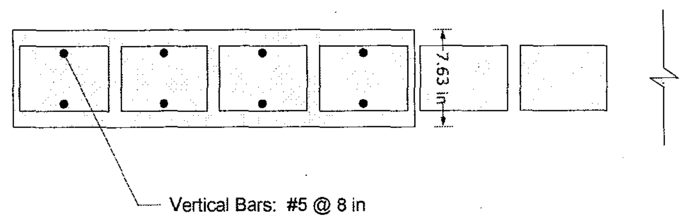
Jamb 1
CFSD HS Kitchen Expansion

Job # 10048

63

Design Detail

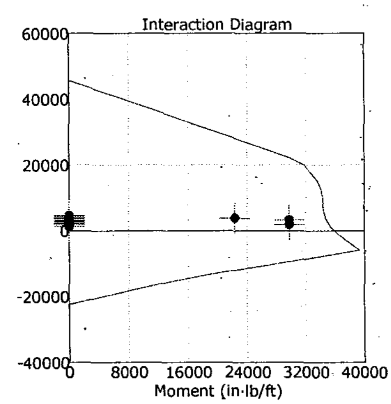




Check Summary

Ratio	Check	Provided	Required	Combination
----- Strength Checks -----				
✓ 0.333	Axial Compression	1.18 k	0.39 k	1.0D + 1.0L
✓ 0.000	Axial Tension	24 k	0 k	1.0D + 1.0L
✓ 0.141	Shear	38.73 psi	5.48 psi	1.0D + 1.0W
✓ 0.855	Axial+Flexure	34849 in·lb/ft	29779 in·lb/ft	1.0D + 1.0W

Interaction Diagram



Criteria

Building Code	MSCJ-05 (ASD)
Load Combination	ASCE 7-05 (ASD)
Seismic R Value	3.50
Amplify Axial Stress For Slenderness	Yes
f'm	1500 psi
fy	60000 psi
Specify Wall Weight Manually	No
Block Weight	Normal weight
Design As Clay Masonry	No
Include Wall Self-Weight	Yes
Neglect Lateral Load on Parapet	No
Include Wall Wt In Virtual Eccentricity	No
Always use I-cracked	No

Load Combinations

ASCE 7-05 (ASD)

1.0D + 1.0L
1.0D + 1.0W
1.0D
1.0D + 0.75L + 0.75W
1.0D + 0.75L
0.6D + 1.0W
0.6D

Loads Summary

Load Set	Source	Axial Unifo...	Axial Pt Lo...	Pt Ld Eff W...	Eccentricity	Lateral Pre...	Top Lateral...	Parapet Pr...	Lateral Uni...	Lat Unif Ld...	Moment
Roof	Dead	0 lb/ft	1.87 k	2.67 ft	0 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft
Roof	Live	0 lb/ft	1.7 k	2.67 ft	0 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft
Wind	Wind	0 lb/ft	0 k	1 ft	0 in	18 psf	18 psf	18 psf	337 lb/ft	8 ft	0 in·lb/ft
Wall	Dead	0 lb/ft	4.5 k	2.67 ft	0 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft

Art Cantrell
Holben, Martin .White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

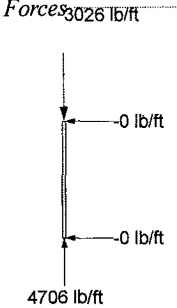
Jamb 1
CFSD HS Kitchen Expansion

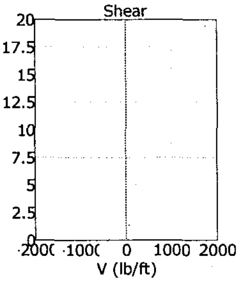
Job # 10048

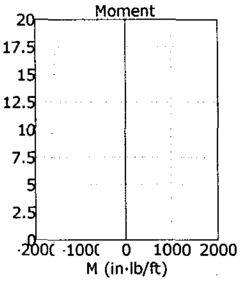
64

Load Combination: 1.0D + 1.0L

Design Forces



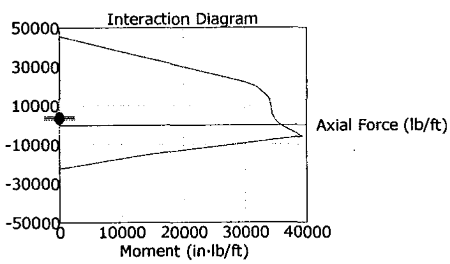




Factored Loads

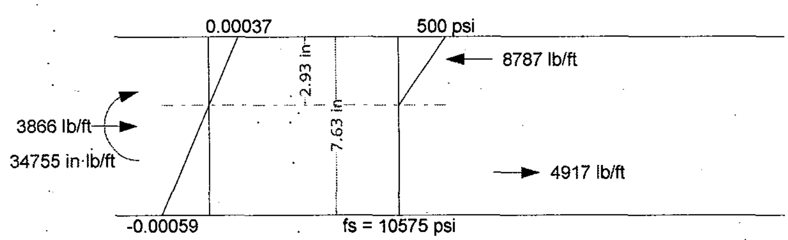
Wall Weight	1680 lb/ft
Effective Eccentricity	0 in
Applied Eccentricity	-0 in

Axial/Flexure Checks



Internal State at Max Moment Capacity for P = 3866 lb/ft

COMPRESSION controlled (fm = Fm = 500 psi) k = 0.384



Combined Axial Flexure (@ base) [MSJC-05 2.3.3.2.2, 2.3.

$P = 4706$ lb / ft

$M_a = 34639$ in·lb / ft (from interaction diagram given P)

$M = 0$ in·lb / ft $\leq$ $M_a = 34639$ in·lb / ft ✓

Combined Axial Flexure (@ max M) [MSJC-05 2.3.3.2.2, 2.

$P = 3866$ lb / ft

$M_a = 34755$ in·lb / ft (from interaction diagram given P)

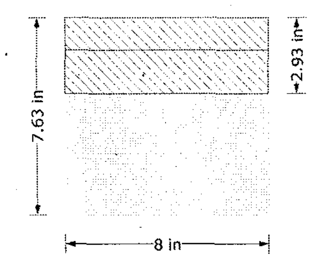
$M = 0$ in·lb / ft $\leq$ $M_a = 34755$ in·lb / ft ✓

Combined Axial Flexure (@ top) [MSJC-05 2.3.3.2.2, 2.3.2.

$P = 3026$ lb / ft

$M_a = 34938$ in·lb / ft (from interaction diagram given P)

$M = 0$ in·lb / ft $\leq$ $M_a = 34938$ in·lb / ft ✓



Other Checks

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{d} = \frac{(0 \text{ lb / ft})}{(5.81 \text{ in})} = 0$ psi

$F_v = \sqrt{f'_m} = \sqrt{(1500 \text{ psi})} = 38.73$ psi ($\leq$ 50 psi)

$f_v = 0$ psi $\leq$ $F_v = 38.73$ psi ✓

Axial Compression (@ base) [MSJC-05 2.3.3.2.1]

$F_s = 24000$ psi (Grade 60 reinf)

$A_{st} = 0$ in² / in (bars are not tied)

$h / r = (20 \text{ ft}) / (2.2 \text{ in}) = 109.0089 > 99$

$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s] \left[\frac{70 r}{h}\right]$ ^ 2

$= [0.25 (1500 \text{ psi}) (0.63 \text{ ft}^2 / \text{ft}) + 0.65 (0 \text{ in}^2 / \text{in}) (24000 \text{ psi})] \left[\frac{70 (2.2 \text{ in})}{(20 \text{ ft})}\right]$ ^ 2

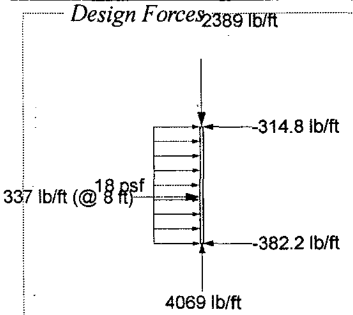
$= 14137$ lb / ft

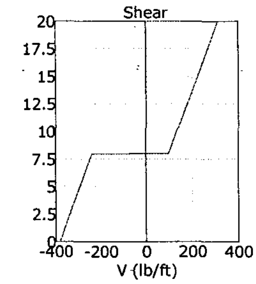
$P = 4706$ lb / ft $\leq$ $P_a = 14137$ lb / ft ✓

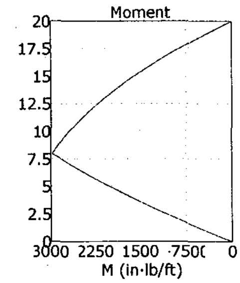
Art Cantrell Holben, Martin _White 123 Easy Street AnyTown, USA 00000 (000) 000-0000	Jamb 1 CFSD HS Kitchen Expansion	Job # 10048 65

Load Combination: 1.0D + 1.0W

Design Forces

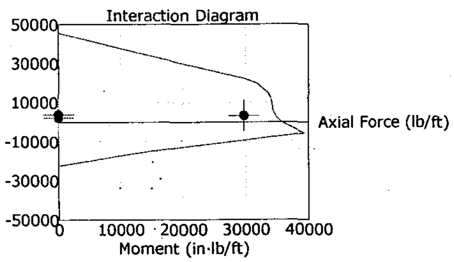






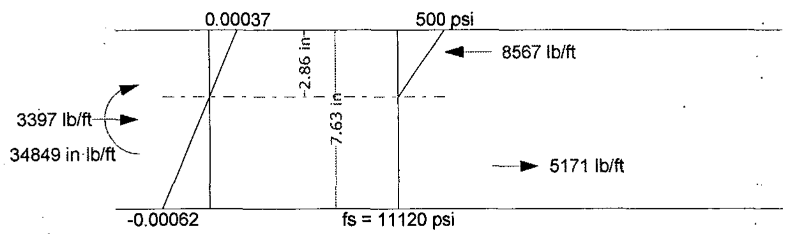
Factored Loads	
Wall Weight	1680 lb/ft
Effective Eccentricity	0 in
Applied Eccentricity	-0 in

Axial/Flexure Checks



Internal State at Max Moment Capacity for P = 3397 lb/ft

COMPRESSION controlled (fm = Fm = 500 psi) k = 0.375



Combined Axial Flexure (@ base) [MSJC-05 2.3.3.2.2, 2.3.

$P = 4069$ lb / ft

$M_a = 34726$ in·lb / ft (from interaction diagram given P)

$M = 0$ in·lb / ft $\leq$ $M_a = 34726$ in·lb / ft ✓

Combined Axial Flexure (@ max M) [MSJC-05 2.3.3.2.2, 2.

$P = 3397$ lb / ft

$M_a = 34849$ in·lb / ft (from interaction diagram given P)

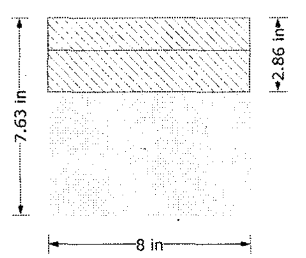
$M = 29779$ in·lb / ft $\leq$ $M_a = 34849$ in·lb / ft ✓

Combined Axial Flexure (@ top) [MSJC-05 2.3.3.2.2, 2.3.2.

$P = 2389$ lb / ft

$M_a = 35104$ in·lb / ft (from interaction diagram given P)

$M = 0$ in·lb / ft $\leq$ $M_a = 35104$ in·lb / ft ✓



Other Checks

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{d} = \frac{(382.2\ \text{lb / ft})}{(5.81\ \text{in})} = 5.48$ psi

$F_v = \sqrt{f'_m} = \sqrt{(1500\ \text{psi})} = 38.73$ psi ($\leq$ 50 psi)

$f_v = 5.48$ psi $\leq$ $F_v = 38.73$ psi ✓

Axial Compression (@ base) [MSJC-05 2.3.3.2.1]

$F_s = 24000$ psi (Grade 60 reinf)

$A_{st} = 0$ in² / in (bars are not tied)

$h / r = (20\ \text{ft}) / (2.2\ \text{in}) = 109.0089 > 99$

$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s] \left[\frac{70 r}{h}\right] \wedge 2$

$= [0.25 (1500\ \text{psi}) (0.63\ \text{ft}^2 / \text{ft}) + 0.65 (0\ \text{in}^2 / \text{in}) (24000\ \text{psi})] \left[\frac{70 (2.2\ \text{in})}{(20\ \text{ft})}\right] \wedge 2$

$= 14137$ lb / ft

$P = 4069$ lb / ft $\leq$ $P_a = 14137$ lb / ft ✓

(12) MASONRY LINTEL $L = 5$ FT 8" CMU

$TH_{MAS} = 8$ FT

$TW_{RF} = 24/2 = 12$ FT

ROOF LOADS: $w_{Dr} = (22\text{PSF})\,12\text{FT} = 264$ PLF

$w_{Lr} = (20\text{PSF})\,12\text{FT} = 240$ PLF

WALL LOADS: $w_D = (75\text{PSF})(8\text{FT}) = 600$ PLF

→ USE 24" DP. LINTEL w/ (2) #5 BOTT (SEE CALC)

JAMB $L = 2.67$ FT HT RF = 14 FT HT LIN = 9 FT

ROOF LOADS: $P_{Dr} = 264\,(5/2) = 660$ LB

$P_{Lr} = 240\,(5/2) = 600$ LB

WALL LOADS: $P_D = 600\,(5/2) = 1500$ LB

LATERAL: $w_w = (18\text{PSF})(9/2 + 5/2)(5/2)/2.67\text{FT} = 118$ PLF @ 9 FT

→ USE 8" CMU w/ 4 #5 BARS, 2 IN EA OF 2 CELLS (SEE CALC)

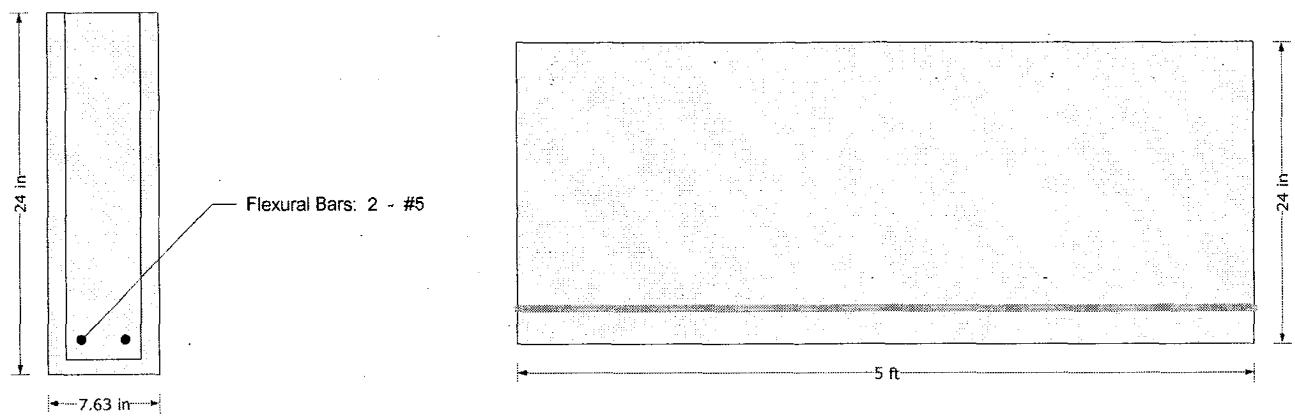
Art Cantrell
Holben, Martin .White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

Lintel 2
CFSD HS Kitchen Expansion

Job # 10048

67

Design Detail



Check Summary

Ratio	Check	Provided	Required	Combination
----- Details -----				
✓ 0.263	Bar Clear Spacing	0.63 in	2.38 in	N/A
✓ 0.750	Bar Cover	2 in	1.5 in	N/A
----- Strength -----				
✓ 0.431	Shear	38.73 psi	16.69 psi	1.0D + 1.0L
✓ 0.158	Flexure	21.8 ft·k	3.45 ft·k	1.0D + 1.0L
✓ 0.031	Deflection	0.1 in	0 in	1.0D + 1.0L

Criteria

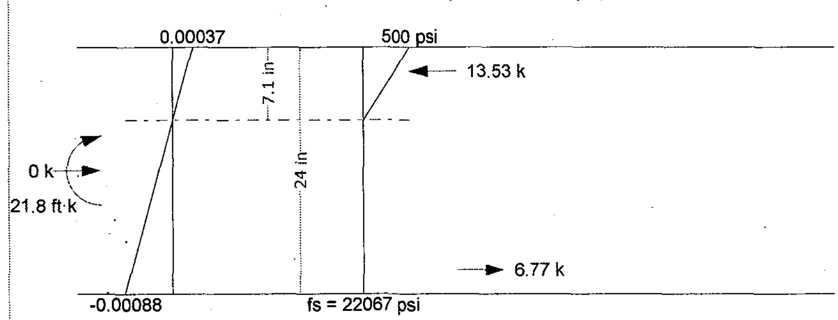
Building Code	MSCJ-05 (ASD)
Load Combination	ASCE 7-05 (ASD)
Seismic R Value	3.50
f'm	1500 psi
fy	60000 psi
Average Unit Weight	100 lb/ft³
Design As Clay Masonry	No
Exposed To Earth Or Weather	Yes
Take Shear at 'd/2' From Support	No
Enforce Deflection Check	Yes
Include Beam Self-Weight	No
Include Weight From Wall Above	No
Allow Arching Action	No
Point Load Dispersion Angle	30°
Uniform Load Dispersion Angle	45°

Load Combinations

ASCE 7-05 (ASD)
1.0D + 1.0L
1.0D
1.0D + 0.75L
0.6D

Internal State At Max Moment Capacity

Internal State at Max Moment Capacity
COMPRESSION controlled (fm = Fm = 500 psi) k = 0.296



Notes

Notes
➔ Lateral support requirements of MSJC-02 2.3.3.4.4 are not checked.
➔ Bearing length should be at least 4 inches in the direction of span (MSJC-02 2.3.3.4.3).
➔ Assumes bars are not epoxy coated.

Loads Summary

Load Set	Source	Uniform Load	Point Load	Pt Load Offset From Center
Roof	Dead	264 lb/ft	0 k	0 ft
Roof	Live	240 lb/ft	0 k	0 ft
Wall	Dead	600 lb/ft	0 k	0 ft

Detail Checks

Bar Clear Spacing [MSJC-05 1.12.3.1]

d_b = 0.63 in < 1.0

s_{clear} = 2.38 in ≥ s_{clear_min} = 0.63 in ✓

Bar Cover [MSJC-05 1.12.4.1]

cover = 2 in ≥ $cover_{min}$ = 1.5 in (Exposed to earth or weather, bars No. 5 or smaller) ✓

Development/Splice Lengths

Bar Development/Splice Lengths [MSJC-05 2.1.10]

$$l_d = \frac{0.13\, d_b \wedge 2\, f_y\, \gamma}{K\sqrt{f'_m}}$$

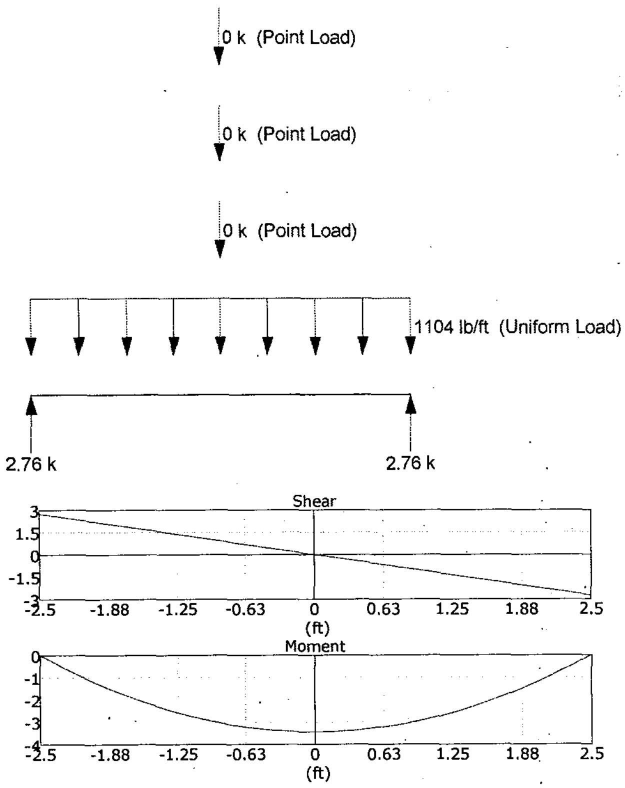
$$= \frac{0.13\,(0.63\text{ in}) \wedge 2\,(60000\text{ psi})\,(1.0)}{(2\text{ in})\sqrt{1500\text{ psi}}}$$

$$= 39.33\text{ in}$$

(000) 000-0000

Load Combination: 1.0D + 1.0L

Design Forces



Factored Loads	
Uniform Load	1104 lb/ft
Self Weight	0 lb/ft
Wall Above Peak Weight	0 lb/ft

Checks

Flexure [MSJC-05 2.3.3.2.2, 2.3.2.1]

M_a = 21.8 ft·k

M = 3.45 ft·k ≤ M_a = 21.8 ft·k ✓

Shear [MSJC-05 2.3.5]

$f_v = \dfrac{V}{b\,d} = \dfrac{(2.76\text{ k})}{(7.63\text{ in})(21.69\text{ in})}$ = 16.69 psi

$F_v = \sqrt{f'_m} = \sqrt{(1500\text{ psi})}$ = 38.73 psi (≤ 50 psi)

f_v = 16.69 psi ≤ F_v = 38.73 psi ✓

Deflection [MSJC-05 1.10.1]

$n = E_s / E_m$ = (29000000 psi) / (1350000 psi) = 21.4815

$I_{cr} = \dfrac{b\,[k\,d]\,^\wedge 3}{3} + n\,A_s\,[d - k\,d]\,^\wedge 2$

$= \dfrac{(7.63\text{ in})\,[(0.2958)\,(21.69\text{ in})]\,^\wedge 3}{3} + (21.4815)\,(0.61\text{ in}^2)\,[(21.69\text{ in}) - (0.2958)\,(21.69\text{ in})]\,^\wedge 2$

= 3745 in ^ 4

$I_g = \dfrac{b\,d_T\,^\wedge 3}{12} = \dfrac{(7.63\text{ in})\,(24\text{ in})\,^\wedge 3}{12}$ = 8784 in ^ 4 (just for comparison)

$\delta = \dfrac{5\,M_{ser}\,L\,^\wedge 2}{48\,E_m\,I_{cr}} = \dfrac{5\,(3.45\text{ ft·k})\,(5\text{ ft})\,^\wedge 2}{48\,(1350000\text{ psi})\,(3745\text{ in}\,^\wedge 4)}$ = 0 in

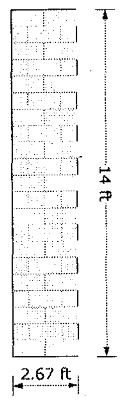
L / 600 = (5 ft) / 600 = 0.1 in ≤ 0.3

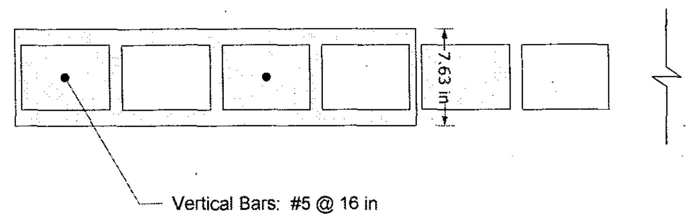
δ = 0 in ≤ δ_a = 0.1 in ✓

(000) 000-0000

70

Design Detail

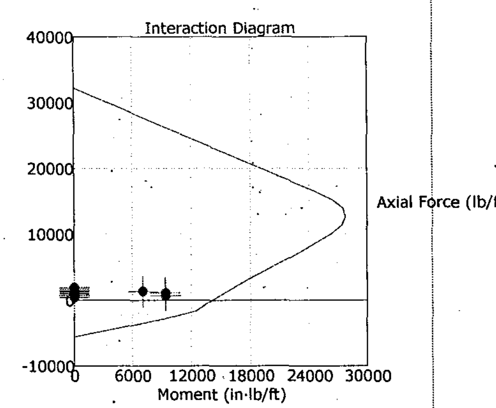




Check Summary

Ratio	Check	Provided	Required	Combination
----- Strength Checks -----				
✓ 0.107	Axial Compression	1.53 k	0.16 k	1.0D + 1.0L
✓ 0.000	Axial Tension	24 k	0 k	1.0D + 1.0L
✓ 0.114	Shear	38.73 psi	4.41 psi	1.0D + 1.0W
✓ 0.633	Axial+Flexure	14864 in·lb/ft	9411 in·lb/ft	0.6D + 1.0W

Interaction Diagram



Criteria

Building Code	MSCJ-05 (ASD)
Load Combination	ASCE 7-05 (ASD)
Seismic R Value	3.50
Amplify Axial Stress For Slenderness	Yes
f'm	1500 psi
fy	60000 psi
Specify Wall Weight Manually	No
Block Weight	Normal weight
Design As Clay Masonry	No
Include Wall Self-Weight	Yes
Neglect Lateral Load on Parapet	No
Include Wall Wt In Virtual Eccentricity	No
Always use I-cracked	No

Load Combinations

ASCE 7-05 (ASD)
1.0D + 1.0L
1.0D + 1.0W
1.0D
1.0D + 0.75L + 0.75W
1.0D + 0.75L
0.6D + 1.0W
0.6D

Loads Summary

Load Set	Source	Axial Unifo...	Axial Pt Lo...	Pt Ld Eff W...	Eccentricity	Lateral Pre...	Top Lateral...	Parapet Pr...	Lateral Uni...	Lat Unif Ld...	Moment
Roof	Dead	0 lb/ft	0.66 k	2.67 ft	0 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft
Roof	Live	0 lb/ft	0.6 k	2.67 ft	0 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft
Wind	Wind	0 lb/ft	0 k	1 ft	0 in	18 psf	18 psf	18 psf	118 lb/ft	9 ft	0 in·lb/ft
Wall	Dead	0 lb/ft	1.5 k	2.67 ft	0 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft

Art Cantrell
Holben, Martin White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

Jamb 2
CFSD HS Kitchen Expansion

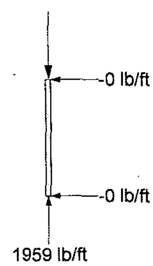
Job # 10048

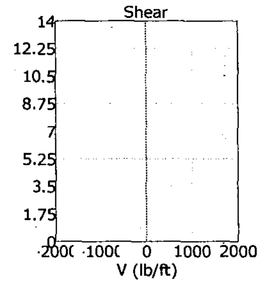
71

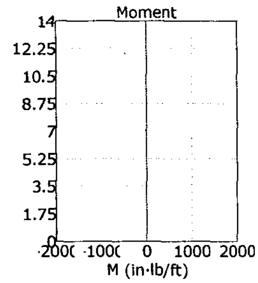
Load Combination: 1.0D + 1.0L

Design Forces

1035 lb/ft



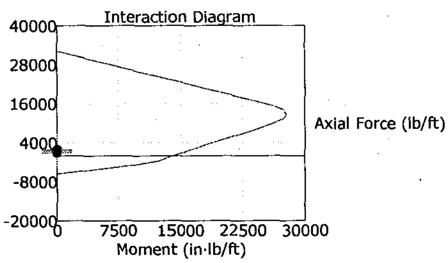




Factored Loads

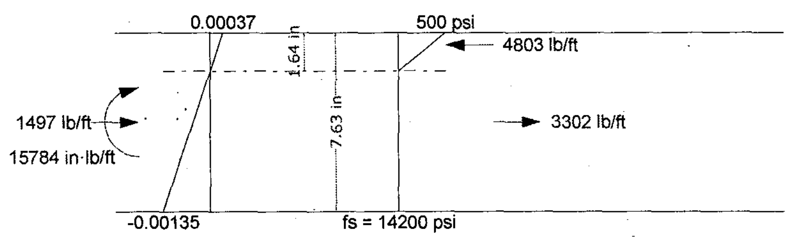
Wall Weight	924 lb/ft
Effective Eccentricity	0 in
Applied Eccentricity	-0 in

Axial/Flexure Checks



Internal State at Max Moment Capacity for P = 1497 lb/ft

COMPRESSION controlled (fm = Fm = 500 psi) k = 0.215



Combined Axial Flexure (@ base) [MSJC-05 2.3.3.2.2, 2.3.

$P = 1959$ lb / ft

$M_a = 16322$ in·lb / ft (from interaction diagram given P)

$M = 0$ in·lb / ft $\leq$ $M_a = 16322$ in·lb / ft ✓

Combined Axial Flexure (@ max M) [MSJC-05 2.3.3.2.2, 2.

$P = 1497$ lb / ft

$M_a = 15784$ in·lb / ft (from interaction diagram given P)

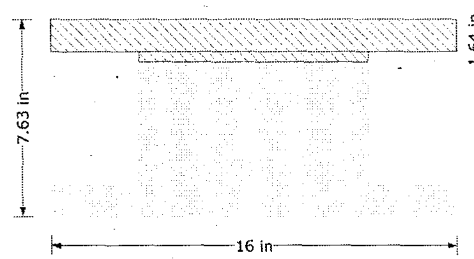
$M = 0$ in·lb / ft $\leq$ $M_a = 15784$ in·lb / ft ✓

Combined Axial Flexure (@ top) [MSJC-05 2.3.3.2.2, 2.3.2.

$P = 1035$ lb / ft

$M_a = 15258$ in·lb / ft (from interaction diagram given P)

$M = 0$ in·lb / ft $\leq$ $M_a = 15258$ in·lb / ft ✓



Other Checks

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{d} = \frac{(0 \text{ lb / ft})}{(3.81 \text{ in})} = 0$ psi

$F_v = \sqrt{f'_m} = \sqrt{(1500 \text{ psi})} = 38.73$ psi ($\leq$ 50 psi)

$f_v = 0$ psi $\leq$ $F_v = 38.73$ psi ✓

Axial Compression (@ base) [MSJC-05 2.3.3.2.1]

$F_s = 24000$ psi (Grade 60 reinf)

$A_{st} = 0$ in² / in (bars are not tied)

$h / r = (14 \text{ ft}) / (2.44 \text{ in}) = 68.850 \leq 99$

$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s]\left[1 - \left[\frac{h}{140 r}\right]^2\right]$

$= [0.25 (1500 \text{ psi}) (0.45 \text{ ft}^2 / \text{ft}) + 0.65 (0 \text{ in}^2 / \text{in}) (24000 \text{ psi})]\left[1 - \left[\frac{(14 \text{ ft})}{140 (2.44 \text{ in})}\right]^2\right]$

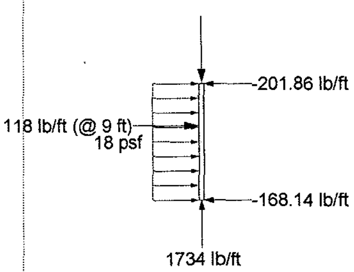
$= 18352$ lb / ft

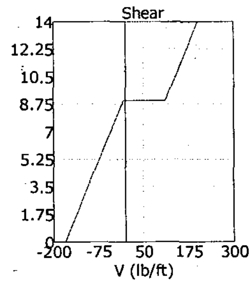
$P = 1959$ lb / ft $\leq$ $P_a = 18352$ lb / ft ✓

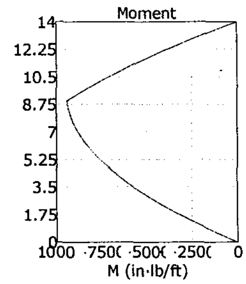
(000) 000-0000

Load Combination: 1.0D + 1.0W

Design Forces



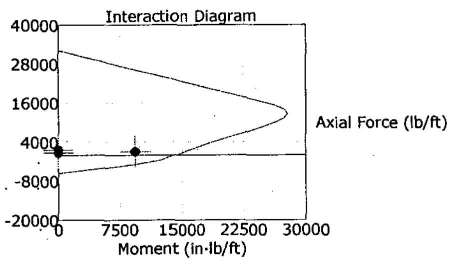




Factored Loads

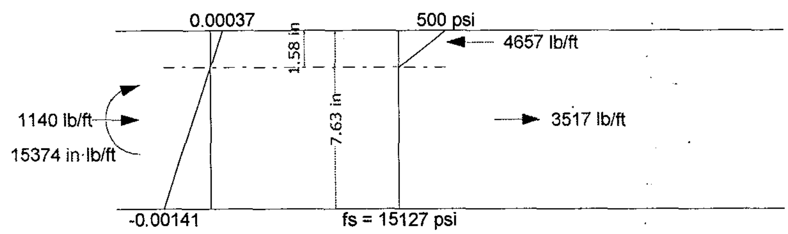
Wall Weight	924 lb/ft
Effective Eccentricity	0 in
Applied Eccentricity	-0 in

Axial/Flexure Checks



Internal State at Max Moment Capacity for P = 1140 lb/ft

COMPRESSION controlled (fm = Fm = 500 psi) k = 0.208



Combined Axial Flexure (@ base) [MSJC-05 2.3.3.2.2, 2.3.

$P = 1734 \text{ lb / ft}$

$M_a = 16060 \text{ in·lb / ft}$ (from interaction diagram given P)

$M = 0 \text{ in·lb / ft} \le M_a = 16060 \text{ in·lb / ft}$ ✓

Combined Axial Flexure (@ max M) [MSJC-05 2.3.3.2.2, 2.

$P = 1140 \text{ lb / ft}$

$M_a = 15374 \text{ in·lb / ft}$ (from interaction diagram given P)

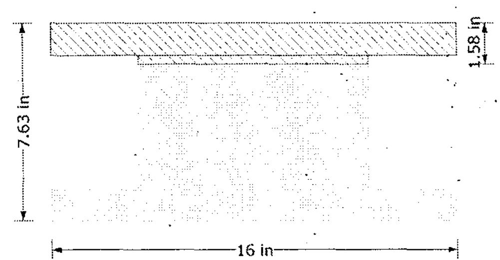
$M = 9411 \text{ in·lb / ft} \le M_a = 15374 \text{ in·lb / ft}$ ✓

Combined Axial Flexure (@ top) [MSJC-05 2.3.3.2.2, 2.3.2.

$P = 810 \text{ lb / ft}$

$M_a = 15008 \text{ in·lb / ft}$ (from interaction diagram given P)

$M = 0 \text{ in·lb / ft} \le M_a = 15008 \text{ in·lb / ft}$ ✓



Other Checks

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{d} = \frac{(201.9 \text{ lb / ft})}{(3.81 \text{ in})} = 4.41 \text{ psi}$

$F_v = \sqrt{f'_m} = \sqrt{(1500 \text{ psi})} = 38.73 \text{ psi}$ ($\le$ 50 psi)

$f_v = 4.41 \text{ psi} \le F_v = 38.73 \text{ psi}$ ✓

Axial Compression (@ base) [MSJC-05 2.3.3.2.1]

$F_s = 24000 \text{ psi}$ (Grade 60 reinf)

$A_{st} = 0 \text{ in}^2 / \text{in}$ (bars are not tied)

$h / r = (14 \text{ ft}) / (2.44 \text{ in}) = 68.850 \le 99$

$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s]\left[1 - \left[\frac{h}{140 r}\right]^2\right]$

$= [0.25 (1500 \text{ psi}) (0.45 \text{ ft}^2 / \text{ft}) + 0.65 (0 \text{ in}^2 / \text{in}) (24000 \text{ psi})]\left[1 - \left[\frac{(14 \text{ ft})}{140 (2.44 \text{ in})}\right]^2\right]$

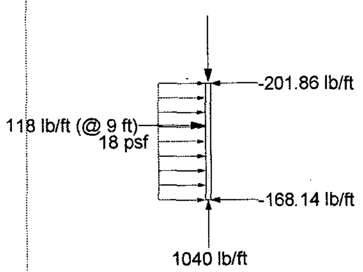
$= 18352 \text{ lb / ft}$

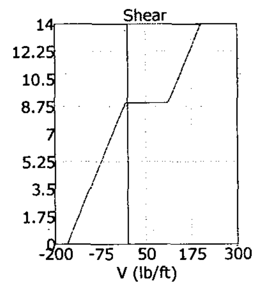
$P = 1734 \text{ lb / ft} \le P_a = 18352 \text{ lb / ft}$ ✓

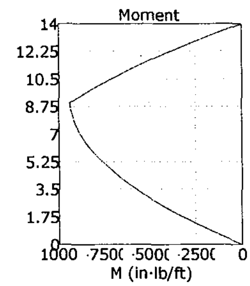
(000) 000-0000

Load Combination: 0.6D + 1.0W

Design Forces



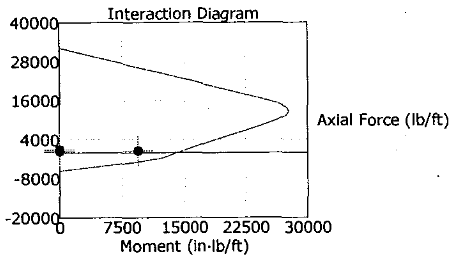




Factored Loads

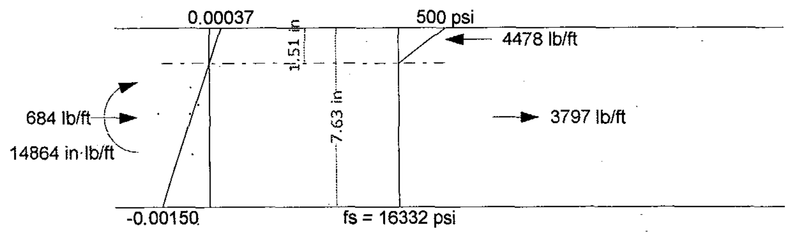
Wall Weight	554.4 lb/ft
Effective Eccentricity	0 in
Applied Eccentricity	-0 in

Axial/Flexure Checks



Internal State at Max Moment Capacity for P = 684 lb/ft

COMPRESSION controlled (fm = Fm = 500 psi) k = 0.198



Combined Axial Flexure (@ base) [MSJC-05 2.3.3.2.2, 2.3.

$P = 1040 \text{ lb/ft}$

$M_a = 15264 \text{ in·lb/ft}$ (from interaction diagram given P)

$M = 0 \text{ in·lb/ft} \le M_a = 15264 \text{ in·lb/ft}$ ✓

Combined Axial Flexure (@ max M) [MSJC-05 2.3.3.2.2, 2.

$P = 684 \text{ lb/ft}$

$M_a = 14864 \text{ in·lb/ft}$ (from interaction diagram given P)

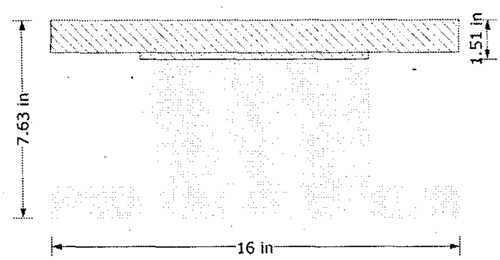
$M = 9411 \text{ in·lb/ft} \le M_a = 14864 \text{ in·lb/ft}$ ✓

Combined Axial Flexure (@ top) [MSJC-05 2.3.3.2.2, 2.3.2.

$P = 486 \text{ lb/ft}$

$M_a = 14651 \text{ in·lb/ft}$ (from interaction diagram given P)

$M = 0 \text{ in·lb/ft} \le M_a = 14651 \text{ in·lb/ft}$ ✓



Other Checks

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{d} = \frac{(201.9 \text{ lb/ft})}{(3.81 \text{ in})} = 4.41 \text{ psi}$

$F_v = \sqrt{f'_m} = \sqrt{(1500 \text{ psi})} = 38.73 \text{ psi}$ ($\le$ 50 psi)

$f_v = 4.41 \text{ psi} \le F_v = 38.73 \text{ psi}$ ✓

Axial Compression (@ base) [MSJC-05 2.3.3.2.1]

$F_s = 24000 \text{ psi}$ (Grade 60 reinf)

$A_{st} = 0 \text{ in}^2/\text{in}$ (bars are not tied)

$h / r = (14 \text{ ft}) / (2.44 \text{ in}) = 68.850 \le 99$

$P_a = [0.25 f'_m A_n + 0.65 A_{st} F_s]\left[1 - \left[\frac{h}{140 r}\right]^2\right]$

$= [0.25 (1500 \text{ psi}) (0.45 \text{ ft}^2/\text{ft}) + 0.65 (0 \text{ in}^2/\text{in}) (24000 \text{ psi})]\left[1 - \left[\frac{(14 \text{ ft})}{140 (2.44 \text{ in})}\right]^2\right]$

$= 18352 \text{ lb/ft}$

$P = 1040 \text{ lb/ft} \le P_a = 18352 \text{ lb/ft}$ ✓

(L3) MASONRY LINTEL $L = 8_{FT}$ 8" CMU
$TH_{MAS} = 9_{FT}$
$TW_{RF} = 34/2 = 17_{FT}$

ROOF LOADS: $w_{D1} = 374$ (FROM (L1))
$w_{L1} = 340$

WALL LOADS: $w_D = (75_{PSF})(9_{FT}) = 675_{PLF}$

→ USE 32" DP LINTEL w/ (2) #5 BOTT & #4 STIR. @16"
(SEE CALC)

JAMB $L = 2.0_{FT}$ $HT_{RF} = 20_{FT}$ $HT_{LIN} = 14_{FT}$

ROOF LOADS: $P_{D1} = 374(8/2) = 1496_{LB}$
$P_{L1} = 340(8/2) = 1360_{LB}$

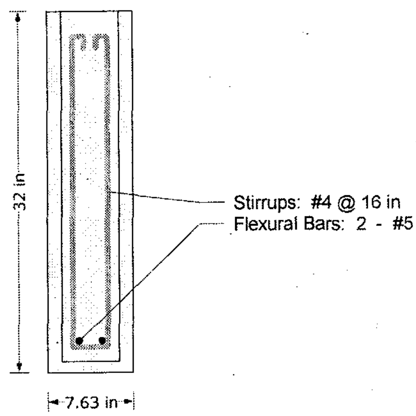
WALL LOADS: $P_D = 675(8/2) = 2700_{LB}$

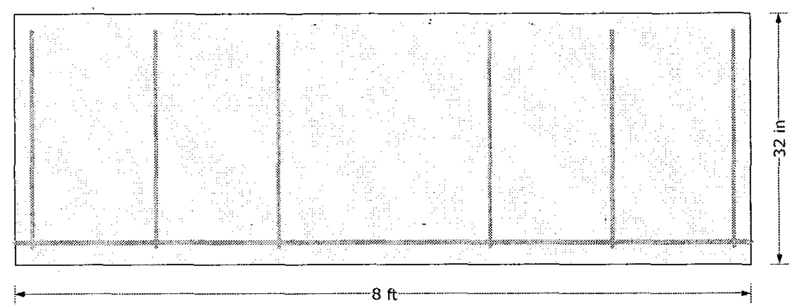
LATERAL: $w_w = (18_{PSF})(6/2 + 6/2)(8/2)/2.0_{FT} = 216_{PLF}$

→ USE 8" CMU w/ 6 #5 BARS, 2 IN. EA. OF 3 CELLS
(SEE CALC)

(000) 000-0000

Design Detail





Check Summary

Ratio	Check	Provided	Required	Combination
----- Details -----				
✓ 0.455	Bar Clear Spacing	0.63 in	1.38 in	N/A
✓ 0.600	Bar Cover	2.5 in	1.5 in	N/A
----- Strength -----				
✓ 0.215	Shear	116.2 psi	24.96 psi	1.0D + 1.0L
✓ 0.344	Flexure	32.34 ft·k	11.11 ft·k	1.0D + 1.0L
✓ 0.082	Deflection	0.16 in	0.01 in	1.0D + 1.0L

Criteria

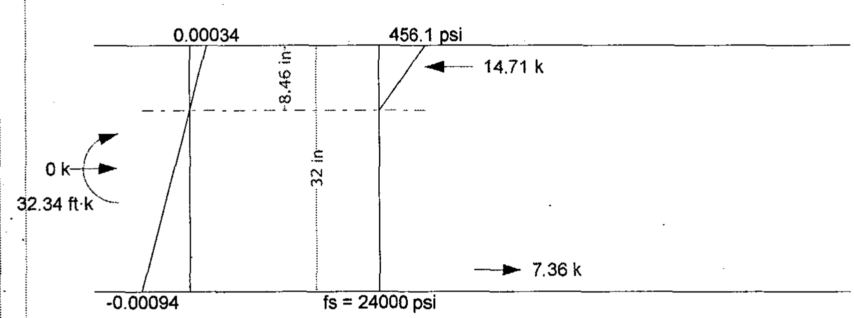
Building Code	MSCJ-05 (ASD)
Load Combination	ASCE 7-05 (ASD)
Seismic R Value	3.50
f'm	1500 psi
fy	60000 psi
Average Unit Weight	100 lb/ft³
Design As Clay Masonry	No
Exposed To Earth Or Weather	Yes
Take Shear at 'd/2' From Support	No
Enforce Deflection Check	Yes
Include Beam Self-Weight	No
Include Weight From Wall Above	No
Allow Arching Action	No
Point Load Dispersion Angle	30°
Uniform Load Dispersion Angle	45°

Load Combinations

ASCE 7-05 (ASD)
1.0D + 1.0L
1.0D
1.0D + 0.75L
0.6D

Internal State At Max Moment Capacity

Internal State at Max Moment Capacity
TENSION controlled (fs = Fs = 24000 psi) k = 0.264



Notes

Notes
➔ Lateral support requirements of MSJC-02 2.3.3.4.4 are not checked.
➔ Bearing length should be at least 4 inches in the direction of span (MSJC-02 2.3.3.4.3).
➔ Assumes bars are not epoxy coated.

Loads Summary

Load Set	Source	Uniform Load	Point Load	Pt Load Offset From Center
Roof	Dead	374 lb/ft	0 k	0 ft
Roof	Live	340 lb/ft	0 k	0 ft
Wall	Dead	675 lb/ft	0 k	0 ft

Detail Checks

Bar Clear Spacing [MSJC-05 1.12.3.1]

d_b = 0.63 in < 1.0

s_{clear} = 1.38 in ≥ s_{clear_min} = 0.63 in ✓

Bar Cover [MSJC-05 1.12.4.1]

cover = 2.5 in ≥ $cover_{min}$ = 1.5 in (Exposed to earth or weather, bars No. 5 or smaller) ✓

Development/Splice Lengths

Bar Development/Splice Lengths [MSJC-05 2.1.10]

$$l_d = \frac{0.13\, d_b \wedge 2\, f_y\, \gamma}{K \sqrt{f'_m}}$$

$$= \frac{0.13\,(0.63\text{ in}) \wedge 2\,(60000\text{ psi})\,(1.0)}{(1.38\text{ in})\sqrt{1500\text{ psi}}}$$

= 57.21 in

Stirrup Development/Splice Lengths [MSJC-05 2.1.10]

$$l_d = \frac{0.13\, d_b \wedge 2\, f_y\, \gamma}{K \sqrt{f'_m}}$$

$$= \frac{0.13\,(0.5\text{ in}) \wedge 2\,(60000\text{ psi})\,(1.0)}{(2.5\text{ in})\sqrt{1500\text{ psi}}}$$

= 20.14 in

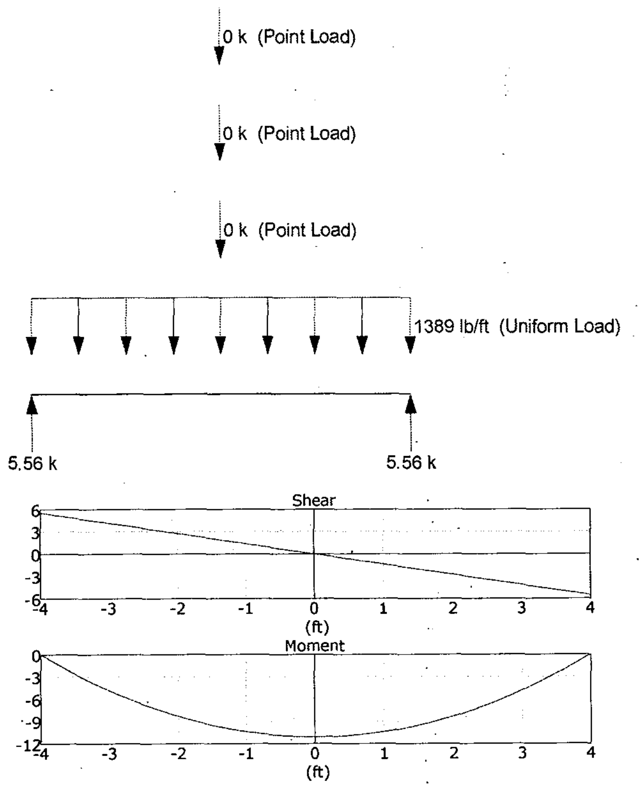
Eff_Embed = 0.5 l_d = 0.5 (20.14 in) = 10.0698 (when ended in a standard hook)

Eff_Embed = 0.33 l_d = 0.33 (20.14 in) = 6.6460 (when hooked 135 deg around a flexural bar, No. 5 & smaller only)

(000) 000-0000

Load Combination: 1.0D + 1.0L

Design Forces



Factored Loads

Uniform Load	1389 lb/ft
Self Weight	0 lb/ft
Wall Above Peak Weight	0 lb/ft

Checks

Flexure [MSJC-05 2.3.3.2.2, 2.3.2.1]

M_a = 32.34 ft·k

M = 11.11 ft·k $\leq$ M_a = 32.34 ft·k ✓

Shear [MSJC-05 2.3.5]

$f_v = \dfrac{V}{b\,d} = \dfrac{(5.56\text{ k})}{(7.63\text{ in})(29.19\text{ in})} = 24.96\text{ psi}$

$A_v = V\,s\,/\,F_s\,d = (5.56\text{ k})(16\text{ in})/(24000\text{ psi})(29.19\text{ in}) = 0.13\text{ in}^2$

$A_{v\_prov} = 0.4\text{ in}^2 \geq A_v = 0.13\text{ in}^2$

Provided shear reinf. is sufficient; masonry stresses must still satisfy 2.3.5.2.3

$F_v = 3\sqrt{f'_m} = 3\sqrt{(1500\text{ psi})} = 116.2\text{ psi}$ ($\leq$ 150 psi)

$f_v = 24.96\text{ psi} \leq F_v = 116.2\text{ psi}$ ✓

Deflection [MSJC-05 1.10.1]

$n = E_s / E_m = (29000000\text{ psi})/(1350000\text{ psi}) = 21.4815$

$I_{cr} = \dfrac{b\,[k\,d]^{\wedge}3}{3} + n\,A_s\,[d - k\,d]^{\wedge}2$

$= \dfrac{(7.63\text{ in})\,[(0.2644)(29.19\text{ in})]^{\wedge}3}{3} + (21.4815)(0.61\text{ in}^2)\,[(29.19\text{ in}) - (0.2644)(29.19\text{ in})]^{\wedge}2$

$= 7244\text{ in}^{\wedge}4$

$I_g = \dfrac{b\,d_T{}^{\wedge}3}{12} = \dfrac{(7.63\text{ in})(32\text{ in})^{\wedge}3}{12} = 20821\text{ in}^{\wedge}4$ (just for comparison)

$\delta = \dfrac{5\,M_{ser}\,L^{\wedge}2}{48\,E_m\,I_{cr}} = \dfrac{5\,(11.11\text{ ft·k})(8\text{ ft})^{\wedge}2}{48\,(1350000\text{ psi})(7244\text{ in}^{\wedge}4)} = 0.01\text{ in}$

$L / 600 = (8\text{ ft}) / 600 = 0.16\text{ in} \leq 0.3$

$\delta = 0.01\text{ in} \leq \delta_a = 0.16\text{ in}$ ✓

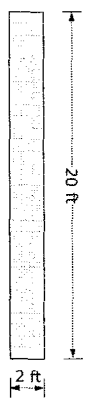
Art Cantrell
Holben, Martin _White
123 Easy Street
AnyTown, USA 00000
(000) 000-0000

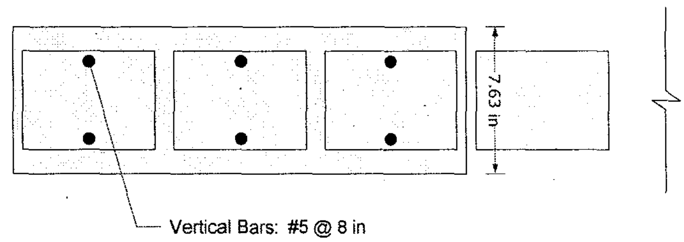
Jamb 3
CFSD HS Kitchen Expansion

Job # 10048

78

Design Detail

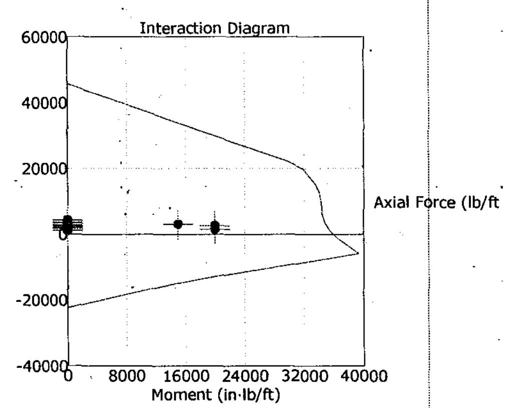




Check Summary

Ratio	Check	Provided	Required	Combination
----- Strength Checks -----				
✓ 0.315	Axial Compression	1.18 k	0.37 k	1.0D + 1.0L
✓ 0.000	Axial Tension	24 k	0 k	1.0D + 1.0L
✓ 0.123	Shear	38.73 psi	4.75 psi	1.0D + 1.0W
✓ 0.570	Axial+Flexure	35026 in·lb/ft	19975 in·lb/ft	1.0D + 1.0W

Interaction Diagram



Criteria

Building Code	MSCJ-05 (ASD)
Load Combination	ASCE 7-05 (ASD)
Seismic R Value	3.50
Amplify Axial Stress For Slenderness	Yes
f'm	1500 psi
fy	60000 psi
Specify Wall Weight Manually	No
Block Weight	Normal weight
Design As Clay Masonry	No
Include Wall Self-Weight	Yes
Neglect Lateral Load on Parapet	No
Include Wall Wt In Virtual Eccentricity	No
Always use I-cracked	No

Load Combinations

ASCE 7-05 (ASD)
1.0D + 1.0L
1.0D + 1.0W
1.0D
1.0D + 0.75L + 0.75W
1.0D + 0.75L
0.6D + 1.0W
0.6D

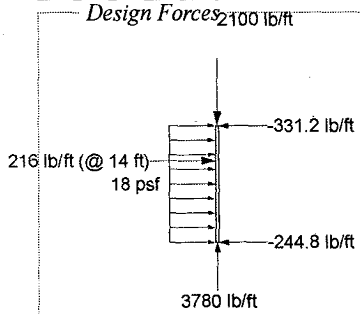
Loads Summary

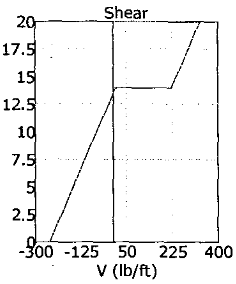
Load Set	Source	Axial Unifo...	Axial Pt Lo...	Pt Ld Eff W...	Eccentricity	Lateral Pre...	Top Lateral...	Parapet Pr...	Lateral Uni...	Lat Unif Ld...	Moment
Roof	Dead	0 lb/ft	1.5 k	2 ft	0 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft
Roof	Live	0 lb/ft	1.36 k	2 ft	0 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft
Wind	Wind	0 lb/ft	0 k	1 ft	0 in	18 psf	18 psf	18 psf	216 lb/ft	14 ft	0 in·lb/ft
Wall	Dead	0 lb/ft	2.7 k	2 ft	0 in	0 psf	0 psf	0 psf	0 lb/ft	1 ft	0 in·lb/ft

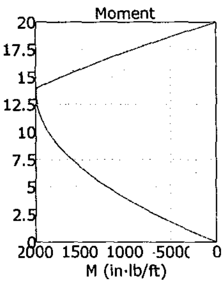
(000) 000-0000

Load Combination: 1.0D + 1.0W

Design Forces



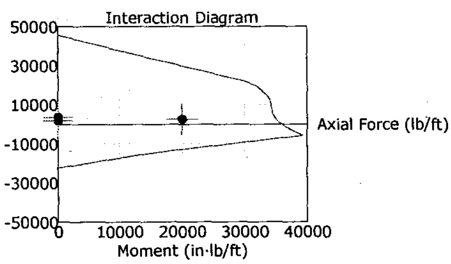




Factored Loads

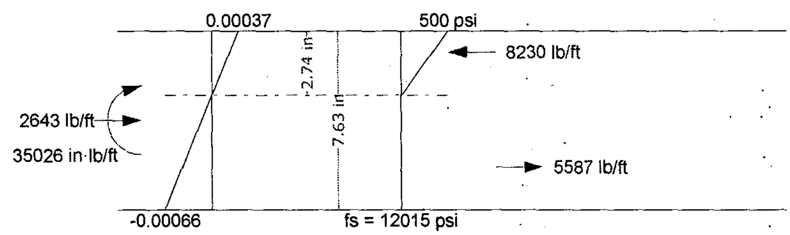
Wall Weight	1680 lb/ft
Effective Eccentricity	0 in
Applied Eccentricity	-0 in

Axial/Flexure Checks



Internal State at Max Moment Capacity for P = 2643 lb/ft

COMPRESSION controlled (fm = Fm = 500 psi) k = 0.360



Combined Axial Flexure (@ base) [MSJC-05 2.3.3.2.2, 2.3.

$P = 3780\ lb/ft$

$M_a = 34781\ in{\cdot}lb/ft$ (from interaction diagram given P)

$M = 0\ in{\cdot}lb/ft \leq M_a = 34781\ in{\cdot}lb/ft$ ✓

Combined Axial Flexure (@ max M) [MSJC-05 2.3.3.2.2, 2.

$P = 2643\ lb/ft$

$M_a = 35026\ in{\cdot}lb/ft$ (from interaction diagram given P)

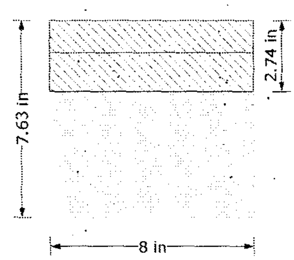
$M = 19975\ in{\cdot}lb/ft \leq M_a = 35026\ in{\cdot}lb/ft$ ✓

Combined Axial Flexure (@ top) [MSJC-05 2.3.3.2.2, 2.3.2.

$P = 2100\ lb/ft$

$M_a = 35181\ in{\cdot}lb/ft$ (from interaction diagram given P)

$M = 0\ in{\cdot}lb/ft \leq M_a = 35181\ in{\cdot}lb/ft$ ✓



Other Checks

Shear [MSJC-05 2.3.5]

$f_v = \frac{V}{d} = \frac{(331.2\ lb/ft)}{(5.81\ in)} = 4.75\ psi$

$F_v = \sqrt{f'_m} = \sqrt{(1500\ psi)} = 38.73\ psi \quad (\leq 50\ psi)$

$f_v = 4.75\ psi \leq F_v = 38.73\ psi$ ✓

Axial Compression (@ base) [MSJC-05 2.3.3.2.1]

$F_s = 24000\ psi$ (Grade 60 reinf)

$A_{st} = 0\ in^2/in$ (bars are not tied)

$h/r = (20\ ft)/(2.2\ in) = 109.0340 > 99$

$P_a = [0.25\, f'_m A_n + 0.65\, A_{st} F_s]\left[\frac{70\, r}{h}\right]\hat{}\,2$

$= [0.25\,(1500\ psi)\,(0.64\ ft^2/ft) + 0.65\,(0\ in^2/in)\,(24000\ psi)]\left[\frac{70\,(2.2\ in)}{(20\ ft)}\right]\hat{}\,2$

$= 14142\ lb/ft$

$P = 3780\ lb/ft \leq P_a = 14142\ lb/ft$ ✓