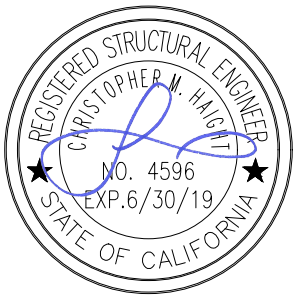
STRUCTURAL CALCULATIONS

Children's Ministry Lancaster Baptist Church
4020 Lancaster Blvd.
Lancaster, CA



Prepared for:
Bickel Group Architecture
3600 SW Birch St #120
Newport Beach, CA 92660

Prepared by:
COFFMAN ENGINEERS, INC.
1401 Dove St, Suite 250
Newport Beach, CA 92660

May 7, 2019
CEI Project #181257

Coffman Engineers

STRUCTURAL CALCULATION INDEX

Project: LBC Children's Ministry TI
CEI #181257

VOLUME 1

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LOAD TAKE-OFF

DEAD →

14" TJI	3.5 psf
GLULAMS	2.5
STEEL BEAMS	2.0
PLYWD 1/2"	1.5
CEILING	2.0
INSULATION	2.0
MEP	2.0
MISC.	+ 1.5
	17 psf

LIVE →

CLASSROOMS	40 psf
HALLWAYS	+ 15
	55 psf

ROOF	**20 psf**

PARTITIONS	5 psf	(ROOF)
	10 psf	(FLOOR)

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project	by	sheet no.
location	date	
client	checked	job no.
	date	

4020 E Lancaster Blvd, Lancaster, CA 93535, USA

Latitude, Longitude: 34.69579969999999, -118.0585858

Date	4/22/2019, 10:59:34 AM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.6	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	0.9	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	0.6	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.5	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	1.559	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.574	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.669	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.691	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.991	Mapped value of the risk coefficient at short periods

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Seismic Design Base Shear: Equivalent Lateral Force Procedure

Determine value of the Seismic Design Base Shear, V. Based on ASCE 7-10.
Requirements for Seismic Design Category A are not considered.

Input Data

Variables determined by building configuration

		II	Risk Category per 2015 IBC Table 1604.5
I_e	=	1.00	Seismic Importance factor per Table 1.5-2
C_t	=	0.020	Building Period parameter per Table 12.8-2
x	=	0.75	Building Period parameter per Table 12.8-2
R	=	6.50	Response Modif. Coeff. per Table 12.2-1
h_n	=	32.00	Height above base to the uppermost level in the main portion of the building, (ft)
		Yes	Diaphragms per Section 11.6: Flexible w/ $L_f \leq 40$ ft -or- Non-Flexible

Variables determined by site and location

		D	Site Class per Table 20.3-1
T_L	=	8	Long-period Transition per Figure 22-12
S_s*	=	1.500	Spectral Accel. for short periods per Figure 22-1
S_1*	=	0.600	Spectral Accel. for 1-sec. period per Figure 22-2
F_a*	=	1.000	Site Coefficient per Table 11.4-1
F_v*	=	1.500	Site Coefficient per Table 11.4-2

*Use USGS website to get precise values:
http://earthquake.usgs.gov/designmaps/us/application.php

Output Data

Approximate fundamental period, T_a (sec.) per Section 12.8.2.1

$T_a = C_t\ (h_n)^x = 0.269$ (Equation 12.8-7)

Design spectral response acceleration coefficients per Sections 11.4.3 and 11.4.4

$S_{MS} = F_a S_s = 1.500$ $S_{DS} = (2/3)S_{MS} = 1.000$ (Equation 11.4-1 and Equation 11.4-3)

$S_{M1} = F_v S_1 = 0.900$ $S_{D1} = (2/3)S_{M1} = 0.600$ (Equation 11.4-2 and Equation 11.4-4)

Response Spectrum Transition Periods (sec.) per Section 11.4.5

$T_0 = 0.2\ (S_{D1}/S_{DS}) = 0.12$ (Section 11.4.5)

$T_S = S_{D1}/S_{DS} = 0.6$ $0.8\ T_s = 0.48$ (Section 11.4.5)

Variables determined by Sections 11.6 and 12.8.2

		D	Seismic Design Category per Section 11.6
$C_{u\,max}$	=	1.40	Period upper limit coeff. per Table 12.8-1
C_u	=	1.00	Period coeff. used

Upper limit fundamental period for base shear determination, T (sec.) per Section 12.8.2

$T = T_a \times C_u = 0.269$ (Section 12.8.2)

Seismic Base Shear, V per ASCE Section 12.8.1

$C_s = \dfrac{S_{DS}}{(R/I_e)} = 0.154$ $V = C_s W = 0.154\ W$ (Equation 12.8-2 and Equation 12.8-1)

$C_{s\,min} = 0.044\ S_{DS}\ I_e = 0.044$ $V = C_s W = 0.044\ W$ (Equation 12.8-5 and Equation 12.8-1)

$C_{s\,min} = \dfrac{0.5S_1}{(R/I_e)} = 0.046$ $V = C_s W = 0.046\ W$ (Equation 12.8-6 and Equation 12.8-1)

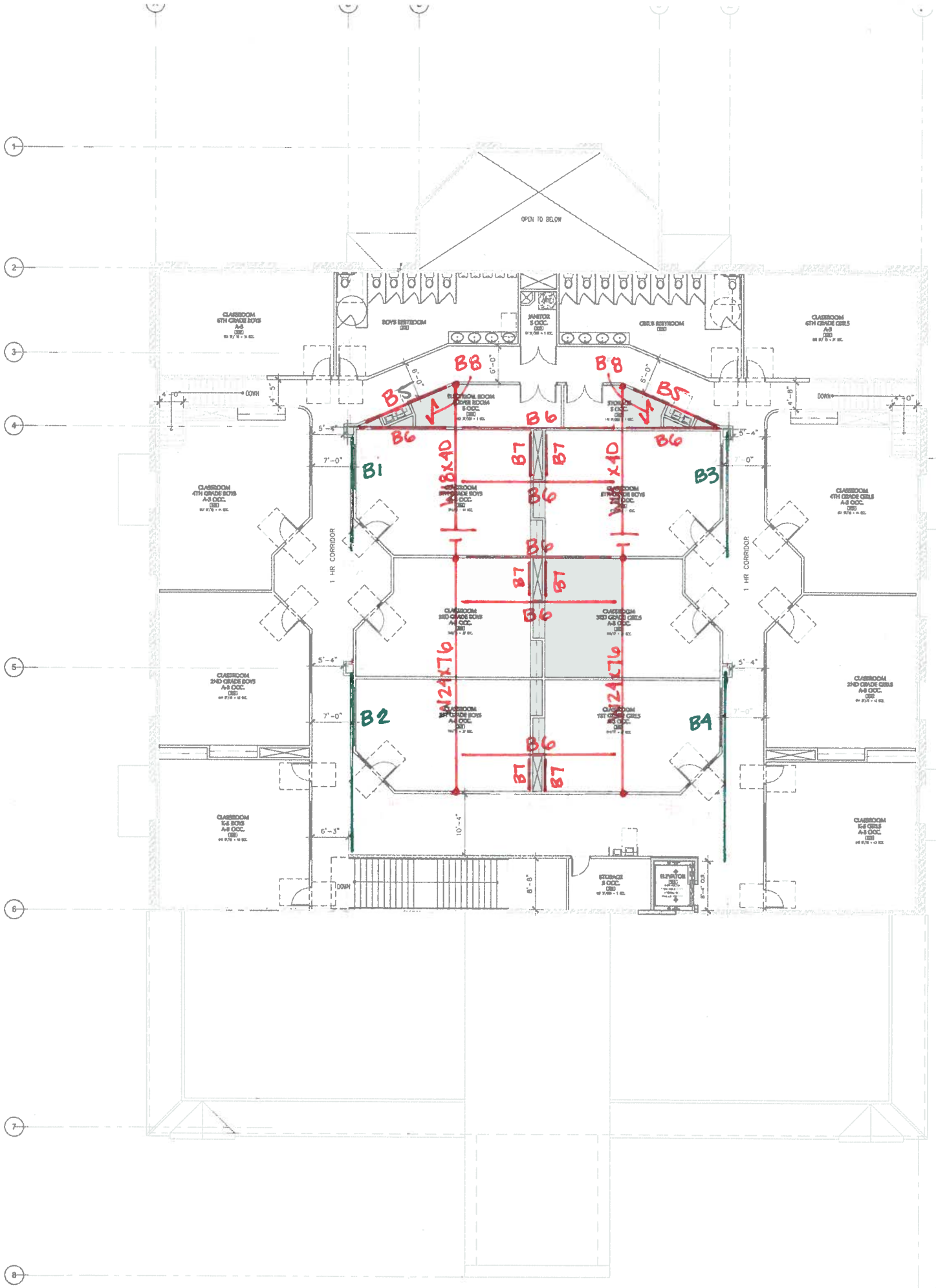
For $T \leq T_L$:

$C_{s\,max} = \dfrac{S_{D1}}{T(R/I_e)} = 0.343$ $V = C_s W = 0.343\ W$ (Equation 12.8-3 and Equation 12.8-1)

For $T > T_L$:

$C_{s\,max} = \dfrac{S_{D1}T_L}{T^2(R/I_e)} = n/a$ $V = C_s W = n/a\ W$ (Equation 12.8-4 and Equation 12.8-1)

$$\boxed{V = C_s W = 0.154\ W}$$


OPEN TO BELOW
CLASSROOM 6TH GRADE BOYS A-3
BOYS RESTROOM
JANITOR 3 OCC.
GIRLS RESTROOM
CLASSROOM 6TH GRADE GIRLS A-3
ELECTRICAL ROOM COVER ROOM 5 OCC.
STORAGE 5 OCC.
CLASSROOM 4TH GRADE BOYS A-3 OCC.
CLASSROOM 4TH GRADE GIRLS A-3 OCC.
CLASSROOM 3RD GRADE BOYS A-4 OCC.
CLASSROOM 3RD GRADE GIRLS A-3 OCC.
CLASSROOM 2ND GRADE BOYS A-3 OCC.
CLASSROOM 2ND GRADE GIRLS A-3 OCC.
CLASSROOM 1ST GRADE BOYS A-4 OCC.
CLASSROOM 1ST GRADE GIRLS A-3 OCC.
CLASSROOM K-5 BOYS A-3 OCC.
CLASSROOM K-5 GIRLS A-3 OCC.
1 HR CORRIDOR
1 HR CORRIDOR
STORAGE 3 OCC.
ELEVATOR
DOWN
DOWN
DOWN
B1
B2
B3
B4
B5
B6
B7
B8
18X40
24X76
6'-0"
4'-5"
4'-0"
5'-4"
7'-0"
6'-3"
10'-4"
8'-8"
8'-1" CLR.

HOPE-AMUNDSON
Structural Engineers

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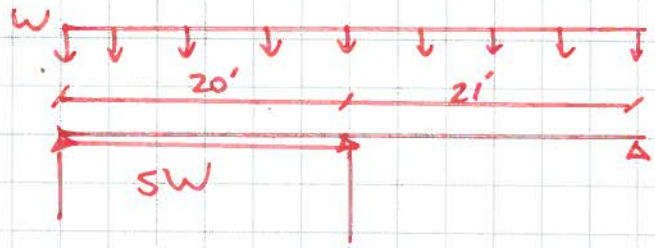
Project ____
Project No. ____
Engineer ____ Date ____

SLB Loads:

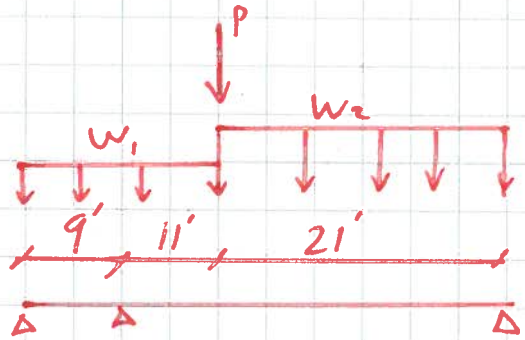
B1/B3 → LINE B/E



$w_D = 17psf(21'/2 + 32'/2) = 459plf$

$w_L = 55psf(21'/2 + 32'/2) = 1485plf$

B2/B4 → LINE B/E



$w_{1D} = 17psf(32'/2) = 272plf$

$w_{1L} = 55psf(32'/2) = 880plf$

$w_{2D} = w_{1D} + 17psf(21'/2) = 459plf$

$w_{2L} = w_{1L} + 55psf(21'/2) = 1485plf$

$P_D = 17psf(22'/2)(11'/2) = 1{,}029\#$

$P_L = 55psf(22'/2)(11'/2) = 3{,}328\#$

See following pages for beam checks

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Sawn Lumber/Glulam Beam Analysis

B1: 8.75 x 34.5

Member Size: 8 3/4x34 1/2 **GLB**
Combination = 24F-V4
Fv = 265 psi
Fb = 2400 psi
E = 1,800,000 psi

Point Load:
Location a (ft) = 0.0
Point Load:
Location a (ft) = 0.0
Partial Uniform Load:
Location a (ft) = 0.0
Location b (ft) = 0.0

SPAN (ft) = 21.00
Uniform Load:
wDL (#/ft) = 459
wTL (#/ft) = 1485
PDL (#) = 0
PTL (#) = 0
PDL (#) = 0
PTL (#) = 0
wDL (#/ft) = 0
wTL (#/ft) = 0

R1 = 15,593 # A = 301.9 in²
R2 = 15,593 # Sx = 1,735.8 in³
V@ d = 11,323 # Ix = 29,942 in^4
Mmax = 81,861 ft-#
Cd = 1.25 Cv = 0.85291
fv = 56 psi < F'v = Fv Cd = 331 psi OK
fb = 566 psi < F'b = Fb Cd Cv = 2,559 psi OK
Deflection Criteria: Total Load L/ 240
Live Load L/ 360
Δ DL = 0.0372 in = L/ 6773
Δ LL = 0.0832 in = L/ 3030
Δ TL = 0.1204 in = L/ 2093

Use 8 3/4x34 1/2 GLB C = 0 in.

B2: 8.75 x 34.5

Member Size: 8 3/4x34 1/2 **GLB**
Combination = 24F-V4
Fv = 265 psi
Fb = 2400 psi
E = 1,800,000 psi

Point Load:
Location a (ft) = 11.0
Point Load:
Location a (ft) = 0.0
Partial Uniform Load:
Location a (ft) = 11.0
Location b (ft) = 32.0

SPAN (ft) = 32
Uniform Load:
wDL (#/ft) = 272
wTL (#/ft) = 880
PDL (#) = 1029
PTL (#) = 3328
PDL (#) = 0
PTL (#) = 0
wDL (#/ft) = 187
wTL (#/ft) = 605

R1 = 20,433 # A = 301.9 in²
R2 = 23,760 # Sx = 1,735.8 in³
V@ d = 19,491 # Ix = 29,942 in^4
Mmax = 190,080 ft-#
Cd = 1.25 Cv = 0.81773
fv = 97 psi < F'v = Fv Cd = 331 psi OK
fb = 1,314 psi < F'b = Fb Cd Cv = 2,453 psi OK
Deflection Criteria: Total Load L/ 240
Live Load L/ 360
Δ DL = 0.1988 in = L/ 1931
Δ LL = 0.4445 in = L/ 864
Δ TL = 0.6433 in = L/ 597

Use 8 3/4x34 1/2 GLB C = 1/4 in.

B3: 8.75 x 34.5

Member Size: 8 3/4x34 1/2 **GLB**
Combination = 24F-V4
Fv = 265 psi
Fb = 2400 psi
E = 1,800,000 psi

Point Load:
Location a (ft) = 0.0
Point Load:
Location a (ft) = 0.0
Partial Uniform Load:
Location a (ft) = 0.0
Location b (ft) = 0.0

SPAN (ft) = 21
Uniform Load:
wDL (#/ft) = 459
wTL (#/ft) = 1485
PDL (#) = 0
PTL (#) = 0
PDL (#) = 0
PTL (#) = 0
wDL (#/ft) = 0
wTL (#/ft) = 0

R1 = 15,593 # A = 301.9 in²
R2 = 15,593 # Sx = 1,735.8 in³
V@ d = 11,323 # Ix = 29,942 in^4
Mmax = 81,861 ft-#
Cd = 1.25 Cv = 0.85291
fv = 56 psi < F'v = Fv Cd = 331 psi OK
fb = 566 psi < F'b = Fb Cd Cv = 2,559 psi OK
Deflection Criteria: Total Load L/ 240
Live Load L/ 360
Δ DL = 0.0372 in = L/ 6773
Δ LL = 0.0832 in = L/ 3030
Δ TL = 0.1204 in = L/ 2093

Use 8 3/4x34 1/2 GLB C = 0 in.

B4: 8.75 x34.5

Member Size: 8 3/4x34 1/2 **GLB**
Combination = 24F-V4
Fv = 265 psi
Fb = 2400 psi
E = 1,800,000 psi

Point Load:
Location a (ft) = 11.0
Point Load:
Location a (ft) = 0.0
Partial Uniform Load:
Location a (ft) = 11.0
Location b (ft) = 32.0

SPAN (ft) = 32
Uniform Load:
wDL (#/ft) = 131
wTL (#/ft) = 277
PDL (#) = 1029
PTL (#) = 3328
PDL (#) = 0
PTL (#) = 0
wDL (#/ft) = 187
wTL (#/ft) = 605

R1 = 10,785 # A = 301.9 in²
R2 = 14,112 # Sx = 1,735.8 in³
V@ d = 11,576 # Ix = 29,942 in^4
Mmax = 101,871 ft-#
Cd = 1.25 Cv = 0.81773
fv = 58 psi < F'v = Fv Cd = 331 psi OK
fb = 704 psi < F'b = Fb Cd Cv = 2,453 psi OK
Deflection Criteria: Total Load L/ 240
Live Load L/ 360
Δ DL = 0.0887 in = L/ 4331
Δ LL = 0.134 in = L/ 2866
Δ TL = 0.2227 in = L/ 1725

Use 8 3/4x34 1/2 GLB C = 1/4 in.

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Sawn Lumber/Glulam Beam Analysis

B5: 6.75 x 18

Member Size: 6 3/4x18 **GLB**
Combination = 24F-V4
Fv = 265 psi
Fb = 2400 psi
E = 1,800,000 psi

Point Load:
Location a (ft) = 0.0
Point Load:
Location a (ft) = 0.0
Partial Uniform Load:
Location a (ft) = 0.0
Location b (ft) = 0.0

SPAN (ft) = 20.00

Uniform Load:
wDL (#/ft) = 306
wTL (#/ft) = 1296

PDL (#) = 0
PTL (#) = 0
PDL (#) = 0
PTL (#) = 0
wDL (#/ft) = 0
wTL (#/ft) = 0

R1 = 12,960 # A = 121.5 in²
R2 = 12,960 # Sx = 364.5 in³
V@ d = 11,016 # Ix = 3,281 in^4
Mmax = 64,800 ft-#
Cd = 1.25 Cv = 0.93875
fv = 136 psi < F'v = Fv Cd = 331 psi OK
fb = 2,133 psi < F'b = Fb Cd Cv = 2,816 psi OK

Deflection Criteria: Total Load L/ 240
Live Load L/ 360
Δ DL = 0.1863 in = L/ 1288
Δ LL = 0.6026 in = L/ 398
Δ TL = 0.7889 in = L/ 304

Use 6 3/4x18 GLB C = 1/4 in.

B6: 3.125 x 13.5

Member Size: 3 1/8x13 1/2 **GLB**
Combination = 24F-V4
Fv = 265 psi
Fb = 2400 psi
E = 1,800,000 psi

Point Load:
Location a (ft) = 8.75
Point Load:
Location a (ft) = 11.25
Partial Uniform Load:
Location a (ft) = 0.0
Location b (ft) = 0.0

SPAN (ft) = 20.00

Uniform Load:
wDL (#/ft) = 23
wTL (#/ft) = 96

PDL (#) = 85
PTL (#) = 360
PDL (#) = 85
PTL (#) = 360
wDL (#/ft) = 0
wTL (#/ft) = 0

R1 = 1,320 # A = 42.2 in²
R2 = 1,320 # Sx = 94.9 in³
V@ d = 1,212 # Ix = 641 in^4
Mmax = 7,950 ft-#
Cd = 1.25 Cv = 1
fv = 43 psi < F'v = Fv Cd = 331 psi OK
fb = 1,005 psi < F'b = Fb Cd Cv = 3,000 psi OK

Deflection Criteria: Total Load L/ 240
Live Load L/ 360
Δ DL = 0.112 in = L/ 2143
Δ LL = 0.3623 in = L/ 662
Δ TL = 0.4743 in = L/ 506

Use 3 1/8x13 1/2 GLB C = 1/4 in.

B7: 4x8

Member Size: 4x8
Grade = No.2
Fv = 180 psi
Fb = 900 psi
E = 1,600,000 psi

Point Load:
Location a (ft) = 0.0
Point Load:
Location a (ft) = 0.0
Partial Uniform Load:
Location a (ft) = 0.0
Location b (ft) = 0.0

SPAN (ft) = 8.00

Uniform Load:
wDL (#/ft) = 85
wTL (#/ft) = 360

PDL (#) = 0
PTL (#) = 0
PDL (#) = 0
PTL (#) = 0
wDL (#/ft) = 0
wTL (#/ft) = 0

R1 = 1,440 # A = 25.4 in²
R2 = 1,440 # Sx = 30.7 in³
V@ d = 1,223 # Ix = 111 in^4
Mmax = 2,880 ft-#
Cd = 1.25 Cf = 1.3
fv = 72 psi < F'v = Fv Cd = 225 psi OK
fb = 1,127 psi < F'b = Fb Cd Cf = 1,463 psi OK

Deflection Criteria: Total Load L/ 240
Live Load L/ 360
Δ DL = 0.044 in = L/ 2183
Δ LL = 0.1423 in = L/ 675
Δ TL = 0.1863 in = L/ 515

Use 4x8

B8: 2x10

Member Size: 2x10
Grade = No.2
Fv = 180 psi
Fb = 900 psi
E = 1,600,000 psi

Point Load:
Location a (ft) = 0.0
Point Load:
Location a (ft) = 0.0
Partial Uniform Load:
Location a (ft) = 0.0
Location b (ft) = 0.0

SPAN (ft) = 8

Uniform Load:
wDL (#/ft) = 23
wTL (#/ft) = 96

PDL (#) = 0
PTL (#) = 0
PDL (#) = 0
PTL (#) = 0
wDL (#/ft) = 0
wTL (#/ft) = 0

R1 = 384 # A = 13.9 in²
R2 = 384 # Sx = 21.4 in³
V@ d = 310 # Ix = 99 in^4
Mmax = 768 ft-#
Cd = 1.25 Cf = 1.1
fv = 34 psi < F'v = Fv Cd = 225 psi OK
fb = 431 psi < F'b = Fb Cd Cf = 1,238 psi OK

Deflection Criteria: Total Load L/ 240
Live Load L/ 360
Δ DL = 0.0132 in = L/ 7286
Δ LL = 0.0426 in = L/ 2252
Δ TL = 0.0558 in = L/ 1720

Use 2x10

RedSpec™ by RedBuilt™
v7.1.9

Project: Project
Location: Lancaster Baptist Church
Folder: Folder
Date: 5/7/19 4:47 PM
Designer: CG
Comment: Hangers Per Mfr.

Type: (N) Typ. 14" Joi

14" Red-I65™ @ 16" o.c. with Glued Sheathing

This product meets or exceeds the set design controls for the application and loads listed

DESIGN CONTROLS	%	Design	Allow.	DOL	Combination	Pattern	Pass/Fail
Shear (lb)	42%	1078	2540	Floor(100%)	1.0D+1.0L	All Spans	PASS
Positive Moment (ft-lb)	70%	5660	8030	Floor(100%)	1.0D+1.0L	All Spans	PASS

DEFLECTIONS (in)	%	Design	Allow.	Design	Allow.	Combination	Pattern	Pass/Fail
Span Live	70%	0.491	0.700	L / 514	L / 360	1.0D+1.0L	All Spans	PASS
Span Total	60%	0.630	1.050	L / 400	L / 240	1.0D+1.0L	All Spans	PASS

FloorChoice™ Rating: 4.4

Performance rating is based on: 32 oc (7/8", 1") sheathing, glued and nailed, simple span, rigid supports. RedSpec has not performed a structural analysis of the sheathing.

SUPPORTS

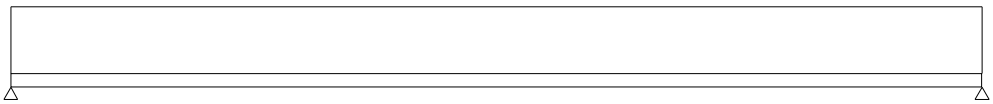
	Support 1	Support 2
Live Reaction, Critical (lb) (DOL%)	840 (100)	840 (100)
Dead Reaction (lb)	238	238
Total Reaction (lb) (DOL%)	1078 (100)	1078 (100)
Bearing	Flush	Flush
Support	Wall	Wall
Req'd Bearing, No Stiffeners (in)	1.75	1.75
Req'd Bearing, Stiffeners (in)	-	-

HANGERS

	Model	Top	Face	Member	Header	Size
Left	None Selected					
Right	None Selected					

SPANS AND LOADS

Dimensions represent horizontal design spans.



21'- 0.0"

APPLICATION LOADS

Type	Units	DOL	Live	Dead	Partition	Tributary	Member Type
Uniform	psf	Floor(100%)	55	17	5	16"	Glued Floor Joist

NOTES

- Building code and design methodology: 2015 IBC ASD (US).
- Deflection analysis is based on composite action with 32 oc (7/8", 1") sheathing, glued and nailed.
- Continuous lateral support required at top edge. Lateral support at bottom edge shall be per RedBuilt recommendations.

P:\SD\18Jobs\181257 LBC Children's Ministry TI\1 Central Folders\03 Calcs\RedSpec\LBC Ministry.red

The products noted are intended for interior, untreated, non-corrosive applications with normal temperatures and dry conditions of use, and must be installed in accordance with local building code requirements and RedBuilt™ recommendations. The loads, spans, and spacing have been provided by others and must be approved for the specific application by the design professional for the project. Unless otherwise noted, this output has not been reviewed by a RedBuilt™ associate. PRODUCT SUBSTITUTION VOIDS THIS ANALYSIS.

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Title Block Line 1
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Title Block Line 6

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 7 MAY 2019, 4:56PM

Steel Beam

File = P:\SD\18Jobs\1VQKSM~X\1VZOAA~E\0AMZQC~8\Enercalc\LBC Childrens Ministry.ec6 .
Software copyright ENERCALC, INC. 1983-2019, Build:10.18.12.31 .

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Description : W24x76

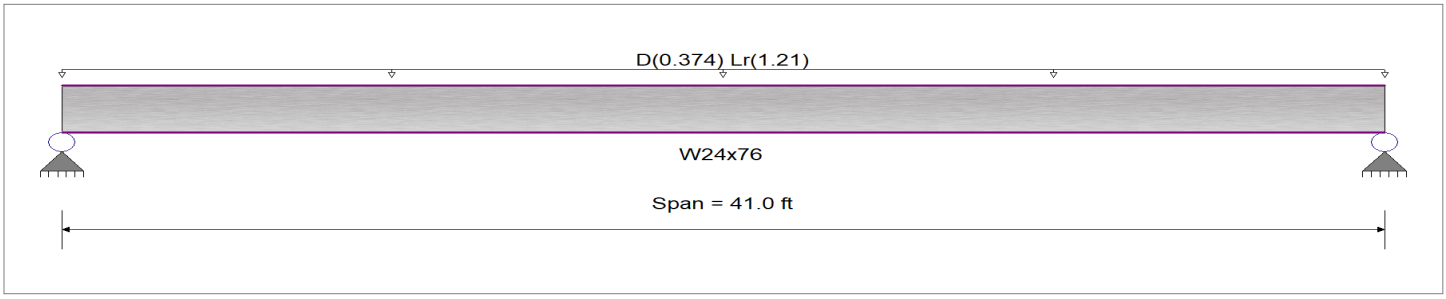
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Load Resistance Factor Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.0170, Lr = 0.0550 ksf, Tributary Width = 22.0 ft, (Floor)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.668 : 1	Maximum Shear Stress Ratio =	0.155 : 1
Section used for this span	W24x76	Section used for this span	W24x76
Mu : Applied	501.106 k-ft	Vu : Applied	48.888 k
Mn * Phi : Allowable	750.000 k-ft	Vn * Phi : Allowable	315.480 k
Load Combination	+1.20D+1.60Lr	Load Combination	+1.20D+1.60Lr
Location of maximum on span	20.500 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 Transient Deflection	1.268 in	Ratio =	387 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	1.661 in	Ratio =	296 >=240
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	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 41.00 ft	1	0.147	0.034	110.02		110.02	833.33	750.00	1.00	1.00	10.73	315.48	315.48
+1.20D+0.50Lr													
Dsgn. L = 41.00 ft	1	0.295	0.068	221.43		221.43	833.33	750.00	1.00	1.00	21.60	315.48	315.48
+1.20D													
Dsgn. L = 41.00 ft	1	0.126	0.029	94.30		94.30	833.33	750.00	1.00	1.00	9.20	315.48	315.48
+1.20D+1.60Lr													
Dsgn. L = 41.00 ft	1	0.668	0.155	501.11		501.11	833.33	750.00	1.00	1.00	48.89	315.48	315.48
+0.90D													
Dsgn. L = 41.00 ft	1	0.094	0.022	70.73		70.73	833.33	750.00	1.00	1.00	6.90	315.48	315.48

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	1.6613	20.617		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	32.472	32.472
Overall MINimum	4.600	4.600
D Only	7.667	7.667
+D+Lr	32.472	32.472
+D+0.750Lr	26.271	26.271
+0.60D	4.600	4.600

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

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 7 MAY 2019, 4:57PM

Steel Beam

File = P:\SD\18Jobs\1VQKSM~X\1VZOAA~E\0AMZQC~8\Enercalc\LBC Childrens Ministry.ec6 .
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Description : W18x40

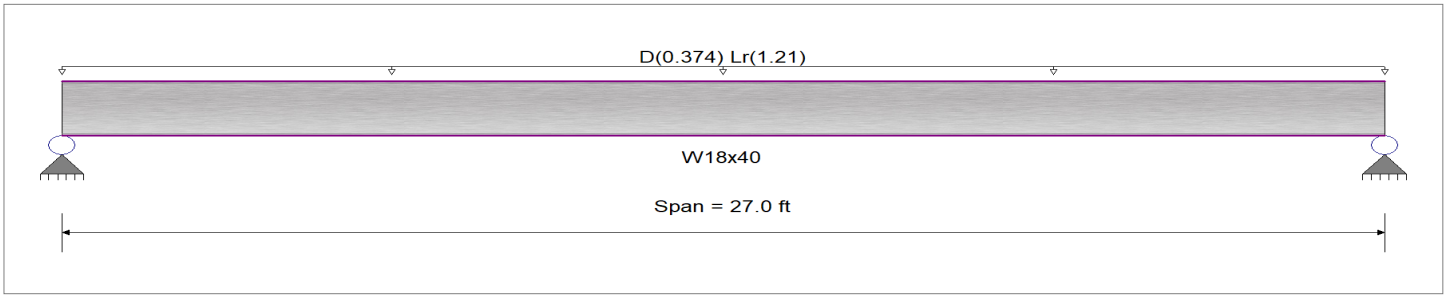
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Load Resistance Factor Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.0170, Lr = 0.0550 ksf, Tributary Width = 22.0 ft, (Floor)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.739 : 1	Maximum Shear Stress Ratio =	0.190 : 1
Section used for this span	W18x40	Section used for this span	W18x40
Mu : Applied	217.315 k-ft	Vu : Applied	32.195 k
Mn * Phi : Allowable	294.000 k-ft	Vn * Phi : Allowable	169.155 k
Load Combination	+1.20D+1.60Lr	Load Combination	+1.20D+1.60Lr
Location of maximum on span	13.500 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 Transient Deflection	0.818 in	Ratio =	395 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	1.072 in	Ratio =	302 >=240
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	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 27.00 ft	1	0.162	0.042	47.71		47.71	326.67	294.00	1.00	1.00	7.07	169.16	169.16
+1.20D+0.50Lr													
Dsgn. L = 27.00 ft	1	0.327	0.084	96.03		96.03	326.67	294.00	1.00	1.00	14.23	169.16	169.16
+1.20D													
Dsgn. L = 27.00 ft	1	0.139	0.036	40.90		40.90	326.67	294.00	1.00	1.00	6.06	169.16	169.16
+1.20D+1.60Lr													
Dsgn. L = 27.00 ft	1	0.739	0.190	217.31		217.31	326.67	294.00	1.00	1.00	32.19	169.16	169.16
+0.90D													
Dsgn. L = 27.00 ft	1	0.104	0.027	30.67		30.67	326.67	294.00	1.00	1.00	4.54	169.16	169.16

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	1.0721	13.577		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	21.384	21.384
Overall MINimum	3.029	3.029
D Only	5.049	5.049
+D+Lr	21.384	21.384
+D+0.750Lr	17.300	17.300
+0.60D	3.029	3.029

HOPE-AMUNDSON
Structural Engineers

Sheet ____ of ____
Project ____
Project No. ____
Engineer ____ Date ____

(M) Col Axial Loads:

DL = 17 psf
LL = 55 psf
} SEE LOAD TAKE-OFF

Grid 4.5 Posts = [1.2(17) + 1.6(55)][4½ + 23½](21')

= 78.6k HSS 5x5x¼ @ 15' ϕP_n = 99.2k OK

Grid 5.5 Posts = [1.2(17) + 1.6(55)](21')(23½)

= 31.9k HSS 4x4x¼ @ 15' ϕP_n = 54.3k OK

Title Block Line 1
You can change this area
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and then using the "Printing &
Title Block" selection.
Title Block Line 6

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 7 MAY 2019, 5:01PM

Steel Base Plate

File = P:\SD\18Jobs\1VQKSM~X\1VZOAA~E\0AMZQC~8\Enercalc\LBC Childrens Ministry.ec6 .
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Description : HSS 4X4X1/4

Code References

Calculations per AISC Design Guide # 1, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

General Information

Material Properties

AISC Design Method	Load Resistance Factor Design		Φ c : LRFD Resistance Factor	0.650
Steel Plate Fy	=	36.0 ksi		
Concrete Support f'c	=	3.0 ksi		
Assumed Bearing Area :	Full Bearing		Nominal Bearing Fp per J8	3.060 ksi

Column & Plate

Column Properties

Steel Section : HSS4x4x1/4

Depth	4 in		Area	3.37 in^2
Width	4 in		Ixx	in^4
Flange Thickness	0.233 in		Iyy	in^4
Web Thickness	in			

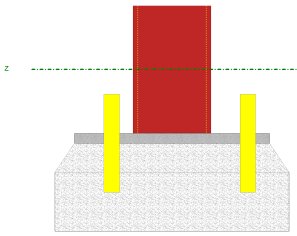
Plate Dimensions

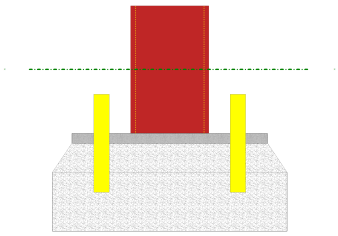
N : Length	10.0	in
B : Width	10.0	in
Thickness	0.50	in

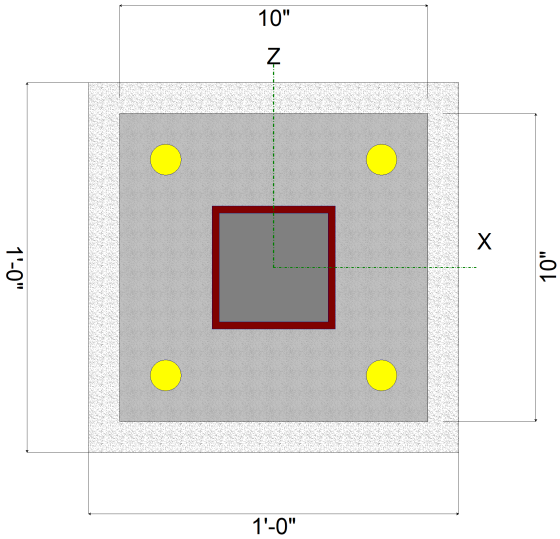
Column assumed welded to base plate.

Support Dimensions

Width along "X"	12.0	in
Length along "Z'	12.0	in





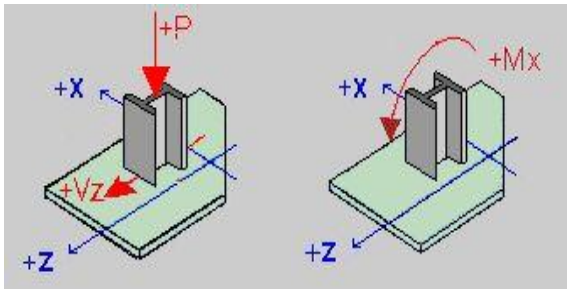


Applied Loads

	P-Y	V-Z	M-X
D : Dead Load	5.0 k	k	k-ft
L : Live	17.0 k	k	k-ft
Lr : Roof Live	k	k	k-ft
S : Snow	k	k	k-ft
W : Wind	k	k	k-ft
E : Earthquake	k	k	k-ft
H : Lateral Earth	k	k	k-ft

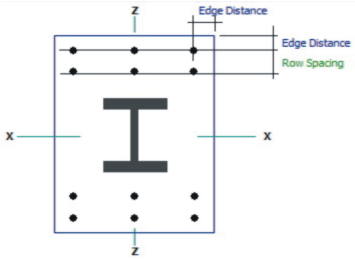
" P " = Gravity load, "+" sign is downward.

"+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.



Anchor Bolts

Anchor Bolt or Rod Description	5/8"	
Max of Tension or Pullout Capacity.....		k
Shear Capacity.....		k
Edge distance : bolt to plate.....	1.50	in
Number of Bolts in each Row.....	2.0	
Number of Bolt Rows.....	1.0	



Steel Base Plate

File = P:\SD\18Jobs\1VQKSM~X\1VZOAA~E\0AMZQC~8\Enercalc\LBC Childrens Ministry.ec6 .
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Description : HSS 4X4X1/4

GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	Load Resistance Factor Design
Governing Load Combination	+1.20D+0.50Lr+1.60L+1.60H
Governing Load Case Type	Axial Load Only
Governing STRESS RATIO	0.5252
Design Plate Size	10" x 10" x 0 -1/2"

Pu : Axial	0.000	k
Mu : Moment	0.000	k-ft

Mu : Max. Moment	1.595	k-in
fb : Max. Bending Stress	17.016	ksi
Fb : Allowable : Fy * Phi	32.400	ksi
Bending Stress Ratio	0.525	

Bending Stress OK

fu : Max. Plate Bearing Stress	0.332	ksi
Fp : Allowable :	1.989	ksi
Bearing Stress Ratio	0.167	

Bearing Stress OK

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

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 7 MAY 2019, 5:02PM

Steel Base Plate

File = P:\SD\18Jobs\1VQKSM~X\1VZOAA~E\0AMZQC~8\Enercalc\LBC Childrens Ministry.ec6 .
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Description : HSS 5x5x1/4

Code References

Calculations per AISC Design Guide # 1, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

General Information

Material Properties

AISC Design Method	Load Resistance Factor Design		Φ c : LRFD Resistance Factor	0.650
Steel Plate Fy	=	36.0 ksi		
Concrete Support f'c	=	3.0 ksi		
Assumed Bearing Area :	Full Bearing		Nominal Bearing Fp per J8	0.0 ksi

Column & Plate

Column Properties

Steel Section : HSS5x5x1/4

Depth	5 in	Area	4.3 in^2	
Width	5 in	Ixx	in^4	
Flange Thickness	0.233 in	Iyy	in^4	
Web Thickness	in			

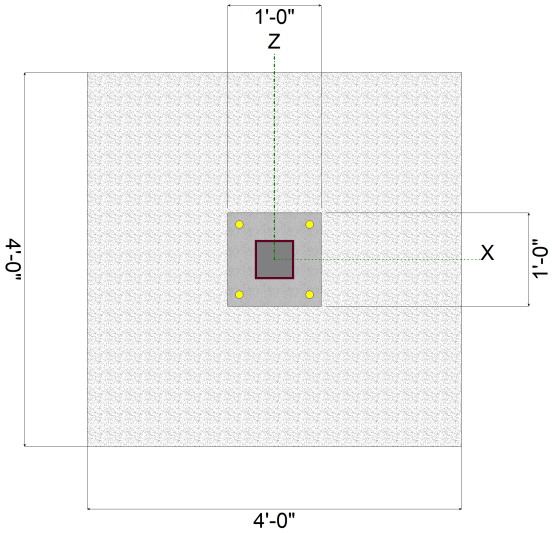
Plate Dimensions

N : Length	12.0	in
B : Width	12.0	in
Thickness	0.6250	in

Column assumed welded to base plate.

Support Dimensions

Width along "X"	48.0	in
Length along "Z'	48.0	in

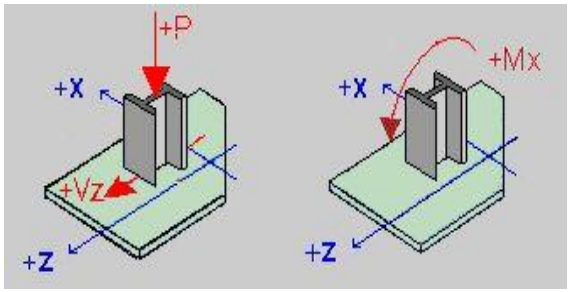


Applied Loads

	P-Y	V-Z	M-X
D : Dead Load	12.0 k	k	k-ft
L : Live	37.0 k	k	k-ft
Lr : Roof Live	k	k	k-ft
S : Snow	k	k	k-ft
W : Wind	k	k	k-ft
E : Earthquake	k	k	k-ft
H : Lateral Earth	k	k	k-ft

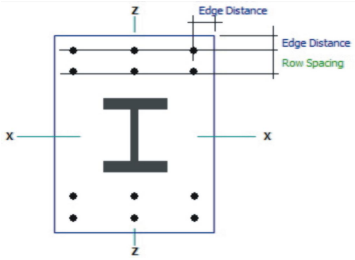
" P " = Gravity load, "+" sign is downward.

"+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.



Anchor Bolts

Anchor Bolt or Rod Description	5/8"	
Max of Tension or Pullout Capacity.....		k
Shear Capacity.....		k
Edge distance : bolt to plate.....	1.50	in
Number of Bolts in each Row.....	2.0	
Number of Bolt Rows.....	1.0	



Steel Base Plate

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Description : HSS 5x5x1/4

GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	Load Resistance Factor Design
Governing Load Combination	Overall Results
Governing Load Case Type	----------
Governing STRESS RATIO	0.0
Design Plate Size	1'-0" x 1'-0" x 0 -5/8

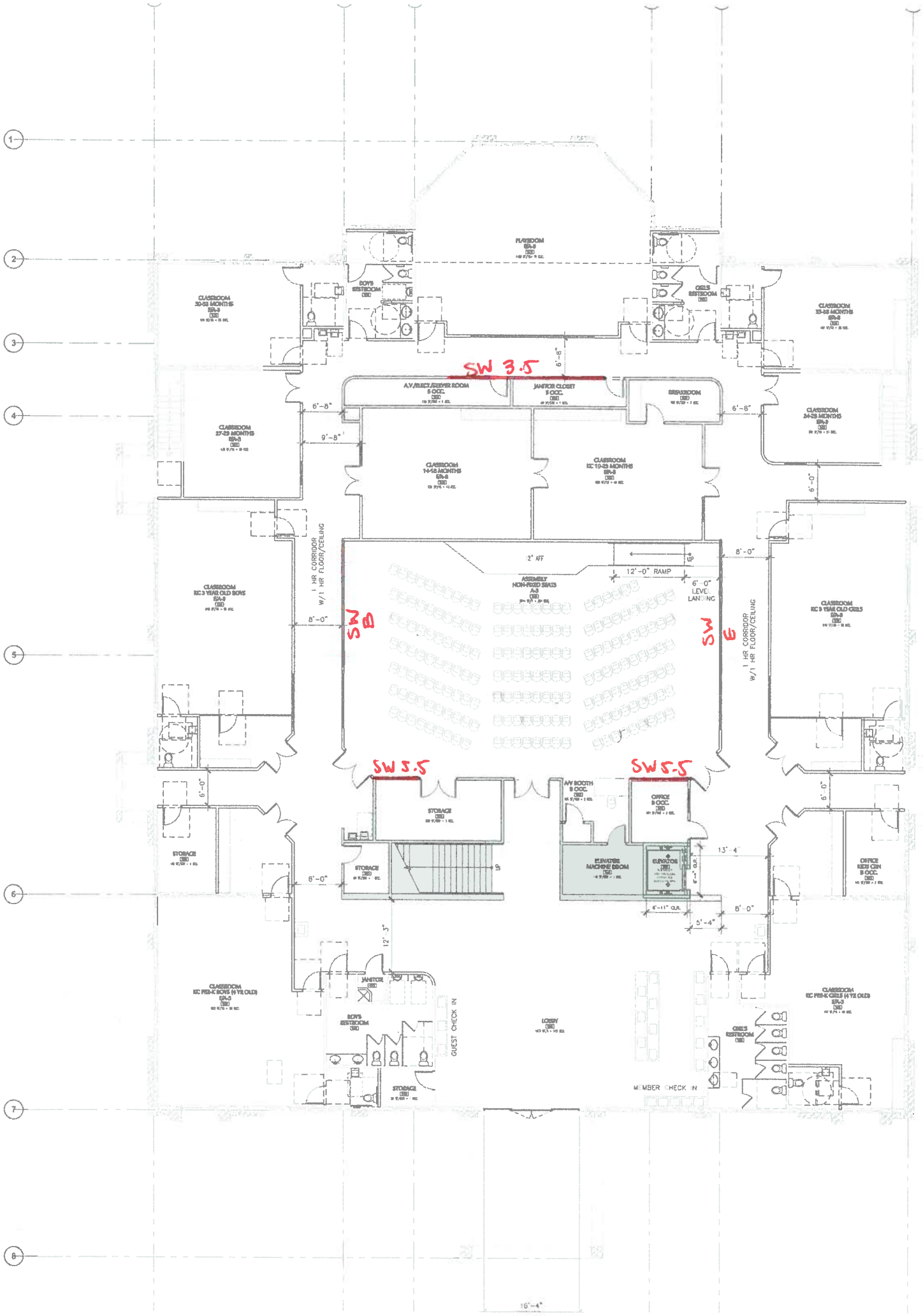
Pu : Axial	0.000	k
Mu : Moment	0.000	k-ft

Mu : Max. Moment	0.000	k-in
fb : Max. Bending Stress	0.000	ksi
Fb : Allowable : Fy * Phi	0.000	ksi
Bending Stress Ratio	0.000	

Bending Stress OK

fu : Max. Plate Bearing Stress	0.000	ksi
Fp : Allowable :	0.000	ksi
Bearing Stress Ratio	0.000	

Bearing Stress OK


SW 3.5
SW B
SW E
SW 5.5
SW 5.5
CLASSROOM 30-33 MONTHS
CLASSROOM 33-36 MONTHS
CLASSROOM 27-29 MONTHS
CLASSROOM 34-36 MONTHS
CLASSROOM 14-18 MONTHS
CLASSROOM EC 19-23 MONTHS
CLASSROOM EC 3 YEAR OLD BOYS
CLASSROOM EC 3 YEAR OLD GIRLS
CLASSROOM EC PRE-K BOYS (4 YR OLD)
CLASSROOM EC PRE-K GIRLS (4 YR OLD)
BOYS RESTROOM
GIRLS RESTROOM
A.V./ELECT./SERVER ROOM
JANITOR CLOSET
BREAKROOM
ASSEMBLY NON-FIXED SEATS
2" AFF
12'-0" RAMP
6'-0" LEVEL LANDING
UP
1 HR CORRIDOR W/1 HR FLOOR/CEILING
STORAGE
A/V BOOTH
OFFICE
ELEVATOR MACHINE ROOM
ELEVATOR
OFFICE KIDS CIN
JANITOR
GUEST CHECK IN
LOBBY
MEMBER CHECK IN
6'-8"
9'-8"
8'-0"
6'-0"
13'-4"
5'-11" CLR.
5'-4"
12'-3"
15'-4"

DIAPHRAGM ANALYSIS

- TRIB MEZZ AREA = 65' x 110' = 7150 SF
- MEZZ MASS = 7150 SF x 17 psf + 10 psf partitions
  = 193 k
- C_s FROM SEISMIC LOADS = 193 k x 0.154 = 30 k (ULT)
  = 21 k (ASD)

SW FORCES

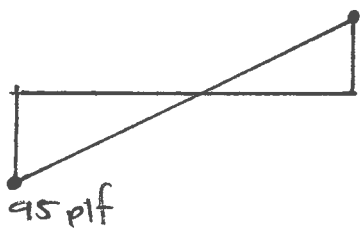
N/S

W_V = 110' (27 psf) (0.154) = 457 plf x 0.7 = 320 plf

Wall force = 320 plf x (65'/2) = 10.4 (ASD)

DIA. SHEAR

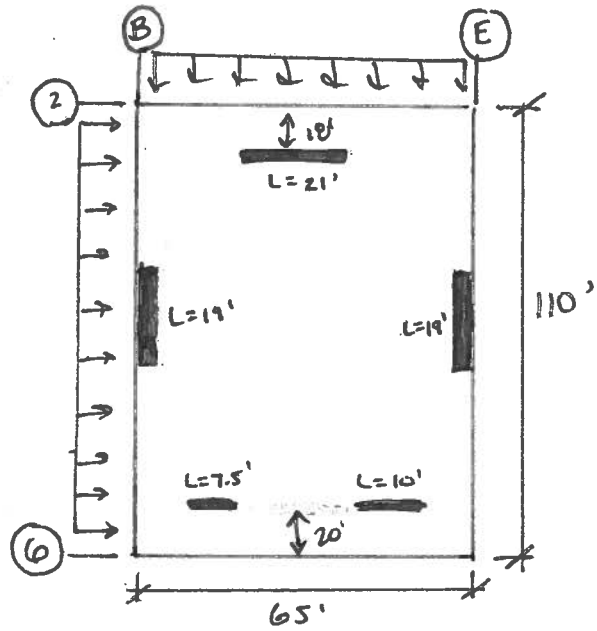
10.4 k / 110' = 95 plf



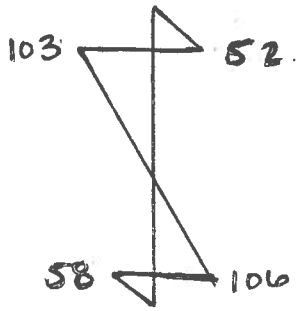
E/W

W_H = 65' x 27 psf x 0.154 x 0.7 = 189 plf

PLAN



DIA. SHEAR



SW FORCES — LINE 3.5 = (52 + 103) plf x 65'
= 10.1 k

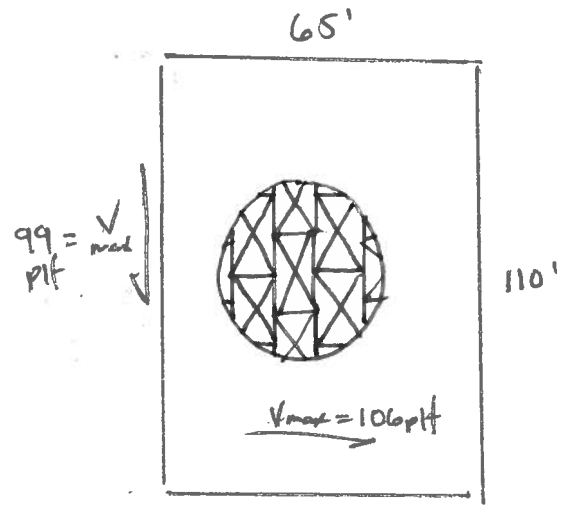
LINE 5.5 = (58 + 106) plf x 65'
= 10.7 k

COFFMAN ENGINEERS	project	by	sheet no.
	location	date	
	client	checked	job no.
		date	

DIAPHRAGM DESIGN

- DIA SHEAR MAX = 106 plf

- USE STRUCT I
  10d
  3/4" THK MATCH (E)
  BLOCKED
  6" EN
  4" BN
  10" FIELD
  = 850 plf / 2 = 425 plf



- ASPECT RATIO = 110 : 65
  1.7 : 1 < 4 : 1

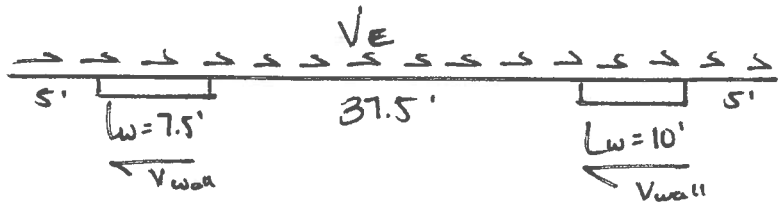
- USE 42/29 APA RATING.
  MATCH EXISTING

COFFMAN ENGINEERS	project	by	sheet no.
	location	date	
	client	checked	job no.
		date	

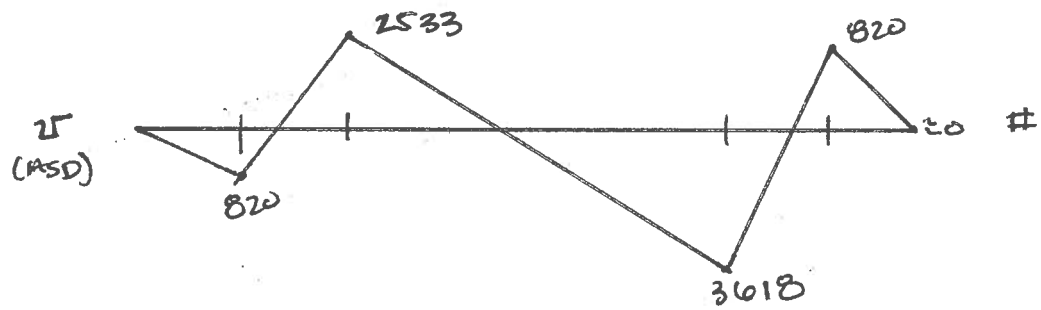
DRAGS & ANCHORAGE

$$V_E = 10.7 \times 1000 / 65' = 164 \text{ plf}$$

☐ LINE 5.5 →



$$V_{wall} = 10.7^k \times 1000 / 17.5' = 611 \text{ plf}$$



$$3618^{\#} / (F'_v = 180 \text{ psi} \times C_d = 1.6) = \underline{\underline{12.6 \text{ in.}^2}} \text{ REQ'D}$$

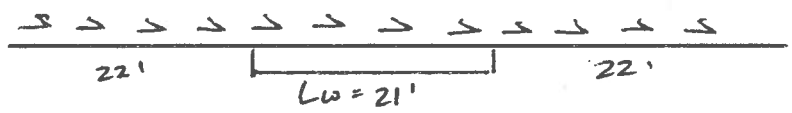
$= 288$ psi

∴ 2X10 MIN. USE 3X6 FOR SIZE.

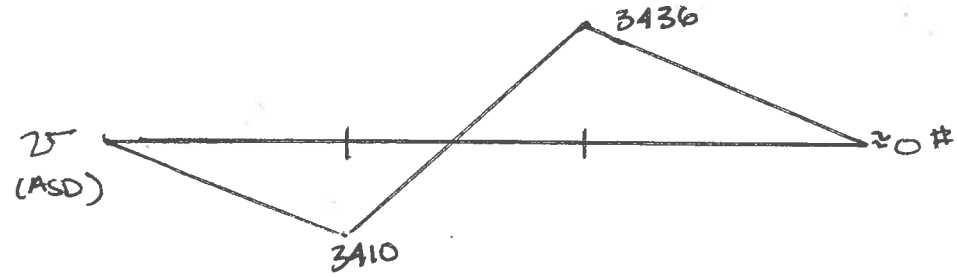
W/ LTP5 @ 24" OC MAX @ SW + MSTI 36 (FALL = 3000#)

$$V_E = 10.1^k \times 1000 / 65' = 155 \text{ plf}$$

☐ LINE 3.5 →



$$V_{wall} = 12100^{\#} / 21' = 481 \text{ plf}$$



COFFMAN ENGINEERS	project	by	sheet no.
	location	date	
	client	checked	job no.
		date	

DRAGS & ANCHORAGE CONT.

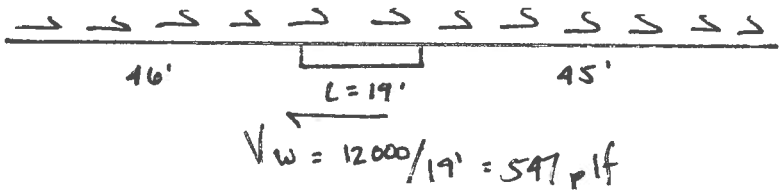
3436# / (F'V = 265 psi × C_d = 1.6 = 424 psi) = 8.1 in.² REQ'D

∴ 6 3/4 x 18 ok

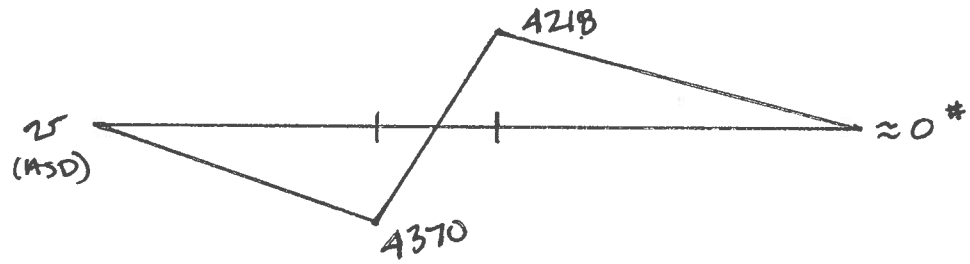
w/ MST136 (T_{all} = 3800 #)

☐ LINE B/E →

V_E = 10.4^k x 1000 / 110' = 95. plf



V_w = 12000/19' = 547 plf



4370 # / F'V = 424 psi = 10.3 in.² REQ'D

∴ 8 3/4 x 34 1/2 ok

w/ LTP5 @ 32" OC MAX WHERE WALL OCCURS.

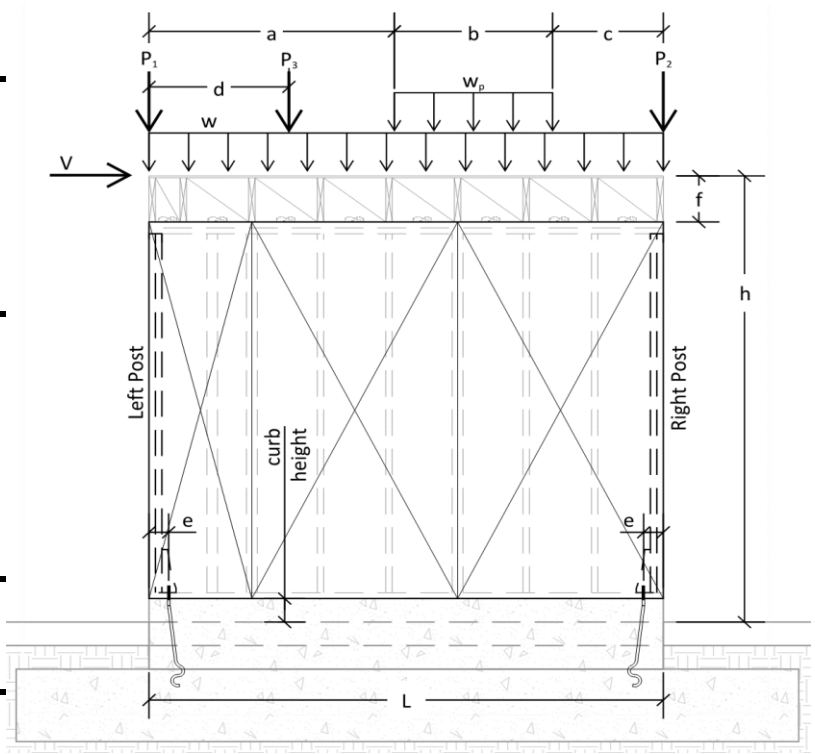
& MST ± 48 STRAP @ STEEL BM (T_{ALL} = 5070 #)

COFFMAN ENGINEERS	project	by	sheet no.
	location	date	
	client	checked	job no.
		date	

JOB			
SHEET NO.		OF	
CALCULATED BY		DATE	
CHECKED BY		DATE	

One-Story Wood Shear Wall Design

Wall ID: B/E



Assumptions

- Allowable Stress Design
- Load Combos: D + 0.6W
  D + 0.75(0.6W) + 0.75L_r
  (1.0 + 0.14S_{DS})D + 0.7E
  0.6D + 0.6W
  (0.6 - 0.14S_{DS})D + 0.7E

Vertical Loads

	D	L_r				
w:	476	1540	plf	a:	0	ft
w_p:	0	0	plf	b:	19	ft
P_1:	5100	16500	lb	c:	0	ft
P_2:	5100	16500	lb	d:	0	ft
P_3:	0	0	lb	e:	0.5	ft
w_{wall}:	15		psf			

Lateral Loads

V_E:	10400	lb	V_W:	0	lb
S_{DS}:	1.000	g			

Wall Dimensions

Wall Length, L:	19	ft			
Floor-to-Roof Height, h:	14	ft			
Roof Framing Height, f:	1	ft			
Nominal wall width:	6x		Curb height:	0	in

Plywood/OSB Sheathing

Sheathing Layers:	One side of wall	$0.7v_E$:	383	plf	**15/32" Structural I Panels**
Sheathing Grade:	Structural I	$0.6v_W$:	0	plf	**One side of wall**
Sheathing Thickness:	15/32"	$v_{allowable}$:	510	plf (seismic) OK	**Edge Nails: 10d @ 4" OC**
Minimum Nail Size:	10d	$v_{allowable}$:	715	plf (wind) OK	**Field Nails: 10d @ 12" OC**
Wall Aspect Ratio:	0.7:1 < 2:1, OK				

Check Bundled Studs or Posts at Wall Ends for Compression

	Left Post		Right Post	
Size, Species, & Grade:	4x6	DF-L #2	Steel	DF-L #2
Unbraced Height & Slenderness Ratio:	12.625 ft	28 < 50 OK	12.625 ft	#N/A
Load Case:	**Seismic**	**Wind**	**Seismic**	**Wind**
Axial Load, P_a:	11,323 lb	WIND NEGLECTED FOR INTERIOR CONDITION OF NEW CONSTR.	11,323 lb	WIND NEGLECTED FOR INTERIOR CONDITION OF NEW CONSTR.
Axial Stress, f_c:	588 psi		#N/A	
Allowable Stress, F'_c:	589 psi		#N/A	
Check Axial:	589 > 588 OK		#N/A	
Allowable Bearing, F'_{cp}:	625 psi		625 psi	
Check Sill Crushing:	625 > 588 OK		#N/A	

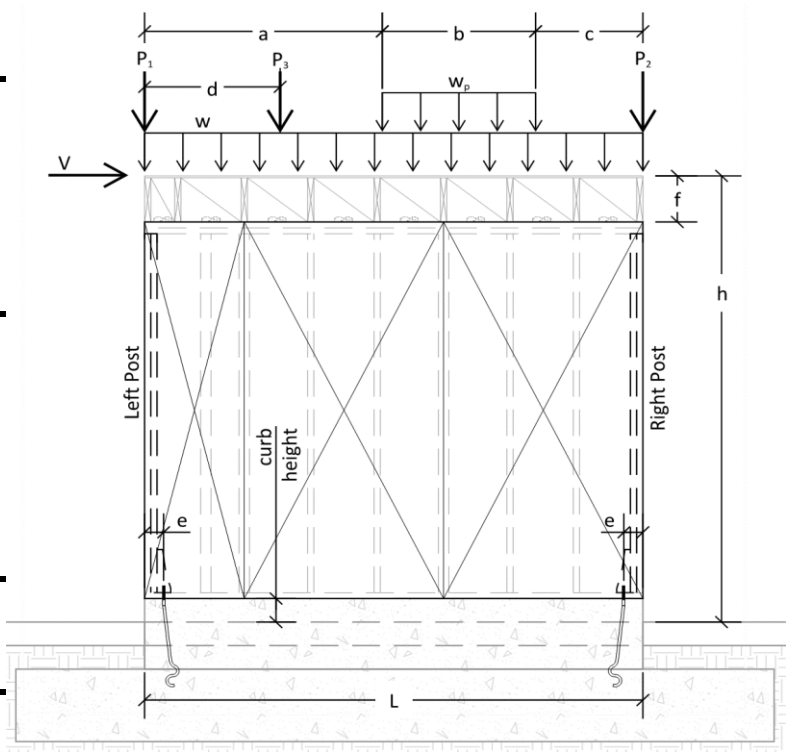
Check Holdowns at Each Wall End

	Left Holdown		Right Holdown	
Member & Thickness at Holdown:	4x6	b = 3.5 in	Steel	#N/A
Holdown & Capacity:	HDU2-SDS2.5	3,075 lb	#N/A	#N/A
Load Case:	**Seismic**	**Wind**	**Seismic**	**Wind**
Uplift, T_a:	88 lb	0 lb	88 lb	0 lb
Check Uplift:	OK	OK	#N/A	#N/A

JOB			
SHEET NO.		OF	
CALCULATED BY		DATE	
CHECKED BY		DATE	

One-Story Wood Shear Wall Design

Wall ID: 3.5



Assumptions

- Allowable Stress Design
- Load Combos: D + 0.6W
  D + 0.75(0.6W) + 0.75L_r
  (1.0 + 0.14S_{DS})D + 0.7E
  0.6D + 0.6W
  (0.6 - 0.14S_{DS})D + 0.7E

Vertical Loads

	D	L_r				
w:	306	990	plf	a:	0	ft
w_p:	0	0	plf	b:	21	ft
P_1:	4250	13750	lb	c:	0	ft
P_2:	4250	13750	lb	d:	0	ft
P_3:	0	0	lb	e:	0.5	ft
w_{wall}:	15		psf			

Lateral Loads

V_E:	10100	lb	V_W:	0	lb
S_{DS}:	1.000	g			

Wall Dimensions

Wall Length, L:	21	ft
Floor-to-Roof Height, h:	14	ft
Roof Framing Height, f:	1	ft
Nominal wall width:	6x	
Curb height:	0	in

Plywood/OSB Sheathing

Sheathing Layers:	One side of wall	$0.7v_E$:	337 plf	**15/32" Structural I Panels**
Sheathing Grade:	Structural I	$0.6v_W$:	0 plf	**One side of wall**
Sheathing Thickness:	15/32"	$v_{allowable}$:	340 plf (seismic) OK	**Edge Nails: 10d @ 6" OC**
Minimum Nail Size:	10d	$v_{allowable}$:	475 plf (wind) OK	**Field Nails: 10d @ 12" OC**
Wall Aspect Ratio:	## < 2:1, OK			

Check Bundled Studs or Posts at Wall Ends for Compression

	Left Post		Right Post	
Size, Species, & Grade:	Steel	DF-L #2	Steel	DF-L #2
Unbraced Height & Slenderness Ratio:	12.625 ft	#N/A	12.625 ft	#N/A
Load Case:	**Seismic**	**Wind**	**Seismic**	**Wind**
Axial Load, P_a:	9,673 lb	WIND NEGLECTED FOR INTERIOR CONDITION OF NEW CONSTR.	9,673 lb	WIND NEGLECTED FOR INTERIOR CONDITION OF NEW CONSTR.
Axial Stress, f_c:	#N/A		#N/A	
Allowable Stress, F'_c:	#N/A		#N/A	
Check Axial:	#N/A		#N/A	
Allowable Bearing, F'_{cp}:	625 psi		625 psi	
Check Sill Crushing:	#N/A		#N/A	

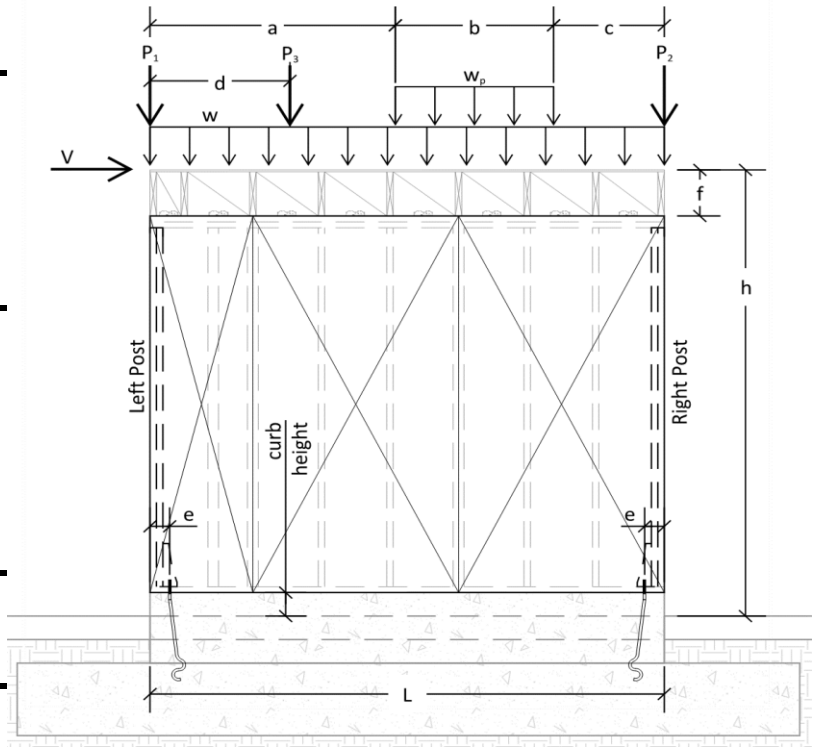
Check Holdowns at Each Wall End

	Left Holdown		Right Holdown	
Member & Thickness at Holdown:	Steel	#N/A	Steel	#N/A
Holdown & Capacity:	#N/A	#N/A	#N/A	#N/A
Load Case:	**Seismic**	**Wind**	**Seismic**	**Wind**
Uplift, T_a:	347 lb	0 lb	347 lb	0 lb
Check Uplift:	#N/A	#N/A	#N/A	#N/A

JOB			
SHEET NO.		OF	
CALCULATED BY		DATE	
CHECKED BY		DATE	

One-Story Wood Shear Wall Design

Wall ID: 5.5



Assumptions

- Allowable Stress Design
- Load Combos: D + 0.6W
  D + 0.75(0.6W) + 0.75L_r
  (1.0 + 0.14S_{DS})D + 0.7E
  0.6D + 0.6W
  (0.6 - 0.14S_{DS})D + 0.7E

Vertical Loads

	D	L_r				
w:	272	880	plf	a:	0	ft
w_p:	0	0	plf	b:	7.5	ft
P_1:	1275	4125	lb	c:	0	ft
P_2:	1275	4125	lb	d:	0	ft
P_3:	0	0	lb	e:	0.5	ft
w_{wall}:	15		psf			

Lateral Loads

V_E:	4280	lb	V_W:	0	lb
S_{DS}:	1.000	g			

Wall Dimensions

Wall Length, L:	7.5	ft
Floor-to-Roof Height, h:	14	ft
Roof Framing Height, f:	1	ft
Nominal wall width:	6x	
Curb height:	0	in

Plywood/OSB Sheathing

Sheathing Layers:	One side of wall	$0.7v_E$:	399	plf	**15/32" Structural I Panels**
Sheathing Grade:	Structural I	$0.6v_W$:	0	plf	**One side of wall**
Sheathing Thickness:	15/32"	$v_{allowable}$:	510	plf (seismic) OK	**Edge Nails: 10d @ 4" OC**
Minimum Nail Size:	10d	$v_{allowable}$:	715	plf (wind) OK	**Field Nails: 10d @ 12" OC**
Wall Aspect Ratio:	1.7:1 < 2:1, OK				

Check Bundled Studs or Posts at Wall Ends for Compression

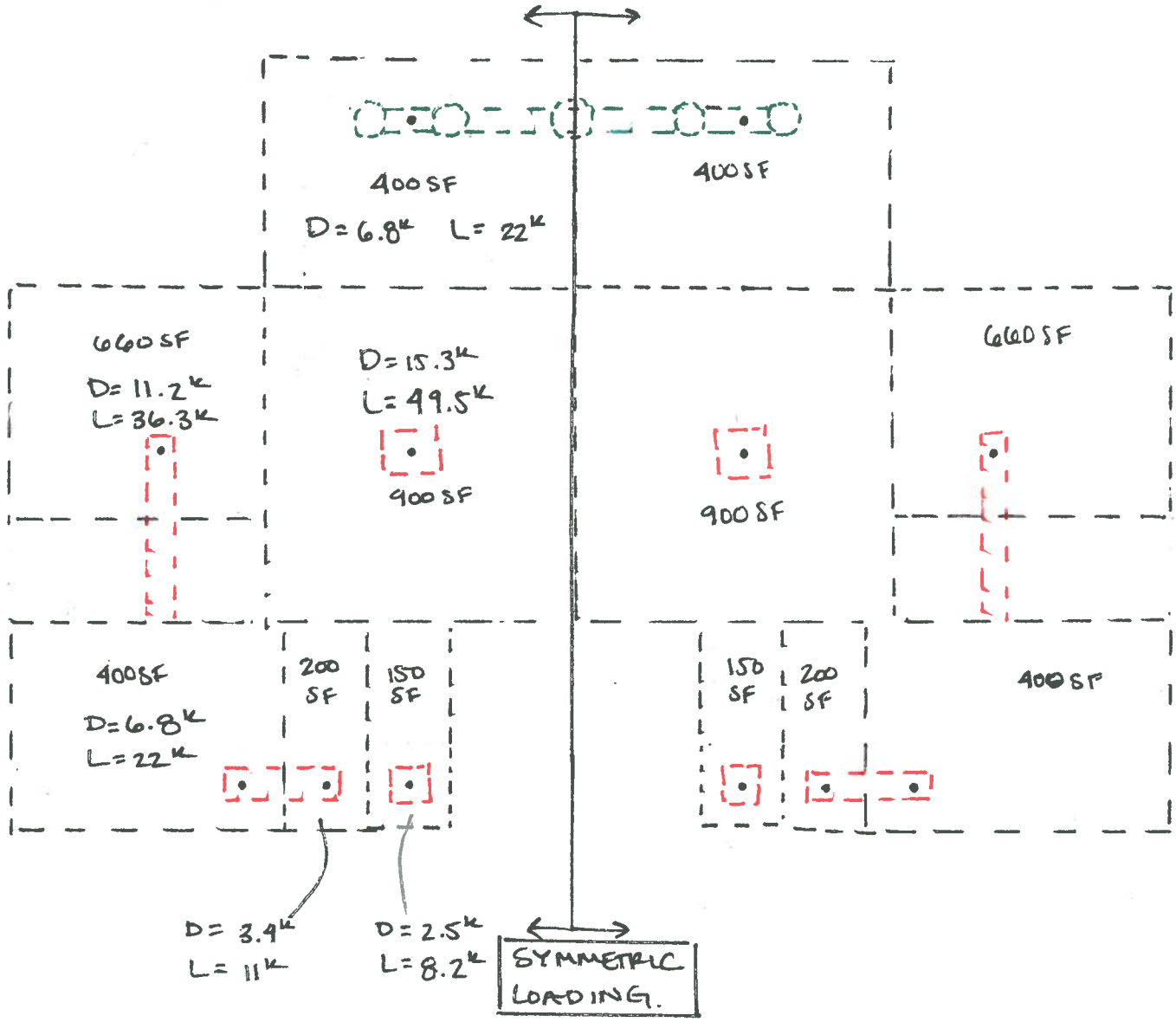
	Left Post		Right Post	
Size, Species, & Grade:	4x6	DF-L #2	4x6	DF-L #2
Unbraced Height & Slenderness Ratio:	12.625 ft	28 < 50 OK	12.625 ft	28 < 50 OK
Load Case:	**Seismic**	**Wind**	**Seismic**	**Wind**
Axial Load, P_a:	7,446 lb	WIND NEGLECTED FOR INTERIOR CONDITION OF NEW CONSTR.	7,446 lb	WIND NEGLECTED FOR INTERIOR CONDITION OF NEW CONSTR.
Axial Stress, f_c:	387 psi		387 psi	
Allowable Stress, F'_c:	589 psi		589 psi	
Check Axial:	589 > 387 OK		589 > 387 OK	
Allowable Bearing, F'_{cp}:	625 psi		625 psi	
Check Sill Crushing:	625 > 387 OK		625 > 387 OK	

Check Holdowns at Each Wall End

	Left Holdown		Right Holdown	
Member & Thickness at Holdown:	4x6	b = 3.5 in	4x6	b = 3.5 in
Holdown & Capacity:	HDU5-SDS2.5	5,645 lb	HDU5-SDS2.5	5,645 lb
Load Case:	**Seismic**	**Wind**	**Seismic**	**Wind**
Uplift, T_a:	4,500 lb	0 lb	4,500 lb	0 lb
Check Uplift:	OK	OK	OK	OK

FOUNDATION LOAD DETERMINATION

• = COL LOCATION
– – = Trib AREA



Tmax Uplift = 4.5k per Wood Shear Walls

☐ Check 10" Ø x 20' DP CONC PILE CAPACITY

$$T_{cap} = 500\,psf \times \frac{10''}{12} \times \pi \times 20' = \underline{\underline{26^k \text{ each}}}$$

NOTE: DEFERRED APPROVAL OF MICROPILES FOR FOUNDATION SYSTEM.

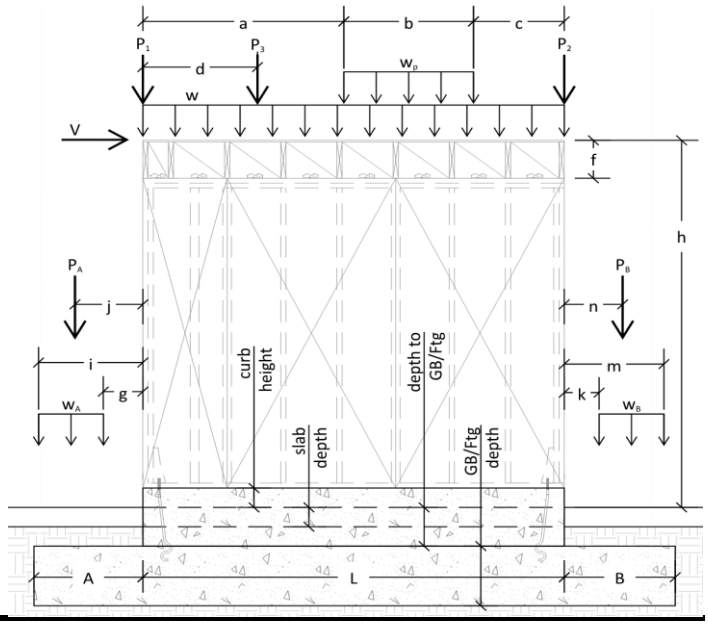
COFFMAN ENGINEERS	project	by	sheet no.
	location	date	
	client	checked	job no.
		date	

JOB			
SHEET NO.	1	OF	2
CALCULATED BY		DATE	
CHECKED BY		DATE	

One-Story Shear Wall Grade Beam Design (page 1)

Vertical Loads

	D	L_r				
w:	272	880	plf	a:	0.00	ft
w_p:	0	0	plf	b:	7.50	ft
P_1:	1275	4125	lb	c:	0.00	ft
P_2:	1275	4125	lb	d:	0.00	ft
P_3:	0	0	lb	g:	0.00	ft
w_{wall}:	15		psf	i:	2.00	ft
w_A:	0	0	plf	j:	2.00	ft
w_B:	0	0	plf	k:	0.00	ft
P_A:	0	0	lb	m:	2.00	ft
P_B:	0	0	lb	n:	2.00	ft



Lateral Loads (Strength)

V_E:	4280	lb
V_W:	0	lb
S_{DS}:	1.000	g
h:	14.00	ft

Grade Beam Properties

slab depth:	4	in	GB depth:	24	in	A:	2.00	ft	$d_{top\ bars}$: 20 in
slab width:	5	ft	GB width:	18	in	B:	2.00	ft	$d_{bott\ bars}$: 20 in
slab weight:	2.0	k	f'_c:	4.3	ksi	l_w:	7.5	ft	Allow. Soil Brg: 2.50 ksf
GB Weight:	5.2	k	f_y (long.):	60	ksi	l_{total}:	11.5	ft	Allow. Soil Brg: 3.33 ksf @ 1.33x
			f_y (stirrups):	40	ksi				

Stability and Soil Pressure Check

------- LOAD CASE 1 : DL ---------

Total Weight = 13.2 k

FORCES FROM THE RIGHT

0.90*M(R)= 69 ft-k > M (OT) O.K.

FORCES FROM THE LEFT

0.90*M(R)= 69 ft-k > M (OT) O.K.

--------LOAD CASE 2 : D+L ---------

Total Weight = 28.1 k

FORCES FROM THE RIGHT

0.90*M(R)= 145 ft-k > M (OT) O.K.

FORCES FROM THE LEFT

0.90*M(R)= 145 ft-k > M (OT) O.K.

USE 18.0 W X 24.0 D GRD BM
W/2-#5 TOP & 3-#5 BOT
W/1- #4 Legs @ 18.0 in o/c

BEARING PRESSURE NOT CONSIDERED FOR GRADE BEAMS.
MICROPILES TO BE DESIGNED FOR GRAVITY LOADING.

----------LOAD CASE 3 : DUCTILITY ----------

Design for: Ω_o = 3.0

a. Maximum anticipated forces: Ω_o * M(OT) = 205 k-ft

Occurs when Ω_o * M(OT) < 0.90*M(R)TL

b. Maximum force that can be delivered to the system:

Occurs when Ω_o * M(OT) > 0.90*M(R)TL +Mu = Total Wt*(X)*1.4

FORCES FROM THE LEFT

0.90*M(R)= 75 ft-k < M (OT)

Reaction Pt. = 0.0 ft:

+Mu (X2) = 895 in-k

-Mu (X1) = 21 in-k

FORCES FROM THE RIGHT

0.90*M(R)= 75 ft-k < M (OT)

Reaction Pt. = 0.0 ft:

+Mu (X1) = 895 in-k

-Mu (X2) = 21 in-k

One-Story Shear Wall Grade Beam Design (page 2)

Reinforcing Check

USE 18.0 W X 24.0 D GRD BM
W/2-#5 TOP & 3-#5 BOT
W/1- #4 Legs @ 18.0 in o/c

Top Longitudinal Bars

From Load Cases 1 & 2 :

-Mu max = 21 in-k ==> -As req = 0.02 in^2 < 200/Fy = 1.20 in^2

-As req times 1.33 = 0.03 in^2

-As req < 200/Fy : Therefore Ductility requirements indicated above need to be checked.

From Load Case 3 :

-Mu max = 21 in-k (same as Load Cases 1 & 2)

From Load Cases 1 & 2 : -As req = 0.03 in^2 <---CONTROLS

From Load Case 3 : -As req = 0.02 in^2

With 200/Fy : -As req = 1.20 in^2

Note : Less than 200/Fy Reinforcing can be used since Grade Beam
will remain elastic under Maximum Anticipated Forces.

With:

(2) #5 TOP : -As = 0.62 in^2 > 0.03 in^2 øMn = 660 in-k

Bottom Longitudinal Bars

From Load Cases 1 & 2 :

+Mu max = 431 in-k ==> +As req = 0.40 in^2 < 200/Fy = 1.20 in^2

+As req times 1.33 = 0.54 in^2

+As req < 200/Fy : Therefore Ductility requirements indicated above need to be checked.

From Load Case 3 :

+Mu max = 895 in-k ==> +As req = 0.84 in^2

From Load Cases 1 & 2 : +As req = 0.54 in^2

From Load Case 3 : +As req = 0.84 in^2 <---CONTROLS

With 200/Fy : +As req = 1.20 in^2

Note : Less than 200/Fy Reinforcing can be used since Grade Beam
will remain elastic under Maximum Anticipated Forces.

With:

(3) #5 BOT : +As = 0.93 in^2 > 0.84 in^2 øMn = 983 in-k

Shear

Vu max = 18 k

øVc/2 = 20 k > Vu

Stirrups Not Required

With:

(1) #4 [

USE : 18 in Spacing

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JOB
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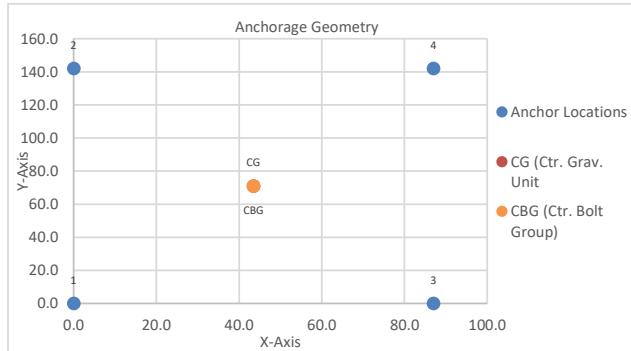
Equipment Anchorage Forces

Determine anchor forces at equipment support points. Based on ASCE 7-10, Chapters 11, 12, and 13

Input Data

Equipment Data and Information

	RTU 5		Equipment name or ID
W_t	=	3.75	Operating weight, kips
B_x	=	87	Long dimension of unit, in.
B_y	=	142	Short dimension of unit, in.
B_z	=	50	Height of unit, in.
CG_x	=	43.5	"x" coordinate of center of gravity of unit, in.
CG_y	=	71	"y" coordinate of center of gravity of unit, in.
CG_z	=	33.333	"z" coordinate or height of center of gravity of unit above anchorage plane, in.
N	=	4	Number of anchors



Seismic Factors

F_p	=	0.30	W_t	Seismic design force per section 13.3.1
S_{DS}	=	1.000		Spectral acceleration for short period per Eqn. 11.4-3
Ω_0	=	1.00		Overstrength factor per ASCE 7-10 Supplement No. 1, Table 13.5-1 or Table 13.6-1

Anchor Locations

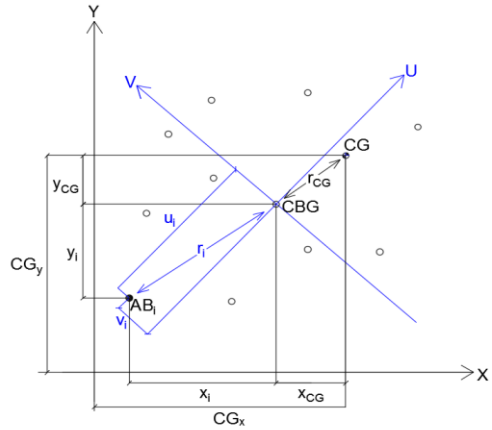
Loc.	x (in)	y (in)
1	0.0	0.0
2	0.0	142.0
3	87.0	0.0
4	87.0	142.0

Output Data - Anchor Forces

Anchor Group Design

Per ASCE 7-10 12.4.3.2 and ACI 318-14 section 17.2.3.5.3, CBG is center of anchor bolt group

E_h	=	$1.0 \times F_p \times W_t \times \Omega_o$	=	**1,125 lb**	
$\Sigma F_{z,min}$	=	$(0.9 \times W_t) - (0.2 S_{DS} \times W_t)$	=	**2,625 lb**	2015 IBC Eq. 16-7
$\Sigma F_{z,max}$	=	$(1.2 \times W_t) + (0.2 S_{DS} \times W_t)$	=	**5,250 lb**	2015 IBC Eq. 16-5
$M_{seis.}$	=	$E_h \times CG_z$	=	**37,500 in-lb**	Overturning due to seismic force
$M_{grav.x}$	=	$\Sigma F_{z,min} \times y_{CG}$	=	**0 in-lb**	Overturning due to weight eccentricity
$M_{grav.y}$	=	$\Sigma F_{z,min} \times x_{CG}$	=	**0 in-lb**	Overturning due to weight eccentricity
T_z	=	$E_h \times r_{CG}$	=	**0 in-lb**	Addt'l torsion due to weight eccentricity



Random bolt locations for visual purposes only

U,V directions vary and are based on worst case seismic loading direction for bolt location within bolt pattern

Single Anchor Design - For vertically cantilevered systems

Worst case horizontal loading direction per ASCE 7-10, 13.3.1 has been considered for each anchor

Note 1: u,v coordinate system created for worst case loading direction and is unique to each anchor

Note 2: $M_{seis.\ v}$ and $M_{seis.\ u}$ are components of $M_{seis.}$ in u,v, dir.

Note 3: u, v, I_u, I_v vary and correspond to worst seismic loading direction for each A.B. location

Over-strength level tension (N) = $\frac{M_{seis.v} \times u_i}{I_v} + \frac{M_{seis.u} \times v_i}{I_u} + \frac{M_{grav.x} \times y_i}{I_x} + \frac{M_{grav.y} \times x_i}{I_y} - \frac{\Sigma F_{z,min}}{N}$ = **0 lb**

Over-strength level shear (V) = $\frac{T_z \times r_i}{\Sigma r_i^{\,2}} + \frac{E_h}{N}$ = **281 lb**

Proprietary software is provided by anchor manufacturers to design anchor groups and single anchors:

CAPACITY OF 3/8" DIA. LAG SCREW = 100# x (Cd = 1.6) = 160# EA.

Typ. Unit Anchorage, Anchor Forces All Dir.

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

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 7 MAY 2019, 4:32PM

Steel Beam

File = C:\PROGRA~2\ENERCA~1ENERCALC_6 .
Software copyright ENERCALC, INC. 1983-2019, Build:10.18.12.31 .

Lic. # : KW-06003501 Licensee : COFFMAN ENGINEERS

Description : Steel Guide Rail Support

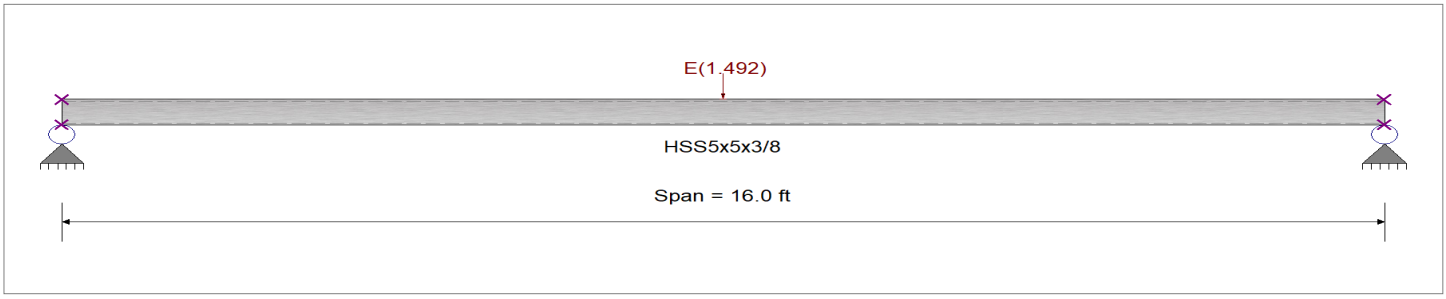
CODE REFERENCES

Calculations per
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Minor Axis Bending

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Load(s) for Span Number 1
Point Load : E = 1.492 k @ 8.0 ft, (Elevator Seismic Force)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.158 : 1	Maximum Shear Stress Ratio =	0.011 : 1
Section used for this span	HSS5x5x3/8	Section used for this span	HSS5x5x3/8
Ma : Applied	4.178 k-ft	Va : Applied	0.5222 k
Mn / Omega : Allowable	26.447 k-ft	Vn/Omega : Allowable	49.566 k
Load Combination	E Only * 0.70	Load Combination	E Only * 0.70
Location of maximum on span	8.000 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 Transient Deflection	0.351 in	Ratio =	546	>=0
Max Upward Transient Deflection	0.000 in	Ratio =	0	>=0
Max Downward Total Deflection	0.246 in	Ratio =	781	>=0
Max Upward Total Deflection	0.000 in	Ratio =	0	>=0

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	Mny	Mny/Omega	Cb	Rm	Va Max	Vny	Vny/Omega
Dsgn. L = 16.00 ft	1		0.000				44.17	26.45	1.00	1.00	-0.00	82.78	49.57
E Only * 0.70													
Dsgn. L = 16.00 ft	1	0.158	0.011	4.18		4.18	44.17	26.45	1.32	1.00	0.52	82.78	49.57
E Only * 0.5250													
Dsgn. L = 16.00 ft	1	0.118	0.008	3.13		3.13	44.17	26.45	1.32	1.00	0.39	82.78	49.57

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
E Only	1	0.3511	8.000		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.746	0.746
Overall MINimum	0.392	0.392
E Only * 0.70	0.522	0.522
E Only * 0.5250	0.392	0.392
E Only	0.746	0.746