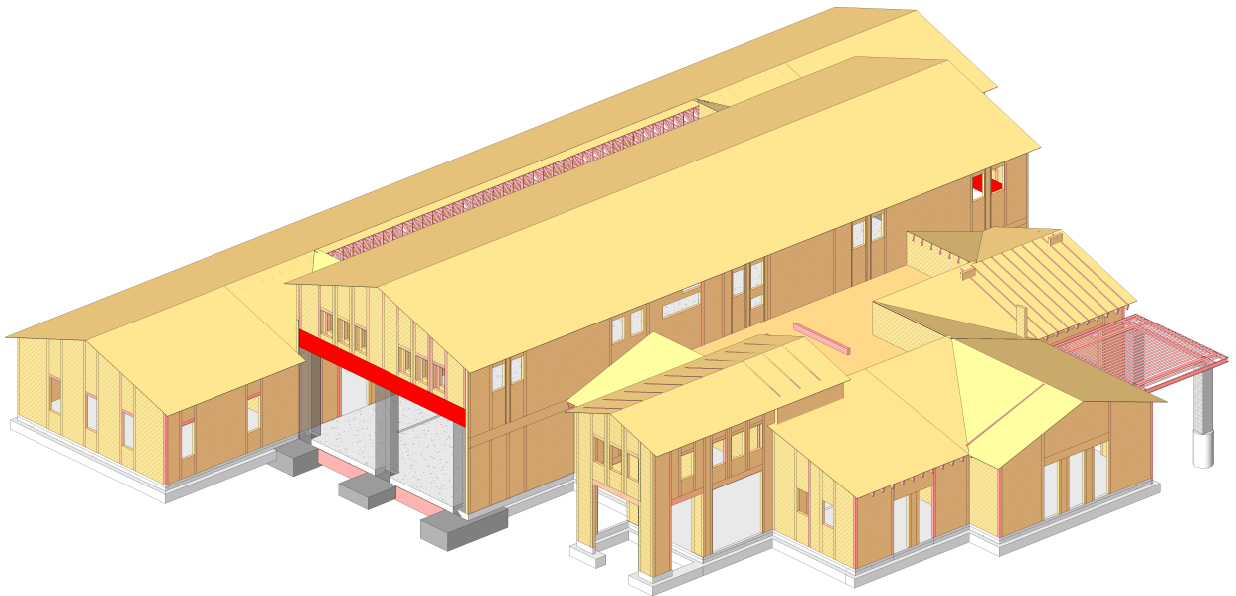

STRUCTURAL CALCULATIONS

for

FIRE STATION 46

Valencia, CA

100% CD SUBMITTAL



Prepared by:



725 S. Figueroa St.
Los Angeles, California 90017
(213) 315-2277

SBI Job # 25534

May 05, 2026

TABLE OF CONTENTS

FIRE STATION 46

SBI JOB NO.: 25534

1	GRAVITY DESIGN	4
1.1	LOADING CRITERIA.....	5
1.2	TYPICAL WOOD JOIST DESIGN	14
1.3	WOOD BEAM DESIGN	30
1.4	WOOD POST DESIGN.....	61
1.5	WOOD STUD WALL DESIGN	65
2	LATERAL DESIGN.....	74
2.1	SEISMIC LOADING CRITERIA.....	77
2.2	DESIGN BASE SHEAR	80
2.3	PLYWOOD SHEAR WALL DESIGN.....	94
2.4	SHEAR WALL HOLDOWN DESIGN.....	101
2.5	DIAPHRAGM DESIGN.....	114
2.6	CHORD DESIGN	120
2.7	DRAG DESIGN.....	122
2.8	CONCRETE MOMENT FRAME DESIGN	132
3	FOUNDATION DESIGN.....	307
3.1	GRAVITY FOOTING DESIGN	308
3.2	MOMENT FRAME FOOTING DESIGN.....	315
3.3	SHEAR WALL FOOTING DESIGN	340
4	MISCELLANEOUS	357
4.1	CANOPY DESIGN.....	358

4.2	TRASH ENCLOSURE DESIGN	384
4.3	SITE WALL DESIGN	399
4.4	FUEL DISPENSING AREA DESIGN	416
4.5	FUEL DISPENSING CANOPY DESIGN	435
4.6	REVERSE APPARATUS DESIGN.....	457
4.7	UNIT	488
4.8	AWNING	500
4.9	LOW WALL DESIGN.....	520
4.10	LOADING ON PREFAB WOOD TRUSSES.....	527
4.11	CHECK MEMBERS FOR 4" DIAMTER FIRE SPRINKLER LINE.....	

1 GRAVITY DESIGN

1.1 LOADING CRITERIA



Project: Fire Station 46
 SBI Job No.:
 Description: Loading Criteria
 Date: 5/28/2025

MAIN BUILDING

Level: Apparatus Roof-Without PV
 Dead Load:

Item	Diaphragm	Joist	Truss	Seismic
Standing Seam (22GA)	4.0			4.0
1/4" Densedeck	1.0			1.0
6" Rigid Insulation	1.5			1.5
19/32" Plywood Diaphragm	2.0			2.0
Ceiling/MEP	5.0			5.0
Sprinkler	3.0			3.0
Misc	5.0			3.0
Wood Truss @ 2'-0" o.c.			4.0	4.0
	21.5 psf	21.5 psf	25.5 psf	23.5 psf

Live Load: Roof 20.0 psf (Reducible)
 Dead loads has been increased to account for slope roof

Level: Apparatus Roof- With PV
 Dead Load:

Item	Diaphragm	Joist	Truss	Seismic
Solar	6			6
Standing Seam (22GA)	4.0			4.0
1/4" Densedeck	1.0			1.0
6" Rigid Insulation	1.5			1.5
19/32" Plywood Diaphragm	2.0			2.0
Ceiling/MEP	5.0			5.0
Sprinkler	3.0			3.0
Misc	5.0			3.0
Wood Truss @ 2'-0" o.c.			4.0	4.0
	27.5 psf	27.5 psf	31.5 psf	29.5 psf

Live Load: Roof 20.0 psf (Reducible)
 Dead loads has been increased to account for slope roof

Level: Flat Roof
 Dead Load:

Item	Gravity	Joist	Truss	Seismic
Roofing	4.0			4.0
1/4" Densedeck	1.0			1.0
6" Rigid Insulation	1.5			1.5
19/32" Plywood Diaphragm	2.0			2.0
Ceiling/MEP	5.0			5.0
Sprinkler	3.0			3.0
Misc	1.5			1.5
Joist Framing		2.0		2.0
Mechanical Units (Seismic Only)	0.0			6.0
Partitions (Seismic Only)	0.0			5.0
	18.0 psf	20.0 psf	20.0 psf	31.0 psf

Live Load: Mechanical Roof 50.0 psf (Non-Reducible)

Level: Sloped Roof Without Solar Panel

Dead Load:

Item	Gravity	Joist	Truss	Seismic
Standing Seam (22GA)	4.0			4.0
1/4" Densedeck	1.0			1.0
6" Rigid Insultation	1.5			1.5
19/32" Plywood Diaphragm	2.0			2.0
Ceiling/MEP	5.0			5.0
Sprinkler	3.0			3.0
Misc	1.5			1.5
Wood Truss @ 24" o.c.			4.0	4.0
Partitions (Seismic Only)	0.0			5.0
18.0 psf 18.0 psf 22.0 psf 27.0 psf				

Live Load: Roof

20.0 psf

(Reducible)

Dead loads has been increased to account for slope roof

Level: Sloped Roof With Solar Panel

Dead Load:

Item	Gravity	Joist	Truss	Seismic
Solar Panel	6.0			6.0
Standing Seam (22GA)	4.0			4.0
1/4" Densedeck	1.0			1.0
6" Rigid Insultation	1.5			1.5
19/32" Plywood Diaphragm	2.0			2.0
Ceiling/MEP	5.0			5.0
Sprinkler	3.0			3.0
Misc	1.5			1.5
Wood Truss @ 24" o.c.			4.0	4.0
Partitions (Seismic Only)	0.0			5.0
24.0 psf 24.0 psf 28.0 psf 33.0 psf				

Live Load: Roof

20.0 psf

(Reducible)

Dead loads has been increased to account for slope roof

RESERVE APPARATUS

Level: Reserve Apparatus Roof

Dead Load:

Item	Diaphragm	Joist	Truss	Seismic
Standing Seam (22GA)	4.0			4.0
1/4" Densedeck	1.0			1.0
6" Rigid Insultation	1.5			1.5
19/32" Plywood Diaphragm	2.0			2.0
Ceiling/MEP	5.0			4.0
Sprinkler	3.0			2.0
Misc	3.0			2.0
Wood Truss @ 2'-0" o.c.			4.0	4.0
19.5 psf 19.5 psf 23.5 psf 20.5 psf				

Live Load: Roof

20.0 psf

(Reducible)

Dead loads has been increased to account for slope roof

Fuel rea
Flat Roof Building

Level:

Dead Load:

Item	Diaphragm	Joist	Truss	Seismic
Roofing	4.0			4.0
1/4" Densedeck	1.0			1.0
6" Rigid Insultation	1.5			1.5
19/32" Plywood Diaphragm	2.0			2.0
Ceiling/MEP	5.0			4.0
Sprinkler	3.0			2.0
Misc	3.0			2.0
2x Joist @ 16" o.c.		3.0		3.0
	19.5 psf	22.5 psf	22.5 psf	19.5 psf

Live Load: Roof 20.0 psf (Reducible)

Level: Canopy

Dead Load:

Item	Diaphragm	Joist	Column	Seismic
Roofing	4			4
1/4" Densedeck	1			1
6" Rigid Insultation	1.5			1.5
3" Metal Deck	3			3
Ceiling/MEP	3			3
Sprinkler	3			3
Misc	3			3
Steel Beam		3		3
Steel Columns			3	3
	18.5	21.5	24.5	24.5

Live Load: Roof 20.0 psf (Reducible)

Dead loads has been increased to account for slope roof

2x12 @ 16" o.c

$$1.5" * 11.25" * 32 \text{ pcf} / (12 * 12) / (16" / 12) = 2.9 \text{ psf}$$

6x12 between truss

$$5.5" * 11.5" * 32 \text{ pcf} / (12 * 12) / 38' / 2 = 0.18 \text{ psf}$$

Wood Truss @ 8'-0" o.c

Assume 6x 12 T & B & vertical & diagonal

$$5.5" * 11.5" * 32 / (12 * 12) * 3 / 8 \text{ ft} = 5.0 \text{ psf}$$

interior walls

Pick A= 465 ft²

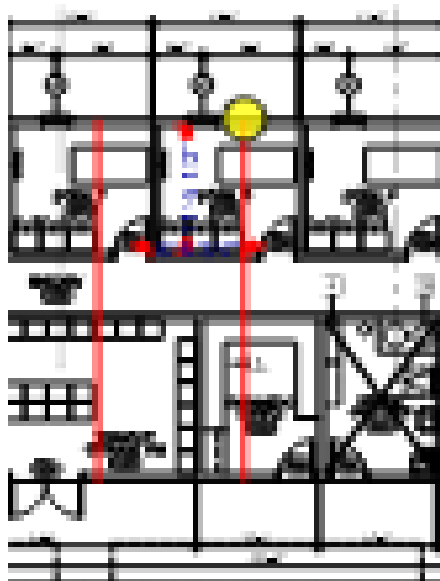
Total interior wall Length= 11ft+ 8 ft+ 13ft +13 ft= 45 ft

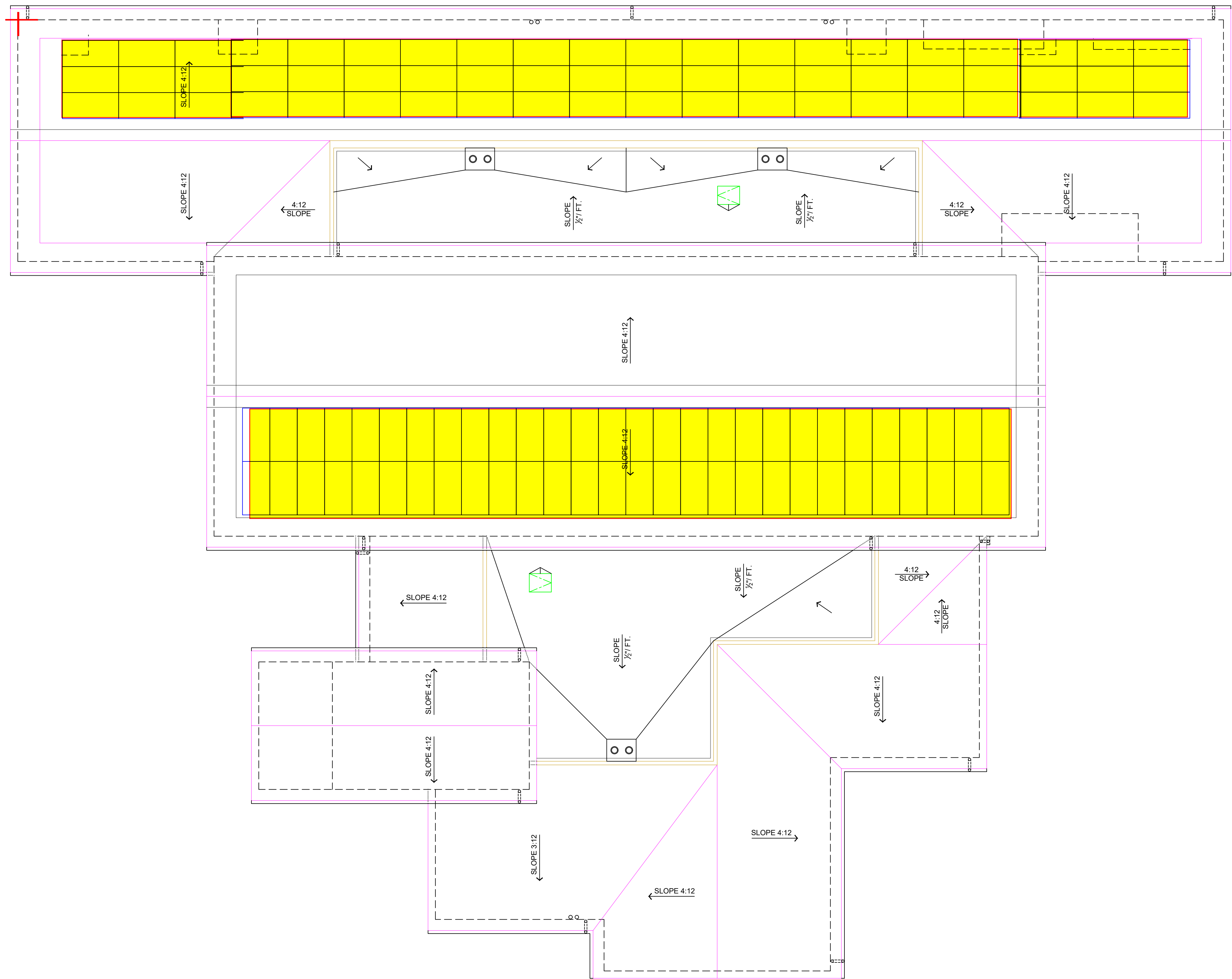
Wall height 11 ft:

Total Wall = 45 ft* 11 ft/2= 247 ft²

Total W= 247 ft²* 8 psf= 1980 lb

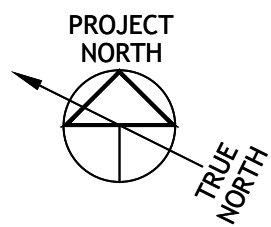
Interior wall lb/ft²= 1980 lb/462 ft²= 4.2 psf use 5 psf for Seismic Weight





ROOF PLAN


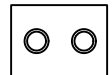
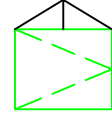
SCALE: 1/8"=1'-0"



NOTES

ROOF PLAN

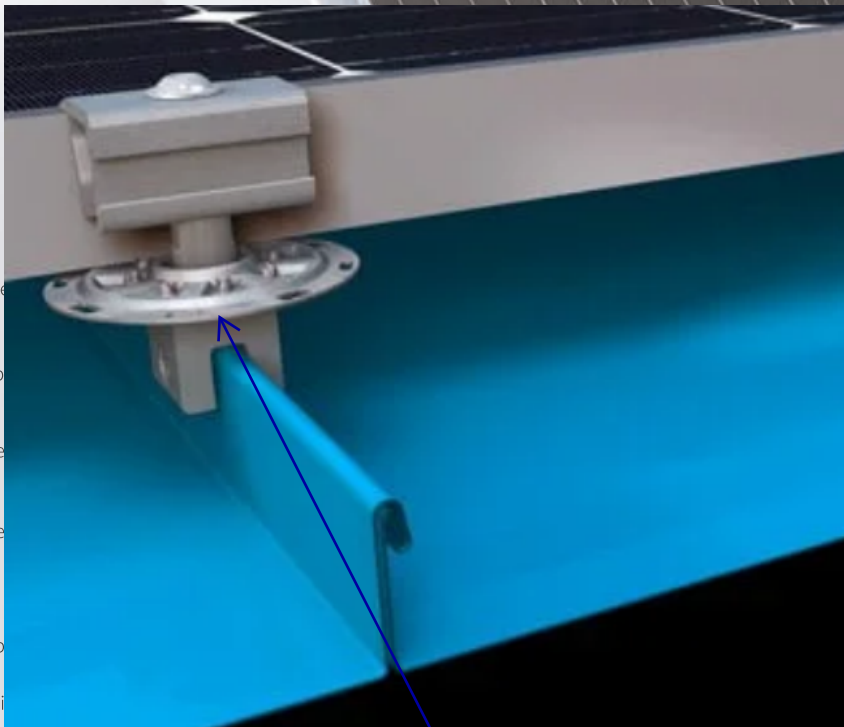
LEGEND

-  STANDING SEAM METAL ROOF
-  ROOF DRAIN AND OVERFLOW
-  ROOF ACCESS HATCH

1/16" = 1'-0"

600W **LB**
Series2.54 psf
panel

TOTAL = 2.54 PSF + 3 PSF =
5.54 PSF LESS THAN 6 PSF
WE ASSUMED



n-type Bifacial Double Glass
High Efficiency Mono Module
JAM72D40 LB

575-600

< 3.0 psf
S-5 Clamp
(per email)

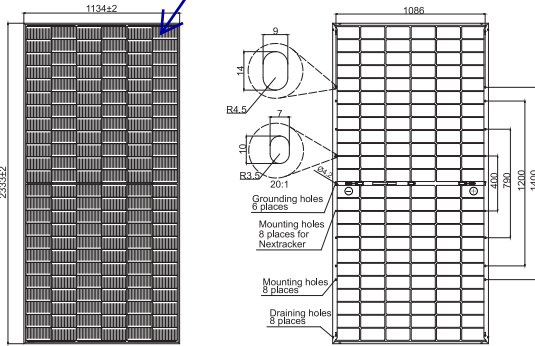
Comprehensive Certificates

- IEC 61215, IEC 61730
- ISO 9001: 2015 Quality management systems
- ISO 14001: 2015 Environmental management systems
- ISO 45001: 2018 Occupational health and safety management systems
- IEC 62941: 2019 Terrestrial photovoltaic (PV) modules - Quality system for PV module manufacturing





600W

575-600
JAM72D40LB
Series

Remark: customized frame color and cable length available upon request

Cell	Mono-16BB
Weight	32.5kg
Dimensions	2333±2mm×1134±2mm×30±1mm
Cable Cross Section Size	4mm ² (IEC), 12 AWG(UL)
No. of cells	144(6×24)
Junction Box	IP68, 3 diodes
Connector	QC 4.10-35I/ MC4-EVO2A
Cable Length (Including Connector)	Portrait: 300mm(+)/400mm(-); 800mm(+)/800mm(-)(Leapfrog) Landscape: 1400mm(+)/1400mm(-)
Front Glass/Back Glass	2.0mm/2.0mm
Packaging Configuration	36pcs/Pallet, 720pcs/40HQ Container

ELECTRICAL PARAMETERS AT STC

TYPE	JAM72D40 -575/LB	JAM72D40 -580/LB	JAM72D40 -585/LB	JAM72D40 -590/LB	JAM72D40 -595/LB	JAM72D40 -600/LB
Rated Maximum Power(P _{max}) [W]	575	580	585	590	595	600
Open Circuit Voltage(V _{oc}) [V]	51.40	51.60	51.80	52.00	52.20	52.40
Maximum Power Voltage(V _{mp}) [V]	42.88	43.06	43.24	43.41	43.59	43.76
Short Circuit Current(I _{sc}) [A]	14.16	14.23	14.29	14.35	14.42	14.48
Maximum Power Current(I _{mp}) [A]	13.41	13.47	13.53	13.59	13.65	13.71
Module Efficiency [%]	21.7	21.9	22.1	22.3	22.5	22.7
Power Tolerance	0~+5W					
Temperature Coefficient of I _{sc} (α _{Isc})	+0.046%/ °C					
Temperature Coefficient of V _{oc} (β _{Voc})	-0.260%/ °C					
Temperature Coefficient of P _{max} (γ _{Pmp})	-0.300%/ °C					
STC	Irradiance 1000W/m ² , cell temperature 25 °C, AM1.5G					

Remark: Electrical data in this catalog do not refer to a single module and they are not part of the offer. They only serve for comparison among different module types.

ELECTRICAL CHARACTERISTICS WITH 10% SOLAR IRRADIATION RATIO

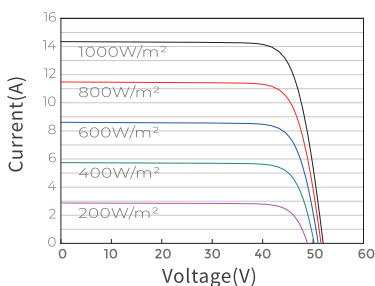
TYPE	JAM72D40 -575/LB	JAM72D40 -580/LB	JAM72D40 -585/LB	JAM72D40 -590/LB	JAM72D40 -595/LB	JAM72D40 -600/LB
Rated Max Power(P _{max}) [W]	621	626	632	637	643	648
Open Circuit Voltage(V _{oc}) [V]	51.40	51.60	51.80	52.00	52.20	52.40
Max Power Voltage(V _{mp}) [V]	42.88	43.06	43.24	43.41	43.59	43.76
Short Circuit Current(I _{sc}) [A]	15.30	15.36	15.43	15.50	15.57	15.64
Max Power Current(I _{mp}) [A]	14.48	14.55	14.61	14.68	14.74	14.81
Irradiation Ratio (rear/front)	10%					

*For NextTracker installations, maximum static load please take compatibility approve letter between JA Solar and NextTracker for reference.

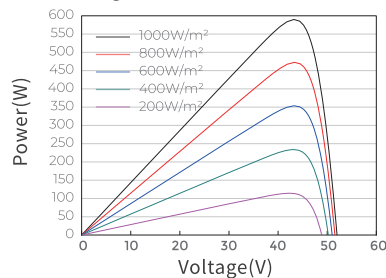
**Bifaciality=P_{max, rear}/Rated P_{max, front}

CHARACTERISTICS

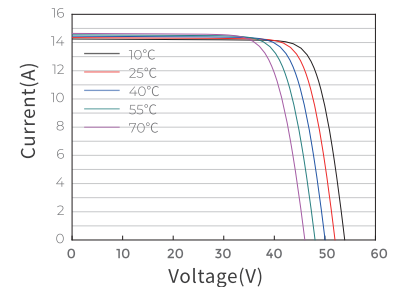
Current-Voltage Curve JAM72D40-590/LB



Power-Voltage Curve JAM72D40-590/LB

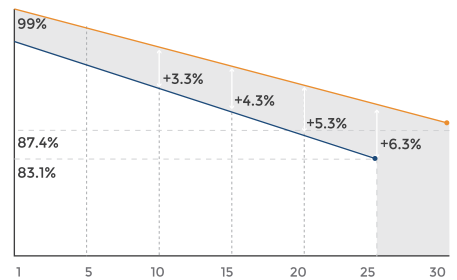


Current-Voltage Curve JAM72D40-590/LB



Superior Warranty

1% 1st-year Degradation
0.4% Annual Degradation Over 30 years




- n-type Bifacial Double Glass Module Linear Performance Warranty
- Standard Module Linear Performance Warranty

OPERATING CONDITIONS

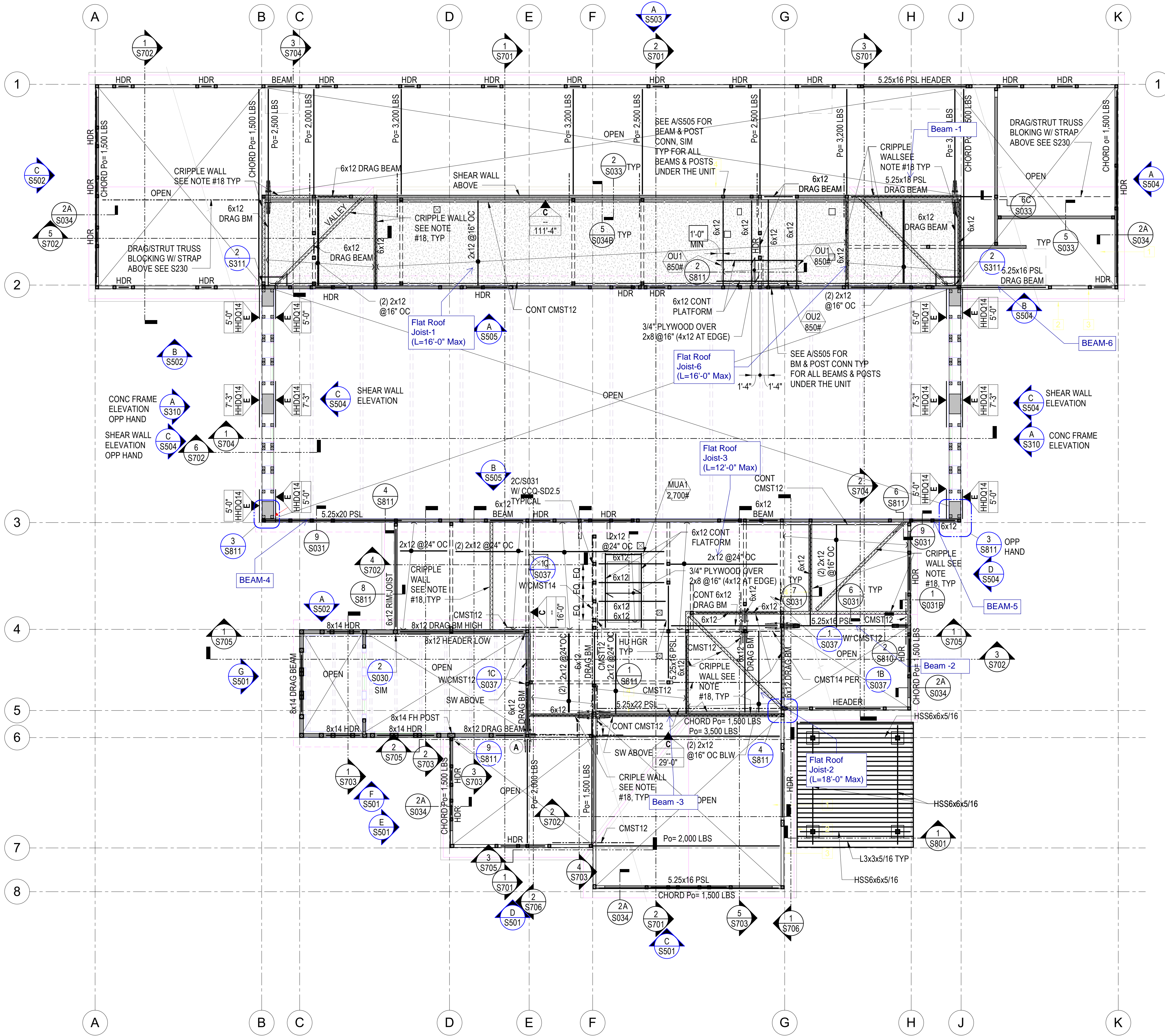
Maximum System Voltage	1500V DC
Operating Temperature	-40 °C ~ +85 °C
Maximum Series Fuse Rating	30A
Maximum Static Load, Front*	5400Pa (112 lb/ft ²)
Maximum Static Load, Back*	2400Pa (50 lb/ft ²)
NOCT	45±2 °C
Bifaciality**	80%±10%
Fire Performance	UL Type 29

◆ AI Overview

The S-5! PV Kit 2.0, which includes the Mid and EdgeGrab options, generally weighs around **0.18 lbs to 0.5 lbs** (82 to 227 grams), depending on the specific components and whether it's the Mid or Edge version, [according to Solaris-Shop.com](#) and [Unbound Solar](#). The Mid version, for example, is listed as 0.18 lbs, while the Edge version is listed as 0.5 lbs. 



1.2 TYPICAL WOOD JOIST DESIGN



ROOF JOISTS & BEAM KEY MAP

SEE PAGE # 17 FOR JOIST DESIGN
PAGE # 30 FOR BEAM DESIGN
& PAGE #51 FOR HEADER DESIGN

FRAMING PLAN NOTES

- FOR GENERAL NOTES SEE S0.0 SERIES AND TYPICAL DETAILS SEE S0 SERIES SHEETS.
- VERIFY ALL DIMENSIONS PRIOR TO START OF WORK. SEE ARCHITECTURAL DRAWINGS FOR REMAINDER OF DIMENSIONS NOT SHOWN ON THIS PLAN.
- WHERE A SHEAR WALL IS SHOWN AS PORTION OF A WALL, THE REMAINDER OF THE WALL (INCLUDING ABOVE AND BELOW OPENINGS, PARAPETS, ETC) SHALL BE SHEATHED WITH THE SAME SHEATHING AND NAILING SCHEDULE ALONG THE ENTIRE WALL. IN ADDITION, CORNERS OF ALL OPENINGS IN SHEAR WALL SHALL BE STRAPPED.
- ALL SHEAR WALLS AND SHEAR TRANSFER NAILS SHALL BE COMMON WIRE NAILS. SINKER AND BOX NAILS ARE NOT PERMITTED.
- HOLDOWN CONNECTORS SHALL BE TIGHTENED JUST PRIOR TO COVERING THE WALL FRAMING.
- INDICATES BEARING AND/OR SHEAR WALL BELOW.
- ROOF DIAPHRAGM SHEATHING AND NAILING SHALL BE PER SHEET S240
- TYPICAL WOOD STUD WALL: SEE NOTE #8 ON SHEET S210
- ALL HARDWARE ARE BY "SIMPSON" TYPICAL OR APPROVED EQUAL.
- ALL SAWN WOOD FRAMING EXPOSED TO WEATHER SHALL BE P.T.D.F.
- HDR INDICATES HEADER PER SCHEDULE SEE DETAIL UNO.
- INDICATES ROOF JOIST
- ROOF DIAPHRAGM NAILING TO BE INSPECTED BEFORE COVERING, FACE GRAIN OF PLYWOOD SHALL BE PERPENDICULAR TO SUPPORT.
- ALL NAILS ARE COMMON NAILS, UNO.
- CMST INDICATES SIMPSON CMST14 STRAP, UNO
- INDICATES SHEAR WALL TYPE PER SCHEDULE
- INDICATES HOLDOWN TYPE EACH END OF SHEAR WALL PANEL SEE SCHEDULE FOR HD HOLDOWNS UNO
- INDICATES SHEAR WALL DESIGN LENGTH
- SEE MECH DWGS FOR EQUIPMENT MOUNTED ON FLAT ROOFS, SEE S036 FOR TYPICAL MECHANICAL UNIT PLATFORM.
- INDICATES 2x6 STUD WALL @16"OC (CRIPPLE WALL) WITH PLYWOOD TYPE C PER SCHEDULE MIN SEE DETAILS & FOR S030
- BOTTOM CONNECTION. SEE DETAIL FOR TOP CONNECTION
- Po= INDICATES AXIAL SEISMIC DRAG STRUT LOAD (SERVICE) IN LBS. CONTRACTOR SHALL DESIGN TRUSS TO RESIST THESE LOADS IN COMBINATION WITH GRAVITY LOADS IN ACCORDANCE WITH APPLICABLE BUILDING CODE AND ALL LISTED CRITERIA. WHERE REQUIRED BY ANALYSIS, PROVIDE DOUBLE OR MORE TRUSSES (PROVIDE MIN DOUBLE TRUSS), SEE UNO
- INDICATES MECHANICAL UNIT TYPE
- INDICATES MECHANICAL UNIT OPERATING WT IN LBS
- T1 INDICATES ROOF WOOD TRUSSES AER DESIGN-BUILT. REFER TO S0.38 SERIES FOR TRUSS PROFILE AND GENERAL NOTES FOR DESIGN CRITERIA, TYP.
- SOLAR PANEL, FRAMING & THEIR CONNECTIONS TO STRUCTURAL MEMBERS BY OTHERS
- INDICATES DRAG/STRUT STRAP AT DRAG BEAM / TRUSS OR BLOCKS AND EXTENT PER PLAN, AT BEAMS & AT TRUSSES CMST14 UNO
- INDICATES CHORD STRAP. PROVIDE MIN 4x4 BLOCKING UNDER STRAP FULL LENGTH OF THE STRAP CMST14 UNO

STRAP NAILING SCHEDULE

CMST14 - USE (2) ROWS OF 10d NAILS @ 3 1/2" OC UNO
CMST12 - USE (2) ROWS OF 10d NAILS @ 3 1/2" OC UNO
CMSTC16 - USE (2) ROWS OF 16d SINKERS @ 3" OC UNO
SEE FOR SPLICE DETAIL WHERE LENGTH SPECIFIED ON PLAN EXCEEDS AVAILABLE STRAP LENGTH PER MANUFACTURER

- INDICATES TOPPLATE SPLICE @ TOP OF WALL ABOVE PER USE TPS-3 AT ALL SHEAR AND BEARING WALLS UNLESS NOTED OTHERWISE
- DRAG BEAM INDICATES DRAG BEAM TO SHEAR WALL PER WITH CMST14 UNO
- DRAG BEAM INDICATES DRAG BEAM TO DRAG BEAM PER WITH (2) HDU11-SDS2.5 EA SIDE (TOTAL 4) UNO
- INDICATES DRAG BEAM TO DRAG TRUSS PER WITH (2) HD7B EA SIDE (TOTAL 4) UNO

FLAT ROOF FRAMING PLAN

SCALE: 1/8" = 1'-0"

1

WILLIAM LOYD JONES
ARCHITECT

9415 culver boulevard
culver city, california
90232

TEL 310 392 3995

S0
saiful-bouquet
structural engineer

726 S. Figueroa St.,
3rd floor
Los Angeles, CA 90017
www.saifulbouquet.com
(213) 315-2277
Project #25534

LOW ROOF FRAMING PLAN

FIRE STATION 46

MISSION VILLAGE
COUNTY OF LOS ANGELES FIRE DEPARTMENT
VALENCIA, CALIFORNIA



THE ABOVE DRAWINGS AND SPECIFICATIONS AND ALL DESIGN AND CONSTRUCTION DETAILS ARE THE PROPERTY OF WILLIAM LOYD JONES ARCHITECT AND SHALL REMAIN THE PROPERTY OF THE ARCHITECT. NO PART THEREOF SHALL BE COPIED, REPRODUCED, OR USED IN CONNECTION WITH ANY WORK OR PROJECT OTHER THAN THE SPECIFIC PROJECT FOR WHICH THEY HAVE BEEN PREPARED. ANY VIOLATION OF THESE TERMS SHALL BE CONSIDERED A BREACH OF CONTRACT AND SHALL BE SUBJECT TO LEGAL ACTION.

WRITTEN DIMENSIONS ON THESE DRAWINGS SHALL HAVE PRECEDENCE OVER SCALED DIMENSIONS. CONTRACTORS SHALL VERIFY AND BE RESPONSIBLE FOR ALL DIMENSIONS AND CONDITIONS ON THE JOB AND THE OFFICE MUST BE NOTIFIED OF ANY VARIATIONS FROM THE DIMENSIONS AND CONDITIONS SHOWN IN THESE DRAWINGS. SHOP DETAILS MUST BE SUBMITTED TO THE OFFICE FOR APPROVAL, BEFORE PROCEEDING WITH FABRICATION.

Date	Issue Date
Drawn	
Checked	
Scale	AS NOTED
Job. No.	Project Number

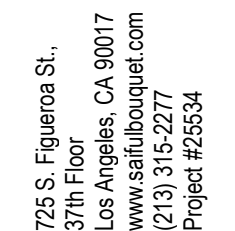
S220

10% DESIGN DEVELOPMENT SUBMITTAL

SCALE: 1/8" = 1'-0"

1

TEL 310 392 3995



PLAN

MISSION VILLAGE
COUNTY OF LOS ANGELES FIRE DEPARTMENT
VALENCIA, CALIFORNIA



WRITTEN DIMENSIONS ON THESE DRAWINGS SHALL HAVE PRECEDENCE OVER SCALED DIMENSIONS. CONTRACTORS SHALL VERIFY AND BE RESPONSIBLE FOR ALL DIMENSIONS AND CONDITIONS ON THE JOB AND THIS OFFICE MUST BE NOTIFIED OF ANY VARIATIONS FROM THE DIMENSIONS AND CONDITIONS SHOWN BY THESE DRAWINGS. SHOP DETAILS MUST BE SUBMITTED TO THIS OFFICE FOR APPROVAL BEFORE PROCEEDING WITH FABRICATION.

Date	Issue Date
Drawn	
Checked	
Scale	AS NOTED
Job. No.	Project Number

S230

ADDITIONAL

Joist-1



$$Wdl = 20 \text{ psf} \times 16/12 = 26 \text{ lb/ft}$$

$$Wll = 50 \text{ psf} \times 16/12 = 67 \text{ lb/ft}$$

2x12 @ 16" o.c

$$DCR = 0.95 < 1.0 \text{ (bending)}$$

$$L/del = 433 > 240 \text{ OK}$$

Joist-2



$$Wdl = (20 + 18) \text{ psf} \times 16/12 = 44 \text{ lb/ft}$$

$$Wll = 20 \text{ psf} \times 16/12 = 26 \text{ lb/ft}$$

(2) 2x12 @ 16" o.c

$$DCR = 0.5 < 1.0 \text{ (bending)}$$

$$L/del = 714 > 240 \text{ OK}$$

Joist-3



$$Wdl = 20 \text{ psf} \times 16/12 = 26 \text{ lb/ft}$$

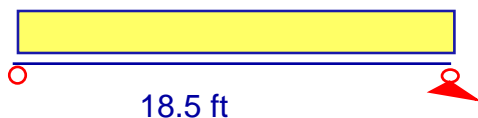
$$Wll = 50 \text{ psf} \times 16/12 = 67 \text{ lb/ft}$$

2x12 @ 24" o.c

$$DCR = 0.82 < 1.0 \text{ (bending)}$$

$$L/del = 672 > 240 \text{ OK}$$

Joist-4



$$Wdl = 20 \text{ psf} \times 24/12 = 40 \text{ lb/ft}$$

$$Wll = 50 \text{ psf} \times 24/12 = 100 \text{ lb/ft}$$

(2) 2x12 @ 24" o.c

$$DCR = 0.94 < 1.0 \text{ (bending)}$$

$$L/del = 378 > 240 \text{ OK}$$

Joist-5



$$Wdl = 18 \text{ psf} \times 24/12 = 36 \text{ lb/ft}$$

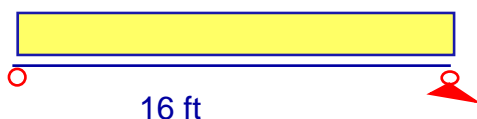
$$Wll = 20 \text{ psf} \times 24/12 = 40 \text{ lb/ft}$$

2x12 @ 24" o.c

$$DCR = 0.79 < 1.0 \text{ (bending)}$$

$$L/del = 522 > 240 \text{ OK}$$

Joist-6



$$Wdl = 18 \text{ psf} \times 4\text{ft} + 20 \text{ psf} \times 2 + 15\text{psf} \times 4\text{ft} = 172 \text{ lb/ft}$$

$$Wll = 20 \text{ psf} \times 4\text{ft} + 50 \text{ psf} \times 1\text{ft} = 130 \text{ lb/ft}$$

6x12

$$DCR = 0.79 < 1.0 \text{ (bending)}$$

$$L/del = 522 > 240 \text{ OK}$$

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build: 20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Flat Roof Joist -1 Typical Wood Joist at Flat Roof L= 16 ft Max @ 2x12 @ 16

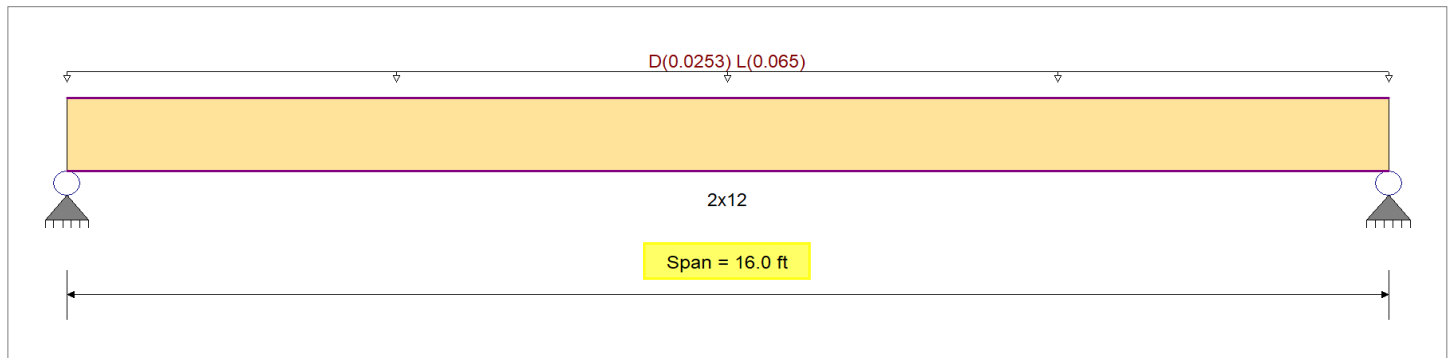
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design	Fb +	1,000.0 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16	Fb -	1,000.0 psi	Ebend- xx	1,700.0ksi
	Fc - Prll	1,500.0 psi	Eminbend - xx	620.0ksi
Wood Species : Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade : No.1	Fv	180.0 psi		
	Ft	675.0 psi	Density	31.210pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.02530, L = 0.0650, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.953	1	Maximum Shear Stress Ratio	=	0.315	1
Section used for this span		2x12		Section used for this span		2x12	
fb: Actual	=	1,095.91 psi		fv: Actual	=	56.71 psi	
F'b	=	1,150.00 psi		F'v	=	180.00 psi	
Load Combination		+D+L		Load Combination		+D+L	
Location of maximum on span	=	8.000 ft		Location of maximum on span	=	15.066 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.319 in	Ratio =	602 >= 360	Span: 1 : L Only			
Max Upward Transient Deflection	0 in	Ratio =	0 < 360	n/a			
Max Downward Total Deflection	0.443 in	Ratio =	433 >= 240	Span: 1 : +D+L			
Max Upward Total Deflection	0 in	Ratio =	0 < 240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values				
		Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only																0.0	0.00	0.0	0.0
	Length = 16.0 ft	1	0.297	0.098	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.15	0.81	307.0	1,035.0	0.18	15.9	162.0	
+D+L						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0	
	Length = 16.0 ft	1	0.953	0.315	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.15	2.89	1,095.9	1,150.0	0.64	56.7	180.0	
+D+0.750L						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0	
	Length = 16.0 ft	1	0.625	0.207	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.15	2.37	898.7	1,437.5	0.52	46.5	225.0	
+0.60D						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0	
	Length = 16.0 ft	1	0.100	0.033	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.15	0.49	184.2	1,840.0	0.11	9.5	288.0	

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Flat Roof Joist -1 Typical Wood Joist at Flat Roof L= 16 ft Max @ 2x12 @ 16

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.4427	8.058		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.722	0.722
Max Upward from Load Combinations	0.722	0.722
Max Upward from Load Cases	0.520	0.520
D Only	0.202	0.202
+D+L	0.722	0.722
+D+0.750L	0.592	0.592
+0.60D	0.121	0.121
L Only	0.520	0.520

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Joist-2 : Typical Wood Joist at sloped Roof L= 18 ft Max @ 2x12 @ 16

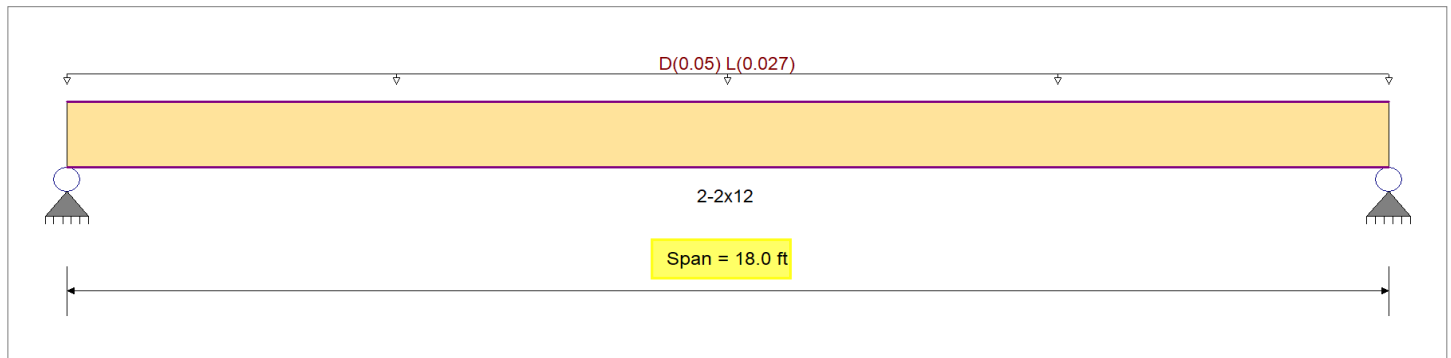
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design	Fb +	1,000.0 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16	Fb -	1,000.0 psi	Ebend- xx	1,700.0ksi
	Fc - Prll	1,500.0 psi	Eminbend - xx	620.0ksi
Wood Species : Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade : No.1	Fv	180.0 psi		
	Ft	675.0 psi	Density	31.210pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.050, L = 0.0270 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.514 : 1	Maximum Shear Stress Ratio	=	0.154 : 1
Section used for this span		2-2x12	Section used for this span		2-2x12
fb: Actual	=	591.36 psi	fv: Actual	=	27.65 psi
F'b	=	1,150.00 psi	F'v	=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.000 ft	Location of maximum on span	=	17.080 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection	0.106 in	Ratio = 2037 >= 360	Span: 1 : L Only		
Max Upward Transient Deflection	0 in	Ratio = 0 < 360	n/a		
Max Downward Total Deflection	0.302 in	Ratio = 714 >= 240	Span: 1 : +D+L		
Max Upward Total Deflection	0 in	Ratio = 0 < 240	n/a		

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only															0.0	0.00	0.0	0.0
Length = 18.0 ft	1		0.371	0.111	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.15	2.03	384.0	1,035.0	0.40	18.0	162.0
+D+L						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 18.0 ft	1		0.514	0.154	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.15	3.12	591.4	1,150.0	0.62	27.7	180.0
+D+0.750L						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 18.0 ft	1		0.375	0.112	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.15	2.85	539.5	1,437.5	0.57	25.2	225.0
+0.60D						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 18.0 ft	1		0.125	0.037	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.15	1.22	230.4	1,840.0	0.24	10.8	288.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Joist-2 : Typical Wood Joist at sloped Roof L= 18 ft Max @ 2x12 @ 16

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.3023	9.066		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.693	0.693
Max Upward from Load Combinations	0.693	0.693
Max Upward from Load Cases	0.450	0.450
D Only	0.450	0.450
+D+L	0.693	0.693
+D+0.750L	0.632	0.632
+0.60D	0.270	0.270
L Only	0.243	0.243

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Joist -3: Typical Wood Joist at Flat Roof L= 12 ft Max 2x12 @ 24

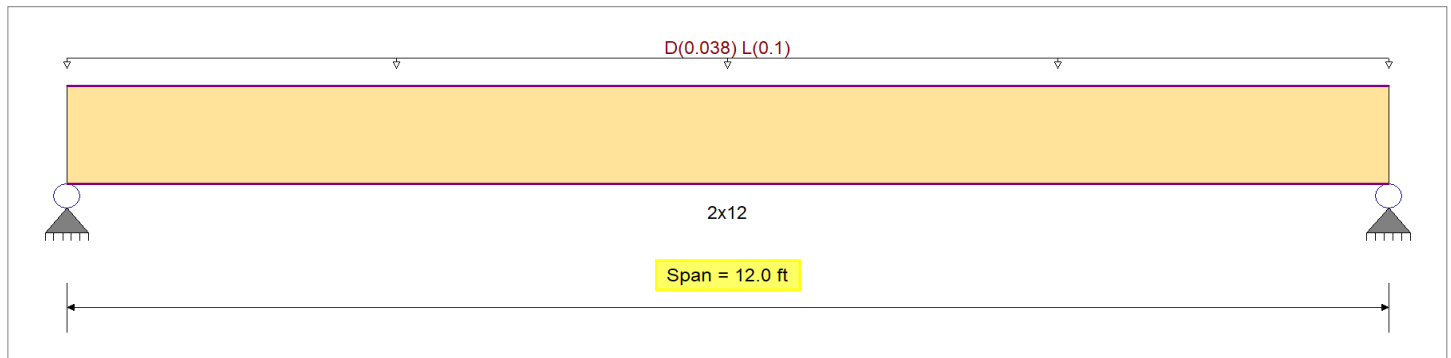
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design	Fb +	1,000.0 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16	Fb -	1,000.0 psi	Ebend- xx	1,700.0ksi
	Fc - Prll	1,500.0 psi	Eminbend - xx	620.0ksi
Wood Species : Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade : No.1	Fv	180.0 psi		
	Ft	675.0 psi	Density	31.210pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.0380, L = 0.10 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.819 : 1	Maximum Shear Stress Ratio	=	0.346 : 1
Section used for this span		2x12	Section used for this span		2x12
fb: Actual	=	942.08psi	fv: Actual	=	62.32 psi
F'b	=	1,150.00psi	F'v	=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	6.000ft	Location of maximum on span	=	0.000ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection	0.155 in	Ratio = 928 >=360	Span: 1 : L Only		
Max Upward Transient Deflection	0 in	Ratio = 0 <360	n/a		
Max Downward Total Deflection	0.214 in	Ratio = 672 >=240	Span: 1 : +D+L		
Max Upward Total Deflection	0 in	Ratio = 0 <240	n/a		

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values				
		Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only																0.0	0.00	0.0	0.0
	Length = 12.0 ft	1	0.251	0.106	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.15	0.68	259.4	1,035.0	0.19	17.2	162.0	
+D+L						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0	
	Length = 12.0 ft	1	0.819	0.346	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.15	2.48	942.1	1,150.0	0.70	62.3	180.0	
+D+0.750L						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0	
	Length = 12.0 ft	1	0.537	0.227	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.15	2.03	771.4	1,437.5	0.57	51.0	225.0	
+0.60D						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0	
	Length = 12.0 ft	1	0.085	0.036	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.15	0.41	155.6	1,840.0	0.12	10.3	288.0	

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Joist -3: Typical Wood Joist at Flat Roof L= 12 ft Max 2x12 @ 24

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.2140	6.044		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.828	0.828
Max Upward from Load Combinations	0.828	0.828
Max Upward from Load Cases	0.600	0.600
D Only	0.228	0.228
+D+L	0.828	0.828
+D+0.750L	0.678	0.678
+0.60D	0.137	0.137
L Only	0.600	0.600

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Joist 4: Typical Wood Joist at Flat Roof L= 19 ft Max (2)2x12 @24

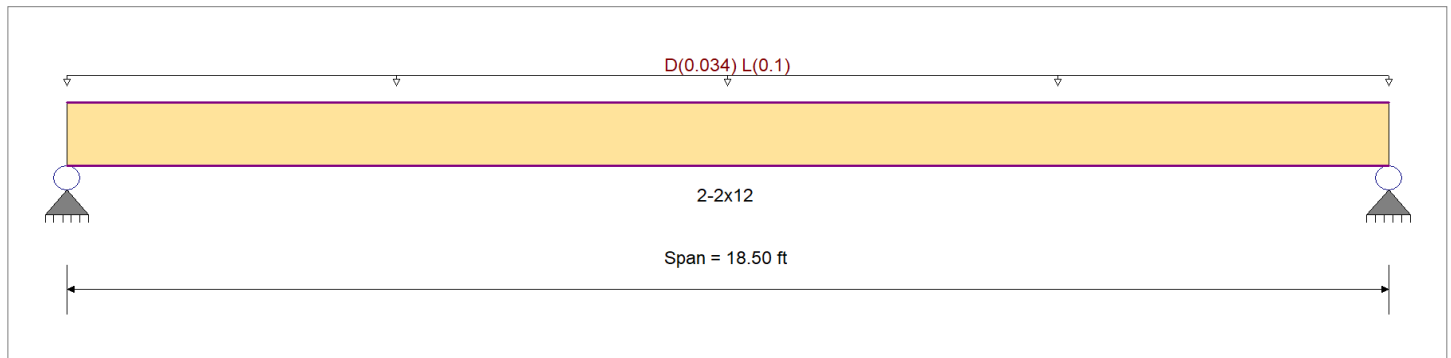
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design	Fb +	1,000.0 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16	Fb -	1,000.0 psi	Ebend- xx	1,700.0ksi
	Fc - Prll	1,500.0 psi	Eminbend - xx	620.0ksi
Wood Species : Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade : No.1	Fv	180.0 psi		
	Ft	675.0 psi	Density	31.210pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.0340, L = 0.10 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.945	1	Maximum Shear Stress Ratio	=	0.277	1
Section used for this span		2-2x12		Section used for this span		2-2x12	
fb: Actual	=	1,087.09psi		fv: Actual	=	49.86 psi	
F'b	=	1,150.00psi		F'v	=	180.00 psi	
Load Combination		+D+L		Load Combination		+D+L	
Location of maximum on span	=	9.250ft		Location of maximum on span	=	17.622 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.438 in	Ratio =	506 >= 360	Span: 1 : L Only			
Max Upward Transient Deflection	0 in	Ratio =	0 < 360	n/a			
Max Downward Total Deflection	0.587 in	Ratio =	378 >= 240	Span: 1 : +D+L			
Max Upward Total Deflection	0 in	Ratio =	0 < 240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 18.50 ft	1	0.267	0.078	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.15	1.45	275.8	1,035.0	0.28	12.7	162.0
+D+L					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 18.50 ft	1	0.945	0.277	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.15	5.73	1,087.1	1,150.0	1.12	49.9	180.0
+D+0.750L					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 18.50 ft	1	0.615	0.180	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.15	4.66	884.3	1,437.5	0.91	40.6	225.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 18.50 ft	1	0.090	0.026	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.15	0.87	165.5	1,840.0	0.17	7.6	288.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Joist 4: Typical Wood Joist at Flat Roof L= 19 ft Max (2)2x12 @24

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.5870	9.318		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.240	1.240
Max Upward from Load Combinations	1.240	1.240
Max Upward from Load Cases	0.925	0.925
D Only	0.315	0.315
+D+L	1.240	1.240
+D+0.750L	1.008	1.008
+0.60D	0.189	0.189
L Only	0.925	0.925

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Joist-5 : Typical Wood Joist at sloped Roof L= 15 ft Max @ 2x12 @ 24

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

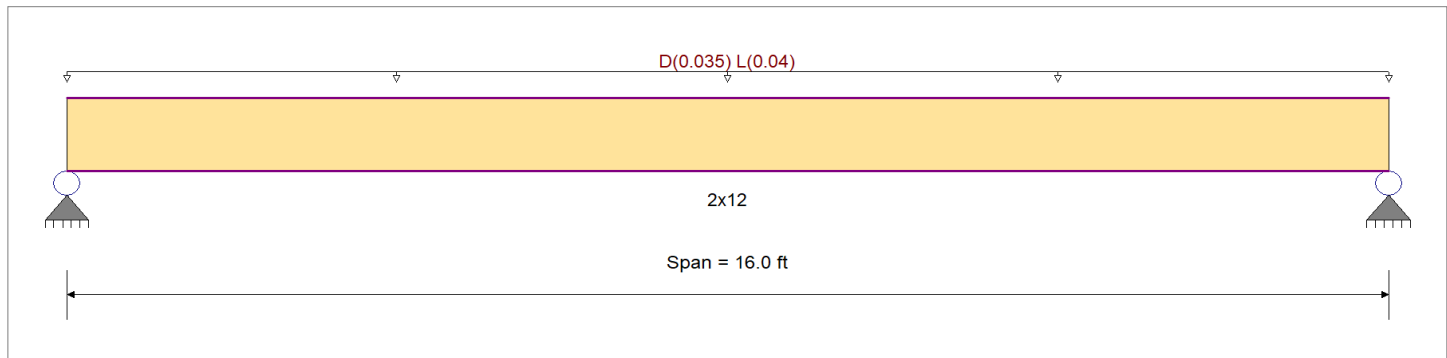
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
 Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 1,000.0 psi
 Fb - 1,000.0 psi
 Fc - Prll 1,500.0 psi
 Fc - Perp 625.0 psi
 Fv 180.0 psi
 Ft 675.0 psi
 E : Modulus of Elasticity
 Ebend- xx 1,700.0 ksi
 Eminbend - xx 620.0 ksi
 Density 31.210pcf
 Repetitive Member Stress Increase



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.0350, L = 0.040, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.791	1	Section used for this span	=	0.262	1
fb: Actual	=	910.22	psi	fv: Actual	=	47.10	psi
F'b	=	1,150.00	psi	F'v	=	180.00	psi
Load Combination	=	+D+L		Load Combination	=	+D+L	
Location of maximum on span	=	8.000ft		Location of maximum on span	=	15.066ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.196 in	Ratio =	979 >=360	Span: 1 : L Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.368 in	Ratio =	522 >=240	Span: 1 : +D+L			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only															0.0	0.00	0.0	0.0
Length = 16.0 ft	1		0.410	0.136	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.15	1.12	424.8	1,035.0	0.25	22.0	162.0
+D+L						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 16.0 ft	1		0.791	0.262	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.15	2.40	910.2	1,150.0	0.53	47.1	180.0
+D+0.750L						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 16.0 ft	1		0.549	0.181	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.15	2.08	788.9	1,437.5	0.46	40.8	225.0
+0.60D						1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 16.0 ft	1		0.139	0.046	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.15	0.67	254.9	1,840.0	0.15	13.2	288.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Joist-5 : Typical Wood Joist at sloped Roof L= 15 ft Max @ 2x12 @ 24

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.3677	8.058		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.600	0.600
Max Upward from Load Combinations	0.600	0.600
Max Upward from Load Cases	0.320	0.320
D Only	0.280	0.280
+D+L	0.600	0.600
+D+0.750L	0.520	0.520
+0.60D	0.168	0.168
L Only	0.320	0.320

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Joist 7: Typical Wood 6x12 at Flat Roof L= 19 ft under crippe wall

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

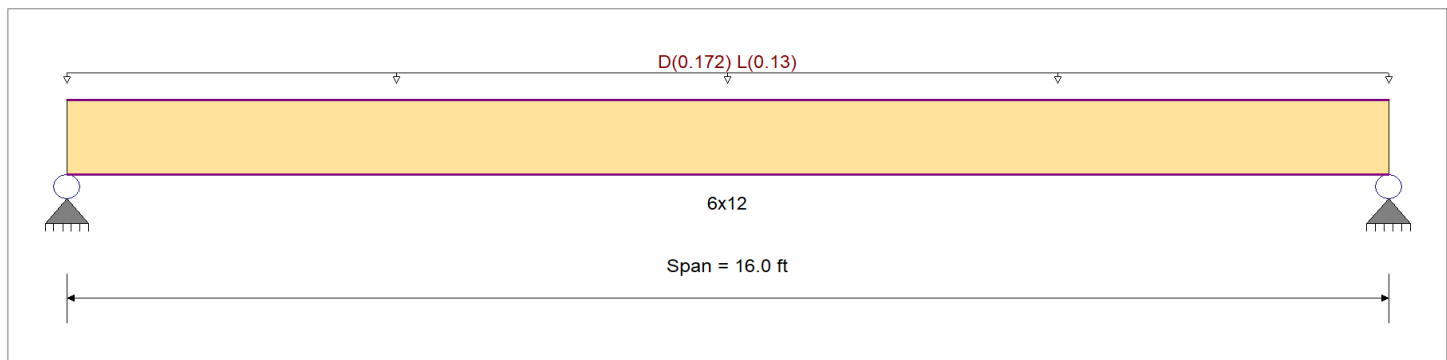
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
 Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 1,350.0 psi E : Modulus of Elasticity
 Fb - 1,350.0 psi Ebend- xx 1,600.0ksi
 Fc - Prll 925.0 psi Eminbend - xx 580.0ksi
 Fc - Perp 625.0 psi
 Fv 170.0 psi
 Ft 675.0 psi Density 31.210pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.1720, L = 0.130 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.709	1	Section used for this span	=	0.298	1
fb: Actual	=	956.60psi		fv: Actual	=	50.60 psi	
F'b	=	1,350.00psi		F'v	=	170.00 psi	
Load Combination		+D+L		Load Combination		+D+L	
Location of maximum on span	=	8.000ft		Location of maximum on span	=	15.066ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.173 in	Ratio =	1110 >=360	Span: 1 : L Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.402 in	Ratio =	478 >=240	Span: 1 : +D+L			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.448	0.188	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.50	544.8	1,215.0	1.22	28.8	153.0
+D+L					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.709	0.298	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	9.66	956.6	1,350.0	2.13	50.6	170.0
+D+0.750L					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.506	0.213	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.62	853.7	1,687.5	1.90	45.2	212.5
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.151	0.064	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.30	326.9	2,160.0	0.73	17.3	272.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Joist 7: Typical Wood 6x12 at Flat Roof L= 19 ft under cripel wall

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.4016	8.058		0.0000	0.000

Vertical Reactions

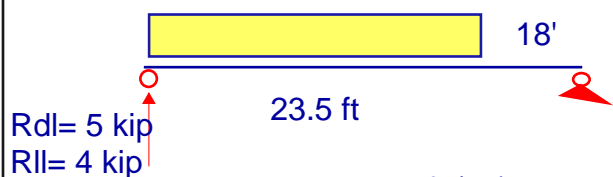
Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.416	2.416
Max Upward from Load Combinations	2.416	2.416
Max Upward from Load Cases	1.376	1.376
D Only	1.376	1.376
+D+L	2.416	2.416
+D+0.750L	2.156	2.156
+0.60D	0.826	0.826
L Only	1.040	1.040

1.3 WOOD BEAM DESIGN

Beam-1



$$Wdl = 24 \text{ psf} \cdot 18 \text{ ft}/2 + 20 \text{ psf} \cdot 7/2 + 18 \text{ psf} \cdot 10 \text{ ft}/2 + 15 \text{ psf} \cdot 6 \text{ ft} = 466 \text{ lb/ft}$$

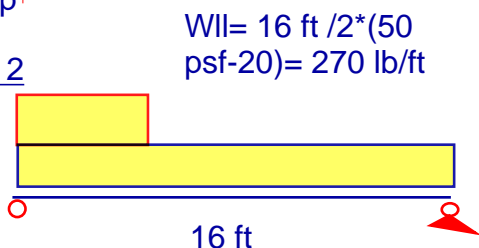
$$Wll = 20 \text{ psf} \cdot (18 \text{ ft}/2 + 10 \text{ ft}/2) = 380 \text{ lb/ft}$$

5 1/4x 18 PSL

DCR= 0.6 < 1.0 (bending)

L/del= 310 > 240 OK

Beam 2

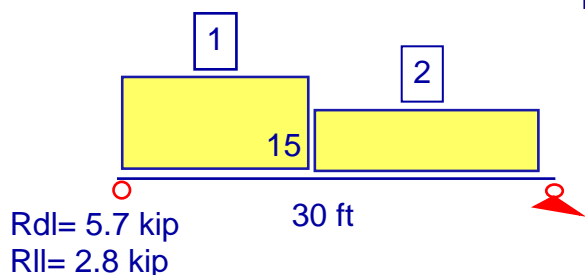


$$Wll = 16 \text{ ft} / 2 \cdot (50 \text{ psf} - 20) = 270 \text{ lb/ft}$$

$$Wdl = 18 \text{ psf} \cdot 18 \text{ ft}/2 + 20 \text{ psf} \cdot 16/2 + 18 \text{ psf} \cdot 16 \text{ ft}/2 = 466 \text{ lb/ft}$$

$$Wll = 20 \text{ psf} \cdot (18 \text{ ft}/2 + 16 \text{ ft}/2) = 340 \text{ lb/ft}$$

Beam-3



1

$$Wdl = 20 \text{ psf} \cdot 6 \text{ ft}/2 + 15 \text{ psf} \cdot 8 \text{ ft} = 180 \text{ lb/ft}$$

$$Wll = 50 \text{ psf} \cdot 6 \text{ ft}/2 = 150 \text{ lb/ft}$$

2

$$Wdl = 20 \text{ psf} \cdot 16 \text{ ft}/2 + 18 \text{ psf} \cdot 16/2 + 15 \text{ psf} \cdot 8 \text{ ft} = 424 \text{ lb/ft}$$

$$Wll = 20 \text{ psf} \cdot 16 \text{ ft}/2 = 160 \text{ lb/ft}$$

PSL 5.25x22

DCR= 0.47 < 1.0 (bending)

L/del= 375 > 240 OK

Beam-6



$$Wdl = 24 \text{ psf} \cdot 12 \text{ ft}/2 = 144 \text{ lb/ft}$$

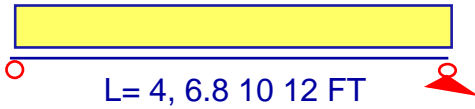
$$Wll = 20 \text{ psf} \cdot 12/2 = 120 \text{ lb/ft}$$

5 1/4x 9 1/2 PSL

DCR= 0.28 < 1.0 (bending)

L/del= 540 > 240 OK

Header-1

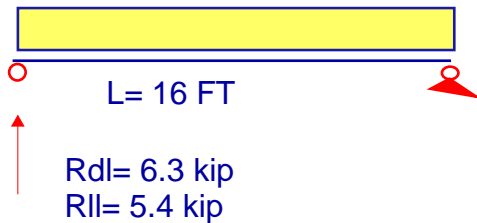


HEADER SCHEDULE		
MAXIMUM OPENING WIDTH	MIN NOMINAL HEADER "D"	
	NON-BEARING	BEARING
4'-0"	4"	6"
6'-0"	6"	8"
8'-0"	8"	10"
10'-0"	10"	12"
12'-0"	12"	14"
14'-0"	AS SHOWN ON PLAN	AS SHOWN ON PLAN

$$Wdl = 15 \text{ psf} \cdot 12 \text{ ft} + 21 \text{ psf} \cdot 37 \text{ ft}/2 + 20 \text{ psf} \cdot 16 \text{ ft}/2 = 728 \text{ lb/ft}$$

$$Wll = 16 \text{ psf} \cdot 37 \text{ ft}/2 + 50 \text{ psf} \cdot 14 \text{ ft}/2 = 646 \text{ lb/ft}$$

Header-2



$$Wdl = 15 \text{ psf} \cdot 12 \text{ ft} + 21 \text{ psf} \cdot 37 \text{ ft}/2 + 20 \text{ psf} \cdot 16 \text{ ft}/2 = 728 \text{ lb/ft}$$

$$Wll = 16 \text{ psf} \cdot 37 \text{ ft}/2 + 50 \text{ psf} \cdot 14 \text{ ft}/2 = 646 \text{ lb/ft}$$

5.25 X16 PSL OK

PSL 5.25x16

$$DCR = 0.8 < 1.0 \text{ (bending)}$$

$$L/del = 250 > 240 \text{ OK}$$

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: BEAM-1

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

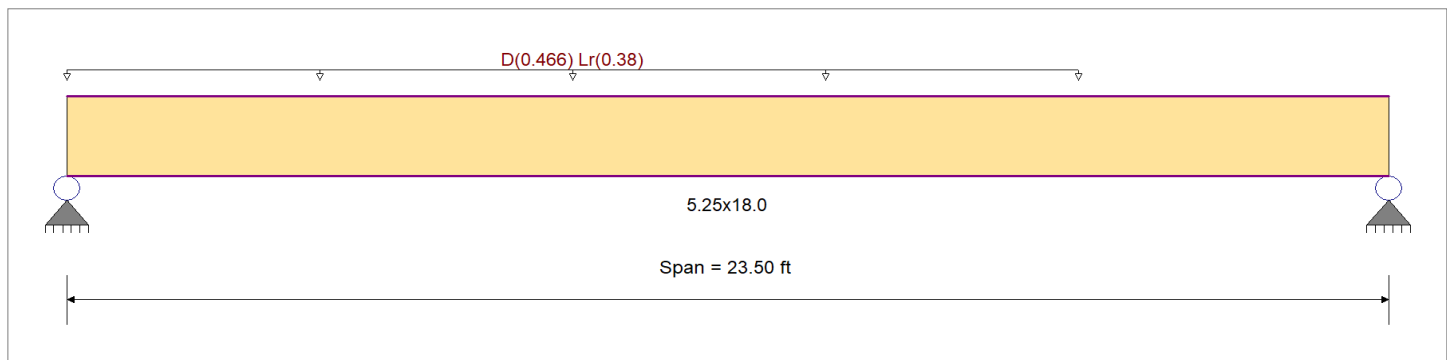
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : iLevel Truss Joist
 Wood Grade : Parallam PSL 2.2E

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,900.0 psi	E : Modulus of Elasticity
Fb -	2,900.0 psi	Ebend- xx
Fc - Prll	2,900.0 psi	Eminbend - xx
Fc - Perp	750.0 psi	
Fv	290.0 psi	
Ft	2,025.0 psi	Density
		45.070pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Load for Span Number 1

Uniform Load : D = 0.4660, Lr = 0.380 k/ft, Extent = 0.0 --> 18.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.637	1	Section used for this span	=	0.357	1
fb: Actual	=	2,208.55	psi	fv: Actual	=	129.56	psi
F'b	=	3,465.47	psi	F'v	=	362.50	psi
Load Combination	=	+D+Lr		Load Combination	=	+D+Lr	
Location of maximum on span	=	11.064	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	0.408 in	Ratio =	690 >= 360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 < 360	n/a			
Max Downward Total Deflection	0.909 in	Ratio =	310 >= 240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 < 240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0			
Length = 23.50 ft	1	0.488	0.273	0.90	1.00	1.00	1.00	0.956	1.00	1.00	1.00	28.74	1,216.5	2,495.1	4.50	71.4	261.0
+D+Lr														0.0			
Length = 23.50 ft	1	0.637	0.357	1.25	1.00	1.00	1.00	0.956	1.00	1.00	1.00	52.18	2,208.6	3,465.5	8.16	129.6	362.5
+D+0.750Lr														0.0			
Length = 23.50 ft	1	0.566	0.317	1.25	1.00	1.00	1.00	0.956	1.00	1.00	1.00	46.32	1,960.5	3,465.5	7.25	115.0	362.5
+0.60D														0.0			
Length = 23.50 ft	1	0.165	0.092	1.60	1.00	1.00	1.00	0.956	1.00	1.00	1.00	17.24	729.9	4,435.8	2.70	42.8	464.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: BEAM-1

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.9090	11.578		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	9.396	5.832
Max Upward from Load Combinations	9.396	5.832
Max Upward from Load Cases	5.176	3.212
D Only	5.176	3.212
+D+Lr	9.396	5.832
+D+0.750Lr	8.341	5.177
+0.60D	3.105	1.927
Lr Only	4.220	2.620

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Beam 2

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

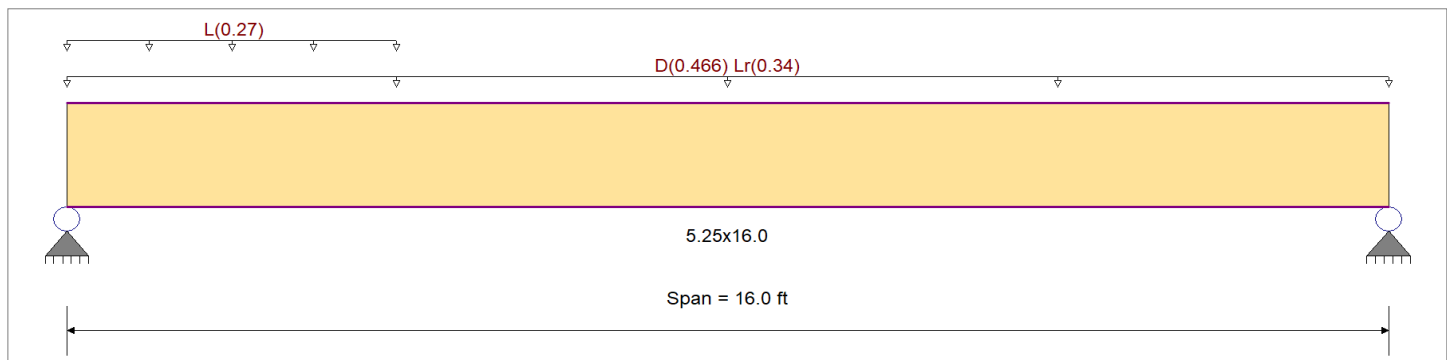
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : iLevel Truss Joist
 Wood Grade : Parallam PSL 2.0E

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 2900 psi E : Modulus of Elasticity
 Fb - 2900 psi Ebend- xx 2000ksi
 Fc - Prll 2900 psi Eminbend - xx 1016.535ksi
 Fc - Perp 750 psi
 Fv 290 psi
 Ft 2025 psi Density 45.07pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.4660, Lr = 0.340, Tributary Width = 1.0 ft

Uniform Load : L = 0.270 k/ft, Extent = 0.0 --> 4.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	5.25x16.0		Section used for this span	=	5.25x16.0	
fb: Actual	=	1,381.71 psi		fv: Actual	=	96.65 psi	
F'b	=	3,511.07 psi		F'v	=	362.50 psi	
Load Combination	=	+D+Lr		Load Combination	=	+D+Lr	
Location of maximum on span	=	8.000ft		Location of maximum on span	=	14.715 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.141 in	Ratio =	1364 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.334 in	Ratio =	575 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.316	0.214	0.90	1.00	1.00	1.00	0.969	1.00	1.00	1.00	14.91	798.9	2,528.0	3.13	55.9	261.0
+D+L														0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.305	0.230	1.00	1.00	1.00	1.00	0.969	1.00	1.00	1.00	16.01	857.8	2,808.9	3.73	66.6	290.0
+D+Lr														0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.394	0.267	1.25	1.00	1.00	1.00	0.969	1.00	1.00	1.00	25.79	1,381.7	3,511.1	5.41	96.7	362.5
+D+0.750Lr+0.750L														0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.364	0.261	1.25	1.00	1.00	1.00	0.969	1.00	1.00	1.00	23.89	1,279.8	3,511.1	5.29	94.5	362.5
+D+0.750L														0.0	0.00	0.0	0.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Beam 2

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 16.0 ft	1		0.261	0.192	1.15	1.00	1.00	1.00	0.969	1.00	1.00	1.00	15.73	842.8	3,230.2	3.58	63.9	333.5
+0.60D						1.00	1.00	1.00	0.969	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1		0.107	0.072	1.60	1.00	1.00	1.00	0.969	1.00	1.00	1.00	8.95	479.3	4,494.2	1.88	33.5	464.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.3335	8.058		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	6.477	6.448
Max Upward from Load Combinations	6.477	6.448
Max Upward from Load Cases	3.728	3.728
D Only	3.728	3.728
+D+L	4.673	3.863
+D+Lr	6.448	6.448
+D+0.750Lr+0.750L	6.477	5.869
+D+0.750L	4.437	3.829
+0.60D	2.237	2.237
Lr Only	2.720	2.720
L Only	0.945	0.135

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Beam -3

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

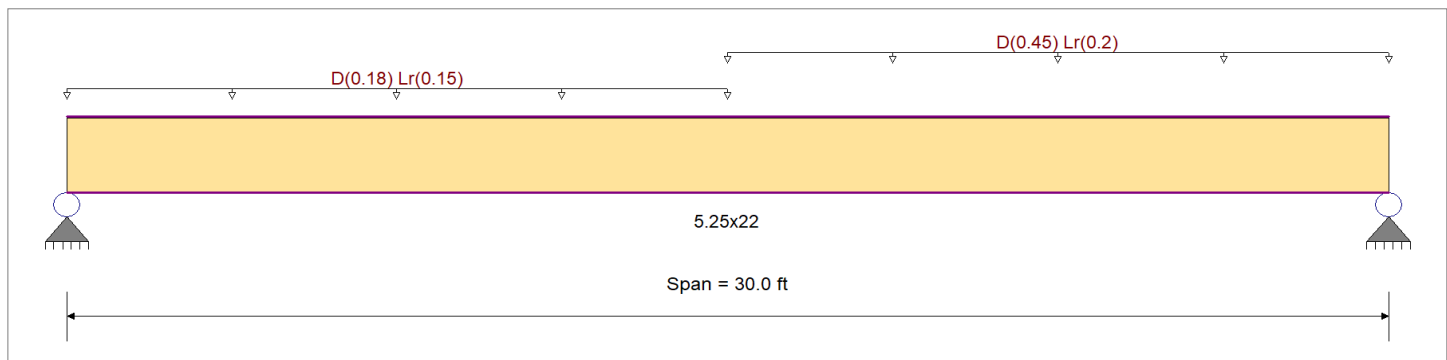
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : iLevel Truss Joist
 Wood Grade : Parallam PSL 2.0E

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 2900 psi E : Modulus of Elasticity
 Fb - 2900 psi Ebend- xx 2000ksi
 Fc - Prll 2900 psi Eminbend - xx 1016.535ksi
 Fc - Perp 750 psi
 Fv 290 psi
 Ft 2025 psi Density 45.07pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Load for Span Number 1

Uniform Load : D = 0.180, Lr = 0.150 k/ft, Extent = 0.0 --> 15.0 ft, Tributary Width = 1.0 ft

Uniform Load : D = 0.450, Lr = 0.20 k/ft, Extent = 15.0 --> 30.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	5.25x22		Section used for this span	=	5.25x22	
fb: Actual	=	1,593.37 psi		fv: Actual	=	96.25 psi	
F'b	=	3,389.13 psi		F'v	=	362.50 psi	
Load Combination	=	+D+Lr		Load Combination	=	+D+Lr	
Location of maximum on span	=	16.861 ft		Location of maximum on span	=	28.248 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.344 in	Ratio =	1045 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.965 in	Ratio =	373 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0			
Length = 30.0 ft	1	0.425	0.246	0.90	1.00	1.00	1.00	0.935	1.00	1.00	1.00	36.58	1,036.4	2,440.2	4.95	64.3	261.0
+D+Lr														0.0			
Length = 30.0 ft	1	0.470	0.266	1.25	1.00	1.00	1.00	0.935	1.00	1.00	1.00	56.23	1,593.4	3,389.1	7.41	96.3	362.5
+D+0.750Lr														0.0			
Length = 30.0 ft	1	0.429	0.243	1.25	1.00	1.00	1.00	0.935	1.00	1.00	1.00	51.31	1,453.9	3,389.1	6.80	88.3	362.5
+0.60D														0.0			
Length = 30.0 ft	1	0.143	0.083	1.60	1.00	1.00	1.00	0.935	1.00	1.00	1.00	21.95	621.8	4,338.1	2.97	38.6	464.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Beam -3

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.9647	15.438		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	6.150	8.550
Max Upward from Load Combinations	6.150	8.550
Max Upward from Load Cases	3.713	5.738
D Only	3.713	5.738
+D+Lr	6.150	8.550
+D+0.750Lr	5.541	7.847
+0.60D	2.228	3.443
Lr Only	2.438	2.813

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: BEAM-6

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

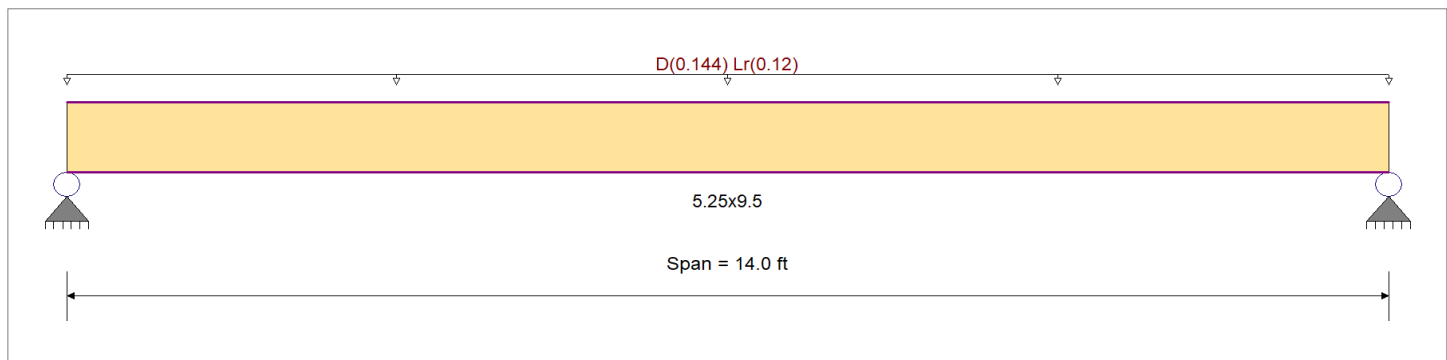
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : iLevel Truss Joist
 Wood Grade : Parallam PSL 2.2E

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 2,900.0 psi
 Fb - 2,900.0 psi
 Fc - Prll 2,900.0 psi
 Fc - Perp 750.0 psi
 Fv 290.0 psi
 Ft 2,025.0 psi
 E : Modulus of Elasticity
 Ebend- xx 2,200.0 ksi
 Eminbend - xx 1,118.19 ksi
 Density 45.070 pcf



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.1440, Lr = 0.120, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.280	1	Section used for this span	=	0.145	1
fb: Actual	=	1,040.99	psi	fv: Actual	=	52.42	psi
F'b	=	3,720.23	psi	F'v	=	362.50	psi
Load Combination	=	+D+Lr		Load Combination	=	+D+Lr	
Location of maximum on span	=	7.000ft		Location of maximum on span	=	13.234 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.126 in	Ratio =	1328 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.295 in	Ratio =	570 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 14.0 ft	1	0.222	0.115	0.90	1.00	1.00	1.00	1.026	1.00	1.00	1.00	3.91	594.2	2,678.6	0.99	29.9	261.0
+D+Lr					1.00	1.00	1.00	1.026	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 14.0 ft	1	0.280	0.145	1.25	1.00	1.00	1.00	1.026	1.00	1.00	1.00	6.85	1,041.0	3,720.2	1.74	52.4	362.5
+D+0.750Lr					1.00	1.00	1.00	1.026	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 14.0 ft	1	0.250	0.129	1.25	1.00	1.00	1.00	1.026	1.00	1.00	1.00	6.12	929.3	3,720.2	1.56	46.8	362.5
+0.60D					1.00	1.00	1.00	1.026	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 14.0 ft	1	0.075	0.039	1.60	1.00	1.00	1.00	1.026	1.00	1.00	1.00	2.35	356.5	4,761.9	0.60	18.0	464.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: BEAM-6

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.2946	7.051		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.957	1.957
Max Upward from Load Combinations	1.957	1.957
Max Upward from Load Cases	1.117	1.117
D Only	1.117	1.117
+D+Lr	1.957	1.957
+D+0.750Lr	1.747	1.747
+0.60D	0.670	0.670
Lr Only	0.840	0.840

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-2 L=18 FT

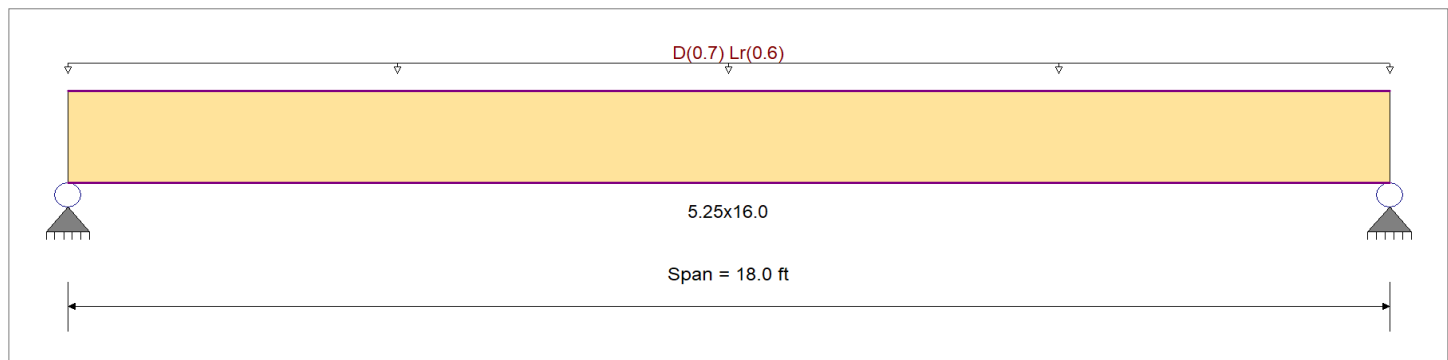
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design	Fb +	2,900.0 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
	Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species : iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade : Parallam PSL 2.0E	Fv	290.0 psi		
	Ft	2,025.0 psi	Density	45.070 pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.70, Lr = 0.60 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.803 : 1	Maximum Shear Stress Ratio	=	0.492 : 1
Section used for this span	=	5.25x16.0	Section used for this span	=	5.25x16.0
fb: Actual	=	2,820.54 psi	fv: Actual	=	178.43 psi
F'b	=	3,511.07 psi	F'v	=	362.50 psi
Load Combination	=	+D+Lr	Load Combination	=	+D+Lr
Location of maximum on span	=	9.000 ft	Location of maximum on span	=	16.686 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection	0.398 in	Ratio = 543 >= 360	Span: 1 : Lr Only		
Max Upward Transient Deflection	0 in	Ratio = 0 < 360	n/a		
Max Downward Total Deflection	0.862 in	Ratio = 250 >= 240	Span: 1 : +D+Lr		
Max Upward Total Deflection	0 in	Ratio = 0 < 240	n/a		

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only															0.0		0.00	0.0
Length = 18.0 ft	1		0.601	0.368	0.90	1.00	1.00	1.00	0.969	1.00	1.00	1.00	28.35	1,518.8	2,528.0	5.38	96.1	261.0
+D+Lr															0.0		0.00	0.0
Length = 18.0 ft	1		0.803	0.492	1.25	1.00	1.00	1.00	0.969	1.00	1.00	1.00	52.65	2,820.5	3,511.1	9.99	178.4	362.5
+D+0.750Lr															0.0		0.00	0.0
Length = 18.0 ft	1		0.711	0.435	1.25	1.00	1.00	1.00	0.969	1.00	1.00	1.00	46.58	2,495.1	3,511.1	8.84	157.8	362.5
+0.60D															0.0		0.00	0.0
Length = 18.0 ft	1		0.203	0.124	1.60	1.00	1.00	1.00	0.969	1.00	1.00	1.00	17.01	911.3	4,494.2	3.23	57.6	464.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-2 L=18 FT

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.8617	9.066		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	11.700	11.700
Max Upward from Load Combinations	11.700	11.700
Max Upward from Load Cases	6.300	6.300
D Only	6.300	6.300
+D+Lr	11.700	11.700
+D+0.750Lr	10.350	10.350
+0.60D	3.780	3.780
Lr Only	5.400	5.400

BEAM -4
Ring Beam @ lines 3 & B

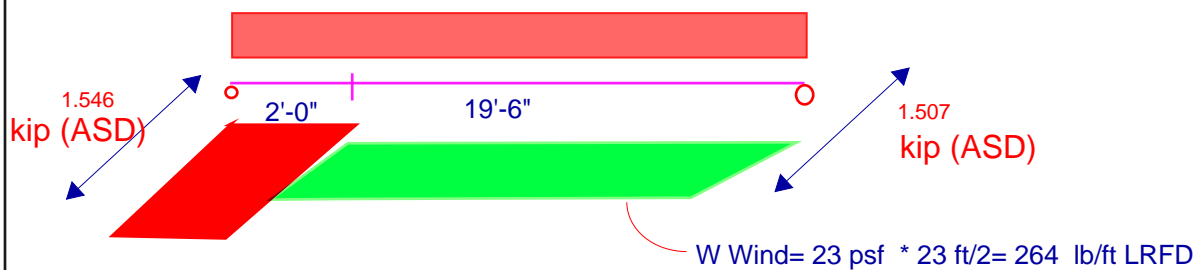
$$W \text{ Roof DL} = 24 \text{ psf} \times 38 \text{ ft} / 2 = 456 \text{ lb/ft}$$

$$W \text{ wall DL} = 15 \text{ psf} \times 10 \text{ ft} = 150 \text{ lb/ft}$$

$$W \text{ Roof LL} = 20 \text{ psf} \times 38 \text{ ft} / 2 = 380 \text{ lb/ft}$$

$$W \text{ DL} = 456 \text{ lb/ft} + 150 \text{ lb/ft} = 606 \text{ lb/ft}$$

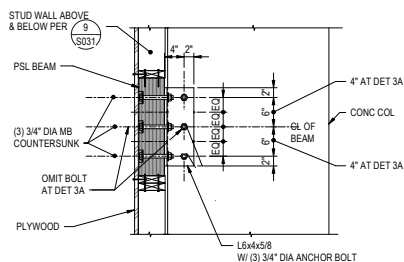
$$W \text{ Roof LL} = 380 \text{ lb/ft}$$



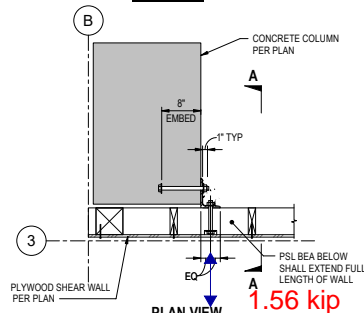
$$W \text{ Wind} = 26.6 \text{ psf} \times 23 \text{ ft} / 2 = 306 \text{ lb/ft LRFD}$$

$$5 \text{ 1/4} \times 20 \text{ PSL } 19' \times 12' / (0.42 \times 1.4'') = 384 > 360 \text{ OK}$$

Connection design



DETAIL A-A



WALL BEAM TO CONCRETE COLUMN CONNECTION

SCALE: 1" = 1'-0"

2

Angle Design:

$$M_{max} = 1.6 \text{ kip} \times 2' = 3.2 \text{ kip}'$$

$$S_{req} = 3.2 \text{ kip}' / 0.6 \times 36 \text{ ksi} = 0.15 \text{ in}^3$$

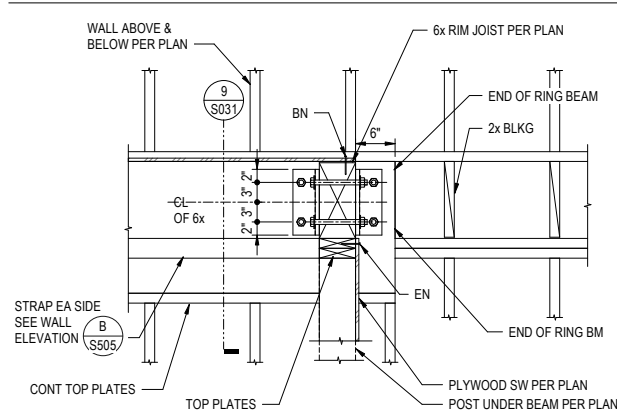
try angle L 8x4x 5/8

$$S_p = 14'' \times (5/8)^2 / 6 = 0.9 \text{ in}^3 > S_{req} = 0.15 \text{ in}^3$$

Check bearing on wood beam:

$$A \text{ bearing} = (1.25^2 \times 3.14) / 4 = 1.23 \text{ in}^2 / \text{bolt}$$

$$P \text{ bearing} = 750 \text{ psi} \times 1.23 \text{ in}^2 \times 3 \times 1.6 = 4.4 \text{ kip} > F = 1.56 \text{ kip OK}$$



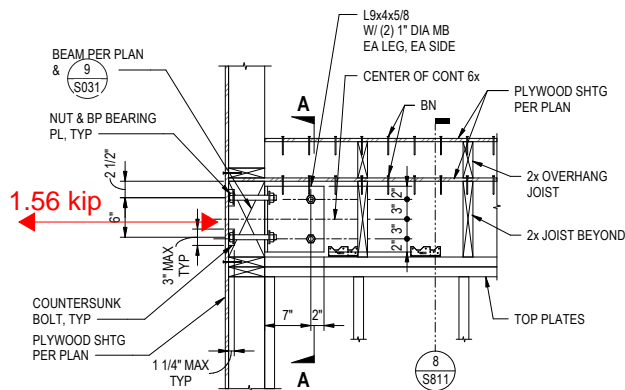
Connection Design:

By inspection from detail 2/S811 the angles, number of bolts at this detail 4/S811 OK.

Checking shear in 6x12

3/4" diameter MB

$$V_a = 2 * (3340 \text{ lb/bolt}) * 1.6 = 10 \text{ kip} > F = 1.6 \text{ kip OK}$$



WALL TO BEAM CONNECTION DETAIL

4

SCALE: 1" = 1'-0"

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: BEAM-4

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

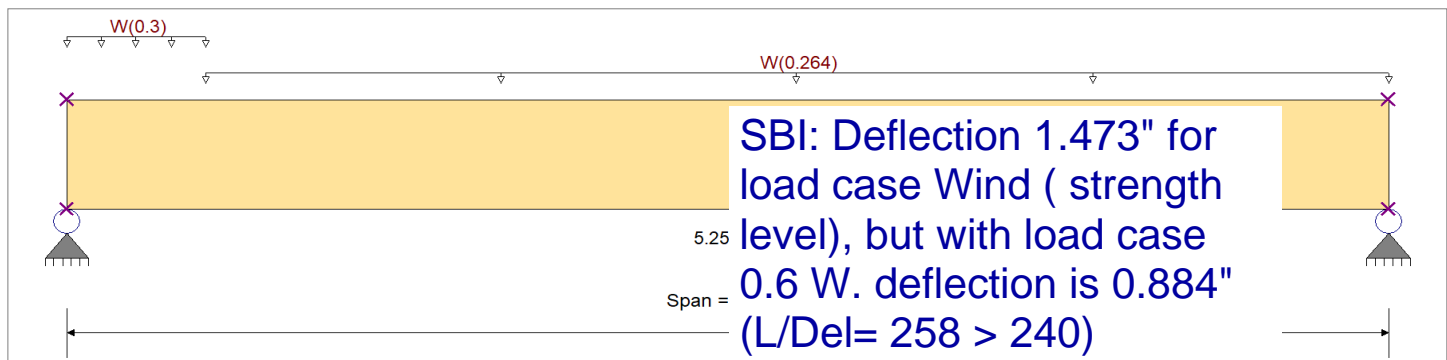
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : iLevel Truss Joist
 Wood Grade : Parallam PSL 2.2E

Beam Bracing : Completely Unbraced

Fb + 2,900.0 psi
 Fb - 2,900.0 psi
 Fc - Prll 2,900.0 psi
 Fc - Perp 750.0 psi
 Fv 290.0 psi
 Ft 2,025.0 psi
 E : Modulus of Elasticity
 Ebend- xx 2,200.0 ksi
 Eminbend - xx 1,118.19 ksi
 Density 45.070 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Load for Span Number 1

Uniform Load : W = 0.2640 k/ft, Extent = 2.0 --> 19.0 ft, Tributary Width = 1.0 ft

Uniform Load : W = 0.30 k/ft, Extent = 0.0 --> 2.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

										Design N.G.			
Maximum Bending Stress Ratio					=	0.214 1					Maximum Shear Stress Ratio		
Section used for this span					=	5.25x20					Section used for this span		
fb: Actual					=	936.41 psi					fv: Actual		
F'b					=	4,384.22 psi					F'v		
Load Combination					=	+0.60W					Load Combination		
Location of maximum on span					=	9.500ft					Location of maximum on span		
Span # where maximum occurs					=	Span # 1					Span # where maximum occurs		
Maximum Deflection													
Max Downward Transient Deflection						1.473 in Ratio = 154 < 360					Span: 1 : W Only		
Max Upward Transient Deflection						0 in Ratio = 0 < 360					n/a		
Max Downward Total Deflection						0.884 in Ratio = 258 >= 240					Span: 1 : +0.60W		
Max Upward Total Deflection						0 in Ratio = 0 < 240					n/a		

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 19.0 ft	1			0.90	1.00	1.00	1.00	0.945	1.00	1.00	1.00			0.0	0.00	0.0	0.0
+0.60W					1.00	1.00	1.00	0.945	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 19.0 ft	1	0.214	0.038	1.60	1.00	1.00	1.00	0.945	1.00	1.00	1.00	7.17	936.4	4,384.2	1.25	17.8	464.0
+0.450W					1.00	1.00	1.00	0.945	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 19.0 ft	1	0.160	0.029	1.60	1.00	1.00	1.00	0.945	1.00	1.00	1.00	5.38	702.3	4,384.2	0.93	13.4	464.0

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: BEAM-4

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	1.4728	9.500		0.0000	0.000

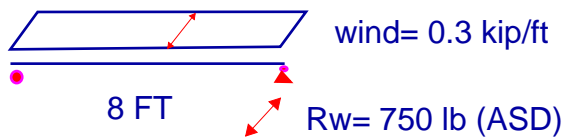
Vertical Reactions

Support notation : Far left is #1

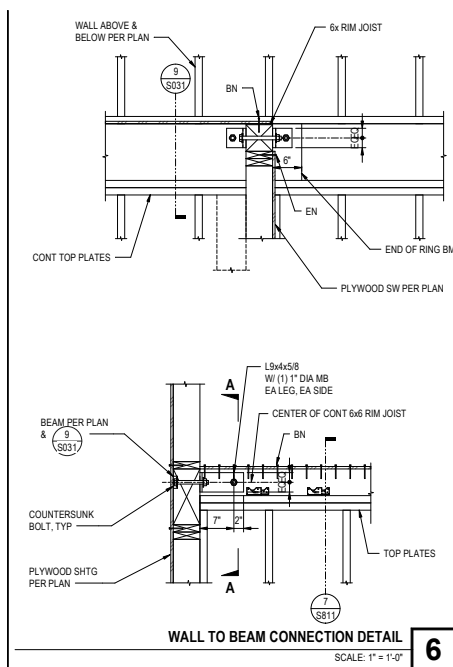
Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.576	2.512
Max Upward from Load Combinations	1.546	1.507
Max Upward from Load Cases	2.576	2.512
+0.60W	1.546	1.507
+0.450W	1.159	1.130
W Only	2.576	2.512

BEAM -5



6x12 is OK
DCR=0.138
L/Del= 1467



Angle Design:

$M_{max} = 0.75 \text{ kip} \cdot 2'' = 1.5 \text{ kip}''$
 $S_{req} = 1.5 \text{ kip}'' / 0.6 \cdot 36 \text{ ksi} = 0.06 \text{ in}^3$
try angle L 8x4x 5/8

$S_p = 4'' \times (5/8)^2 / 6 = 0.26 \text{ in}^3 > S_{req} = 0.06 \text{ in}^3$

Check bearing on wood beam:

A bearing= $(1.25^2 \cdot 3.14) / 4 = 1.23 \text{ in}^2 / \text{bolt}$

P bearing= $750 \text{ psi} \cdot 1.23 \text{ in}^2 \cdot 2 \cdot 1.6 = 2.9 \text{ kip} > F = 0.75 \text{ kip}$
OK

Checking shear in 6x6

3/4" diameter MB

$V_a = 1 \cdot (3340 \text{ lb/bolt}) \cdot 1.6 = 5.3 \text{ kip} > F = 0.75 \text{ kip}$ OK

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: BEAM-5

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

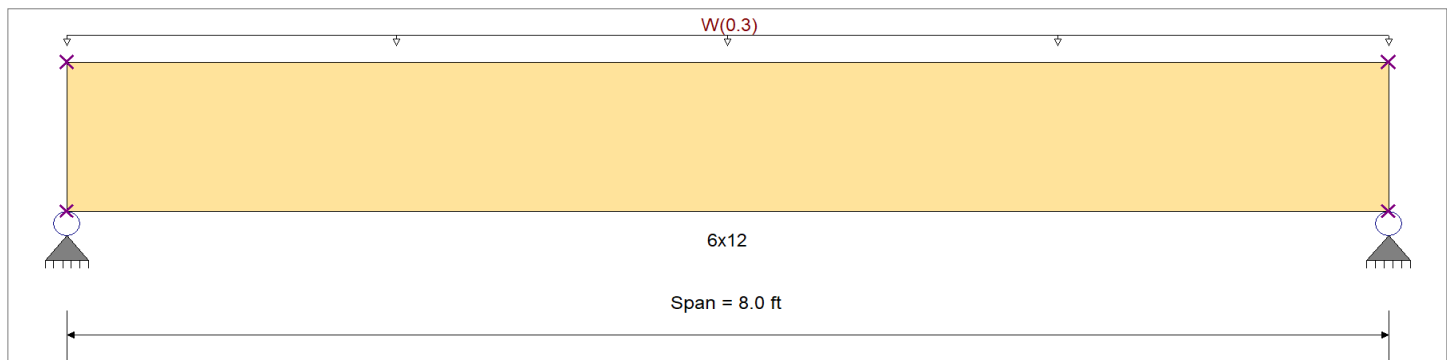
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
 Wood Grade : No.1

Beam Bracing : Completely Unbraced

Fb + 1,350.0 psi E : Modulus of Elasticity
 Fb - 1,350.0 psi Ebend- xx 1,600.0ksi
 Fc - Prll 925.0 psi Eminbend - xx 580.0ksi
 Fc - Perp 625.0 psi
 Fv 170.0 psi
 Ft 675.0 psi Density 31.210pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Load for Span Number 1

Uniform Load : W = 0.30 k/ft, Extent = 0.0 --> 8.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.138	1	Section used for this span	=	0.048	1
fb: Actual	=	298.04psi		fv: Actual	=	13.09 psi	
F'b	=	2,160.00psi		F'v	=	272.00 psi	
Load Combination	=	+0.60W		Load Combination	=	+0.60W	
Location of maximum on span	=	4.000ft		Location of maximum on span	=	7.066 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.109 in	Ratio =	880 >=360	Span: 1 : W Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.065 in	Ratio =	1467 >=240	Span: 1 : +0.60W			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 8.0 ft	1			0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.138	0.048	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.44	298.0	2,160.0	0.55	13.1	272.0
+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.103	0.036	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.08	223.5	2,160.0	0.41	9.8	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.1090	4.029		0.0000	0.000

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: BEAM-5

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.200	1.200
Max Upward from Load Combinations	0.720	0.720
Max Upward from Load Cases	1.200	1.200
+0.60W	0.720	0.720
+0.450W	0.540	0.540
W Only	1.200	1.200

Wood Beam

Project File: FS 46 enercalc.ec6

LIC#: KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-1 L=4 ft

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

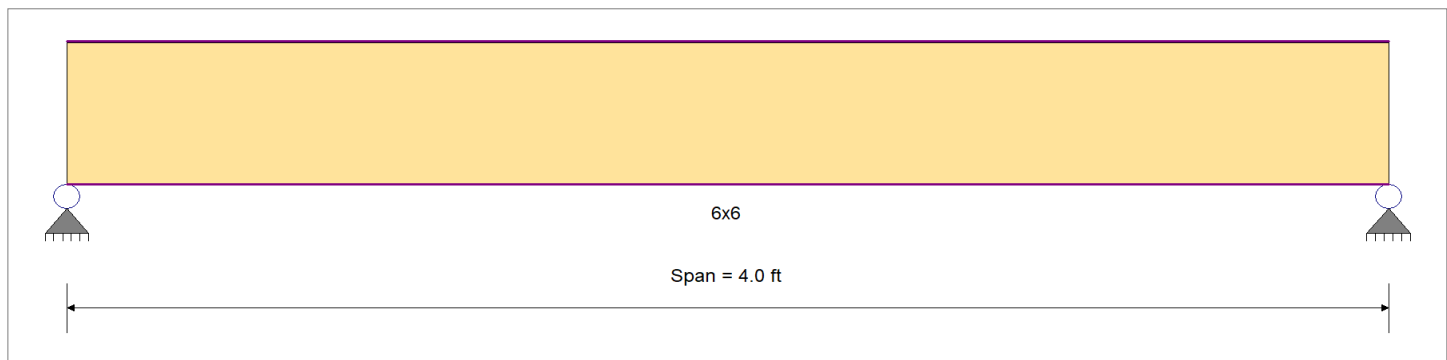
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
 Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 1,350.0 psi E : Modulus of Elasticity
 Fb - 1,350.0 psi Ebend- xx 1,600.0ksi
 Fc - Prll 925.0 psi Eminbend - xx 580.0ksi
 Fc - Perp 625.0 psi
 Fv 170.0 psi
 Ft 675.0 psi Density 31.210pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.667	1	Section used for this span	=	0.469	1
fb: Actual	=	1,125.17	psi	fv: Actual	=	99.75	psi
F'b	=	1,687.50	psi	F'v	=	212.50	psi
Load Combination	=	+D+Lr		Load Combination	=	+D+Lr	
Location of maximum on span	=	2.000ft		Location of maximum on span	=	3.547 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.028 in	Ratio =	1684 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.062 in	Ratio =	777 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only															0.0		0.0	0.0
Length = 4.0 ft	1		0.499	0.351	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.40	605.9	1,215.0	1.08	53.7	153.0
+D+Lr						1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	1		0.667	0.469	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.60	1,125.2	1,687.5	2.01	99.8	212.5
+D+0.750Lr						1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	1		0.590	0.415	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.30	995.3	1,687.5	1.78	88.2	212.5
+0.60D						1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	1		0.168	0.118	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.84	363.5	2,160.0	0.65	32.2	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0617	2.015		0.0000	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-1 L=4 ft

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.600	2.600
Max Upward from Load Combinations	2.600	2.600
Max Upward from Load Cases	1.400	1.400
D Only	1.400	1.400
+D+Lr	2.600	2.600
+D+0.750Lr	2.300	2.300
+0.60D	0.840	0.840
Lr Only	1.200	1.200

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: FS 46 enercalc.ec6

LIC#: KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-1 L=6 ft

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

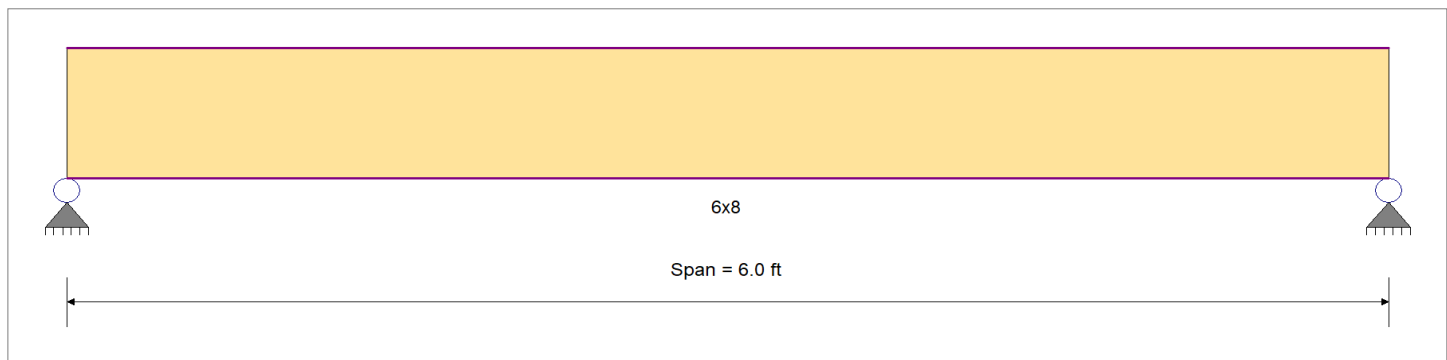
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1,350.0 psi	E : Modulus of Elasticity	
Fb -	1,350.0 psi	Ebend- xx	1,600.0ksi
Fc - Prll	925.0 psi	Eminbend - xx	580.0ksi
Fc - Perp	625.0 psi		
Fv	170.0 psi		
Ft	675.0 psi	Density	31.210pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.807	1	Section used for this span	=	0.531	1
fb: Actual	=	1,361.45	psi	fv: Actual	=	112.83	psi
F'b	=	1,687.50	psi	F'v	=	212.50	psi
Load Combination		+D+Lr		Load Combination		+D+Lr	
Location of maximum on span	=	3.000ft		Location of maximum on span	=	0.000ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.057 in	Ratio =	1265 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.123 in	Ratio =	584 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 6.0 ft	1	0.603	0.397	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.15	733.1	1,215.0	1.67	60.8	153.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.0 ft	1	0.807	0.531	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.85	1,361.5	1,687.5	3.10	112.8	212.5
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.0 ft	1	0.714	0.470	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.18	1,204.4	1,687.5	2.74	99.8	212.5
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.0 ft	1	0.204	0.134	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.89	439.9	2,160.0	1.00	36.5	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.1232	3.022		0.0000	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-1 L=6 ft

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.900	3.900
Max Upward from Load Combinations	3.900	3.900
Max Upward from Load Cases	2.100	2.100
D Only	2.100	2.100
+D+Lr	3.900	3.900
+D+0.750Lr	3.450	3.450
+0.60D	1.260	1.260
Lr Only	1.800	1.800

Wood Beam

Project File: FS 46 enercalc.ec6

LIC#: KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-1 L=8 ft

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

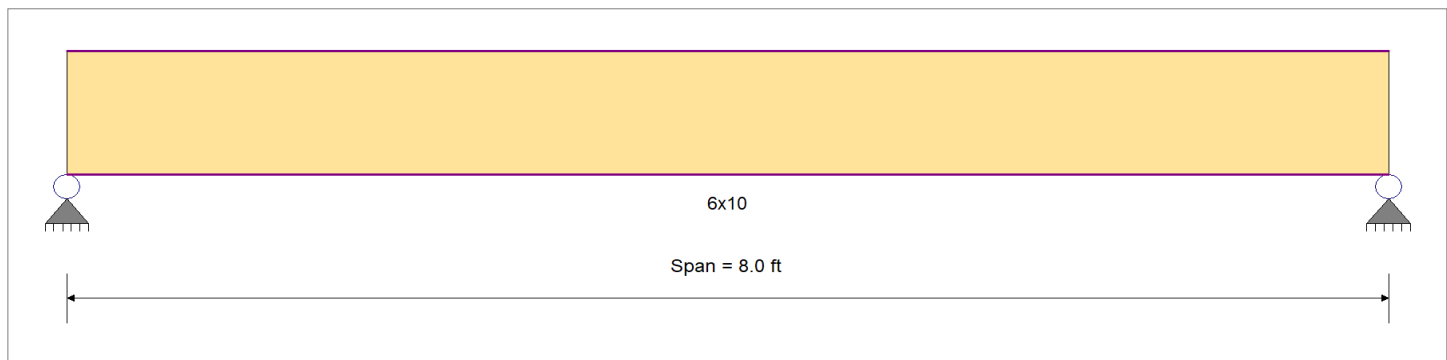
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
 Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

F_b + 1,350.0 psi
 F_b - 1,350.0 psi
 F_c - Prll 925.0 psi
 F_c - Perp 625.0 psi
 F_v 170.0 psi
 F_t 675.0 psi
 E : Modulus of Elasticity
 E_{bend}- xx 1,600.0ksi
 E_{minbend} - xx 580.0ksi
 Density 31.210pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.894	1	Section used for this span	=	0.564	1
fb: Actual	=	1,508.54	psi	fv: Actual	=	119.86	psi
F'b	=	1,687.50	psi	F'v	=	212.50	psi
Load Combination		+D+Lr		Load Combination		+D+Lr	
Location of maximum on span	=	4.000ft		Location of maximum on span	=	7.212 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.088 in	Ratio =	1085 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.192 in	Ratio =	500 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.669	0.422	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.60	812.3	1,215.0	2.25	64.5	153.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.894	0.564	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	10.40	1,508.5	1,687.5	4.18	119.9	212.5
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.791	0.499	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	9.20	1,334.5	1,687.5	3.69	106.0	212.5
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.226	0.142	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.36	487.4	2,160.0	1.35	38.7	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.1917	4.029		0.0000	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-1 L=8 ft

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	5.200	5.200
Max Upward from Load Combinations	5.200	5.200
Max Upward from Load Cases	2.800	2.800
D Only	2.800	2.800
+D+Lr	5.200	5.200
+D+0.750Lr	4.600	4.600
+0.60D	1.680	1.680
Lr Only	2.400	2.400

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-1 L=10 ft

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

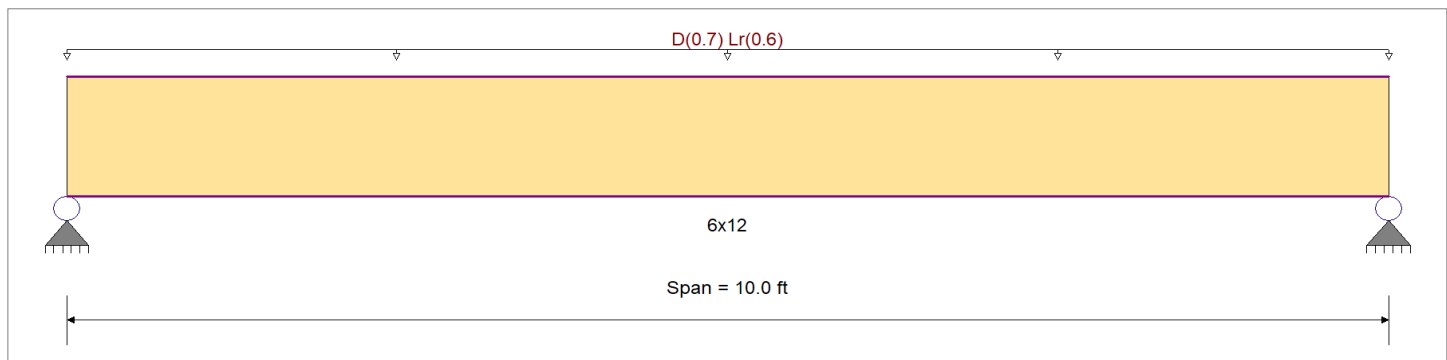
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
 Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 1,350.0 psi E : Modulus of Elasticity
 Fb - 1,350.0 psi Ebend- xx 1,600.0ksi
 Fc - Prll 925.0 psi Eminbend - xx 580.0ksi
 Fc - Perp 625.0 psi
 Fv 170.0 psi
 Ft 675.0 psi Density 31.210pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Load for Span Number 1

Uniform Load : D = 0.70, Lr = 0.60 k/ft, Extent = 0.0 --> 10.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.953	1	Section used for this span	=	0.588	1
fb: Actual	=	1,608.52psi		fv: Actual	=	124.90 psi	
F'b	=	1,687.50psi		F'v	=	212.50 psi	
Load Combination		+D+Lr		Load Combination		+D+Lr	
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 Transient Deflection	0.122 in	Ratio =	985 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.264 in	Ratio =	454 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.713	0.440	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.75	866.1	1,215.0	2.84	67.3	153.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.953	0.588	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	16.25	1,608.5	1,687.5	5.27	124.9	212.5
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.843	0.520	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	14.38	1,422.9	1,687.5	4.66	110.5	212.5
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.241	0.148	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.25	519.7	2,160.0	1.70	40.4	272.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-1 L=10 ft

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.2638	5.036		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	6.500	6.500
Max Upward from Load Combinations	6.500	6.500
Max Upward from Load Cases	3.500	3.500
D Only	3.500	3.500
+D+Lr	6.500	6.500
+D+0.750Lr	5.750	5.750
+0.60D	2.100	2.100
Lr Only	3.000	3.000

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-1 L=12

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

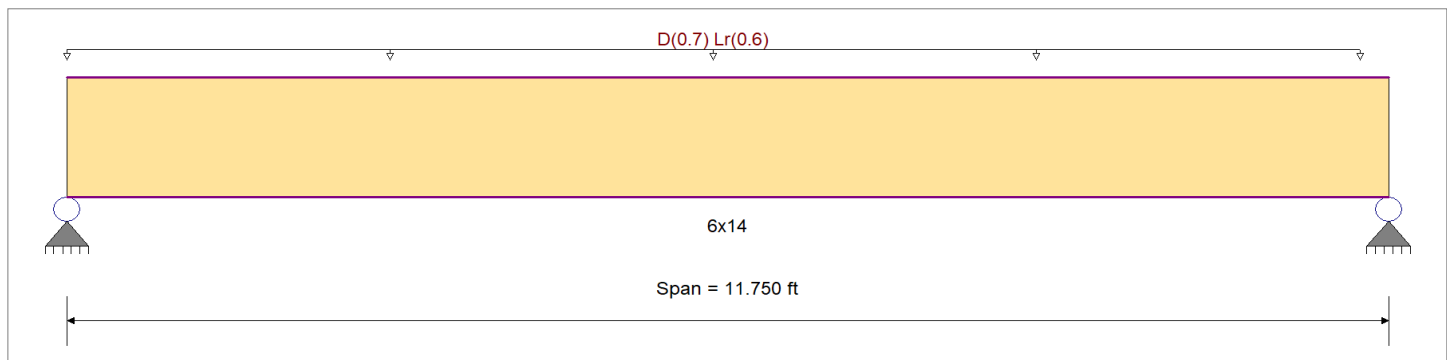
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
 Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 1,350.0 psi
 Fb - 1,350.0 psi
 Fc - Prll 925.0 psi
 Fc - Perp 625.0 psi
 Fv 170.0 psi
 Ft 675.0 psi
 E : Modulus of Elasticity
 Ebend- xx 1,600.0ksi
 Eminbend - xx 580.0ksi
 Density 31.210pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Load for Span Number 1

Uniform Load : D = 0.70, Lr = 0.60 k/ft, Extent = 0.0 --> 11.50 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.967	1	Section used for this span	=	0.589	1
fb: Actual	=	1,610.04	psi	fv: Actual	=	125.08	psi
F'b	=	1,665.56	psi	F'v	=	212.50	psi
Load Combination	=	+D+Lr		Load Combination	=	+D+Lr	
Location of maximum on span	=	5.875	ft	Location of maximum on span	=	10.635	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.143 in	Ratio =	983 >= 360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 < 360	n/a			
Max Downward Total Deflection	0.310 in	Ratio =	454 >= 240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 < 240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0			
Length = 11.750 ft	1	0.723	0.440	0.90	1.00	1.00	1.00	0.987	1.00	1.00	1.00	12.07	866.9	1,199.2	3.33	67.4	153.0
+D+Lr														0.0			
Length = 11.750 ft	1	0.967	0.589	1.25	1.00	1.00	1.00	0.987	1.00	1.00	1.00	22.41	1,610.0	1,665.6	6.19	125.1	212.5
+D+0.750Lr														0.0			
Length = 11.750 ft	1	0.855	0.521	1.25	1.00	1.00	1.00	0.987	1.00	1.00	1.00	19.83	1,424.3	1,665.6	5.48	110.6	212.5
+0.60D														0.0			
Length = 11.750 ft	1	0.244	0.149	1.60	1.00	1.00	1.00	0.987	1.00	1.00	1.00	7.24	520.2	2,131.9	2.00	40.4	272.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HEADER-1 L=12

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.3105	5.875		0.0000	0.000

Vertical Reactions

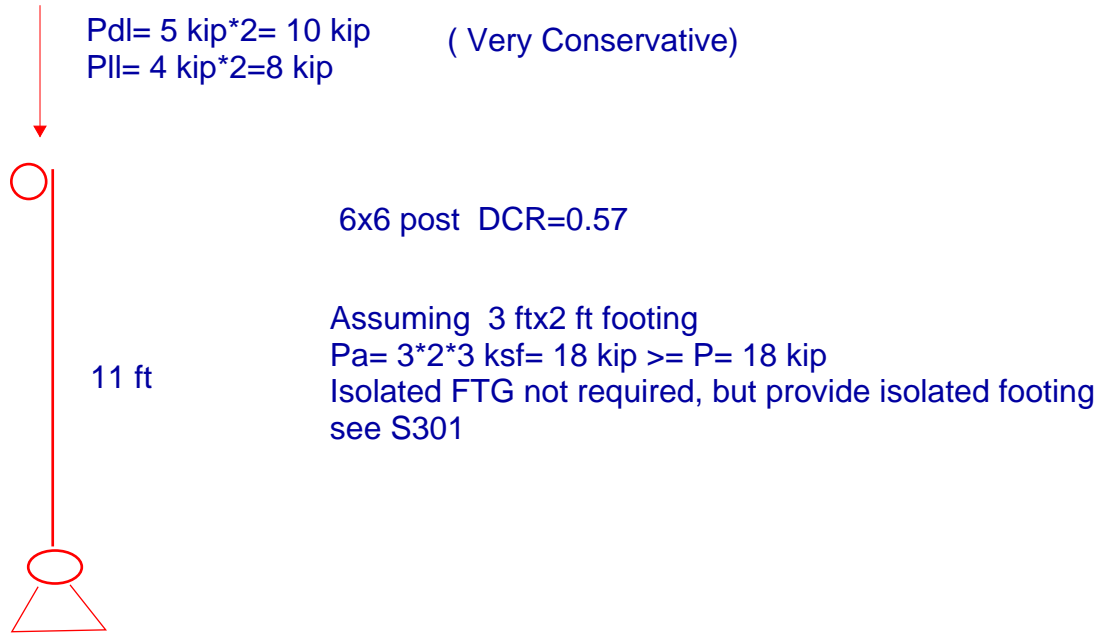
Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	7.634	7.316
Max Upward from Load Combinations	7.634	7.316
Max Upward from Load Cases	4.111	3.939
D Only	4.111	3.939
+D+Lr	7.634	7.316
+D+0.750Lr	6.753	6.472
+0.60D	2.466	2.364
Lr Only	3.523	3.377

1.4 WOOD POST DESIGN

Typical wood post 6x6



Wood Column

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Typical Post 6x6 H=11 ft

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
 Load Combinations Used : ASCE 7-16

General Information

Analysis Method		Allowable Stress Design		Wood Section Name		6x6	
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber	
Overall Column Height		11 ft		Wood Member Type		Sawn	
(Used for non-slender calculations)							
Wood Species				Exact Width		5.50 in	
Wood Grade				Exact Depth		5.50 in	
Fb +		1,500.0 psi	Fv	150.0 psi	Area	30.250 in^2	Allow Stress Modification Factors
Fb -		1,500.0 psi	Ft	1,000.0 psi	Ix	76.255 in^4	Cf or Cv for Bending 1.0
Fc - Prll		1,000.0 psi	Density	33.0 pcf	Iy	76.255 in^4	Cf or Cv for Compression 1.0
Fc - Perp		1,000.0 psi					Cf or Cv for Tension 1.0
							Cm : Wet Use Factor 1.0
							Ct : Temperature Fact 1.0
							Cfu : Flat Use Factor 1.0
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			Kf : Built-up columns 1.0
Basic		1,400.0	1,400.0	1,400.0 ksi			Use Cr : Repetitive ? No
Minimum		1,400.0	1,400.0				
Brace condition for deflection (buckling) along columns :							
				X-X (width) axis :	Unbraced Length for buckling ABOUT Y-Y Axis = 11 ft, k		
				Y-Y (depth) axis :	Unbraced Length for buckling ABOUT X-X Axis = 11 ft, k		

Applied Loads

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

Column self weight included : 76.255 lbs * Dead Load Factor

AXIAL LOADS . . .

Axial Load at 11.0 ft, D = 10.0, Lr = 8.0 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.5799 : 1	Maximum SERVICE Lateral Load Reactions . .	
Load Combination	+D+Lr	Top along Y-Y	0.0 k
Governing NDS Formula	Comp Only, f_c/F_c'	Bottom along Y-Y	0.0 k
Location of max.above base	0.0 ft	Top along X-X	0.0 k
At maximum location values are .		Bottom along X-X	0.0 k
Applied Axial	18.076 k	Maximum SERVICE Load Lateral Deflections . .	
Applied Mx	0.0 k-ft	Along Y-Y	0.0 in at 0.0 ft above base
Applied My	0.0 k-ft	for load combination : n/a	
Fc : Allowable	1,030.48 psi	Along X-X	0.0 in at 0.0 ft above base
		for load combination : n/a	
PASS Maximum Shear Stress Ratio =	0.0 : 1	Other Factors used to calculate allowable stresses . .	
Load Combination	+0.60D	Bending	Compression
Location of max.above base	11.0 ft	Tension	
Applied Design Shear	0.0 psi		
Allowable Shear	240.0 psi		

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.883	0.4190	PASS	0.0 ft	0.0	PASS	11.0 ft
+D+Lr	1.250	0.824	0.5799	PASS	0.0 ft	0.0	PASS	11.0 ft
+D+0.750Lr	1.250	0.824	0.5157	PASS	0.0 ft	0.0	PASS	11.0 ft
+0.60D	1.600	0.762	0.1640	PASS	0.0 ft	0.0	PASS	11.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		Y-Y Axis Reaction		Axial Reaction	My - End Moments		Mx - End Moments	
	@ Base	@ Top	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top
D Only					10.076				
+D+Lr					18.076				
+D+0.750Lr					16.076				

Wood Column

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Typical Post 6x6 H=11 ft

Maximum Reactions

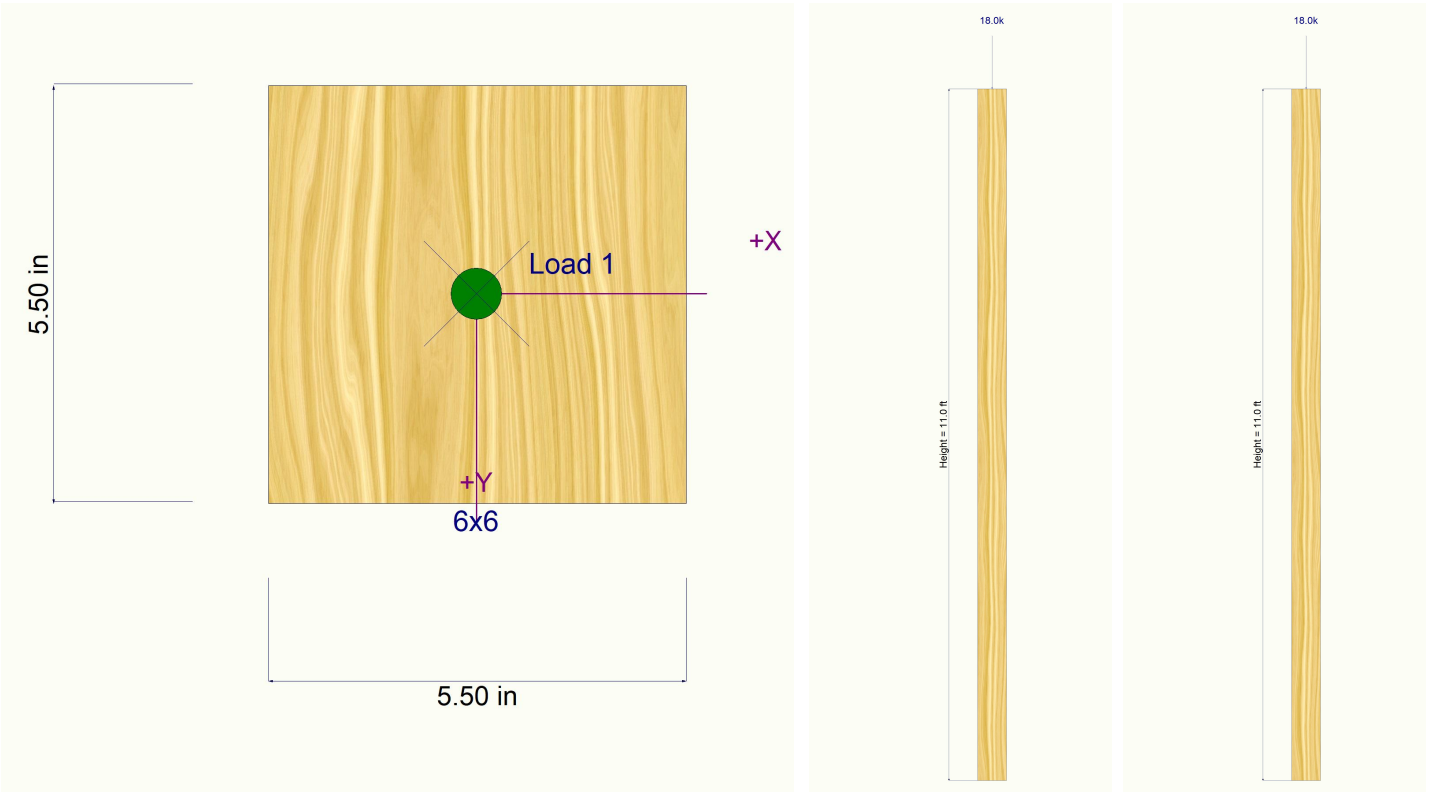
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
+0.60D						6.046					
Lr Only						8.000					

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000ft	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000ft	0.000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000ft	0.000 in	0.000 ft

Sketches

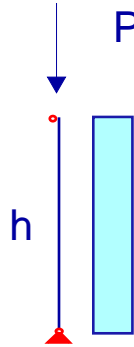


1.5 WOOD STUD WALL DESIGN

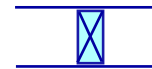
Stud wall design at F/S501

$$P_d = 22.5 \text{ psf} \cdot 16/12 \cdot 18 \text{ ft}/2 = 263 \text{ lb}$$

$$P_{II} = 20 \text{ psf} * 16/12 * 18 \text{ ft} / 2 = 234 \text{ lb}$$



$$W_{\text{wind}} = 30 \text{ psf} \cdot 16/12 = 40 \text{ lb/ft (LRFD)}$$



$$h = 21' - 0'' - (0' - 3'' + 2.5'' + 0' - 6'') = 20'$$

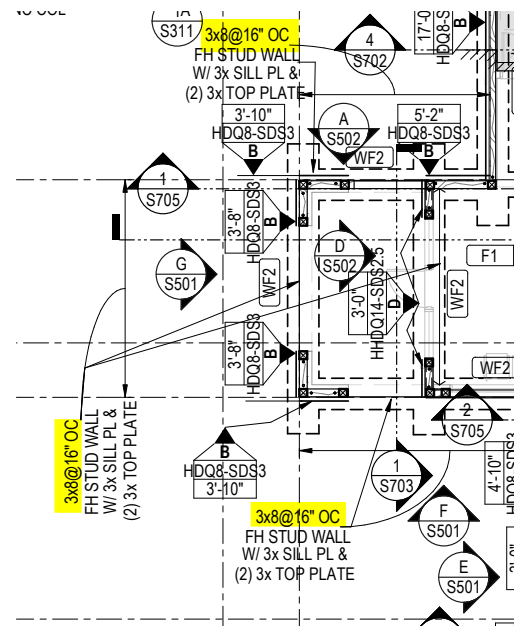
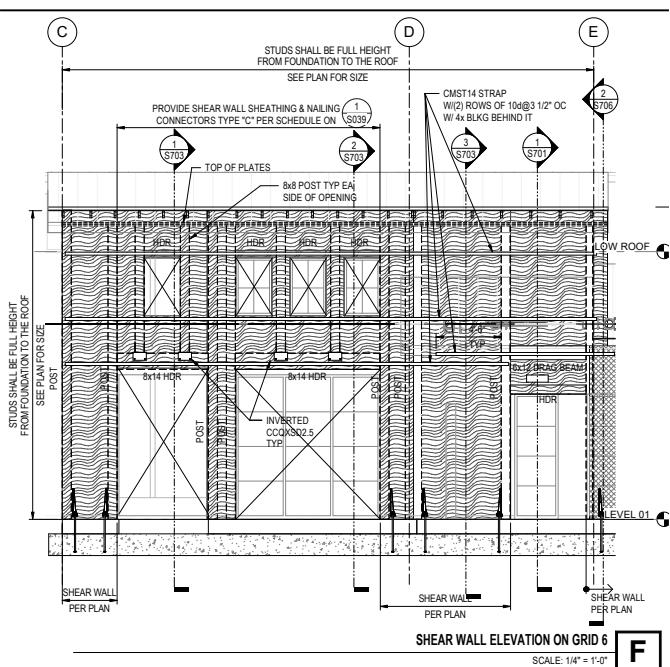
2x6 @ 16 " DCR= 0.92

$$\Delta = 2.3'' \times 0.42 = 0.96'' \quad L/\Delta = 17.25 \text{ ft} \times 12/0.97'' = 214 < 360 \text{ NG}$$

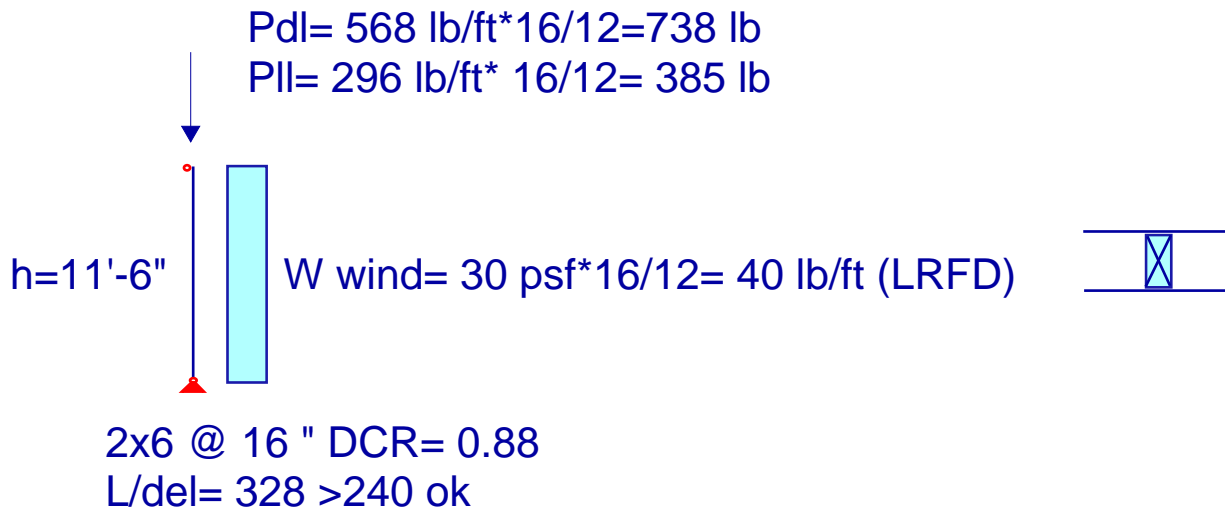
3x8 @ 16 " DCR= 0.4

$$\text{del} = 1 \times 0.42 = 0.42'' \quad L/\text{Del} = 20 \text{ ft} \times 12/0.42'' = 571 > 360$$

Provide 3x8 @ 16" o.c OK

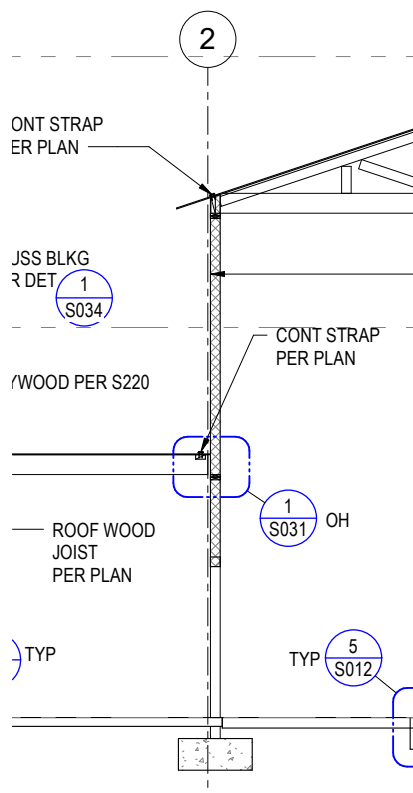
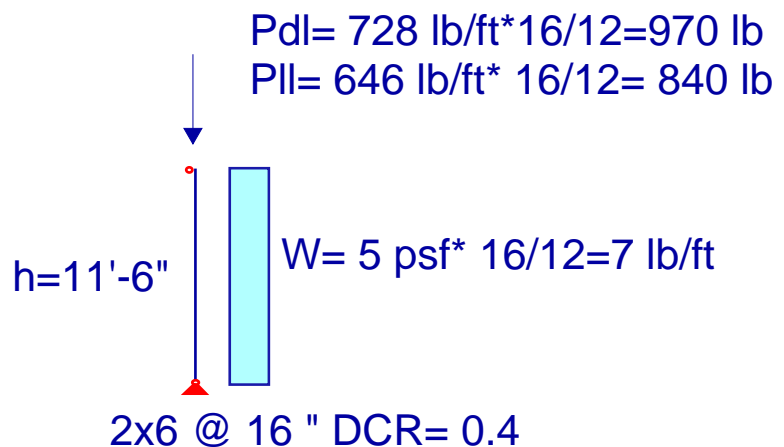


Typical Exterior Wall Design



$Wdl = 15 \text{ psf} \times 12 \text{ ft} + 21 \text{ psf} \times 37 \text{ ft} / 2 = 568 \text{ lb/ft}$
 $WII = 16 \text{ psf} \times 37 \text{ ft} / 2 = 296 \text{ lb/ft}$

Typical Interior Wall Design



8. TYPICAL WOOD STUDS SHALL BE AS FOLLOWS, UNLESS NOTED OTHERWISE ON PLAN

SHEAR WALL:	2x6@16" AT ALL EXTERIOR WALL, UNO 2x6 @16" AT INTERIOR WALL UNO
EXTERIOR WALL (EXCLUDING THE APPARATUS RESERVE BUILDING):	2x6@16" UNO
EXTERIOR WALL FOR APPARATUS RESERVE BUILDING:	2x8" @16" OC
INTERIOR BEARING WALL:	2x6@16" AND LARGER AS INDICATED BY ARCH
INTERIOR NON-BEARING WALL:	2x4 @16" FOR HEIGHTS UP TO 10' OR 2x6@16" AND LARGER AS INDICATED BY ARCH

Wood Column

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Stud Wall at main building Elevation F on S501

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
 Load Combinations Used : ASCE 7-16

General Information

Analysis Method		Allowable Stress Design		Wood Section Name		3x8	
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber	
Overall Column Height		20 ft		Wood Member Type		Sawn	
(Used for non-slender calculations)							
Wood Species		Douglas Fir-Larch		Exact Width		2.50 in	
Wood Grade		No.1		Exact Depth		7.250 in	
Fb +		1,000.0 psi	Fv	180.0 psi	Area		18.125 in^2
Fb -		1,000.0 psi	Ft	675.0 psi	Ix		79.391 in^4
Fc - Prll		1,500.0 psi	Density	31.210 pcf	Iy		9.440 in^4
Fc - Perp		625.0 psi					
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			
Basic		1,700.0	1,700.0	1,700.0 ksi		Cf or Cv for Bending 1.20	
Minimum		620.0	620.0			Cf or Cv for Compression 1.050	
						Cf or Cv for Tension 1.20	
						Cm : Wet Use Factor 1.0	
						Ct : Temperature Fact 1.0	
						Cfu : Flat Use Factor 1.0	
						Kf : Built-up columns 1.0	
						Use Cr : Repetitive ? No	
Brace condition for deflection (buckling) along columns :							
				X-X (width) axis : Fully braced against buckling ABOUT Y-Y Axis			
				Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 20 ft, k			

Applied Loads

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

Column self weight included : 78.567 lbs * Dead Load Factor

AXIAL LOADS . . .

Axial Load at 20.0 ft, D = 0.260, Lr = 0.2340 k

BENDING LOADS . . .

Lat. Uniform Load creating Mx-x, W = 0.040 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.3585 : 1**
 Load Combination +D+0.60W
 Governing NDS Formula Comp + Mxx, NDS Eq. 3.9-3
 Location of max.above base 9.933 ft
 At maximum location values are .
 Applied Axial 0.3386 k
 Applied Mx 1.20 k-ft
 Applied My 0.0 k-ft
 Fc : Allowable 445.897 psi

Maximum SERVICE Lateral Load Reactions . .
 Top along Y-Y 0.40 k Bottom along Y-Y 0.40 k
 Top along X-X 0.0 k Bottom along X-X 0.0 k

Maximum SERVICE Load Lateral Deflections . . .
 Along Y-Y 1.078 in at 10.067 ft above base
 for load combination : W Only
 Along X-X 0.0 in at 0.0 ft above base
 for load combination : n/a

Other Factors used to calculate allowable stresses . . .
 Bending Compression Tension

PASS Maximum Shear Stress Ratio = **0.06897 : 1**
 Load Combination +D+0.60W
 Location of max.above base 20.0 ft
 Applied Design Shear 19.862 psi
 Allowable Shear 288.0 psi

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.302	0.04364	PASS	0.0 ft	0.0	PASS	20.0 ft
+D+Lr	1.250	0.223	0.07183	PASS	0.0 ft	0.0	PASS	20.0 ft
+D+0.750Lr	1.250	0.223	0.06449	PASS	0.0 ft	0.0	PASS	20.0 ft
+D+0.60W	1.600	0.177	0.3585	PASS	9.933 ft	0.06897	PASS	20.0 ft
+D+0.750Lr+0.450W	1.600	0.177	0.2776	PASS	9.933 ft	0.05172	PASS	20.0 ft
+D+0.450W	1.600	0.177	0.2693	PASS	9.933 ft	0.05172	PASS	20.0 ft
+0.60D+0.60W	1.600	0.177	0.3515	PASS	9.933 ft	0.06897	PASS	20.0 ft
+0.60D	1.600	0.177	0.02514	PASS	0.0 ft	0.0	PASS	20.0 ft

Wood Column

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Stud Wall at main building Elevation F on S501

Maximum Reactions

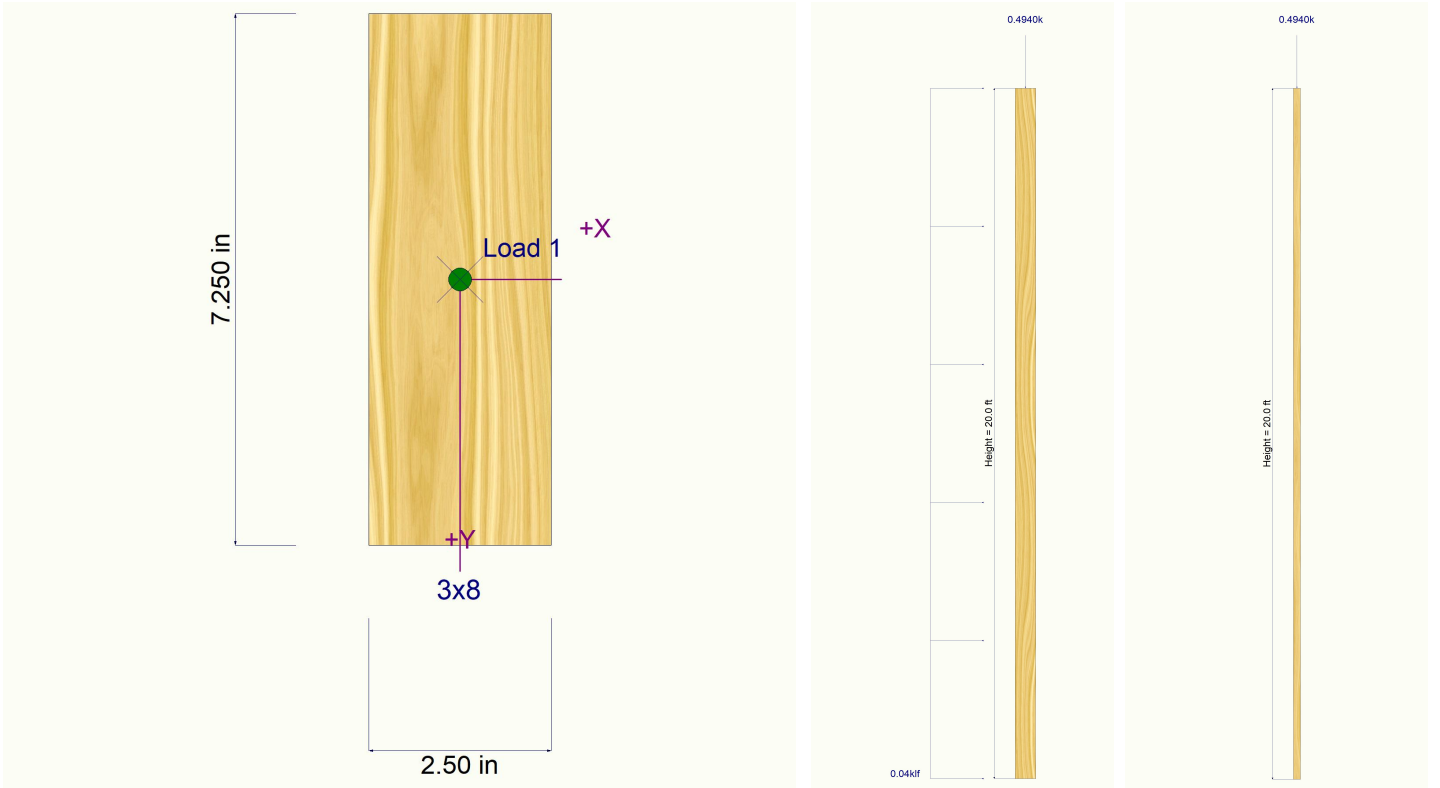
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
D Only						0.339					
+D+Lr						0.573					
+D+0.750Lr						0.514					
+D+0.60W				0.240	0.240	0.339					
+D+0.750Lr+0.450W				0.180	0.180	0.514					
+D+0.450W				0.180	0.180	0.339					
+0.60D+0.60W				0.240	0.240	0.203					
+0.60D						0.203					
Lr Only						0.234					
W Only				0.400	0.400						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.000 in	0.000ft	0.000 in	0.000 ft
+D+Lr	0.000 in	0.000ft	0.000 in	0.000 ft
+D+0.750Lr	0.000 in	0.000ft	0.000 in	0.000 ft
+D+0.60W	0.000 in	0.000ft	0.647 in	10.067 ft
+D+0.750Lr+0.450W	0.000 in	0.000ft	0.485 in	10.067 ft
+D+0.450W	0.000 in	0.000ft	0.485 in	10.067 ft
+0.60D+0.60W	0.000 in	0.000ft	0.647 in	10.067 ft
+0.60D	0.000 in	0.000ft	0.000 in	0.000 ft
Lr Only	0.000 in	0.000ft	0.000 in	0.000 ft
W Only	0.000 in	0.000ft	1.078 in	10.067 ft

Sketches



Wood Column

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Typical Interior Wall stud Design

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
 Load Combinations Used : ASCE 7-16

General Information

Analysis Method	Allowable Stress Design			Wood Section Name	2x6	
End Fixities	Top & Bottom Pinned			Wood Grading/Manuf.	Graded Lumber	
Overall Column Height	11.5 ft			Wood Member Type	Sawn	
(Used for non-slender calculations)						
Wood Species	Douglas Fir-Larch			Exact Width	1.50 in	Allow Stress Modification Factors
Wood Grade	No.1			Exact Depth	5.50 in	
Fb +	1,000.0 psi	Fv	180.0 psi	Area	8.250 in^2	Cf or Cv for Bending 1.30
Fb -	1,000.0 psi	Ft	675.0 psi	Ix	20.797 in^4	Cf or Cv for Compression 1.10
Fc - Prll	1,500.0 psi	Density	31.210 pcf	Iy	1.547 in^4	Cf or Cv for Tension 1.30
Fc - Perp	625.0 psi					Cm : Wet Use Factor 1.0
						Ct : Temperature Fact 1.0
E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial			Cfu : Flat Use Factor 1.0
	Basic	1,700.0	1,700.0	1,700.0 ksi		Kf : Built-up columns 1.0
	Minimum	620.0	620.0			Use Cr : Repetitive ? No
Brace condition for deflection (buckling) along columns :						
				X-X (width) axis :	Fully braced against buckling ABOUT Y-Y Axis	
				Y-Y (depth) axis :	Unbraced Length for buckling ABOUT X-X Axis = 11.5 ft,	

Applied Loads

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

Column self weight included : 20.563 lbs * Dead Load Factor

AXIAL LOADS . . .

Axial Load at 11.50 ft, D = 0.90, Lr = 0.80 k

BENDING LOADS . . .

Lat. Uniform Load creating Mx-x, L = 0.0070 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.2858 : 1	Maximum SERVICE Lateral Load Reactions . .	
Load Combination	+D+Lr	Top along Y-Y	0.04025 k
Governing NDS Formula	Comp Only, fc/Fc'	Bottom along Y-Y	0.04025 k
Location of max.above base	0.0 ft	Top along X-X	0.0 k
At maximum location values are .		Bottom along X-X	0.0 k
Applied Axial	1.721 k	Maximum SERVICE Load Lateral Deflections . . .	
Applied Mx	0.0 k-ft	Along Y-Y	0.07876 in at 5.789 ft above base
Applied My	0.0 k-ft	for load combination : +D+L	
Fc : Allowable	729.64 psi	Along X-X	0.0 in at 0.0 ft above base
		for load combination : n/a	
PASS Maximum Shear Stress Ratio =	0.04066 : 1	Other Factors used to calculate allowable stresses . . .	
Load Combination	+D+L	Bending	Compression
Location of max.above base	0.0 ft	Tension	
Applied Design Shear	7.318 psi		
Allowable Shear	180.0 psi		

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.465	0.1618	PASS	0.0 ft	0.0	PASS	11.50 ft
+D+L	1.000	0.427	0.1889	PASS	5.711 ft	0.04066	PASS	0.0 ft
+D+Lr	1.250	0.354	0.2858	PASS	0.0 ft	0.0	PASS	11.50 ft
+D+0.750Lr+0.750L	1.250	0.354	0.2526	PASS	0.07718 ft	0.02439	PASS	0.0 ft
+D+0.750L	1.150	0.380	0.1547	PASS	0.0 ft	0.02652	PASS	0.0 ft
+0.60D	1.600	0.284	0.08927	PASS	0.0 ft	0.0	PASS	11.50 ft

Wood Column

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Typical Interior Wall stud Design

Maximum Reactions

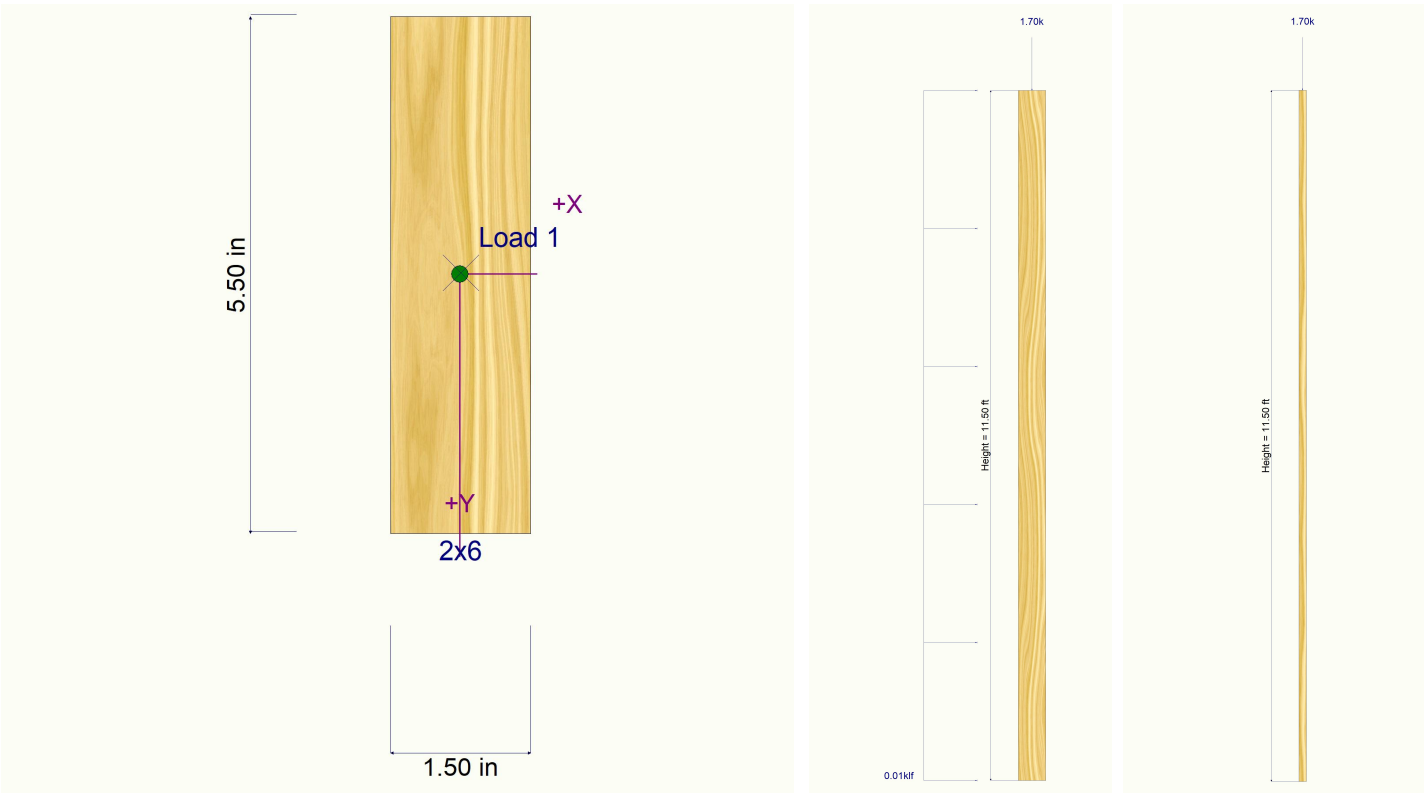
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
D Only						0.921					
+D+L				0.040	0.040	0.921					
+D+Lr						1.721					
+D+0.750Lr+0.750L				0.030	0.030	1.521					
+D+0.750L				0.030	0.030	0.921					
+0.60D						0.552					
Lr Only						0.800					
L Only				0.040	0.040						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000ft	0.000 in	0.000ft
+D+L	0.0000 in	0.000ft	0.079 in	5.789ft
+D+Lr	0.0000 in	0.000ft	0.000 in	0.000ft
+D+0.750Lr+0.750L	0.0000 in	0.000ft	0.059 in	5.789ft
+D+0.750L	0.0000 in	0.000ft	0.059 in	5.789ft
+0.60D	0.0000 in	0.000ft	0.000 in	0.000ft
Lr Only	0.0000 in	0.000ft	0.000 in	0.000ft
L Only	0.0000 in	0.000ft	0.079 in	5.789ft

Sketches



Wood Column

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Typical Exterior Wall stud Design

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
 Load Combinations Used : ASCE 7-16

General Information

Analysis Method	Allowable Stress Design			Wood Section Name	2x6	
End Fixities	Top & Bottom Pinned			Wood Grading/Manuf.	Graded Lumber	
Overall Column Height	11.25 ft			Wood Member Type	Sawn	
(Used for non-slender calculations)						
Wood Species	Douglas Fir-Larch			Exact Width	1.50 in	Allow Stress Modification Factors
Wood Grade	No.1			Exact Depth	5.50 in	
Fb +	1,000.0 psi	Fv	180.0 psi	Area	8.250 in^2	Cf or Cv for Bending 1.30
Fb -	1,000.0 psi	Ft	675.0 psi	Ix	20.797 in^4	Cf or Cv for Compression 1.10
Fc - Prll	1,500.0 psi	Density	31.210 pcf	Iy	1.547 in^4	Cf or Cv for Tension 1.30
Fc - Perp	625.0 psi					Cm : Wet Use Factor 1.0
						Ct : Temperature Fact 1.0
E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial			Cfu : Flat Use Factor 1.0
	Basic	1,700.0	1,700.0	1,700.0 ksi		Kf : Built-up columns 1.0
	Minimum	620.0	620.0			Use Cr : Repetitive ? No
Brace condition for deflection (buckling) along columns :						
				X-X (width) axis :	Fully braced against buckling ABOUT Y-Y Axis	
				Y-Y (depth) axis :	Unbraced Length for buckling ABOUT X-X Axis = 11.25 ft	

Applied Loads

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

Column self weight included : 20.116 lbs * Dead Load Factor

AXIAL LOADS . . .

Axial Load at 11.250 ft, D = 0.750, Lr = 0.40 k

BENDING LOADS . . .

Lat. Uniform Load creating Mx-x, L = 0.040 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.8845 : 1	Maximum SERVICE Lateral Load Reactions . .	
Load Combination	+D+L	Top along Y-Y	0.2250 k
Governing NDS Formula Comp + Mxx, NDS Eq. 3.9-3		Bottom along Y-Y	0.2250 k
Location of max.above base	5.663 ft	Top along X-X	0.0 k
At maximum location values are .		Bottom along X-X	0.0 k
Applied Axial	0.7701 k	Maximum SERVICE Load Lateral Deflections . . .	
Applied Mx	0.6328 k-ft	Along Y-Y	0.4122 in at 5.663 ft above base
Applied My	0.0 k-ft	for load combination : +D+L	
Fc : Allowable	730.04 psi	Along X-X	0.0 in at 0.0 ft above base
		for load combination : n/a	
PASS Maximum Shear Stress Ratio =	0.2273 : 1	Other Factors used to calculate allowable stresses . . .	
Load Combination	+D+L	Bending	Compression
Location of max.above base	0.0 ft	Tension	
Applied Design Shear	40.909 psi		
Allowable Shear	180.0 psi		

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.481	0.1308	PASS	0.0 ft	0.0	PASS	11.250 ft
+D+L	1.000	0.442	0.8845	PASS	5.663 ft	0.2273	PASS	0.0 ft
+D+Lr	1.250	0.367	0.1871	PASS	0.0 ft	0.0	PASS	11.250 ft
+D+0.750Lr+0.750L	1.250	0.367	0.5767	PASS	5.587 ft	0.1364	PASS	11.250 ft
+D+0.750L	1.150	0.394	0.5818	PASS	5.663 ft	0.1482	PASS	11.250 ft
+0.60D	1.600	0.296	0.07177	PASS	0.0 ft	0.0	PASS	11.250 ft

Wood Column

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Typical Exterior Wall stud Design

Maximum Reactions

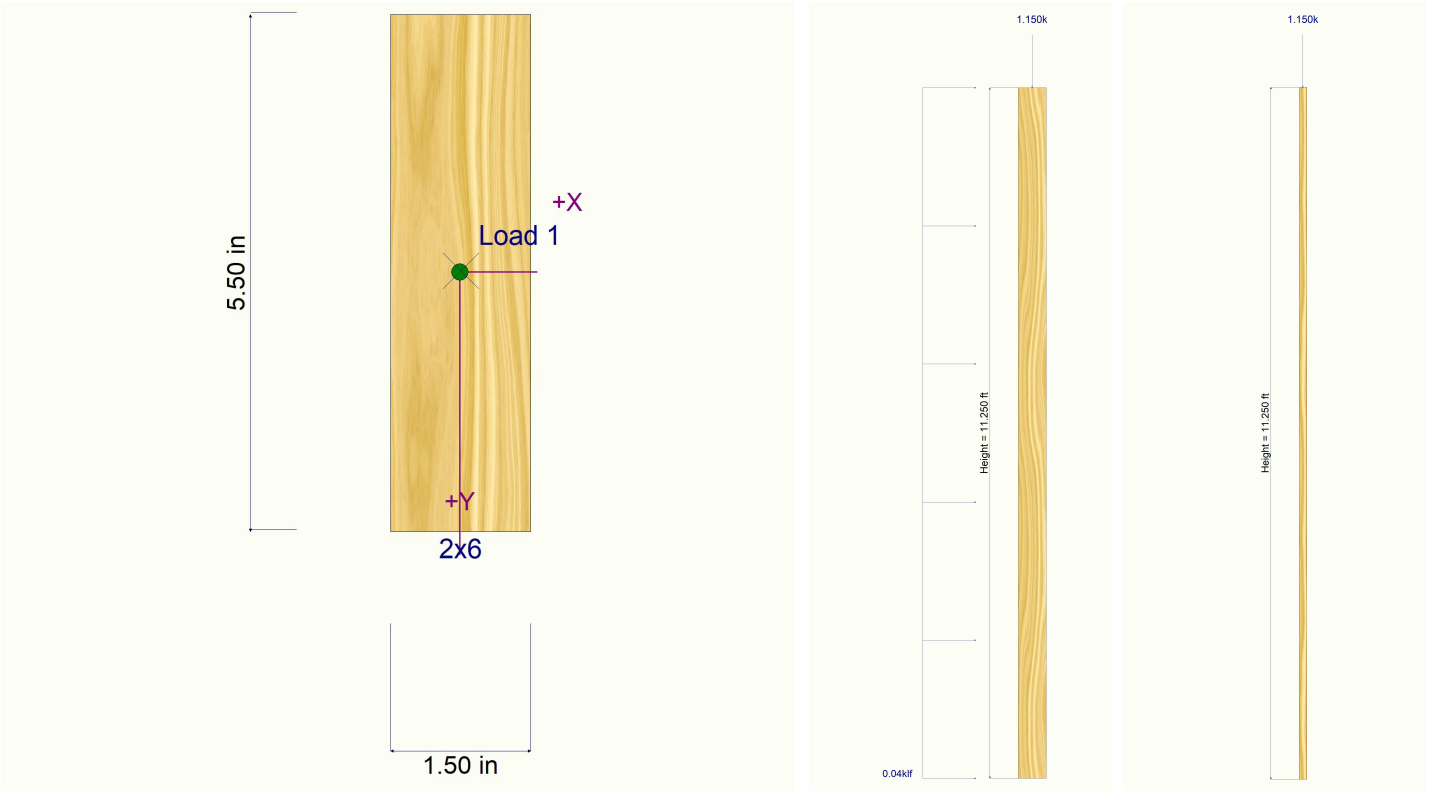
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
D Only						0.770					
+D+L				0.225	0.225	0.770					
+D+Lr						1.170					
+D+0.750Lr+0.750L				0.169	0.169	1.070					
+D+0.750L				0.169	0.169	0.770					
+0.60D						0.462					
Lr Only						0.400					
L Only				0.225	0.225						

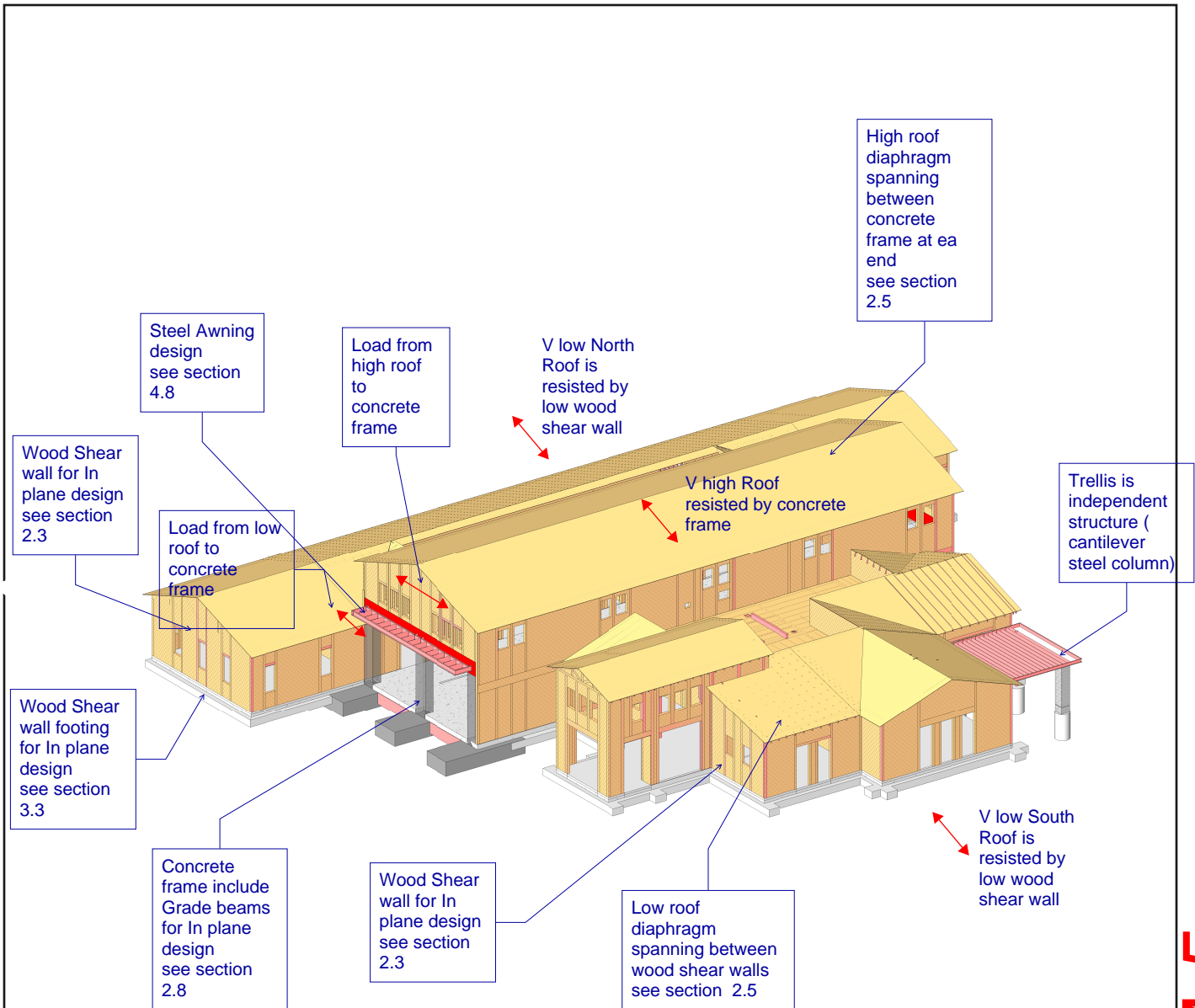
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000ft	0.412 in	5.663 ft
+D+Lr	0.0000 in	0.000ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L	0.0000 in	0.000ft	0.309 in	5.663 ft
+D+0.750L	0.0000 in	0.000ft	0.309 in	5.663 ft
+0.60D	0.0000 in	0.000ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000ft	0.412 in	5.663 ft

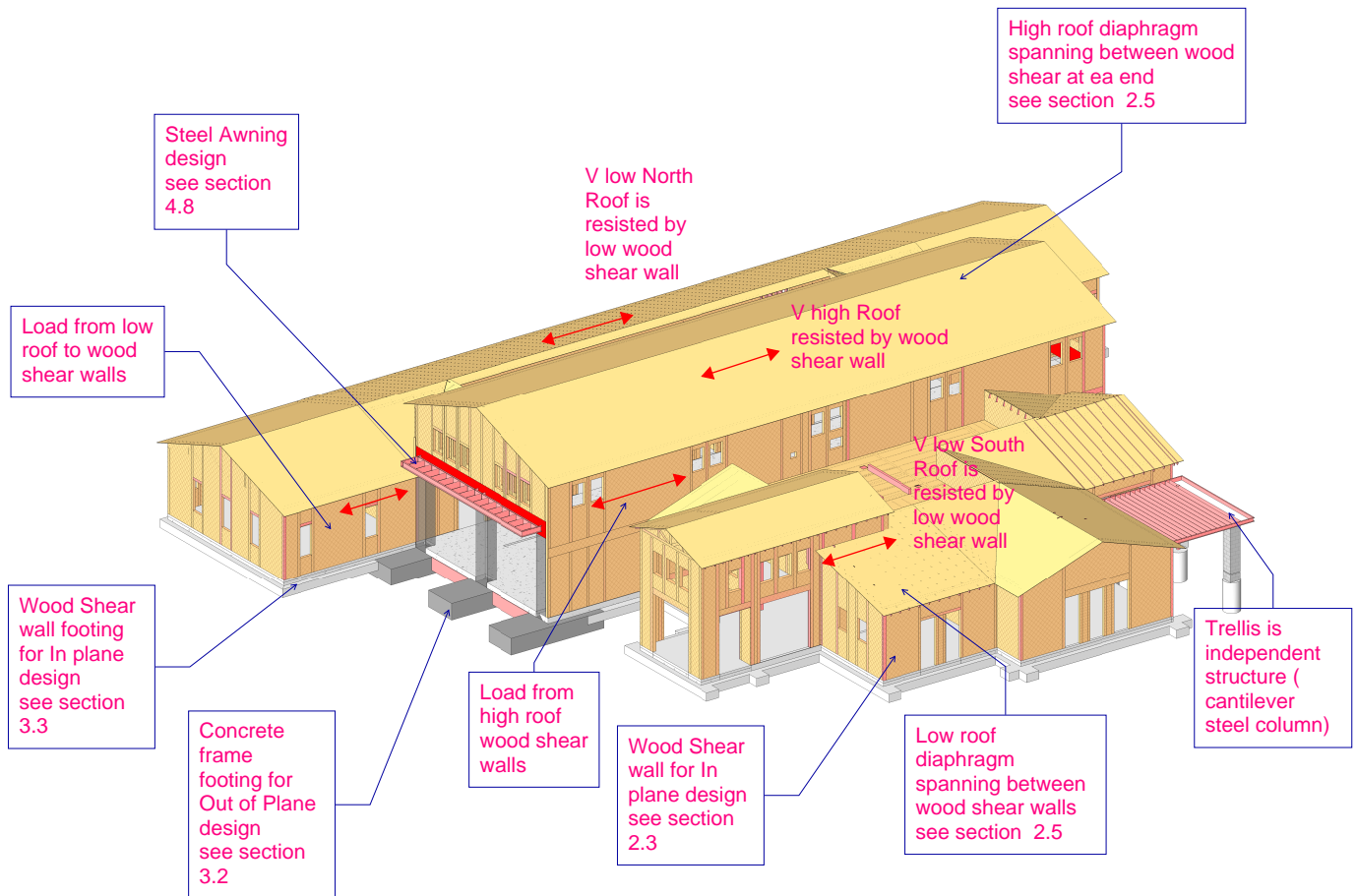
Sketches



2 LATERAL DESIGN



LATERAL LOAD PATH: NORTH SOUTH DIRECTION



LATERAL LOAD PATH : EAST-WEST DIRECTION

2.1 SEISMIC LOADING CRITERIA

CODE LEVEL EARTHQUAKE DESIGN DATA:

SITE COORDINATES	= 34.4143°N, 118.6020°W
MAPPED SPECTRAL RESPONSE ACCELERATION, SS	= 2.263g
MAPPED SPECTRAL RESPONSE ACCELERATION, S1	= 0.817g
SITE CLASS	= C
DESIGN SPECTRAL RESPONSE COEFFICIENT, SDS	= 1.601g
DESIGN SPECTRAL RESPONSE COEFFICIENT, SD1	= 0.710g
RISK CATEGORY	= IV
IMPORTANCE FACTOR, I _e	= 1.5
SEISMIC DESIGN CATEGORY	= F
SEISMIC RESPONSE COEFFICIENT, CS	= 0.369
ANALYTICAL PROCEDURE:	=EQUIVALENT LATERAL FORCE PROCEDURE
SEISMIC-FORCE RESISTING SYSTEM	= LIGHT FRAME BEARING WOOD PANEL SHEAR WALLS
RESPONSE MODIFICATION FACTOR, R	= 6 1/2
DEFLECTION AMPLIFICATION FACTOR, C _d	= 4
OVERSTRENGTH FACTOR, Ω _o	= 3
DESIGN BASE SHEAR	
MAIN BUILDING	= 154 KIP
RESERVE APPARATUS	= 26.5 KIP

BUILDING DESIGN:

X-X DIRECTION (E-W DIRECTION) LIGHT FRAMING BEARING WOOD SW R= 6.5

Y-Y DIRECTION (N-S DIRECTION) LIGHT WOOD SW R= 6.5, SCMF R= 8.0

DESIGNN R=6.5

CS= 0.369

**TABLE 4.5-2: Design Acceleration Parameters based on ASCE 7-16 Sections 21.4 and 21.5
 (Latitude: 34.4143 Longitude: -118.6020)**

ACCELERATION PARAMETER	VALUE (g)
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S	2.263
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1	0.817
MCE_R Spectral Response Acceleration at Short Periods, S_{MS}	2.401
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1}	1.066
Design Spectral Response Acceleration at Short Periods, S_{DS}	1.601
Design Spectral Response Acceleration at 1-second Period, S_{D1}	0.710
MCE_G peak ground acceleration adjusted for site class effects, PGA_M	0.991

ENGEO, INC. PROJECT NO. 6538.100312, DATED MAY 14, 2025

2.2 DESIGN BASE SHEAR

BASE SHEAR

2022 CBC / ASCE 7-16 Equivalent Lateral Force Procedure

Project: 0
SBI Job No.: 0

Building: FS #46
LFRS: Wood Shear Wall
Direction: X

Source of Input for Seismic Ground Motion Values: Code-Based (Mapped)

Building Data

Risk Category = IV Table 1604.5, 2019 CBC Importance Factor, I_e = 1.50 Table 1.5-2

Seismic Ground Motion Values

Section 11.4

S_s = 2.263 From geotech or Figs.22-1 to 8
 S_1 = 0.817 From geotech or Figs.22-1 to 8
Site Class = C From geotech or Table 20.3-1
 T_L = 8 sec Figs.22-14 to 17

F_a = 1.20 Table 11.4-1
 F_v = 1.40 Table 11.4-2
 $S_{MS} = F_a S_s$ = 2.716 Eq. 11.4-1 **2.401**
 $S_{M1} = F_v S_1$ = 1.144 Eq. 11.4-2 **1.066**
 $S_{DS} = (2/3) S_{MS}$ = ~~1.810~~ Eq. 11.4-3 **1.601**
 $S_{D1} = (2/3) S_{M1}$ = 0.763 Eq. 11.4-4 **0.710**

Seismic Design Category

F Tables 11.6-1 & 2

No

Building Period

Section 12.8.2

C_t = 0.02 Table 12.8-2
 x = 0.75 Table 12.8-2
 h_n = 30.00 ft Height of Building
 T_b = 0.000 sec From Analysis
(Input zero to use T_a)

$T_a = C_t h_n^x$ = 0.256 sec Eq. 12.8-7
 C_u = 1.40 Table 12.8-1
 $T_{a, max} = C_u T_a$ = 0.359 sec Section. 12.8.2

Period = 0.256 sec <-- used for design
0.256 sec <-- used for drift
Section. 12.8.6.2

Base Shear

Section 12.8

W = 136 kips Total Structure Weight
 R = 6.5 Table 12.2-1
 C_d = 4 Table 12.2-1

For Design Only

C_s = $S_{DS} / (R/I_e)$ = ~~0.418~~ **0.369** Eq. 12.8-2
 $C_{s, max}$ = $SD1 / [T (R/I_e)]$ = ~~0.686~~ **0.64** Eq. 12.8-3, for $T \leq T_L$
 $C_{s, max}$ = $S_{D1} T_L / [T^2 (R/I_e)]$ = N/A Eq. 12.8-4, for $T > T_L$
 $C_{s, min}$ = $\max\{0.01, 0.044 S_{DS} I_e\}$ = N/A Eq. 12.8-5, if $S_1 < 0.6g$
 $C_{s, min}$ = $\max\{0.01, 0.044 S_{DS} I_e, 0.5 S_1 / (R/I_e)\}$ = ~~0.119~~ **0.106** Eq. 12.8-5 and 12.8-6, if $S_1 \geq 0.6g$

Use, C_s = ~~0.418~~ **0.369**

$$0.044 * 1.601 * 1.5 = 0.106$$

$$0.5 * 0.817 / (6.5 / 1.5) = 0.09 \quad V_{design} = C_s W = 50 \text{ kips} \quad \text{Eq. 12.8-1}$$

For Drift Only

C_s = 0.418
 $C_{s, max}$ = 0.686
 $C_{s, max}$ = N/A
 $C_{s, min}$ = N/A
 $C_{s, min}$ = 0.119

Use, C_s =

$V_{drift} = C_s W$ = kips
Allowable Drift = 0.020 h_{sx}
Table 12.12-1

* Note: All references are from ASCE 7-16 unless noted otherwise.

Wind Forces - MWFRS - ASCE 7-05 Method 1 (Section 6.4)

Code: ASCE 7-05

Wind Parameters:

$h = 26.0$ ft

$\theta = 18.0^\circ$

$V = 105.0$ mi/h (Fig. 6-1)

$I = 1.50$ (6.5.5)

$K_{zt} = 1.00$ (6.5.7)

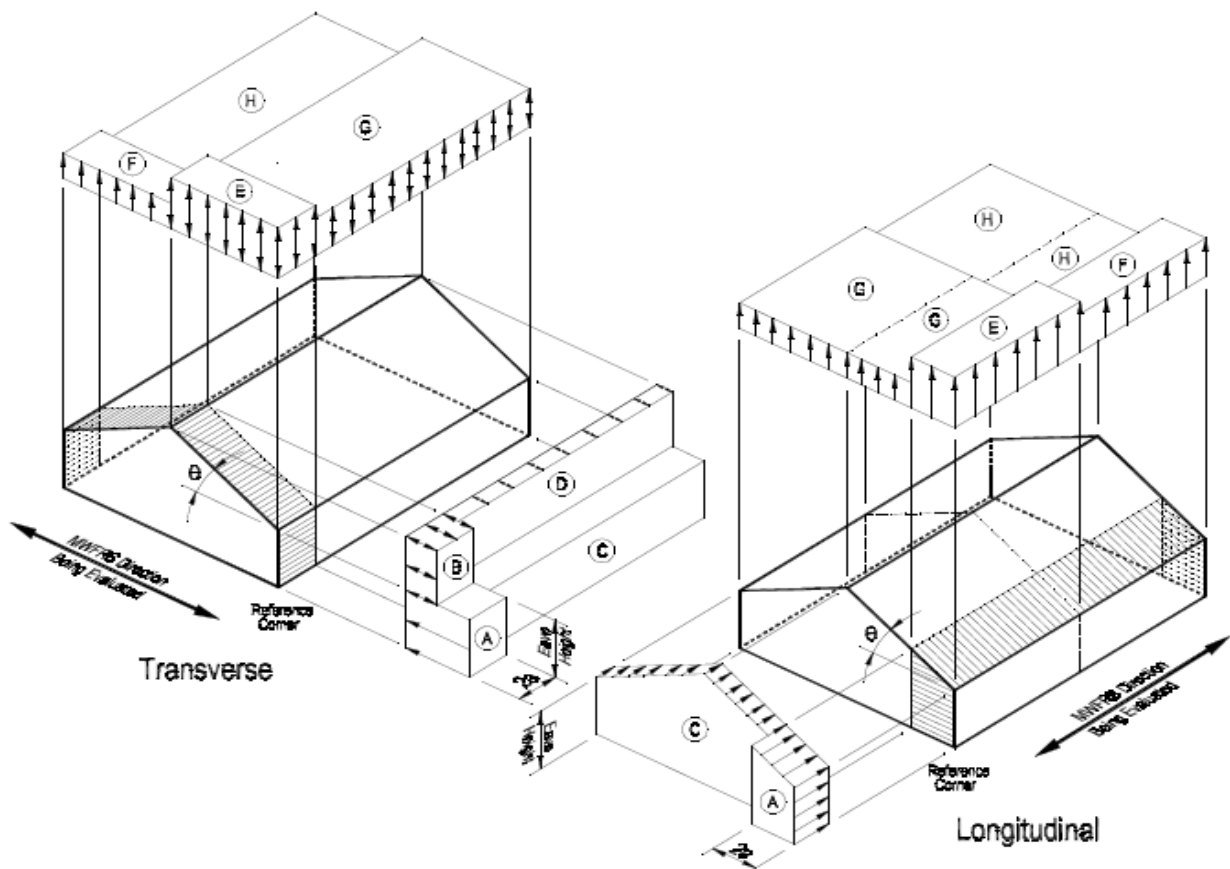
Surface Rough. = C (6.5.6.2)

Exposure: C (6.5.6.3)

$\lambda = 1.36$ (Fig. 6-2)

	Load Case	Zone									
		A	B	C	D	E	F	G	H	EOH	GOH
p _{s30} (psf)	1	24.3	-8.4	16.1	-3.5	-21.1	-14.7	-14.7	-11.1	-29.4	-23.0

	Load Case	Zone									
		A	B	C	D	E	F	G	H	EOH	GOH
p _s (psf)	1	49.57	-17.14	32.84	-7.14	-43.04	-29.99	-29.99	-22.64	-59.98	-46.92



- 1) The load patterns shown shall be applied to each corner of the building in turn as the reference corner.
- 2) For design of the longitudinal MWFRS use $\theta=0^\circ$, and locate the zone E/F, G/H boundary at the mid-length of the building.
- 3) Load Cases 1 and 2 must be checked for $25^\circ < \theta \leq 45^\circ$.
- 4) Plus and minus signs signify pressures acting towards and away from the projected surfaces, respectively.
- 5) The total horizontal load shall not be less than that determined by assuming $p_s=0$ in zones B & D
- 6) Where zone E or G falls on a roof overhang on the windward side of the building, use EOH and GOH for the pressure on the horizontal projections of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied.
- 7) $a = 10\%$ of the least horiz. dimension or $0.4h$, whichever is smaller, but not less than 4% of least horiz. dimension or 3 ft.

Wind Load in Short Direction (N/S)

$V_{wind} = (162 \text{ ft}) * 6 \text{ ft} * 33 \text{ psf} + 112 \text{ ft} * (6 \text{ ft}) * 33 \text{ psf} = 32 \text{ kip} + 22 \text{ kip} = 54 \text{ kip} < V_{seismic} = V_{wood} + V_{Concrete}$
Frame = $162 \text{ kip} + (23 \text{ kip}) * 2 = 208 \text{ kip}$ SEISMIC GOVERNED

From page # 96

From page # 137

STRUCTURAL DESIGN

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

RISK CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: <ul style="list-style-type: none"> • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Risk Categories I, III and IV.
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: <ul style="list-style-type: none"> • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures containing one or more public assembly spaces, each having an occupant load greater than 300 and a cumulative occupant load of these public assembly spaces of greater than 2,500. • Buildings and other structures containing Group E or Group I-4 occupancies or combination thereof, with an occupant load greater than 250. • Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500. • Group I-2, Condition 1 occupancies with 50 or more care recipients. • Group I-2, Condition 2 occupancies not having emergency surgery or emergency treatment facilities. • [OSHPD 2] Skilled nursing facilities, intermediate care facilities, Group I-2 occupancy with 50 or more care recipients. • [OSHPD 5] Acute psychiatric hospitals, Group I-2 occupancy with 50 or more care recipients. • Group I-3 occupancies. • Any other occupancy with an occupant load greater than 5,000.^a • Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV. • Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that: <ul style="list-style-type: none"> • Exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the <i>California Fire Code</i>; and • Are sufficient to pose a threat to the public if released.^b
IV	Buildings and other structures designated as essential facilities, including but not limited to: <ul style="list-style-type: none"> • Group I-2, Condition 2 occupancies having emergency surgery or emergency treatment facilities. • Ambulatory care facilities having emergency surgery or emergency treatment facilities. • Fire, rescue, ambulance and police stations and emergency vehicle garages • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communications and operations centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures. • Buildings and other structures containing quantities of highly toxic materials that: <ul style="list-style-type: none"> • Exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the <i>California Fire Code</i>; and • Are sufficient to pose a threat to the public if released.^b • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water storage facilities and pump structures required to maintain water pressure for fire suppression.

- a. For purposes of occupant load calculation, occupancies required by Table 1004.5 to use gross floor area calculations shall be permitted to use net floor areas to determine the total occupant load.
- b. Where approved by the building official, the classification of buildings and other structures as Risk Category III or IV based on their quantities of toxic, highly toxic or explosive materials is permitted to be reduced to Risk Category II, provided that it can be demonstrated by a hazard assessment in accordance with Section 1.5.3 of ASCE 7 that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public.

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n (ft) Limits ^d				
					Seismic Design Category				
					B	C	D ^e	E ^e	F ^f
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls ^{g,h}	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls ^g	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls ^g	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls ^g	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls ^g	14.2	4	2½	4	NL	NL	40 ⁱ	40 ⁱ	40 ⁱ
6. Ordinary precast shear walls ^g	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35 ^j	35 ^j	NP ⁱ
4. Special reinforced concrete shear walls ^{g,h}	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls ^g	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls ^g	14.2 and 14.2.2.7	2	2½	2	NL	NP	NP	NP	NP
7. Ordinary plain concrete shear walls ^g	14.2	1½	2½	1½	NL	NP	NP	NP	NP
8. Intermediate precast shear walls ^g	14.2	5	2½	4½	NL	NL	40 ⁱ	40 ⁱ	40 ⁱ
9. Ordinary precast shear walls ^g	14.2	4	2½	4	NL	NP	NP	NP	NP
10. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	160	160	100
11. Steel and concrete composite special concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
12. Steel and concrete composite ordinary braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
13. Steel and concrete composite plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
14. Steel and concrete composite special shear walls	14.3	6	2½	5	NL	NL	160	160	100
15. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½	NL	NL	NP	NP	NP
16. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
17. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP

X DIRECTION (EAST
-WEST)

3. The structure has a fundamental period, T , that does not exceed 0.5 s, as determined using Section 12.8.2;
4. The structure meets the requirements necessary for the redundancy factor, ρ , to be permitted to be taken as 1.0, in accordance with Section 12.3.4.2;
5. The site soil properties are not classified as Site Class E or F, as defined in Section 11.4.3; and
6. The structure is classified as Risk Category I or II, as defined in Section 1.5.1.

12.8.2 Period Determination. The fundamental period of the structure, T , in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period, T , shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a , determined in accordance with Section 12.8.2.1. As an alternative to performing an analysis to determine the fundamental period, T , it is permitted to use the approximate building period, T_a , calculated in accordance with Section 12.8.2.1, directly.

12.8.2.1 Approximate Fundamental Period. The approximate fundamental period (T_a), in seconds, shall be determined from the following equation:

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

where h_n is the structural height as defined in Section 11.2 and the coefficients C_t and x are determined from Table 12.8-2.

Alternatively, it is permitted to determine the approximate fundamental period (T_a), in seconds, from the following equation

for structures not exceeding 12 stories above the base as defined in Section 11.2 where the seismic force-resisting system consists entirely of concrete or steel moment-resisting frames and the average story height is at least 10 ft (3 m):

$$T_a = 0.1 N \quad (12.8-8)$$

where N = number of stories above the base.

The approximate fundamental period, T_a , in seconds, for masonry or concrete shear wall structures not exceeding 120 ft (36.6 m) in height is permitted to be determined from Eq. (12.8-9) as follows:

$$T_a = \frac{C_q}{\sqrt{C_w}} h_n \quad (12.8-9)$$

where

$C_q = 0.0019$ ft (0.00058 m)

C_w is calculated from Eq. (12.8-10) as follows:

$$C_w = \frac{100}{A_B} \sum_{i=1}^x \frac{A_i}{\left[1 + 0.83 \left(\frac{h_n}{D_i}\right)^2\right]} \quad (12.8-10)$$

where

A_B = area of base of structure [ft² (m²)];

A_i = web area of shear wall i [ft² (m²)];

D_i = length of shear wall i [ft (m)]; and

x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

12.8.3 Vertical Distribution of Seismic Forces. The lateral seismic force (F_x) (kip or kN) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \quad (12.8-11)$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (12.8-12)$$

where

C_{vx} = vertical distribution factor;

V = total design lateral force or shear at the base of the structure [kip (kN)];

w_i and w_x = portion of the total effective seismic weight of the structure (W) located or assigned to level i or x ;

h_i and h_x = height [ft (m)] from the base to level i or x ; and

k = an exponent related to the structure period as follows:

- for structures that have a period of 0.5 s or less, $k = 1$;
- for structures that have a period of 2.5 s or more, $k = 2$; and
- for structures that have a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

12.8.4 Horizontal Distribution of Forces. The seismic design story shear in any story (V_x) [kip (kN)] shall be determined from the following equation:

$$V_x = \sum_{i=x}^n F_i \quad (12.8-13)$$

Table 12.8-1 Coefficient for Upper Limit on Calculated Period

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

Table 12.8-2 Values of Approximate Period Parameters C_t and x

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 (0.0724) ^a	0.8
Concrete moment-resisting frames	0.016 (0.0466) ^a	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731) ^a	0.75
Steel buckling-restrained braced frames	0.03 (0.0731) ^a	0.75
All other structural systems	0.02 (0.0488) ^a	0.75

^aMetric equivalents are shown in parentheses.

3. The structure has a fundamental period, T , that does not exceed 0.5 s, as determined using Section 12.8.2;
4. The structure meets the requirements necessary for the redundancy factor, ρ , to be permitted to be taken as 1.0, in accordance with Section 12.3.4.2;
5. The site soil properties are not classified as Site Class E or F, as defined in Section 11.4.3; and
6. The structure is classified as Risk Category I or II, as defined in Section 1.5.1.

12.8.2 Period Determination. The fundamental period of the structure, T , in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period, T , shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a , determined in accordance with Section 12.8.2.1. As an alternative to performing an analysis to determine the fundamental period, T , it is permitted to use the approximate building period, T_a , calculated in accordance with Section 12.8.2.1, directly.

12.8.2.1 Approximate Fundamental Period. The approximate fundamental period (T_a), in seconds, shall be determined from the following equation:

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

where h_n is the structural height as defined in Section 11.2 and the coefficients C_t and x are determined from Table 12.8-2.

Alternatively, it is permitted to determine the approximate fundamental period (T_a), in seconds, from the following equation

for structures not exceeding 12 stories above the base as defined in Section 11.2 where the seismic force-resisting system consists entirely of concrete or steel moment-resisting frames and the average story height is at least 10 ft (3 m):

$$T_a = 0.1 N \quad (12.8-8)$$

where N = number of stories above the base.

The approximate fundamental period, T_a , in seconds, for masonry or concrete shear wall structures not exceeding 120 ft (36.6 m) in height is permitted to be determined from Eq. (12.8-9) as follows:

$$T_a = \frac{C_q}{\sqrt{C_w}} h_n \quad (12.8-9)$$

where

$C_q = 0.0019$ ft (0.00058 m)

C_w is calculated from Eq. (12.8-10) as follows:

$$C_w = \frac{100}{A_B} \sum_{i=1}^x \frac{A_i}{\left[1 + 0.83 \left(\frac{h_n}{D_i}\right)^2\right]} \quad (12.8-10)$$

where

A_B = area of base of structure [ft² (m²)];

A_i = web area of shear wall i [ft² (m²)];

D_i = length of shear wall i [ft (m)]; and

x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

12.8.3 Vertical Distribution of Seismic Forces. The lateral seismic force (F_x) (kip or kN) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \quad (12.8-11)$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (12.8-12)$$

where

C_{vx} = vertical distribution factor;

V = total design lateral force or shear at the base of the structure [kip (kN)];

w_i and w_x = portion of the total effective seismic weight of the structure (W) located or assigned to level i or x ;

h_i and h_x = height [ft (m)] from the base to level i or x ; and

k = an exponent related to the structure period as follows:

- for structures that have a period of 0.5 s or less, $k = 1$;
- for structures that have a period of 2.5 s or more, $k = 2$; and
- for structures that have a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

12.8.4 Horizontal Distribution of Forces. The seismic design story shear in any story (V_x) [kip (kN)] shall be determined from the following equation:

$$V_x = \sum_{i=x}^n F_i \quad (12.8-13)$$

Table 12.8-1 Coefficient for Upper Limit on Calculated Period

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

Table 12.8-2 Values of Approximate Period Parameters C_t and x

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 (0.0724) ^a	0.8
Concrete moment-resisting frames	0.016 (0.0466) ^a	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731) ^a	0.75
Steel buckling-restrained braced frames	0.03 (0.0731) ^a	0.75
All other structural systems	0.02 (0.0488) ^a	0.75

^aMetric equivalents are shown in parentheses.

BUILDING SEISMIC MASS CALCULATION

WALL SEISMIC WEIGHT CALCULATION

Exterior Wall (EAST/WEST) X-X Direction:

Low

High

Low

High

East + West = (9732 lb+ 3590) + (1107 lb + 3590 lb)= (9732 lb+ 1107 lb) + (3590 lb+3590 lb)= 10839 + 7180 lb
W E/W= 18019 lb

Exterior Wall (South/North) Y-Y Direction:

Low

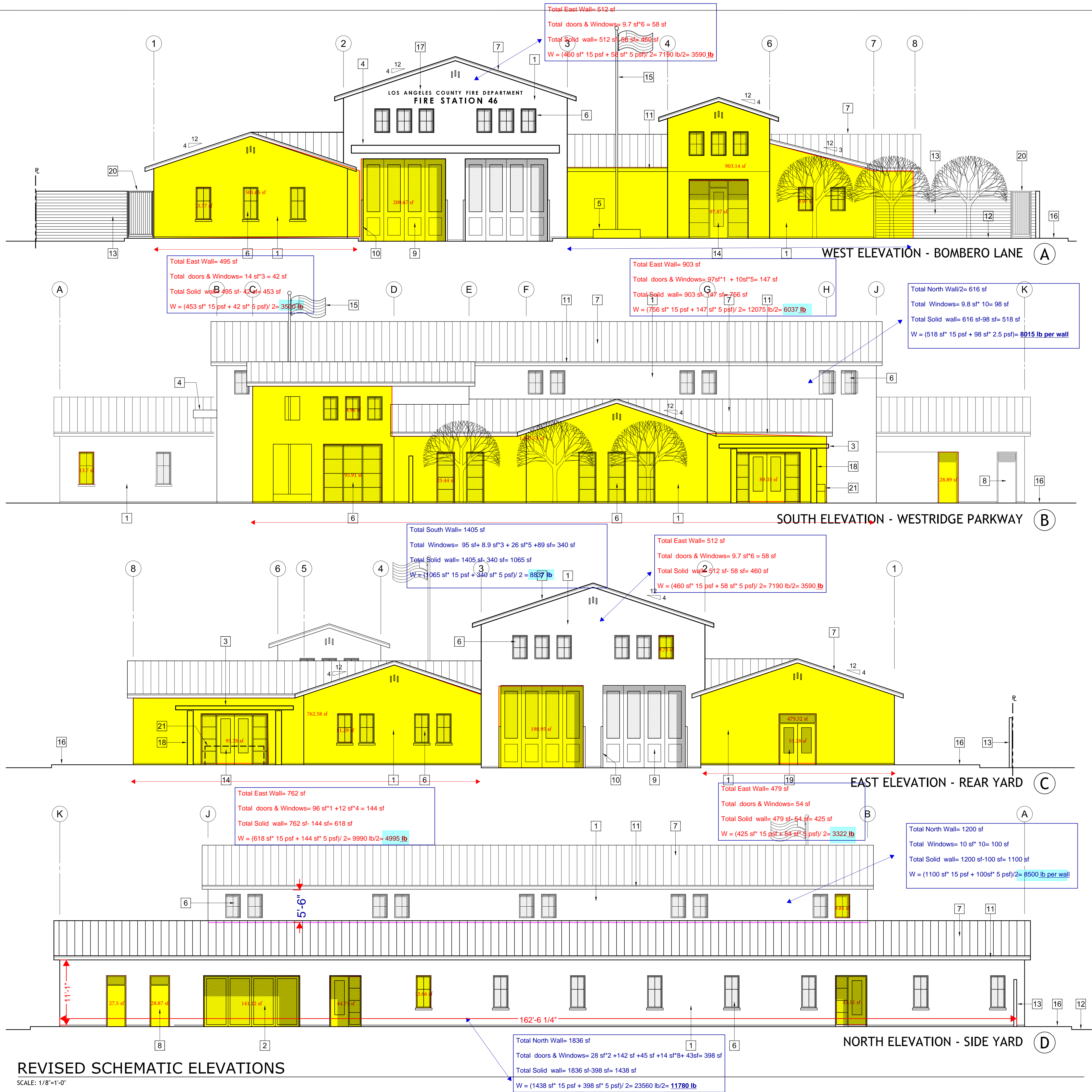
High

Low

High

South + North = (14842 lb+ 9590) + (11780 lb + 9590 lb)= (14842 lb+ 11780 lb) + (9590 lb+9590 lb)= 26622 +19180 lb
W N/S= 45802 lb

Total Building Walls = WE/W + W N/S= 18 kip + 46 kip= **51 kip**



NOTES

1. CEMENT PLASTER w/ SAND FINISH, TYP.
2. RETRACTABLE GLASS DOORS.
3. PATIO CANOPY w/ ROUND CONCRETE COLUMNS.
4. STEEL BEAM OVER APPARATUS DOORS.
5. CONCRETE FLAGPOLE BASE.
6. TYP. ALUMINUM WINDOW w/ TINTED DOUBLE GLAZING.
7. TYP. STANDING SEAM METAL ROOF.
8. STEEL DOOR w/ TRANSOM VENT.
9. TYP. STEEL BI-FOLD APPARATUS ROOM DOORS WITH PRIMED & PAINTED STEEL FRAMES & TINTED DOUBLE GLAZING.
10. TYP. PAINTED / GALV. STEEL ANGLE CORNER GUARD (8 LOCATIONS).
11. METAL FASCIA / GUTTER.
12. FINISH GRADE.
13. CMU WALL.
14. ALUMINUM STOREFRONT DOORS & WINDOWS.
15. FLAGPOLE.
16. CONCRETE PAVING/SIDEWALK.
17. BUILDING SIGNAGE - TYP. CAST ALUMINUM LETTERS.
18. TYP. ROUND CONCRETE COLUMN.
19. LOUVERED DOORS w/ TRANSOM VENT.
20. STEEL GATE.
21. BBO.

WILLIAM LOYD JONES
ARCHITECT

9415 culver boulevard
culver city, california
90230

TEL 310 392 3995

REVISED SCHEMATIC ELEVATIONS

FIRE STATION 46
COUNTY OF LOS ANGELES FIRE DEPARTMENT
26720 BOMBERO LANE
VALENCIA, CALIFORNIA

FIRE POINT

Issue

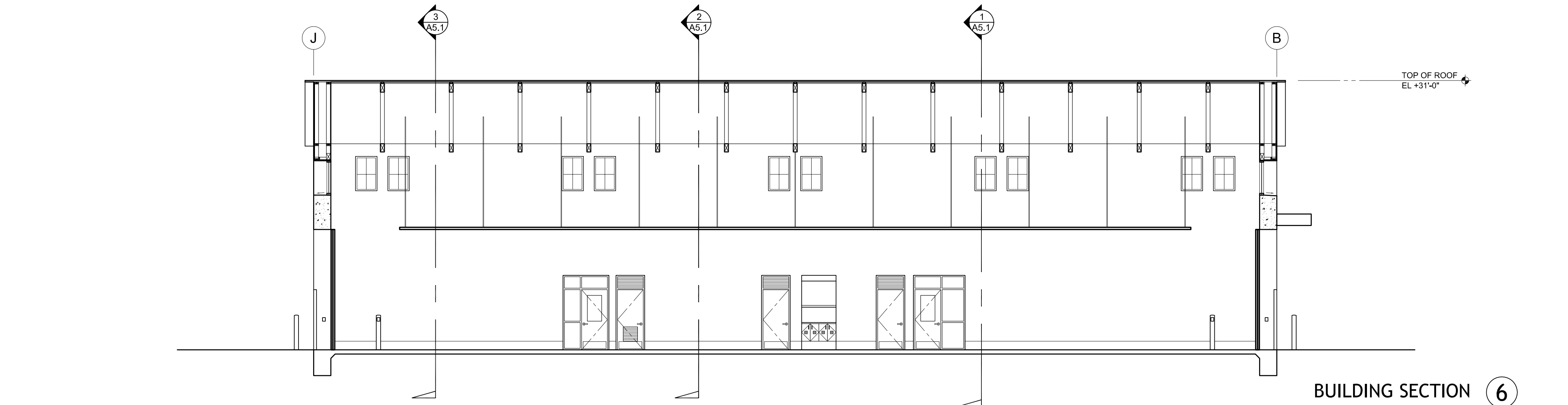
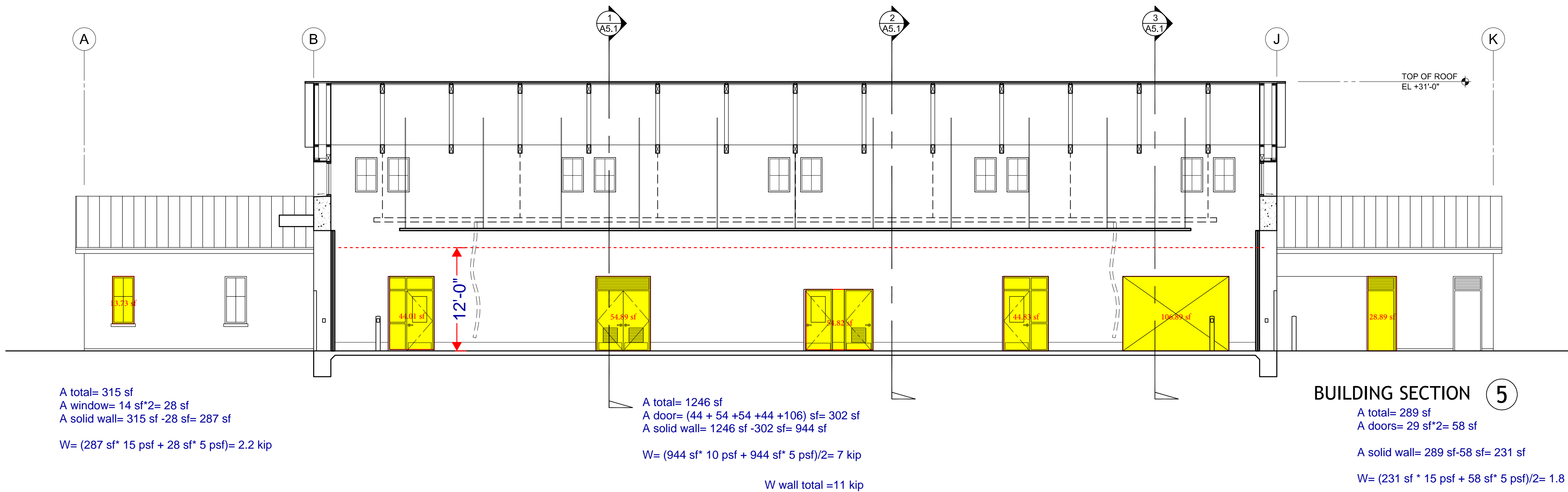
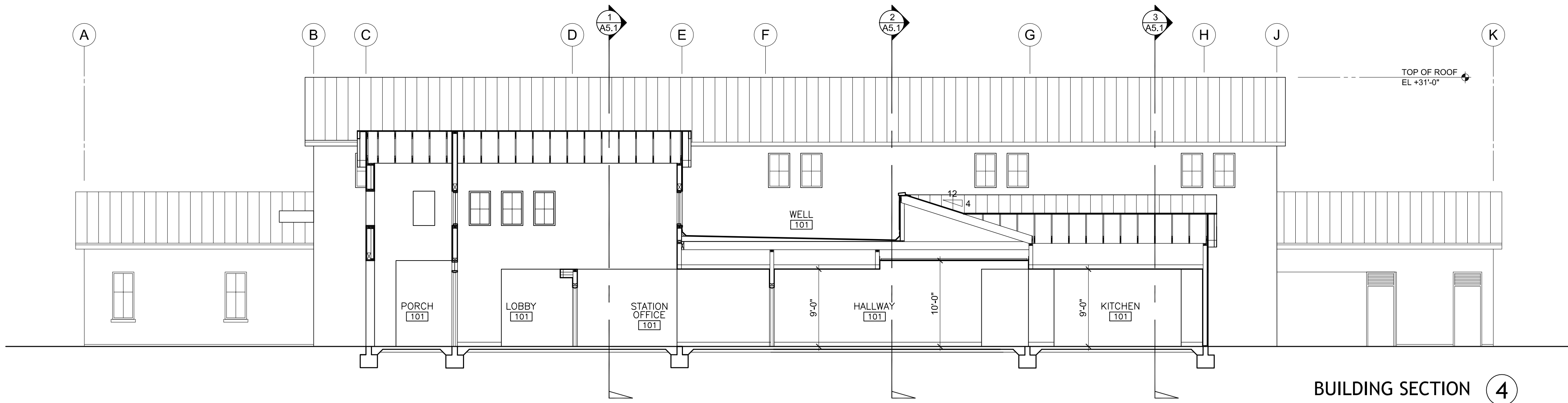
SCHEMATIC ELEVATIONS	06.DEC.24
REVISION	09.DEC.24
REVISION	07.JAN.25
REVISION	21.JAN.25
REVISION	30.JAN.25
REVISION	06.FEB.25
REVISION	24.FEB.25
REVISION	03.MAR.25
LACoFD APPROVED	11.MAR.25
REVISION	19.MAR.25

THE ABOVE DRAWINGS AND SPECIFICATIONS AND THEIR DESIGN AND ARRANGEMENTS REPRESENTED THEREBY ARE AND SHALL REMAIN THE PROPERTY OF THE ARCHITECT AND NO PART THEREOF SHALL BE COPIED, REPRODUCED, OR OTHERWISE USED IN ANY MANNER WITHOUT THE WRITTEN CONSENT OF THE ARCHITECT. ANY SUCH VIOLATION SHALL BE CONSIDERED A BREACH OF THE ARCHITECT'S PROFESSIONAL ETHICS AND SHALL BE SUBJECT TO THE DISCIPLINARY ACTION OF THE ARCHITECT'S BOARD. THE ARCHITECT'S PROFESSIONAL ETHICS AND SHALL BE SUBJECT TO THE DISCIPLINARY ACTION OF THE ARCHITECT'S BOARD. THE ARCHITECT'S PROFESSIONAL ETHICS AND SHALL BE SUBJECT TO THE DISCIPLINARY ACTION OF THE ARCHITECT'S BOARD.

Date	27.NOV.24
Drawn	KC
Checked	WLJ
Scale	1/8" = 1'-0"
Job. No.	

A4.1

ADDENDUM 5



BUILDING SECTIONS
SCALE: 1/8"=1'-0"

NOTES

**WILLIAM LOYD JONES
ARCHITECT**

9415 culver boulevard
culver city, california
9 0 2 3 2

TEL 310 392 3995

BUILDING SECTIONS

FIRE STATION 46
COUNTY OF LOS ANGELES FIRE DEPARTMENT
26720 BOMBERO LANE
VALENCIA, CALIFORNIA

F I R E P O I N T

Issue

DESIGN	18.DEC.23
REVISION	17.APR.24
REVISION	22.APR.24
REVISION	14.NOV.24
REVISION	09.DEC.24
REVISION	06.FEB.25
REVISION	24.FEB.25
REVISION	03.MAR.25
LACoFD APPROVED	11.MAR.25
REVISION	19.MAR.25

THE ABOVE DRAWINGS AND SPECIFICATIONS AND THEIR DESIGN AND ARRANGEMENTS REPRESENTED THEREBY ARE AND SHALL REMAIN THE PROPERTY OF THE ARCHITECT AND NO PART THEREOF SHALL BE COPIED, REPRODUCED, OR IN ANY MANNER USED FOR ANY OTHER PROJECT OTHER THAN THE SPECIFIC PROJECT FOR WHICH THEY HAVE BEEN PREPARED, AND NO PART THEREOF SHALL BE LOANED, RENTED, REPRODUCED, COPIED, OR OTHERWISE USED IN ANY MANNER WITHOUT THE WRITTEN CONSENT OF THE ARCHITECT. VISUAL CONFERENCE WITH THESE DRAWINGS OR SPECIFICATIONS SHALL CONSTITUTE CONCLUSIVE EVIDENCE OF ACCEPTANCE OF THESE DRAWINGS.

WRITTEN DIMENSIONS ON THESE DRAWINGS SHALL HAVE PRECEDENCE OVER SCALE DIMENSIONS. CONTRACTORS SHALL VERIFY AND BE RESPONSIBLE FOR ALL DIMENSIONS AND CONDITIONS SHOWN ON THESE DRAWINGS AND CONDITIONS SHOWN IN THESE DRAWINGS. SHOP DETAILS MUST BE SUBMITTED TO THIS OFFICE FOR APPROVAL BEFORE PROCEEDING WITH FABRICATION.

Copyright 2003

Date	17.APR.24
Drawn	KC
Checked	WLJ
Scale	1/8" = 1'-0"
Job. No.	

A5.2

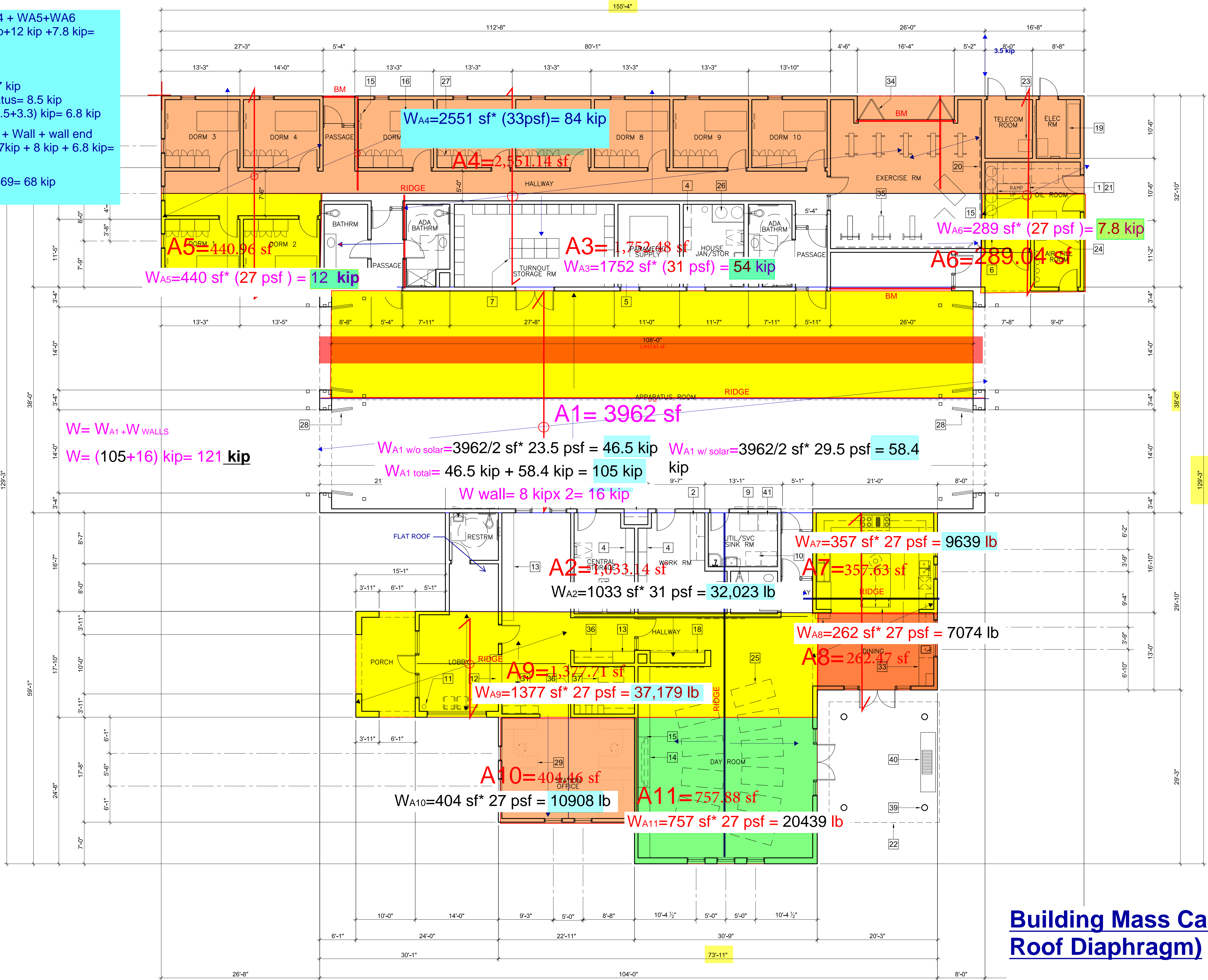
ADDENDUM 5

Wr= WA3+WA4 + WA5+WA6
= 54 kip+ 84 kip+12 kip +7.8 kip=
= 158 kip

Wall:
North wall: 11.7 kip
Wall at Apparatus= 8.5 kip
Wall at end= (3.5+3.3) kip= 6.8 kip

Wtotal= W roof + Wall + wall end
= 158 kip + 11.7kip + 8 kip + 6.8 kip=
185 kip

V= 185 kip* 0.369= 68 kip



W Roof= WA2+WA7 +WA9+WA8 +WA10+WA11
= 32 kip+ 9.6 kip +37 kip+ 7 kip +10 kip +20 kip= 116kip
W walls= 8.8 kip + 6.0 kip + 5 kip= 20 kip
W total = W roof + Wwalls = 116 kip + 20 kip= 136 kip
V= 0.369 * 136 kip= 50 kip

REVISED SCHEMATIC FLOOR PLAN - 13,150 SF
SCALE: 1/8"=1'-0"

Building Mass Calculation (Walls & Roof Diaphragm)

SOLAR AT HIGH ROOF & LOW ROOF

A2

NOTES

- 55 GALLON DRUMS (N.I.C.).
- WORK BENCH / SHELVES.
- FLAMMABLE STORAGE CABINET.
- TYP. SHELVES.
- PARAMEDIC CABINET.
- AIR COMPRESSOR.
- TURNOUT GEAR STORAGE SYSTEM 24"x20"x72".
- CHARGING CABINET.
- ICE MACHINE.
- WASHER/DRYER STACKABLE COMBO. (N.I.C.).
- FIRE SPRINKLER RISER.
- COUNTER - ADA ACCESSIBLE.
- CABINET.
- DAY ROOM CABINET.
- FLAT SCREEN TV (N.I.C.).
- TYPICAL CAPTAIN BED.
- TELECOM CABINET.
- DISPLAY CABINET.
- MAIN SWITCH BD. PANEL ON 4" CONC. PAD.
- MIRROR WALL.
- PRE-FABRICATED OIL PAN CONTAINMENT CORP IN-LINE 4 DRUMS POLY SPILL PALLET 3000 WITH RAMP.
- PATIO CANOPY.
- MDT COMMUNICATIONS UNIT (LACoFD).
- OXYGEN COMPRESSOR (N.I.C.).
- TYPICAL LOUNGE CHAIR (N.I.C.).
- 100 GALLON WATER HEATER.
- TYP. DORM LOCKER.
- AUTOMATIC STEEL BI-FOLD DOORS 14' x 14'.
- TYP. WORK STATION (N.I.C.).
- STAINLESS STEEL KITCHEN.
- FILES.
- CHAIR STORAGE.
- STAINLESS STEEL COFFEE AREA.
- RETRACTABLE GLASS DOORS.
- EXERCISE EQUIPMENT (N.I.C.).
- BASE CABINET w/ UPPERS.
- COPIER (N.I.C.).
- PHONE CABINET.
- TYP. ROUND CONC. COLUMN.
- BUILT-IN BBQ.
- VENDING MACHINE (N.I.C.).

- Apparatus Roof
- Flat Roof
- The rest colors Sloped Roof

WILLIAM LOYD JONES ARCHITECT

9415 culver boulevard
culver city, california
90232

TEL 310 392 3995

REVISED SCHEMATIC FLOOR PLAN

FIRE STATION 46
COUNTY OF LOS ANGELES FIRE DEPARTMENT
26720 BOMBERO LANE
VALENCIA, CALIFORNIA

FIVE POINT

Issue	
DESIGN	18.DEC.23
REVISION	17.APR.24
REVISION	22.APR.24
REVISION	14.NOV.24
REVISION	09.DEC.24
REVISION	06.FEB.25
REVISION	24.FEB.25
REVISION	03.MAR.25
LACoFD APPROVED	11.MAR.25

THE ABOVE DRAWINGS AND SPECIFICATIONS AND THEIR DESIGN AND ARRANGEMENTS REPRESENTED THEREBY ARE AND SHALL REMAIN THE PROPERTY OF THE ARCHITECT AND NO PART THEREOF SHALL BE COPIED, REPRODUCED, OR IN ANY MANNER USED FOR ANY OTHER PROJECT OTHER THAN THE SPECIFIC PROJECT FOR WHICH THEY HAVE BEEN PREPARED, AND WITHOUT THE WRITTEN CONSENT OF THE ARCHITECT. VISUAL CONSTRUCTION OF THESE DRAWINGS OR SPECIFICATIONS SHALL CONSTITUTE CONVICTION OF A CRIME UNDER PENAL CODE SECTION 148.5.

WRITTEN DIMENSIONS ON THESE DRAWINGS SHALL HAVE PRECEDENCE OVER SCALE DIMENSIONS. CONTRACTORS SHALL VERIFY AND BE RESPONSIBLE FOR ALL DIMENSIONS AND CONDITIONS OF THE EXISTING STRUCTURE. ANY VARIATIONS FROM THE DIMENSIONS AND CONDITIONS SHOWN ON THESE DRAWINGS, SHOP DETAILS MUST BE SUBMITTED TO THE OFFICE FOR APPROVAL BEFORE PROCEEDING WITH FABRICATION.

Copyright 2025

Date	17.APR.24
Drawn	KC
Checked	WLJ
Scale	1/16" = 1'-0"
Ob. No.	

ADDENDUM 5

Table 12.3-1 Horizontal Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	Torsional Irregularity: Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 16.3.4	D, E, and F B, C, D, E, and F C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	Extreme Torsional Irregularity: Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.3.4.2 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 16.3.4	E and F D D B, C, and D C and D C and D D B, C, and D
2.	Reentrant Corner Irregularity: Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	Diaphragm Discontinuity Irregularity: Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	Out-of-Plane Offset Irregularity: Out-of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.	12.3.3.3 12.3.3.4 12.7.3 Table 12.6-1 16.3.4	B, C, D, E, and F D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	Nonparallel System Irregularity: Nonparallel system irregularity is defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.	12.5.3 12.7.3 Table 12.6-1 16.3.4	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Table 12.3-2 Vertical Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	Stiffness-Soft Story Irregularity: Stiffness-soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	Stiffness-Extreme Soft Story Irregularity: Stiffness-extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
2.	Weight (Mass) Irregularity: Weight (mass) irregularity is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
3.	Vertical Geometric Irregularity: Vertical geometric irregularity is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
4.	In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity: In-plane discontinuity in vertical lateral force-resisting element irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on supporting structural elements.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
5a.	Discontinuity in Lateral Strength-Weak Story Irregularity: Discontinuity in lateral strength-weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
5b.	Discontinuity in Lateral Strength-Extreme Weak Story Irregularity: Discontinuity in lateral strength-extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

12.3.3.4 Increase in Forces Caused by Irregularities for Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E, or F and having a horizontal structural irregularity of Type 1a, 1b, 2, 3, or 4 in Table 12.3-1 or a vertical structural irregularity of Type 4 in Table 12.3-2, the design forces determined from Section 12.10.1.1 shall be increased 25% for the following elements of the seismic force-resisting system:

1. Connections of diaphragms to vertical elements and to collectors and
2. Collectors and their connections, including connections to vertical elements, of the seismic force-resisting system.

EXCEPTION: Forces calculated using the seismic load effects, including overstrength of Section 12.4.3, need not be increased.

2.3 PLYWOOD SHEAR WALL DESIGN

WOOD STRUCTURE (WS) PANEL SHEAR WALL SCHEDULE									
SHEAR WALL MARK	NO. OF SIDES OF SHTG	DESIGN SHEAR	STRUCT'L WSP THICKNESS	SILL PLATE SIZE (6)	STUD AND BLK'G SIZE AT ADJOINING PANEL EDGES (5)	ANCHORS AT SILL PLATE TO CONCRETE SLAB OR WALL	WS PANEL NAILING (3)	FASTENER AT (7) & (8) SILL PL TO RIM JOIST OR BLOCKING MIN PENETRATIONS SHOWN ARE INTO JOIST / OR BLOCK	A35/LTP-4 AT RIM JOIST OR BLOCKING TO DBL TOP PLATE
A	1	340 PLF	1/2"	2x6	2x6	5/8"Ø @ 32" OC AB W/ 1/4"x3"x3" WASHER PL	10d @ 6" OC EDGE NAILING (EN) 10d @ 12" OC FIELD NAILING (FN)	NAIL: 16d @ 4" OC LAG SCREWS: 1/4"Ø W/ 2" MIN PENET @ 8" SIMPSON SDS 1/4"x6" (2" MIN PENET) @ 12" OC	16" OC
B	1	510 PLF	1/2"	3x6	3x6	5/8"Ø @ 16" OC AB W/ 1/4"x3"x3" WASHER PL	10d @ 4" OC EDGE NAILING (EN) 10d @ 12" OC FIELD NAILING (FN)	LAG SCREWS: 1/4"Ø W/ 2" MIN PENET @ 4" OC SDS 1/4"x6" (2" MIN PENET) @ 8" OC	12" OC
C	1	665 PLF	1/2"	3x6	3x6	5/8"Ø @ 16" OC AB W/ 1/4"x3"x3" WASHER PL	10d @ 3" OC EDGE NAILING (EN) 10d @ 12" OC FIELD NAILING (FN)	LAG SCREWS: 1/4"Ø W/ 2" MIN PENET @ 4" OC SDS 1/4"x6" (2" MIN PENET) @ 6" OC	9" OC
D	1	870 PLF	1/2"	3x6	3x6	5/8"Ø @ 16" OC AB W/ 1/4"x3"x3" WASHER PL	10d @ 2" OC EDGE NAILING (EN) 10d @ 12" OC FIELD NAILING (FN)	LAG SCREWS: 1/4"Ø W/ 2" MIN PENET @ 3" OC SDS 1/4"x6" (2" MIN PENET) @ 5" OC	BOTH SIDES @ 14" OC (STAG)
E	2	1020 PLF	1/2"	3x6	3x6	5/8"Ø @ 16" OC AB W/ 1/4"x3"x3" WASHER PL	10d @ 4" OC EDGE NAILING (EN) 10d @ 12" OC FIELD NAILING (FN)	LAG SCREWS: 3/8"Ø W/ 3" MIN PENET @ 3" OC SDS 1/4"x6" (2" MIN PENET) @ 4" OC	BOTH SIDES @ 12" OC (STAG)
F	2	1330 PLF	1/2"	3x6	3x6	3/4" Ø @ 16" OC AB W/ 5/16"x3"x3" WASHER PL	10d @ 3" OC EDGE NAILING (EN) 10d @ 12" OC FIELD NAILING (FN)	LAG SCREWS: 3/8"Ø W/ 3" MIN PENET @ 2 1/2" OC SDS 1/4"x6" (2" MIN PENET) @ 3" OC	BOTH SIDES @ 9" OC (STAG)

Table 4.3.4 Maximum Shear Wall Aspect Ratios

Shear Wall Sheathing Type	Maximum h/b, Ratio
Wood structural panels, unblocked	2:1
Wood structural panels, blocked	3.5:1
Particleboard, blocked	2:1
Diagonal sheathing, conventional	2:1
Gypsum wallboard	2:1 ¹
Portland cement plaster	2:1 ¹
Structural Fiberboard	3.5:1

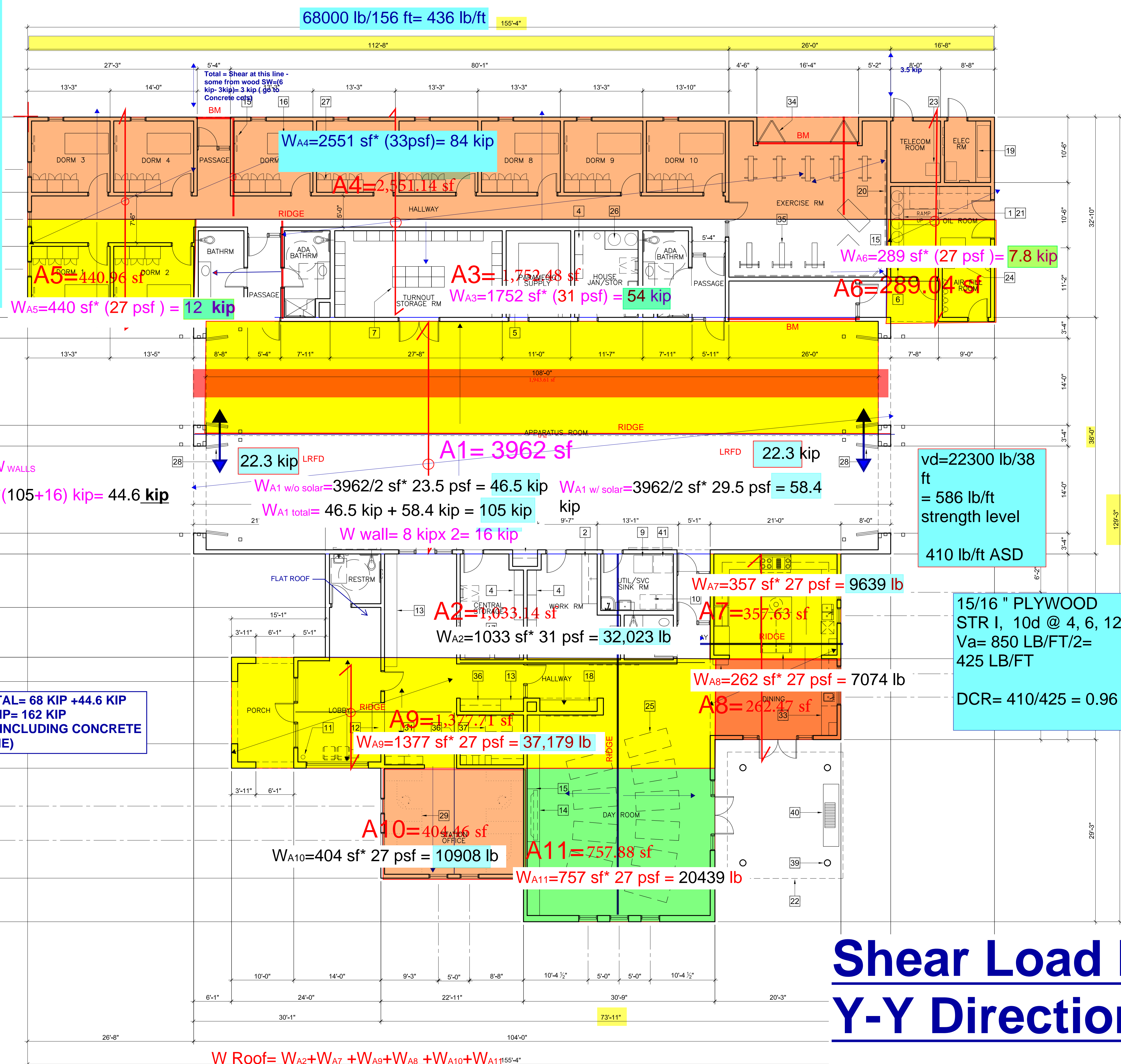
¹ Walls having aspect ratios exceeding 1.5:1 shall be blocked shear walls.

W_r= W_{A3}+W_{A4} + W_{A5}+W_{A6}
= 54 kip+ 84 kip+12 kip+7.8 kip=
= 158 kip

Wall:
North wall: 11.7 kip
Wall at Apparatus=8.5 kip
Wall at end= (3.5+3.3) kip=
6.8 kip

W_{total}= W roof + Wall + wall end
= 158 kip + 11.7kip + 8 kip + 6.8 kip=
185 kip

V= 185 kip* 0.369= 68 kip



W= W_{A1} +W WALLS
V= 0.369*(105+16) kip= 44.6 kip

V TOTAL= 68 KIP +44.6 KIP
+50 KIP= 162 KIP
(NOT INCLUDING CONCRETE FRAME)

W Roof= W_{A2}+W_{A7} +W_{A9}+W_{A8} +W_{A10}+W_{A11}
= 32 kip+ 9.6 kip +37 kip+ 7 kip +10 kip +20 kip= 116kip
W walls= 8.8 kip + 6.0 kip + 5 kip= 20 kip
W total = W roof + Wwalls = 116 kip + 20 kip= 136 kip
V= 0.369 * 136 kip= 50 kip

REVISED SCHEMATIC FLOOR PLAN - 13,150 SF

Shear Load load in Y-Y Direction

SOLAR AT HIGH ROOF & LOW ROOF

NOTES

- 55 GALLON DRUMS (N.I.C.).
- WORK BENCH / SHELVES.
- FLAMMABLE STORAGE CABINET.
- TYP. SHELVES.
- PARAMEDIC CABINET.
- AIR COMPRESSOR.
- TURNOUT GEAR STORAGE SYSTEM 24"x20"x72".
- CHARGING CABINET.
- ICE MACHINE.
- WASHER/DRYER STACKABLE COMBO. (N.I.C.).
- FIRE SPRINKLER RISER.
- COUNTER - ADA ACCESSIBLE.
- CABINET.
- DAY ROOM CABINET.
- FLAT SCREEN TV (N.I.C.).
- TYPICAL CAPTAIN BED.
- TELECOM CABINET.
- DISPLAY CABINET.
- MAIN SWITCH BD. PANEL ON 4" CONC. PAD.
- MIRROR WALL.
- PRE-FABRICATED OIL PAN CONTAINMENT CORP IN-LINE 4 DRUMS POLY SPILL PALLET 3000 WITH RAMP.
- PATIO CANOPY.
- MDT COMMUNICATIONS UNIT (LACoFD).
- OXYGEN COMPRESSOR (N.I.C.).
- TYPICAL LOUNGE CHAIR (N.I.C.).
- 100 GALLON WATER HEATER.
- TYP. DORM LOCKER.
- AUTOMATIC STEEL BI-FOLD DOORS 14' x 14'.
- TYP. WORK STATION (N.I.C.).
- STAINLESS STEEL KITCHEN.
- FILES.
- CHAIR STORAGE.
- STAINLESS STEEL COFFEE AREA.
- RETRACTABLE GLASS DOORS.
- EXERCISE EQUIPMENT (N.I.C.).
- BASE CABINET w/ UPPERS.
- COPIER (N.I.C.).
- PHONE CABINET.
- TYP. ROUND CONC. COLUMN.
- BUILT-IN BBQ.
- VENDING MACHINE (N.I.C.).

Apparatus Roof

Flat Roof

Sloped Roof

The rest colors

15/16 " PLYWOOD STR I, 10d @ 2 1/2, 4, 12
Va= 1280 LB/FT/2= 640 LB/FT
DCR= 410/640 = 0.64

WILLIAM LOYD JONES ARCHITECT

9415 culver boulevard
culver city, california
90232

TEL 310 392 3995

REVISED SCHEMATIC FLOOR PLAN

FIRE STATION 46
COUNTY OF LOS ANGELES FIRE DEPARTMENT
26720 BOMBERO LANE
VALENCIA, CALIFORNIA

FIVE POINT

Issue	
DESIGN	18.DEC.23
REVISION	17.APR.24
REVISION	22.APR.24
REVISION	14.NOV.24
REVISION	09.DEC.24
REVISION	06.FEB.25
REVISION	24.FEB.25
REVISION	03.MAR.25
LACoFD APPROVED	11.MAR.25

THE ABOVE DRAWINGS AND SPECIFICATIONS AND THEIR DESIGN AND ARRANGEMENTS REPRESENTED THEREBY ARE AND SHALL REMAIN THE PROPERTY OF THE ARCHITECT AND NO PART THEREOF SHALL BE COPIED, REPRODUCED, OR IN ANY MANNER USED FOR ANY OTHER PROJECT OTHER THAN THE SPECIFIC PROJECT FOR WHICH THEY HAVE BEEN PREPARED, AND WITHOUT THE WRITTEN CONSENT OF THE ARCHITECT. VISUAL CONCLUSION WITH THESE DRAWINGS OR SPECIFICATIONS SHALL CONSTITUTE ACCEPTANCE OF THE DRAWINGS AND SPECIFICATIONS BY THE USER.

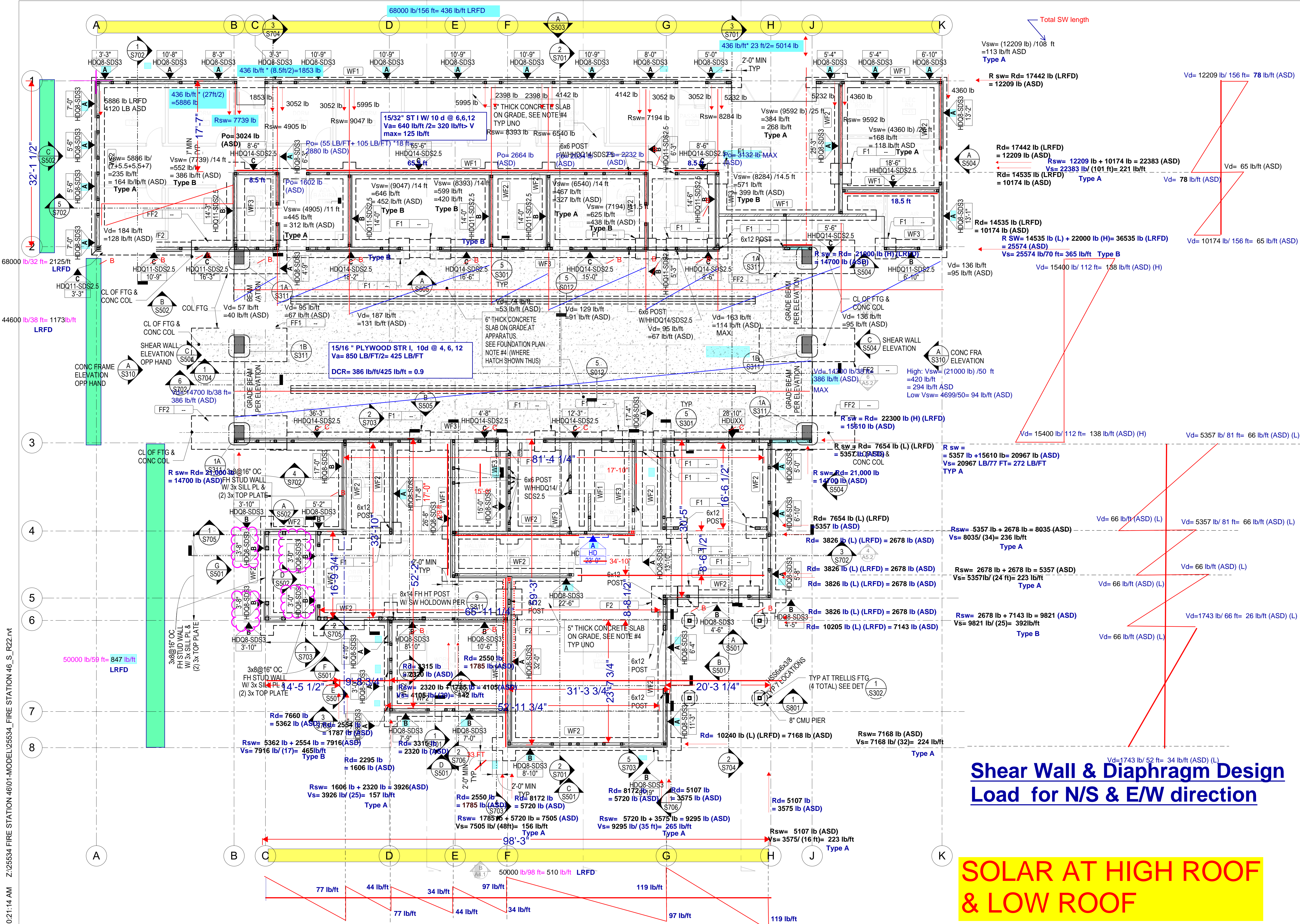
WRITTEN DIMENSIONS ON THESE DRAWINGS SHALL HAVE PRECEDENCE OVER SPOKE DIMENSIONS. DIMENSIONS SHALL BE IN FEET AND INCHES. ALL DIMENSIONS AND VARIATIONS FROM THE DIMENSIONS AND CONDITIONS SHOWN IN THESE DRAWINGS, SHOP DETAILS MUST BE SUBMITTED TO THE OFFICE FOR APPROVAL BEFORE PROCEEDING WITH FABRICATION.

Copyright 2025

Date	17.APR.24
Drawn	KC
Checked	WLJ
Scale	1/16" = 1'-0"
Job. No.	

A2

ADDENDUM 5



Shear Wall & Diaphragm Design Load for N/S & E/W direction

**SOLAR AT HIGH ROOF
& LOW ROOF**

SCALE: 1/8" = 1'-0"

1

**WILLIAM LOYD JONES
ARCHITECT**
9415 culver boulevard
culver city, california
90232
TEL 310 392 3995

so
saiful-bouquet
structural engineers
725 S. Figueroa St.,
37th floor
Los Angeles, CA 90017
www.saifulbouquet.com
(213) 315-2277
Project #25534

FOUNDATION PLAN
FIRE STATION 46
MISSION VILLAGE
COUNTY OF LOS ANGELES FIRE DEPARTMENT
VALENCIA, CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
No. S3289
Exp. 12/31/25
STRUCTURAL
STATE OF CALIFORNIA

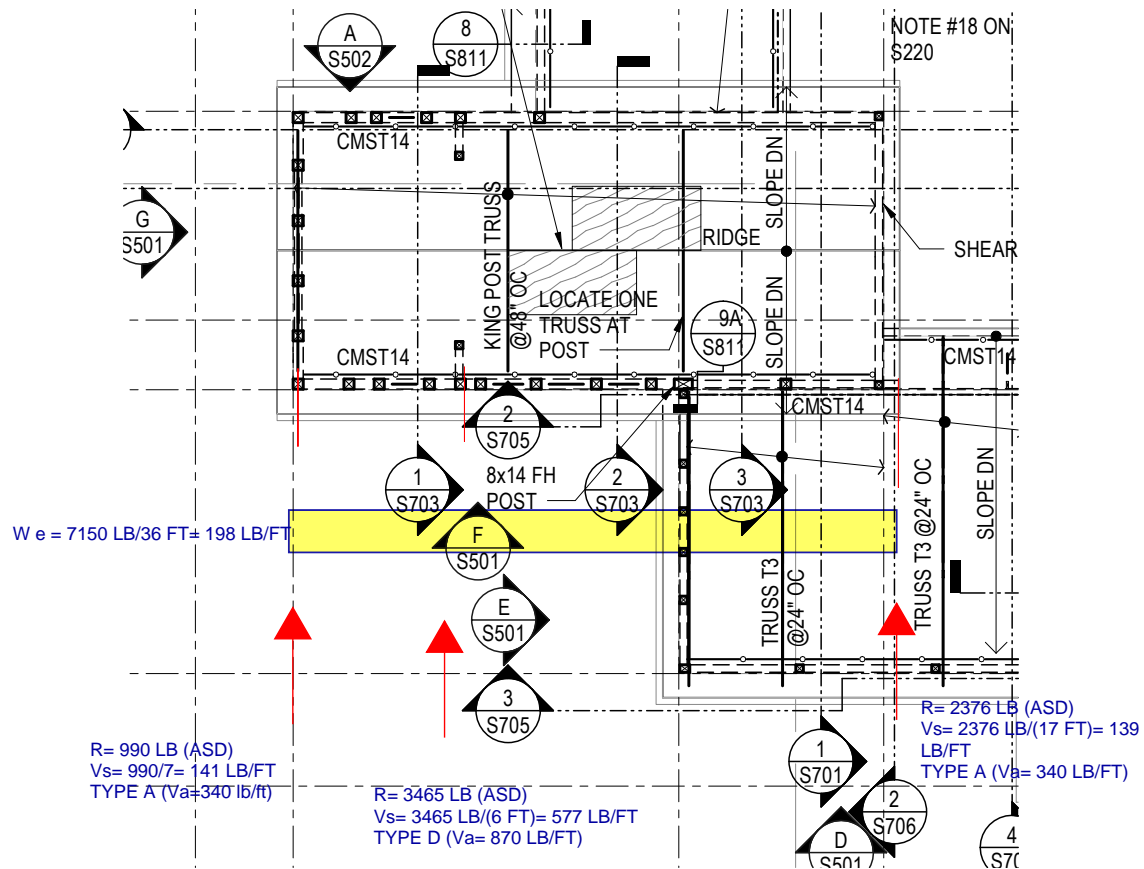
PROGRESS SET

THE ABOVE DRAWINGS AND SPECIFICATIONS AND ALL DESIGN AND
ARRANGEMENTS REPRESENTED THEREIN ARE AND SHALL REMAIN THE
PROPERTY OF THE ARCHITECT AND NO PART THEREOF SHALL BE COPIED,
DISCLOSED TO OTHERS OR USED IN CONNECTION WITH ANY WORK OR PROJECT
OTHER THAN THE SPECIFIC PROJECT FOR WHICH THEY HAVE BEEN PREPARED.
CONTACT WITH THESE DRAWINGS OR SPECIFICATIONS SHALL CONSTITUTE
CONCLUSIVE EVIDENCE OF ACCEPTANCE OF THE INSTRUCTIONS.
WRITTEN DIMENSIONS ON THESE DRAWINGS SHALL HAVE PRECEDENCE OVER
SCALED DIMENSIONS. CONTRACTORS SHALL VERIFY AND BE RESPONSIBLE
FOR ALL DIMENSIONS AND CONDITIONS ON THE JOB AND THE OFFICE MUST
BE NOTIFIED OF ANY VARIATIONS FROM THE DIMENSIONS AND CONDITIONS
SHOWN IN THESE DRAWINGS. SHOP DETAILS MUST BE SUBMITTED TO THIS
OFFICE FOR APPROVAL, BEFORE PROCEEDING WITH FABRICATION.
10/20/2025

Date	Issue Date
Drawn	
Checked	
Scale	AS NOTED
Job No.	Project Number

S210

APPENDIX 5
DESIGN DEVELOPMENT



W DL roof= 812 SF* 20 PSF= 16240 LB
W wall= 15 PSF *8 FT* (16 FT +32 FT)*2= 11520 LB
W total= 27760 LB

$$V = 0.368 * 27760 \text{ LB} = 10215 \text{ LB (LRFD)}$$
$$= 7150 \text{ LB (ASD)}$$

Shear load at front entrance in N/S direction

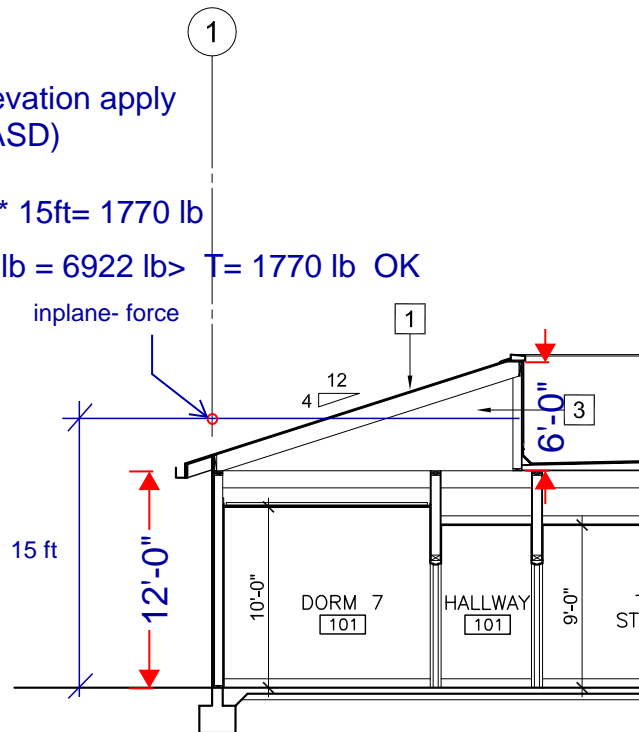
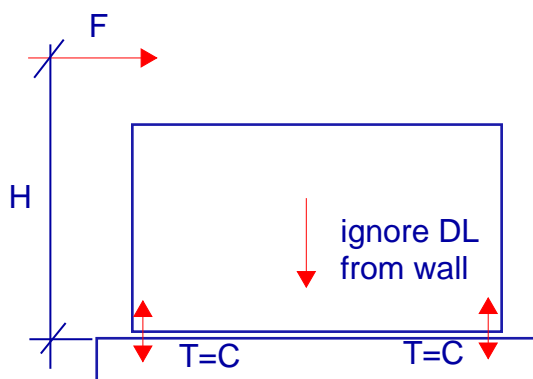
2.4 SHEAR WALL HOLDOWN DESIGN

Typical Shear walls at line 1

All walls is the same height and same elevation apply loads $H = 15$ ft and same $V_s = 118$ lb/ft (ASD)

$$T/C = F \cdot H/L = V_s \cdot L \cdot H/L = V_s \cdot H = 118 \text{ lb/ft} \cdot 15 \text{ ft} = 1770 \text{ lb}$$

HDQ8-SDS3 with 6x 6 $T_a = (9230) \cdot 0.75 \text{ lb} = 6922 \text{ lb} > T = 1770 \text{ lb}$ OK



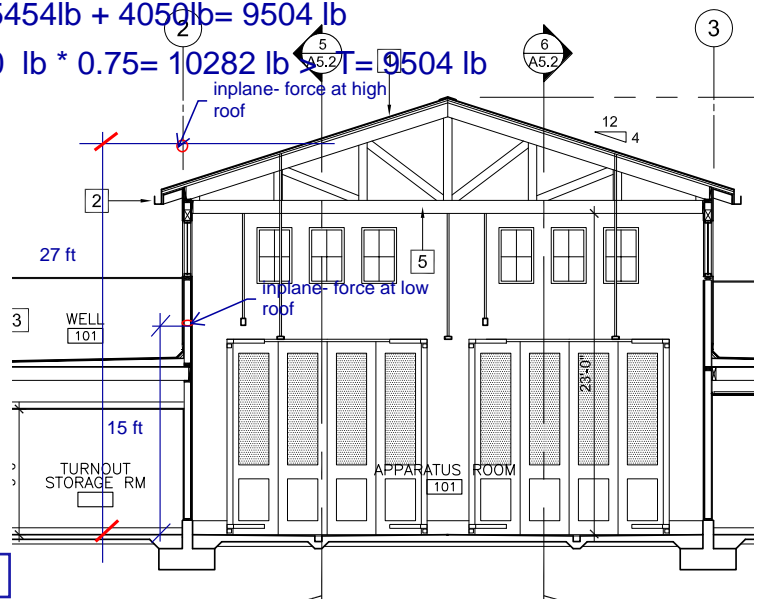
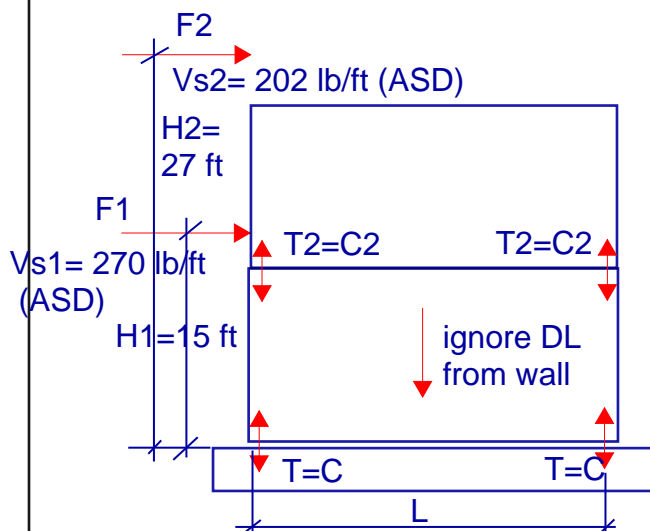
Typical Shear walls at line 2

$$T_2/C_2 = F \cdot H/L = V_s \cdot L \cdot H/L = V_s \cdot H = 202 \text{ lb/ft} \cdot (27 \text{ ft} - 12 \text{ ft}) = 3030 \text{ lb}$$

$$T/C = (F_2 \cdot H_2 + F_1 \cdot H_1)/L = (V_{s2} \cdot L \cdot H_2 + V_{s1} \cdot L \cdot H_1)/L = V_{s2} \cdot H_2 + V_{s1} \cdot H_1$$

$$= 202 \text{ lb/ft} \cdot 27 \text{ ft} + 270 \text{ lb/ft} \cdot 15 \text{ ft} = 5454 \text{ lb} + 4050 \text{ lb} = 9504 \text{ lb}$$

HDQ14-SDS2.5 with 6x 6 $T_a = 13710 \text{ lb} \cdot 0.75 = 10282 \text{ lb} > T = 9504 \text{ lb}$



Typical Shear walls at line 3

$$T2/C2 = F \cdot H/L = V_s \cdot L \cdot H/L = V_s \cdot H = 420 \text{ lb/ft} \cdot (27\text{ft}-12\text{ft}) = 6300 \text{ lb}$$

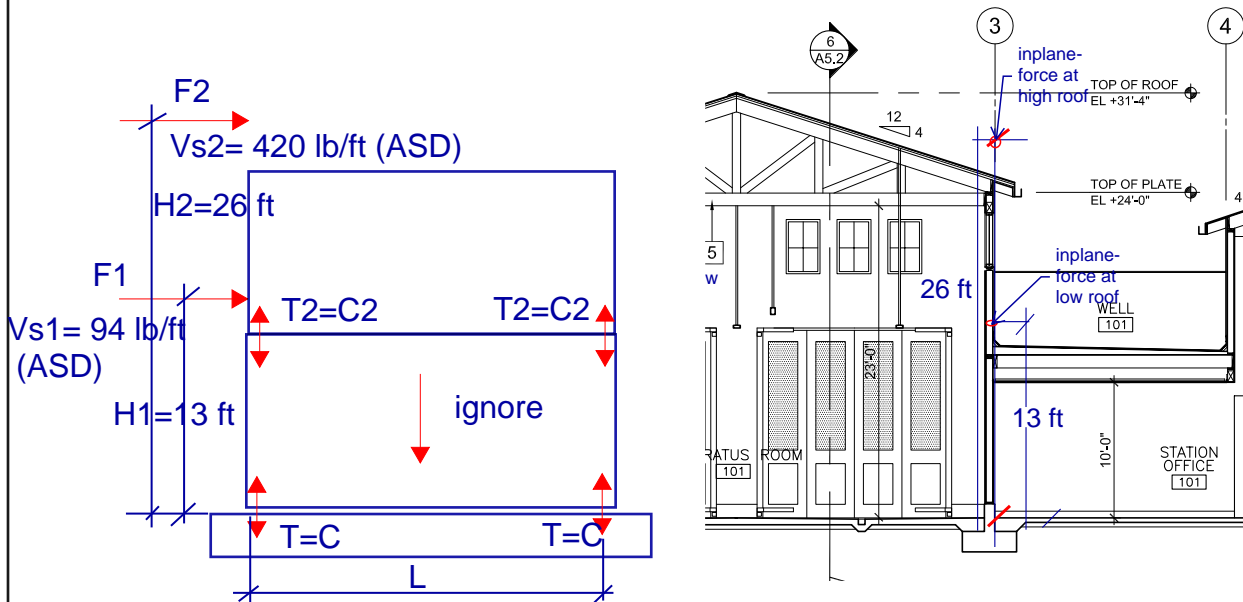
Conservative
ly, L=10 ft

$$T/C = (F2 \cdot H2 + F1 \cdot H1)/L = (V_s2 \cdot L \cdot H2 + V_s1 \cdot L \cdot H1)/L = V_s2 \cdot H2 + V_s1 \cdot H1$$

$$= 420 \text{ lb/ft} \cdot 26 \text{ ft} + 94 \text{ lb/ft} \cdot 12 \text{ ft} = 10920 + 1128 = 12048 \text{ lb}$$

$$T \text{ dl} = (0.6 - 0.14 \cdot Sds) \cdot (15 \text{ psf} \cdot (24 \text{ ft} \cdot L) + 25.5 \text{ psf} \cdot (18 \text{ ft} \cdot L)) \cdot L/2 / (L) = (0.37)(600 + 459) \cdot L/2 = 391 \cdot 10 \text{ ft}/2 = 1955 \text{ lb}$$

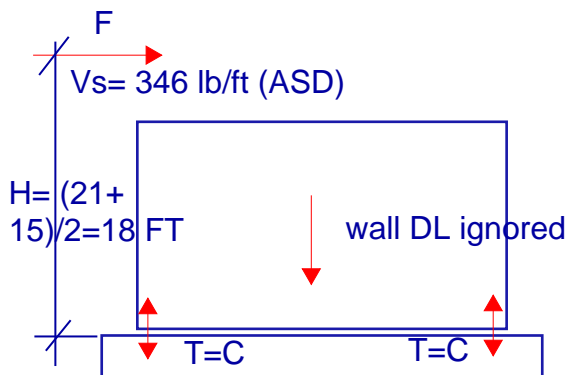
$$\text{HDQ14-SDS2.5 with 6x 6 Ta} = 13710 \cdot 0.75 \text{ lb} = 10282 > T = 12048 \text{ lb} - 1955 \text{ lb} = 10093 \text{ lb OK}$$



Typical Shear walls at line 4,6 &8

$$T/C = F \cdot H/L = V_s \cdot L \cdot H/L = V_s \cdot H = 346 \text{ lb/ft} \cdot 18\text{ft} = 6228 \text{ lb}$$

$$\text{HDQ8-SDS3 with 6x 6 Ta} = 9230 \text{ lb} \cdot 0.75 = 6922 > T = 6228 \text{ lb}$$



$$V = (420 \text{ lb/ft} + 94 \text{ lb/ft}) = 514 \text{ lb/ft (ASD)}$$

k = stiffness of the anchorage = F / δ (deflection / elongation)

Shear Walls in a Line E/W Direction Line 3

Check SW drift

(bending) (shear) (wall anchorage slip)

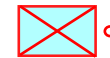
$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h}{b}\Delta_a \quad (C4.3.2-2)$$

δ

(1) HHDQ14-SDS2.5

$$k = (13710 \text{ lb/0.107"} = 128,130 \text{ lb/in}$$

$$V = \frac{8h^3}{EAb_{SW1}} + \frac{h}{1000G_a} + \frac{h^2}{kb_{SW1}}$$



End Post A= 5.5*7.5= 41.25 in^2

$$V = \frac{\delta}{\frac{8(24')^3}{(1,400,000)(16.5)(4')} + \frac{(24')}{1000(14,000)} + \frac{(24')^2}{(64,924)(4')}} + \frac{0.0000012}{0.0000012} + \frac{0.00028}{0.0000012}$$

δ

$$514 \text{ lb/ft } V = \frac{0.00038}{0.00038} \Rightarrow$$

$$= 0.00038 * 514 \text{ lb/ft} = 0.196"$$

$$\text{Drift} = 0.196 * Cd/le = 0.196 * 4/1.5 = 0.52" \leq 1.5\% * 24 \text{ ft} * 12 = 4.32"$$

ADDENDUM 5

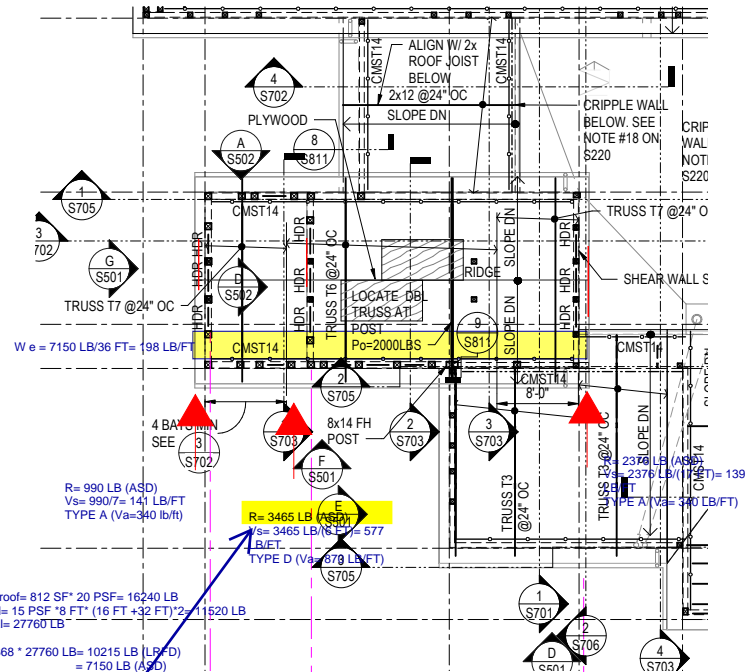
JUSTIFY ASPECT RATIO LESS THAN 3.5:1

1. Design shear walls per NDS section 4.3.5.2 " Force-Transfer Shear Walls" , wall height to wall width are defined per figure 4E. All shear wall aspect ratio is less than 3.5:1.0. Please see attached calculation for clarification.
2. Boundary members for force transfer around opening such collectors ,drag header beams/ straps are designed for force transfer around openings.

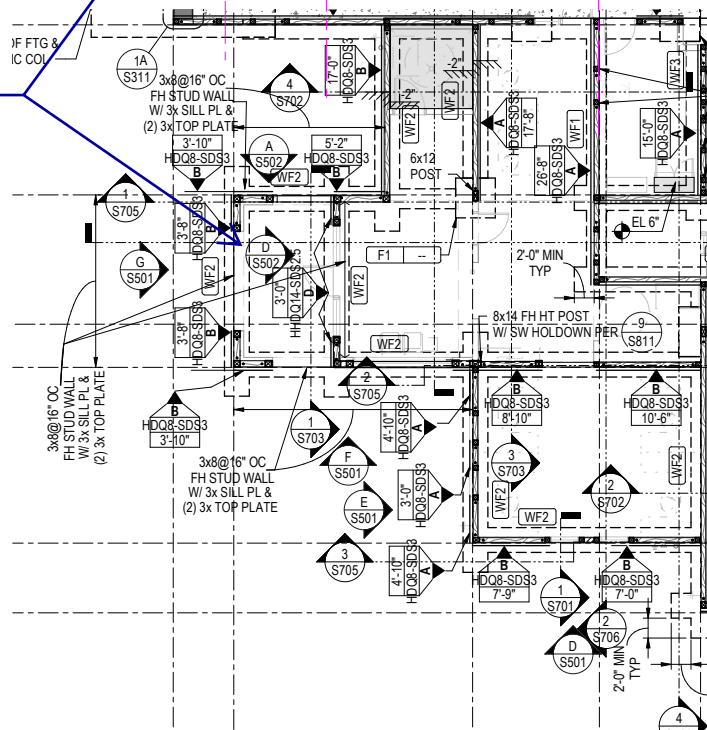
JUSTIFY HOLDOWN WITH 25 % REDUCTION

1. See attached calculation for justify of holdowns after 25% reduction

Narrow SW design in N/S direction

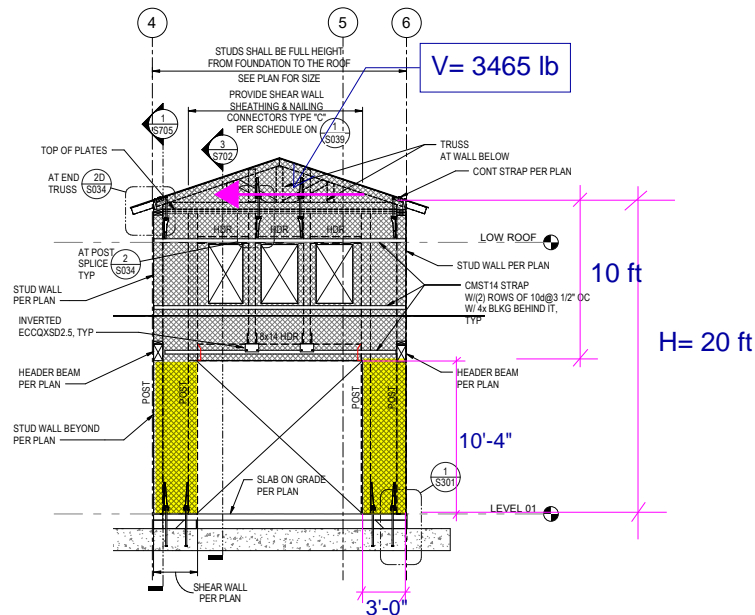


DESIGN
WALL
D/S502
WORST
CASE



Shear load at front entrance in N/S direction

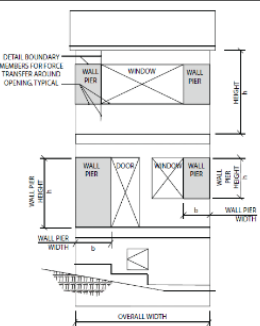
D S502



4.3.5.2 Force-transfer Shear Walls: Where shear walls with openings are designed for force transfer around the openings, the aspect ratio limitations of 4.3.4.4 shall apply as illustrated in Figure 4E. Design for force transfer shall be based on a rational analysis. The following limitations shall apply:

1. The length of each wall pier shall not be less than 2'.
2. A full-height wall segment shall be located at each end of a force-transfer shear wall.
3. Where out-of-plane offsets occur, portions of the wall on each side of the offset shall be considered as separate force-transfer shear walls.
4. Collectors for shear transfer shall be provided through the full length of the force-transfer shear wall.

Figure 4E Typical Shear Wall Height-to-Width Ratio for Shear Walls Designed for Force Transfer Around Openings



4

LATERAL FORCE-RESISTING SYSTEMS

Check Shear Wall Aspect Ratios

$$H_{pier}/L_{pier} = 10.3 \text{ ft}/3 \text{ ft} = 3.4 < 3.5 \text{ OK}$$

4.3.4.4 Aspect Ratio of Force-transfer Shear Walls: The aspect ratio limitations of Table 4.3.4 shall apply to the overall shear wall including openings and to each wall pier at the sides of openings. The height of a wall pier with an opening on one side shall be defined as the clear height of the pier at the side of the opening. The height of a wall pier with an opening on each side shall be defined as the larger of the clear heights of the pier at the sides of the openings. The length of a wall pier shall be defined as the sheathed length of the pier. Wall piers with aspect ratios exceeding 3:1 shall not be considered as portions of force-transfer shear walls.

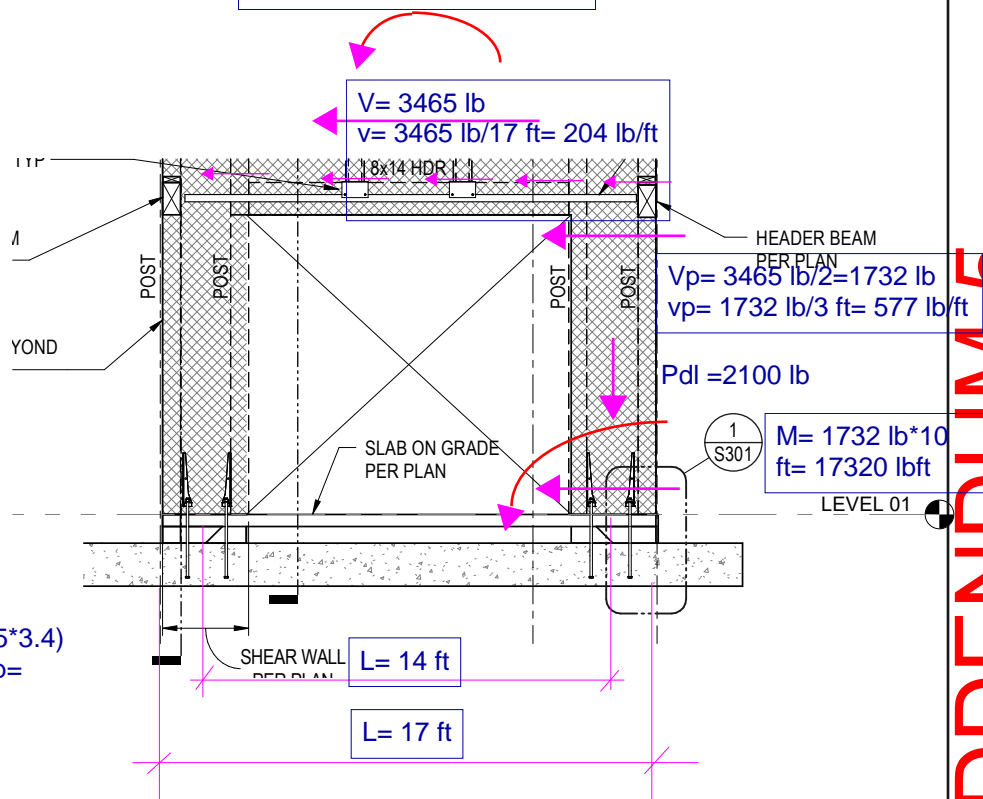
Table 4.3.4 Maximum Shear Wall Aspect Ratios

Shear Wall Sheathing Type	Maximum h/b, Ratio
Wood structural panels, unblocked	2:1
Wood structural panels, blocked	3.5:1
Particleboard, blocked	2:1
Diagonal sheathing, conventional	2:1
Gypsum wallboard	2:1 ¹
Portland cement plaster	2:1 ¹
Structural Fiberboard	3.5:1

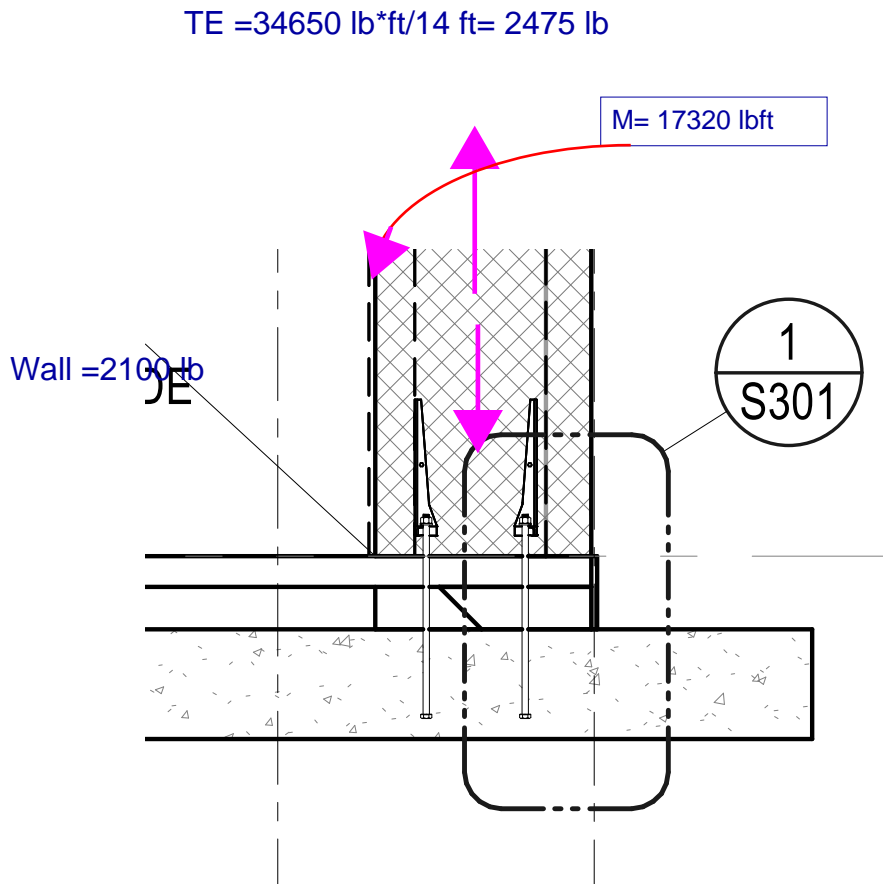
1 Walls having aspect ratios exceeding 1.5:1 shall be blocked shear walls.

$A = 280 \text{ ft}^2$
 $Pdl = 280 \text{ ft}^2 \times 15 \text{ psf} = 4200 \text{ lb}$
 $Pdl \text{ each pier} = 4200 \text{ lb} / 2 = 2100 \text{ lb}$
 $vp = 577 \text{ lb/ft}$
 $\text{Type D } Va = 870 \text{ lb/ft} \times (1.25 - 0.125 \times 3.4)$
 $= 870 \text{ lb/ft} \times (0.825) = 717 \text{ lb/ft} > vp =$
 577 lb/ft OK

$$M = 3465 \text{ lb} \cdot 10 \text{ ft} = 34650 \text{ lbft}$$



ADDENDUM 5

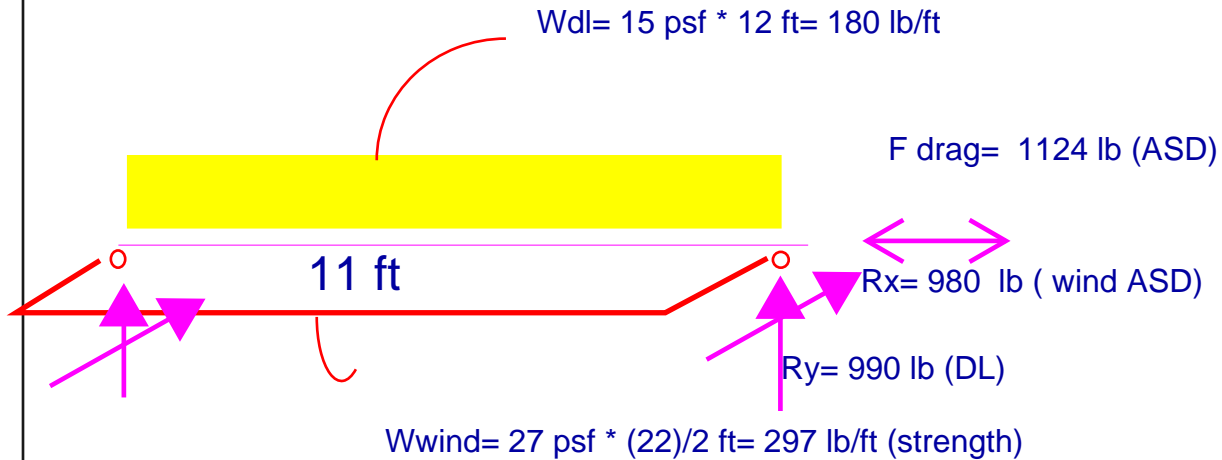


$$TE \text{ total} = 2475 \text{ lb} / 2 + 17320 \text{ lbft} / 3 \text{ ft} = 1238 \text{ lb} + 5773 \text{ lb} = 7011 \text{ lb}$$

$$P_{dl} = (0.6 - 0.14 \cdot S_{ds}) \cdot W_{wall} = (0.6 - 0.14 \cdot 1.6) \cdot 2100 = 789 \text{ lb}$$

$$T_{\text{Net / holdown}} = 7011 \text{ lb} - 789 \text{ lb} = 6222 \text{ lb} < 0.75 \cdot 13710 \text{ lb} = 10,282 \text{ lb}, \text{ HHDQ14-SDS2.5 OK}$$

DRAG HEADER DESIGN



DRAG LOAD FROM HEADER TO PIER SHEAR WALL

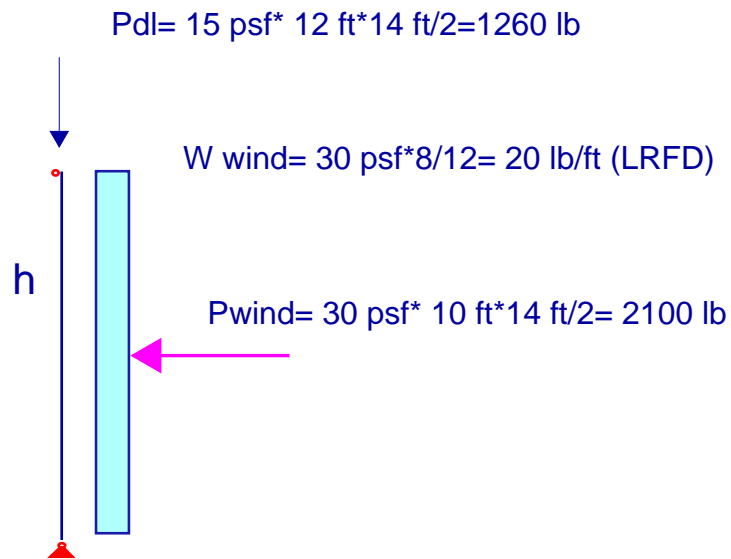
F drag= 204 lb/ft * 14 ft= 1732 lb= 1124 lb

DCR= 0.2 <1.0 8x14 is OK

Strap CMST14 Ta= 6472 lb for Lend= 60 " (partial nailing)

L end = 30" Ta= 6472 lb/2= 3236 lb

End Post design G/S501



$$h = 21'-0" - (0'-3" + 2.5" + 0'-6") = 20'$$

2x6 @ 16 " DCR= 0.92

$$\text{del} = 2.3'' \times 0.42 = 0.96'' \quad L/\text{Del} = 17.25 \text{ ft} \times 12 / 0.97'' = 214 < 360 \text{ NG}$$

8x8 @ 16 " DCR=0.7

$$\text{del} = 1.5'' \times 0.42 = 0.63'' \quad L/\text{Del} = 20 \text{ ft} \times 12 / 0.63'' = 380 > 360$$

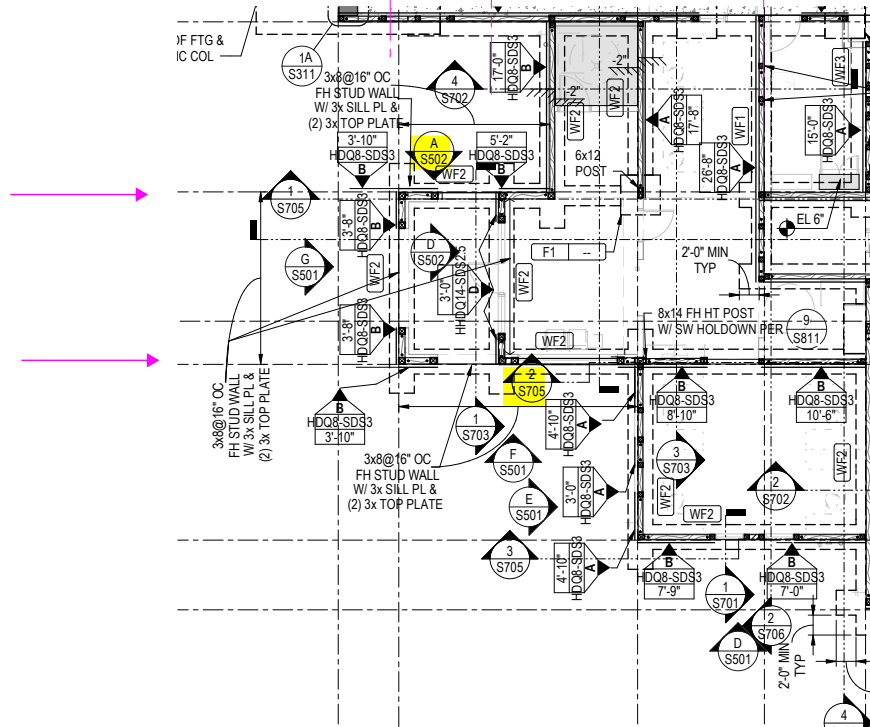
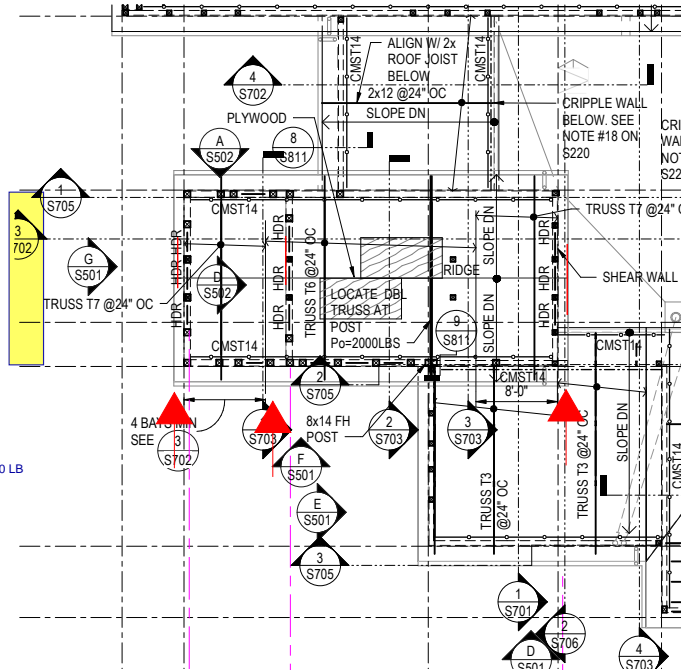
Provide 8x8 post is OK

Narrow SW design in E/W direction

$F_e = 7150 \text{ LB}/2 = 3575 \text{ lb (ASD)}$
 $W_e = 7150 \text{ LB}/17 \text{ FT} = 420 \text{ LB}/\text{FT (ASD)}$
 $W_e = 7150 \text{ LB}/2 = 3575 \text{ lb (ASD)}$

 $W \text{ DL roof} = 812 \text{ SF} \times 20 \text{ PSF} = 16240 \text{ LB}$
 $W \text{ wall} = 15 \text{ PSF} \times 8 \text{ FT} \times (16 \text{ FT} + 32 \text{ FT}) \times 2 = 11520 \text{ LB}$
 $W \text{ total} = 27760 \text{ LB}$

 $V = 0.368 \times 27760 \text{ LB} = 10215 \text{ LB (LRFD)}$
 $= 7150 \text{ LB (ASD)}$



Shear load at front entrance in N/S direction

ADDENDUM 5

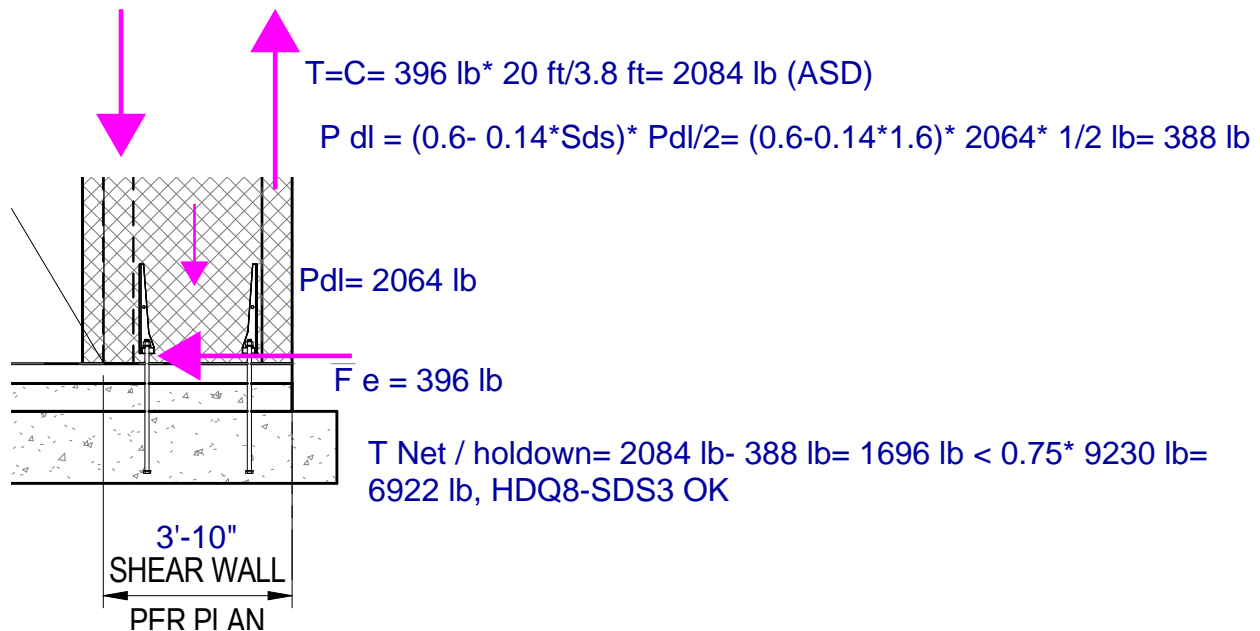
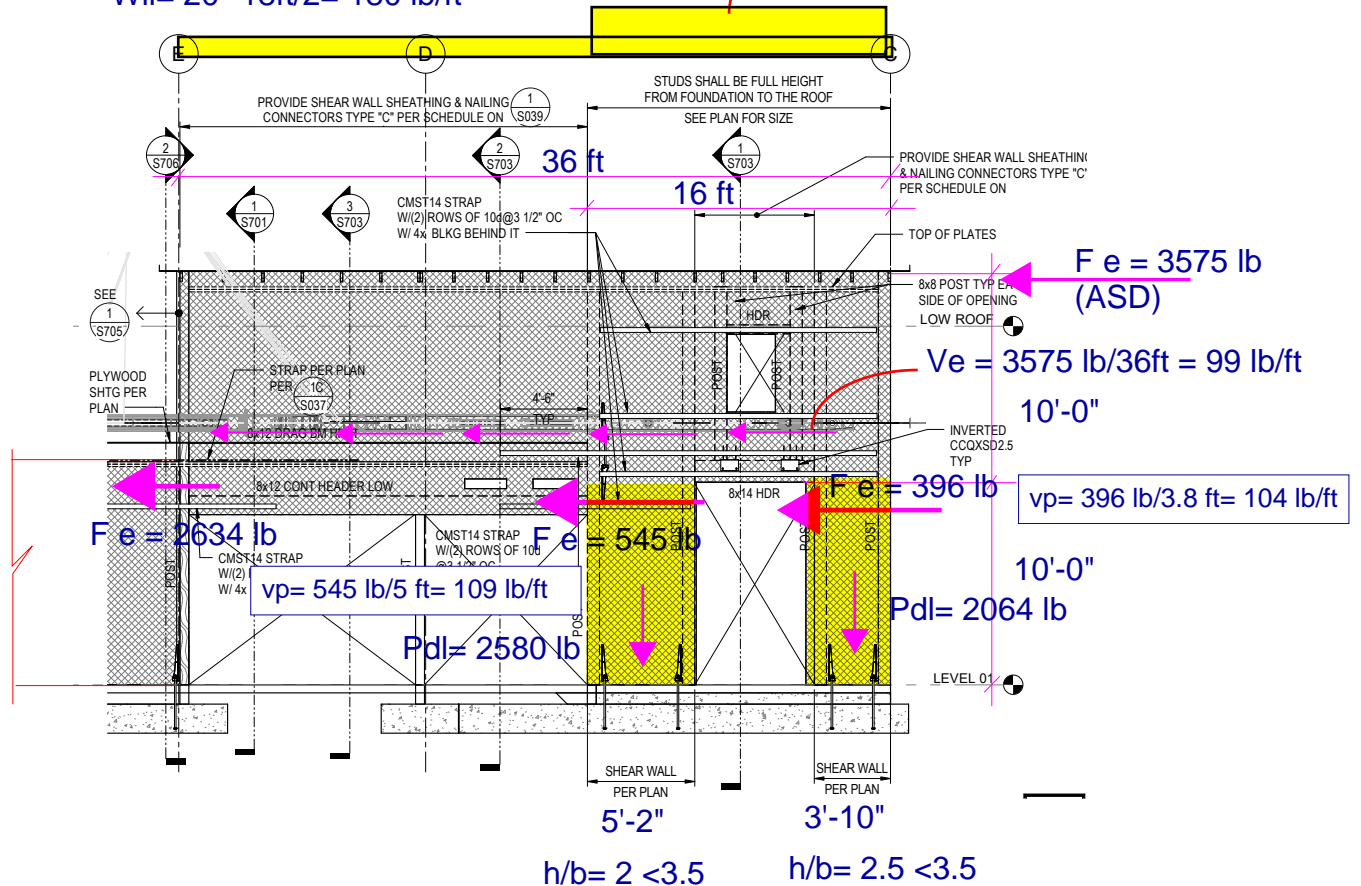
A S502

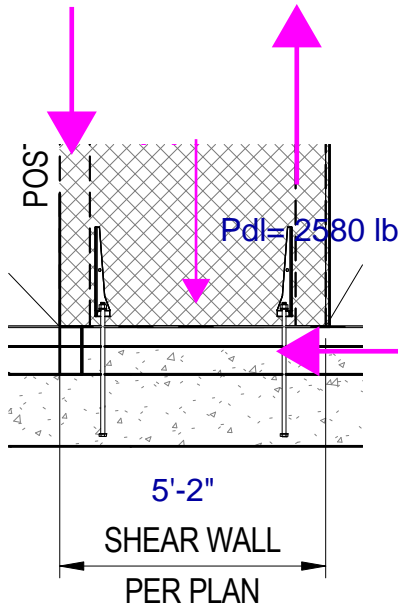
$$Wdl = 24\text{psf} * 18\text{ft}/2 * 15\text{psf} * 20\text{ft} = 516\text{ lb/ft}$$

$$Wll = 20 * 18\text{ft}/2 = 180\text{ lb/ft}$$

$$Wdl = 24\text{psf} * 18\text{ft}/2 * 15\text{psf} * 12\text{ft} = 396\text{ lb/ft}$$

$$Wll = 20 * 18\text{ft}/2 = 180\text{ lb/ft}$$





$$T=C= 545 \text{ lb} \cdot 20 \text{ ft}/5 \text{ ft}= 2180 \text{ lb (ASD)}$$

$$P_{dl} = (0.6 - 0.14 \cdot S_{ds}) \cdot P_{dl}/2 = (0.6 - 0.14 \cdot 1.6) \cdot 2580 \cdot 1/2 \text{ lb} = 485 \text{ lb}$$

$$F_e = 545 \text{ lb}$$

$$T_{\text{Net / holdown}} = 2180 \text{ lb} - 485 \text{ lb} = 1695 \text{ lb} < 0.75 \cdot 9230 \text{ lb} = 6922 \text{ lb, HDQ8-SDS3 OK}$$

$$T_{\text{drag}} = 2638 \text{ lb (ASD)}$$

$$\text{Strap CMST14 } T_a = 6472 \text{ lb} > T_{\text{drag}} = 2638 \text{ lb OK}$$

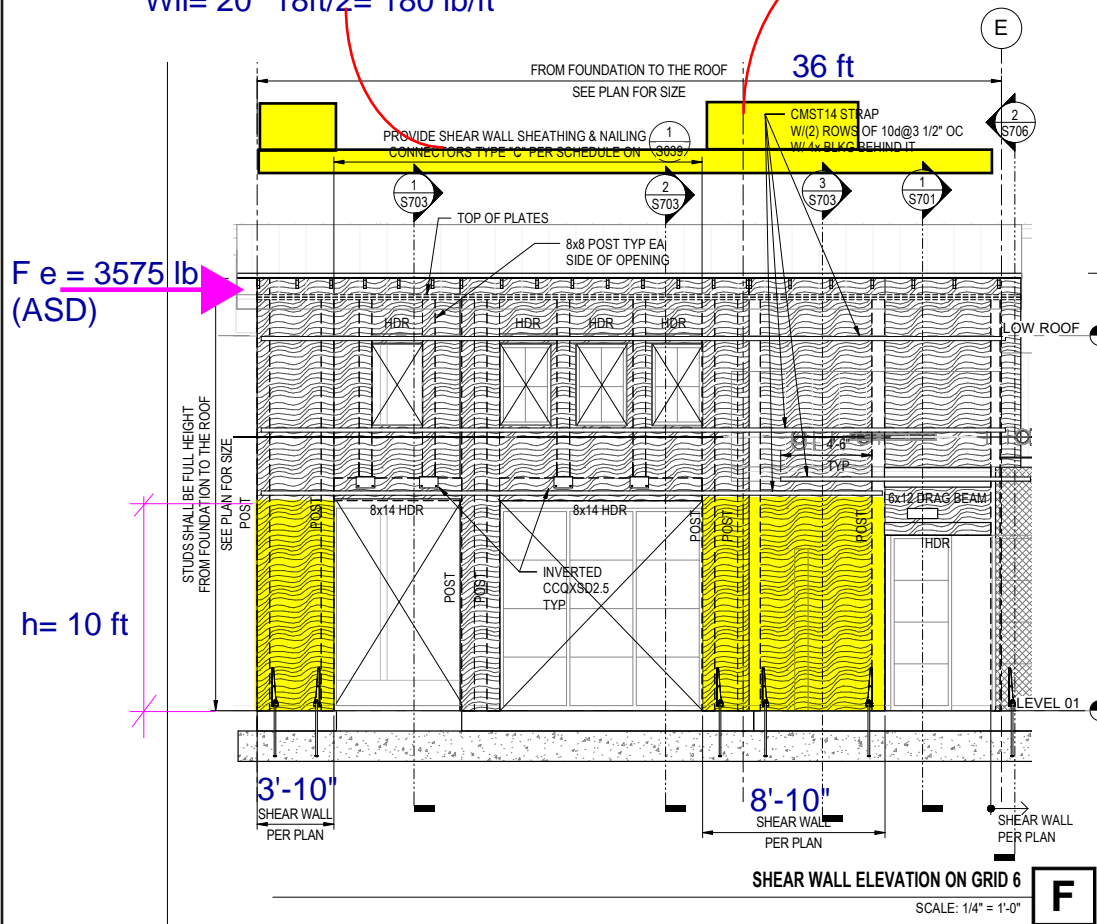
F S501

$$Wdl = 24\text{psf} * 18\text{ft}/2 * 15\text{psf} * 12\text{ft} = 396\text{ lb/ft}$$

$$Wll = 20 * 18\text{ft}/2 = 180\text{ lb/ft}$$

$$Wdl = 24\text{psf} * 18\text{ft}/2 * 15\text{psf} * 20\text{ft} = 516\text{ lb/ft}$$

$$Wll = 20 * 18\text{ft}/2 = 180\text{ lb/ft}$$



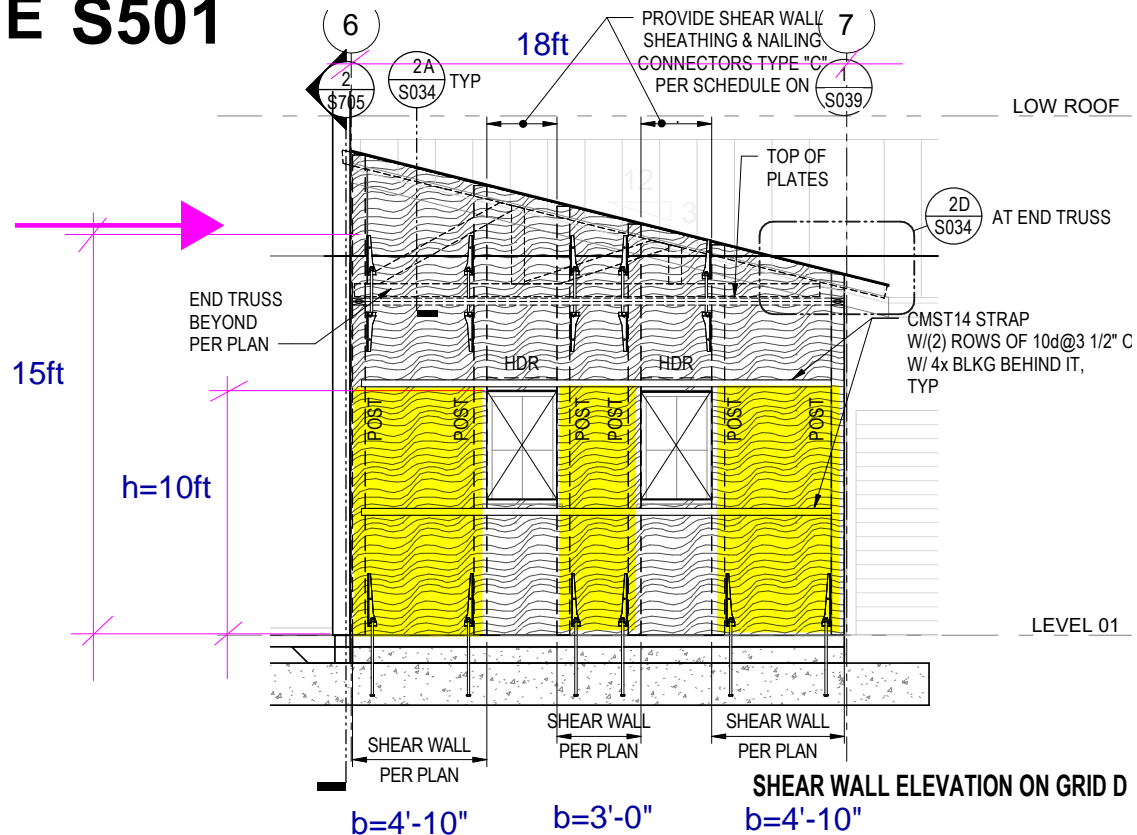
$$h/b = 10\text{ ft}/3'-10" = 2.6 < 3.5$$

$$h/b = 10\text{ ft}/8'-10" = 1.1 < 3.5$$

By Inspection from walls on wall elevation A/S502, Holdown is oK with 25% reduction

ADDENDUM 5

E S501



$$h/b = 10 \text{ ft} / 4' - 10" = 2.1 < 3.5 \text{ OK}$$

$$h/b = 10 \text{ ft} / 3' - 0" = 3.3 < 3.5 \text{ OK}$$

$$h/b = 10 \text{ ft} / 4' - 10" = 2.1 < 3.5 \text{ OK}$$

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Column

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: DRAG HEADER BM ON D/S502

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Analysis Method		Allowable Stress Design		Wood Section Name		8x14	
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber	
Overall Column Height		11 ft		Wood Member Type		Sawn	
(Used for non-slender calculations)							
Wood Species		Douglas Fir-Larch		Exact Width		7.50 in	
Wood Grade		No.1		Exact Depth		13.50 in	
Fb +		1200 psi	Fv	170 psi	Area		101.250 in^2
Fb -		1200 psi	Ft	825 psi	Ix		1,537.73 in^4
Fc - Prll		1000 psi	Density	31.21 pcf	Iy		474.609 in^4
Fc - Perp		625 psi					
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			
Basic		1600	1600	1600 ksi		Cf or Cv for Bending 0.9870	
Minimum		580	580			Cf or Cv for Compression0.9870	
						Cf or Cv for Tension 0.9870	
						Cm : Wet Use Factor 1.0	
						Ct : Temperature Fact 1.0	
						Cfu : Flat Use Factor 1.0	
						Kf : Built-up columns 1.0	
						Use Cr : Repetitive ? No	
Brace condition for deflection (buckling) along columns :							
				X-X (width) axis :		Unbraced Length for buckling ABOUT Y-Y Axis = 11 ft, k	
				Y-Y (depth) axis :		Unbraced Length for buckling ABOUT X-X Axis = 11 ft, k	

Applied Loads

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

Column self weight included : 241.390 lbs * Dead Load Factor

AXIAL LOADS . . .

Axial Load at 11.0 ft, E = 1.10 k

BENDING LOADS . . .

Lat. Uniform Load creating Mx-x, D = 0.180 k/ft

Lat. Uniform Load creating My-y, W = 0.2970 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.2115 : 1**
Load Combination +D+0.60W
Governing NDS Formula $\phi P_n + M_{xx} + M_{yy}$, NDS Eq. 3.9-
Location of max.above base 5.463 ft
At maximum location values are .
Applied Axial 0.2414 k
Applied Mx 2.722 k-ft
Applied My 2.695 k-ft
Fc : Allowable 1,077.07 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.990 k	Bottom along Y-Y	0.990 k
Top along X-X	1.634 k	Bottom along X-X	1.634 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y	0.02436 in	at	5.537 ft	above base
for load combination : D Only				
Along X-X	0.1302 in	at	5.537 ft	above base
for load combination : W Only				

Other Factors used to calculate allowable stresses . . .

<u>Bending</u>	<u>Compression</u>	<u>Tension</u>
----------------	--------------------	----------------

PASS Maximum Shear Stress Ratio = **0.09586 : 1**
Load Combination D Only
Location of max.above base 0.0 ft
Applied Design Shear 14.667 psi
Allowable Shear 153.0 psi

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.841	0.1353	PASS	5.463 ft	0.09586	PASS	0.0 ft
+D+0.60W	1.600	0.682	0.2115	PASS	5.463 ft	0.05392	PASS	0.0 ft
+D+0.70E	1.600	0.682	0.07666	PASS	5.463 ft	0.05392	PASS	0.0 ft
+D+0.450W	1.600	0.682	0.1778	PASS	5.463 ft	0.05392	PASS	0.0 ft
+D+0.5250E	1.600	0.682	0.07660	PASS	5.463 ft	0.05392	PASS	0.0 ft
+0.60D+0.60W	1.600	0.682	0.1808	PASS	5.537 ft	0.05338	PASS	0.0 ft
+0.60D+0.70E	1.600	0.682	0.04601	PASS	5.537 ft	0.03235	PASS	0.0 ft

Wood Column

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: DRAG HEADER BM ON D/S502

Maximum Reactions

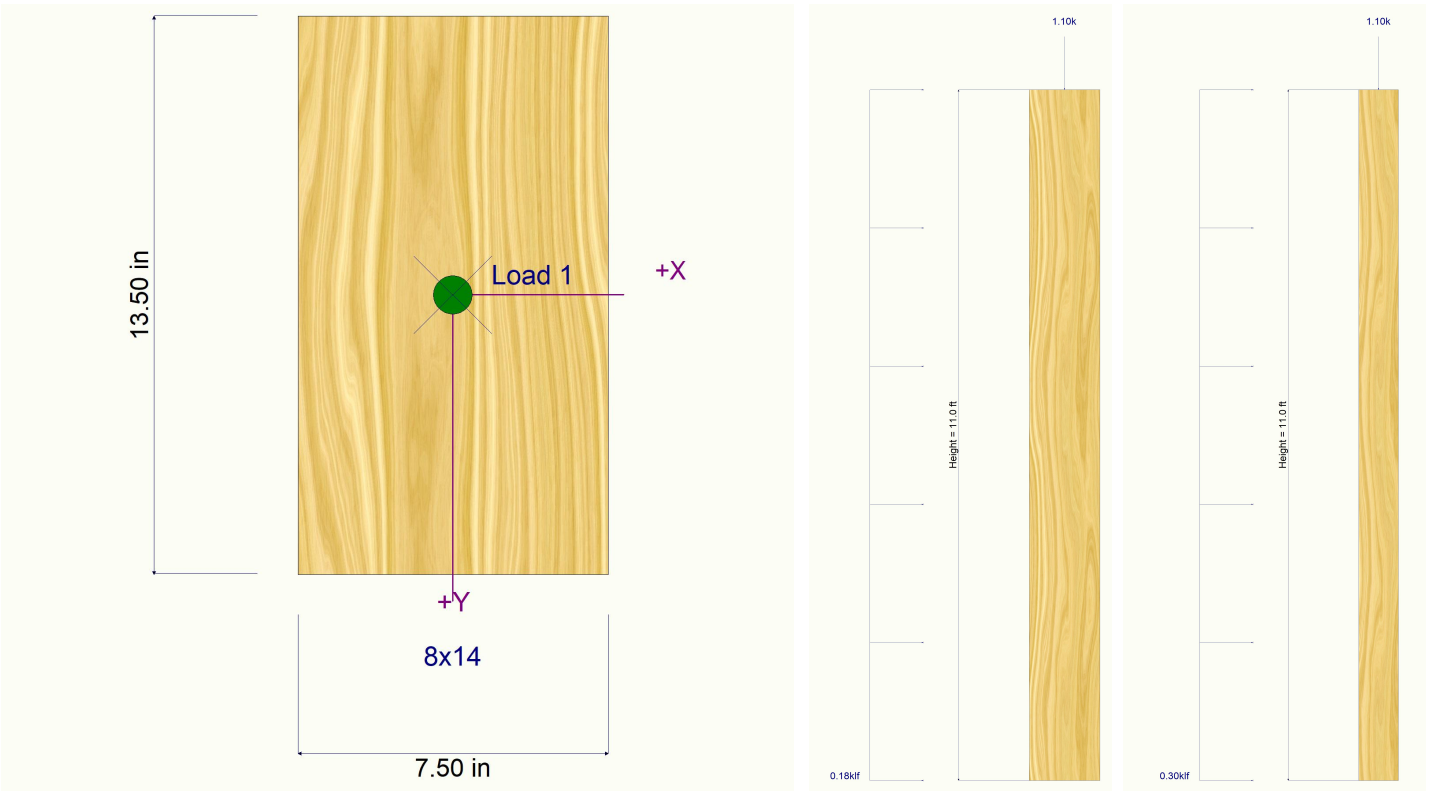
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		Y-Y Axis Reaction		Axial Reaction	My - End Moments k-ft		Mx - End Moments	
	@ Base	@ Top	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top
D Only			0.990	0.990	0.241				
+D+0.60W	0.980	0.980	0.990	0.990	0.241				
+D+0.70E			0.990	0.990	1.011				
+D+0.450W	0.735	0.735	0.990	0.990	0.241				
+D+0.5250E			0.990	0.990	0.819				
+0.60D+0.60W	0.980	0.980	0.594	0.594	0.145				
+0.60D+0.70E			0.594	0.594	0.915				
W Only	1.634	1.633							
E Only					1.100				

Maximum Deflections for Load Combinations

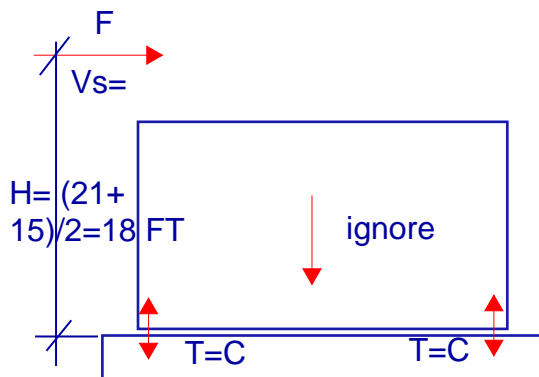
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000ft	0.024 in	5.537 ft
+D+0.60W	0.0781 in	5.537ft	0.024 in	5.537 ft
+D+0.70E	0.0000 in	0.000ft	0.024 in	5.537 ft
+D+0.450W	0.0586 in	5.537ft	0.024 in	5.537 ft
+D+0.5250E	0.0000 in	0.000ft	0.024 in	5.537 ft
+0.60D+0.60W	0.0781 in	5.537ft	0.015 in	5.537 ft
+0.60D+0.70E	0.0000 in	0.000ft	0.015 in	5.537 ft
W Only	0.1302 in	5.537ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000ft	0.000 in	0.000 ft

Sketches



ADDENDUM 5

Shear wall holdown in N/S direction



$$T/C = F \cdot H/L = V_s \cdot L \cdot H/L = V_s \cdot H = V_s \cdot 18 \text{ ft}$$

V_s less than 492 lb/ft, HHDQ11-SDS2.5 $T_a = 11810 \text{ lb} \cdot 0.75 = 8857 \text{ lb} = T = v \text{ lb/ft} \cdot 18 \text{ ft} =$

V_s less than 384 lb/ft, HDQ8-SDS3 $T_a = 9230 \cdot 0.75 \text{ lb} = 6922 \text{ lb} = T = v \text{ lb/ft} \cdot 18 \text{ ft}$

$V = 374 \text{ lb/ft}$ (ASD) k = stiffness of the anchorage = F / δ (deflection / elongation)

Shear Walls in a Line N/S Direction Line E.5

(bending) (shear) (wall anchorage slip)

Check SW drift

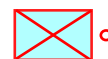
$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h}{b}\Delta_a \quad (\text{C4.3.2-2})$$

δ

(1) HHDQ11-SDS2.5

$k = (13015 \text{ lb}/0.107") = 128,130 \text{ lb/in}$

$$V = \frac{8h^3}{EAb_{sw1}} + \frac{h}{1000G_a} + \frac{h^2}{kb_{sw1}}$$



End Post A = $5.5 \times 7.5 = 41.25 \text{ in}^2$

$$V = \frac{\delta}{\frac{8(14')^3}{(1,400,000)(41.25 \text{ in}^2)(14 \text{ ft})} + \frac{(14')}{1000(14,000)} + \frac{(14')^2}{(64,924)(14 \text{ ft})}}$$

δ

0.000023 0.0000007 0.00011

374 lb/ft $V = \frac{\delta}{0.00013} \Rightarrow$

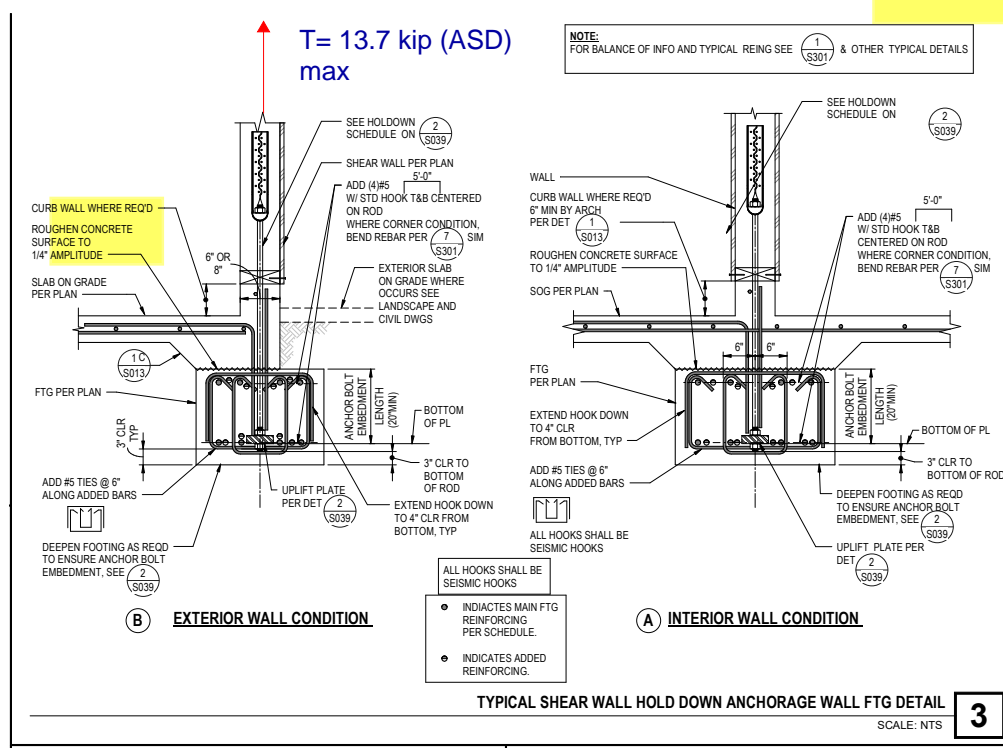
$$\delta = 0.00013 \times 374 \text{ lb/ft} = 0.048"$$

$$\text{Drift} = 0.048 \times 107 \text{ Cd/le} = 0.048 \times 4/1.5 = 0.22" \leq 0.375 = 3/8"$$

ADDENDUM 5

Shear Wall Anchorage Design

Typical Anchorage For Shear at Main Building



Loading for Shear Wall anchorage Design:

1" Diameter $F_y=36$ ksi $T_u = 1.2 F_y \cdot A_g = 1.2 \cdot 36 \text{ ksi} \cdot 0.79 \text{ in}^2 = 34 \text{ kip}$.

Wood Holdown $T_u = 13.7 \text{ kip} \cdot 1.4 = 19 \text{ kip}$ (Design For this Failure Mode)

Omega Load= $(3.0-1/2) 13.7 \cdot 1.4 = 47 \text{ kip}$

SECTION 17.4.3 PULL OUT STRENGTH

$$N_{pn} = \psi_{cpn} N_p \quad (17.4.31)$$

$$\psi_{cp} = 1.0 \quad (\text{CRACK SERVICE}) \quad 17.4.36$$

$$N_p = 8 A_{brg} f'_c \quad (17.4.3.4)$$

$$N_p = 8 \times (4" \times 4") \times 3 \text{ ksi} = 384 \text{ kip}$$

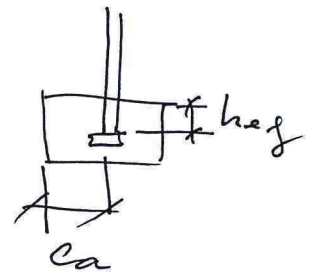
$$\rightarrow \phi N_{pn} = 0.75 \times 1.0 \times 384 = 288 \text{ kip} > T_u = 19 \text{ kip}$$

SECTION 17.4.4 CONCRETE SIDE FACE BLOW OUT

$$h_{ef} = (20" - 1.5") = 18.5"$$

$$C_a = \frac{24}{2} = 12"$$

$$2.5 \times C_a = 30" > h_{ef}$$



\hookrightarrow NO SIDE BLOW OUT OCCURS

STRENGTH OF ANCHOR REINF

REINF. EA. SIDE OF A.B. $0.5 h_{ef}$

$$(0.5 \times 18.5 \times 2) = 18.5"$$

#5 RE @ 6" o.c

$\square \leftarrow 2 \text{ leg}$

$$\left(\frac{18.5}{6} \times 2 \right) = 6$$

$$\phi N_u = \phi P_n = 0.75 \times 60 \text{ ksi} \times 6 \times 0.31 = 84 \text{ kip} > 19 \text{ kip}$$

ANCHOR . UPLIFT CHECK

$$T_u = 19 \text{ kip}$$

$$q_u = \frac{19}{(4 \times 4)} = 1.2 \text{ ksi}$$

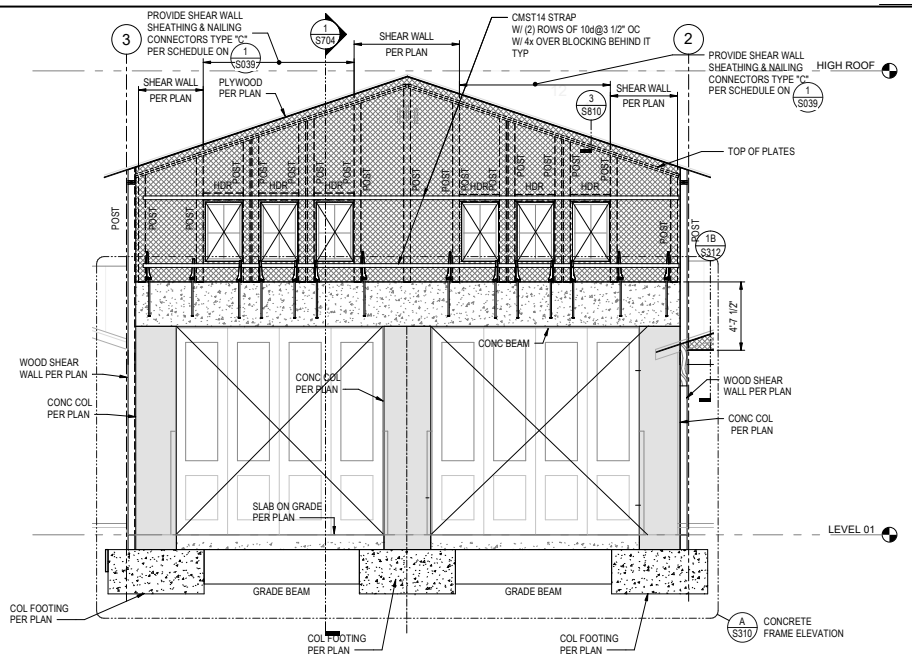
$$M_u = \frac{w_u l^2}{8} = \frac{1.2 \times 2^2}{8} = 2.4 \text{ kip"}^2$$

$$Z_{req} = \frac{2.4 \text{ kip"}^2}{0.9 \times 36 \text{ ksi}} = 0.07 \text{ in}^3$$

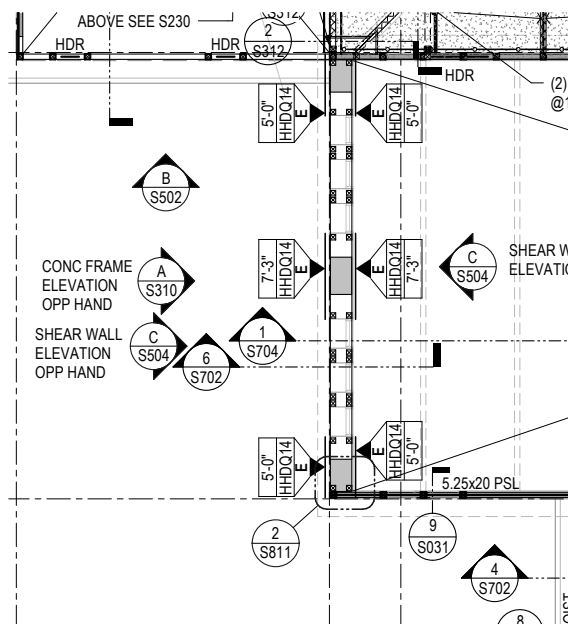
$$Z = \frac{bd^2}{4} \rightarrow d = 0.24 \text{ "}$$

USE PL 3/8 " x 4 x 4 x 3/8

CHECK SHEAR WALL HOLDOWN AT EA END OF APPARATUS ROOM



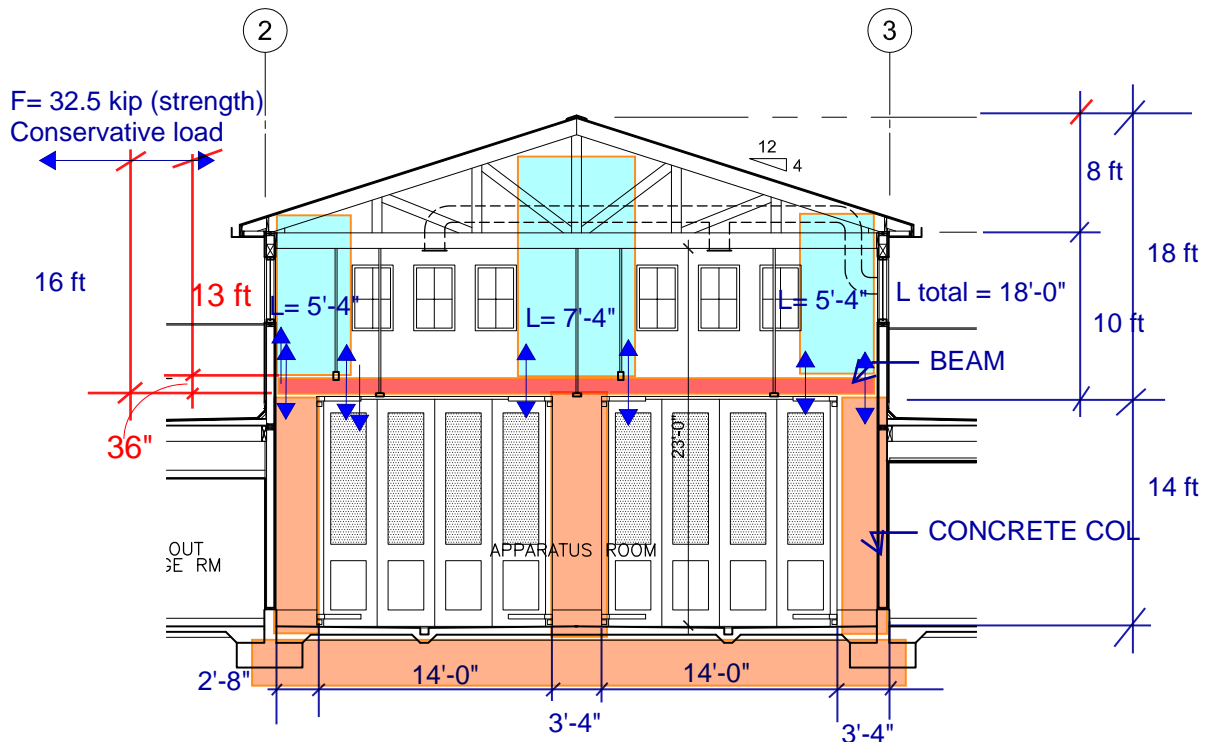
SHEAR WALL ELEVATION NEAR GRID J **C**
SCALE: 1/4" = 1'-0"



CHECK SHEAR WALL EA END OF APPARATUS ROOM

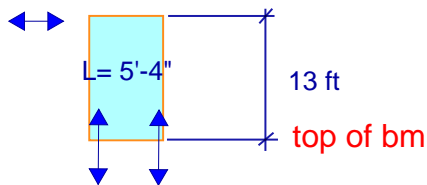
$$F = V/2 * 1.3 = 22.3 * 1.3 \text{ kip} = 29 \text{ kip (strength)}$$

rho = 1.3



$$v = F/L = 29 \text{ kip} * 1000/18 \text{ ft} = 1611 \text{ lb/ft (LRFD)} \quad 1127 \text{ lb/ft (ASD)}$$

$$F1 = 1611 \text{ lb/ft} * 5'-4" = 8538 \text{ lb LRFD}$$

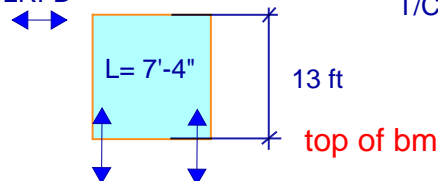


$$T/C = 8538 \text{ lb} * 13 \text{ ft} / 4'-4" = 22 \text{ kip LRFD}$$

$$= 22 \text{ kip} / 1.4 = 15.7 \text{ kip (ASD)}$$

Double SW w/ HHQ14-SD2.5, Ta = 13.7 kip

$$F1 = 1611 \text{ lb/ft} * 7'-4" = 11760 \text{ lb LRFD}$$

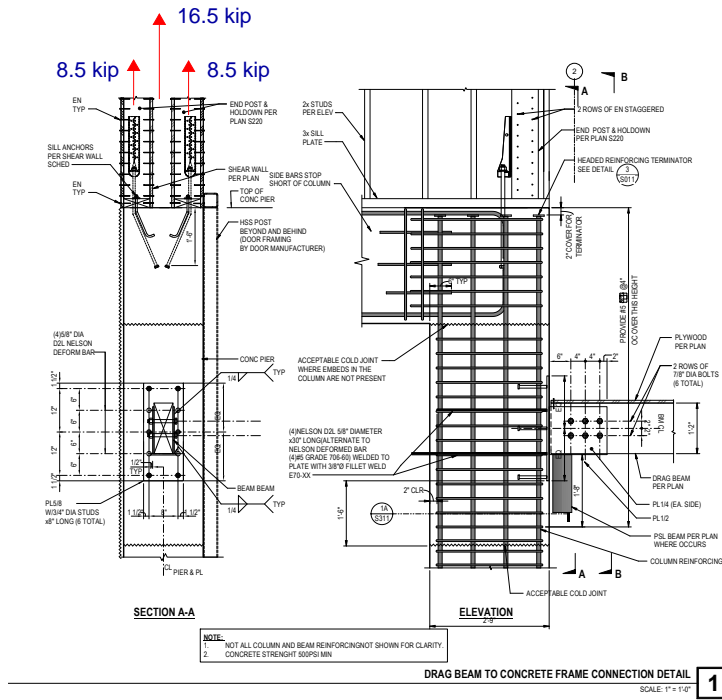


$$T/C = 11760 \text{ lb} * 13 \text{ ft} / 6'-4" = 23.5 \text{ kip (LRFD)}$$

$$= 23.5 \text{ kip} / 1.4 = 16.5 \text{ kip (ASD)} < 2 * 13.7 \text{ kip} = 27.4 \text{ kip}$$

F	2	1320 PLF	1/2	3x	3x
---	---	----------	-----	----	----

$$v \text{ max} = 1127 \text{ lb/ft (ASD)} < V_a = 1320 \text{ lb/ft DCR} = 1127 \text{ lb/ft} / 1320 \text{ lb/ft} = 0.85$$



Loading for Shear Wall anchorage Design:

1" Diameter $F_y=36$ ksi $T_u= 1.2 F_y \cdot A_g= 1.2 \cdot 36 \text{ ksi} \cdot 0.79 \text{ in}^2= 34 \text{ kip}$.

Wood Holdown $T_u= 13.7 \text{ kip} \cdot 1.4= 19 \text{ kip}$ (Design For this Failure first)

Omega Load= $(3.0-1/2) 8.5 \cdot 1.4= 29 \text{ kip}$
By inspection from typical footing AB, OK

2.5 DIAPHRAGM DESIGN

12.10.1.1 Diaphragm Design Forces. Floor and roof diaphragms shall be designed to resist design seismic forces from the structural analysis but shall not be less than that determined in accordance with Eq. (12.10-1) as follows:

$$F_{px} = \frac{\sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px} \quad (12.10-1)$$

$$= 0.369 W$$

where

F_{px} = the diaphragm design force at level x ;

F_i = the design force applied to level i ;

w_i = the weight tributary to level i ; and

w_{px} = the weight tributary to the diaphragm at level x .

The force determined from Eq. (12.10-1) shall not be less than

$$F_{px} = 0.2 S_{DS} I_e w_{px} \quad (12.10-2)$$

$$= 0.2 * 1.6 * 1.5 W = 0.48 W \text{ (governed)}$$

The force determined from Eq. (12.10-1) need not exceed

$$F_{px} = 0.4 S_{DS} I_e w_{px} \quad (12.10-3)$$

Diaphragm Force = $(0.48/0.368) = 1.3$ Base Shear Force

Diaphragm Design in Y direction (North-South direction)

Diaphragm at Apparatus Room

L=112 ft

W= 38 ft

L/W= 112 ft/38 ft= 2.9 < 4 (Wood structural panel, blocked, table 4.2.4) OK

4.2.4 Diaphragm Aspect Ratios

Size and shape of diaphragms shall be limited to the aspect ratios in Table 4.2.4.

Table 4.2.4 Maximum Diaphragm Aspect Ratios

(Horizontal or Sloped Diaphragms)

Diaphragm Sheathing Type	Maximum L/W Ratio
Wood structural panel, unblocked	3:1
Wood structural panel, blocked	4:1
Single-layer straight lumber sheathing	2:1
Single-layer diagonal lumber sheathing	3:1
Double-layer diagonal lumber sheathing	4:1

4.2.3 Unit Shear Capacities

Tabulated nominal unit shear capacities for seismic design are provided in Column A of Tables 4.2A, 4.2B, 4.2C, and 4.2D; and for wind design in Column B of Tables 4.2A, 4.2B, 4.2C, and 4.2D. The ASD allowable unit shear capacity shall be determined by dividing the tabulated nominal unit shear capacity, modified by applicable footnotes, by the ASD reduction factor of 2.0. The LRFD factored unit resistance shall be determined by multiplying the tabulated nominal unit shear capacity, modified by applicable footnotes, by a resistance factor, ϕ_D , of 0.80. No further increases shall be permitted.

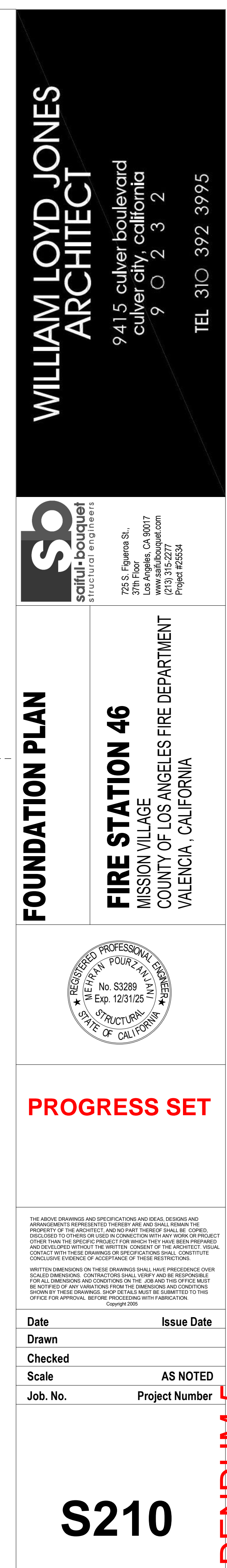
Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms**Blocked Wood Structural Panel Diaphragms^{1,2,3,4,5}**

					A SEISMIC												B WIND							
					Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)												Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)							
					6			4			2-1/2			2			6		4		2-1/2		2	
					Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)												Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)							
					6			6			4			3			6		6		4		3	
Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	v_s	G_a	v_s	G_a	v_s	G_a	v_s	G_a	v_w	v_w	v_w	v_w								
					(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(plf)	(plf)	(plf)								
Structural I	6d	1-1/4	5/16	2		OSB		OSB		OSB		OSB		520	700	1050	1175							
				3		PLY		PLY		PLY		PLY		590	785	1175	1330							
					370	15	12	500	8.5	7.5	750	12	10	840	20	15								
	8d	1-3/8	3/8	2	420	12	9.5	560	7.0	6.0	840	9.5	8.5	950	17	13	755	1010	1485	1680				
				3	540	14	11	720	9.0	7.5	1060	13	10	1200	21	15	840	1120	1680	1890				
					600	12	10	800	7.5	6.5	1200	10	9.0	1350	18	13								
Sheathing and Single-Floor	6d	1-1/4	5/16	2	640	24	17	850	15	12	1280	20	15	1460	31	21	895	1190	1790	2045				
				3	720	20	15	960	12	9.5	1440	16	13	1640	26	18	1010	1345	2015	2295				
					340	15	10	450	9.0	7.0	670	13	9.5	760	21	13	475	630	940	1065				
			3/8	2	380	12	9.0	500	7.0	6.0	760	10	8.0	860	17	12	530	700	1065	1205				
				3	370	13	9.5	500	7.0	6.0	750	10	8.0	840	18	12	520	700	1050	1175				
					420	10	8.0	560	5.5	5.0	840	8.5	7.0	950	14	10	590	785	1175	1330				
	8d	1-3/8	3/8	2	480	15	11	640	9.5	7.5	960	13	9.5	1090	21	13	670	895	1345	1525				
				3	540	12	9.5	720	7.5	6.0	1080	11	8.5	1220	18	12	755	1010	1510	1710				
					510	14	10	680	8.5	7.0	1010	12	9.5	1150	20	13	715	950	1415	1610				
			7/16	2	570	11	9.0	760	7.0	6.0	1140	10	8.0	1290	17	12	800	1065	1595	1805				
				3	540	13	9.5	720	7.5	6.5	1060	11	8.5	1200	19	13	755	1010	1485	1680				
					600	10	8.5	800	6.0	5.5	1200	9.0	7.5	1350	15	11	840	1120	1680	1890				
	10d	1-1/2	15/32	2	580	25	15	770	15	11	1150	21	14	1310	33	18	810	1080	1610	1835				
				3	650	21	14	860	12	9.5	1300	17	12	1470	28	16	910	1205	1820	2060				
					640	21	14	850	13	9.5	1280	18	12	1460	28	17	895	1190	1790	2045				
			19/32	2	720	17	12	960	10	8.0	1440	14	11	1640	24	15	1010	1345	2015	2295				
				3																				

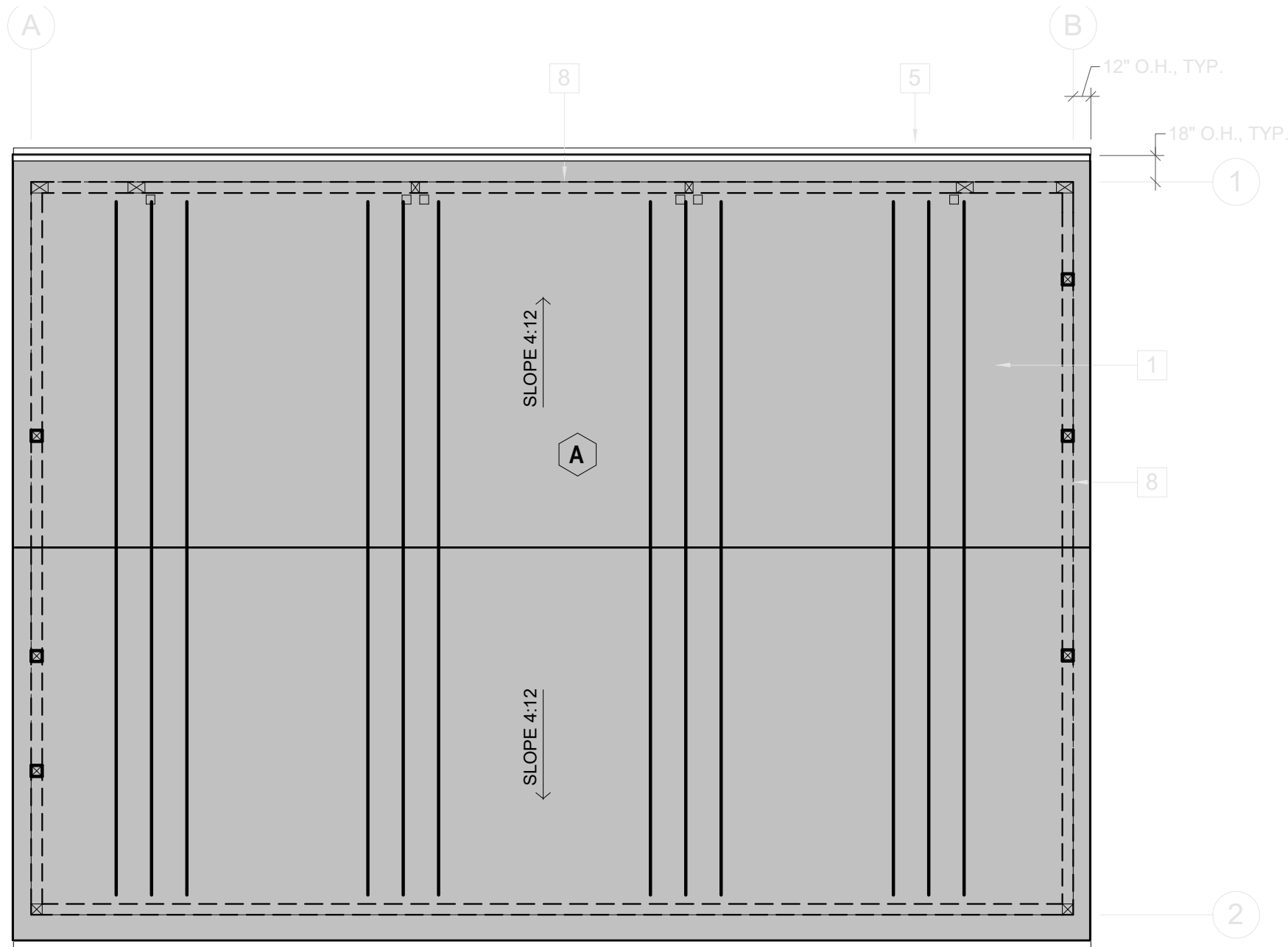
- Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
- For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = $[1 - (0.5 - G)]$, where G = Specific Gravity of the framing lumber from the *NDS* (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
- Apparent shear stiffness values, G_a , are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be multiplied by 1.2.
- Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
- Diaphragm resistance depends on the direction of continuous panel joints with respect to the loading direction and direction of framing members, and is independent of the panel orientation.

	Cases 1&3: Continuous Panel Joints Perpendicular to Framing	Cases 2&4: Continuous Panel Joints Parallel to Framing	Cases 5&6: Continuous Panel Joints Perpendicular and Parallel to Framing
Long Panel Direction Perpendicular to Supports			
Long Panel Direction Parallel to Supports ^a			

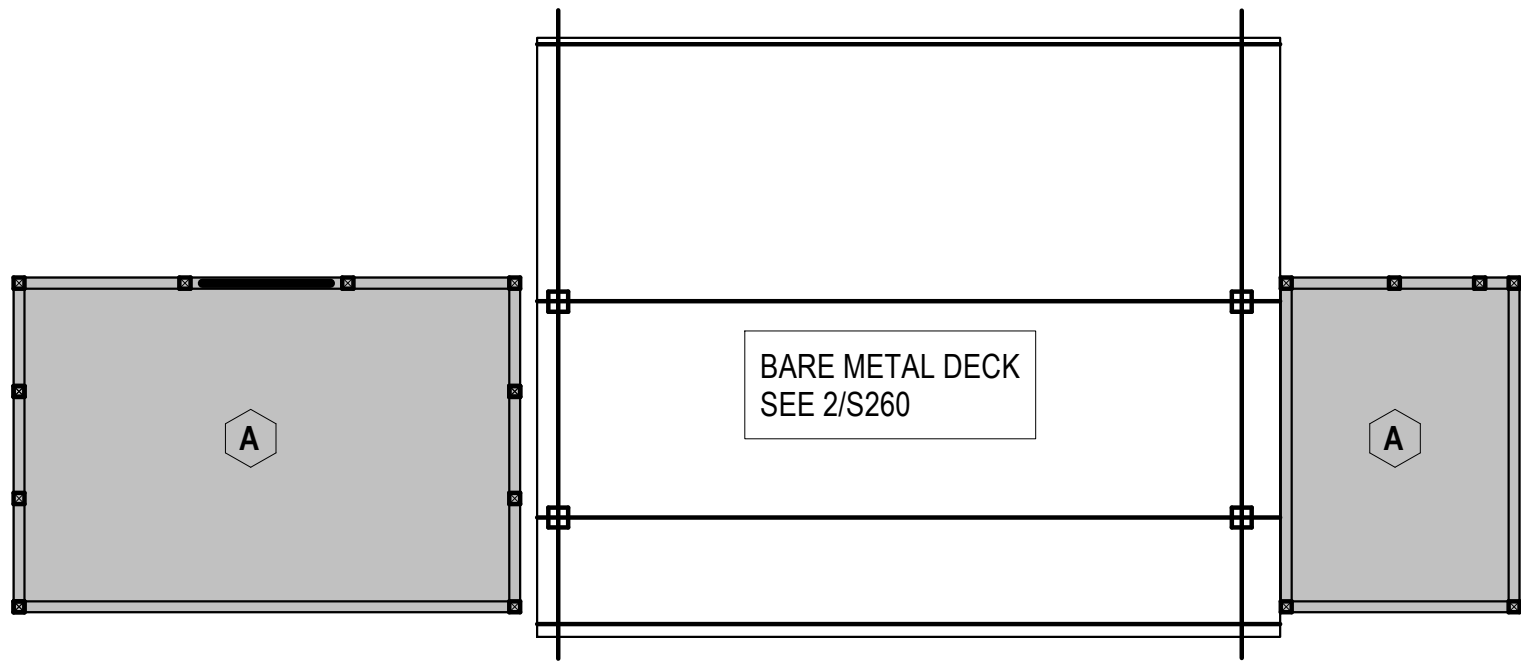
(a) Panel span rating for out-of-plane loads may be lower than the span rating with the long panel direction perpendicular to supports (See Section 3.2.2 and Section 3.2.3)



8/7/2025 2:52:05 PM C:\WORK\FIRE STATION 46\01 MODEL\25534_FIRE STATION 46_S_R22.rvt



ROOF AT RESERVE APPARATUS BUILDING DIAPRHAGM PLAN



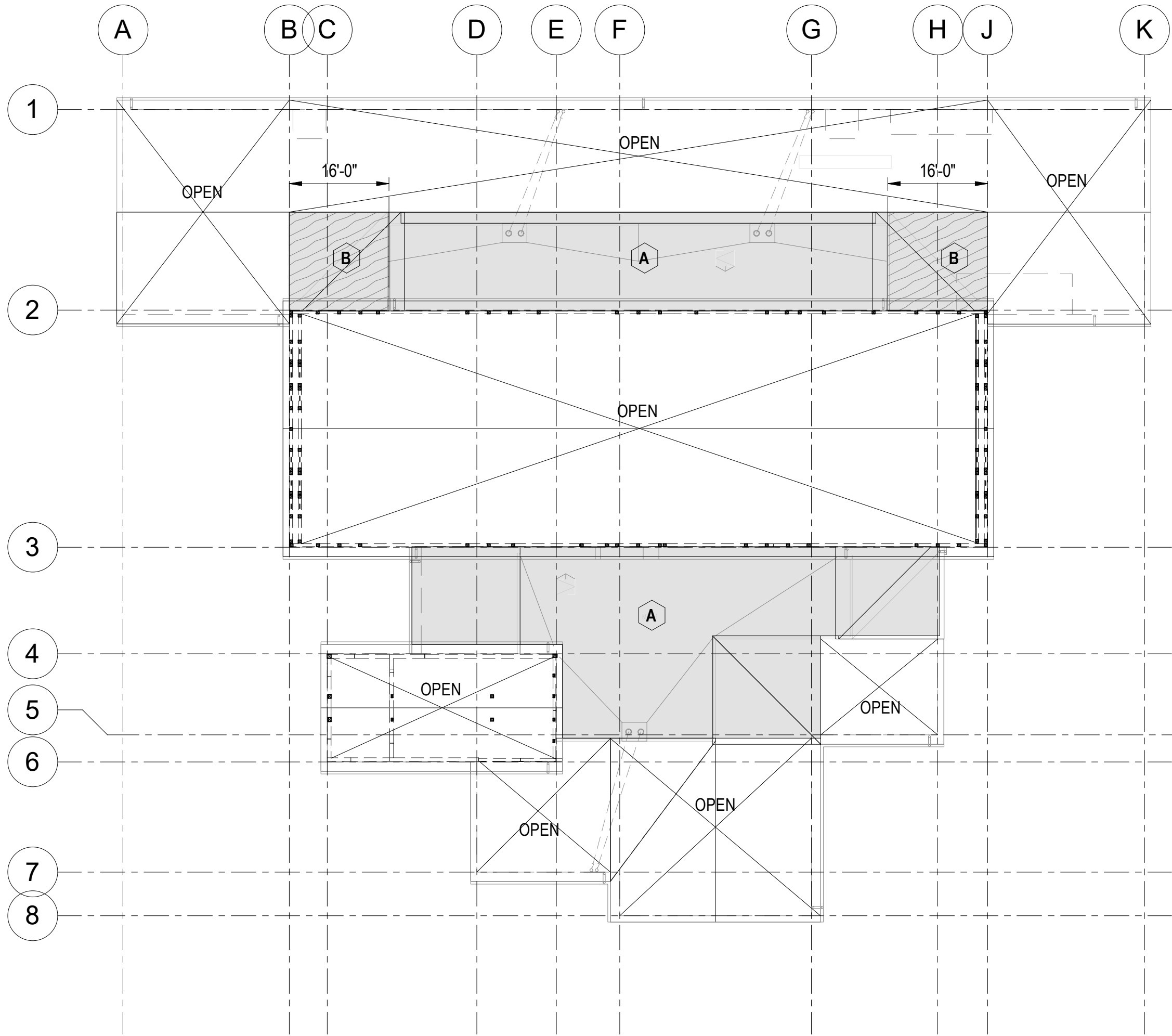
ROOF AT FUEL DISPENSING AREA DIAPRHAGM PLAN

FRAMING PLAN NOTES

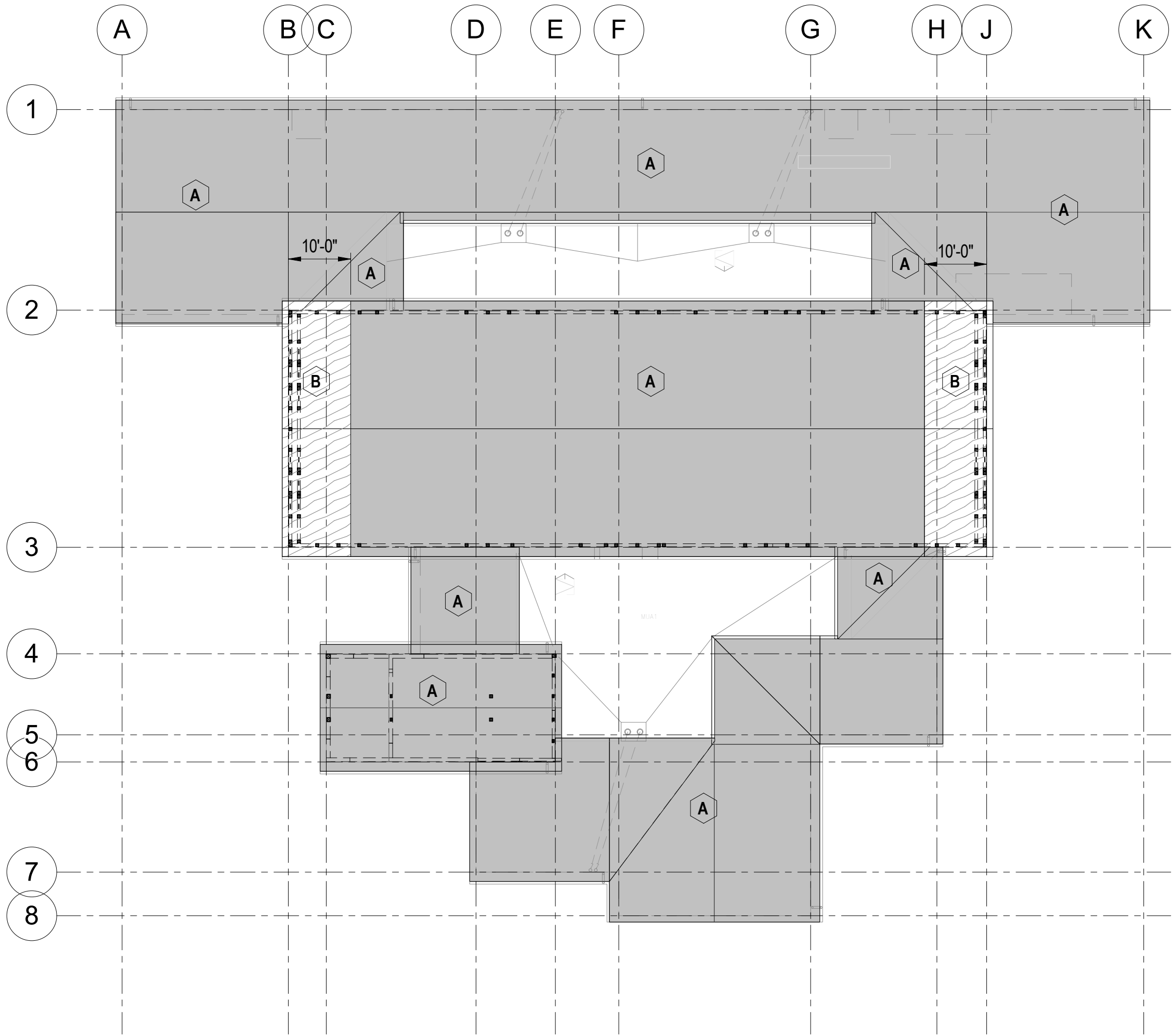
1. ROOF DIAPHRAGM SHEATHING AND NAILING SHALL BE AS FOLLOWS:

- TYPE **A** : 19/32" THICK CDX PLYWOOD (BLOCKED DIAPHRAGM)
10d@4" BN, 10d@6" EN, 10d@12" FN
- TYPE **B** : 19/32" THICK CDX PLYWOOD (BLOCKED DIAPHRAGM)
10d@2 1/2" BN, 10d@4" EN, 10d@12" FN
(USE DOUBLE TRUSSES AT ADJOINING PANELS AND BOUNDARY NAILING)

2. SEE DETAIL **3** FOR REMAINDER OF INFO.



FLAT ROOF AT MAIN BUILDING DIAPHRAGM PLAN



SLOPED ROOF AT MAIN BUILDING DIAPHRAGM PLAN

WILLIAM LOYD JONES
ARCHITECT

9415 culver boulevard
culver city, california
9 0 2 3 2

TEL 310 392 3995

SO
sciful-bouquet
structural engineers

725 S. Figueroa St.,
37th floor
Los Angeles, CA 90017
www.sciful-bouquet.com
(213) 315-2277
Project #25534

ROOF WOOD DIAPHRAGM
PLANS

FIRE STATION 46

MISSION VILLAGE

COUNTY OF LOS ANGELES FIRE DEPARTMENT
VALENCIA, CALIFORNIA



THE ABOVE DRAWINGS AND SPECIFICATIONS AND IDEAL DESIGNS AND ARRANGEMENTS REPRESENTED THEREBY ARE AND SHALL REMAIN THE PROPERTY OF THE ARCHITECT AND NO PART THEREOF SHALL BE COPIED, DISCLOSED TO OTHERS OR USED IN CONNECTION WITH ANY WORK OR PROJECT OTHER THAN THE SPECIFIC PROJECT FOR WHICH THEY HAVE BEEN PREPARED AND DEVELOPED WITHOUT THE WRITTEN CONSENT OF THE ARCHITECT. VERBAL CONTACT WITH THESE DRAWINGS OR SPECIFICATIONS SHALL CONSTITUTE CONCLUSIVE EVIDENCE OF ACCEPTANCE OF THESE INSTRUCTIONS.

WRITTEN DIMENSIONS ON THESE DRAWINGS SHALL HAVE PRECEDENCE OVER SCALED DIMENSIONS. CONTRACTORS SHALL VERIFY AND BE RESPONSIBLE FOR ALL DIMENSIONS AND CONDITIONS ON THE JOB AND THIS OFFICE MUST BE NOTIFIED OF ANY VARIATIONS FROM THE DIMENSIONS AND CONDITIONS SHOWN IN THESE DRAWINGS. SHOP DETAILS MUST BE SUBMITTED TO THIS OFFICE FOR APPROVAL BEFORE PROCEEDING WITH FABRICATION.

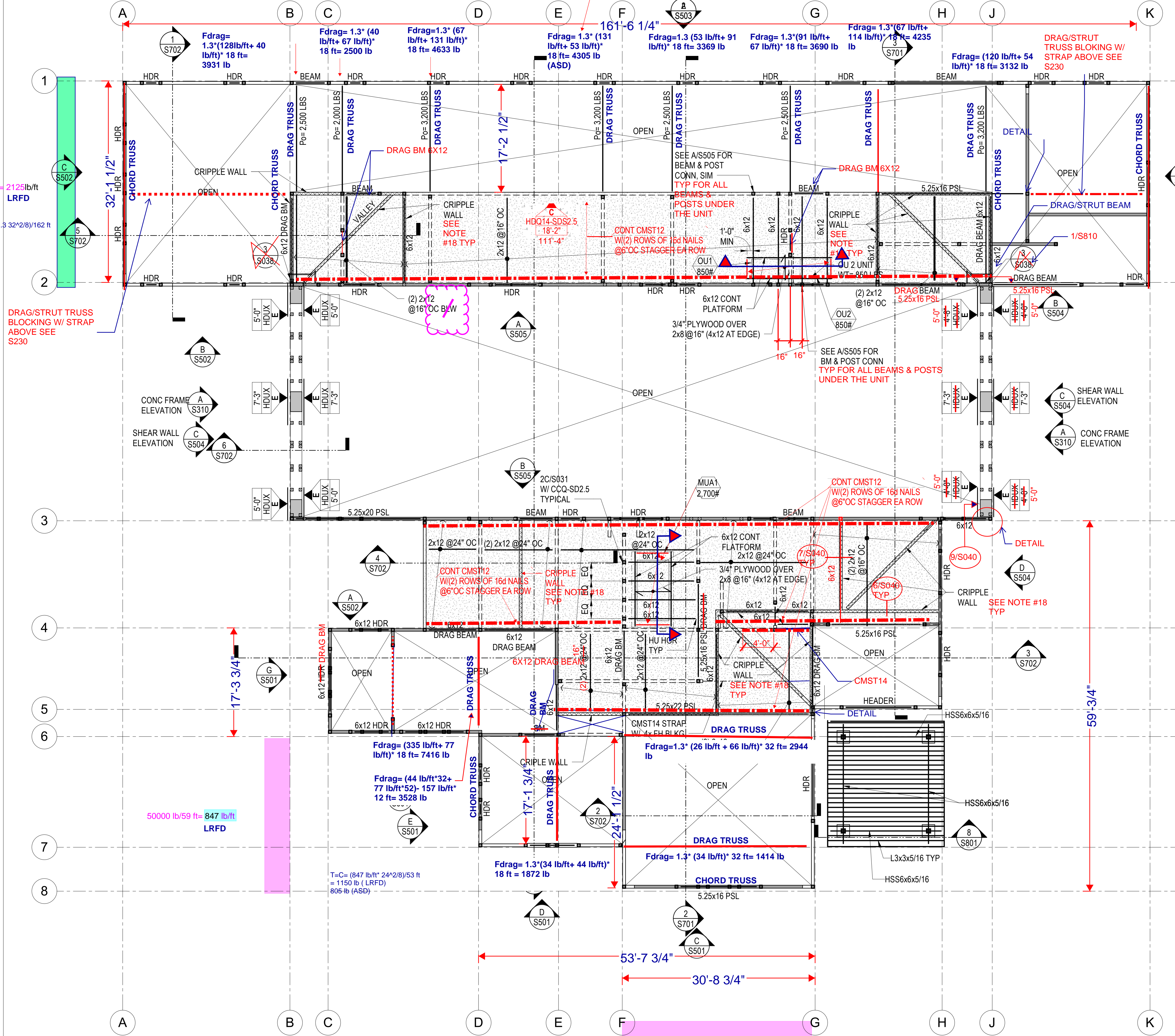
Date	Issue Date
Drawn	
Checked	
Scale	AS NOTED
Job. No.	Project Number

S240

10% DESIGN DEVELOPMENT SUBMITTAL 05/20/2025

2.6 CHORD DESIGN

$V \text{ dia}/Vsw=$
 $0.48W/0.669W= 1.3$



FRAMING PLAN NOTES

- FOR GENERAL NOTES SEE S0.0 SERIES AND TYPICAL DETAILS SEE S1.0 SERIES SHEETS.
- VERIFY ALL DIMENSIONS PRIOR TO START OF WORK. SEE ARCHITECTURAL DRAWINGS FOR REMAINDER OF DIMENSIONS NOT SHOWN ON THIS PLAN.
- WHERE A SHEAR WALL IS SHOWN AS PORTION OF A WALL, THE REMAINDER OF THE WALL (INCLUDING ABOVE AND BELOW OPENINGS, PARAPETS, ETC) SHALL BE SHEATHED WITH THE SAME SHEATHING AND NAILING SCHEDULE ALONG THE ENTIRE WALL. IN ADDITION, CORNERS OF ALL OPENINGS IN SHEAR WALL SHALL BE STRAPPED.
- ALL SHEAR WALLS AND SHEAR TRANSFER NAILS SHALL BE COMMON WIRE NAILS. SINKER AND BOX NAILS ARE NOT PERMITTED.
- HOLDOWN CONNECTORS SHALL BE TIGHTENED JUST PRIOR TO COVERING THE WALL FRAMING.
- INDICATES BEARING AND/OR SHEAR WALL BELOW.
- ROOF DIAPHRAGM SHEATHING AND NAILING SHALL BE PER SHEET S240
- TYPICAL WOOD STUD WALL: SEE NOTE #8 ON SHEET S210
- ALL HARDWARE ARE BY "SIMPSON" TYPICAL OR APPROVED EQUAL.
- ALL SAWN WOOD FRAMING EXPOSED TO WEATHER SHALL BE P.T.D.F.
- HDR INDICATES HEADER PER SCHEDULE SEE DETAIL (1) S030 UNO.
- INDICATES ROOF JOIST
- ROOF DIAPHRAGM NAILING TO BE INSPECTED BEFORE COVERING, FACE GRAIN OF PLYWOOD SHALL BE PERPENDICULAR TO SUPPORT.
- ALL NAILS ARE COMMON NAILS, UNO.
- CMST. INDICATES SIMPSON CMST14 STRAP, UNO
- INDICATES SHEAR WALL TYPE PER SCHEDULE
- INDICATES HOLDOWN TYPE EACH END OF SHEAR WALL PANEL SEE SCHEDULE (1) S030 FOR HD HOLDOWNS (1) S030
- INDICATES SHEAR WALL DESIGN LENGTH
- SEE MECH DWGS FOR EQUIPMENT MOUNTED ON FLAT ROOFS, SEE FOR TYPICAL MECHANICAL UNIT PLATFORM.
- INDICATES 2x6 STUD WALL @16"OC (CRIPPLE WALL)
- Po= INDICATES AXIAL SEISMIC DRAG STRUT LOAD (SERVICE) IN LBS. CONTRACTOR TO DESIGN TRUSS TO RESIST THESE LOADS IN COMBINATION WITH GRAVITY LOADS IN ACCORDANCE WITH APPLICABLE BUILDING CODE AND ALL LISTED CRITERIA. WHERE REQUIRED BY ANALYSIS, PROVIDE DOUBLE OR MORE TRUSSES
- INDICATES MECHANICAL UNIT TYPE
- INDICATES MECHANICAL UNIT OPERATING WT IN LBS
- T1 INDICATES ROOF WOOD TRUSSES AER DESIGN-BUILT. REFER TO S0.38 SERIES FOR TRUSS PROFILE AND GENERAL NOTES FOR DESIGN CRITERIA, TYP.
- SOLAR PANEL, FRAMING & THEIR CONNECTIONS TO STRUCTURAL MEMBERS BY OTHERS

DRAG & CHORD LOADS IN
DIAPHRAGM

FLAT ROOF FRAMING PLAN

SCALE: 1/8" = 1'-0"

1

WILLIAM LOYD JONES
ARCHITECT

9415 culver boulevard
culver city, california
90232

TEL 310 392 3995

SO
saiful-bouquet
structural engineers

726 S. Figueroa St.,
3rd floor
Los Angeles, CA 90017
www.saifulbouquet.com
(213) 315-2277
Project #25534

LOW ROOF FRAMING PLAN

FIRE STATION 46

MISSION VILLAGE
COUNTY OF LOS ANGELES FIRE DEPARTMENT
VALENCIA, CALIFORNIA



THE ABOVE DRAWINGS AND SPECIFICATIONS AND ALL DESIGN AND CONSTRUCTION DETAILS ARE THE PROPERTY OF THE ARCHITECT AND NO PART THEREOF SHALL BE COPIED, REPRODUCED, OR USED IN CONNECTION WITH ANY WORK OR PROJECT OTHER THAN THE SPECIFIC PROJECT FOR WHICH THEY HAVE BEEN PREPARED. ANY SUCH VIOLATION SHALL BE CONSIDERED A VIOLATION OF THE ARCHITECT'S PROFESSIONAL ETHICS AND SHALL BE SUBJECT TO LEGAL ACTION. WRITTEN DIMENSIONS ON THESE DRAWINGS SHALL HAVE PRECEDENCE OVER SCALED DIMENSIONS. CONTRACTORS SHALL VERIFY AND BE RESPONSIBLE FOR ALL DIMENSIONS AND CONDITIONS ON THE JOB AND THE OFFICE MUST BE NOTIFIED OF ANY VARIATIONS FROM THE DIMENSIONS AND CONDITIONS SHOWN IN THESE DRAWINGS. SHOP DETAILS MUST BE SUBMITTED TO THE OFFICE FOR APPROVAL, BEFORE PROCEEDING WITH FABRICATION.

Date	Issue Date
Drawn	
Checked	
Scale	AS NOTED
Job. No.	Project Number

S220

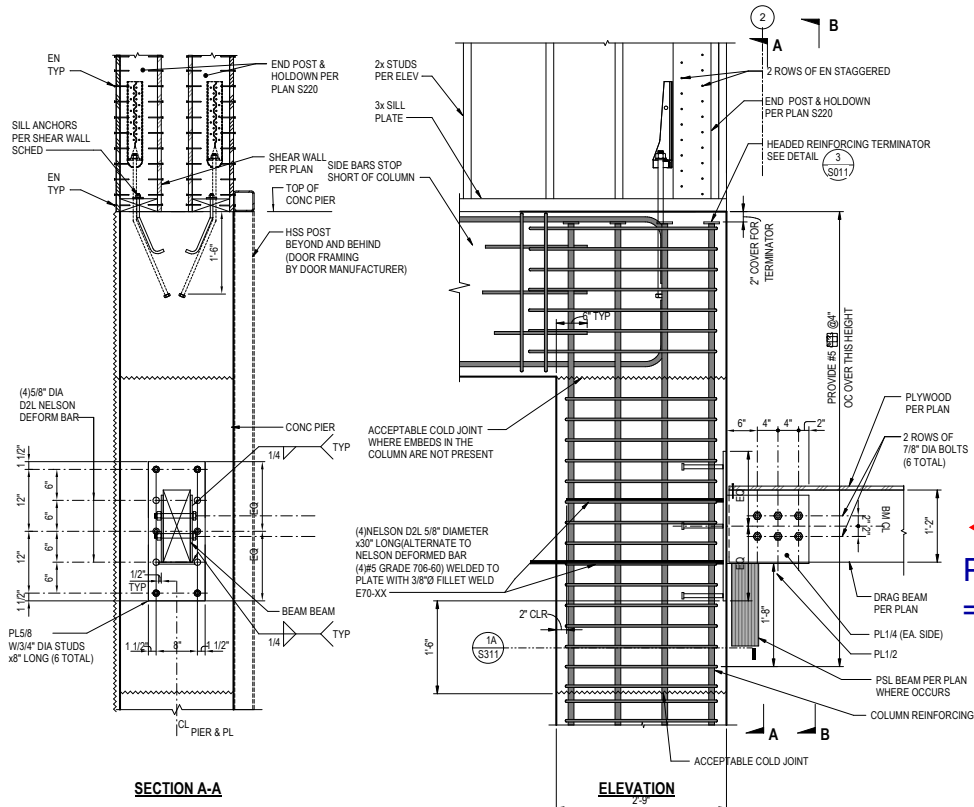
10% DESIGN DEVELOPMENT SUBMITTAL 5/20/2025

2.7 DRAG DESIGN



6X12 Drag BM DCR= 0.2 OK

3C
YP



$P_o = 3500 \text{ lb ASD}$
 $= 4900 \text{ LB LRFD}$

NOTE:
1. NOT ALL COLUMN AND BEAM REINFORCING NOT SHOWN FOR CLARITY.
2. CONCRETE STRENGTH 5000 PSI MIN

DRAG BEAM TO CONCRETE FRAME CONNECTION DETAIL

SCALE: 1" = 1'-0"

1

Checking shear in 6x12

3/4" diameter MB

$$V_a = 6 * (3340 \text{ lb/bolt}) * 1.6 = 32 \text{ kip} > F = 3.5 \text{ kip OK}$$

Checking dowels

5/8" DIA rebars

$$T_a = 4 * 0.6 * 36 \text{ ksi} * 0.31 \text{ in}^2 = 26 \text{ kip} > F = 3.5 \text{ kip OK}$$

Check welding

$$T \text{ welding} = 0.928 * 6 * 3.14 * 0.625 * 3 = 32 \text{ kip} >> F = 3.5 \text{ kip}$$

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Column

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: DRAG BEAM-1

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
 Load Combinations Used : ASCE 7-16

General Information

Analysis Method		Allowable Stress Design		Wood Section Name		6x12		
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber		
Overall Column Height		16 ft		Wood Member Type		Sawn		
(Used for non-slender calculations)								
Wood Species		Douglas Fir-Larch		Exact Width		5.50 in		
Wood Grade		No.1		Exact Depth		11.50 in		
Fb +		1,200.0 psi	Fv	170.0 psi	Area	63.250 in^2	Allow Stress Modification Factors	
Fb -		1,200.0 psi	Ft	825.0 psi	Ix	697.07 in^4	Cf or Cv for Bending	1.0
Fc - Prll		1,000.0 psi	Density	31.210 pcf	Iy	159.443 in^4	Cf or Cv for Compression	1.0
Fc - Perp		625.0 psi					Cf or Cv for Tension	1.0
							Cm : Wet Use Factor	1.0
							Ct : Temperature Fact	1.0
							Cfu : Flat Use Factor	1.0
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			Kf : Built-up columns	1.0
		Basic	1,600.0	1,600.0	1,600.0 ksi		Use Cr : Repetitive ?	No
		Minimum	580.0	580.0				
Brace condition for deflection (buckling) along columns :								
X-X (width) axis : Unbraced Length for buckling ABOUT Y-Y Axis = 4 ft, K								
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 16 ft, K								

Applied Loads

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

Column self weight included : 219.337 lbs * Dead Load Factor

AXIAL LOADS . . .

Axial Load at 16.0 ft, E = 4.90 k

BENDING LOADS . . .

Lat. Uniform Load creating Mx-x, D = 0.0260, LR = 0.0670 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.1977 : 1	Maximum SERVICE Lateral Load Reactions . .			
	Load Combination	+D+Lr	Top along Y-Y	0.7440 k	Bottom along Y-Y 0.7440 k	
	Governing NDS Forumla Comp + Mxx, NDS Eq. 3.9-3		Top along X-X	0.0 k	Bottom along X-X 0.0 k	
	Location of max.above base	8.054 ft	Maximum SERVICE Load Lateral Deflections . . .			
	At maximum location values are .		Along Y-Y	0.1243 in at 8.054 ft	above base	
	Applied Axial	0.2193 k	for load combination : +D+Lr			
	Applied Mx	2.976 k-ft	Along X-X	0.0 in at 0.0 ft	above base	
	Applied My	0.0 k-ft	for load combination : n/a			
	Fc : Allowable	983.69 psi	Other Factors used to calculate allowable stresses . . .			
				<u>Bending</u>	<u>Compression</u>	<u>Tension</u>
PASS	Maximum Shear Stress Ratio =	0.08303 : 1				
	Load Combination	+D+Lr				
	Location of max.above base	0.0 ft				
	Applied Design Shear	17.644 psi				
	Allowable Shear	212.50 psi				

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.859	0.07666	PASS	7.946 ft	0.03224	PASS	0.0 ft
+D+Lr	1.250	0.787	0.1977	PASS	8.054 ft	0.08303	PASS	0.0 ft
+D+0.750Lr	1.250	0.787	0.1621	PASS	7.946 ft	0.06808	PASS	0.0 ft
+0.60D	1.600	0.714	0.02592	PASS	7.946 ft	0.01088	PASS	0.0 ft
+D+0.70E	1.600	0.714	0.05056	PASS	7.946 ft	0.01814	PASS	0.0 ft
+D+0.5250E	1.600	0.714	0.04578	PASS	7.946 ft	0.01814	PASS	0.0 ft
+0.60D+0.70E	1.600	0.714	0.04933	PASS	7.946 ft	0.01088	PASS	0.0 ft

Wood Column

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: DRAG BEAM-1

Maximum Reactions

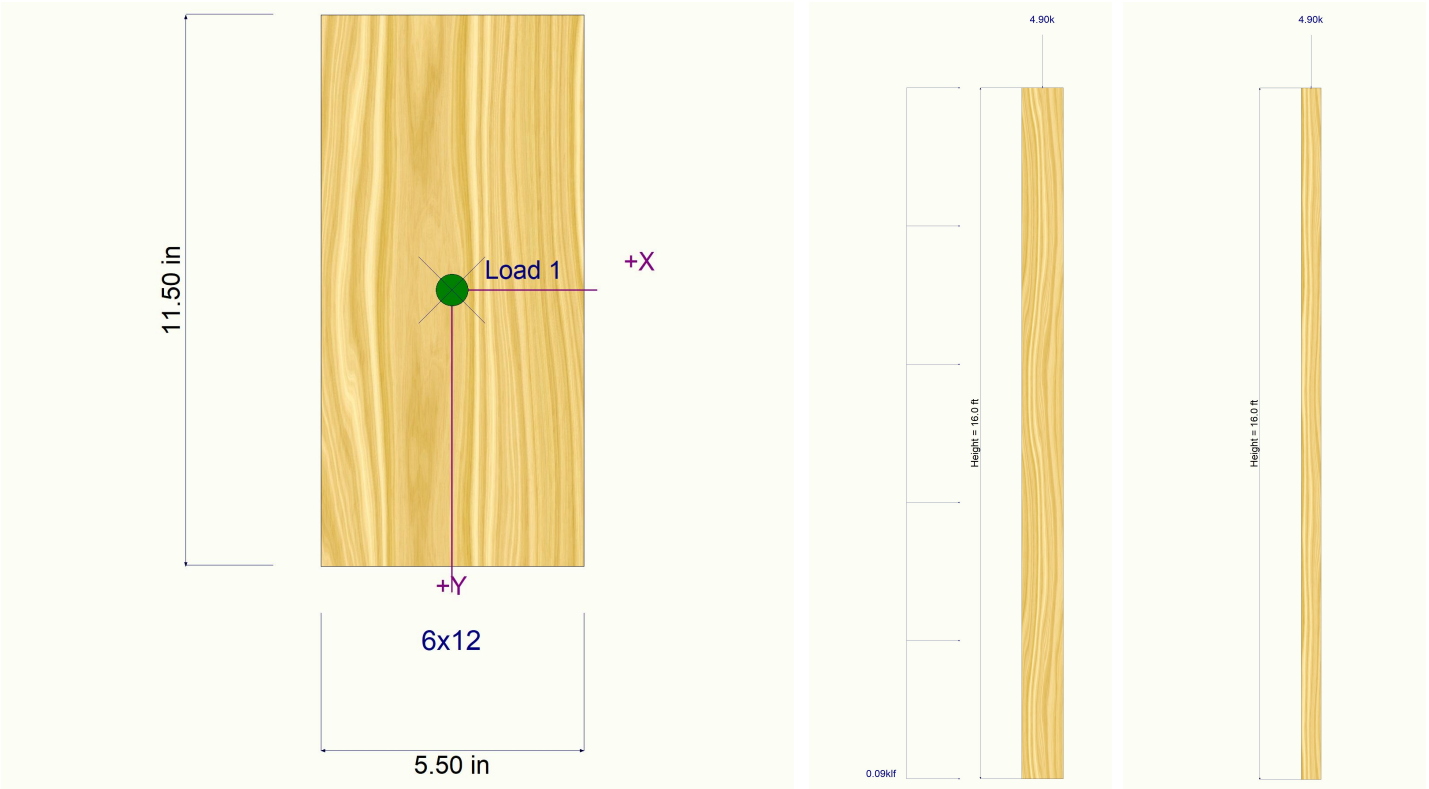
Note: Only non-zero reactions are listed.

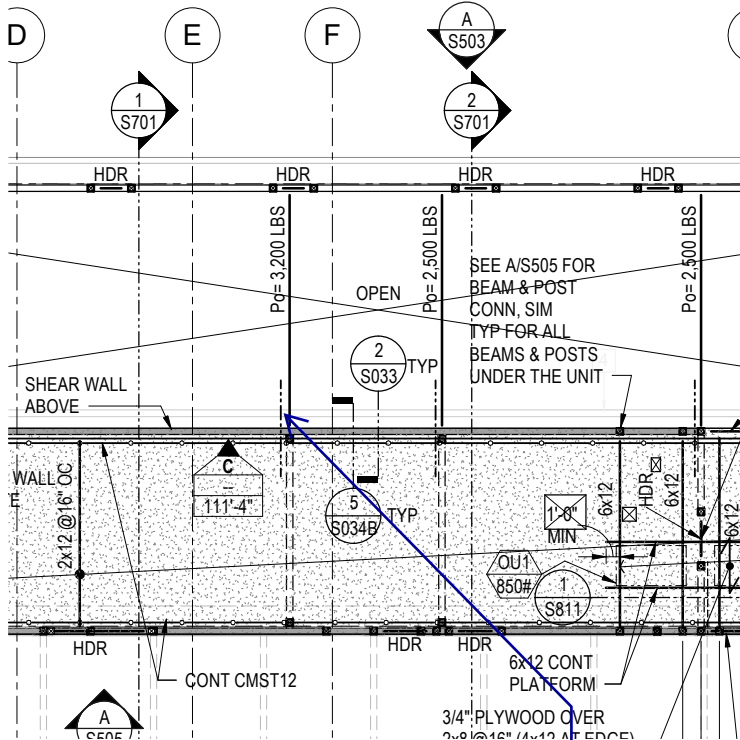
Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
D Only				0.208	0.208	0.219					
+D+Lr				0.744	0.744	0.219					
+D+0.750Lr				0.610	0.610	0.219					
+0.60D				0.125	0.125	0.132					
+D+0.70E				0.208	0.208	3.649					
+D+0.5250E				0.208	0.208	2.792					
+0.60D+0.70E				0.125	0.125	3.562					
Lr Only				0.536	0.536						
E Only						4.900					

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000ft	0.035 in	8.054 ft
+D+Lr	0.0000 in	0.000ft	0.124 in	8.054 ft
+D+0.750Lr	0.0000 in	0.000ft	0.102 in	8.054 ft
+0.60D	0.0000 in	0.000ft	0.021 in	8.054 ft
+D+0.70E	0.0000 in	0.000ft	0.035 in	8.054 ft
+D+0.5250E	0.0000 in	0.000ft	0.035 in	8.054 ft
+0.60D+0.70E	0.0000 in	0.000ft	0.021 in	8.054 ft
Lr Only	0.0000 in	0.000ft	0.090 in	8.054 ft
E Only	0.0000 in	0.000ft	0.000 in	0.000 ft

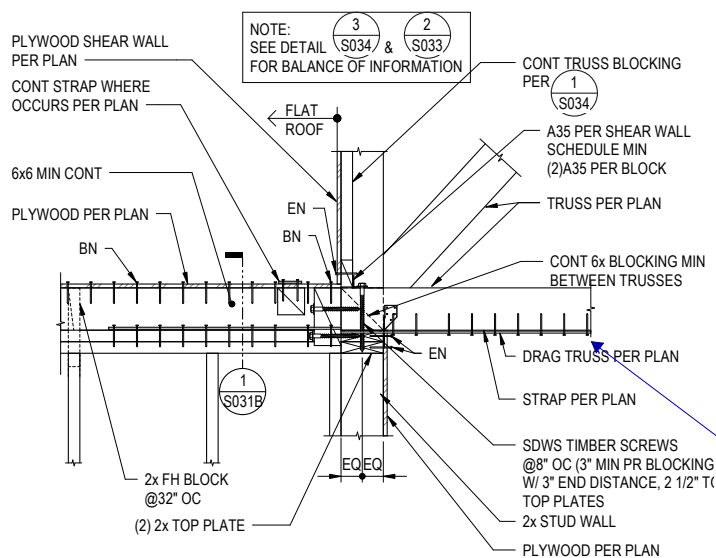
Sketches





**DRAG TRUSS TO
SW CONNECTION**

23. ----- INDICATES DRAG/STRUT STRAP AT DRAG BEAM / TRUSS OR BLOCKS AND EXTENT PER PLAN. (1) AT BEAMS & (3) AT TRUSSES CMST14 UNO



CMST14 $T_a = 6475 \text{ lb} > F = 3200 \text{ LB OK}$

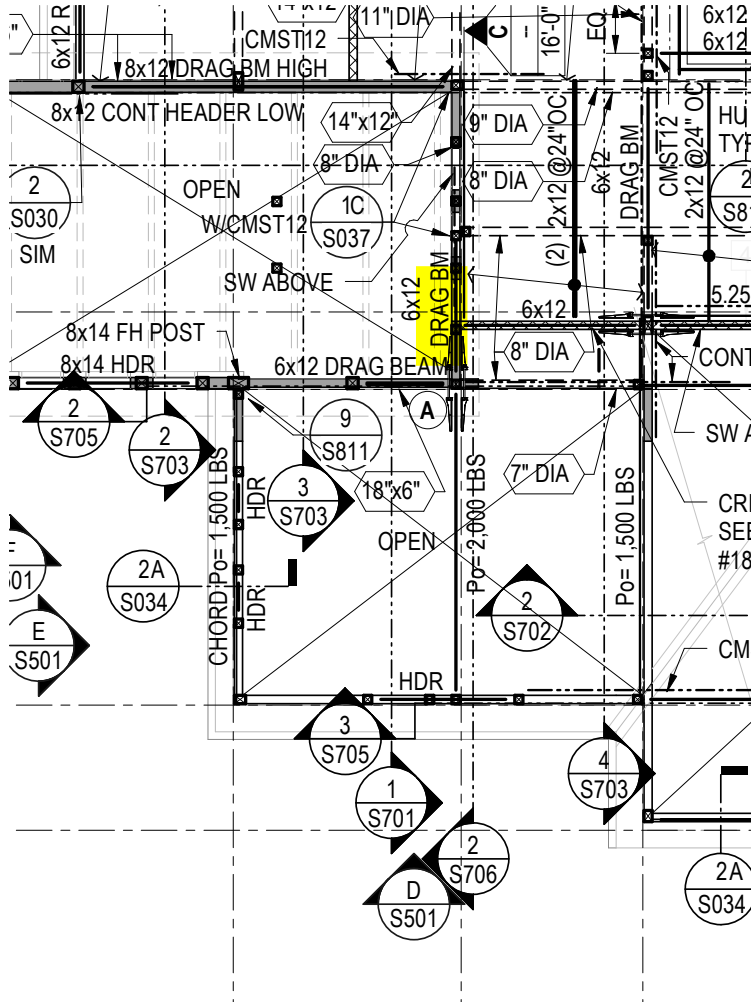
**TYPICAL DRAG /STRUT TRUSS
CONNECTION AT TRANSITION TO FLAT ROOF**

SCALE: NTS

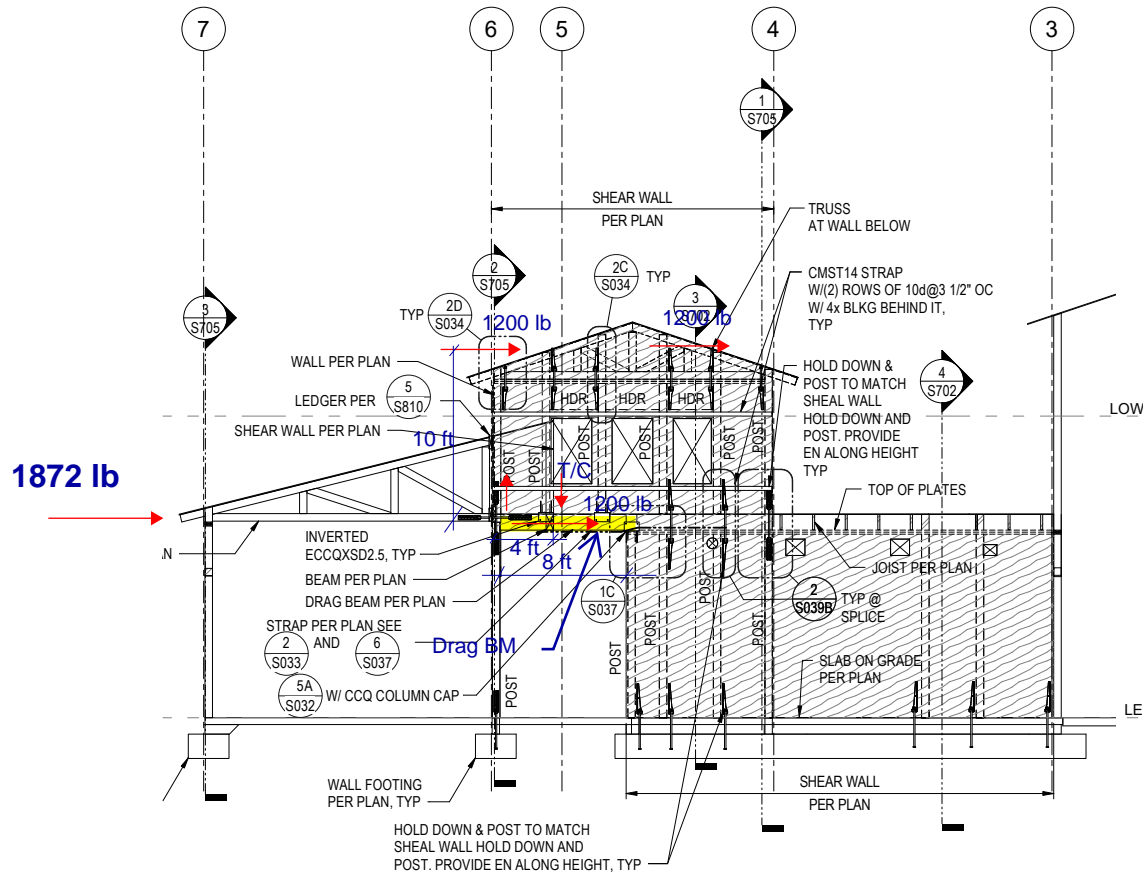
5

ADDENDUM 5

DRAG BM ON 2/S706



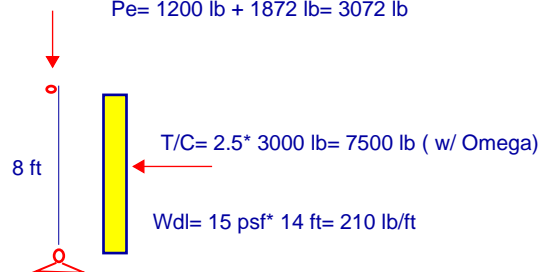
DRAG BM ON 2/S706 CONT



RIIII NING SECTII

$$T/C = 1200 \text{ lb} \cdot 10 \text{ ft} / 4 \text{ ft} = 3000 \text{ lb}$$

$$P_e = 1200 \text{ lb} + 1872 \text{ lb} = 3072 \text{ lb}$$



Wood Column

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: DRAG BM ON 2/S706

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
 Load Combinations Used : IBC 2021

General Information

Analysis Method :		Allowable Stress Design		Wood Section Name		6x12		
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber		
Overall Column Height		8 ft		Wood Member Type		Sawn		
(Used for non-slender calculations)								
Wood Species		Douglas Fir-Larch		Exact Width		5.50 in		
Wood Grade		No.1		Exact Depth		11.50 in		
Fb +		1200 psi	Fv	170 psi	Area	63.250 in^2	Cf or Cv for Bending	1.0
Fb -		1200 psi	Ft	825 psi	Ix	697.07 in^4	Cf or Cv for Compression	1.0
Fc - Prll		1000 psi	Density	31.21 pcf	Iy	159.443 in^4	Cf or Cv for Tension	1.0
Fc - Perp		625 psi					Cm : Wet Use Factor	1.0
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			Ct : Temperature Factor	1.0
		Basic	1600	1600	1600 ksi		Cfu : Flat Use Factor	1.0
		Minimum	580	580			Kf : Built-up columns	1.0 NDS 15.3.2
							Use Cr : Repetitive ?	No
Brace condition for deflection (buckling) along columns :								
				X-X (width) axis :	Unbraced Length for buckling ABOUT Y-Y Axis = 8 ft, K =			
				Y-Y (depth) axis :	Unbraced Length for buckling ABOUT X-X Axis = 8 ft, K =			

Applied Loads

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

Column self weight included : 109.668 lbs * Dead Load Factor

AXIAL LOADS . . .

Axial Load at 8.0 ft, E = 3.10 k

BENDING LOADS . . .

Lat. Uniform Load from 0.0-->8.0 ft creating Mx-x, D = 0.210 k/ft

Lat. Point Load at 4.0 ft creating Mx-x, E = 7.50 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.6364 : 1**
 Load Combination +D+0.70E
 Governing NDS Formula Comp + Mxx, NDS Eq. 3.9-3
 Location of max.above base 3.973 ft
 At maximum location values are . . .
 Applied Axial 2.280 k
 Applied Mx 12.109 k-ft
 Applied My 0.0 k-ft
 Fc : Allowable 1,093.22 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	3.750 k	Bottom along Y-Y	3.750 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y	0.1252 in	at	4.027 ft	above base
for load combination : E Only				
Along X-X	0.0 in	at	0.0 ft	above base
for load combination : n/a				

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension
---------	-------------	---------

PASS Maximum Shear Stress Ratio = **0.3021 : 1**
 Load Combination +D+0.70E
 Location of max.above base 0.0 ft
 Applied Design Shear 82.174 psi
 Allowable Shear 272.0 psi

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.842	0.1550	PASS	4.027 ft	0.1302	PASS	0.0 ft
+D+0.70E	1.600	0.683	0.6364	PASS	3.973 ft	0.3021	PASS	0.0 ft
+D+0.5250E	1.600	0.683	0.4986	PASS	3.973 ft	0.2449	PASS	0.0 ft
+0.60D	1.600	0.683	0.05262	PASS	3.973 ft	0.04394	PASS	0.0 ft
+0.60D+0.70E	1.600	0.683	0.6011	PASS	3.973 ft	0.2728	PASS	0.0 ft

Wood Column

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: DRAG BM ON 2/S706

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
D Only				0.840	0.840	0.110					
+D+0.70E				3.465	3.465	2.280					
+D+0.5250E				2.809	2.809	1.737					
+0.60D				0.504	0.504	0.066					
+0.60D+0.70E				3.129	3.129	2.236					
E Only				3.750	3.750	3.100					

Maximum Deflections for Load Combinations

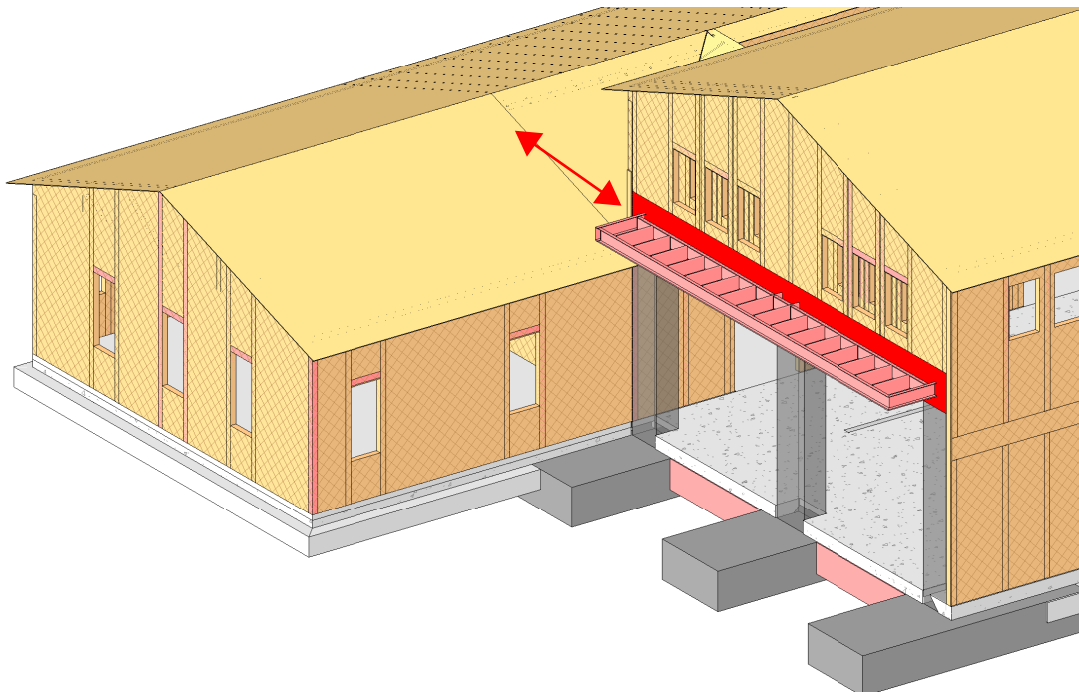
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000ft	0.018 in	4.027 ft
+D+0.70E	0.0000 in	0.000ft	0.105 in	4.027 ft
+D+0.5250E	0.0000 in	0.000ft	0.083 in	4.027 ft
+0.60D	0.0000 in	0.000ft	0.011 in	4.027 ft
+0.60D+0.70E	0.0000 in	0.000ft	0.098 in	4.027 ft
E Only	0.0000 in	0.000ft	0.125 in	4.027 ft

Sketches



2.8 CONCRETE MOMENT FRAME DESIGN

IN PLAN SHEAR DESIGN



Concrete framing Design:

1. Drift $3/8"$ at 14 ff high (see page #134 drift criteria, cal pages 136 & 145)
2. Strong column /weak beams (page #217)
3. Grade beam Design (see page 287)

Item No. : Criteria Section G – “Structural Design Guidelines and Criteria”, Paragraph C.4.d.

Delete : Existing Paragraph numbering as follows:

d. Other Design Considerations

- iii. Design structure to resist lateral earth pressures and surcharge loads and seismic increment when applicable.
- iv. Design external (perimeter) components of subsystem either directly or indirectly exposed to weather to resist applicable potential ambient air differential temperature range from 30°F to 120°F with adequate provisions for noiseless movement in expansion and contraction and prevention of binding, buckling, joint-opening, breakage or undue stress in and between members.
- v. Welded wire fabric shall not be used as reinforcement on this project.
- vi. Deflections:
 - Design the structure to limit deflections to meet code, component criteria, or to the following minimums:
 - Roof live load deflection L/360 of span. Total load deflection L/240 of span.
 - Limit live load deflection of members supporting exterior walls as required by code for different finishes.
 - The structural system and its components shall meet the requirements for noise and vibration. Refer to Chapter R, for noise and vibration.
 - Building separation: The separation between two adjacent buildings shall be based on combined maximum inelastic response displacement plus 1/2".
 - The inelastic lateral drift along the apparatus bay's door openings shall be limited to 3/8" inch maximum at 14 feet above the finish floor under the design lateral force.
- vii. Design/Builder to consider in the design, issues related to crack control, corrosion protection of concrete and steel and decay termite protection of wood elements.

Add portion: New Paragraph numbering as follows:

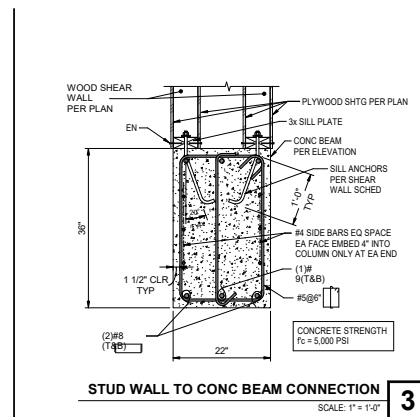
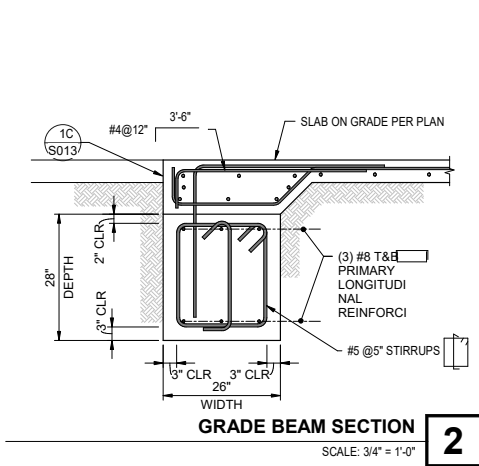
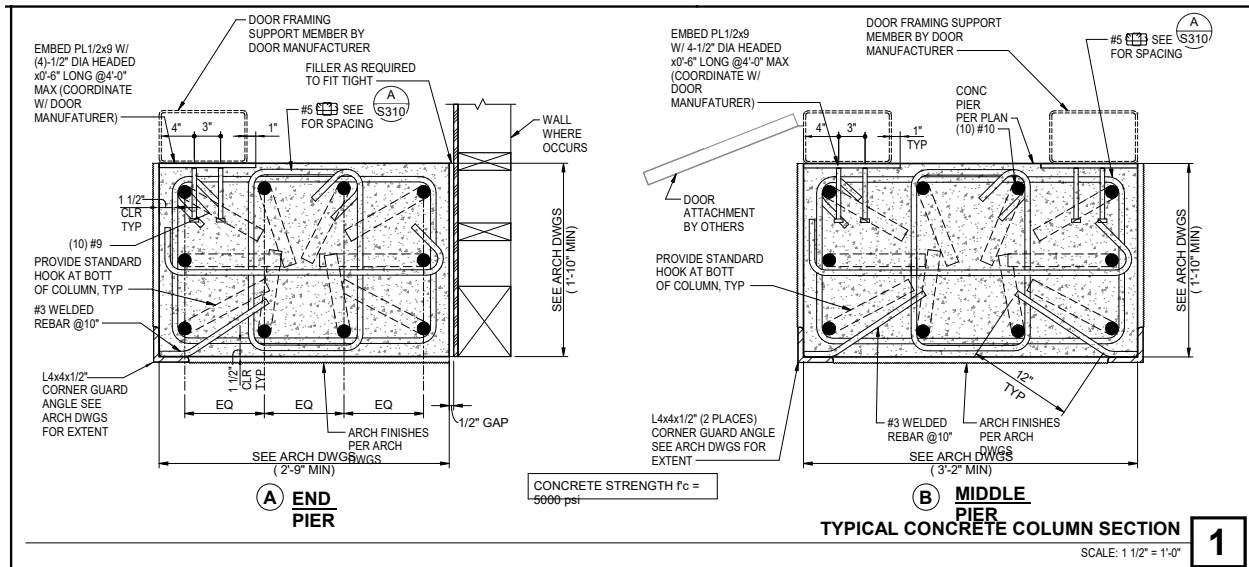
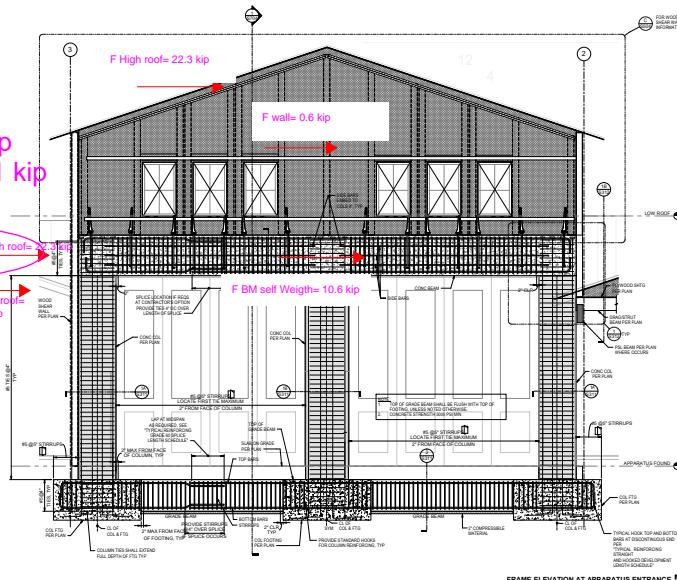
5. Other Design Considerations

- a. Design structure to resist lateral earth pressures and surcharge loads and seismic increment when applicable.
- b. Design external (perimeter) components of subsystem either directly or indirectly exposed to weather to resist applicable potential ambient air differential temperature range from 30°F to 120°F with adequate provisions for noiseless movement in expansion and contraction and

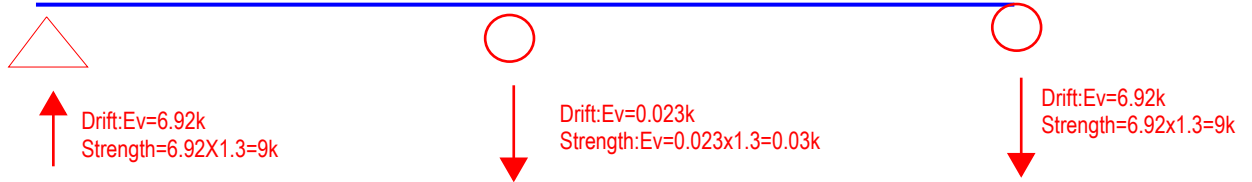
ALL SEISMIC LOAD IS LRFD

F total = (28.5kip
+10.6kip) = 39.1 kip
F = 28.5 kip

F wall = 0.6 kip
F Col = 5.6 kip
F High roof = 22.3 kip
F low roof = 3.7 kip



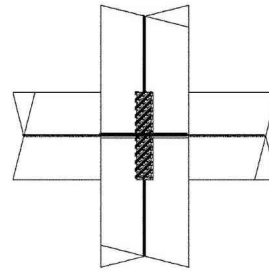
Beam Reactions From SAP2000



10.4.2.2 Stiffness of Reinforced Concrete Beam-Column Moment Frames

10.4.2.2.1 Linear Static and Dynamic Procedures. Beams shall be modeled considering flexural and shear stiffnesses, including the effect of the slab acting as a flange in monolithic construction per Section 10.3.1.3. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Refer to Section 10.3.1.2 to compute the effective stiffnesses. Where joint stiffness is not modeled explicitly, it shall be permitted to be modeled implicitly by adjusting a centerline model (Fig. 10-2):

1. For $\Sigma M_{ColE}/\Sigma M_{BE} > 1.2$, column offsets are rigid and beam offsets are not;
2. For $\Sigma M_{ColE}/\Sigma M_{BE} < 0.8$, beam offsets are rigid and column offsets are not; and
3. For $0.8 \leq \Sigma M_{ColE}/\Sigma M_{BE} \leq 1.2$, half of the beam and column offsets are considered rigid.



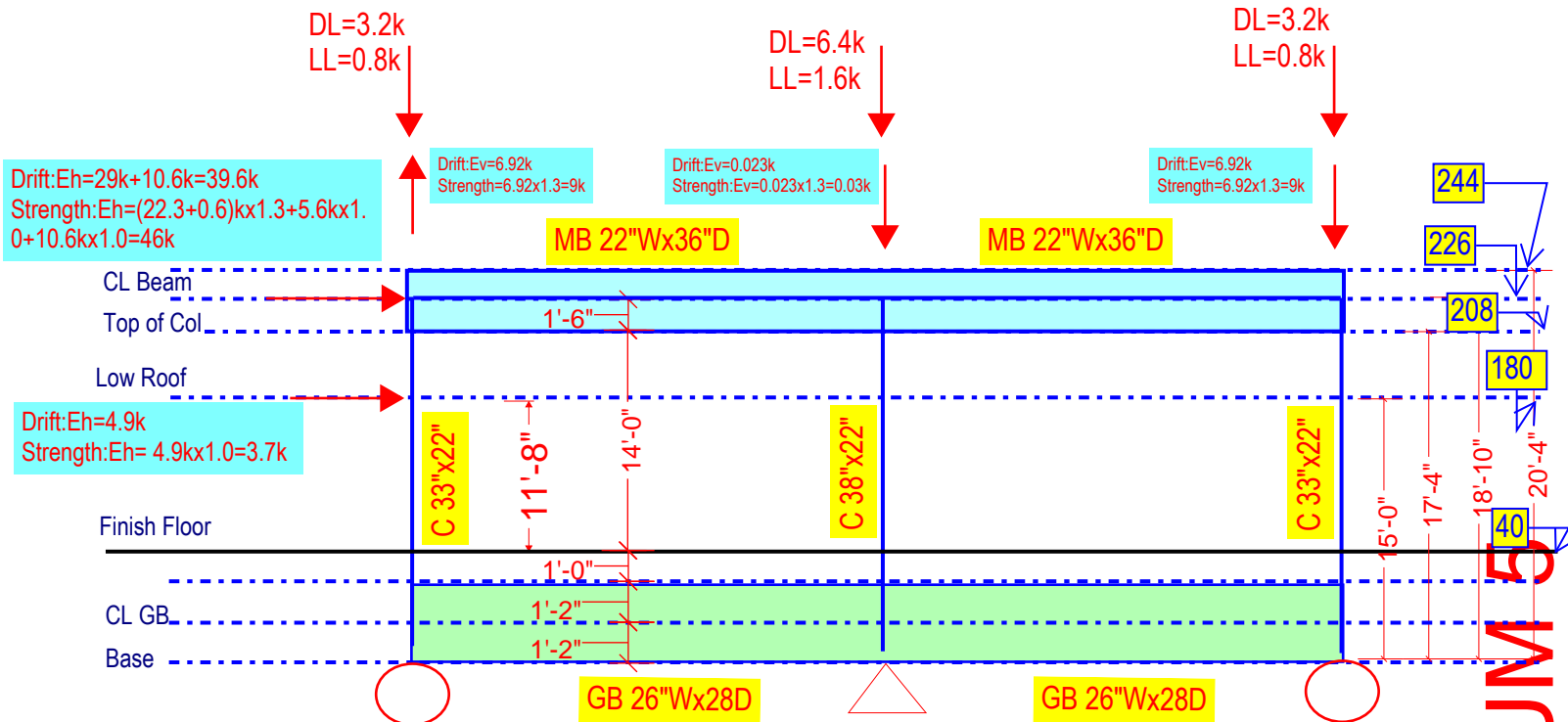
(c) $\Sigma M_{nd}/\Sigma M_{nb} > 1.2$

Cs=0.369 site class C

Rigid End Zone =
100% for column
0% for beam
ASCE41-17
10.4.2.2

F'c = 5000psi
Column: 0.5lg
Beam: 0.5lg

Seismic Loads are applied in both direction in ETABS



Elastic Drift = 0.127 in
Cd=4,
Ie = 1.5
Inelastic Drift = $4 \times 0.127 / 1.5 = 0.339 \text{ in} < 3/8" = 0.375" \dots \dots \text{OK}$

Beam Weight = $150 \text{ pcf} \times 36" / 12 \times 22" / 12 \times 34' - 5" = 28.5 \text{ kip}$
Beam Eh = $W \times Cs = 0.369 \times 28.5 \text{ k} = 10.6 \text{ kip}$

$F_a =$	1.20	Table 11.4-1
$F_v =$	1.40	Table 11.4-2
$S_{MS} = F_a S_s =$	2.716	Eq. 11.4-1
$S_{M1} = F_v S_1 =$	1.144	Eq. 11.4-2
$S_{DS} = (2/3) S_{MS} =$	1.601	Eq. 11.4-3
$S_{D1} = (2/3) S_{M1} =$	0.763	Eq. 11.4-4

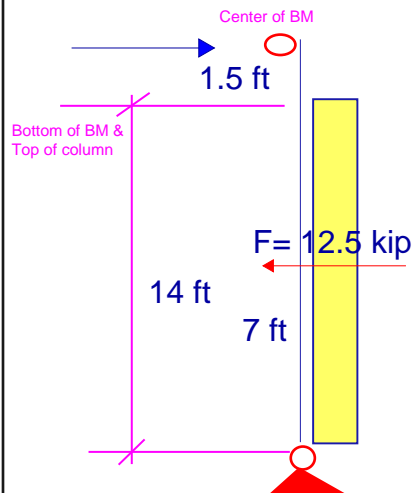
Finalized

Columns Self Weight Seismic load

3 concrete columns

$$F = 12.5 \text{ kip} \times 7 \text{ ft} / (15.5 \text{ ft}) = 5.6 \text{ kip}$$

$$150 \text{ pcf} \times ((35''/12 \times 22''/12 \times 14 \text{ ft}) \times 2 + 38''/12 \times 22''/12 \times 14) = 150 \times (149 + 81) = \underline{\underline{34 \text{ kip}}}$$



$$\text{Column Self Seismic} = (34 \text{ kip}) \times 0.369 = 12.5 \text{ kip (LRFD)}$$

$$\text{Concrete Beam self seismic} = 10.6 \text{ kip (LRFD)}$$

$$\text{Concrete Frame Self Weight Seismic} = 12.5 \text{ kip} + 10.6 \text{ kip} = 23 \text{ kip/frame}$$

Low Roof Seismic load to MF

$$436 \text{ lb/ft} \times 24 \text{ ft} / 2 = 5014 \text{ lb Line J (LRFD)} = 3581 \text{ lb (ASD)}$$

$$436 \text{ lb/ft} \times 34 \text{ ft} / 2 = 7739 \text{ lb Line B (LRFD)} = 5527 \text{ lb (ASD)}$$

At Line B: Assume all low roof go to wood shear wall = $510 \text{ lb/ft} \times 14 \text{ ft} = 7.1 \text{ kip (ASD)} \gg 5.5 \text{ kip}$

At Line J: Since wood shear walls are designed for low roof. Conservatively 75 % load to MF is $3.5 \text{ kip} \times 0.75 = 2.6 \text{ kip (ASD)}$, 3.7 kip (LRFD)

High Roof Seismic Load To MF

$$22.3 \text{ kip (ASD)}, 22.3 \text{ kip (LRFD)}$$

Wood wall above MF

$$\text{Total Wall Area} = 4 \times 38 = 152 \text{ sf}$$

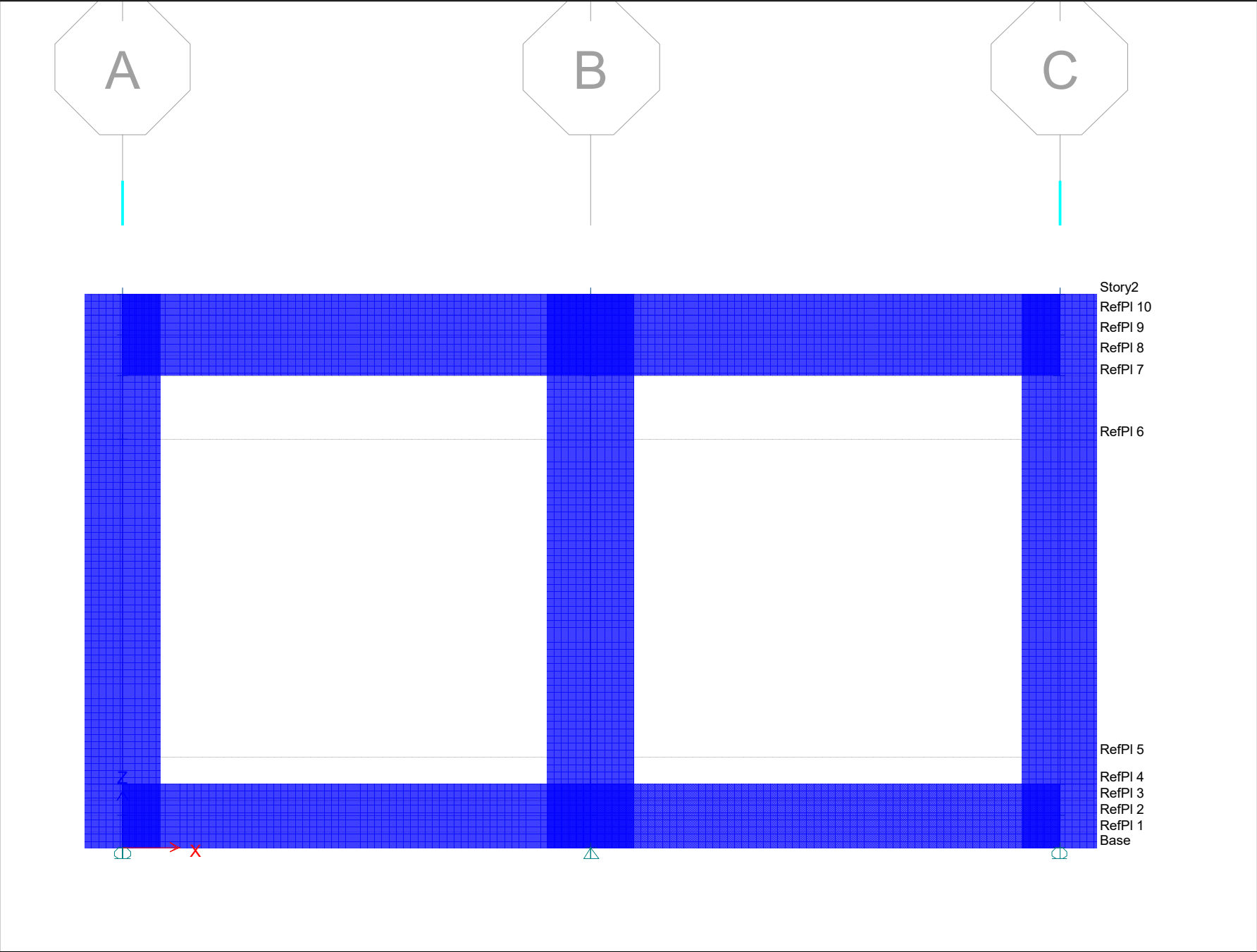
$$\text{Total Window} = 60 \text{ sf}$$

$$\text{Solid wall} = 152 \text{ sf} - 60 \text{ sf} = 92 \text{ sf}$$

$$W = 92 \text{ sf} \times 15 \text{ psf} + 60 \text{ sf} \times 5 \text{ psf} = 1680 \text{ lb}$$

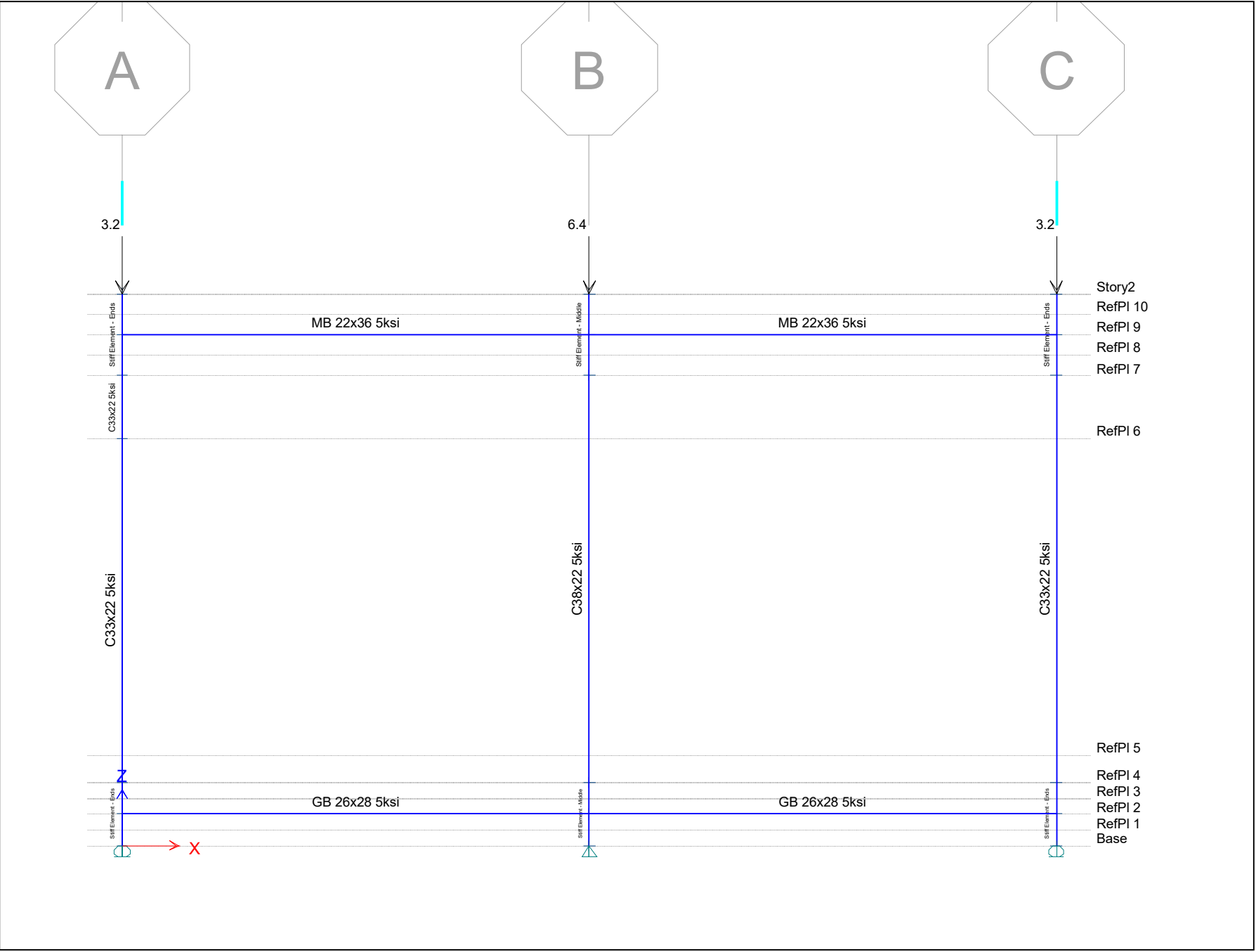
$$F = 0.369 \times 1680 \text{ lb} = 0.6 \text{ kip}$$

$$\begin{aligned} \text{Force to Beam} &= F \text{ roof} + (F \text{ low roof} + F \text{ column Weight}) + F \text{ wall at beam} \\ &= 22.3 \text{ kip} + 5.6 \text{ kip} + 0.6 \text{ kip} = \underline{\underline{28.5 \text{ kip}}} \end{aligned}$$



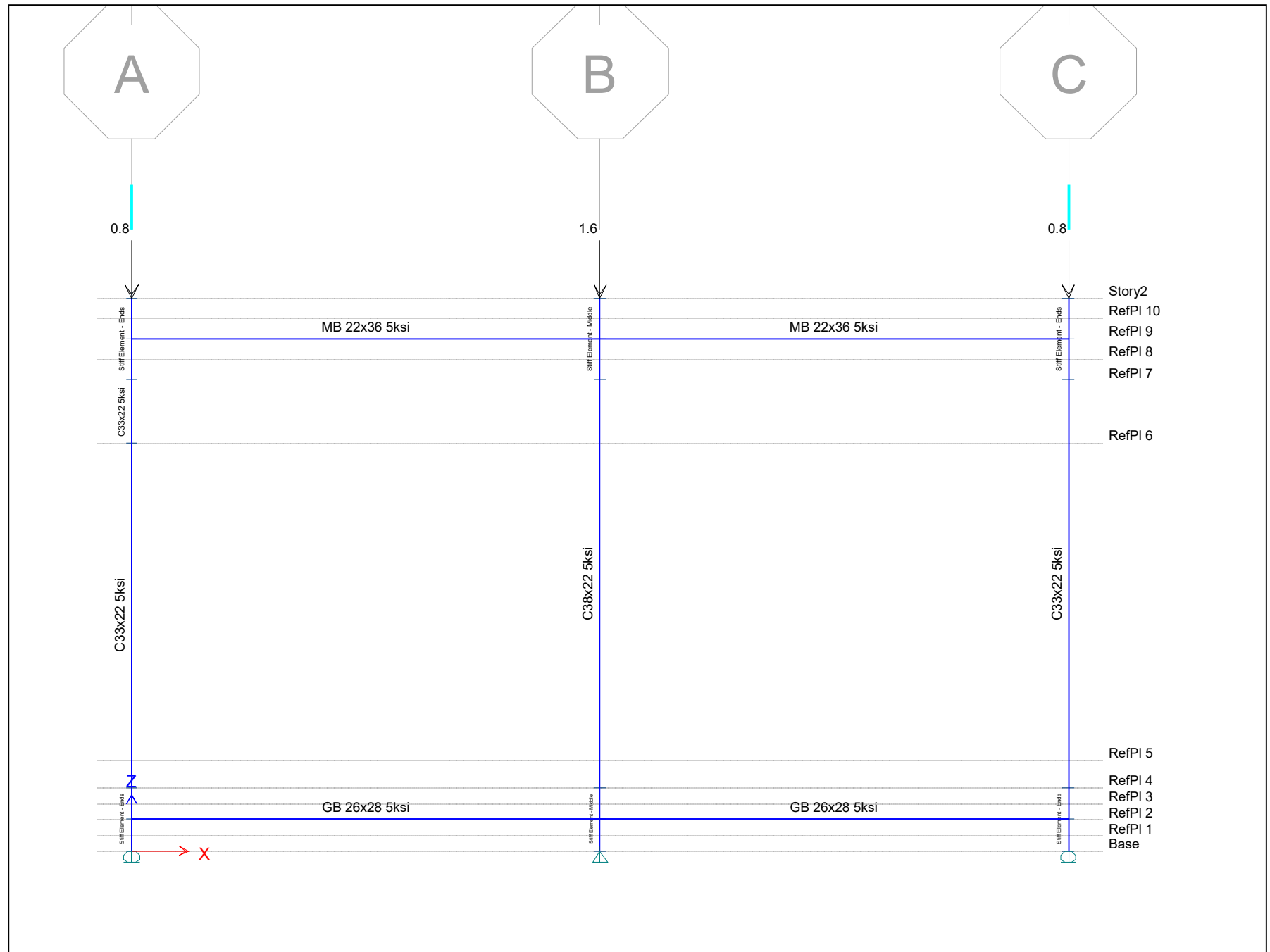
Moment Frame_v34.1 - Drift Level.EDB

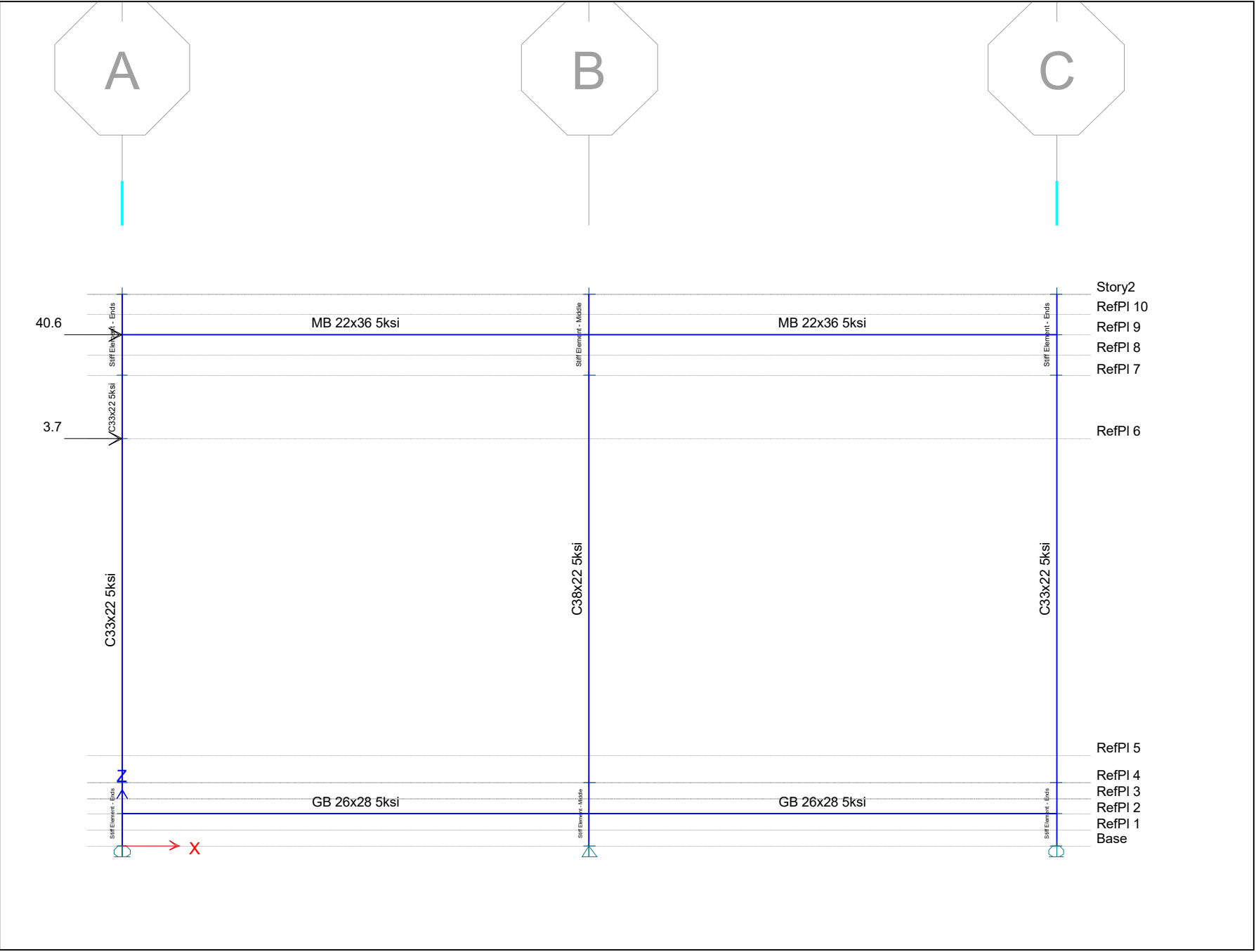
Elevation View - 1



Moment Frame_v34.1 - Drift Level.EDB

Elevation View - 1 Joint Loads (Dead)

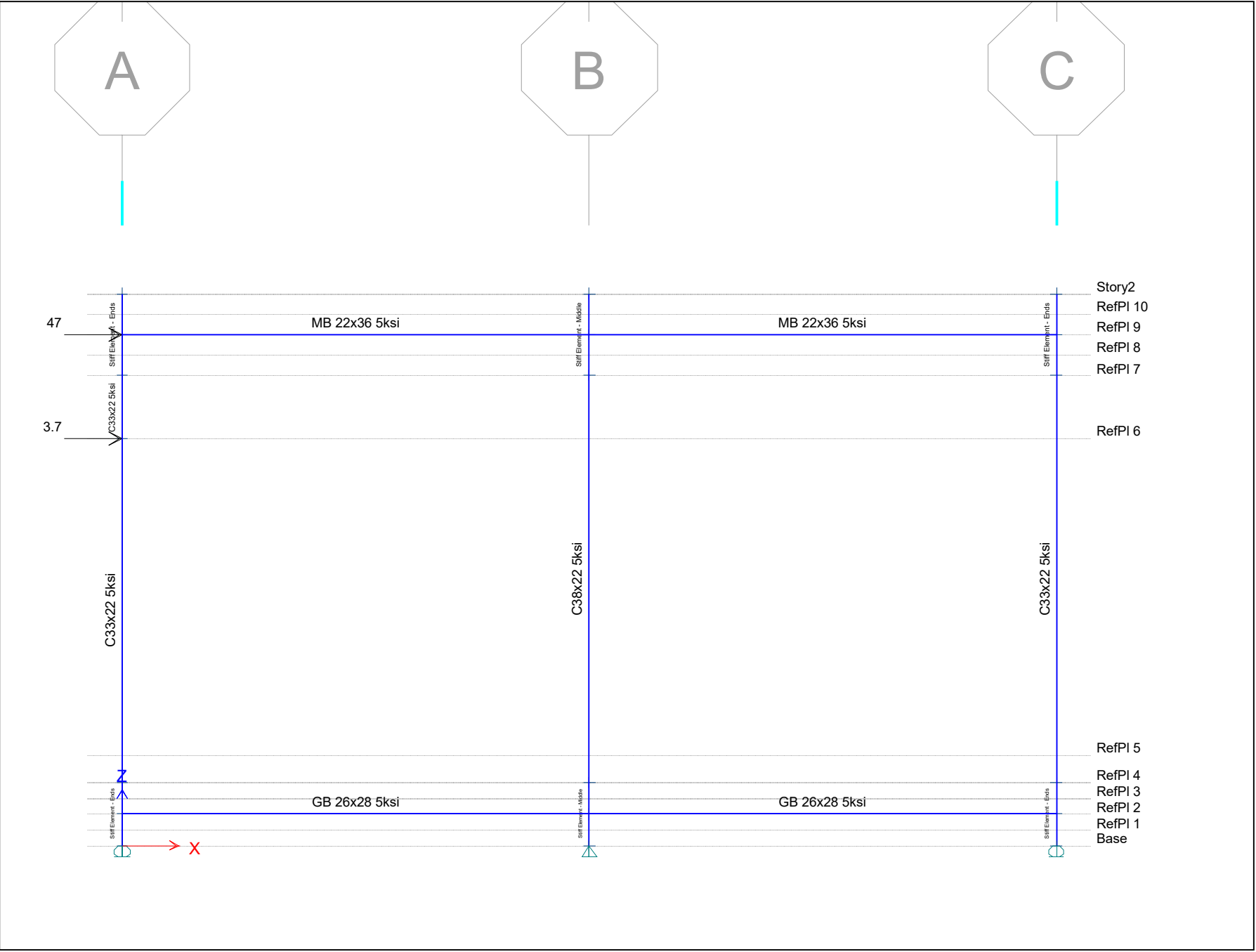




Moment Frame_v34.1 - Drift Level.EDB

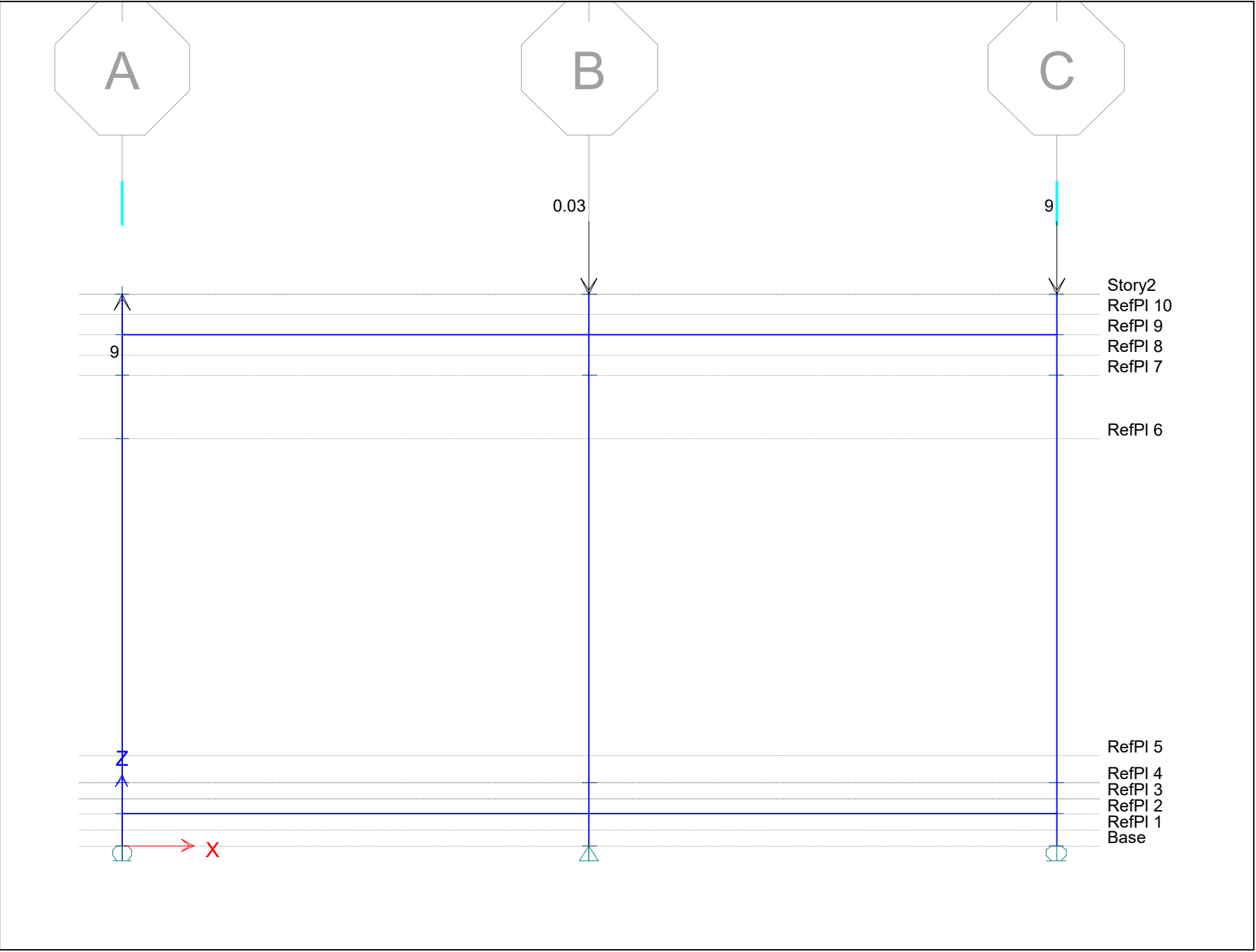
Elevation View - 1 Joint Loads (EQ-x)

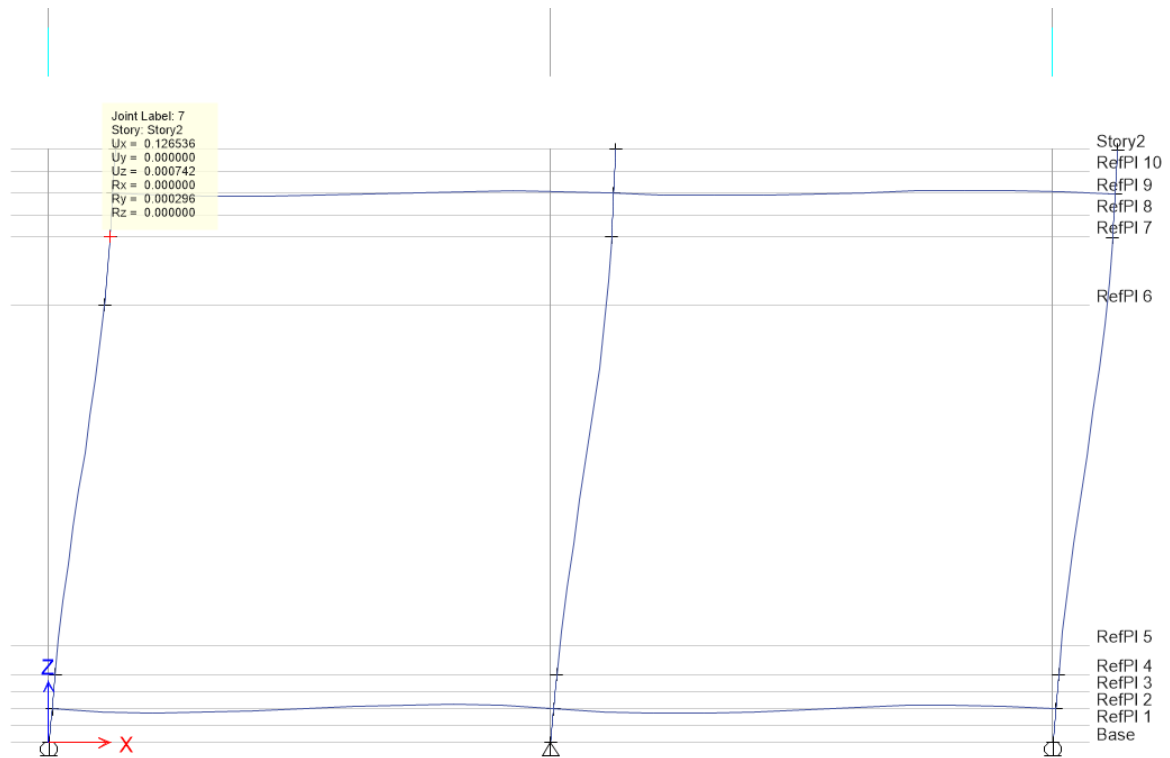




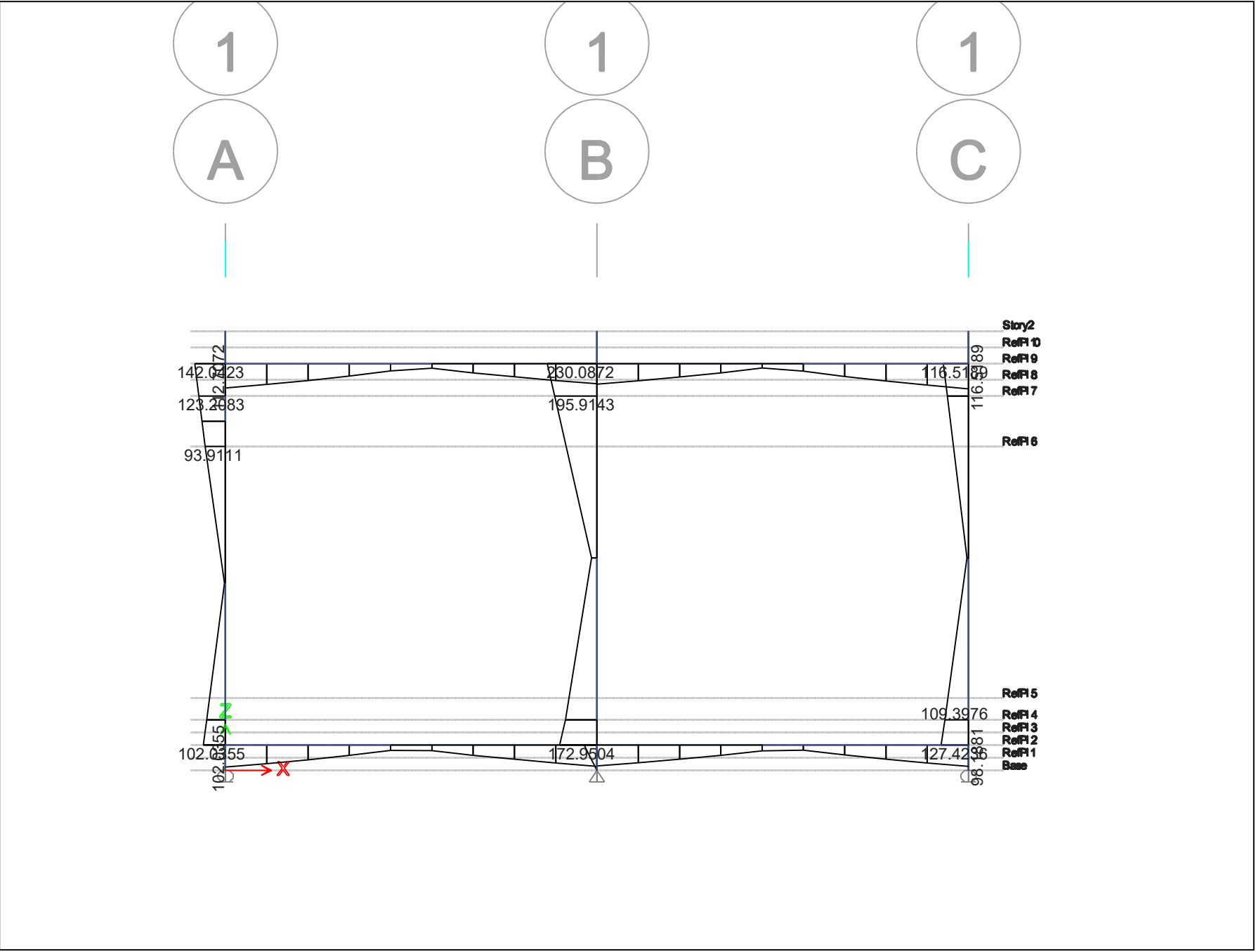
Moment Frame_v34.1 - Strength Level.EDB

Elevation View - 1 Joint Loads (EQ-x)

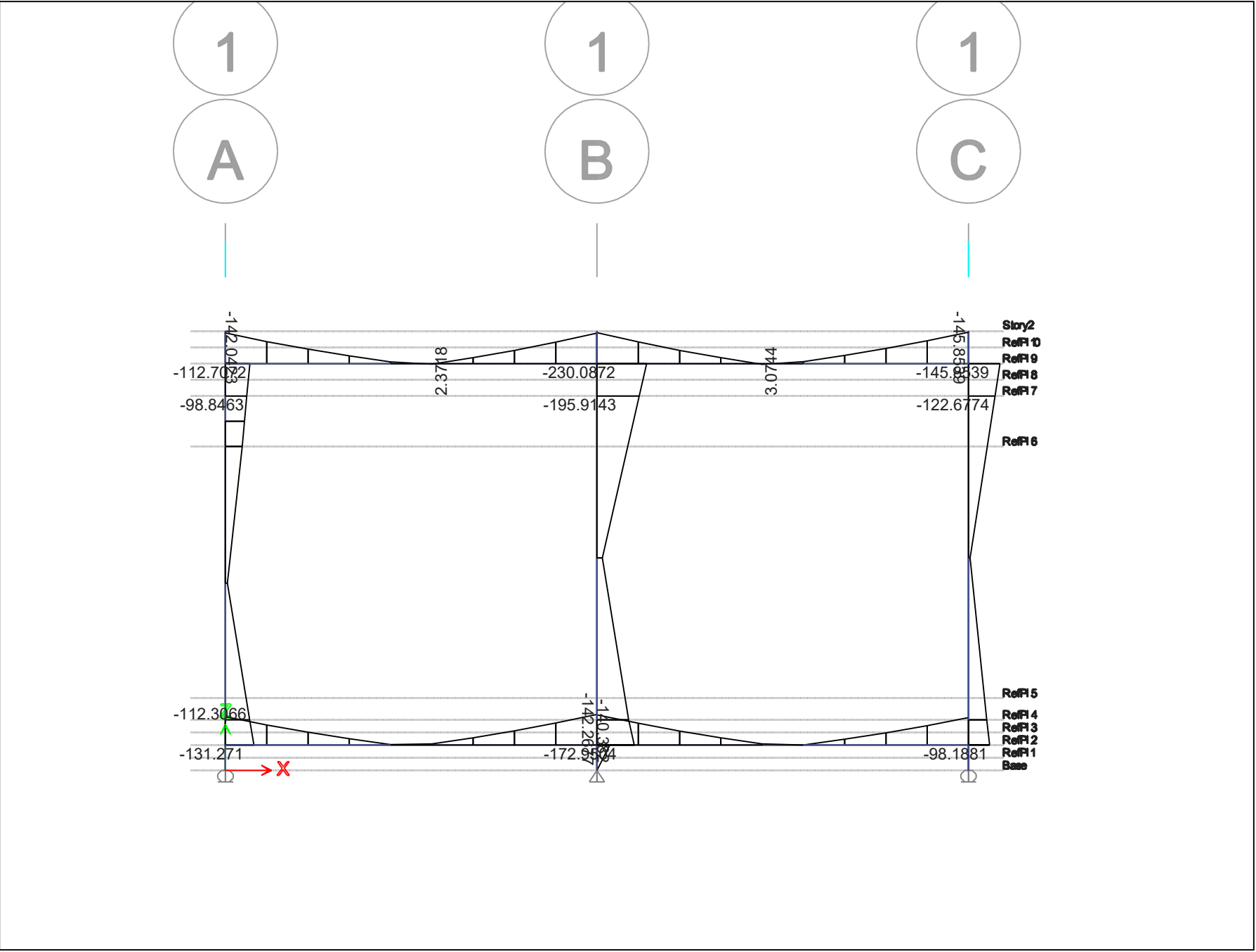




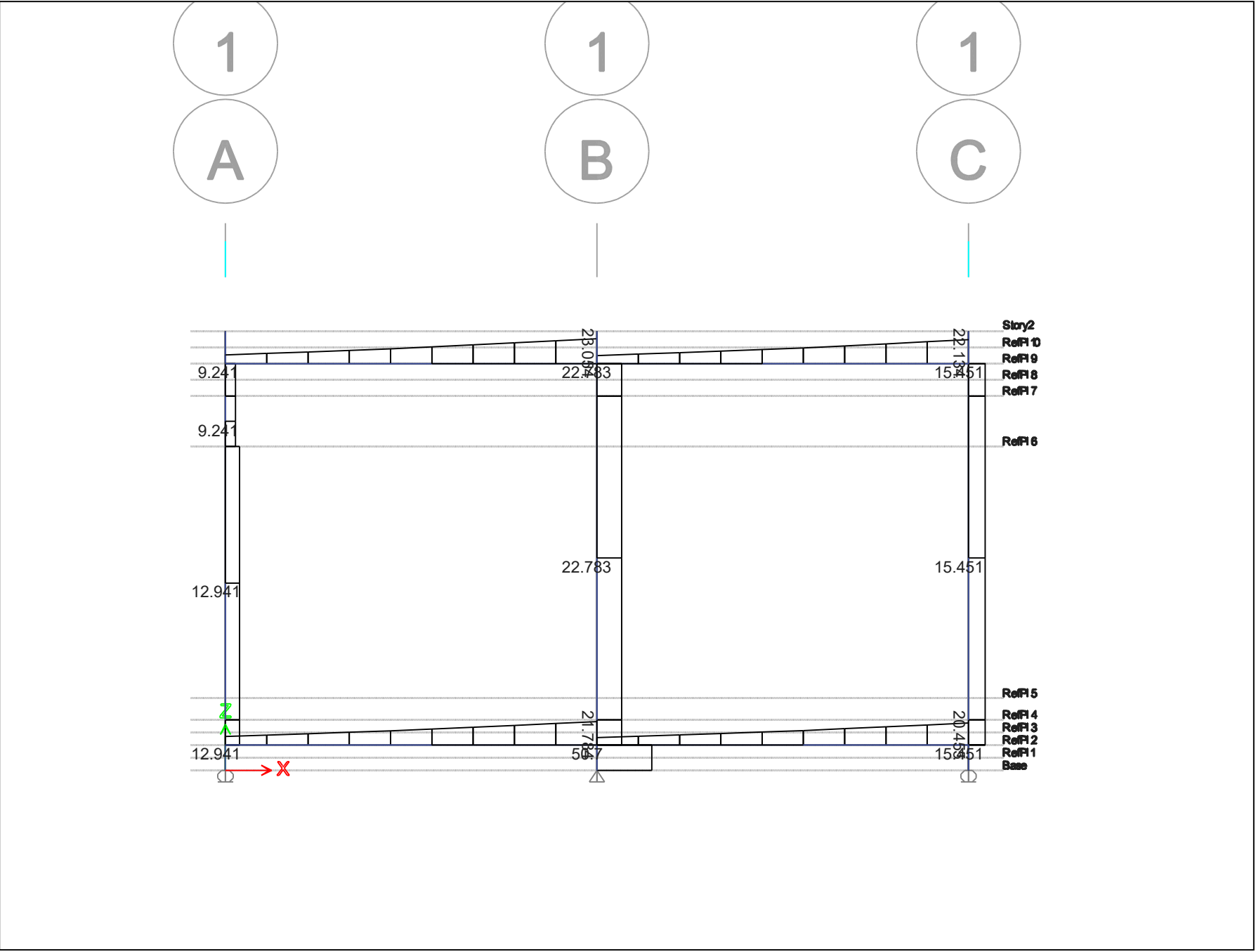
Elastic Drift = 0.127 in
Cd=4,
Ie = 1.5
Inelastic Drift = $4 \times 0.127 / 1.5 = 0.339\text{in} < 3/8" = 0.375"$ OK



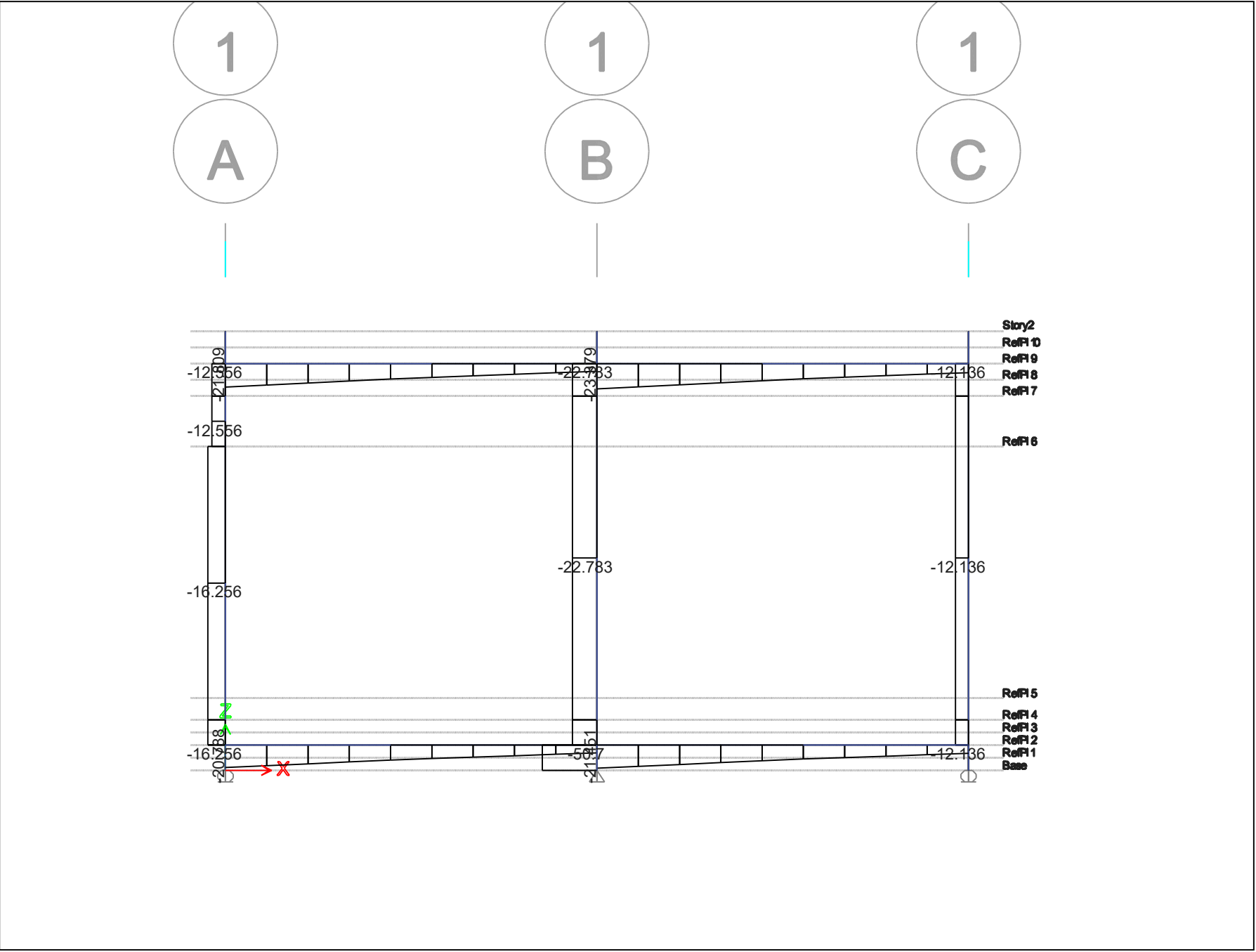
Moment Frame_v34.1 - Strength Level.EDB Elevation View - 1 Moment 3-3 Diagram (Env Strength) [kip-ft]



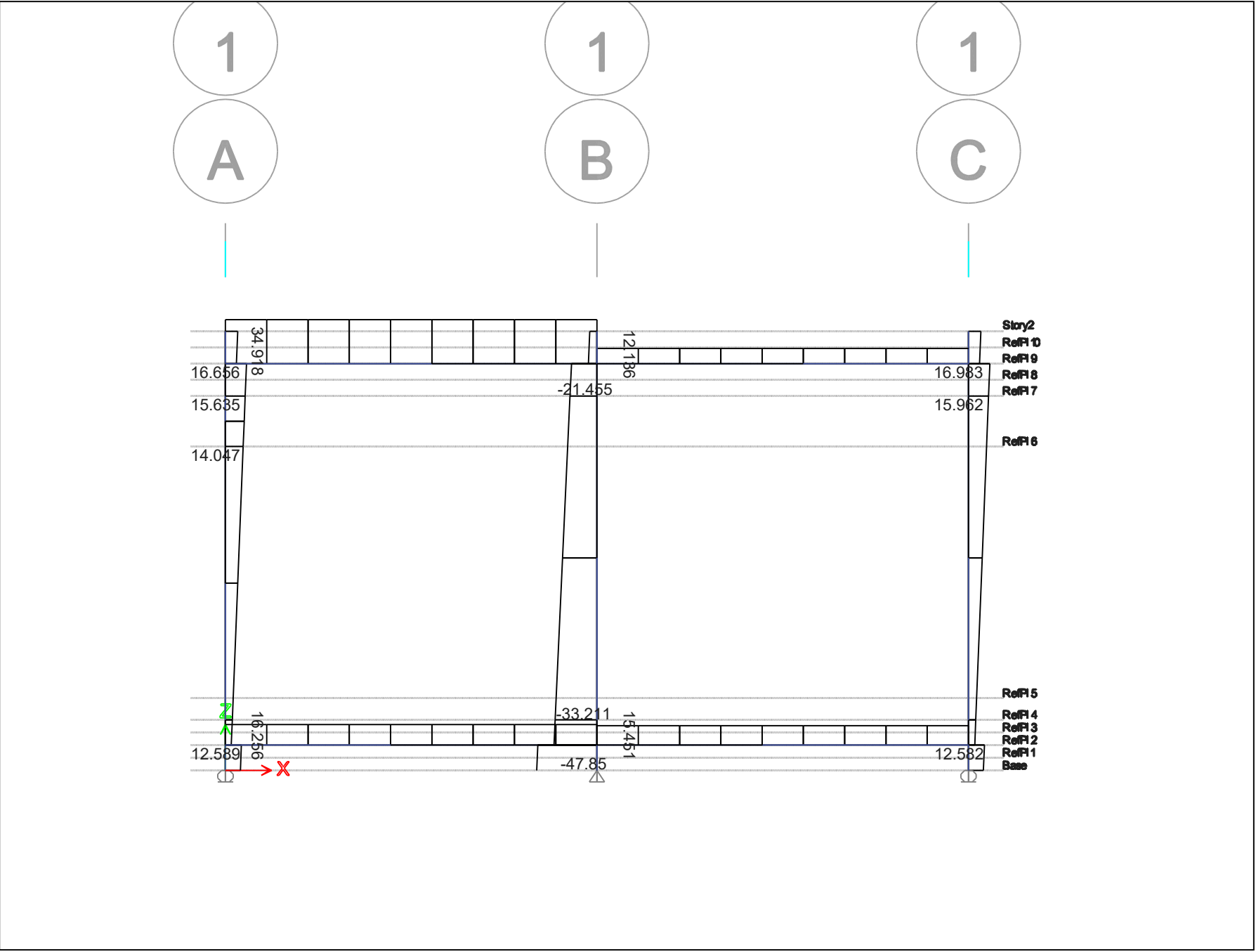
Moment Frame_v34.1 - Strength Level.EDB Elevation View - 1 Moment 3-3 Diagram (Env Strength) [kip-ft]



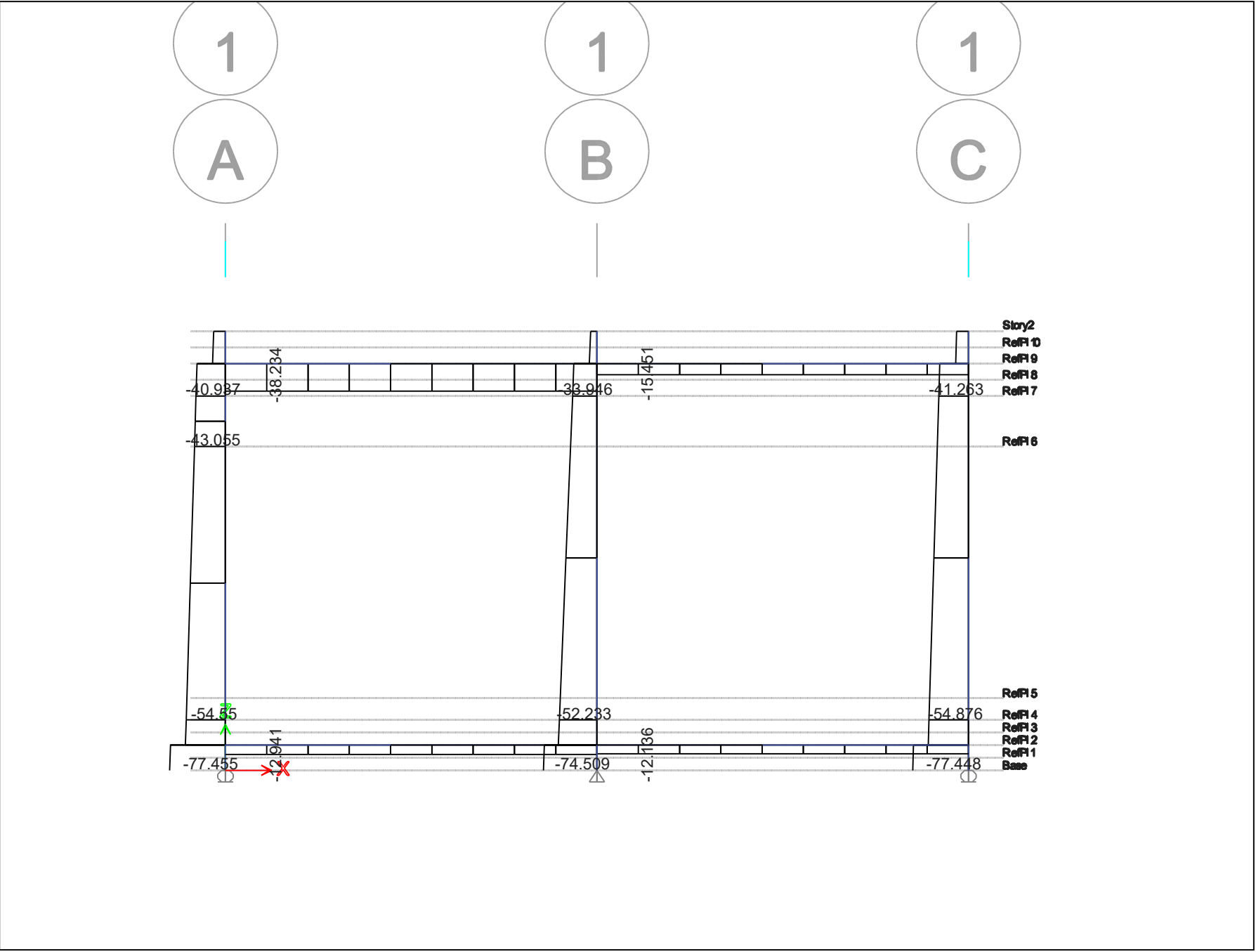
Moment Frame_v34.1 - Strength Level. Elevation View - 1 Shear Force 2-2 Diagram (Env Strength) [kip]



Moment Frame_v34.1 - Strength Level.ENV Elevation View - 1 Shear Force 2-2 Diagram (Env Strength) [kip]



Moment Frame_v34.1 - Strength Level.EDBElevation View - 1 Axial Force Diagram (Env Strength) [kip]



Moment Frame_v34.1 - Strength Level.EDBElevation View - 1 Axial Force Diagram (Env Strength) [kip]

SEISMIC GRADE BEAM DESIGN

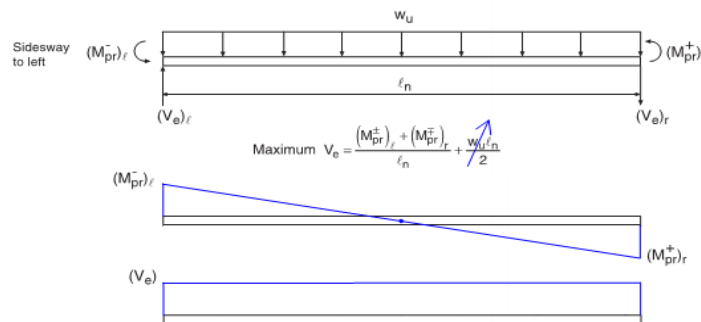
Code: ACI 318-19

Design Parameters

Clear Span, l_n (ft) =	14.25
h (in.) =	36
b_w (in.) =	22
Top cover (in.) =	1.5
Bot cover (in.) =	1.5
d^+ (in.) =	33.0
d^- (in.) =	33.0
f'_c (psi) =	5000
f_y (psi) =	60000

M_u (k-ft) =	147.5
P_u (k-ft) =	17.0 (frame action + $\Omega(V_e - V_{res})/2$)
$M_u < \phi M_n$ =	OK

ϕM_{n+} (k-ft) =	368.5
ϕM_{n-} (k-ft) =	406.1
M_{pr+} (k-ft) =	515.0
M_{pr-} (k-ft) =	552.3
Shear, V_{pr} (kips) =	74.90 ($M_{pr+} - M_{pr-}$)/ L



Reinforcing Bars

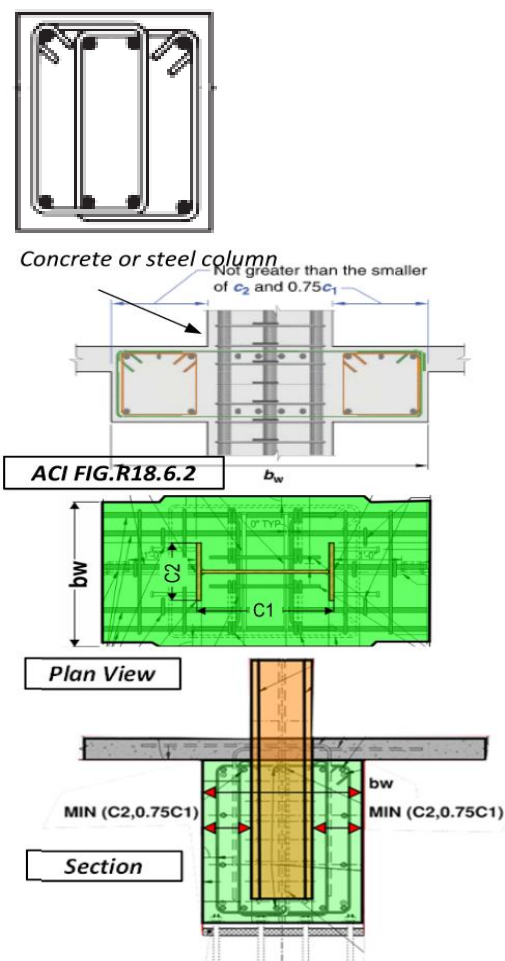
Top Longit. Reinf. =	(2) # 8	1.58 in. ²
	(1) # 9	1.00 in. ²
Bot. Longit. Reinf. =	(2) # 8	1.58 in. ²
	(1) # 9	1.00 in. ²
A_s (in. ²) =	5.16	
Transverse Bar =	# 5	

Scope

ACI 18.6.2.1	Clear Span, $l_n \geq 4d =$	132.00 in.	OK
	Beam Width, $b_w \geq \max(0.3h, 10 \text{ in}) =$	10.80 in	OK
	Projected Width $\leq \min(c_2, 0.75c_1) =$	11.50 in	OK
ACI 18.6.3.1	$A_s \geq A_{s,min}$: $A_{s,min} =$	2.57	OK
	$\rho \leq 0.025$: $\rho =$	0.0065	OK
ACI 18.6.3.3	Spacing of transverse reinforcement enclosing the Lap Splices:		
	$\min\{d/4, 4''\} =$	4 in.	

Transverse Reinforcement

ACI 18.6.4.4	Hoop Spacing shall not exceed min of:		
a)	$d/4 =$	8.25 in.	
b)	$6 * \min.d_{bar} =$	6.00 in.	
c)	$6'' =$	6.00 in.	
	$s_{max} =$	6.00 in.	
	(SPAN A) USE:	6 in.	OK
ACI 18.6.4.1	Span of Hoop Reinf. =	72 in.	(2h)
ACI 18.6.4.6	Where hoops are not required, provide stirrups with seismic hooks at both ends.		
	Stirrup $s_{max} \leq d/2$:	16.50 in.	
	(SPAN B) USE:	12 in.	OK



SEISMIC GRADE BEAM DESIGN

Code: ACI 318-19

Shear Reinforcement

Assume Constant Shear over span of grade beam

Span A

$$A_{v,required} (in.^2) = V_{pr} * s / (f_y * d) = 0.30$$

$$\# \text{ of Legs required} = 0.98$$

$$\# \text{ of Legs provided} = 3 \text{ OK}$$

Span B

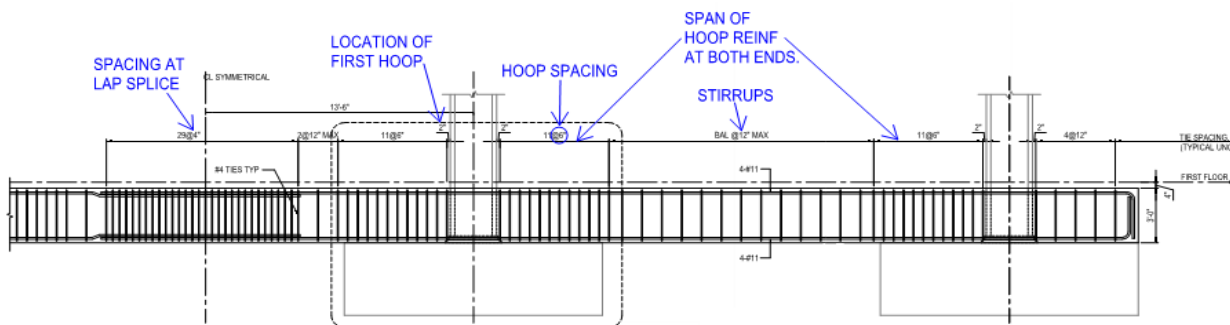
$$A_{v,required} (in.^2) = V_{pr} * s / (f_y * d) = 0.61$$

$$\# \text{ of Legs required} = 1.95$$

$$\# \text{ of Legs provided} = 3 \text{ OK}$$

Summary

<---Face of Column			Face column----->
<-----Span A----->	<-----Span B----->		<-----Span A----->
72.00 in	27.00 in	in	72.00 in
Spacing: 6 in. oc	12 in. oc		6 in. oc
# of Legs: 3	3		3
Transverse Bar: # 5			



Additional Notes:

Lap Bars Shall Not exceed $d/4$ " or 4" spacing for transverse reinforcement (21.5.2.3)

Lap Splices shall not be used:

- within the Joint
- withing a distance $2*h$ from the face of the joint
- At locations where analysis indicates Flexural Yielding

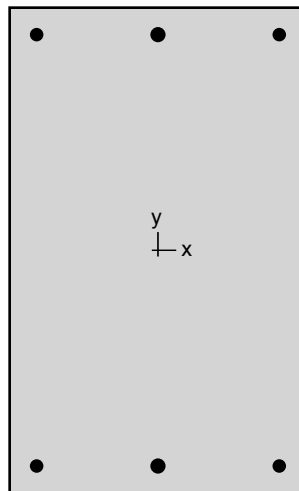
Mechanical Splices shall conform to 21.1.6

Welded Splices shall conform to 21.1.7

Hoop spacing shall be located no more than 2" from face of supporting member (21.5.3.2)



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Station...\SPCOL Mn - Right MB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	792 in ²
I_x	85536 in ⁴
I_y	31944 in ⁴
r_x	10.3923 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

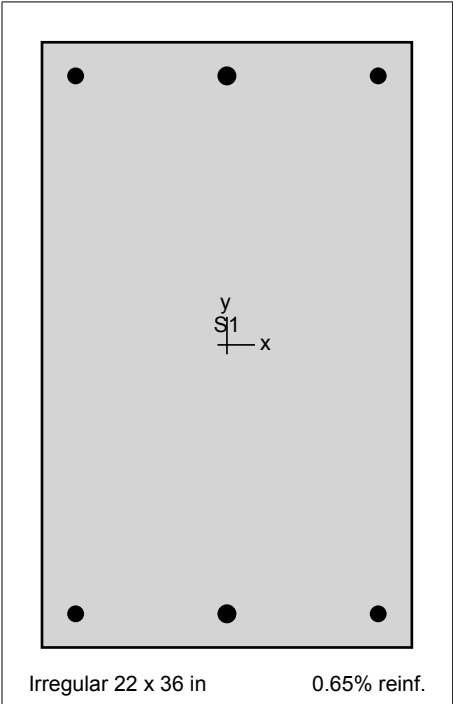


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-18.0	2	11.0	-18.0	3	11.0	18.0
4	-11.0	18.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	5.16 in ²
Rho	0.65 %
Minimum clear spacing	7.93 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-9.0	16.0	0.79	9.0	16.0	0.79	-9.0	-16.0
0.79	9.0	-16.0	1.00	0.0	16.0	1.00	0.0	-16.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	3653.7	0.00	0.00	109.55	34.00	-0.00207	1.00000
X @ Allowable comp.	3653.7	0.00	0.00	109.55	34.00	-0.00207	1.00000
X @ $f_s = 0.0$	2686.9	1124.35	0.00	34.00	34.00	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1957.4	1537.92	0.00	25.28	34.00	0.00103	1.00000
X @ Balanced point	1494.1	1646.29	0.00	20.12	34.00	0.00207	1.00000
X @ Tension control	934.5	1418.02	0.00	12.64	34.00	0.00507	1.00000
X @ Pure bending	0.0	427.80	0.00	2.03	34.00	0.04727	1.00000
X @ Max tension	-309.6	0.00	0.00	0.00	34.00	9.99999	1.00000
-X @ Max compression	3653.7	0.00	0.00	109.55	34.00	-0.00207	1.00000
-X @ Allowable comp.	3653.7	0.00	0.00	109.55	34.00	-0.00207	1.00000
-X @ $f_s = 0.0$	2686.9	-1124.35	0.00	34.00	34.00	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1957.4	-1537.92	0.00	25.28	34.00	0.00103	1.00000
-X @ Balanced point	1494.1	-1646.29	0.00	20.12	34.00	0.00207	1.00000
-X @ Tension control	934.5	-1418.02	0.00	12.64	34.00	0.00507	1.00000
-X @ Pure bending	0.0	-427.80	0.00	2.03	34.00	0.04727	1.00000
-X @ Max tension	-309.6	0.00	0.00	0.00	34.00	9.99999	1.00000

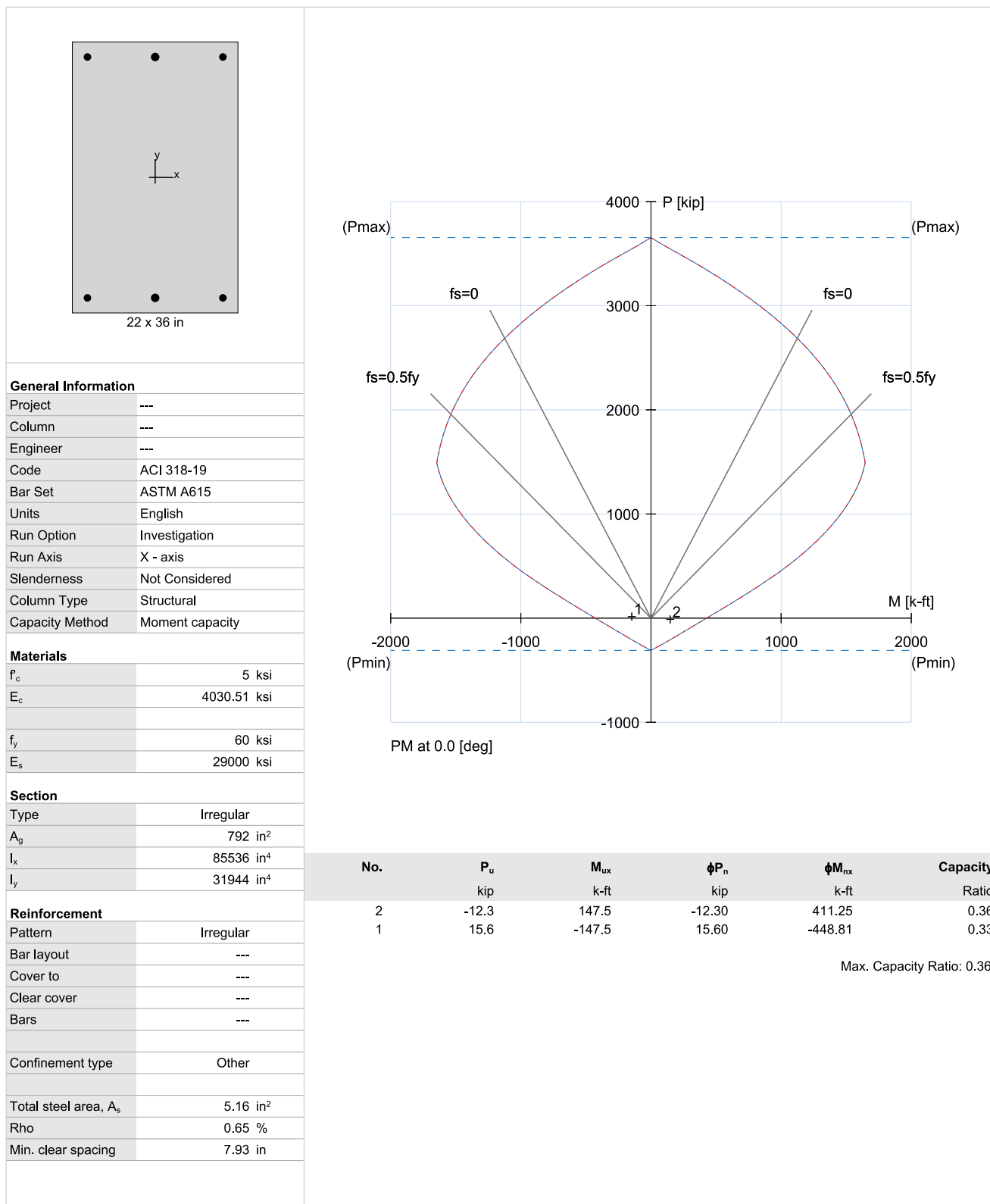
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	15.60	-147.50	15.60	-448.81	2.12	0.04520	1.000	0.33
2	-12.30	147.50	-12.30	411.25	1.96	0.04895	1.000	0.36

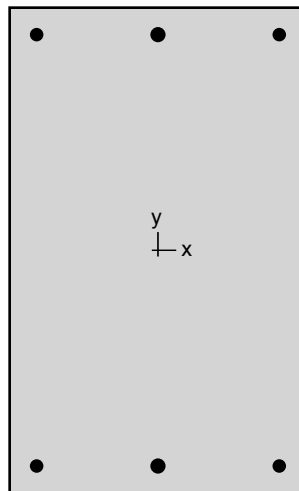
7. Diagrams

7.1. PM at $\theta=0$ [deg]





spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Statio...\SPCOL Mpr - Right MB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	75 ksi
E_s	29000 ksi
ϵ_{ty}	0.00258621 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	792 in ²
I_x	85536 in ⁴
I_y	31944 in ⁴
r_x	10.3923 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

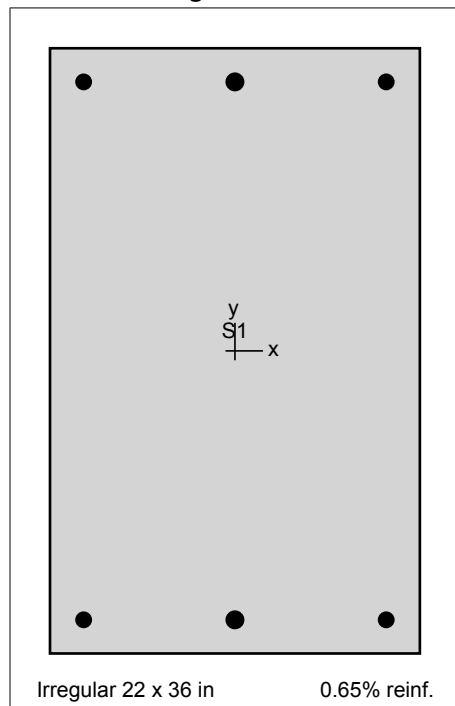


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-18.0	2	11.0	-18.0	3	11.0	18.0
4	-11.0	18.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	5.16 in ²
Rho	0.65 %
Minimum clear spacing	7.93 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-9.0	16.0	0.79	9.0	16.0	0.79	-9.0	-16.0
0.79	9.0	-16.0	1.00	0.0	16.0	1.00	0.0	-16.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	3731.1	0.00	0.00	246.49	34.00	-0.00259	1.00000
X @ Allowable comp.	3731.1	0.00	0.00	246.49	34.00	-0.00259	1.00000
X @ $f_s = 0.0$	2725.6	1175.95	0.00	34.00	34.00	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1862.9	1630.65	0.00	23.76	34.00	0.00129	1.00000
X @ Balanced point	1354.8	1718.73	0.00	18.26	34.00	0.00259	1.00000
X @ Tension control	870.7	1473.18	0.00	11.88	34.00	0.00559	1.00000
X @ Pure bending	0.0	531.40	0.00	2.25	34.00	0.04228	1.00000
X @ Max tension	-387.0	0.00	0.00	0.00	34.00	9.99999	1.00000
-X @ Max compression	3731.1	0.00	0.00	246.49	34.00	-0.00259	1.00000
-X @ Allowable comp.	3731.1	0.00	0.00	246.49	34.00	-0.00259	1.00000
-X @ $f_s = 0.0$	2725.6	-1175.95	0.00	34.00	34.00	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1862.9	-1630.65	0.00	23.76	34.00	0.00129	1.00000
-X @ Balanced point	1354.8	-1718.73	0.00	18.26	34.00	0.00259	1.00000
-X @ Tension control	870.7	-1473.18	0.00	11.88	34.00	0.00559	1.00000
-X @ Pure bending	0.0	-531.40	0.00	2.25	34.00	0.04228	1.00000
-X @ Max tension	-387.0	0.00	0.00	0.00	34.00	9.99999	1.00000

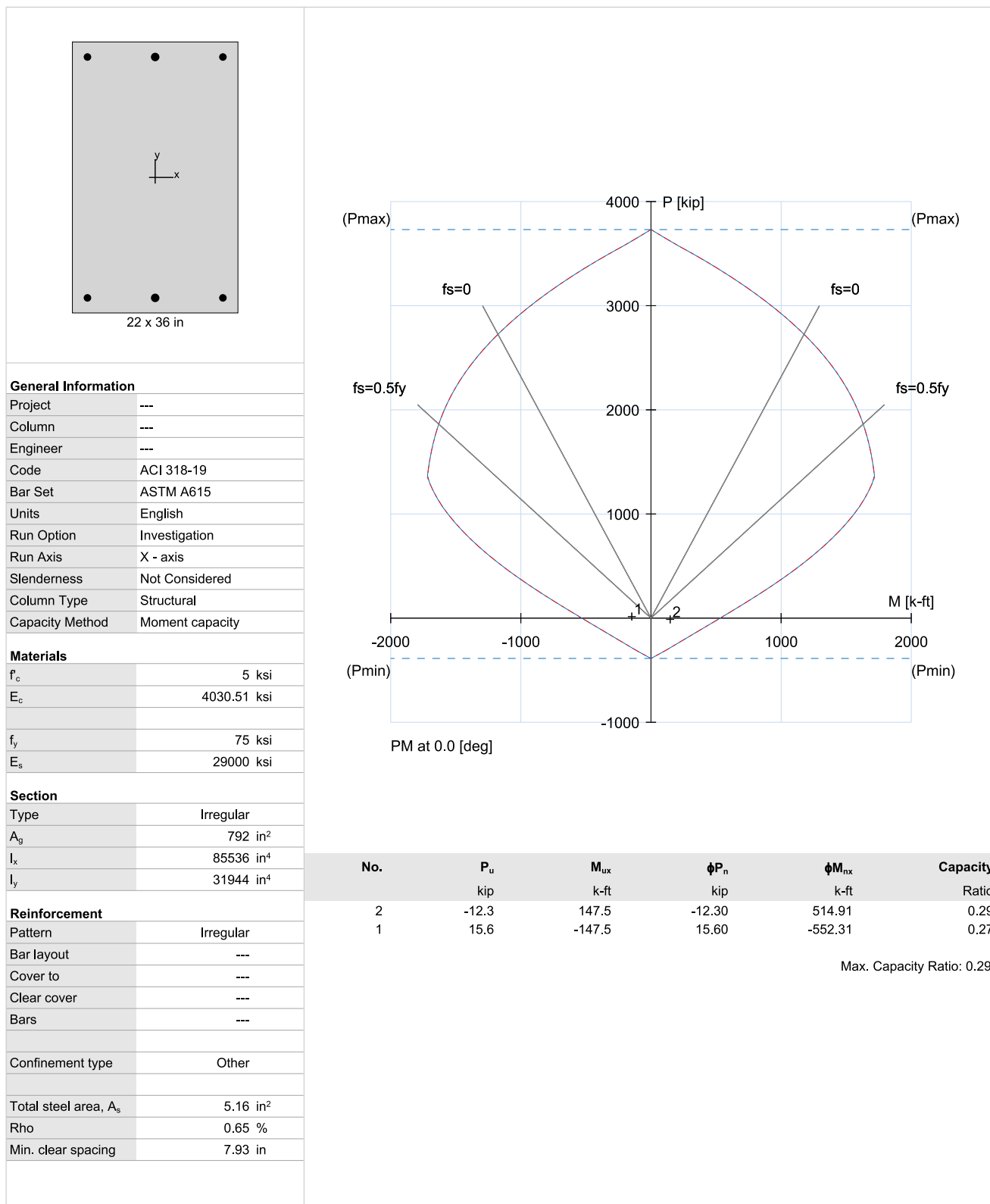
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	15.60	-147.50	15.60	-552.31	2.35	0.04040	1.000	0.27
2	-12.30	147.50	-12.30	514.91	2.18	0.04382	1.000	0.29

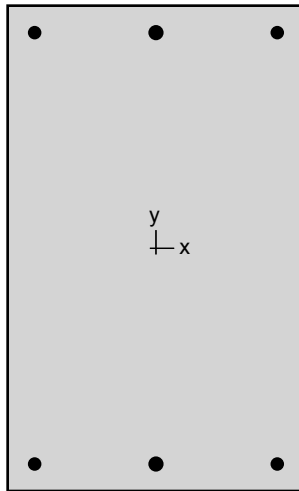
7. Diagrams

7.1. PM at $\theta=0$ [deg]





spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Stat...\SPCOL phiMn - Right MB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	792 in ²
I_x	85536 in ⁴
I_y	31944 in ⁴
r_x	10.3923 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

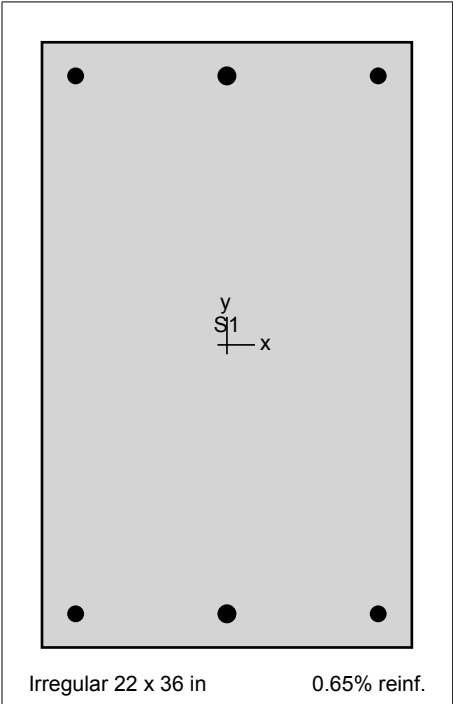


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-18.0	2	11.0	-18.0	3	11.0	18.0
4	-11.0	18.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ , (b)	0.9
Compression controlled ϕ , (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	5.16 in ²
Rho	0.65 %
Minimum clear spacing	7.93 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-9.0	16.0	0.79	9.0	16.0	0.79	-9.0	-16.0
0.79	9.0	-16.0	1.00	0.0	16.0	1.00	0.0	-16.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	2374.9	0.00	0.00	109.55	34.00	-0.00207	0.65000
X @ Allowable comp.	1899.9	592.89	0.00	36.92	34.00	-0.00024	0.65000
X @ $f_s = 0.0$	1746.5	730.83	0.00	34.00	34.00	0.00000	0.65000
X @ $f_s = 0.5 f_y$	1272.3	999.65	0.00	25.28	34.00	0.00103	0.65000
X @ Balanced point	971.2	1070.09	0.00	20.12	34.00	0.00207	0.65000
X @ Tension control	841.1	1276.21	0.00	12.64	34.00	0.00507	0.90000
X @ Pure bending	0.0	385.02	0.00	2.03	34.00	0.04727	0.90000
X @ Max tension	-278.6	0.00	0.00	0.00	34.00	9.99999	0.90000
-X @ Max compression	2374.9	0.00	0.00	109.55	34.00	-0.00207	0.65000
-X @ Allowable comp.	1899.9	-592.89	0.00	36.92	34.00	-0.00024	0.65000
-X @ $f_s = 0.0$	1746.5	-730.83	0.00	34.00	34.00	0.00000	0.65000
-X @ $f_s = 0.5 f_y$	1272.3	-999.65	0.00	25.28	34.00	0.00103	0.65000
-X @ Balanced point	971.2	-1070.09	0.00	20.12	34.00	0.00207	0.65000
-X @ Tension control	841.1	-1276.21	0.00	12.64	34.00	0.00507	0.90000
-X @ Pure bending	0.0	-385.02	0.00	2.03	34.00	0.04727	0.90000
-X @ Max tension	-278.6	0.00	0.00	0.00	34.00	9.99999	0.90000

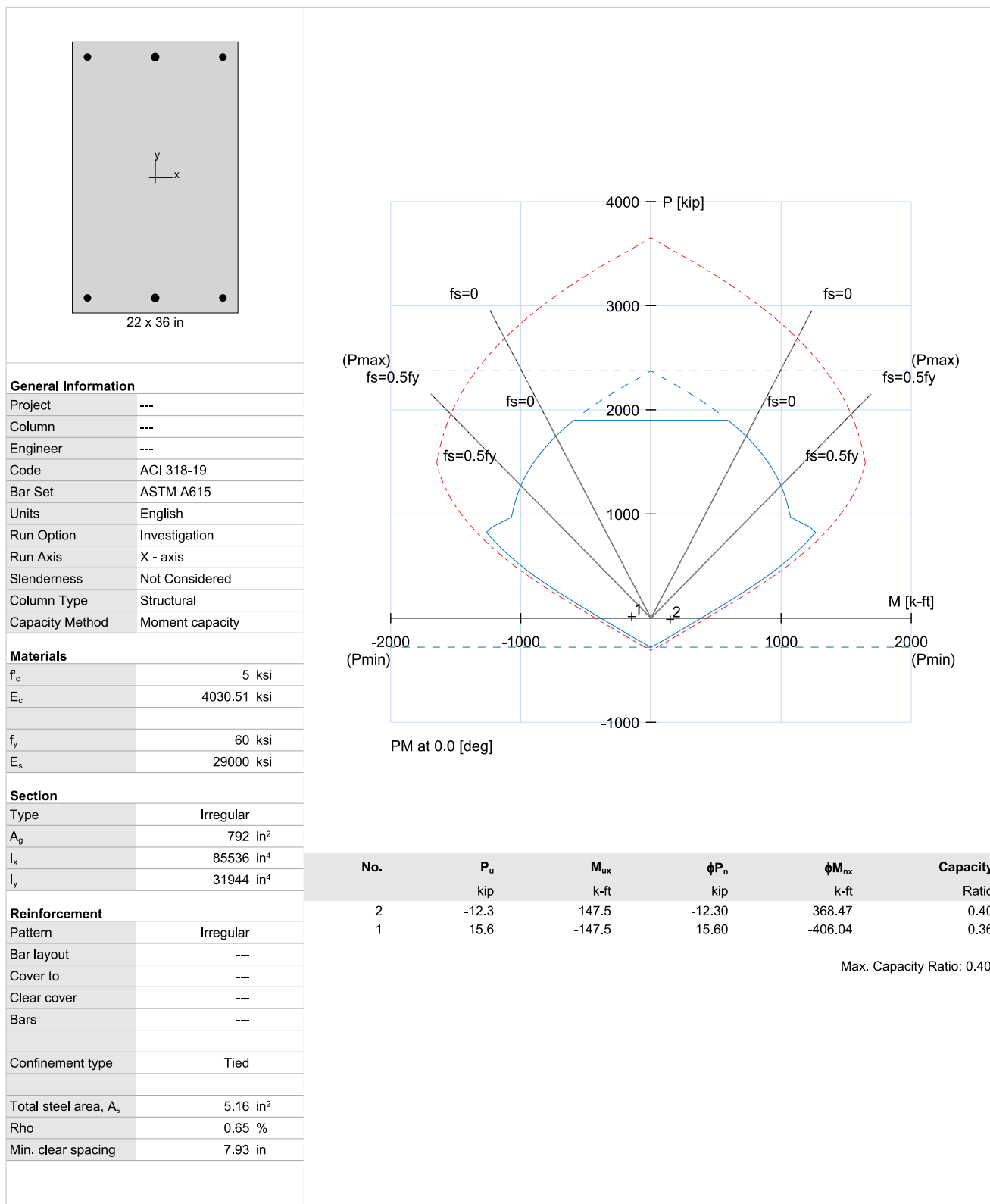
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	15.60	-147.50	15.60	-406.04	2.13	0.04498	0.900	0.36
2	-12.30	147.50	-12.30	368.47	1.96	0.04914	0.900	0.40

7. Diagrams

7.1. PM at $\theta=0$ [deg]



SEISMIC GRADE BEAM DESIGN

Code: ACI 318-19

Design Parameters

Clear Span, l_n (ft) = 14.25

h (in.) = 36

b_w (in.) = 22

Top cover (in.) = 1.5

Bot cover (in.) = 1.5

d^+ (in.) = 33.0

d^- (in.) = 33.0

f'_c (psi) = 5000

f_y (psi) = 60000

M_u (k-ft) = 145.5

P_u (k-ft) = 41.0 (frame action + $\Omega(V_e - V_{res})/2$)

$M_u < \phi M_n$ = OK

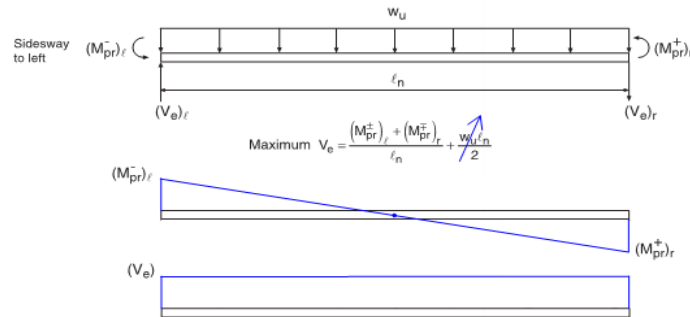
ϕM_{n+} (k-ft) = 338.7

ϕM_{n-} (k-ft) = 435.7

M_{pr+} (k-ft) = 485.2

M_{pr-} (k-ft) = 581.8

Shear, V_{pr} (kips) = 74.88 ($M_{pr+} - M_{pr-}/L$)



Reinforcing Bars

Top Longit. Reinf. = (2) # 8 1.58 in.²

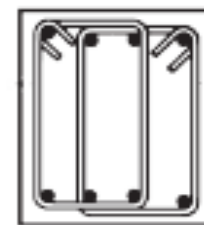
(1) # 9 1.00 in.²

Bot. Longit. Reinf. = (2) # 8 1.58 in.²

(1) # 9 1.00 in.²

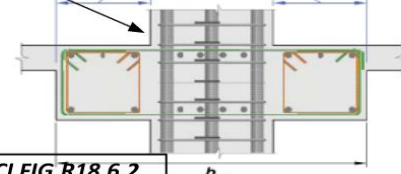
A_s (in.²) = 5.16

Transverse Bar = # 5

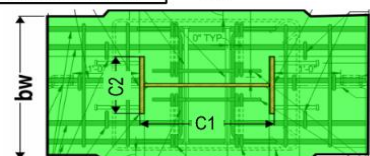


Concrete or steel column

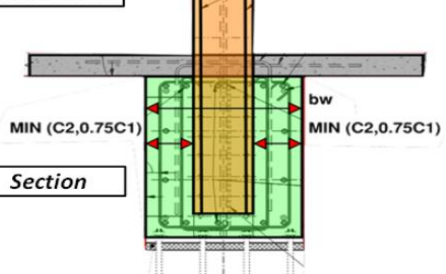
Not greater than the smaller of c_2 and $0.75c_1$



ACI FIG. R18.6.2



Plan View



Section

Scope

ACI 18.6.2.1	Clear Span, $l_n \geq 4d =$	132.00 in.	OK
	Beam Width, $b_w \geq \max(0.3h, 10 \text{ in}) =$	10.80 in	OK
	Projected Width $\leq \min(c_2, 0.75c_1) =$	11.50 in	OK
ACI 18.6.3.1	$A_s \geq A_{s,min}$: $A_{s,min} =$	2.57	OK
	$\rho \leq 0.025$: $\rho =$	0.0065	OK
ACI 18.6.3.3	Spacing of transverse reinforcement enclosing the Lap Splices:		
	$\min\{d/4, 4''\} =$	4 in.	

Transverse Reinforcement

ACI 18.6.4.4	Hoop Spacing shall not exceed min of:		
a)	$d/4 =$	8.25 in.	
b)	$6 * \min.d_{bar} =$	6.00 in.	
c)	$6'' =$	6.00 in.	
	$S_{max} =$	6.00 in.	
	(SPAN A) USE:	6 in.	OK
ACI 18.6.4.1	Span of Hoop Reinf. =	72 in.	(2h)

SEISMIC GRADE BEAM DESIGN

Code: ACI 318-19

ACI 18.6.4.6 Where hoops are not required, provide stirrups with seismic hooks at both ends.
Stirrup $s_{max} \leq d/2$: 16.50 in.
(SPAN B) USE: 12 in. OK

Shear Reinforcement

Assume Constant Shear over span of grade beam
Span A

$$A_{v,required} (in.^2) = V_{pr} * s / (f_y * d) = 0.30$$

$$\# \text{ of Legs required} = 0.98$$

$$\# \text{ of Legs provided} = 3 \text{ OK}$$

Span B

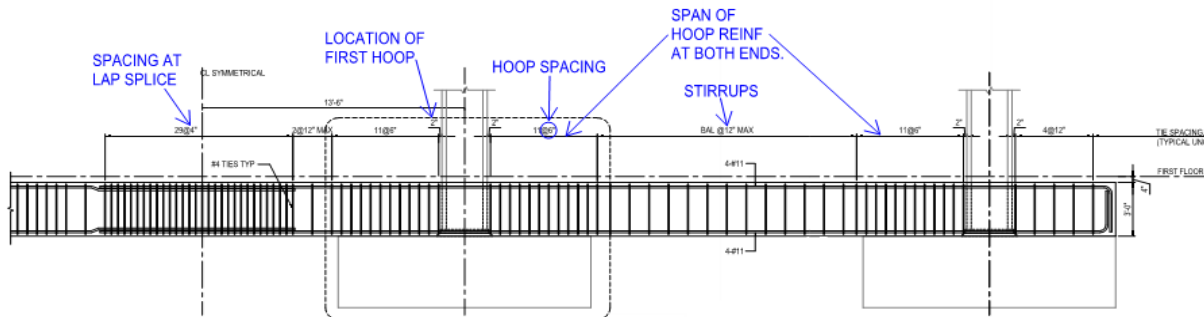
$$A_{v,required} (in.^2) = V_{pr} * s / (f_y * d) = 0.61$$

$$\# \text{ of Legs required} = 1.95$$

$$\# \text{ of Legs provided} = 3 \text{ OK}$$

Summary

<---Face of Column		Face column----->
<-----Span A----->	<-----Span B----->	<-----Span A----->
72.00 in	27.00 in	72.00 in
Spacing: 6 in. oc	12 in. oc	6 in. oc
# of Legs: 3	3	3
Transverse Bar: # 5		

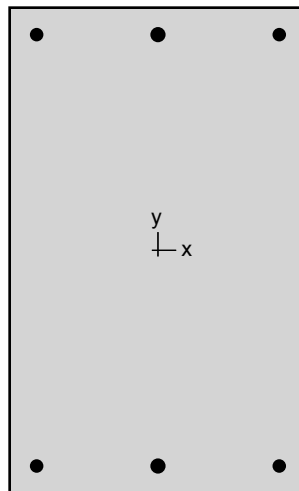


Additional Notes:

- Lap Bars Shall Not exceed $d/4$ " or 4" spacing for transverse reinforcement (21.5.2.3)
Lap Splices shall not be used:
- within the Joint
 - withing a distance $2 * h$ from the face of the joint
 - At locations where analysis indicates Flexural Yielding
- Mechanical Splices shall conform to 21.1.6
Welded Splices shall conform to 21.1.7
Hoop spacing shall be located no more than 2" from face of supporting member (21.5.3.2)



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Station ...\SPCOL Mn - Left MB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	792 in ²
I_x	85536 in ⁴
I_y	31944 in ⁴
r_x	10.3923 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

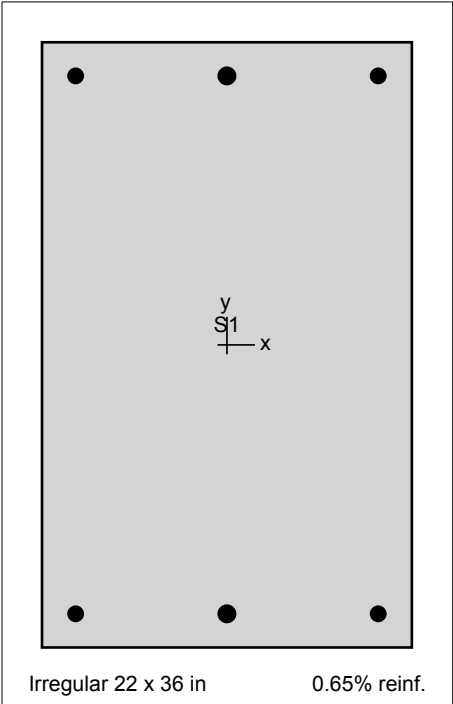


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-18.0	2	11.0	-18.0	3	11.0	18.0
4	-11.0	18.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	5.16 in ²
Rho	0.65 %
Minimum clear spacing	7.93 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-9.0	16.0	0.79	9.0	16.0	0.79	-9.0	-16.0
0.79	9.0	-16.0	1.00	0.0	16.0	1.00	0.0	-16.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	3653.7	0.00	0.00	109.55	34.00	-0.00207	1.00000
X @ Allowable comp.	3653.7	0.00	0.00	109.55	34.00	-0.00207	1.00000
X @ $f_s = 0.0$	2686.9	1124.35	0.00	34.00	34.00	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1957.4	1537.92	0.00	25.28	34.00	0.00103	1.00000
X @ Balanced point	1494.1	1646.29	0.00	20.12	34.00	0.00207	1.00000
X @ Tension control	934.5	1418.02	0.00	12.64	34.00	0.00507	1.00000
X @ Pure bending	0.0	427.80	0.00	2.03	34.00	0.04727	1.00000
X @ Max tension	-309.6	0.00	0.00	0.00	34.00	9.99999	1.00000
-X @ Max compression	3653.7	0.00	0.00	109.55	34.00	-0.00207	1.00000
-X @ Allowable comp.	3653.7	0.00	0.00	109.55	34.00	-0.00207	1.00000
-X @ $f_s = 0.0$	2686.9	-1124.35	0.00	34.00	34.00	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1957.4	-1537.92	0.00	25.28	34.00	0.00103	1.00000
-X @ Balanced point	1494.1	-1646.29	0.00	20.12	34.00	0.00207	1.00000
-X @ Tension control	934.5	-1418.02	0.00	12.64	34.00	0.00507	1.00000
-X @ Pure bending	0.0	-427.80	0.00	2.03	34.00	0.04727	1.00000
-X @ Max tension	-309.6	0.00	0.00	0.00	34.00	9.99999	1.00000

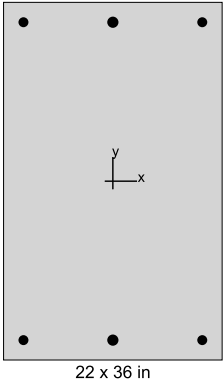
6. Factored Loads and Moments with Corresponding Capacity Ratios

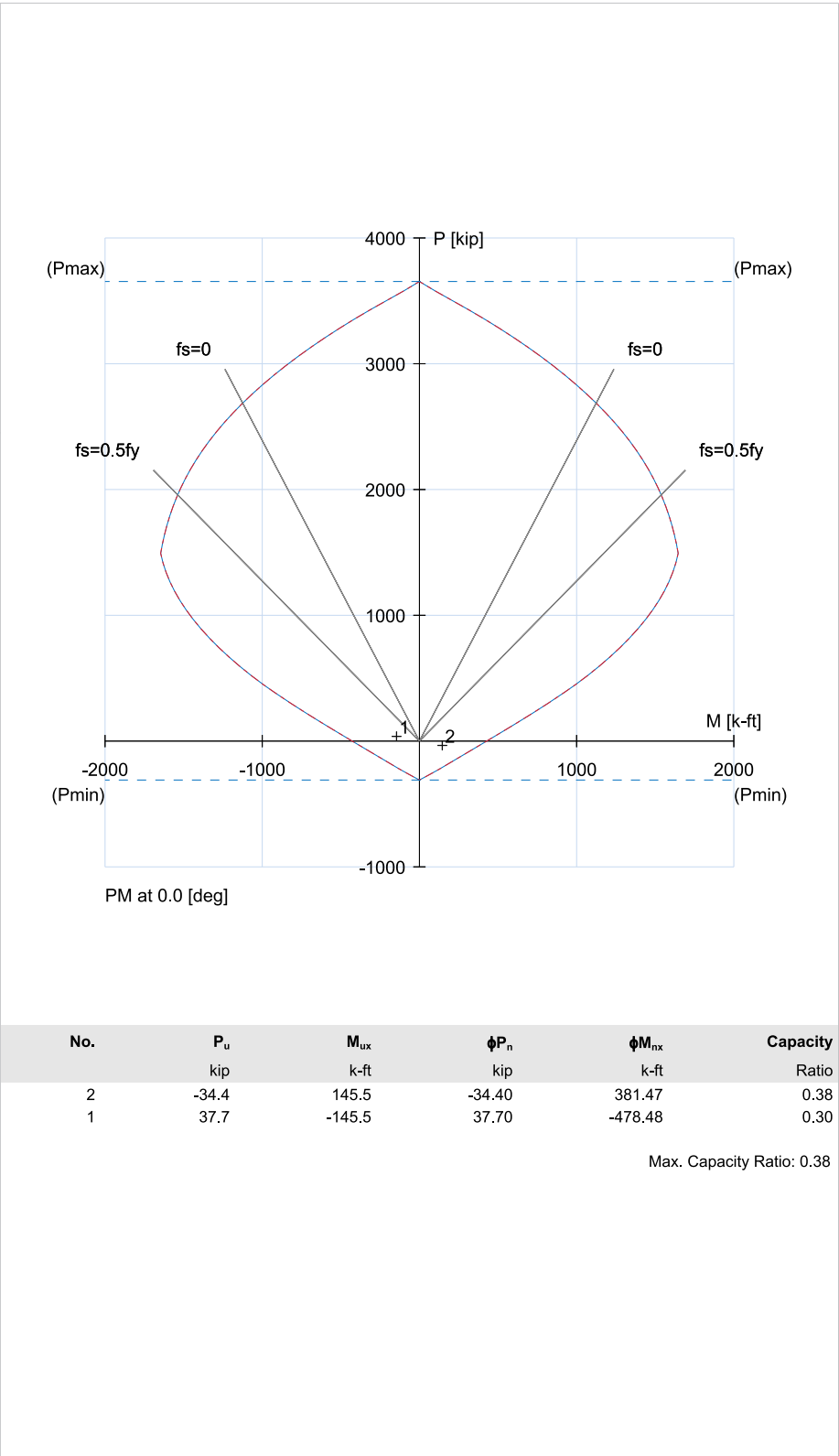
NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	37.70	-145.50	37.70	-478.48	2.25	0.04240	1.000	0.30
2	-34.40	145.50	-34.40	381.47	1.85	0.05207	1.000	0.38

7. Diagrams

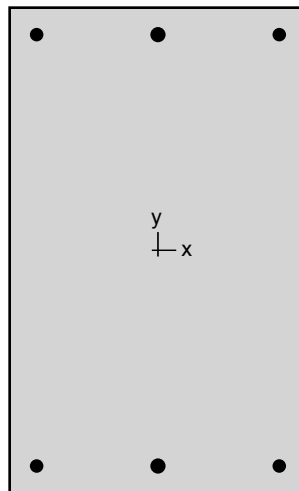
7.1. PM at $\theta=0$ [deg]

	
22 x 36 in	
General Information	
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity
Materials	
f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi
Section	
Type	Irregular
A_g	792 in ²
I_x	85536 in ⁴
I_y	31944 in ⁴
Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Other
Total steel area, A_s	5.16 in ²
Rho	0.65 %
Min. clear spacing	7.93 in





spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Station...\SPCOL Mpr - Left MB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	75 ksi
E_s	29000 ksi
ϵ_{ty}	0.00258621 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	792 in ²
I_x	85536 in ⁴
I_y	31944 in ⁴
r_x	10.3923 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

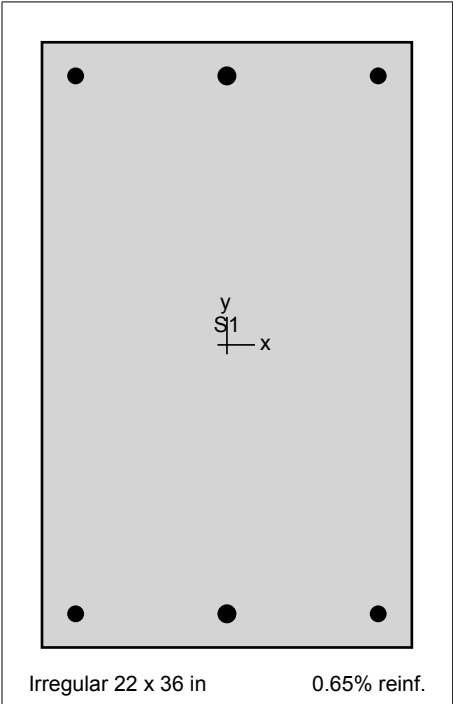


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-18.0	2	11.0	-18.0	3	11.0	18.0
4	-11.0	18.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	5.16 in ²
Rho	0.65 %
Minimum clear spacing	7.93 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-9.0	16.0	0.79	9.0	16.0	0.79	-9.0	-16.0
0.79	9.0	-16.0	1.00	0.0	16.0	1.00	0.0	-16.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	3731.1	0.00	0.00	246.49	34.00	-0.00259	1.00000
X @ Allowable comp.	3731.1	0.00	0.00	246.49	34.00	-0.00259	1.00000
X @ $f_s = 0.0$	2725.6	1175.95	0.00	34.00	34.00	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1862.9	1630.65	0.00	23.76	34.00	0.00129	1.00000
X @ Balanced point	1354.8	1718.73	0.00	18.26	34.00	0.00259	1.00000
X @ Tension control	870.7	1473.18	0.00	11.88	34.00	0.00559	1.00000
X @ Pure bending	0.0	531.40	0.00	2.25	34.00	0.04228	1.00000
X @ Max tension	-387.0	0.00	0.00	0.00	34.00	9.99999	1.00000
-X @ Max compression	3731.1	0.00	0.00	246.49	34.00	-0.00259	1.00000
-X @ Allowable comp.	3731.1	0.00	0.00	246.49	34.00	-0.00259	1.00000
-X @ $f_s = 0.0$	2725.6	-1175.95	0.00	34.00	34.00	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1862.9	-1630.65	0.00	23.76	34.00	0.00129	1.00000
-X @ Balanced point	1354.8	-1718.73	0.00	18.26	34.00	0.00259	1.00000
-X @ Tension control	870.7	-1473.18	0.00	11.88	34.00	0.00559	1.00000
-X @ Pure bending	0.0	-531.40	0.00	2.25	34.00	0.04228	1.00000
-X @ Max tension	-387.0	0.00	0.00	0.00	34.00	9.99999	1.00000

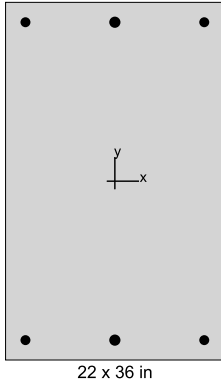
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	37.70	-145.50	37.70	-581.81	2.57	0.03666	1.000	0.25
2	-34.40	145.50	-34.40	485.15	2.05	0.04670	1.000	0.30

7. Diagrams

7.1. PM at $\theta=0$ [deg]



22 x 36 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

f'_c	5 ksi
E_c	4030.51 ksi
f_y	75 ksi
E_s	29000 ksi

Section

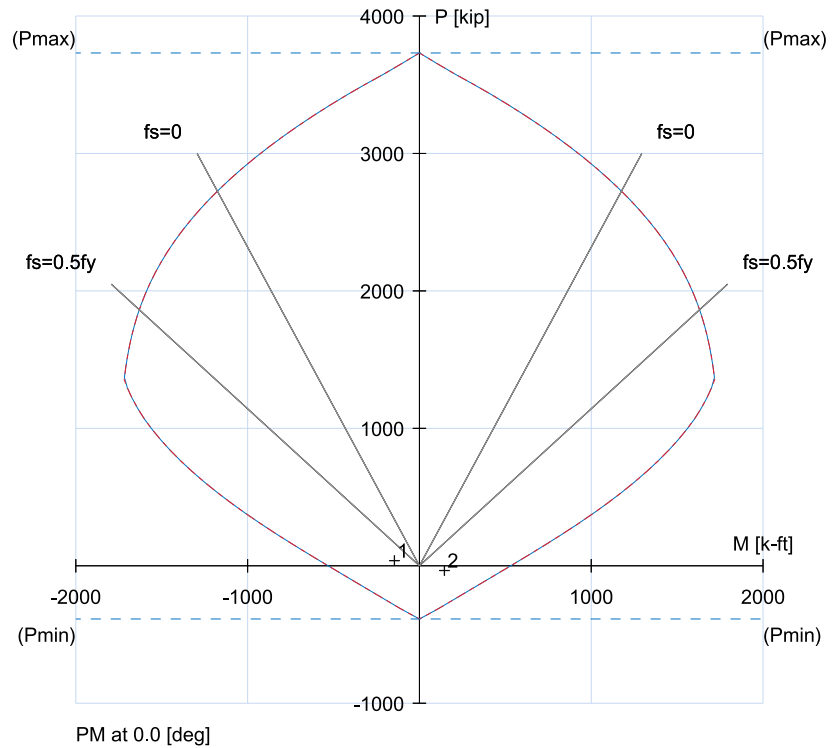
Type	Irregular
A_g	792 in ²
I_x	85536 in ⁴
I_y	31944 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---

Confinement type	Other
------------------	-------

Total steel area, A_s	5.16 in ²
Rho	0.65 %
Min. clear spacing	7.93 in

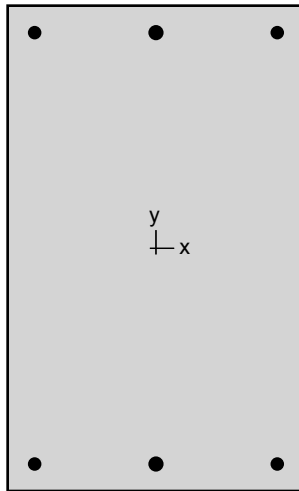


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-34.4	145.5	-34.40	485.15	0.30
1	37.7	-145.5	37.70	-581.81	0.25

Max. Capacity Ratio: 0.30



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

- 1. General Information3
- 2. Material Properties.....3
 - 2.1. Concrete3
 - 2.2. Steel3
- 3. Section.....3
 - 3.1. Shape and Properties.....3
 - 3.2. Section Figure4
 - 3.3. Solids.....4
 - 3.3.1. S1.....4
- 4. Reinforcement4
 - 4.1. Bar Set: ASTM A6154
 - 4.2. Confinement and Factors4
 - 4.3. Arrangement.....5
 - 4.4. Bars Provided5
- 5. Control Points5
- 6. Factored Loads and Moments with Corresponding Capacity Ratios.....5
- 7. Diagrams6
 - 7.1. PM at $\theta=0$ [deg]6

List of Figures

- Figure 1: Column section.....4

1. General Information

File Name	X:\25534-Fire Stati...\SPCOL phiMn - Left MB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	792 in ²
I_x	85536 in ⁴
I_y	31944 in ⁴
r_x	10.3923 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

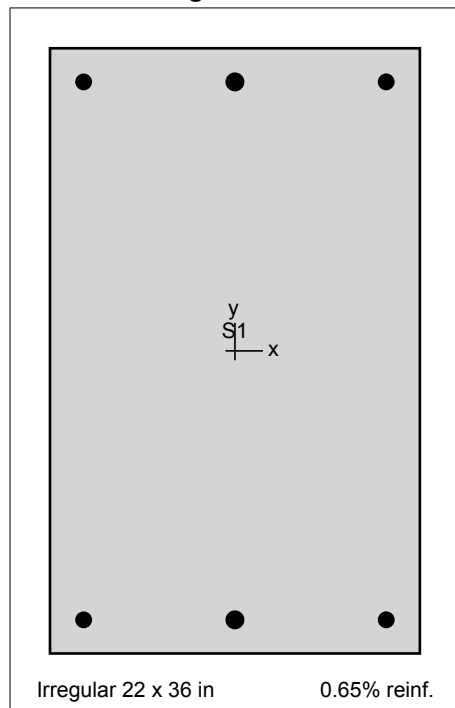


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-18.0	2	11.0	-18.0	3	11.0	18.0
4	-11.0	18.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ , (b)	0.9
Compression controlled ϕ , (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	5.16 in ²
Rho	0.65 %
Minimum clear spacing	7.93 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-9.0	16.0	0.79	9.0	16.0	0.79	-9.0	-16.0
0.79	9.0	-16.0	1.00	0.0	16.0	1.00	0.0	-16.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	2374.9	0.00	0.00	109.55	34.00	-0.00207	0.65000
X @ Allowable comp.	1899.9	592.89	0.00	36.92	34.00	-0.00024	0.65000
X @ $f_s = 0.0$	1746.5	730.83	0.00	34.00	34.00	0.00000	0.65000
X @ $f_s = 0.5 f_y$	1272.3	999.65	0.00	25.28	34.00	0.00103	0.65000
X @ Balanced point	971.2	1070.09	0.00	20.12	34.00	0.00207	0.65000
X @ Tension control	841.1	1276.21	0.00	12.64	34.00	0.00507	0.90000
X @ Pure bending	0.0	385.02	0.00	2.03	34.00	0.04727	0.90000
X @ Max tension	-278.6	0.00	0.00	0.00	34.00	9.99999	0.90000
-X @ Max compression	2374.9	0.00	0.00	109.55	34.00	-0.00207	0.65000
-X @ Allowable comp.	1899.9	-592.89	0.00	36.92	34.00	-0.00024	0.65000
-X @ $f_s = 0.0$	1746.5	-730.83	0.00	34.00	34.00	0.00000	0.65000
-X @ $f_s = 0.5 f_y$	1272.3	-999.65	0.00	25.28	34.00	0.00103	0.65000
-X @ Balanced point	971.2	-1070.09	0.00	20.12	34.00	0.00207	0.65000
-X @ Tension control	841.1	-1276.21	0.00	12.64	34.00	0.00507	0.90000
-X @ Pure bending	0.0	-385.02	0.00	2.03	34.00	0.04727	0.90000
-X @ Max tension	-278.6	0.00	0.00	0.00	34.00	9.99999	0.90000

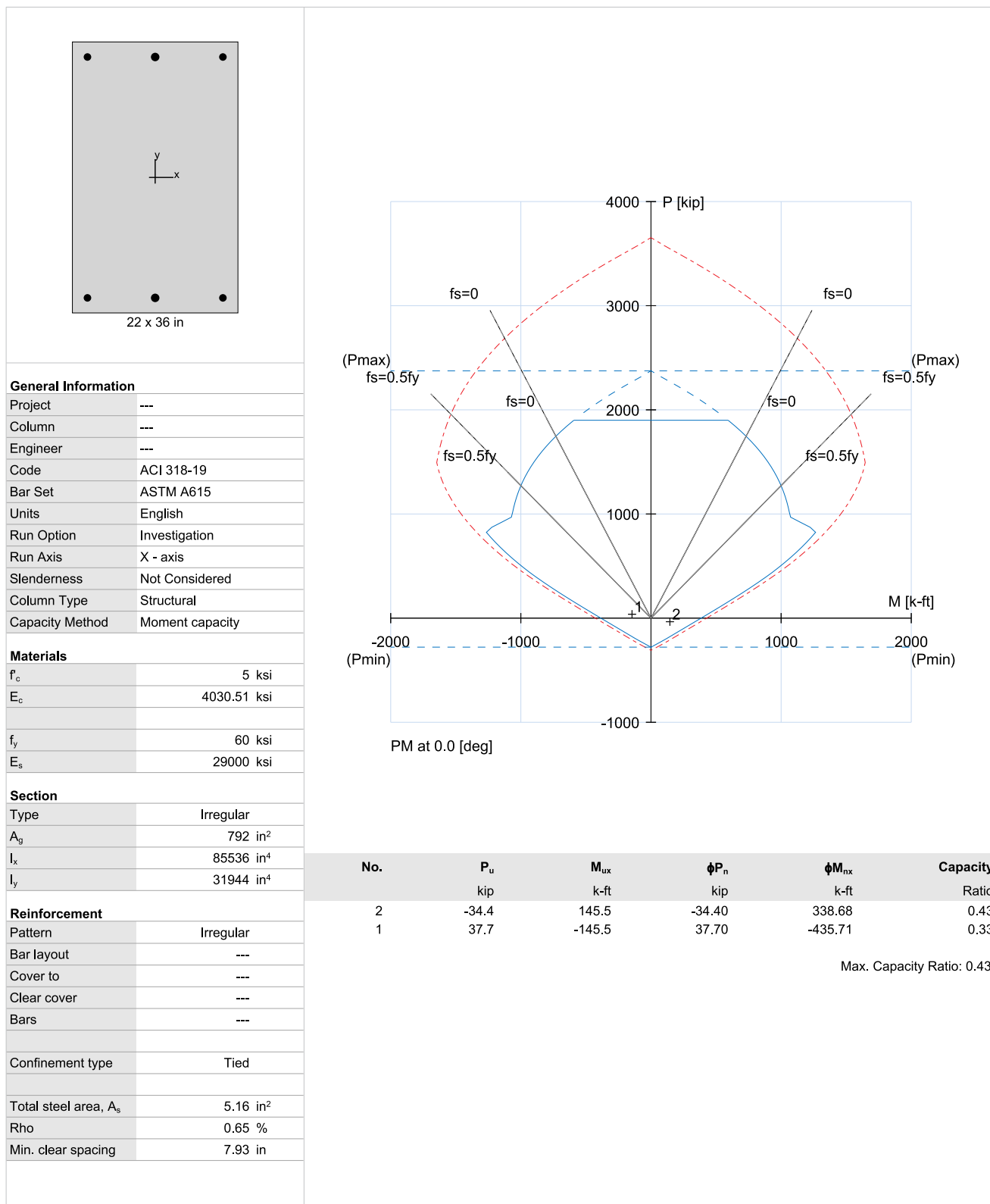
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	37.70	-145.50	37.70	-435.71	2.27	0.04189	0.900	0.33
2	-34.40	145.50	-34.40	338.68	1.83	0.05263	0.900	0.43

7. Diagrams

7.1. PM at $\theta=0$ [deg]



Column Size Parameters

$$f'_c := 5000 \text{ psi}$$

$$f_y := 60000 \text{ psi}$$

$$f_{yt} := 60000 \text{ psi}$$

$$L_n := 14 \text{ ft}$$

Clear Span L_n

$$H := 17.833$$

Story height, Center to Center

$$DbT := 36 \text{ in}$$

Depth of connected beam @TOP

$$WbT := 22 \text{ in}$$

Width of connected beam@TOP

$$DbB := 28 \text{ in}$$

Depth of connected beam @BTM

$$WbB := 26 \text{ in}$$

Width of connected beam@BTM

$$h_c := 33 \text{ in}$$

Col depth

$$b_c := 22 \text{ in}$$

Col width

$$LongBarSize := \text{"\#9"}$$

Longitudinal bar size

$$StirrupSize := \text{"\#5"}$$

Hoop size

$$CC := 1.5 \text{ in}$$

Top Clear Cover

$$Nb := 3$$

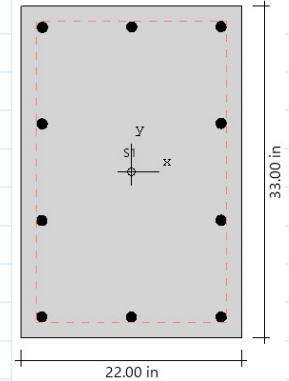
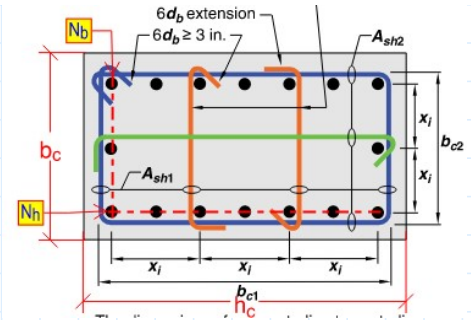
Number of longitudinal bars along side b

$$Nh := 4$$

Number of longitudinal bars along side h

$$n := 2 \cdot (Nb + Nh) - 4 = 10$$

Total Number of longitudinal bars



Column Loads

$$M_u := 146.6 \text{ kip} \cdot \text{ft} \quad \text{Ultimate Moment From Analysis}$$

$$P_{u_{top}} := 43 \text{ kip} \quad \text{Ultimate Axial force @TOP From Analysis}$$

$$P_{u_{btm}} := 85 \cdot \text{kip} \quad \text{Ultimate Axial force @BTM From Analysis}$$

$$V_u := 15.6 \text{ kip} \quad \text{Ultimate Shear force From Analysis}$$

Column Capacity

$$M_{n_{top}} := 709.2 \text{ kip} \cdot \text{ft} \quad \text{Column Mn positive From SPCol}$$

$$M_{n_{btm}} := 827 \text{ kip} \cdot \text{ft} \quad \text{Column Mn negative From SPCol}$$

$$M_{pr_{top}} := 880.3 \text{ kip} \cdot \text{ft} \quad \text{Column Mpr positive From SPCol}$$

$$M_{pr_{btm}} := 991.1 \text{ kip} \cdot \text{ft} \quad \text{Column Mpr negative From SPCol}$$

Beam Capacity TOP

$$M_{nTR} := 448.9 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mn From SPCol}$$

$$M_{nTL} := 0 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mn From SPCol}$$

$$M_{nT} := M_{nTR} + M_{nTL} = 448.9 \text{ kip} \cdot \text{ft}$$

$$M_{prTR} := 552.3 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mpr From SPCol}$$

$$M_{prTL} := 0 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mpr From SPCol}$$

$$M_{prT} := M_{prTR} + M_{prTL} = 552.3 \text{ kip} \cdot \text{ft}$$

Beam Capacity BTM

$$M_{nBR} := 314 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mn From SPCol}$$

$$M_{nBL} := 0 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mn From SPCol}$$

$$M_{nB} := M_{nBR} + M_{nBL} = 314 \text{ kip} \cdot \text{ft}$$

$$M_{prBR} := 385.7 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mpr From SPCol}$$

$$M_{prBL} := 0 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mpr From SPCol}$$

$$M_{prB} := M_{prBR} + M_{prBL} = 385.7 \text{ kip} \cdot \text{ft}$$

$Ld\#9 := 37 \text{ in}$	Seismic Ld for #9		
$Ld\#10 := 42 \text{ in}$	Seismic Ld for #10	Diameter of Bar	Area of Bar
$Ld\#11 := 47 \text{ in}$	Seismic Ld for #11	$db3 := 0.375 \text{ in}$	$ab3 := 0.11 \text{ in}^2$
		$db4 := 0.5 \text{ in}$	$ab4 := 0.2 \text{ in}^2$
		$db5 := 0.625 \text{ in}$	$ab5 := 0.31 \text{ in}^2$
		$db6 := 0.75 \text{ in}$	$ab6 := 0.44 \text{ in}^2$
		$db7 := 0.875 \text{ in}$	$ab7 := 0.6 \text{ in}^2$
		$db8 := 1 \text{ in}$	$ab8 := 0.79 \text{ in}^2$
		$db9 := 1.125 \text{ in}$	$ab9 := 1 \text{ in}^2$
		$db10 := 1.25 \text{ in}$	$ab10 := 1.27 \text{ in}^2$
		$db11 := 1.41 \text{ in}$	$ab11 := 1.56 \text{ in}^2$
$dLb :=$	<pre> if LongBarSize = "#3" return Ld#9 else if LongBarSize = "#10" return Ld#10 else if LongBarSize = "#11" return Ld#11 </pre>	$= 37 \text{ in}$	
$dLb :=$	<pre> if LongBarSize = "#3" return db3 else if LongBarSize = "#4" return db4 else if LongBarSize = "#5" return db5 else if LongBarSize = "#6" return db6 else if LongBarSize = "#7" return db7 else if LongBarSize = "#8" return db8 else if LongBarSize = "#9" return db9 else if LongBarSize = "#10" return db10 else if LongBarSize = "#11" return db11 </pre>	$= 1.125 \text{ in}$	Dia. of one Long. Bar
$aLb :=$	<pre> if LongBarSize = "#3" return ab3 else if LongBarSize = "#4" return ab4 else if LongBarSize = "#5" return ab5 else if LongBarSize = "#6" return ab6 else if LongBarSize = "#7" return ab7 else if LongBarSize = "#8" return ab8 else if LongBarSize = "#9" return ab9 else if LongBarSize = "#10" return ab10 else if LongBarSize = "#11" return ab11 </pre>	$= 1 \text{ in}^2$	Area of one Long. Bar
$dTb :=$	<pre> if StirrupSize = "#3" return db3 else if StirrupSize = "#4" return db4 else if StirrupSize = "#5" return db5 else if StirrupSize = "#6" return db6 else if StirrupSize = "#7" return db7 else if StirrupSize = "#8" return db8 else if StirrupSize = "#9" return db9 else if StirrupSize = "#10" return db10 </pre>	$= 0.625 \text{ in}$	Dia. of one Hoop
$aTb :=$	<pre> if StirrupSize = "#3" return ab3 else if StirrupSize = "#4" return ab4 else if StirrupSize = "#5" return ab5 else if StirrupSize = "#6" return ab6 else if StirrupSize = "#7" return ab7 else if StirrupSize = "#8" return ab8 else if StirrupSize = "#9" return ab9 else if StirrupSize = "#10" return ab10 </pre>	$= 0.31 \text{ in}^2$	Area of one Hoop

$$A_{st} := n \cdot aLb = 10 \text{ in}^2$$

$$d_e := hc - CC - dTb - 0.5 \cdot dLb = 30.313 \text{ in}$$

Effective depth

ACI 18.7.2 Column Dimensional limits

$$ColDimCheck := \begin{cases} \text{if } \min(hc, bc) \geq 12 \cdot \text{in} \wedge \frac{\min(hc, bc)}{\max(hc, bc)} \geq 0.4 & = \text{"O.K"} \\ \text{return "O.K"} \\ \text{else} \\ \text{return "Redesign column Dimensions"} \end{cases}$$

ACI 18.7.3.2 Strong Column Weak Beam Criteria

$$\min(Mn_{bttm}, Mn_{top}) = 709.2 \text{ kip} \cdot \text{ft}$$

$$\frac{6}{5} \cdot \max(MnT, MnB) = 538.68 \text{ kip} \cdot \text{ft}$$

$$MinAstCheck := \begin{cases} \text{if } \min(Mn_{bttm}, Mn_{top}) \geq \frac{6}{5} \cdot \max(MnT, MnB) & = \text{"O.K"} \\ \text{return "O.K"} \\ \text{else} \\ \text{return "Redesign Column"} \end{cases}$$

ACI 18.7.4.1 Min &Max Longitudinal Bar Area

18.7.4 Longitudinal reinforcement

18.7.4.1 Area of longitudinal reinforcement, A_{st} , shall be at least $0.01A_g$ and shall not exceed $0.06A_g$.

$$MinAstCheck := \begin{cases} \text{if } A_{st} \geq 0.01 \cdot hc \cdot bc & = \text{"O.K"} \\ \text{return "O.K"} \\ \text{else} \\ \text{return "Increase Longitudinal bars"} \end{cases}$$

$$MaxAstCheck := \begin{cases} \text{if } A_{st} \leq 0.06 \cdot hc \cdot bc & = \text{"O.K"} \\ \text{return "O.K"} \\ \text{else} \\ \text{return "Reduce Longitudinal bars"} \end{cases}$$

ACI 18.7.4.3 Development Length

18.7.4.3 Over column clear height, longitudinal reinforcement shall be selected such that $1.25\ell_d \leq \ell_u/2$.

$$MaxAstDevLength := \begin{cases} \text{if } 1.25 \cdot Ld \leq \frac{L_n}{2} & = \text{"O.K"} \\ \text{return "O.K"} \\ \text{else} \\ \text{return "Reduce Longitudinal bars size"} \end{cases}$$

ACI 18.7.5.1 Extend of Required Transverse Reinforcements

18.7.5.1 Transverse reinforcement required in 18.7.5.2 through 18.7.5.4 shall be provided over a length ℓ_e from each joint face and on both sides of any section where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior. Length ℓ_e shall be at least the greatest of (a) through (c):

$$L_o := \max\left(hc, \frac{L_n}{16}, 18 \text{ in}\right) = 33 \text{ in}$$

- (a) The depth of the column at the joint face or at the section where flexural yielding is likely to occur
- (b) One-sixth of the clear span of the column
- (c) 18 in.

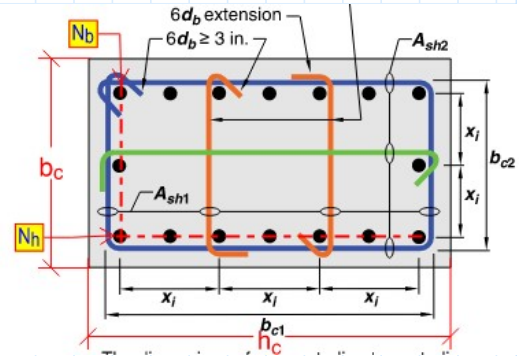
ACI 18.7.5.2 Max Laterally Supported Longitudinal Bar Spacing

$$X_{ih} := \frac{\left(hc - 2 \cdot CC - 2 \cdot dTb - 2 \cdot \frac{dLb}{2} \right)}{N_h - 1} = 9.208 \text{ in}$$

$$X_{ib} := \frac{\left(bc - 2 \cdot CC - 2 \cdot dTb - 2 \cdot \frac{dLb}{2} \right)}{N_b - 1} = 8.313 \text{ in}$$

$$X_i := \max(X_{ih}, X_{ib}) = 9.208 \text{ in}$$

$$MaxAstCheck := \begin{cases} \text{if } X_i \leq 14 \text{ in} \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Reduce Longitudinal bars spacing"} \end{cases} = \text{"O.K"}$$



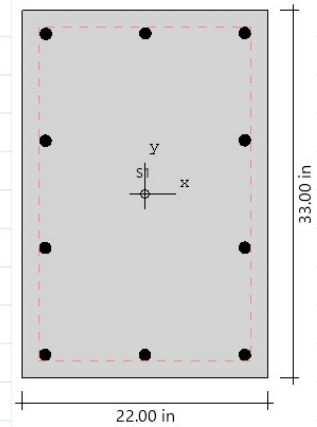
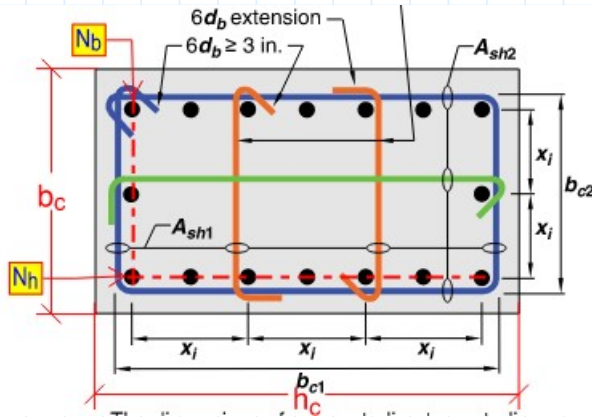
ACI 18.7.5.3 Max Spacing of Transverse Reinforcements Within Lo

$$hx := X_i = 9.208 \text{ in} \quad (\text{max C/C spacing of long. bars laterally supported by corners of stirrups})$$

$$So := \min \left(\min \left(\frac{1}{4} \cdot hc, \frac{1}{4} \cdot bc \right), \begin{cases} \text{if } fy = 60000 \text{ psi} \\ \quad \text{return } 6 \cdot dLb \\ \text{else if } fy = 80000 \text{ psi} \\ \quad \text{return } 5 \cdot dLb \\ \text{else} \\ \quad \text{return "Error"} \end{cases}, 4 + \frac{(14 + hx)}{3} \right) = 6.75 \text{ in}$$

$$So := 4 \text{ in} \quad \text{Spacing of Transverse Reinforcement within length } Lo$$

ACI 18.7.5.4 Amount of Transverse Reinforcements



$$n1 := 10$$

(# of bars supported by corners of rectilinear hoops)

$$Ach := (hc - CC) \cdot (bc - CC) = 645.75 \text{ in}^2$$

$$kf := \begin{cases} \text{if } \frac{f'_c}{25000 \text{ psi}} + 0.6 \geq 1.0 \\ \frac{f'_c}{25000 \text{ psi}} + 0.6 \\ \text{else} \\ 1.0 \end{cases} = 1$$

$$kn := \frac{n1}{n1 - 2} = 1.25$$

Table 18.7.5.4—Transverse reinforcement for columns of special moment frames

Transverse reinforcement	Conditions	Applicable expressions
A_{sh}/sb_c for rectilinear hoop	$P_n \leq 0.3A_g f'_c$ and $f'_c \leq 10,000$ psi	Greater of (a) and (b) $0.3 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (a)
	$P_n > 0.3A_g f'_c$ or $f'_c > 10,000$ psi	Greater of (a), (b), and (c) $0.09 \frac{f'_c}{f_{yt}}$ (b) $0.2k_f k_a \frac{P_n}{f_{yt} A_{ch}}$ (c)
ρ_s for spiral or circular hoop	$P_n \leq 0.3A_g f'_c$ and $f'_c \leq 10,000$ psi	Greater of (d) and (e) $0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (d)
	$P_n > 0.3A_g f'_c$ or $f'_c > 10,000$ psi	Greater of (d), (e), and (f) $0.12 \frac{f'_c}{f_{yt}}$ (e) $0.35k_f \frac{P_n}{f_{yt} A_{ch}}$ (f)

$$a := 0.3 \cdot \left(\frac{hc \cdot bc}{Ach} - 1 \right) \cdot \frac{f'_c}{f_{yt}} = 0.003$$

$$d := 0.45 \cdot \left(\frac{hc \cdot bc}{Ach} - 1 \right) \cdot \frac{f'_c}{f_{yt}} = 0.005$$

$$b := 0.09 \cdot \frac{f'_c}{f_{yt}} = 0.008$$

$$e := 0.12 \cdot \frac{f'_c}{f_{yt}} = 0.01$$

$$c := 0.2 \cdot kf \cdot kn \cdot \frac{\max(Pu_{btm}, Pu_{top})}{f_{yt} \cdot Ach} = 5.485 \cdot 10^{-4}$$

$$f := 0.35 \cdot kf \cdot \frac{\max(Pu_{top}, Pu_{btm})}{f_{yt} \cdot Ach} = 7.678 \cdot 10^{-4}$$

$$Ash1 := \left\| \begin{array}{l} \text{if } \max(Pu_{bttm}, Pu_{top}) \leq 0.3 \cdot hc \cdot bc \cdot f'c \wedge f'c \leq 10000 \text{ psi} \\ \quad \left\| \begin{array}{l} \text{return } \max(a, b) \cdot So \cdot hc \\ \text{else if } \max(Pu_{bttm}, Pu_{top}) > 0.3 \cdot hc \cdot bc \cdot f'c \vee f'c > 10000 \text{ psi} \\ \quad \left\| \text{return } \max(a, b, c) \cdot So \cdot hc \end{array} \right. \end{array} \right\| = 0.99 \text{ in}^2$$

$$Width := hc = 33 \text{ in}$$

$$MinNoLegsAsh1 := \text{ceil} \left(\frac{Ash1}{aTb} \right) = 4$$

Min number of required legs along "hc"

$$Ash1 := MinNoLegsAsh1 \cdot aTb = 1.24 \text{ in}^2$$

$$Ash2 := \left\| \begin{array}{l} \text{if } \max(Pu_{bttm}, Pu_{top}) \leq 0.3 \cdot hc \cdot bc \cdot f'c \wedge f'c \leq 10000 \text{ psi} \\ \quad \left\| \begin{array}{l} \text{return } \max(a, b) \cdot So \cdot bc \\ \text{else if } \max(Pu_{bttm}, Pu_{top}) > 0.3 \cdot hc \cdot bc \cdot f'c \vee f'c > 10000 \text{ psi} \\ \quad \left\| \text{return } \max(a, b, c) \cdot So \cdot bc \end{array} \right. \end{array} \right\| = 0.66 \text{ in}^2$$

$$Depth := bc = 22 \text{ in}$$

$$MinNoLegsAsh2 := \text{ceil} \left(\frac{Ash2}{aTb} \right) = 3$$

Min number of required legs along "bc"

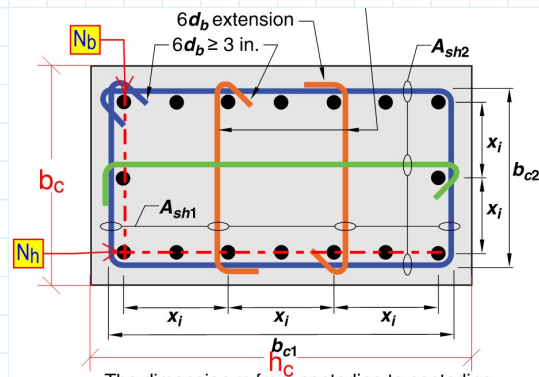
$$Ash2 := MinNoLegsAsh2 \cdot aTb = 0.93 \text{ in}^2$$

ACI 18.7.6.1 Design Shear Reinforcement Beyond Length L_o $V_c=0$

$$V_{pr} := \frac{M_{prT} + M_{prB}}{L_n} = 67 \text{ kip}$$

$$S_{req} := \frac{Ash2 \cdot 0.75 \cdot f_{yt} \cdot d_e}{V_{pr} + V_u} = 15.358 \text{ in}$$

$$S := \min \left(S_{req}, \left\| \begin{array}{l} \text{if } f_y = 60000 \text{ psi} \\ \quad \left\| \begin{array}{l} \text{return } 6 \cdot dLb \\ \text{else if } f_y = 80000 \text{ psi} \\ \quad \left\| \text{return } 5 \cdot dLb \\ \text{else} \\ \quad \left\| \text{return "Error"} \end{array} \right. \end{array} \right. \right\| \right) = 6.75 \text{ in}$$



$$S := 6 \text{ in}$$

Spacing of Transverse Reinforcement beyond length L_o

ACI 18.8.4.3 Nominal Joint Shear Strength

$$A_j := hc \cdot bc = 726 \text{ in}^2$$

$$V_e := \frac{\max(MnT, MnB)}{\min(DbT, DbB)} = 192.386 \text{ kip}$$

$$V_n := 8 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot \text{psi} \cdot A_j = 410.688 \text{ kip}$$

$$\text{Shear} := \begin{cases} \text{if } V_e \leq V_n \\ \text{return "O.K"} \\ \text{else} \\ \text{return "N.G"} \end{cases} = \text{"O.K"}$$

Table 18.8.4.3—Nominal joint shear strength V_n

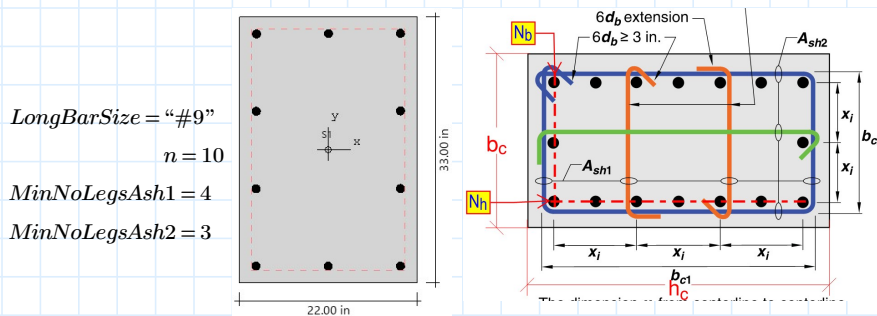
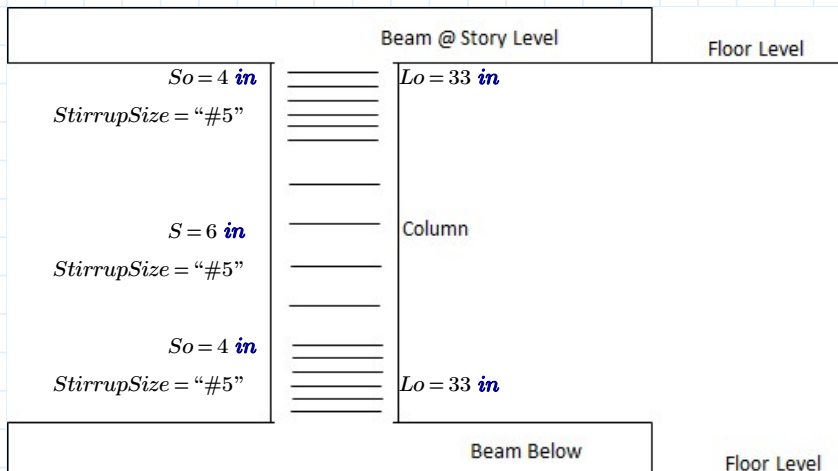
Column	Beam in direction of V_n	Confinement by transverse beams according to 15.2.8	V_n , lb ⁽¹⁾
Continuous or meets 15.2.6	Continuous or meets 15.2.7	Confined	$20\lambda\sqrt{f'_c}A_j$
		Not confined	$15\lambda\sqrt{f'_c}A_j$
	Other	Confined	$15\lambda\sqrt{f'_c}A_j$
		Not confined	$12\lambda\sqrt{f'_c}A_j$
Other	Continuous or meets 15.2.7	Confined	$15\lambda\sqrt{f'_c}A_j$
		Not confined	$12\lambda\sqrt{f'_c}A_j$
	Other	Confined	$12\lambda\sqrt{f'_c}A_j$
		Not confined	$8\lambda\sqrt{f'_c}A_j$

⁽¹⁾ λ shall be 0.75 for lightweight concrete and 1.0 for normalweight concrete. A_j shall be calculated in accordance with 15.4.2.4.

15.4.2.4 Effective cross-sectional area within a joint, A_j , shall be calculated as the product of joint depth and effective joint width. Joint depth shall be the overall depth of the column, h , in the direction of joint shear considered. Effective joint width shall be the overall width of the column where the beam is wider than the column. Where the column is wider than the beam, effective joint width shall not exceed the lesser of (a) and (b):

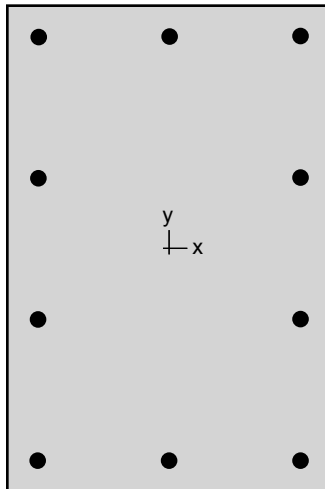
- (a) Beam width plus joint depth
- (b) Twice the perpendicular distance from longitudinal axis of beam to nearest side face of the column

Column Design Summary





spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

- 1. General Information3
- 2. Material Properties.....3
 - 2.1. Concrete3
 - 2.2. Steel3
- 3. Section.....3
 - 3.1. Shape and Properties.....3
 - 3.2. Section Figure4
 - 3.3. Solids.....4
 - 3.3.1. S1.....4
- 4. Reinforcement4
 - 4.1. Bar Set: ASTM A6154
 - 4.2. Confinement and Factors4
 - 4.3. Arrangement.....5
 - 4.4. Bars Provided5
- 5. Control Points5
- 6. Factored Loads and Moments with Corresponding Capacity Ratios.....5
- 7. Diagrams6
 - 7.1. PM at $\theta=0$ [deg]6

List of Figures

- Figure 1: Column section.....4

1. General Information

File Name	X:\25534-Fire Statio...\SPCOL Mn - Col Right.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴
r_x	9.52628 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

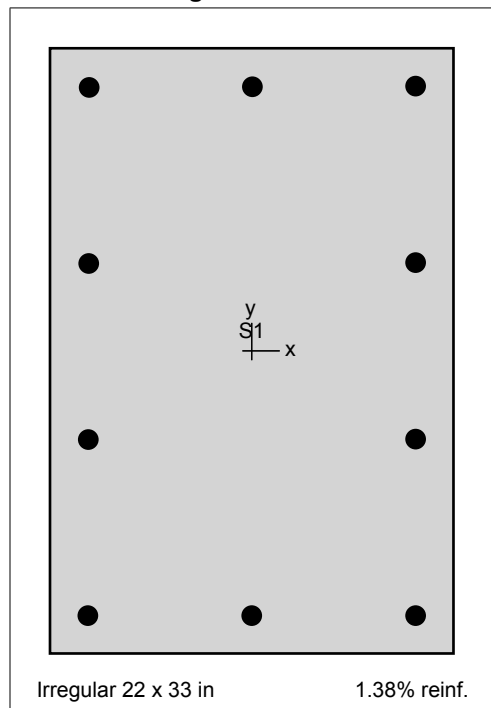


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-16.5	2	11.0	-16.5	3	11.0	16.5
4	-11.0	16.5						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Minimum clear spacing	7.77 in

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
1.00	-8.9	14.4	1.00	0.0	14.4	1.00	8.9	14.4
1.00	-8.9	-14.4	1.00	0.0	-14.4	1.00	8.9	-14.4
1.00	-8.9	-4.8	1.00	-8.9	4.8	1.00	8.9	-4.8
1.00	8.9	4.8						

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	3643.0	0.83	0.83	99.68	30.94	-0.00207	1.00000
X @ Allowable comp.	3643.0	0.83	0.83	99.68	30.94	-0.00207	1.00000
X @ $f_s = 0.0$	2627.1	1018.27	-0.11	30.94	30.94	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1888.5	1381.10	-0.57	23.00	30.94	0.00103	1.00000
X @ Balanced point	1383.5	1497.07	-1.04	18.31	30.94	0.00207	1.00000
X @ Tension control	724.9	1317.13	-0.91	11.50	30.94	0.00507	1.00000
X @ Pure bending	0.0	731.99	-0.89	4.07	30.94	0.01982	1.00000
X @ Max tension	-600.0	-0.89	-0.89	0.00	30.94	9.99999	1.00000
-X @ Max compression	3643.0	0.83	0.83	99.68	30.94	-0.00207	1.00000
-X @ Allowable comp.	3643.0	0.83	0.83	99.68	30.94	-0.00207	1.00000
-X @ $f_s = 0.0$	2626.4	-1017.70	0.89	30.94	30.94	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1887.5	-1381.08	0.87	23.00	30.94	0.00103	1.00000
-X @ Balanced point	1382.3	-1497.65	0.83	18.31	30.94	0.00207	1.00000
-X @ Tension control	724.4	-1317.32	0.64	11.50	30.94	0.00507	1.00000
-X @ Pure bending	0.0	-732.75	1.11	4.09	30.94	0.01970	1.00000
-X @ Max tension	-600.0	-0.89	-0.89	0.00	30.94	9.99999	1.00000

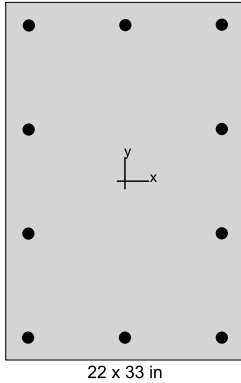
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand P_u kip	M_{ux} k-ft	Capacity ϕP_n kip	ϕM_{nx} k-ft	Parameters at Capacity NA Depth in	ϵ_t	ϕ	Capacity Ratio
1	85.00	-146.60	85.00	-826.94	4.92	0.01586	1.000	0.18
2	-20.00	146.60	-20.00	709.23	3.88	0.02090	1.000	0.21

7. Diagrams

7.1. PM at $\theta=0$ [deg]



General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

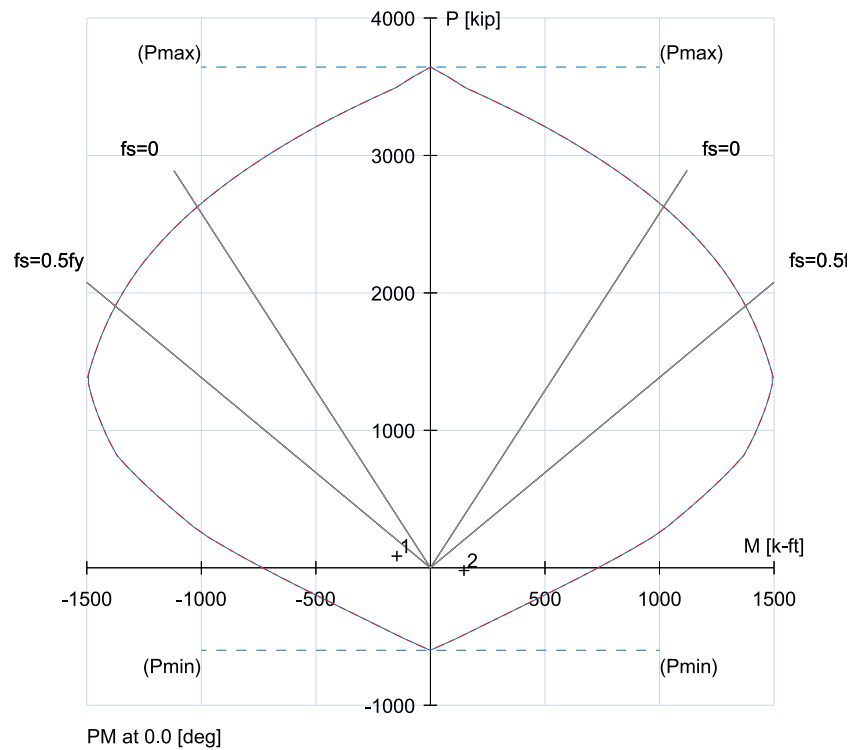
f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Other
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Min. clear spacing	7.77 in

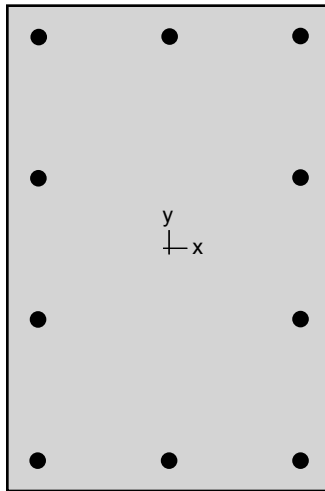


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-20.0	146.6	-20.00	709.23	0.21
1	85.0	-146.6	85.00	-826.94	0.18

Max. Capacity Ratio: 0.21



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

- 1. General Information3
- 2. Material Properties.....3
 - 2.1. Concrete3
 - 2.2. Steel3
- 3. Section.....3
 - 3.1. Shape and Properties.....3
 - 3.2. Section Figure4
 - 3.3. Solids.....4
 - 3.3.1. S1.....4
- 4. Reinforcement4
 - 4.1. Bar Set: ASTM A6154
 - 4.2. Confinement and Factors4
 - 4.3. Arrangement.....5
 - 4.4. Bars Provided5
- 5. Control Points5
- 6. Factored Loads and Moments with Corresponding Capacity Ratios.....5
- 7. Diagrams6
 - 7.1. PM at $\theta=0$ [deg]6

List of Figures

- Figure 1: Column section.....4

1. General Information

File Name	X:\25534-Fire Stati...\SPCOL Mpr - Col Right.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	75 ksi
E_s	29000 ksi
ϵ_{ty}	0.00258621 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴
r_x	9.52628 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

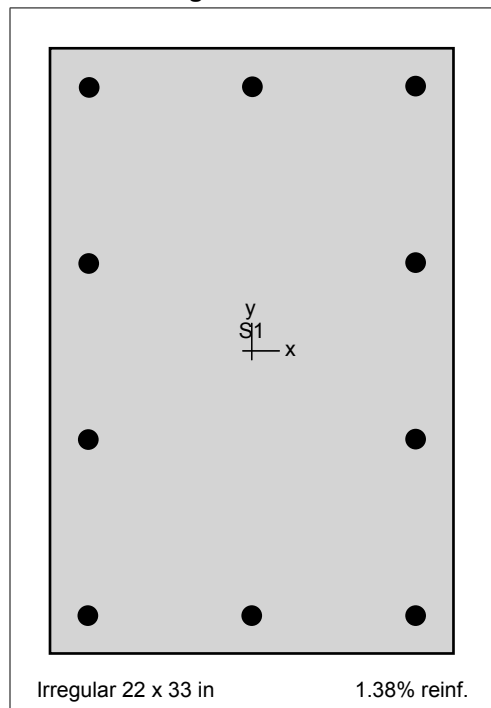


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-16.5	2	11.0	-16.5	3	11.0	16.5
4	-11.0	16.5						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Minimum clear spacing	7.77 in

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
1.00	-8.9	14.4	1.00	0.0	14.4	1.00	8.9	14.4
1.00	-8.9	-14.4	1.00	0.0	-14.4	1.00	8.9	-14.4
1.00	-8.9	-4.8	1.00	-8.9	4.8	1.00	8.9	-4.8
1.00	8.9	4.8						

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	3793.0	1.06	1.06	224.28	30.94	-0.00259	1.00000
X @ Allowable comp.	3793.0	1.06	1.06	224.28	30.94	-0.00259	1.00000
X @ $f_s = 0.0$	2672.1	1072.41	-0.11	30.94	30.94	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1791.4	1475.69	-0.69	21.62	30.94	0.00129	1.00000
X @ Balanced point	1224.8	1582.09	-1.28	16.61	30.94	0.00259	1.00000
X @ Tension control	618.0	1383.31	-1.13	10.81	30.94	0.00559	1.00000
X @ Pure bending	0.0	901.77	-1.12	5.11	30.94	0.01516	1.00000
X @ Max tension	-750.0	-1.12	-1.12	0.00	30.94	9.99999	1.00000
-X @ Max compression	3793.0	1.06	1.06	224.28	30.94	-0.00259	1.00000
-X @ Allowable comp.	3793.0	1.06	1.06	224.28	30.94	-0.00259	1.00000
-X @ $f_s = 0.0$	2671.4	-1071.70	1.03	30.94	30.94	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1790.4	-1475.69	0.99	21.62	30.94	0.00129	1.00000
-X @ Balanced point	1223.5	-1582.82	0.95	16.61	30.94	0.00259	1.00000
-X @ Tension control	616.5	-1382.61	1.12	10.81	30.94	0.00559	1.00000
-X @ Pure bending	0.0	-902.67	0.87	5.13	30.94	0.01510	1.00000
-X @ Max tension	-750.0	-1.12	-1.12	0.00	30.94	9.99999	1.00000

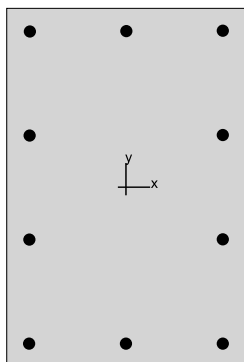
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand P_u kip	M_{ux} k-ft	Capacity ϕP_n kip	ϕM_{nx} k-ft	Parameters at Capacity NA Depth in	ϵ_t	ϕ	Capacity Ratio
1	85.00	-146.60	85.00	-991.04	6.05	0.01235	1.000	0.15
2	-20.00	146.60	-20.00	880.26	4.90	0.01593	1.000	0.17

7. Diagrams

7.1. PM at $\theta=0$ [deg]



22 x 33 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

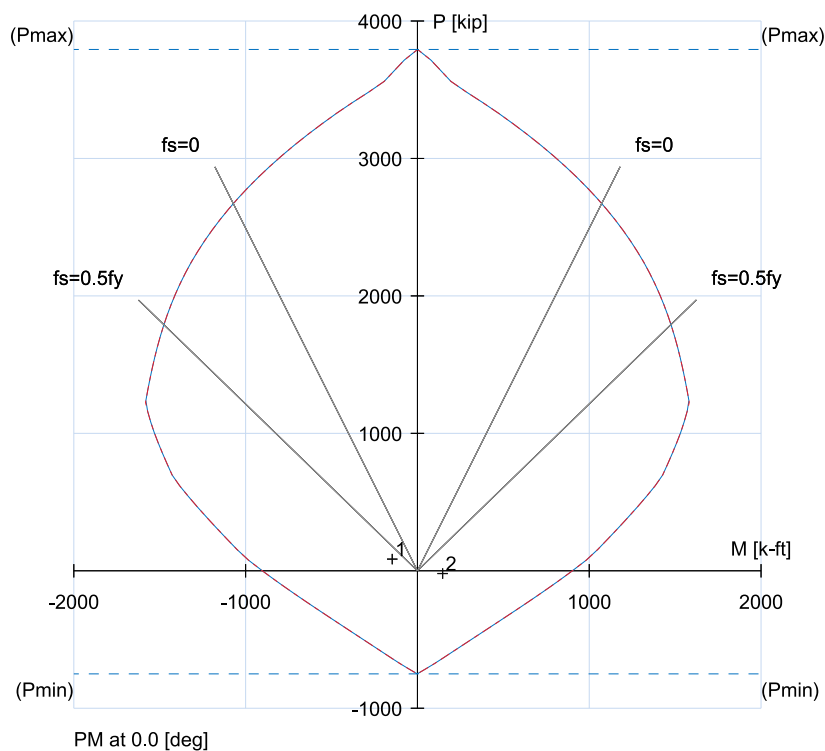
f'_c	5 ksi
E_c	4030.51 ksi
f_y	75 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Other
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Min. clear spacing	7.77 in

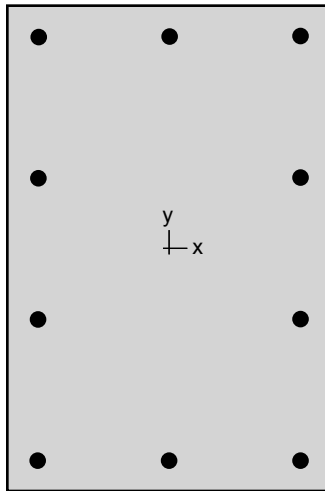


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-20.0	146.6	-20.00	880.26	0.17
1	85.0	-146.6	85.00	-991.04	0.15

Max. Capacity Ratio: 0.17



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Sta...\SPCOL phiMn - Col Right.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴
r_x	9.52628 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

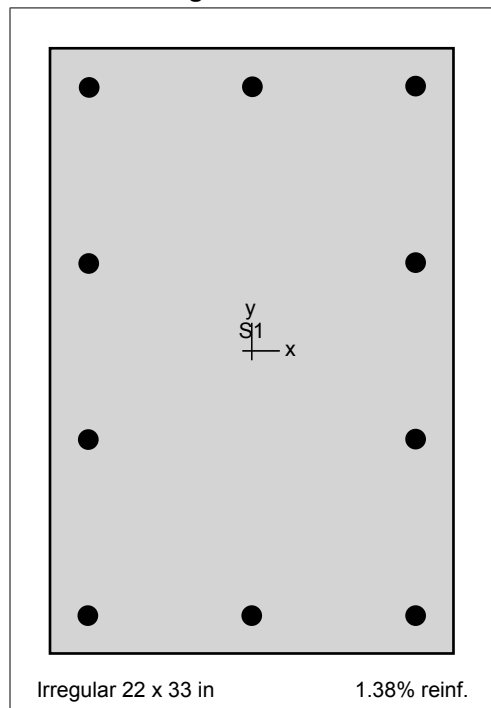


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-16.5	2	11.0	-16.5	3	11.0	16.5
4	-11.0	16.5						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ , (b)	0.9
Compression controlled ϕ , (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Minimum clear spacing	7.77 in

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
1.00	-8.9	14.4	1.00	0.0	14.4	1.00	8.9	14.4
1.00	-8.9	-14.4	1.00	0.0	-14.4	1.00	8.9	-14.4
1.00	-8.9	-4.8	1.00	-8.9	4.8	1.00	8.9	-4.8
1.00	8.9	4.8						

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d_t Depth in	ϵ_t	ϕ
X @ Max compression	2367.9	0.54	0.54	99.68	30.94	-0.00207	0.65000
X @ Allowable comp.	1894.4	514.46	0.02	34.21	30.94	-0.00029	0.65000
X @ $f_s = 0.0$	1707.6	661.88	-0.07	30.94	30.94	0.00000	0.65000
X @ $f_s = 0.5 f_y$	1227.5	897.71	-0.37	23.00	30.94	0.00103	0.65000
X @ Balanced point	899.3	973.10	-0.68	18.31	30.94	0.00207	0.65000
X @ Tension control	652.4	1185.42	-0.82	11.50	30.94	0.00507	0.90000
X @ Pure bending	0.0	658.79	-0.81	4.07	30.94	0.01982	0.90000
X @ Max tension	-540.0	-0.81	-0.81	0.00	30.94	9.99999	0.90000
-X @ Max compression	2367.9	0.54	0.54	99.68	30.94	-0.00207	0.65000
-X @ Allowable comp.	1894.4	-513.62	0.59	34.21	30.94	-0.00029	0.65000
-X @ $f_s = 0.0$	1707.2	-661.51	0.58	30.94	30.94	0.00000	0.65000
-X @ $f_s = 0.5 f_y$	1226.9	-897.70	0.56	23.00	30.94	0.00103	0.65000
-X @ Balanced point	898.5	-973.47	0.54	18.31	30.94	0.00207	0.65000
-X @ Tension control	652.0	-1185.59	0.58	11.50	30.94	0.00507	0.90000
-X @ Pure bending	0.0	-659.47	1.00	4.09	30.94	0.01970	0.90000
-X @ Max tension	-540.0	-0.81	-0.81	0.00	30.94	9.99999	0.90000

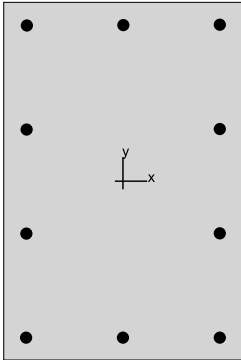
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand P_u kip	M_{ux} k-ft	Capacity ϕP_n kip	ϕM_{nx} k-ft	Parameters at Capacity NA Depth in	ϵ_t	ϕ	Capacity Ratio
1	85.00	-146.60	85.00	-753.42	5.02	0.01549	0.900	0.19
2	-20.00	146.60	-20.00	636.02	3.86	0.02102	0.900	0.23

7. Diagrams

7.1. PM at $\theta=0$ [deg]



22 x 33 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi

Section

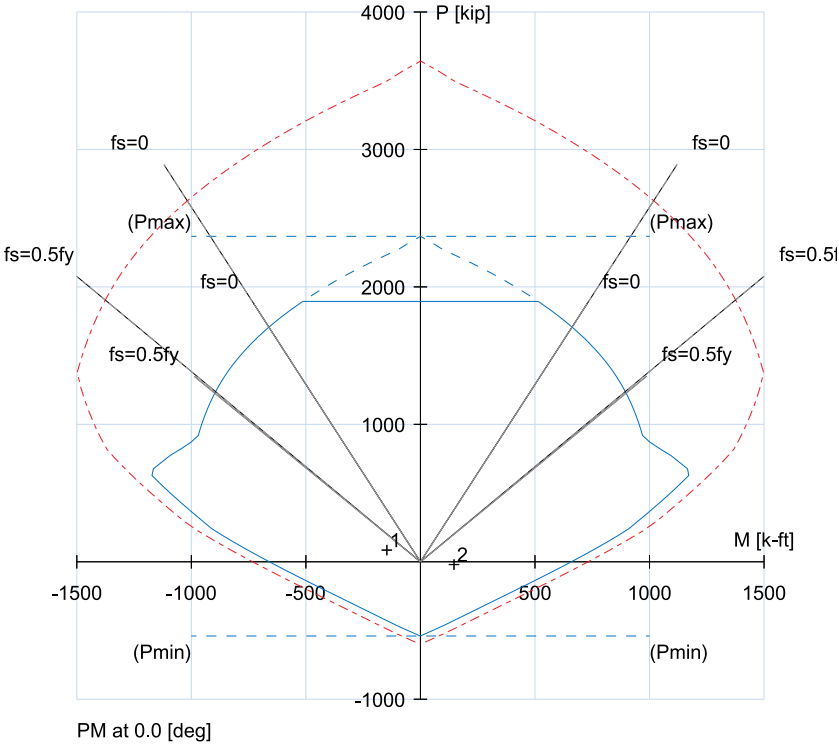
Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---

Confinement type Tied

Total steel area, A_s	10.00 in ²
Rho	1.38 %
Min. clear spacing	7.77 in



No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-20.0	146.6	-20.00	636.02	0.23
1	85.0	-146.6	85.00	-753.42	0.19

Max. Capacity Ratio: 0.23

Column Size Parameters

$$f'_c := 5000 \text{ psi}$$

$$f_y := 60000 \text{ psi}$$

$$f_{yt} := 60000 \text{ psi}$$

$$L_n := 14 \text{ ft}$$

Clear Span L_n

$$H := 17.833$$

Story height, Center to Center

$$D_{bT} := 36 \text{ in}$$

Depth of connected beam @TOP

$$W_{bT} := 22 \text{ in}$$

Width of connected beam@TOP

$$D_{bB} := 28 \text{ in}$$

Depth of connected beam @BTM

$$W_{bB} := 26 \text{ in}$$

Width of connected beam@BTM

$$h_c := 38 \text{ in}$$

Col depth

$$b_c := 22 \text{ in}$$

Col width

$$LongBarSize := \text{"\#10"}$$

Longitudinal bar size

$$StirrupSize := \text{"\#5"}$$

Hoop size

$$CC := 1.5 \text{ in}$$

Top Clear Cover

$$N_b := 3$$

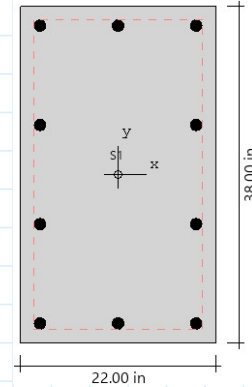
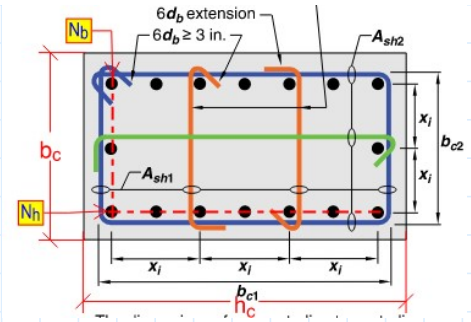
Number of longitudinal bars along side b

$$N_h := 4$$

Number of longitudinal bars along side h

$$n := 2 \cdot (N_b + N_h) - 4 = 10$$

Total Number of longitudinal bars



Column Loads

$$M_u := 223.3 \text{ kip} \cdot \text{ft} \quad \text{Ultimate Moment From Analysis}$$

$$P_{u_{top}} := 37.4 \text{ kip} \quad \text{Ultimate Axial force @TOP From Analysis}$$

$$P_{u_{btm}} := 81.2 \text{ kip} \quad \text{Ultimate Axial force @BTM From Analysis}$$

$$V_u := 49.3 \text{ kip} \quad \text{Ultimate Shear force From Analysis}$$

Column Capacity

$$M_{n_{top}} := 1124.9 \text{ kip} \cdot \text{ft} \quad \text{Column Mn positive From SPCol}$$

$$M_{n_{btm}} := 1180.3 \text{ kip} \cdot \text{ft} \quad \text{Column Mn negative From SPCol}$$

$$M_{pr_{top}} := 1369.9 \text{ kip} \cdot \text{ft} \quad \text{Column Mpr positive From SPCol}$$

$$M_{pr_{btm}} := 1421.2 \text{ kip} \cdot \text{ft} \quad \text{Column Mpr negative From SPCol}$$

Beam Capacity TOP

$$M_{nTR} := 448.9 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mn From SPCol}$$

$$M_{nTL} := 478.5 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mn From SPCol}$$

$$M_{nT} := M_{nTR} + M_{nTL} = 927.4 \text{ kip} \cdot \text{ft}$$

$$M_{prTR} := 552.3 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mpr From SPCol}$$

$$M_{prTL} := 581.8 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mpr From SPCol}$$

$$M_{prT} := M_{prTR} + M_{prTL} = 1134.1 \text{ kip} \cdot \text{ft}$$

Beam Capacity BTM

$$M_{nBR} := 314 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mn From SPCol}$$

$$M_{nBL} := 314.8 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mn From SPCol}$$

$$M_{nB} := M_{nBR} + M_{nBL} = 628.8 \text{ kip} \cdot \text{ft}$$

$$M_{prBR} := 385.7 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mpr From SPCol}$$

$$M_{prBL} := 386.5 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mpr From SPCol}$$

$$M_{prB} := M_{prBR} + M_{prBL} = 772.2 \text{ kip} \cdot \text{ft}$$

$Ld\#9 := 37 \text{ in}$	Seismic Ld for #9				
$Ld\#10 := 42 \text{ in}$	Seismic Ld for #10		Diameter of Bar	Area of Bar	
$Ld\#11 := 47 \text{ in}$	Seismic Ld for #11		$db3 := 0.375 \text{ in}$	$ab3 := 0.11 \text{ in}^2$	
			$db4 := 0.5 \text{ in}$	$ab4 := 0.2 \text{ in}^2$	
			$db5 := 0.625 \text{ in}$	$ab5 := 0.31 \text{ in}^2$	
			$db6 := 0.75 \text{ in}$	$ab6 := 0.44 \text{ in}^2$	
			$db7 := 0.875 \text{ in}$	$ab7 := 0.6 \text{ in}^2$	
			$db8 := 1 \text{ in}$	$ab8 := 0.79 \text{ in}^2$	
			$db9 := 1.125 \text{ in}$	$ab9 := 1 \text{ in}^2$	
			$db10 := 1.25 \text{ in}$	$ab10 := 1.27 \text{ in}^2$	
			$db11 := 1.41 \text{ in}$	$ab11 := 1.56 \text{ in}^2$	
$dLb :=$	if LongBarSize = "#3"	= 1.25 in	$aLb :=$	if LongBarSize = "#3"	= 1.27 in ²
	return db3	Dia. of one		return ab3	Area of one
	else if LongBarSize = "#4"	Long. Bar		else if LongBarSize = "#4"	Long. Bar
	return db4			return ab4	
	else if LongBarSize = "#5"			else if LongBarSize = "#5"	
	return db5			return ab5	
	else if LongBarSize = "#6"			else if LongBarSize = "#6"	
	return db6			return ab6	
	else if LongBarSize = "#7"			else if LongBarSize = "#7"	
	return db7			return ab7	
	else if LongBarSize = "#8"			else if LongBarSize = "#8"	
	return db8			return ab8	
	else if LongBarSize = "#9"			else if LongBarSize = "#9"	
	return db9			return ab9	
	else if LongBarSize = "#10"			else if LongBarSize = "#10"	
	return db10			return ab10	
	else if LongBarSize = "#11"			else if LongBarSize = "#11"	
	return db11			return ab11	
$dTb :=$	if StirrupSize = "#3"	= 0.625 in	$aTb :=$	if StirrupSize = "#3"	= 0.31 in ²
	return db3	Dia. of one		return ab3	Area of one
	else if StirrupSize = "#4"	Hoop		else if StirrupSize = "#4"	Hoop
	return db4			return ab4	
	else if StirrupSize = "#5"			else if StirrupSize = "#5"	
	return db5			return ab5	
	else if StirrupSize = "#6"			else if StirrupSize = "#6"	
	return db6			return ab6	
	else if StirrupSize = "#7"			else if StirrupSize = "#7"	
	return db7			return ab7	
	else if StirrupSize = "#8"			else if StirrupSize = "#8"	
	return db8			return ab8	
	else if StirrupSize = "#9"			else if StirrupSize = "#9"	
	return db9			return ab9	
	else if StirrupSize = "#10"			else if StirrupSize = "#10"	
	return db10			return ab10	

$$A_{st} := n \cdot aLb = 12.7 \text{ in}^2$$

$$de := hc - CC - dTb - 0.5 \cdot dLb = 35.25 \text{ in}$$

Effective depth

ACI 18.7.2 Column Dimensional limits

$$ColDimCheck := \begin{cases} \text{if } \min(hc, bc) \geq 12 \cdot \text{in} \wedge \frac{\min(hc, bc)}{\max(hc, bc)} \geq 0.4 \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Redesign column Dimensions"} \end{cases} = \text{"O.K"}$$

ACI 18.7.3.2 Strong Column Weak Beam Criteria

$$\min(Mn_{bttm}, Mn_{top}) = 1124.9 \text{ kip} \cdot \text{ft}$$

$$\frac{6}{5} \cdot \max(MnT, MnB) = 1112.88 \text{ kip} \cdot \text{ft}$$

$$MinAstCheck := \begin{cases} \text{if } \min(Mn_{bttm}, Mn_{top}) \geq \frac{6}{5} \cdot \max(MnT, MnB) \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Redesign Column"} \end{cases} = \text{"O.K"}$$

ACI 18.7.4.1 Min &Max Longitudinal Bar Area

18.7.4 Longitudinal reinforcement

18.7.4.1 Area of longitudinal reinforcement, A_{st} , shall be at least $0.01A_g$ and shall not exceed $0.06A_g$.

$$MinAstCheck := \begin{cases} \text{if } A_{st} \geq 0.01 \cdot hc \cdot bc \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Increase Longitudinal bars"} \end{cases} = \text{"O.K"}$$

$$MaxAstCheck := \begin{cases} \text{if } A_{st} \leq 0.06 \cdot hc \cdot bc \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Reduce Longitudinal bars"} \end{cases} = \text{"O.K"}$$

ACI 18.7.4.3 Development Length

18.7.4.3 Over column clear height, longitudinal reinforcement shall be selected such that $1.25\ell_d \leq \ell_u/2$.

$$MaxAstDevLength := \begin{cases} \text{if } 1.25 \cdot Ld \leq \frac{L_n}{2} \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Reduce Longitudinal bars size"} \end{cases} = \text{"O.K"}$$

ACI 18.7.5.1 Extend of Required Transverse Reinforcements

$$Lo := \max\left(hc, \frac{L_n}{16}, 18 \text{ in}\right) = 38 \text{ in}$$

18.7.5.1 Transverse reinforcement required in 18.7.5.2 through 18.7.5.4 shall be provided over a length ℓ_e from each joint face and on both sides of any section where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior. Length ℓ_e shall be at least the greatest of (a) through (c):

- (a) The depth of the column at the joint face or at the section where flexural yielding is likely to occur
- (b) One-sixth of the clear span of the column
- (c) 18 in.

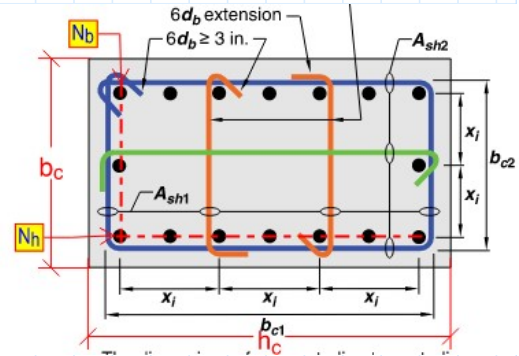
ACI 18.7.5.2 Max Laterally Supported Longitudinal Bar Spacing

$$X_{ih} := \frac{\left(h_c - 2 \cdot CC - 2 \cdot dTb - 2 \cdot \frac{dLb}{2} \right)}{N_h - 1} = 10.833 \text{ in}$$

$$X_{ib} := \frac{\left(b_c - 2 \cdot CC - 2 \cdot dTb - 2 \cdot \frac{dLb}{2} \right)}{N_b - 1} = 8.25 \text{ in}$$

$$X_i := \max(X_{ih}, X_{ib}) = 10.833 \text{ in}$$

$$MaxAstCheck := \begin{cases} \text{if } X_i \leq 14 \text{ in} \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Reduce Longitudinal bars spacing"} \end{cases} = \text{"O.K"}$$



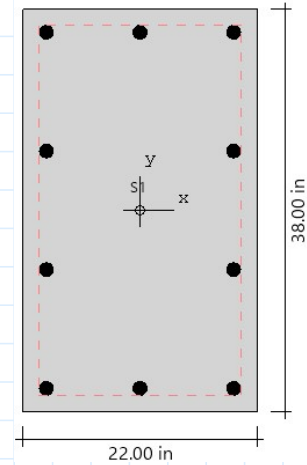
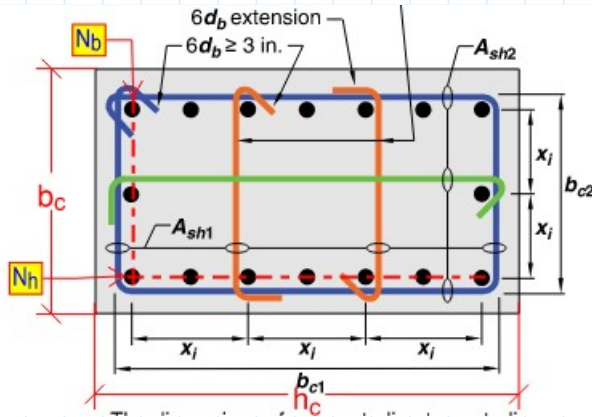
ACI 18.7.5.3 Max Spacing of Transverse Reinforcements Within Lo

$$hx := X_i = 10.833 \text{ in} \quad (\text{max C/C spacing of long. bars laterally supported by corners of stirrups})$$

$$So := \min \left(\min \left(\frac{1}{4} \cdot h_c, \frac{1}{4} \cdot b_c \right), \begin{cases} \text{if } f_y = 60000 \text{ psi} \\ \quad \text{return } 6 \cdot dLb \\ \text{else if } f_y = 80000 \text{ psi} \\ \quad \text{return } 5 \cdot dLb \\ \text{else} \\ \quad \text{return "Error"} \end{cases}, 4 + \frac{(14 + hx)}{3} \right) = 7.5 \text{ in}$$

$$So := 4 \text{ in} \quad \text{Spacing of Transverse Reinforcement within length } L_o$$

ACI 18.7.5.4 Amount of Transverse Reinforcements



$$n1 := 10$$

(# of bars supported by corners of rectilinear hoops)

$$Ach := (hc - CC) \cdot (bc - CC) = 748.25 \text{ in}^2$$

$$kf := \begin{cases} \text{if } \frac{f'_c}{25000 \text{ psi}} + 0.6 \geq 1.0 \\ \frac{f'_c}{25000 \text{ psi}} + 0.6 \\ \text{else} \\ 1.0 \end{cases} = 1$$

$$kn := \frac{n1}{n1 - 2} = 1.25$$

Table 18.7.5.4—Transverse reinforcement for columns of special moment frames

Transverse reinforcement	Conditions	Applicable expressions
$A_{sh}/s b_c$ for rectilinear hoop	$P_n \leq 0.3 A_g f'_c$ and $f'_c \leq 10,000$ psi	Greater of (a) and (b) $0.3 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (a)
	$P_n > 0.3 A_g f'_c$ or $f'_c > 10,000$ psi	Greater of (a), (b), and (c) $0.09 \frac{f'_c}{f_{yt}}$ (b) $0.2 k_f k_a \frac{P_n}{f_{yt} A_{ch}}$ (c)
ρ , for spiral or circular hoop	$P_n \leq 0.3 A_g f'_c$ and $f'_c \leq 10,000$ psi	Greater of (d) and (e) $0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (d)
	$P_n > 0.3 A_g f'_c$ or $f'_c > 10,000$ psi	Greater of (d), (e), and (f) $0.12 \frac{f'_c}{f_{yt}}$ (e) $0.35 k_f \frac{P_n}{f_{yt} A_{ch}}$ (f)

$$a := 0.3 \cdot \left(\frac{hc \cdot bc}{Ach} - 1 \right) \cdot \frac{f'_c}{f_{yt}} = 0.003$$

$$d := 0.45 \cdot \left(\frac{hc \cdot bc}{Ach} - 1 \right) \cdot \frac{f'_c}{f_{yt}} = 0.004$$

$$b := 0.09 \cdot \frac{f'_c}{f_{yt}} = 0.008$$

$$e := 0.12 \cdot \frac{f'_c}{f_{yt}} = 0.01$$

$$c := 0.2 \cdot kf \cdot kn \cdot \frac{\max(Pu_{btm}, Pu_{top})}{f_{yt} \cdot Ach} = 4.522 \cdot 10^{-4}$$

$$f := 0.35 \cdot kf \cdot \frac{\max(Pu_{top}, Pu_{btm})}{f_{yt} \cdot Ach} = 6.33 \cdot 10^{-4}$$

$$Ash1 := \left\{ \begin{array}{l} \text{if } \max(Pu_{bttm}, Pu_{top}) \leq 0.3 \cdot hc \cdot bc \cdot f'c \wedge f'c \leq 10000 \text{ psi} \\ \quad \parallel \text{return } \max(a, b) \cdot So \cdot hc \\ \text{else if } \max(Pu_{bttm}, Pu_{top}) > 0.3 \cdot hc \cdot bc \cdot f'c \vee f'c > 10000 \text{ psi} \\ \quad \parallel \text{return } \max(a, b, c) \cdot So \cdot hc \end{array} \right\} = 1.14 \text{ in}^2$$

$$Width := hc = 38 \text{ in}$$

$$MinNoLegsAsh1 := \text{ceil} \left(\frac{Ash1}{aTb} \right) = 4$$

Min number of required legs along "hc"

$$Ash1 := MinNoLegsAsh1 \cdot aTb = 1.24 \text{ in}^2$$

$$Ash2 := \left\{ \begin{array}{l} \text{if } \max(Pu_{bttm}, Pu_{top}) \leq 0.3 \cdot hc \cdot bc \cdot f'c \wedge f'c \leq 10000 \text{ psi} \\ \quad \parallel \text{return } \max(a, b) \cdot So \cdot bc \\ \text{else if } \max(Pu_{bttm}, Pu_{top}) > 0.3 \cdot hc \cdot bc \cdot f'c \vee f'c > 10000 \text{ psi} \\ \quad \parallel \text{return } \max(a, b, c) \cdot So \cdot bc \end{array} \right\} = 0.66 \text{ in}^2$$

$$Depth := bc = 22 \text{ in}$$

$$MinNoLegsAsh2 := \text{ceil} \left(\frac{Ash2}{aTb} \right) = 3$$

Min number of required legs along "bc"

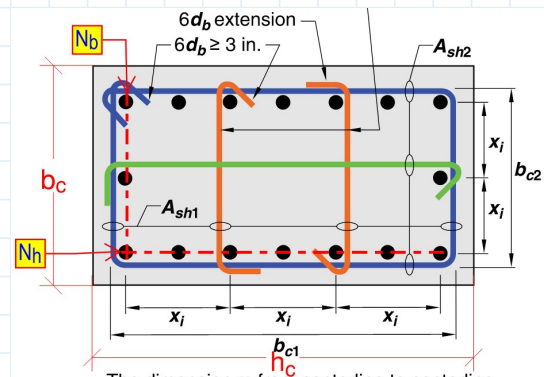
$$Ash2 := MinNoLegsAsh2 \cdot aTb = 0.93 \text{ in}^2$$

ACI 18.7.6.1 Design Shear Reinforcement Beyond Length L_o $V_c=0$

$$Vpr := \frac{MprT + MprB}{Ln} = 136.164 \text{ kip}$$

$$Sreq := \frac{Ash2 \cdot 0.75 \cdot fyt \cdot de}{Vpr + Vu} = 7.954 \text{ in}$$

$$S := \min \left(Sreq, \left\{ \begin{array}{l} \text{if } f_y = 60000 \text{ psi} \\ \quad \parallel \text{return } 6 \cdot dLb \\ \text{else if } f_y = 80000 \text{ psi} \\ \quad \parallel \text{return } 5 \cdot dLb \\ \text{else} \\ \quad \parallel \text{return "Error"} \end{array} \right\} \right) = 7.5 \text{ in}$$



$$S := 6 \text{ in}$$

Spacing of Transverse Reinforcement beyond length L_o

ACI 18.8.4.3 Nominal Joint Shear Strength

$$A_j := h_c \cdot b_c = 836 \text{ in}^2$$

$$V_e := \frac{\max(M_n T, M_n B)}{\min(D_b T, D_b B)} = 397.457 \text{ kip}$$

$$V_n := 8 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot \text{psi} \cdot A_j = 472.913 \text{ kip}$$

$$\text{Shear} := \begin{cases} \text{if } V_e \leq V_n \\ \text{return "O.K"} \\ \text{else} \\ \text{return "N.G"} \end{cases} = \text{"O.K"}$$

Table 18.8.4.3—Nominal joint shear strength V_n

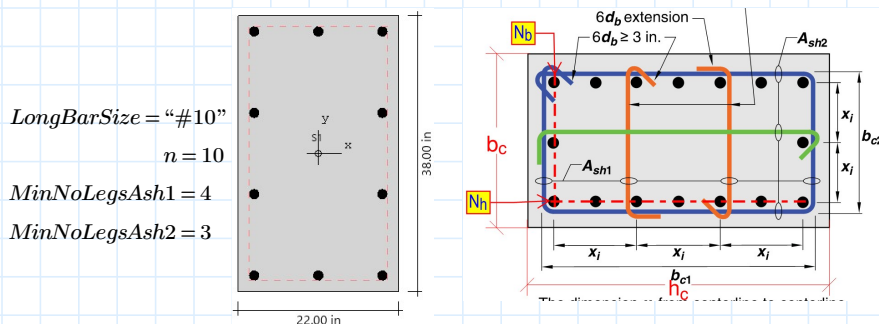
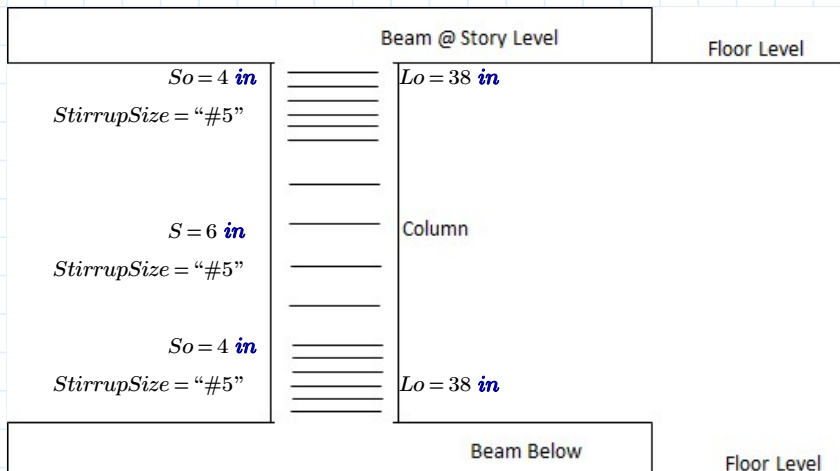
Column	Beam in direction of V_n	Confinement by transverse beams according to 15.2.8	V_n , lb ⁽¹⁾
Continuous or meets 15.2.6	Continuous or meets 15.2.7	Confined	$20\lambda\sqrt{f'_c}A_j$
		Not confined	$15\lambda\sqrt{f'_c}A_j$
	Other	Confined	$15\lambda\sqrt{f'_c}A_j$
		Not confined	$12\lambda\sqrt{f'_c}A_j$
Other	Continuous or meets 15.2.7	Confined	$15\lambda\sqrt{f'_c}A_j$
		Not confined	$12\lambda\sqrt{f'_c}A_j$
	Other	Confined	$12\lambda\sqrt{f'_c}A_j$
		Not confined	$8\lambda\sqrt{f'_c}A_j$

⁽¹⁾ λ shall be 0.75 for lightweight concrete and 1.0 for normalweight concrete. A_j shall be calculated in accordance with 15.4.2.4.

15.4.2.4 Effective cross-sectional area within a joint, A_j , shall be calculated as the product of joint depth and effective joint width. Joint depth shall be the overall depth of the column, h , in the direction of joint shear considered. Effective joint width shall be the overall width of the column where the beam is wider than the column. Where the column is wider than the beam, effective joint width shall not exceed the lesser of (a) and (b):

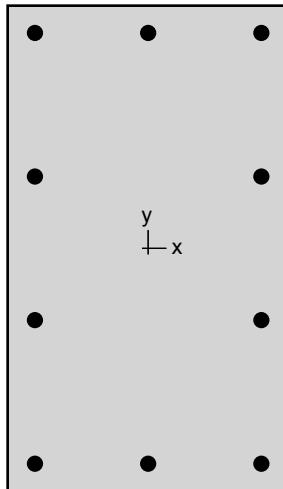
- (a) Beam width plus joint depth
- (b) Twice the perpendicular distance from longitudinal axis of beam to nearest side face of the column

Column Design Summary





spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

- 1. General Information3
- 2. Material Properties.....3
 - 2.1. Concrete3
 - 2.2. Steel3
- 3. Section.....3
 - 3.1. Shape and Properties.....3
 - 3.2. Section Figure4
 - 3.3. Solids.....4
 - 3.3.1. S1.....4
- 4. Reinforcement4
 - 4.1. Bar Set: ASTM A6154
 - 4.2. Confinement and Factors4
 - 4.3. Arrangement.....5
 - 4.4. Bars Provided5
- 5. Control Points5
- 6. Factored Loads and Moments with Corresponding Capacity Ratios.....5
- 7. Diagrams6
 - 7.1. PM at $\theta=0$ [deg]6

List of Figures

- Figure 1: Column section.....4

1. General Information

File Name	X:\25534-Fire Station ...\SPCOL Mn - Middle.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	836 in ²
I_x	100599 in ⁴
I_y	33718.7 in ⁴
r_x	10.9697 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

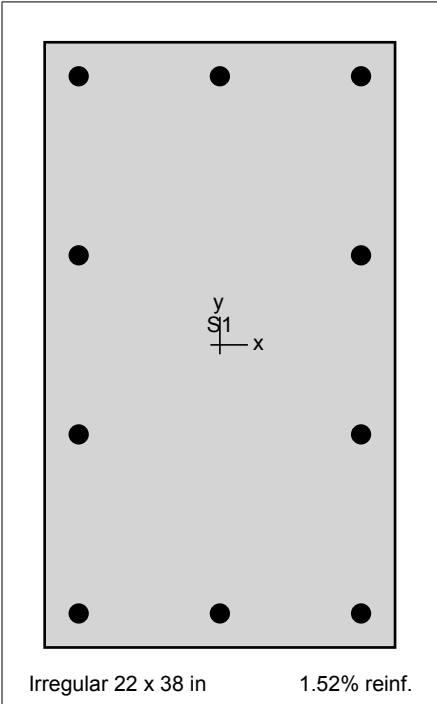


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-19.0	2	11.0	-19.0	3	11.0	19.0
4	-11.0	19.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	12.70 in ²
Rho	1.52 %
Minimum clear spacing	7.59 in

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
1.27	-8.9	16.9	1.27	0.0	16.9	1.27	8.9	16.9
1.27	-8.9	-16.9	1.27	0.0	-16.9	1.27	8.9	-16.9
1.27	-8.9	-5.6	1.27	-8.9	5.6	1.27	8.9	-5.6
1.27	8.9	5.6						

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d_t Depth in	ϵ_t	ϕ
X @ Max compression	4261.0	0.00	0.00	115.56	35.86	-0.00207	1.00000
X @ Allowable comp.	4261.0	0.00	0.00	115.56	35.86	-0.00207	1.00000
X @ $f_s = 0.0$	3081.3	1371.45	0.00	35.86	35.86	0.00000	1.00000
X @ $f_s = 0.5 f_y$	2209.2	1882.88	0.00	26.67	35.86	0.00103	1.00000
X @ Balanced point	1607.0	2060.05	0.00	21.23	35.86	0.00207	1.00000
X @ Tension control	828.1	1826.71	0.00	13.33	35.86	0.00507	1.00000
X @ Pure bending	0.0	1076.67	0.00	4.86	35.86	0.01913	1.00000
X @ Max tension	-762.0	0.00	0.00	0.00	35.86	9.99999	1.00000
-X @ Max compression	4261.0	0.00	0.00	115.56	35.86	-0.00207	1.00000
-X @ Allowable comp.	4261.0	0.00	0.00	115.56	35.86	-0.00207	1.00000
-X @ $f_s = 0.0$	3081.3	-1371.45	0.00	35.86	35.86	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	2209.2	-1882.88	0.00	26.67	35.86	0.00103	1.00000
-X @ Balanced point	1607.0	-2060.05	0.00	21.23	35.86	0.00207	1.00000
-X @ Tension control	828.1	-1826.71	0.00	13.33	35.86	0.00507	1.00000
-X @ Pure bending	0.0	-1076.67	0.00	4.86	35.86	0.01913	1.00000
-X @ Max tension	-762.0	0.00	0.00	0.00	35.86	9.99999	1.00000

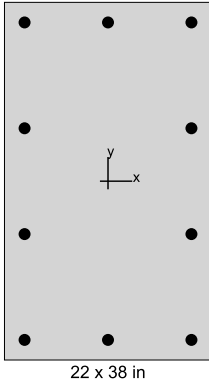
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand P_u kip	M_{ux} k-ft	Capacity ϕP_n kip	ϕM_{nx} k-ft	Parameters at Capacity NA Depth in	ϵ_t	ϕ	Capacity Ratio
1	81.20	-223.30	81.20	-1180.31	5.67	0.01597	1.000	0.19
2	37.40	223.30	37.40	1124.94	5.23	0.01758	1.000	0.20

7. Diagrams

7.1. PM at $\theta=0$ [deg]



General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

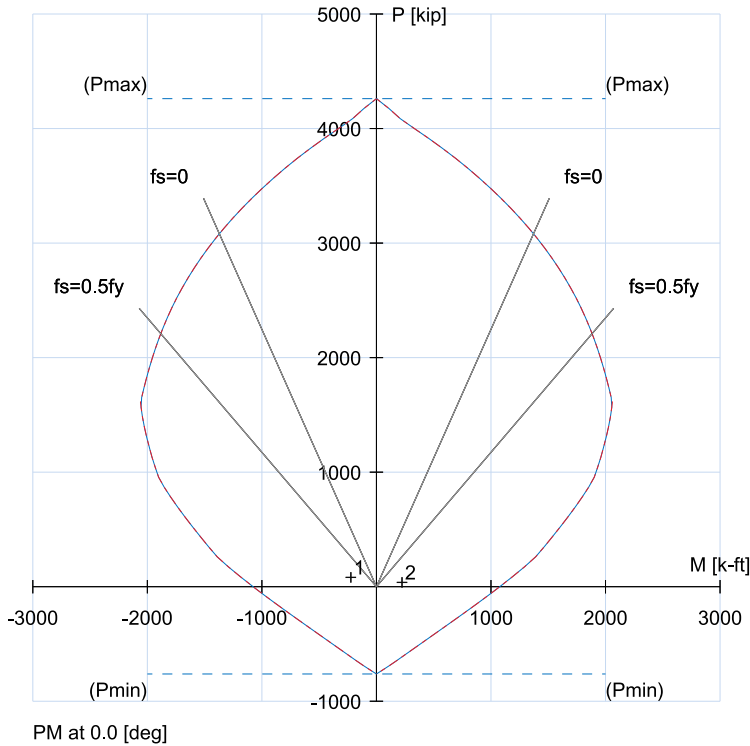
f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	836 in ²
I_x	100599 in ⁴
I_y	33718.7 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Other
Total steel area, A_s	12.70 in ²
Rho	1.52 %
Min. clear spacing	7.59 in

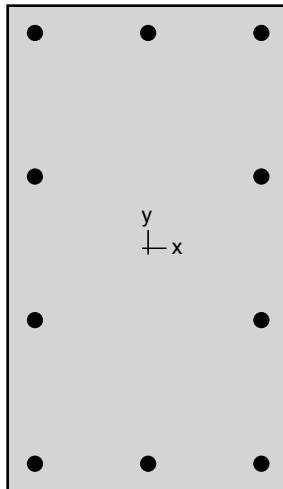


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	37.4	223.3	37.40	1124.94	0.20
1	81.2	-223.3	81.20	-1180.31	0.19

Max. Capacity Ratio: 0.20



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Station ...\SPCOL Mpr - Middle.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	75 ksi
E_s	29000 ksi
ϵ_{ty}	0.00258621 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	836 in ²
I_x	100599 in ⁴
I_y	33718.7 in ⁴
r_x	10.9697 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

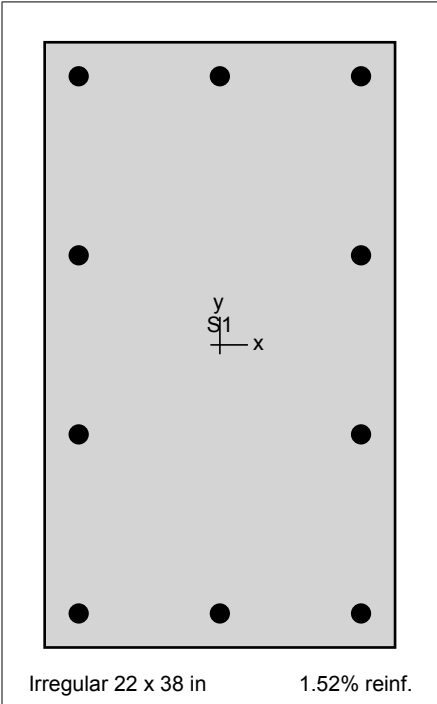


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-19.0	2	11.0	-19.0	3	11.0	19.0
4	-11.0	19.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	12.70 in ²
Rho	1.52 %
Minimum clear spacing	7.59 in

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
1.27	-8.9	16.9	1.27	0.0	16.9	1.27	8.9	16.9
1.27	-8.9	-16.9	1.27	0.0	-16.9	1.27	8.9	-16.9
1.27	-8.9	-5.6	1.27	-8.9	5.6	1.27	8.9	-5.6
1.27	8.9	5.6						

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	4451.5	0.00	0.00	260.02	35.86	-0.00259	1.00000
X @ Allowable comp.	4451.5	0.00	0.00	260.02	35.86	-0.00259	1.00000
X @ $f_s = 0.0$	3138.4	1451.76	0.00	35.86	35.86	0.00000	1.00000
X @ $f_s = 0.5 f_y$	2097.4	2023.10	0.00	25.06	35.86	0.00129	1.00000
X @ Balanced point	1419.7	2191.91	0.00	19.26	35.86	0.00259	1.00000
X @ Tension control	704.9	1940.05	0.00	12.53	35.86	0.00559	1.00000
X @ Pure bending	0.0	1324.93	0.00	6.22	35.86	0.01430	1.00000
X @ Max tension	-952.5	0.00	0.00	0.00	35.86	9.99999	1.00000
-X @ Max compression	4451.5	0.00	0.00	260.02	35.86	-0.00259	1.00000
-X @ Allowable comp.	4451.5	0.00	0.00	260.02	35.86	-0.00259	1.00000
-X @ $f_s = 0.0$	3138.4	-1451.76	0.00	35.86	35.86	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	2097.4	-2023.10	0.00	25.06	35.86	0.00129	1.00000
-X @ Balanced point	1419.7	-2191.91	0.00	19.26	35.86	0.00259	1.00000
-X @ Tension control	704.9	-1940.05	0.00	12.53	35.86	0.00559	1.00000
-X @ Pure bending	0.0	-1324.93	0.00	6.22	35.86	0.01430	1.00000
-X @ Max tension	-952.5	0.00	0.00	0.00	35.86	9.99999	1.00000

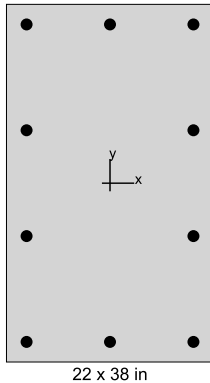
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand P_u kip	M_{ux} k-ft	Capacity ϕP_n kip	ϕM_{nx} k-ft	Parameters at Capacity NA Depth in	ϵ_t	ϕ	Capacity Ratio
1	81.20	-223.30	81.20	-1421.20	7.11	0.01212	1.000	0.16
2	37.40	223.30	37.40	1369.85	6.63	0.01324	1.000	0.16

7. Diagrams

7.1. PM at $\theta=0$ [deg]



22 x 38 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

f'_c	5 ksi
E_c	4030.51 ksi
f_y	75 ksi
E_s	29000 ksi

Section

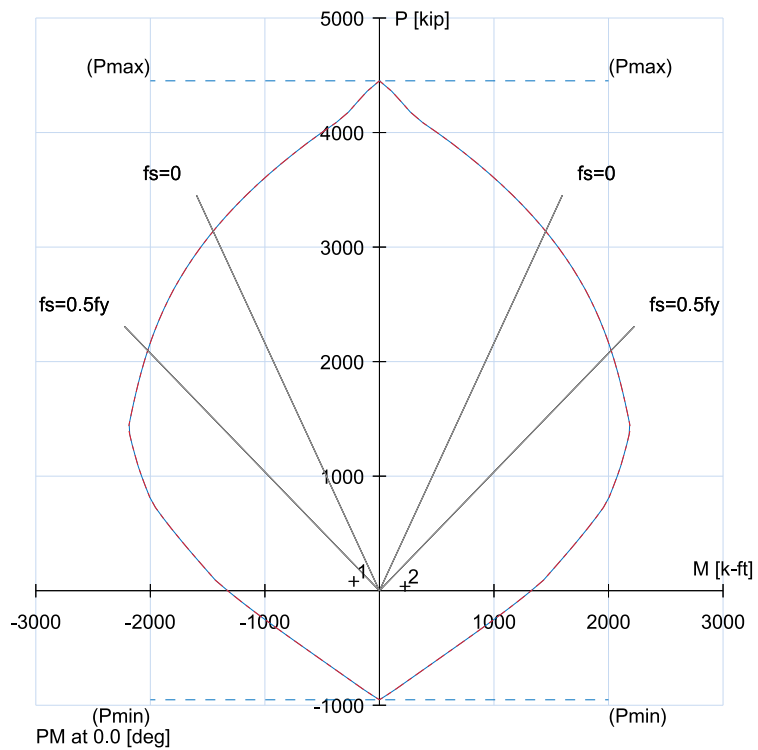
Type	Irregular
A_g	836 in ²
I_x	100599 in ⁴
I_y	33718.7 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---

Confinement type	Other
------------------	-------

Total steel area, A_s	12.70 in ²
Rho	1.52 %
Min. clear spacing	7.59 in

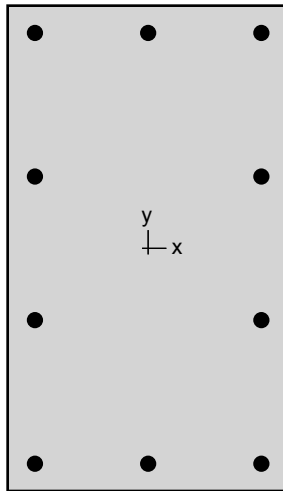


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
1	81.2	-223.3	81.20	-1421.20	0.16
2	37.4	223.3	37.40	1369.85	0.16

Max. Capacity Ratio: 0.16



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Statio...\SPCOL phiMn - Middle.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	836 in ²
I_x	100599 in ⁴
I_y	33718.7 in ⁴
r_x	10.9697 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

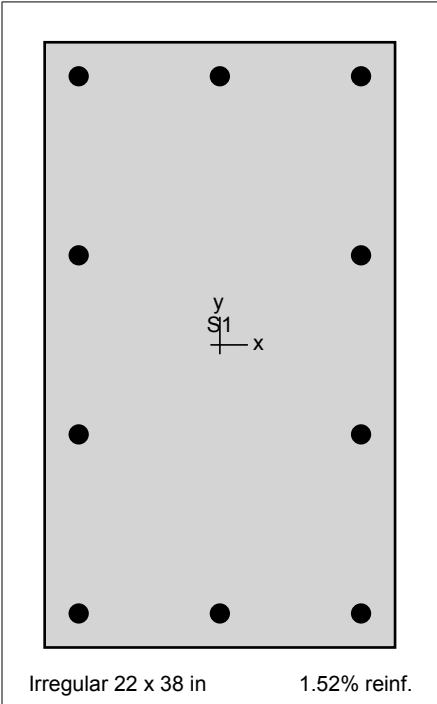


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-19.0	2	11.0	-19.0	3	11.0	19.0
4	-11.0	19.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ , (b)	0.9
Compression controlled ϕ , (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	12.70 in ²
Rho	1.52 %
Minimum clear spacing	7.59 in

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
1.27	-8.9	16.9	1.27	0.0	16.9	1.27	8.9	16.9
1.27	-8.9	-16.9	1.27	0.0	-16.9	1.27	8.9	-16.9
1.27	-8.9	-5.6	1.27	-8.9	5.6	1.27	8.9	-5.6
1.27	8.9	5.6						

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	2769.7	0.00	0.00	115.56	35.86	-0.00207	0.65000
X @ Allowable comp.	2215.7	695.05	0.00	39.54	35.86	-0.00028	0.65000
X @ $f_s = 0.0$	2002.8	891.44	0.00	35.86	35.86	0.00000	0.65000
X @ $f_s = 0.5 f_y$	1436.0	1223.88	0.00	26.67	35.86	0.00103	0.65000
X @ Balanced point	1044.6	1339.03	0.00	21.23	35.86	0.00207	0.65000
X @ Tension control	745.3	1644.04	0.00	13.33	35.86	0.00507	0.90000
X @ Pure bending	0.0	969.00	0.00	4.86	35.86	0.01913	0.90000
X @ Max tension	-685.8	0.00	0.00	0.00	35.86	9.99999	0.90000
-X @ Max compression	2769.7	0.00	0.00	115.56	35.86	-0.00207	0.65000
-X @ Allowable comp.	2215.7	-695.05	0.00	39.54	35.86	-0.00028	0.65000
-X @ $f_s = 0.0$	2002.8	-891.44	0.00	35.86	35.86	0.00000	0.65000
-X @ $f_s = 0.5 f_y$	1436.0	-1223.88	0.00	26.67	35.86	0.00103	0.65000
-X @ Balanced point	1044.6	-1339.03	0.00	21.23	35.86	0.00207	0.65000
-X @ Tension control	745.3	-1644.04	0.00	13.33	35.86	0.00507	0.90000
-X @ Pure bending	0.0	-969.00	0.00	4.86	35.86	0.01913	0.90000
-X @ Max tension	-685.8	0.00	0.00	0.00	35.86	9.99999	0.90000

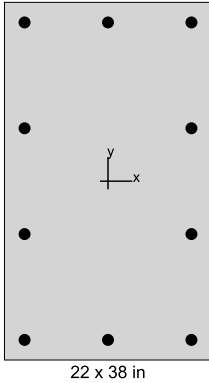
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	81.20	-223.30	81.20	-1072.47	5.76	0.01566	0.900	0.21
2	37.40	223.30	37.40	1017.20	5.27	0.01742	0.900	0.22

7. Diagrams

7.1. PM at $\theta=0$ [deg]



General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

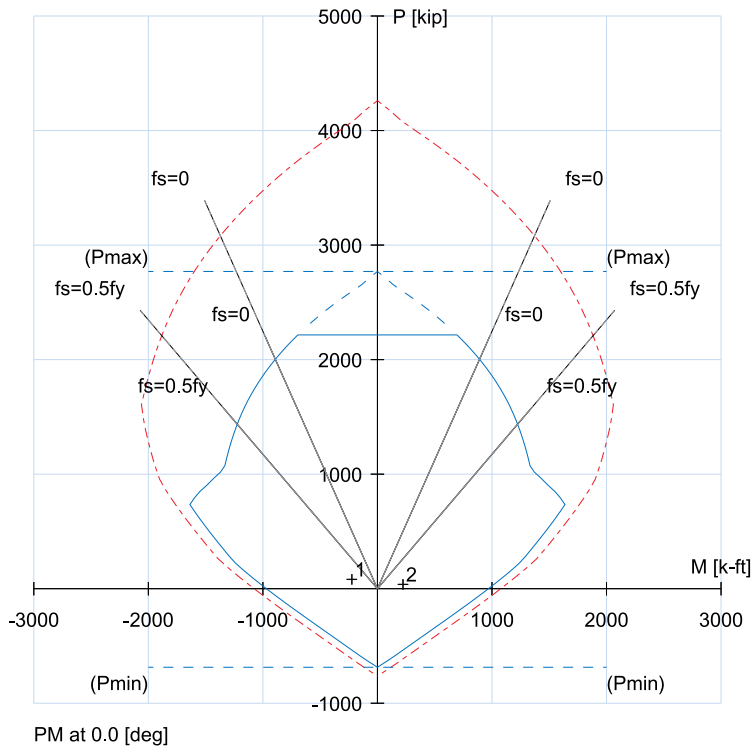
f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	836 in ²
I_x	100599 in ⁴
I_y	33718.7 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	12.70 in ²
Rho	1.52 %
Min. clear spacing	7.59 in



No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	37.4	223.3	37.40	1017.20	0.22
1	81.2	-223.3	81.20	-1072.47	0.21

Max. Capacity Ratio: 0.22

Column Size Parameters

$$f'_c := 5000 \text{ psi}$$

$$f_y := 60000 \text{ psi}$$

$$f_{yt} := 60000 \text{ psi}$$

$$L_n := 14 \text{ ft}$$

Clear Span L_n

$$H := 17.833$$

Story height, Center to Center

$$D_{bT} := 36 \text{ in}$$

Depth of connected beam @TOP

$$W_{bT} := 22 \text{ in}$$

Width of connected beam@TOP

$$D_{bB} := 28 \text{ in}$$

Depth of connected beam @BTM

$$W_{bB} := 26 \text{ in}$$

Width of connected beam@BTM

$$h_c := 33 \text{ in}$$

Col depth

$$b_c := 22 \text{ in}$$

Col width

$$LongBarSize := \text{"\#9"}$$

Longitudinal bar size

$$StirrupSize := \text{"\#5"}$$

Hoop size

$$CC := 1.5 \text{ in}$$

Top Clear Cover

$$N_b := 3$$

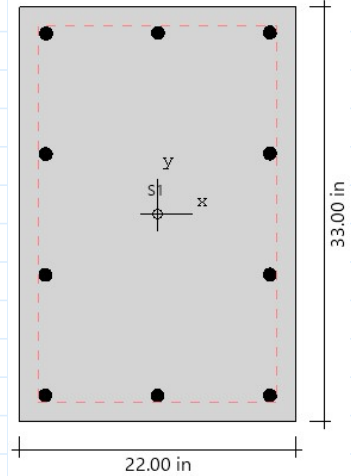
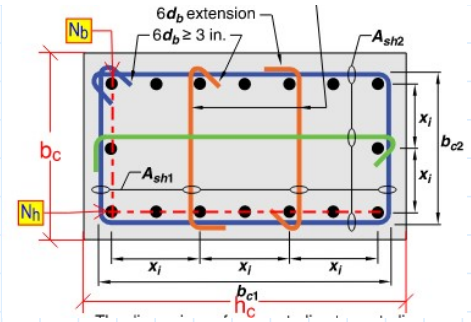
Number of longitudinal bars along side b

$$N_h := 4$$

Number of longitudinal bars along side h

$$n := 2 \cdot (N_b + N_h) - 4 = 10$$

Total Number of longitudinal bars



Column Loads

$$M_u := 142.3 \text{ kip} \cdot \text{ft} \quad \text{Ultimate Moment From Analysis}$$

$$P_{u_{top}} := 42.66 \text{ kip} \quad \text{Ultimate Axial force @TOP From Analysis}$$

$$P_{u_{btm}} := 85 \cdot \text{kip} \quad \text{Ultimate Axial force @BTM From Analysis}$$

$$V_u := 16.4 \text{ kip} \quad \text{Ultimate Shear force From Analysis}$$

Column Capacity

$$M_{n_{top}} := 709.5 \text{ kip} \cdot \text{ft} \quad \text{Column Mn positive From SPCol}$$

$$M_{n_{btm}} := 827 \text{ kip} \cdot \text{ft} \quad \text{Column Mn negative From SPCol}$$

$$M_{pr_{top}} := 880.5 \text{ kip} \cdot \text{ft} \quad \text{Column Mpr positive From SPCol}$$

$$M_{pr_{btm}} := 991 \text{ kip} \cdot \text{ft} \quad \text{Column Mpr negative From SPCol}$$

Beam Capacity TOP

$$M_{nTR} := 0 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mn From SPCol}$$

$$M_{nTL} := 478.5 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mn From SPCol}$$

$$M_{nT} := M_{nTR} + M_{nTL} = 478.5 \text{ kip} \cdot \text{ft}$$

$$M_{prTR} := 0 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mpr From SPCol}$$

$$M_{prTL} := 581.8 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mpr From SPCol}$$

$$M_{prT} := M_{prTR} + M_{prTL} = 581.8 \text{ kip} \cdot \text{ft}$$

Beam Capacity BTM

$$M_{nBR} := 0 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mn From SPCol}$$

$$M_{nBL} := 314.8 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mn From SPCol}$$

$$M_{nB} := M_{nBR} + M_{nBL} = 314.8 \text{ kip} \cdot \text{ft}$$

$$M_{prBR} := 0 \text{ kip} \cdot \text{ft} \quad \text{Max Right Beam Mpr From SPCol}$$

$$M_{prBL} := 386.5 \text{ kip} \cdot \text{ft} \quad \text{Max Left Beam Mpr From SPCol}$$

$$M_{prB} := M_{prBR} + M_{prBL} = 386.5 \text{ kip} \cdot \text{ft}$$

$Ld\#9 := 37 \text{ in}$	Seismic Ld for #9		
$Ld\#10 := 42 \text{ in}$	Seismic Ld for #10	Diameter of Bar	Area of Bar
$Ld\#11 := 47 \text{ in}$	Seismic Ld for #11	$db3 := 0.375 \text{ in}$	$ab3 := 0.11 \text{ in}^2$
		$db4 := 0.5 \text{ in}$	$ab4 := 0.2 \text{ in}^2$
		$db5 := 0.625 \text{ in}$	$ab5 := 0.31 \text{ in}^2$
		$db6 := 0.75 \text{ in}$	$ab6 := 0.44 \text{ in}^2$
		$db7 := 0.875 \text{ in}$	$ab7 := 0.6 \text{ in}^2$
		$db8 := 1 \text{ in}$	$ab8 := 0.79 \text{ in}^2$
		$db9 := 1.125 \text{ in}$	$ab9 := 1 \text{ in}^2$
		$db10 := 1.25 \text{ in}$	$ab10 := 1.27 \text{ in}^2$
		$db11 := 1.41 \text{ in}$	$ab11 := 1.56 \text{ in}^2$
$dLb :=$	if $LongBarSize = \text{"#3"}$ return $Ld\#9$ else if $LongBarSize = \text{"#10"}$ return $Ld\#10$ else if $LongBarSize = \text{"#11"}$ return $Ld\#11$	$= 37 \text{ in}$	
		Dia. of one Long. Bar	
$dLb :=$	if $LongBarSize = \text{"#3"}$ return $db3$ else if $LongBarSize = \text{"#4"}$ return $db4$ else if $LongBarSize = \text{"#5"}$ return $db5$ else if $LongBarSize = \text{"#6"}$ return $db6$ else if $LongBarSize = \text{"#7"}$ return $db7$ else if $LongBarSize = \text{"#8"}$ return $db8$ else if $LongBarSize = \text{"#9"}$ return $db9$ else if $LongBarSize = \text{"#10"}$ return $db10$ else if $LongBarSize = \text{"#11"}$ return $db11$	$= 1.125 \text{ in}$	
		Dia. of one Long. Bar	
$aLb :=$	if $LongBarSize = \text{"#3"}$ return $ab3$ else if $LongBarSize = \text{"#4"}$ return $ab4$ else if $LongBarSize = \text{"#5"}$ return $ab5$ else if $LongBarSize = \text{"#6"}$ return $ab6$ else if $LongBarSize = \text{"#7"}$ return $ab7$ else if $LongBarSize = \text{"#8"}$ return $ab8$ else if $LongBarSize = \text{"#9"}$ return $ab9$ else if $LongBarSize = \text{"#10"}$ return $ab10$ else if $LongBarSize = \text{"#11"}$ return $ab11$	$= 1 \text{ in}^2$	
		Area of one Long. Bar	
$dTb :=$	if $StirrupSize = \text{"#3"}$ return $db3$ else if $StirrupSize = \text{"#4"}$ return $db4$ else if $StirrupSize = \text{"#5"}$ return $db5$ else if $StirrupSize = \text{"#6"}$ return $db6$ else if $StirrupSize = \text{"#7"}$ return $db7$ else if $StirrupSize = \text{"#8"}$ return $db8$ else if $StirrupSize = \text{"#9"}$ return $db9$ else if $StirrupSize = \text{"#10"}$ return $db10$	$= 0.625 \text{ in}$	
		Dia. of one Hoop	
$aTb :=$	if $StirrupSize = \text{"#3"}$ return $ab3$ else if $StirrupSize = \text{"#4"}$ return $ab4$ else if $StirrupSize = \text{"#5"}$ return $ab5$ else if $StirrupSize = \text{"#6"}$ return $ab6$ else if $StirrupSize = \text{"#7"}$ return $ab7$ else if $StirrupSize = \text{"#8"}$ return $ab8$ else if $StirrupSize = \text{"#9"}$ return $ab9$ else if $StirrupSize = \text{"#10"}$ return $ab10$	$= 0.31 \text{ in}^2$	
		Area of one Hoop	

$$A_{st} := n \cdot aLb = 10 \text{ in}^2$$

$$de := hc - CC - dTb - 0.5 \cdot dLb = 30.313 \text{ in}$$

Effective depth

ACI 18.7.2 Column Dimensional limits

$$ColDimCheck := \begin{cases} \text{if } \min(hc, bc) \geq 12 \cdot \text{in} \wedge \frac{\min(hc, bc)}{\max(hc, bc)} \geq 0.4 \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Redesign column Dimensions"} \end{cases} = \text{"O.K"}$$

ACI 18.7.3.2 Strong Column Weak Beam Criteria

$$\min(Mn_{bttm}, Mn_{top}) = 709.5 \text{ kip} \cdot \text{ft}$$

$$\frac{6}{5} \cdot \max(MnT, MnB) = 574.2 \text{ kip} \cdot \text{ft}$$

$$MinAstCheck := \begin{cases} \text{if } \min(Mn_{bttm}, Mn_{top}) \geq \frac{6}{5} \cdot \max(MnT, MnB) \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Redesign Column"} \end{cases} = \text{"O.K"}$$

ACI 18.7.4.1 Min &Max Longitudinal Bar Area

18.7.4 Longitudinal reinforcement

18.7.4.1 Area of longitudinal reinforcement, A_{st} , shall be at least $0.01A_g$ and shall not exceed $0.06A_g$.

$$MinAstCheck := \begin{cases} \text{if } A_{st} \geq 0.01 \cdot hc \cdot bc \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Increase Longitudinal bars"} \end{cases} = \text{"O.K"}$$

$$MaxAstCheck := \begin{cases} \text{if } A_{st} \leq 0.06 \cdot hc \cdot bc \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Reduce Longitudinal bars"} \end{cases} = \text{"O.K"}$$

ACI 18.7.4.3 Development Length

18.7.4.3 Over column clear height, longitudinal reinforcement shall be selected such that $1.25l_d \leq \ell_d/2$.

$$MaxAstDevLength := \begin{cases} \text{if } 1.25 \cdot Ld \leq \frac{Ln}{2} \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Reduce Longitudinal bars size"} \end{cases} = \text{"O.K"}$$

ACI 18.7.5.1 Extend of Required Transverse Reinforcements

18.7.5.1 Transverse reinforcement required in 18.7.5.2 through 18.7.5.4 shall be provided over a length ℓ_e from each joint face and on both sides of any section where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior. Length ℓ_e shall be at least the greatest of (a) through (c):

$$Lo := \max\left(hc, \frac{Ln}{16}, 18 \text{ in}\right) = 33 \text{ in}$$

- (a) The depth of the column at the joint face or at the section where flexural yielding is likely to occur
- (b) One-sixth of the clear span of the column
- (c) 18 in.

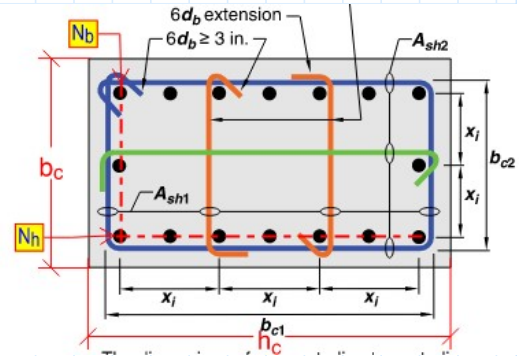
ACI 18.7.5.2 Max Laterally Supported Longitudinal Bar Spacing

$$X_{ih} := \frac{\left(hc - 2 \cdot CC - 2 \cdot dTb - 2 \cdot \frac{dLb}{2} \right)}{N_h - 1} = 9.208 \text{ in}$$

$$X_{ib} := \frac{\left(bc - 2 \cdot CC - 2 \cdot dTb - 2 \cdot \frac{dLb}{2} \right)}{N_b - 1} = 8.313 \text{ in}$$

$$X_i := \max(X_{ih}, X_{ib}) = 9.208 \text{ in}$$

$$MaxAstCheck := \begin{cases} \text{if } X_i \leq 14 \text{ in} \\ \quad \text{return "O.K"} \\ \text{else} \\ \quad \text{return "Reduce Longitudinal bars spacing"} \end{cases} = \text{"O.K"}$$



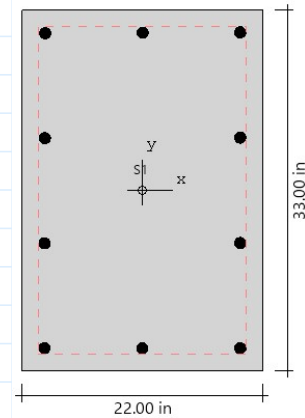
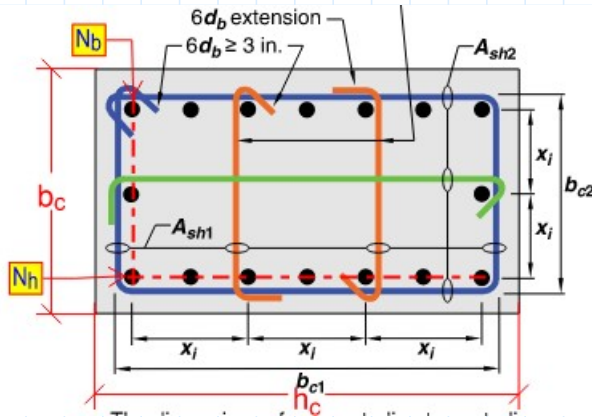
ACI 18.7.5.3 Max Spacing of Transverse Reinforcements Within Lo

$$hx := X_i = 9.208 \text{ in} \quad (\text{max C/C spacing of long. bars laterally supported by corners of stirrups})$$

$$So := \min \left(\min \left(\frac{1}{4} \cdot hc, \frac{1}{4} \cdot bc \right), \begin{cases} \text{if } fy = 60000 \text{ psi} \\ \quad \text{return } 6 \cdot dLb \\ \text{else if } fy = 80000 \text{ psi} \\ \quad \text{return } 5 \cdot dLb \\ \text{else} \\ \quad \text{return "Error"} \end{cases}, 4 + \frac{(14 + hx)}{3} \right) = 6.75 \text{ in}$$

$$So := 4 \text{ in} \quad \text{Spacing of Transverse Reinforcement within length } Lo$$

ACI 18.7.5.4 Amount of Transverse Reinforcements



$$n1 := 10$$

(# of bars supported by corners of rectilinear hoops)

$$Ach := (hc - CC) \cdot (bc - CC) = 645.75 \text{ in}^2$$

$$kf := \begin{cases} \text{if } \frac{f'_c}{25000 \text{ psi}} + 0.6 \geq 1.0 \\ \frac{f'_c}{25000 \text{ psi}} + 0.6 \\ \text{else} \\ 1.0 \end{cases} = 1$$

$$kn := \frac{n1}{n1 - 2} = 1.25$$

Table 18.7.5.4—Transverse reinforcement for columns of special moment frames

Transverse reinforcement	Conditions	Applicable expressions
A_{sh}/sb_c for rectilinear hoop	$P_n \leq 0.3A_g f'_c$ and $f'_c \leq 10,000$ psi	Greater of (a) and (b) $0.3 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (a)
	$P_n > 0.3A_g f'_c$ or $f'_c > 10,000$ psi	Greater of (a), (b), and (c) $0.09 \frac{f'_c}{f_{yt}}$ (b) $0.2k_f k_a \frac{P_n}{f_{yt} A_{ch}}$ (c)
ρ , for spiral or circular hoop	$P_n \leq 0.3A_g f'_c$ and $f'_c \leq 10,000$ psi	Greater of (d) and (e) $0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (d)
	$P_n > 0.3A_g f'_c$ or $f'_c > 10,000$ psi	Greater of (d), (e), and (f) $0.12 \frac{f'_c}{f_{yt}}$ (e) $0.35k_f \frac{P_n}{f_{yt} A_{ch}}$ (f)

$$a := 0.3 \cdot \left(\frac{hc \cdot bc}{Ach} - 1 \right) \cdot \frac{f'_c}{f_{yt}} = 0.003$$

$$d := 0.45 \cdot \left(\frac{hc \cdot bc}{Ach} - 1 \right) \cdot \frac{f'_c}{f_{yt}} = 0.005$$

$$b := 0.09 \cdot \frac{f'_c}{f_{yt}} = 0.008$$

$$e := 0.12 \cdot \frac{f'_c}{f_{yt}} = 0.01$$

$$c := 0.2 \cdot kf \cdot kn \cdot \frac{\max(Pu_{btm}, Pu_{top})}{f_{yt} \cdot Ach} = 5.485 \cdot 10^{-4}$$

$$f := 0.35 \cdot kf \cdot \frac{\max(Pu_{top}, Pu_{btm})}{f_{yt} \cdot Ach} = 7.678 \cdot 10^{-4}$$

$$Ash1 := \left\| \begin{array}{l} \text{if } \max(Pu_{bttm}, Pu_{top}) \leq 0.3 \cdot hc \cdot bc \cdot f'c \wedge f'c \leq 10000 \text{ psi} \\ \quad \left\| \begin{array}{l} \text{return } \max(a, b) \cdot So \cdot hc \\ \text{else if } \max(Pu_{bttm}, Pu_{top}) > 0.3 \cdot hc \cdot bc \cdot f'c \vee f'c > 10000 \text{ psi} \\ \quad \left\| \text{return } \max(a, b, c) \cdot So \cdot hc \end{array} \right. \end{array} \right\| = 0.99 \text{ in}^2$$

$$Width := hc = 33 \text{ in}$$

$$MinNoLegsAsh1 := \text{ceil} \left(\frac{Ash1}{aTb} \right) = 4$$

Min number of required legs along "hc"

$$Ash1 := MinNoLegsAsh1 \cdot aTb = 1.24 \text{ in}^2$$

$$Ash2 := \left\| \begin{array}{l} \text{if } \max(Pu_{bttm}, Pu_{top}) \leq 0.3 \cdot hc \cdot bc \cdot f'c \wedge f'c \leq 10000 \text{ psi} \\ \quad \left\| \begin{array}{l} \text{return } \max(a, b) \cdot So \cdot bc \\ \text{else if } \max(Pu_{bttm}, Pu_{top}) > 0.3 \cdot hc \cdot bc \cdot f'c \vee f'c > 10000 \text{ psi} \\ \quad \left\| \text{return } \max(a, b, c) \cdot So \cdot bc \end{array} \right. \end{array} \right\| = 0.66 \text{ in}^2$$

$$Depth := bc = 22 \text{ in}$$

$$MinNoLegsAsh2 := \text{ceil} \left(\frac{Ash2}{aTb} \right) = 3$$

Min number of required legs along "bc"

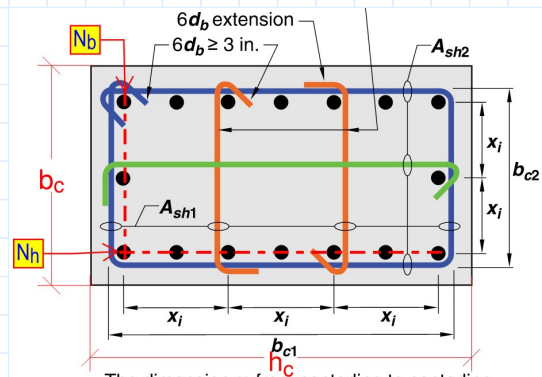
$$Ash2 := MinNoLegsAsh2 \cdot aTb = 0.93 \text{ in}^2$$

ACI 18.7.6.1 Design Shear Reinforcement Beyond Length L_o $V_c=0$

$$Vpr := \frac{MprT + MprB}{Ln} = 69.164 \text{ kip}$$

$$Sreq := \frac{Ash2 \cdot 0.75 \cdot fyt \cdot de}{Vpr + Vu} = 14.826 \text{ in}$$

$$S := \min \left(Sreq, \left\| \begin{array}{l} \text{if } fyt = 60000 \text{ psi} \\ \quad \left\| \begin{array}{l} \text{return } 6 \cdot dLb \\ \text{else if } fyt = 80000 \text{ psi} \\ \quad \left\| \text{return } 5 \cdot dLb \\ \text{else} \\ \quad \left\| \text{return "Error"} \end{array} \right. \end{array} \right. \right\| \right) = 6.75 \text{ in}$$



$$S := 6 \text{ in}$$

Spacing of Transverse Reinforcement beyond length L_o

ACI 18.8.4.3 Nominal Joint Shear Strength

$$A_j := h_c \cdot b_c = 726 \text{ in}^2$$

$$V_e := \frac{\max(M_n T, M_n B)}{\min(D_b T, D_b B)} = 205.071 \text{ kip}$$

$$V_n := 8 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot \text{psi} \cdot A_j = 410.688 \text{ kip}$$

$$\text{Shear} := \begin{cases} \text{if } V_e \leq V_n \\ \text{return "O.K"} \\ \text{else} \\ \text{return "N.G"} \end{cases} = \text{"O.K"}$$

Table 18.8.4.3—Nominal joint shear strength V_n

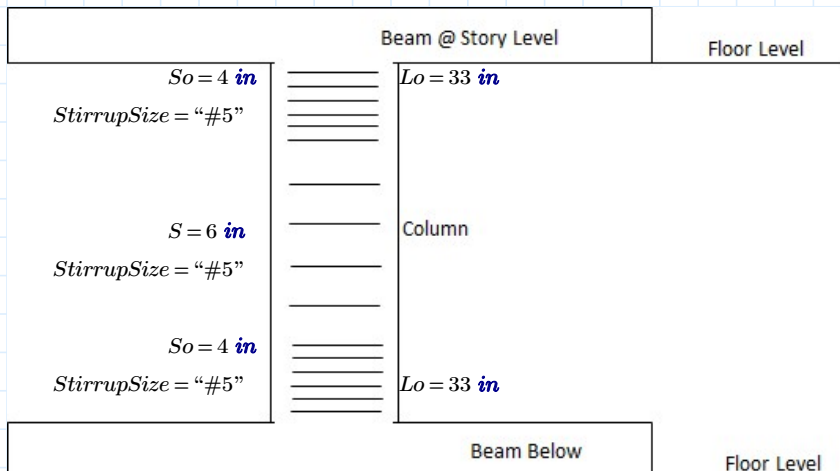
Column	Beam in direction of V_n	Confinement by transverse beams according to 15.2.8	V_n , lb ⁽¹⁾
Continuous or meets 15.2.6	Continuous or meets 15.2.7	Confined	$20\lambda\sqrt{f'_c}A_j$
		Not confined	$15\lambda\sqrt{f'_c}A_j$
	Other	Confined	$15\lambda\sqrt{f'_c}A_j$
		Not confined	$12\lambda\sqrt{f'_c}A_j$
Other	Continuous or meets 15.2.7	Confined	$15\lambda\sqrt{f'_c}A_j$
		Not confined	$12\lambda\sqrt{f'_c}A_j$
	Other	Confined	$12\lambda\sqrt{f'_c}A_j$
		Not confined	$8\lambda\sqrt{f'_c}A_j$

⁽¹⁾ λ shall be 0.75 for lightweight concrete and 1.0 for normalweight concrete. A_j shall be calculated in accordance with 15.4.2.4.

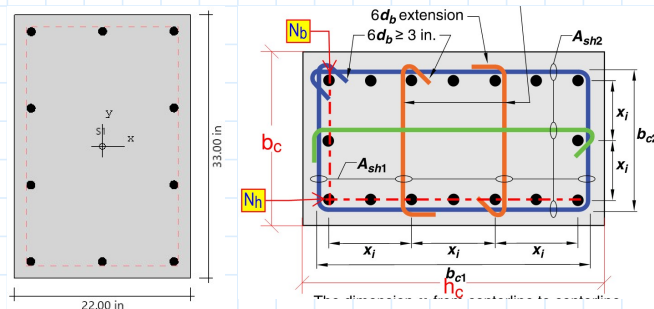
15.4.2.4 Effective cross-sectional area within a joint, A_j , shall be calculated as the product of joint depth and effective joint width. Joint depth shall be the overall depth of the column, h , in the direction of joint shear considered. Effective joint width shall be the overall width of the column where the beam is wider than the column. Where the column is wider than the beam, effective joint width shall not exceed the lesser of (a) and (b):

- (a) Beam width plus joint depth
- (b) Twice the perpendicular distance from longitudinal axis of beam to nearest side face of the column

Column Design Summary

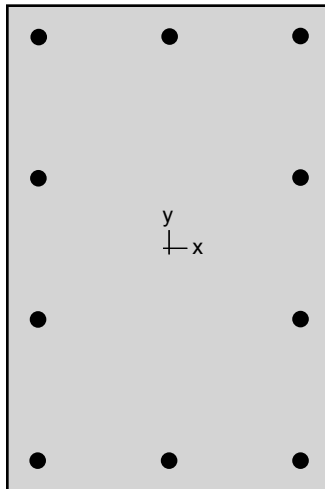


LongBarSize = "#9"
n = 10
MinNoLegsAsh1 = 4
MinNoLegsAsh2 = 3





spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Station...\SPCOL Mn - Col Left.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴
r_x	9.52628 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

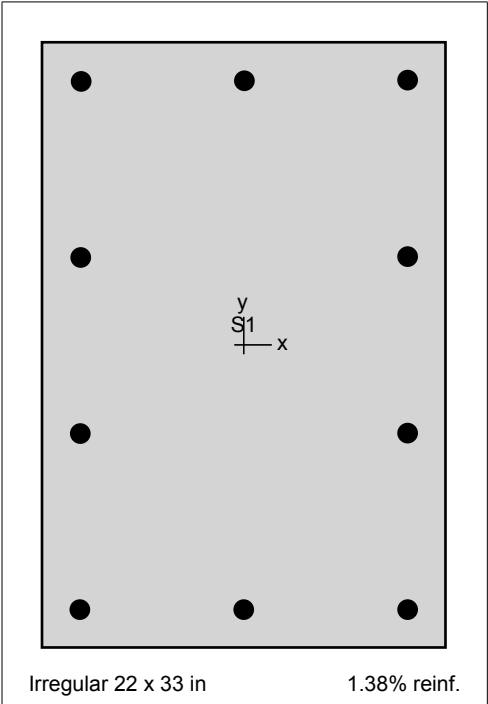


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-16.5	2	11.0	-16.5	3	11.0	16.5
4	-11.0	16.5						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Minimum clear spacing	7.77 in

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
1.00	-8.9	14.4	1.00	0.0	14.4	1.00	8.9	14.4
1.00	-8.9	-14.4	1.00	0.0	-14.4	1.00	8.9	-14.4
1.00	-8.9	-4.8	1.00	-8.9	4.8	1.00	8.9	-4.8
1.00	8.9	4.8						

5. Control Points

About Point	P	X-Moment	Y-Moment	NA Depth	d _t Depth	ϵ_t	ϕ
	kip	k-ft	k-ft	in	in		
X @ Max compression	3643.0	0.83	0.83	99.68	30.94	-0.00207	1.00000
X @ Allowable comp.	3643.0	0.83	0.83	99.68	30.94	-0.00207	1.00000
X @ $f_s = 0.0$	2627.1	1018.27	-0.11	30.94	30.94	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1888.5	1381.10	-0.57	23.00	30.94	0.00103	1.00000
X @ Balanced point	1383.5	1497.07	-1.04	18.31	30.94	0.00207	1.00000
X @ Tension control	724.9	1317.13	-0.91	11.50	30.94	0.00507	1.00000
X @ Pure bending	0.0	731.99	-0.89	4.07	30.94	0.01982	1.00000
X @ Max tension	-600.0	-0.89	-0.89	0.00	30.94	9.99999	1.00000
-X @ Max compression	3643.0	0.83	0.83	99.68	30.94	-0.00207	1.00000
-X @ Allowable comp.	3643.0	0.83	0.83	99.68	30.94	-0.00207	1.00000
-X @ $f_s = 0.0$	2626.4	-1017.70	0.89	30.94	30.94	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1887.5	-1381.08	0.87	23.00	30.94	0.00103	1.00000
-X @ Balanced point	1382.3	-1497.65	0.83	18.31	30.94	0.00207	1.00000
-X @ Tension control	724.4	-1317.32	0.64	11.50	30.94	0.00507	1.00000
-X @ Pure bending	0.0	-732.75	1.11	4.09	30.94	0.01970	1.00000
-X @ Max tension	-600.0	-0.89	-0.89	0.00	30.94	9.99999	1.00000

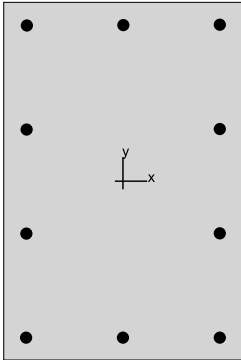
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand P_u kip	M_{ux} k-ft	Capacity ϕP_n kip	ϕM_{nx} k-ft	Parameters at Capacity NA Depth in	ϵ_t	ϕ	Capacity Ratio
1	85.00	-142.30	85.00	-826.94	4.92	0.01586	1.000	0.17
2	-19.80	142.30	-19.80	709.46	3.89	0.02089	1.000	0.20

7. Diagrams

7.1. PM at $\theta=0$ [deg]



22 x 33 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi

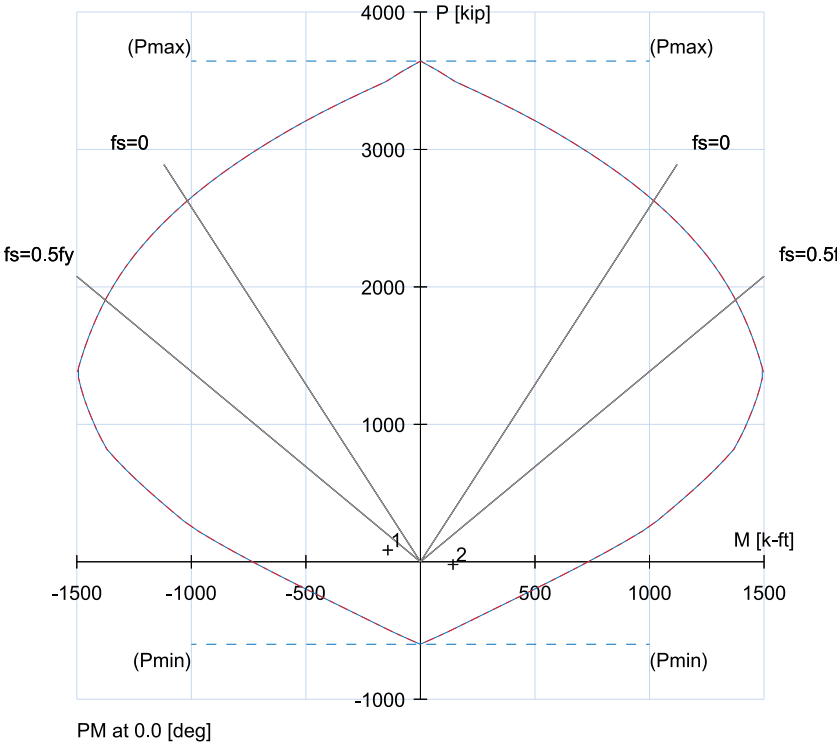
Section

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---

Confinement type	Other
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Min. clear spacing	7.77 in

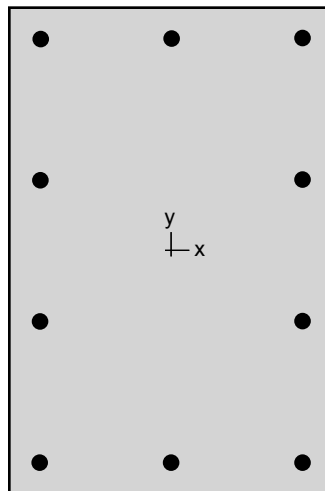


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-19.8	142.3	-19.80	709.46	0.20
1	85.0	-142.3	85.00	-826.94	0.17

Max. Capacity Ratio: 0.20



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

- 1. General Information3
- 2. Material Properties.....3
 - 2.1. Concrete3
 - 2.2. Steel3
- 3. Section.....3
 - 3.1. Shape and Properties.....3
 - 3.2. Section Figure4
 - 3.3. Solids.....4
 - 3.3.1. S1.....4
- 4. Reinforcement4
 - 4.1. Bar Set: ASTM A6154
 - 4.2. Confinement and Factors4
 - 4.3. Arrangement.....5
 - 4.4. Bars Provided5
- 5. Control Points5
- 6. Factored Loads and Moments with Corresponding Capacity Ratios.....5
- 7. Diagrams6
 - 7.1. PM at $\theta=0$ [deg]6

List of Figures

- Figure 1: Column section.....4

1. General Information

File Name	X:\25534-Fire Statio...\SPCOL Mpr - Col Left.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	75 ksi
E_s	29000 ksi
ϵ_{ty}	0.00258621 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴
r_x	9.52628 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

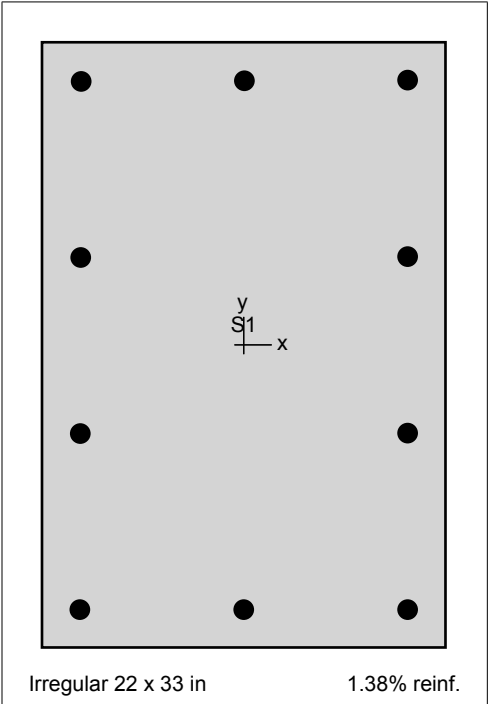


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-16.5	2	11.0	-16.5	3	11.0	16.5
4	-11.0	16.5						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Minimum clear spacing	7.77 in

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
1.00	-8.9	14.4	1.00	0.0	14.4	1.00	8.9	14.4
1.00	-8.9	-14.4	1.00	0.0	-14.4	1.00	8.9	-14.4
1.00	-8.9	-4.8	1.00	-8.9	4.8	1.00	8.9	-4.8
1.00	8.9	4.8						

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d_t Depth in	ϵ_t	ϕ
X @ Max compression	3793.0	1.06	1.06	224.28	30.94	-0.00259	1.00000
X @ Allowable comp.	3793.0	1.06	1.06	224.28	30.94	-0.00259	1.00000
X @ $f_s = 0.0$	2672.1	1072.41	-0.11	30.94	30.94	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1791.4	1475.69	-0.69	21.62	30.94	0.00129	1.00000
X @ Balanced point	1224.8	1582.09	-1.28	16.61	30.94	0.00259	1.00000
X @ Tension control	618.0	1383.31	-1.13	10.81	30.94	0.00559	1.00000
X @ Pure bending	0.0	901.77	-1.12	5.11	30.94	0.01516	1.00000
X @ Max tension	-750.0	-1.12	-1.12	0.00	30.94	9.99999	1.00000
-X @ Max compression	3793.0	1.06	1.06	224.28	30.94	-0.00259	1.00000
-X @ Allowable comp.	3793.0	1.06	1.06	224.28	30.94	-0.00259	1.00000
-X @ $f_s = 0.0$	2671.4	-1071.70	1.03	30.94	30.94	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1790.4	-1475.69	0.99	21.62	30.94	0.00129	1.00000
-X @ Balanced point	1223.5	-1582.82	0.95	16.61	30.94	0.00259	1.00000
-X @ Tension control	616.5	-1382.61	1.12	10.81	30.94	0.00559	1.00000
-X @ Pure bending	0.0	-902.67	0.87	5.13	30.94	0.01510	1.00000
-X @ Max tension	-750.0	-1.12	-1.12	0.00	30.94	9.99999	1.00000

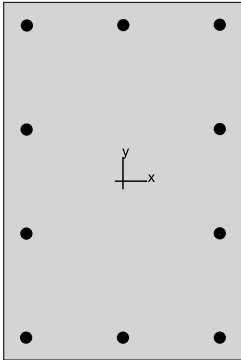
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	85.00	-142.30	85.00	-991.04	6.05	0.01235	1.000	0.14
2	-19.80	142.30	-19.80	880.47	4.90	0.01592	1.000	0.16

7. Diagrams

7.1. PM at $\theta=0$ [deg]



22 x 33 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

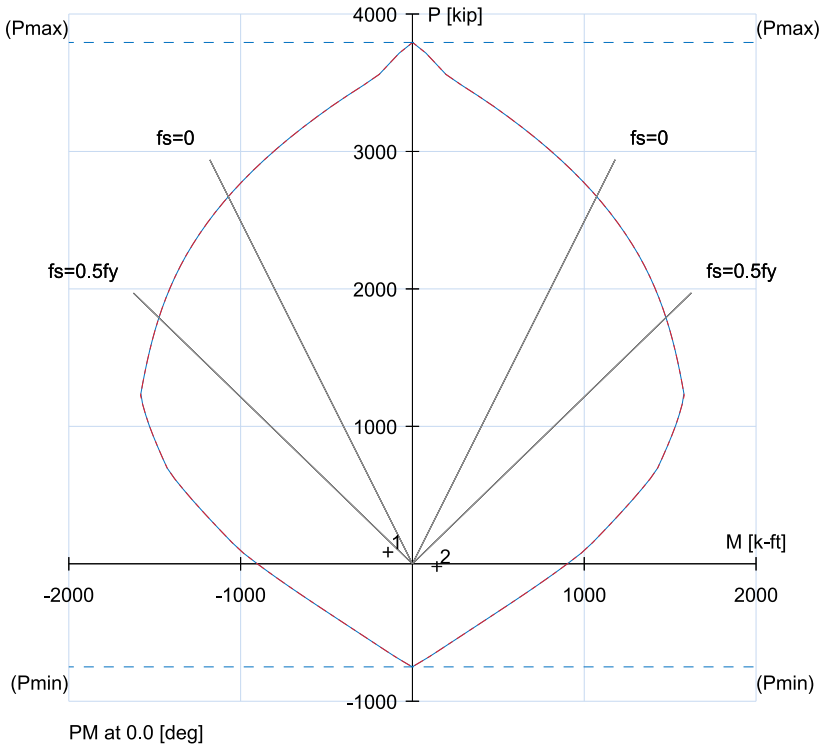
f'_c	5 ksi
E_c	4030.51 ksi
f_y	75 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Other
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Min. clear spacing	7.77 in

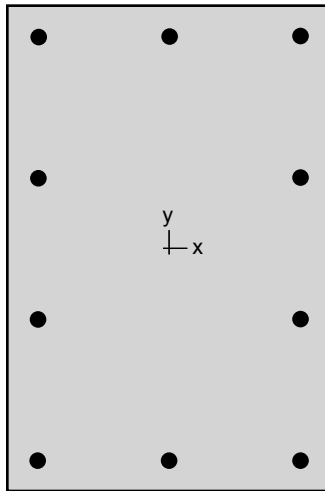


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-19.8	142.3	-19.80	880.47	0.16
1	85.0	-142.3	85.00	-991.04	0.14

Max. Capacity Ratio: 0.16



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Stat...\SPCOL phiMn - Col Left.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴
r_x	9.52628 in
r_y	6.35085 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

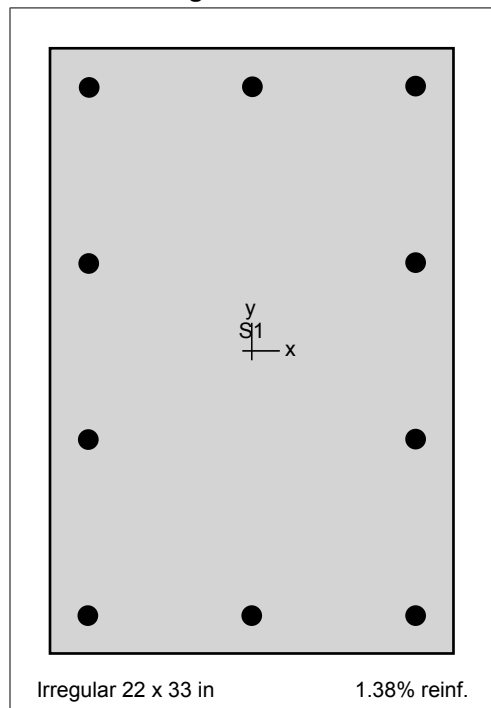


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-11.0	-16.5	2	11.0	-16.5	3	11.0	16.5
4	-11.0	16.5						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ , (b)	0.9
Compression controlled ϕ , (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Minimum clear spacing	7.77 in

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
1.00	-8.9	14.4	1.00	0.0	14.4	1.00	8.9	14.4
1.00	-8.9	-14.4	1.00	0.0	-14.4	1.00	8.9	-14.4
1.00	-8.9	-4.8	1.00	-8.9	4.8	1.00	8.9	-4.8
1.00	8.9	4.8						

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	2367.9	0.54	0.54	99.68	30.94	-0.00207	0.65000
X @ Allowable comp.	1894.4	514.46	0.02	34.21	30.94	-0.00029	0.65000
X @ $f_s = 0.0$	1707.6	661.88	-0.07	30.94	30.94	0.00000	0.65000
X @ $f_s = 0.5 f_y$	1227.5	897.71	-0.37	23.00	30.94	0.00103	0.65000
X @ Balanced point	899.3	973.10	-0.68	18.31	30.94	0.00207	0.65000
X @ Tension control	652.4	1185.42	-0.82	11.50	30.94	0.00507	0.90000
X @ Pure bending	0.0	658.79	-0.81	4.07	30.94	0.01982	0.90000
X @ Max tension	-540.0	-0.81	-0.81	0.00	30.94	9.99999	0.90000
-X @ Max compression	2367.9	0.54	0.54	99.68	30.94	-0.00207	0.65000
-X @ Allowable comp.	1894.4	-513.62	0.59	34.21	30.94	-0.00029	0.65000
-X @ $f_s = 0.0$	1707.2	-661.51	0.58	30.94	30.94	0.00000	0.65000
-X @ $f_s = 0.5 f_y$	1226.9	-897.70	0.56	23.00	30.94	0.00103	0.65000
-X @ Balanced point	898.5	-973.47	0.54	18.31	30.94	0.00207	0.65000
-X @ Tension control	652.0	-1185.59	0.58	11.50	30.94	0.00507	0.90000
-X @ Pure bending	0.0	-659.47	1.00	4.09	30.94	0.01970	0.90000
-X @ Max tension	-540.0	-0.81	-0.81	0.00	30.94	9.99999	0.90000

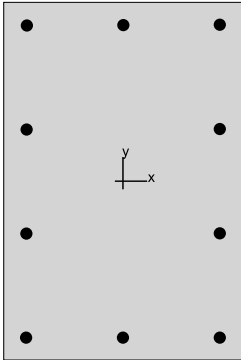
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	85.00	-142.30	85.00	-753.42	5.02	0.01549	0.900	0.19
2	-19.80	142.30	-19.80	636.25	3.87	0.02101	0.900	0.22

7. Diagrams

7.1. PM at $\theta=0$ [deg]



22 x 33 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi

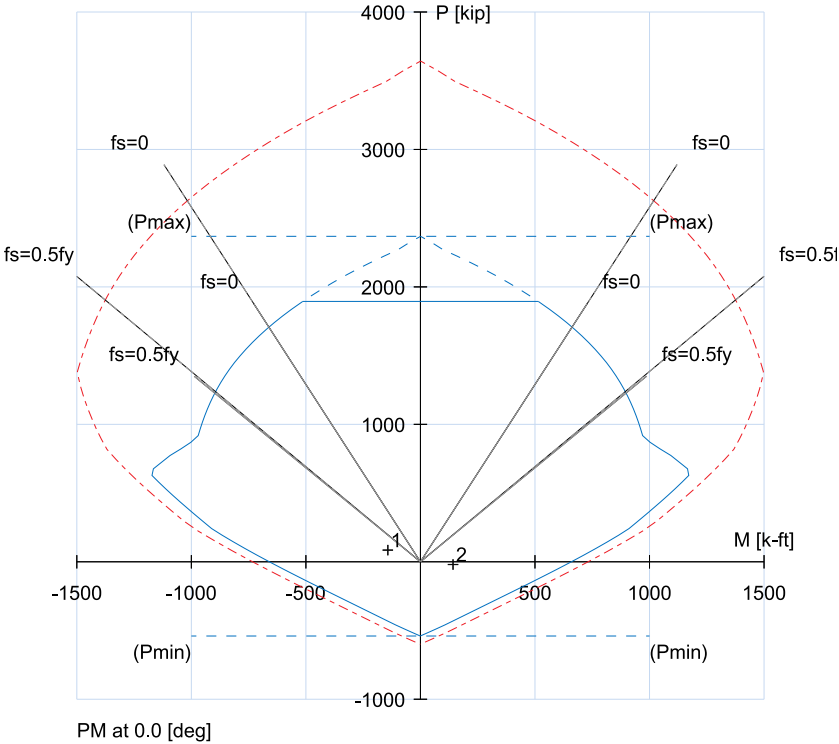
Section

Type	Irregular
A_g	726 in ²
I_x	65884.5 in ⁴
I_y	29282 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---

Confinement type	Tied
Total steel area, A_s	10.00 in ²
Rho	1.38 %
Min. clear spacing	7.77 in



No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-19.8	142.3	-19.80	636.25	0.22
1	85.0	-142.3	85.00	-753.42	0.19

Max. Capacity Ratio: 0.22

SEISMIC GRADE BEAM DESIGN

Code: ACI 318-19

Design Parameters

Clear Span, l_n (ft) =	14.25
h (in.) =	28
b_w (in.) =	26
Top cover (in.) =	2
Bot cover (in.) =	3
d^+ (in.) =	23.9
d^- (in.) =	24.9
f'_c (psi) =	5000
f_y (psi) =	60000

$$M_u \text{ (k-ft)} = 143.8$$

$$P_u \text{ (k-ft)} = 15.6 \quad (\text{frame action} + \Omega(V_e - V_{res})/2)$$

$$M_u < \phi M_n = \text{OK}$$

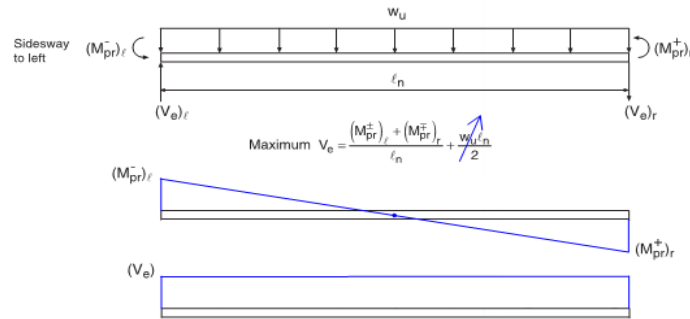
$$\phi M_{n+} \text{ (k-ft)} = 255.5$$

$$\phi M_{n-} \text{ (k-ft)} = 283.7$$

$$M_{pr+} \text{ (k-ft)} = 357.4$$

$$M_{pr-} \text{ (k-ft)} = 385.7$$

$$\text{Shear, } V_{pr} \text{ (kips)} = 52.15 \quad (M_{pr+} - M_{pr-})/L$$



Reinforcing Bars

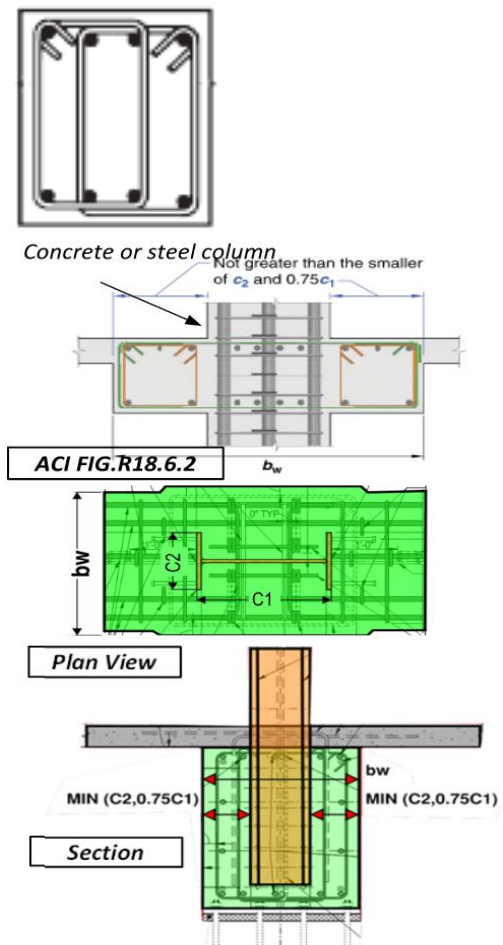
Top Longit. Reinf. =	(3)	# 8	2.37 in.^2
	()	# 8	0.00 in.^2
Bot. Longit. Reinf. =	(3)	# 8	2.37 in.^2
	()	# 8	0.00 in.^2
A_s (in.^2) =	4.74		
Transverse Bar =	# 5		

Scope

ACI 18.6.2.1	Clear Span, $l_n \geq 4d =$	95.50 in.	OK
	Beam Width, $b_w \geq \max(0.3h, 10 \text{ in}) =$	10.00 in	OK
	Projected Width $\leq \min(c_2, 0.75c_1) =$	11.50 in	OK
ACI 18.6.3.1	$A_s \geq A_{s,min}$: $A_{s,min} =$	2.19	OK
	$\rho \leq 0.025$: $\rho =$	0.0065	OK
ACI 18.6.3.3	Spacing of transverse reinforcement enclosing the Lap Splices:		
	$\min\{d/4, 4''\} =$	4 in.	

Transverse Reinforcement

ACI 18.6.4.4	Hoop Spacing shall not exceed min of:		
a)	$d/4 =$	5.97 in.	
b)	$6 * \min.d_{bar} =$	6.00 in.	
c)	$6'' =$	6.00 in.	
	$S_{max} =$	5.97 in.	
	(SPAN A) USE:	5 in.	OK
ACI 18.6.4.1	Span of Hoop Reinf. =	56 in.	(2h)



SEISMIC GRADE BEAM DESIGN

Code: ACI 318-19

ACI 18.6.4.6 Where hoops are not required, provide stirrups with seismic hooks at both ends.
Stirrup $s_{max} \leq d/2$: 11.94 in.
(SPAN B) USE: 6 in. OK

Shear Reinforcement

Assume Constant Shear over span of grade beam
Span A

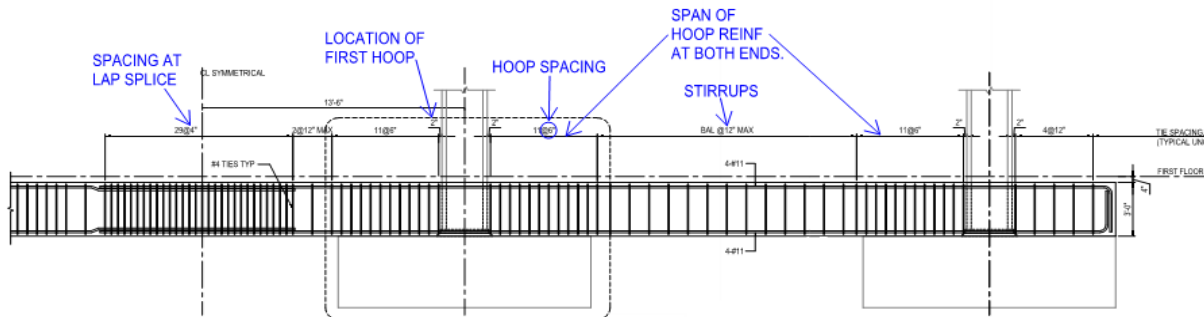
$A_{v,required} (in.^2) = V_{pr} * s / (f_y * d) = 0.24$
of Legs required = 0.78
of Legs provided = 3 OK

Span B

$A_{v,required} (in.^2) = V_{pr} * s / (f_y * d) = 0.29$
of Legs required = 0.94
of Legs provided = 3 OK

Summary

<---Face of Column		Face column----->
<-----Span A----->	<-----Span B----->	<-----Span A----->
56.00 in	59.00 in	56.00 in
Spacing: 5 in. oc	6 in. oc	5 in. oc
# of Legs: 3	3	3
Transverse Bar: # 5		

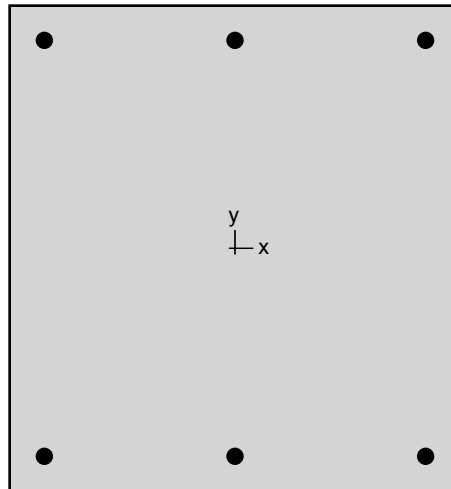


Additional Notes:

Lap Bars Shall Not exceed $d/4$ " or 4" spacing for transverse reinforcement (21.5.2.3)
Lap Splices shall not be used:
a) within the Joint
b) withing a distance $2 * h$ from the face of the joint
c) At locations where analysis indicates Flexural Yielding
Mechanical Splices shall conform to 21.1.6
Welded Splices shall conform to 21.1.7
Hoop spacing shall be located no more than 2" from face of supporting member (21.5.3.2)



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

- 1. General Information3
- 2. Material Properties.....3
 - 2.1. Concrete3
 - 2.2. Steel3
- 3. Section.....3
 - 3.1. Shape and Properties.....3
 - 3.2. Section Figure4
 - 3.3. Solids.....4
 - 3.3.1. S1.....4
- 4. Reinforcement4
 - 4.1. Bar Set: ASTM A6154
 - 4.2. Confinement and Factors4
 - 4.3. Arrangement.....5
 - 4.4. Bars Provided5
- 5. Control Points5
- 6. Factored Loads and Moments with Corresponding Capacity Ratios.....5
- 7. Diagrams6
 - 7.1. PM at $\theta=0$ [deg]6

List of Figures

- Figure 1: Column section.....4

1. General Information

File Name	X:\25534-Fire Station...\SPCOL Mn - Right GB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴
r_x	8.0829 in
r_y	7.50555 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

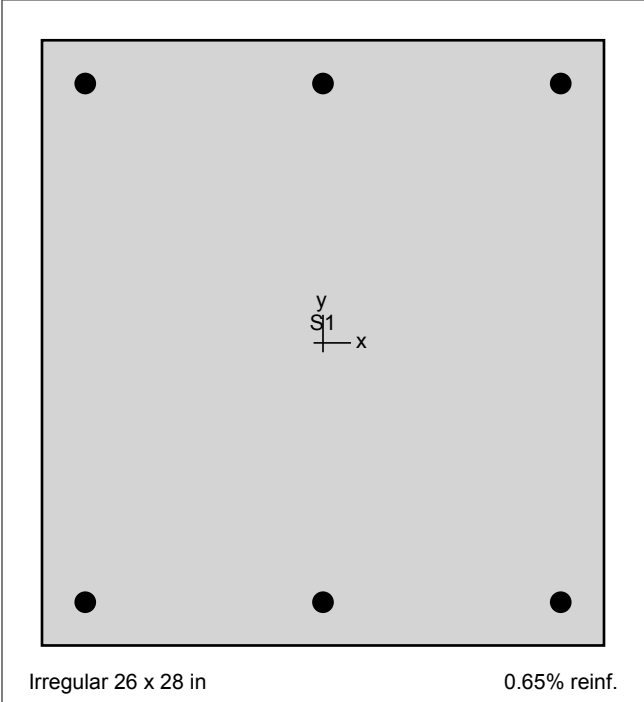


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-13.0	-14.0	2	13.0	-14.0	3	13.0	14.0
4	-13.0	14.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	4.74 in ²
Rho	0.65 %
Minimum clear spacing	10.00 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-11.0	12.0	0.79	0.0	12.0	0.79	11.0	12.0
0.79	-11.0	-12.0	0.79	0.0	-12.0	0.79	11.0	-12.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	3358.3	0.00	0.00	83.77	26.00	-0.00207	1.00000
X @ Allowable comp.	3358.3	0.00	0.00	83.77	26.00	-0.00207	1.00000
X @ $f_s = 0.0$	2430.4	821.71	0.00	26.00	26.00	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1770.0	1095.73	0.00	19.33	26.00	0.00103	1.00000
X @ Balanced point	1350.1	1163.55	0.00	15.39	26.00	0.00207	1.00000
X @ Tension control	844.4	995.87	0.00	9.67	26.00	0.00507	1.00000
X @ Pure bending	0.0	301.47	0.00	1.83	26.00	0.03965	1.00000
X @ Max tension	-284.4	0.00	0.00	0.00	26.00	9.99999	1.00000
-X @ Max compression	3358.3	0.00	0.00	83.77	26.00	-0.00207	1.00000
-X @ Allowable comp.	3358.3	0.00	0.00	83.77	26.00	-0.00207	1.00000
-X @ $f_s = 0.0$	2430.4	-821.71	0.00	26.00	26.00	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1770.0	-1095.73	0.00	19.33	26.00	0.00103	1.00000
-X @ Balanced point	1350.1	-1163.55	0.00	15.39	26.00	0.00207	1.00000
-X @ Tension control	844.4	-995.87	0.00	9.67	26.00	0.00507	1.00000
-X @ Pure bending	0.0	-301.47	0.00	1.83	26.00	0.03965	1.00000
-X @ Max tension	-284.4	0.00	0.00	0.00	26.00	9.99999	1.00000

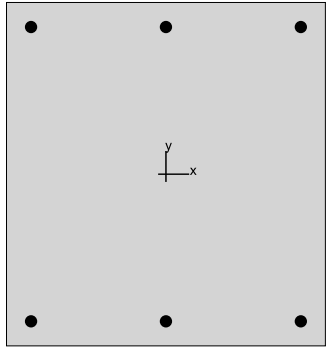
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand P_u kip	M_{ux} k-ft	Capacity ϕP_n kip	ϕM_{nx} k-ft	Parameters at Capacity NA Depth in	ϵ_t	ϕ	Capacity Ratio
1	12.30	-143.80	12.30	-314.00	1.89	0.03831	1.000	0.46
2	-15.60	143.80	-15.60	285.58	1.76	0.04139	1.000	0.50

7. Diagrams

7.1. PM at $\theta=0$ [deg]



26 x 28 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi

Section

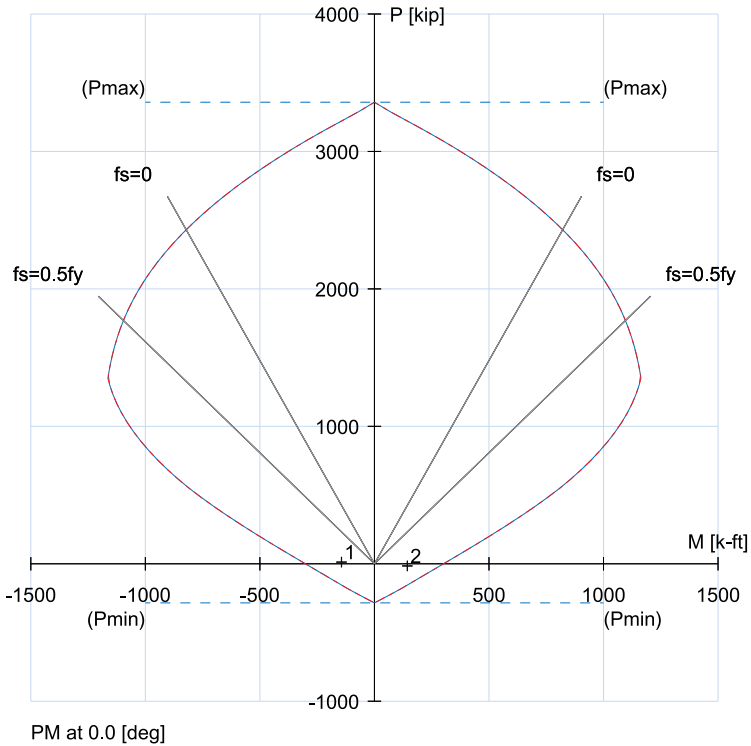
Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---

Confinement type Other

Total steel area, A_s	4.74 in ²
Rho	0.65 %
Min. clear spacing	10.00 in

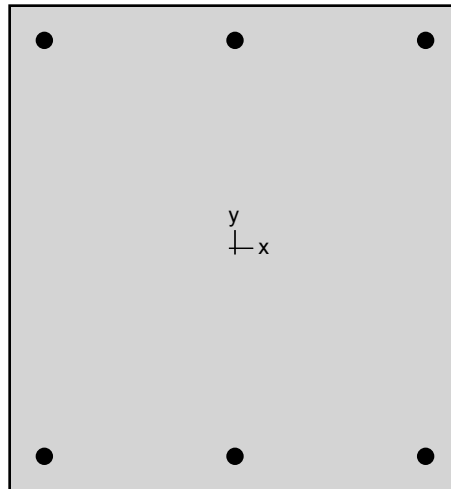


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-15.6	143.8	-15.60	285.58	0.50
1	12.3	-143.8	12.30	-314.00	0.46

Max. Capacity Ratio: 0.50



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

- 1. General Information3
- 2. Material Properties.....3
 - 2.1. Concrete3
 - 2.2. Steel3
- 3. Section.....3
 - 3.1. Shape and Properties.....3
 - 3.2. Section Figure4
 - 3.3. Solids.....4
 - 3.3.1. S1.....4
- 4. Reinforcement4
 - 4.1. Bar Set: ASTM A6154
 - 4.2. Confinement and Factors4
 - 4.3. Arrangement.....5
 - 4.4. Bars Provided5
- 5. Control Points5
- 6. Factored Loads and Moments with Corresponding Capacity Ratios.....5
- 7. Diagrams6
 - 7.1. PM at $\theta=0$ [deg]6

List of Figures

- Figure 1: Column section.....4

1. General Information

File Name	X:\25534-Fire Statio...\SPCOL Mpr - Right GB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	75 ksi
E_s	29000 ksi
ϵ_{ty}	0.00258621 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴
r_x	8.0829 in
r_y	7.50555 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

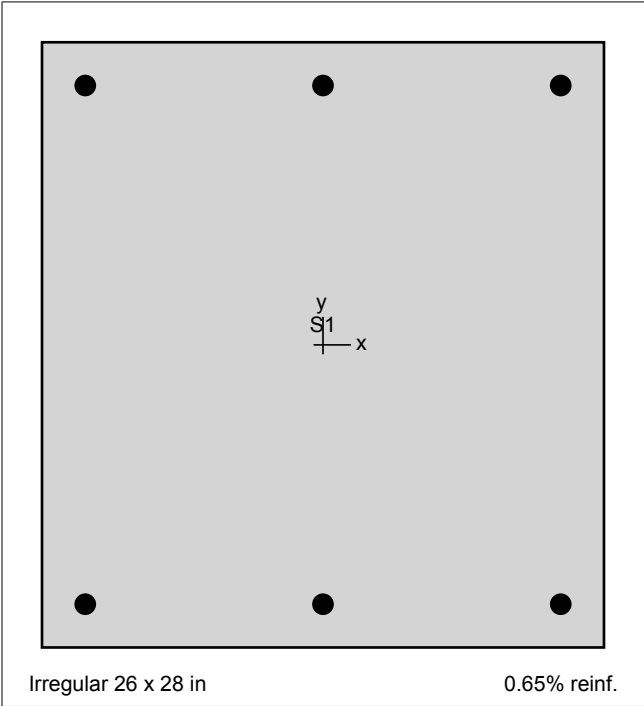


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-13.0	-14.0	2	13.0	-14.0	3	13.0	14.0
4	-13.0	14.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	4.74 in ²
Rho	0.65 %
Minimum clear spacing	10.00 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-11.0	12.0	0.79	0.0	12.0	0.79	11.0	12.0
0.79	-11.0	-12.0	0.79	0.0	-12.0	0.79	11.0	-12.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	3429.4	0.00	0.00	188.49	26.00	-0.00259	1.00000
X @ Allowable comp.	3429.4	0.00	0.00	188.49	26.00	-0.00259	1.00000
X @ $f_s = 0.0$	2465.9	857.25	0.00	26.00	26.00	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1684.8	1157.62	0.00	18.17	26.00	0.00129	1.00000
X @ Balanced point	1223.1	1209.80	0.00	13.96	26.00	0.00259	1.00000
X @ Tension control	775.9	1022.09	0.00	9.08	26.00	0.00559	1.00000
X @ Pure bending	0.0	373.17	0.00	2.01	26.00	0.03589	1.00000
X @ Max tension	-355.5	0.00	0.00	0.00	26.00	9.99999	1.00000
-X @ Max compression	3429.4	0.00	0.00	188.49	26.00	-0.00259	1.00000
-X @ Allowable comp.	3429.4	0.00	0.00	188.49	26.00	-0.00259	1.00000
-X @ $f_s = 0.0$	2465.9	-857.25	0.00	26.00	26.00	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1684.8	-1157.62	0.00	18.17	26.00	0.00129	1.00000
-X @ Balanced point	1223.1	-1209.80	0.00	13.96	26.00	0.00259	1.00000
-X @ Tension control	775.9	-1022.09	0.00	9.08	26.00	0.00559	1.00000
-X @ Pure bending	0.0	-373.17	0.00	2.01	26.00	0.03589	1.00000
-X @ Max tension	-355.5	0.00	0.00	0.00	26.00	9.99999	1.00000

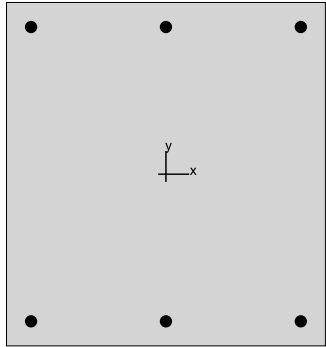
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	12.30	-143.80	12.30	-385.65	2.07	0.03466	1.000	0.37
2	-15.60	143.80	-15.60	357.36	1.93	0.03749	1.000	0.40

7. Diagrams

7.1. PM at $\theta=0$ [deg]



26 x 28 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

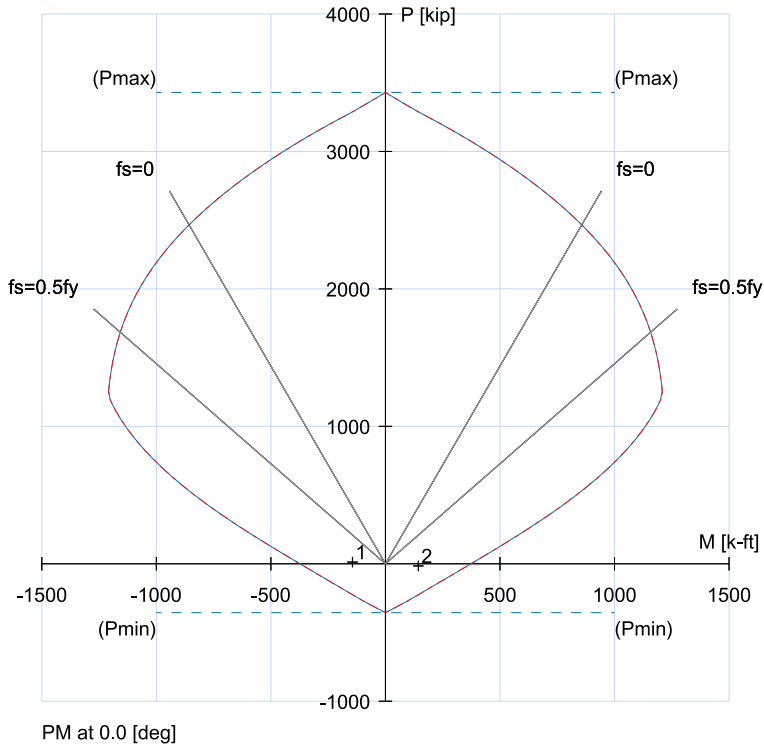
f'_c	5 ksi
E_c	4030.51 ksi
f_y	75 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Other
Total steel area, A_s	4.74 in ²
Rho	0.65 %
Min. clear spacing	10.00 in

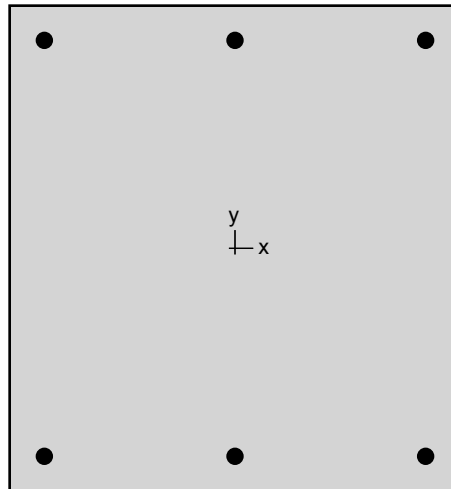


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-15.6	143.8	-15.60	357.36	0.40
1	12.3	-143.8	12.30	-385.65	0.37

Max. Capacity Ratio: 0.40



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Stat...\SPCOL phiMn - Right GB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴
r_x	8.0829 in
r_y	7.50555 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

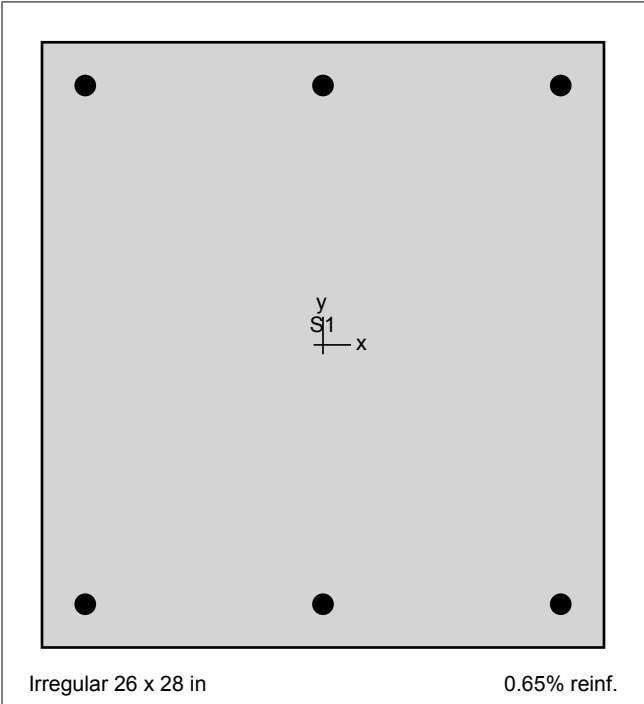


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-13.0	-14.0	2	13.0	-14.0	3	13.0	14.0
4	-13.0	14.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ , (b)	0.9
Compression controlled ϕ , (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	4.74 in ²
Rho	0.65 %
Minimum clear spacing	10.00 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-11.0	12.0	0.79	0.0	12.0	0.79	11.0	12.0
0.79	-11.0	-12.0	0.79	0.0	-12.0	0.79	11.0	-12.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	2182.9	0.00	0.00	83.77	26.00	-0.00207	0.65000
X @ Allowable comp.	1746.3	420.56	0.00	28.68	26.00	-0.00028	0.65000
X @ $f_s = 0.0$	1579.8	534.11	0.00	26.00	26.00	0.00000	0.65000
X @ $f_s = 0.5 f_y$	1150.5	712.22	0.00	19.33	26.00	0.00103	0.65000
X @ Balanced point	877.6	756.31	0.00	15.39	26.00	0.00207	0.65000
X @ Tension control	760.0	896.29	0.00	9.67	26.00	0.00507	0.90000
X @ Pure bending	0.0	271.32	0.00	1.83	26.00	0.03965	0.90000
X @ Max tension	-256.0	0.00	0.00	0.00	26.00	9.99999	0.90000
-X @ Max compression	2182.9	0.00	0.00	83.77	26.00	-0.00207	0.65000
-X @ Allowable comp.	1746.3	-420.56	0.00	28.68	26.00	-0.00028	0.65000
-X @ $f_s = 0.0$	1579.8	-534.11	0.00	26.00	26.00	0.00000	0.65000
-X @ $f_s = 0.5 f_y$	1150.5	-712.22	0.00	19.33	26.00	0.00103	0.65000
-X @ Balanced point	877.6	-756.31	0.00	15.39	26.00	0.00207	0.65000
-X @ Tension control	760.0	-896.29	0.00	9.67	26.00	0.00507	0.90000
-X @ Pure bending	0.0	-271.32	0.00	1.83	26.00	0.03965	0.90000
-X @ Max tension	-256.0	0.00	0.00	0.00	26.00	9.99999	0.90000

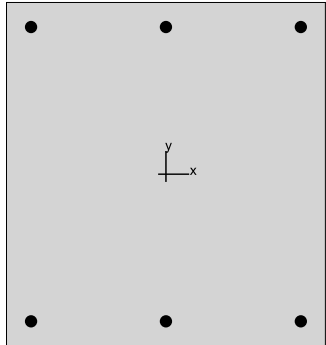
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	12.30	-143.80	12.30	-283.86	1.89	0.03816	0.900	0.51
2	-15.60	143.80	-15.60	255.43	1.75	0.04159	0.900	0.56

7. Diagrams

7.1. PM at $\theta=0$ [deg]



26 x 28 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi

Section

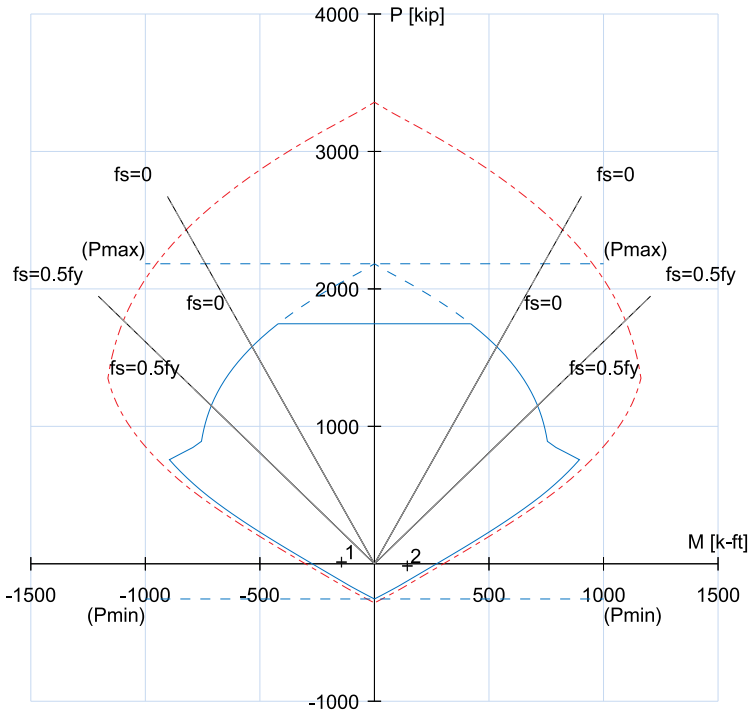
Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---

Confinement type Tied

Total steel area, A_s	4.74 in ²
Rho	0.65 %
Min. clear spacing	10.00 in



PM at 0.0 [deg]

No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-15.6	143.8	-15.60	255.43	0.56
1	12.3	-143.8	12.30	-283.86	0.51

Max. Capacity Ratio: 0.56

SEISMIC GRADE BEAM DESIGN

Code: ACI 318-19

Design Parameters

Clear Span, l_n (ft) =	14.25
h (in.) =	28
b_w (in.) =	26
Top cover (in.) =	2
Bot cover (in.) =	3
d^+ (in.) =	23.9
d^- (in.) =	24.9
f'_c (psi) =	5000
f_y (psi) =	60000

$$M_u \text{ (k-ft)} = 145.8$$

$$P_u \text{ (k-ft)} = 16.4 \quad (\text{frame action} + \Omega(V_e - V_{res})/2)$$

$$M_u < \phi M_n = \text{OK}$$

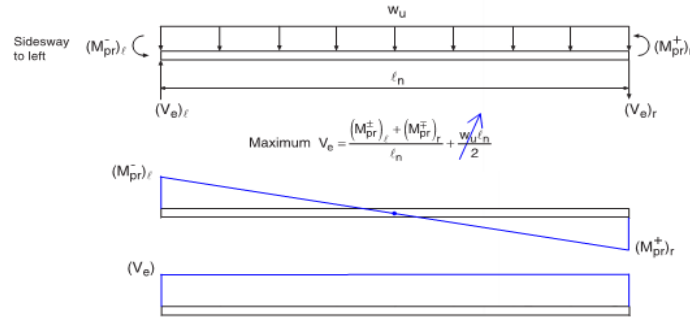
$$\phi M_{n+} \text{ (k-ft)} = 254.6$$

$$\phi M_{n-} \text{ (k-ft)} = 284.7$$

$$M_{pr+} \text{ (k-ft)} = 356.6$$

$$M_{pr-} \text{ (k-ft)} = 386.5$$

$$\text{Shear, } V_{pr} \text{ (kips)} = 52.15 \quad (M_{pr+} - M_{pr-})/L$$



Reinforcing Bars

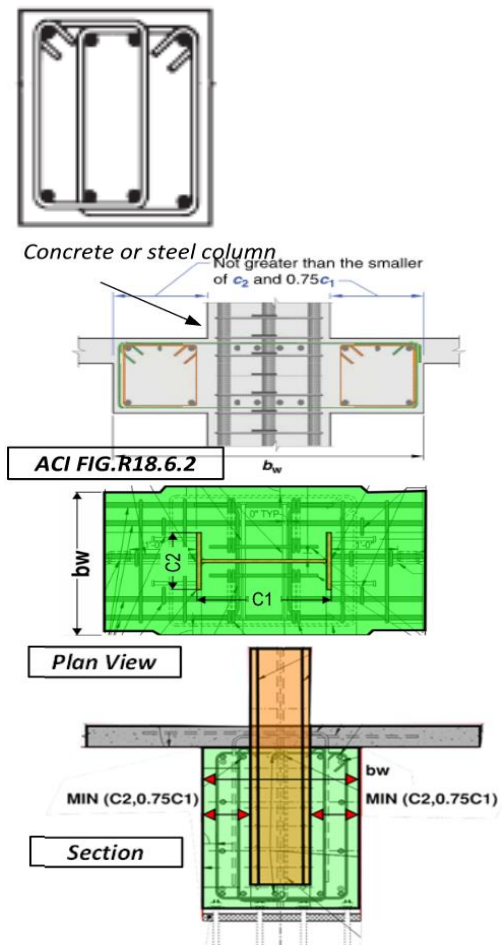
Top Longit. Reinf. =	(3)	# 8	2.37 in.^2
	()	# 8	0.00 in.^2
Bot. Longit. Reinf. =	(3)	# 8	2.37 in.^2
	()	# 8	0.00 in.^2
A_s (in.^2) =	4.74		
Transverse Bar =	# 5		

Scope

ACI 18.6.2.1	Clear Span, $l_n \geq 4d =$	95.50 in.	OK
	Beam Width, $b_w \geq \max(0.3h, 10 \text{ in}) =$	10.00 in	OK
	Projected Width $\leq \min(c_2, 0.75c_1) =$	11.50 in	OK
ACI 18.6.3.1	$A_s \geq A_{s,min}$: $A_{s,min} =$	2.19	OK
	$\rho \leq 0.025$: $\rho =$	0.0065	OK
ACI 18.6.3.3	Spacing of transverse reinforcement enclosing the Lap Splices:		
	$\min\{d/4, 4''\} =$	4 in.	

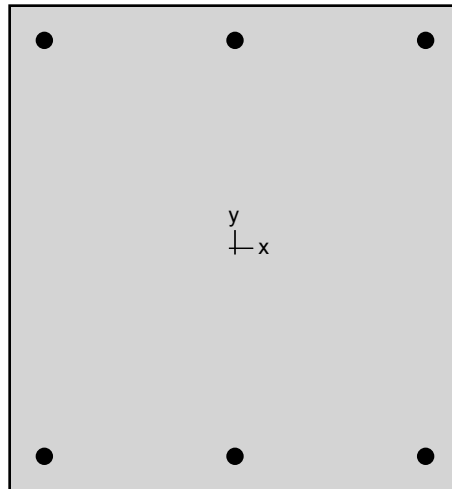
Transverse Reinforcement

ACI 18.6.4.4	Hoop Spacing shall not exceed min of:		
a)	$d/4 =$	5.97 in.	
b)	$6 * \min.d_{bar} =$	6.00 in.	
c)	$6'' =$	6.00 in.	
	$S_{max} =$	5.97 in.	
	(SPAN A) USE:	5 in.	OK
ACI 18.6.4.1	Span of Hoop Reinf. =	56 in.	(2h)





spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

- 1. General Information3
- 2. Material Properties.....3
 - 2.1. Concrete3
 - 2.2. Steel3
- 3. Section.....3
 - 3.1. Shape and Properties.....3
 - 3.2. Section Figure4
 - 3.3. Solids.....4
 - 3.3.1. S1.....4
- 4. Reinforcement4
 - 4.1. Bar Set: ASTM A6154
 - 4.2. Confinement and Factors4
 - 4.3. Arrangement.....5
 - 4.4. Bars Provided5
- 5. Control Points5
- 6. Factored Loads and Moments with Corresponding Capacity Ratios.....5
- 7. Diagrams6
 - 7.1. PM at $\theta=0$ [deg]6

List of Figures

- Figure 1: Column section.....4

1. General Information

File Name	X:\25534-Fire Station ...\SPCOL Mn - Left GB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴
r_x	8.0829 in
r_y	7.50555 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

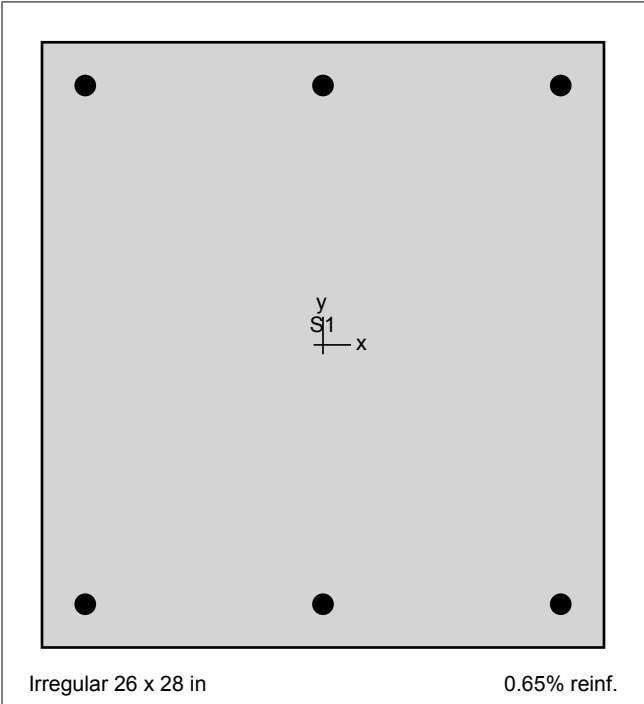


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-13.0	-14.0	2	13.0	-14.0	3	13.0	14.0
4	-13.0	14.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	4.74 in ²
Rho	0.65 %
Minimum clear spacing	10.00 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-11.0	12.0	0.79	0.0	12.0	0.79	11.0	12.0
0.79	-11.0	-12.0	0.79	0.0	-12.0	0.79	11.0	-12.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	3358.3	0.00	0.00	83.77	26.00	-0.00207	1.00000
X @ Allowable comp.	3358.3	0.00	0.00	83.77	26.00	-0.00207	1.00000
X @ $f_s = 0.0$	2430.4	821.71	0.00	26.00	26.00	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1770.0	1095.73	0.00	19.33	26.00	0.00103	1.00000
X @ Balanced point	1350.1	1163.55	0.00	15.39	26.00	0.00207	1.00000
X @ Tension control	844.4	995.87	0.00	9.67	26.00	0.00507	1.00000
X @ Pure bending	0.0	301.47	0.00	1.83	26.00	0.03965	1.00000
X @ Max tension	-284.4	0.00	0.00	0.00	26.00	9.99999	1.00000
-X @ Max compression	3358.3	0.00	0.00	83.77	26.00	-0.00207	1.00000
-X @ Allowable comp.	3358.3	0.00	0.00	83.77	26.00	-0.00207	1.00000
-X @ $f_s = 0.0$	2430.4	-821.71	0.00	26.00	26.00	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1770.0	-1095.73	0.00	19.33	26.00	0.00103	1.00000
-X @ Balanced point	1350.1	-1163.55	0.00	15.39	26.00	0.00207	1.00000
-X @ Tension control	844.4	-995.87	0.00	9.67	26.00	0.00507	1.00000
-X @ Pure bending	0.0	-301.47	0.00	1.83	26.00	0.03965	1.00000
-X @ Max tension	-284.4	0.00	0.00	0.00	26.00	9.99999	1.00000

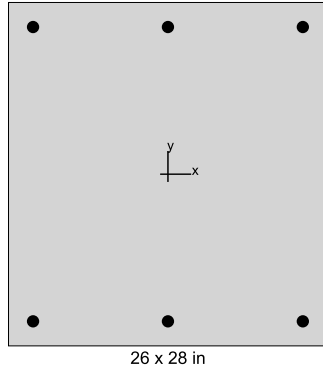
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	13.10	-145.80	13.10	-314.82	1.89	0.03823	1.000	0.46
2	-16.40	145.80	-16.40	284.76	1.75	0.04149	1.000	0.51

7. Diagrams

7.1. PM at $\theta=0$ [deg]



General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

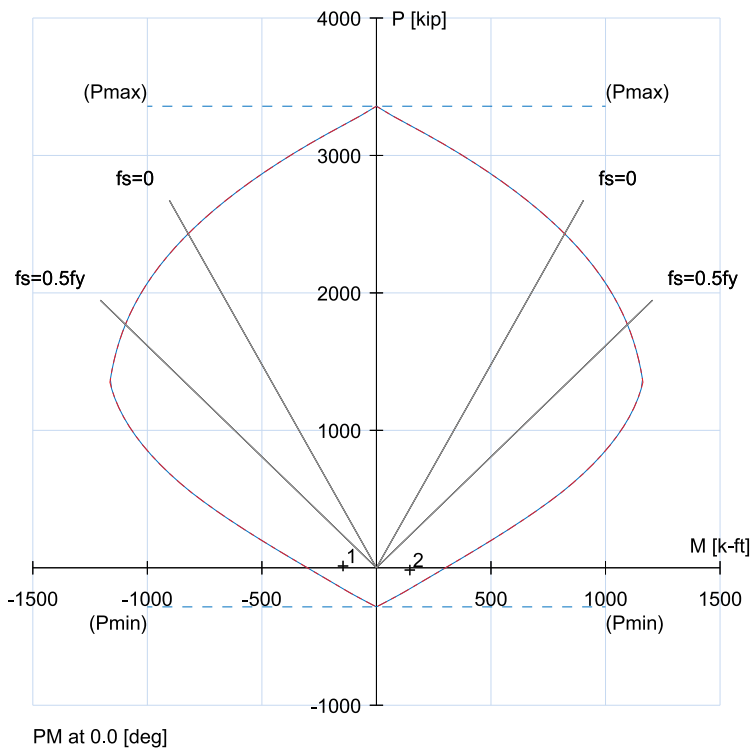
f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Other
Total steel area, A_s	4.74 in ²
Rho	0.65 %
Min. clear spacing	10.00 in

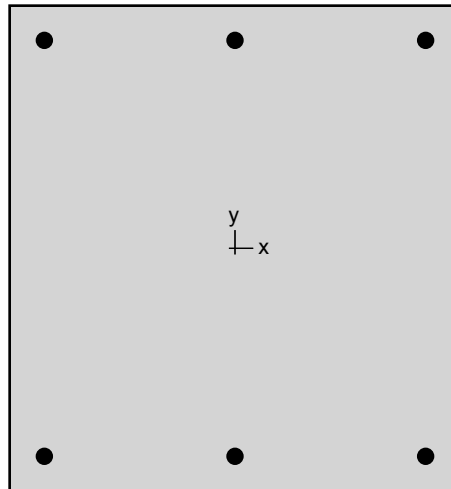


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-16.4	145.8	-16.40	284.76	0.51
1	13.1	-145.8	13.10	-314.82	0.46

Max. Capacity Ratio: 0.51



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

- 1. General Information3
- 2. Material Properties.....3
 - 2.1. Concrete3
 - 2.2. Steel3
- 3. Section.....3
 - 3.1. Shape and Properties.....3
 - 3.2. Section Figure4
 - 3.3. Solids.....4
 - 3.3.1. S1.....4
- 4. Reinforcement4
 - 4.1. Bar Set: ASTM A6154
 - 4.2. Confinement and Factors4
 - 4.3. Arrangement.....5
 - 4.4. Bars Provided5
- 5. Control Points5
- 6. Factored Loads and Moments with Corresponding Capacity Ratios.....5
- 7. Diagrams6
 - 7.1. PM at $\theta=0$ [deg]6

List of Figures

- Figure 1: Column section.....4

1. General Information

File Name	X:\25534-Fire Station...\SPCOL Mpr - Left GB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	75 ksi
E_s	29000 ksi
ϵ_{ty}	0.00258621 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴
r_x	8.0829 in
r_y	7.50555 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

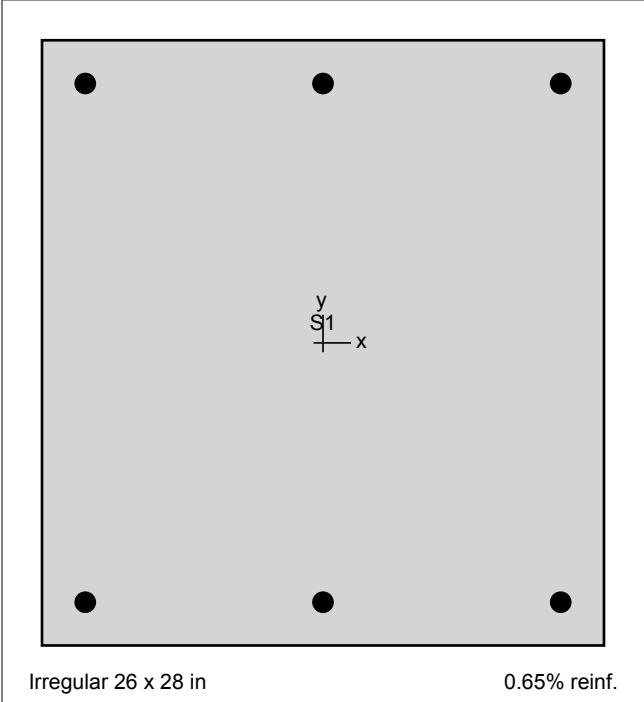


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-13.0	-14.0	2	13.0	-14.0	3	13.0	14.0
4	-13.0	14.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	1
Tension controlled ϕ , (b)	1
Compression controlled ϕ , (c)	1

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	4.74 in ²
Rho	0.65 %
Minimum clear spacing	10.00 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-11.0	12.0	0.79	0.0	12.0	0.79	11.0	12.0
0.79	-11.0	-12.0	0.79	0.0	-12.0	0.79	11.0	-12.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	3429.4	0.00	0.00	188.49	26.00	-0.00259	1.00000
X @ Allowable comp.	3429.4	0.00	0.00	188.49	26.00	-0.00259	1.00000
X @ $f_s = 0.0$	2465.9	857.25	0.00	26.00	26.00	0.00000	1.00000
X @ $f_s = 0.5 f_y$	1684.8	1157.62	0.00	18.17	26.00	0.00129	1.00000
X @ Balanced point	1223.1	1209.80	0.00	13.96	26.00	0.00259	1.00000
X @ Tension control	775.9	1022.09	0.00	9.08	26.00	0.00559	1.00000
X @ Pure bending	0.0	373.17	0.00	2.01	26.00	0.03589	1.00000
X @ Max tension	-355.5	0.00	0.00	0.00	26.00	9.99999	1.00000
-X @ Max compression	3429.4	0.00	0.00	188.49	26.00	-0.00259	1.00000
-X @ Allowable comp.	3429.4	0.00	0.00	188.49	26.00	-0.00259	1.00000
-X @ $f_s = 0.0$	2465.9	-857.25	0.00	26.00	26.00	0.00000	1.00000
-X @ $f_s = 0.5 f_y$	1684.8	-1157.62	0.00	18.17	26.00	0.00129	1.00000
-X @ Balanced point	1223.1	-1209.80	0.00	13.96	26.00	0.00259	1.00000
-X @ Tension control	775.9	-1022.09	0.00	9.08	26.00	0.00559	1.00000
-X @ Pure bending	0.0	-373.17	0.00	2.01	26.00	0.03589	1.00000
-X @ Max tension	-355.5	0.00	0.00	0.00	26.00	9.99999	1.00000

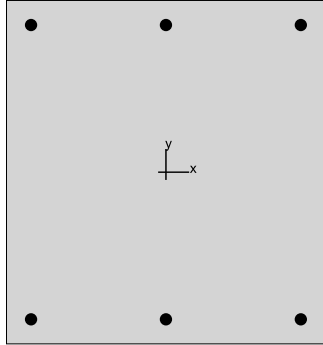
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity Ratio
	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	NA Depth in	ϵ_t	ϕ	
1	13.10	-145.80	13.10	-386.47	2.08	0.03458	1.000	0.38
2	-16.40	145.80	-16.40	356.55	1.92	0.03758	1.000	0.41

7. Diagrams

7.1. PM at $\theta=0$ [deg]



26 x 28 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

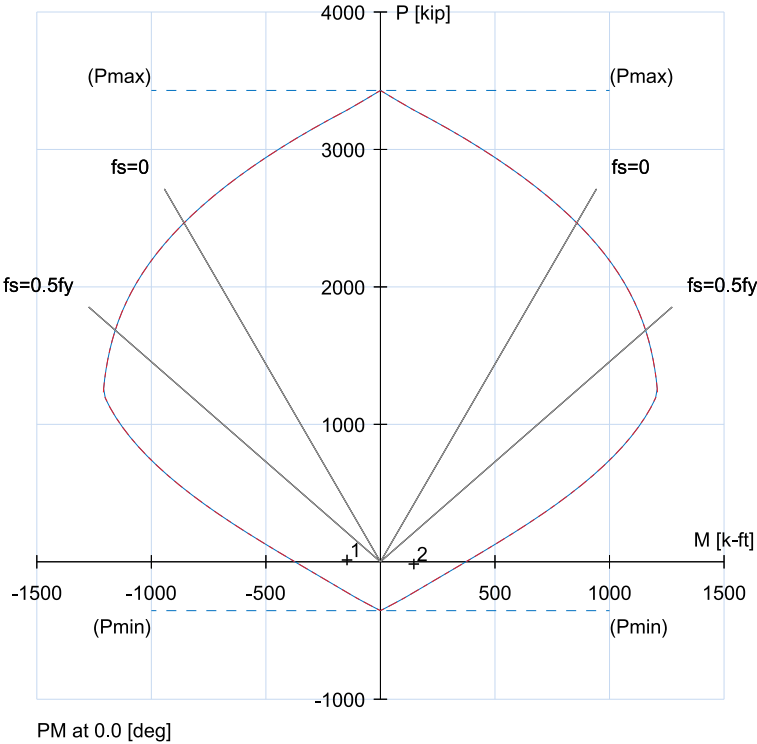
f'_c	5 ksi
E_c	4030.51 ksi
f_y	75 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Other
Total steel area, A_s	4.74 in ²
Rho	0.65 %
Min. clear spacing	10.00 in

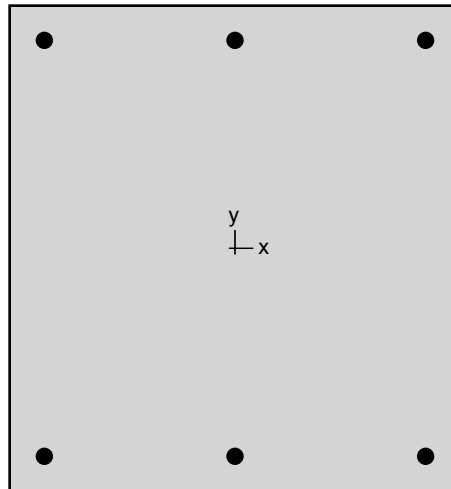


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-16.4	145.8	-16.40	356.55	0.41
1	13.1	-145.8	13.10	-386.47	0.38

Max. Capacity Ratio: 0.41



spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections
Copyright - 1988-2021, STRUCTUREPOINT, LLC.
All rights reserved



StructurePoint

Licensee stated below acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program. Licensed to: Saiful Bouquet - California. License ID: 76342-1083551-2-27A36-2879B

Contents

1. General Information	3
2. Material Properties	3
2.1. Concrete	3
2.2. Steel	3
3. Section	3
3.1. Shape and Properties	3
3.2. Section Figure	4
3.3. Solids	4
3.3.1. S1	4
4. Reinforcement	4
4.1. Bar Set: ASTM A615	4
4.2. Confinement and Factors	4
4.3. Arrangement	5
4.4. Bars Provided	5
5. Control Points	5
6. Factored Loads and Moments with Corresponding Capacity Ratios	5
7. Diagrams	6
7.1. PM at $\theta=0$ [deg]	6

List of Figures

Figure 1: Column section	4
--------------------------------	---

1. General Information

File Name	X:\25534-Fire Stati...\SPCOL phiMn - Left GB.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	5 ksi
E_c	4030.51 ksi
f_c	4.25 ksi
ϵ_u	0.003 in/in
β_1	0.8

2.2. Steel

Type	Standard
f_y	60 ksi
E_s	29000 ksi
ϵ_{ty}	0.00206897 in/in

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴
r_x	8.0829 in
r_y	7.50555 in
X_o	0 in
Y_o	0 in

3.2. Section Figure

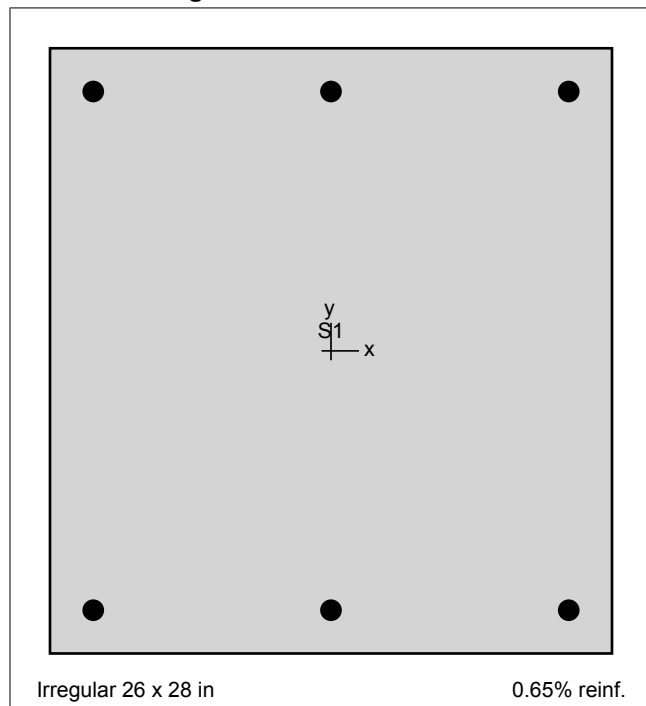


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-13.0	-14.0	2	13.0	-14.0	3	13.0	14.0
4	-13.0	14.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²	Bar	Diameter in	Area in ²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #9 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ , (b)	0.9
Compression controlled ϕ , (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	4.74 in ²
Rho	0.65 %
Minimum clear spacing	10.00 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area in ²	X in	Y in	Area in ²	X in	Y in	Area in ²	X in	Y in
0.79	-11.0	12.0	0.79	0.0	12.0	0.79	11.0	12.0
0.79	-11.0	-12.0	0.79	0.0	-12.0	0.79	11.0	-12.0

5. Control Points

About Point	P kip	X-Moment k-ft	Y-Moment k-ft	NA Depth in	d _t Depth in	ϵ_t	ϕ
X @ Max compression	2182.9	0.00	0.00	83.77	26.00	-0.00207	0.65000
X @ Allowable comp.	1746.3	420.56	0.00	28.68	26.00	-0.00028	0.65000
X @ $f_s = 0.0$	1579.8	534.11	0.00	26.00	26.00	0.00000	0.65000
X @ $f_s = 0.5 f_y$	1150.5	712.22	0.00	19.33	26.00	0.00103	0.65000
X @ Balanced point	877.6	756.31	0.00	15.39	26.00	0.00207	0.65000
X @ Tension control	760.0	896.29	0.00	9.67	26.00	0.00507	0.90000
X @ Pure bending	0.0	271.32	0.00	1.83	26.00	0.03965	0.90000
X @ Max tension	-256.0	0.00	0.00	0.00	26.00	9.99999	0.90000
-X @ Max compression	2182.9	0.00	0.00	83.77	26.00	-0.00207	0.65000
-X @ Allowable comp.	1746.3	-420.56	0.00	28.68	26.00	-0.00028	0.65000
-X @ $f_s = 0.0$	1579.8	-534.11	0.00	26.00	26.00	0.00000	0.65000
-X @ $f_s = 0.5 f_y$	1150.5	-712.22	0.00	19.33	26.00	0.00103	0.65000
-X @ Balanced point	877.6	-756.31	0.00	15.39	26.00	0.00207	0.65000
-X @ Tension control	760.0	-896.29	0.00	9.67	26.00	0.00507	0.90000
-X @ Pure bending	0.0	-271.32	0.00	1.83	26.00	0.03965	0.90000
-X @ Max tension	-256.0	0.00	0.00	0.00	26.00	9.99999	0.90000

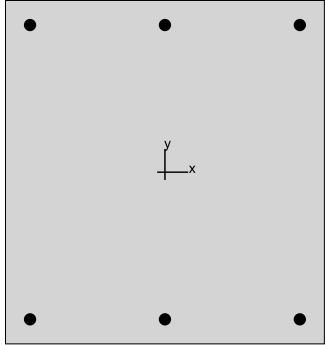
6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand P_u kip	M_{ux} k-ft	Capacity ϕP_n kip	ϕM_{nx} k-ft	Parameters at Capacity NA Depth in	ϵ_t	ϕ	Capacity Ratio
1	13.10	-145.80	13.10	-284.67	1.90	0.03807	0.900	0.51
2	-16.40	145.80	-16.40	254.61	1.75	0.04169	0.900	0.57

7. Diagrams

7.1. PM at $\theta=0$ [deg]



26 x 28 in

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

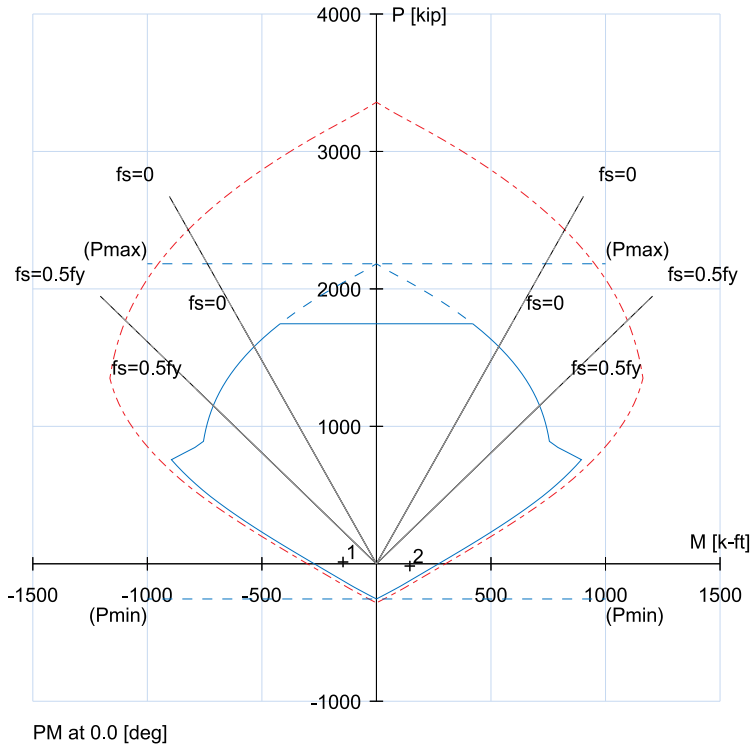
f'_c	5 ksi
E_c	4030.51 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	728 in ²
I_x	47562.7 in ⁴
I_y	41010.7 in ⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	4.74 in ²
Rho	0.65 %
Min. clear spacing	10.00 in

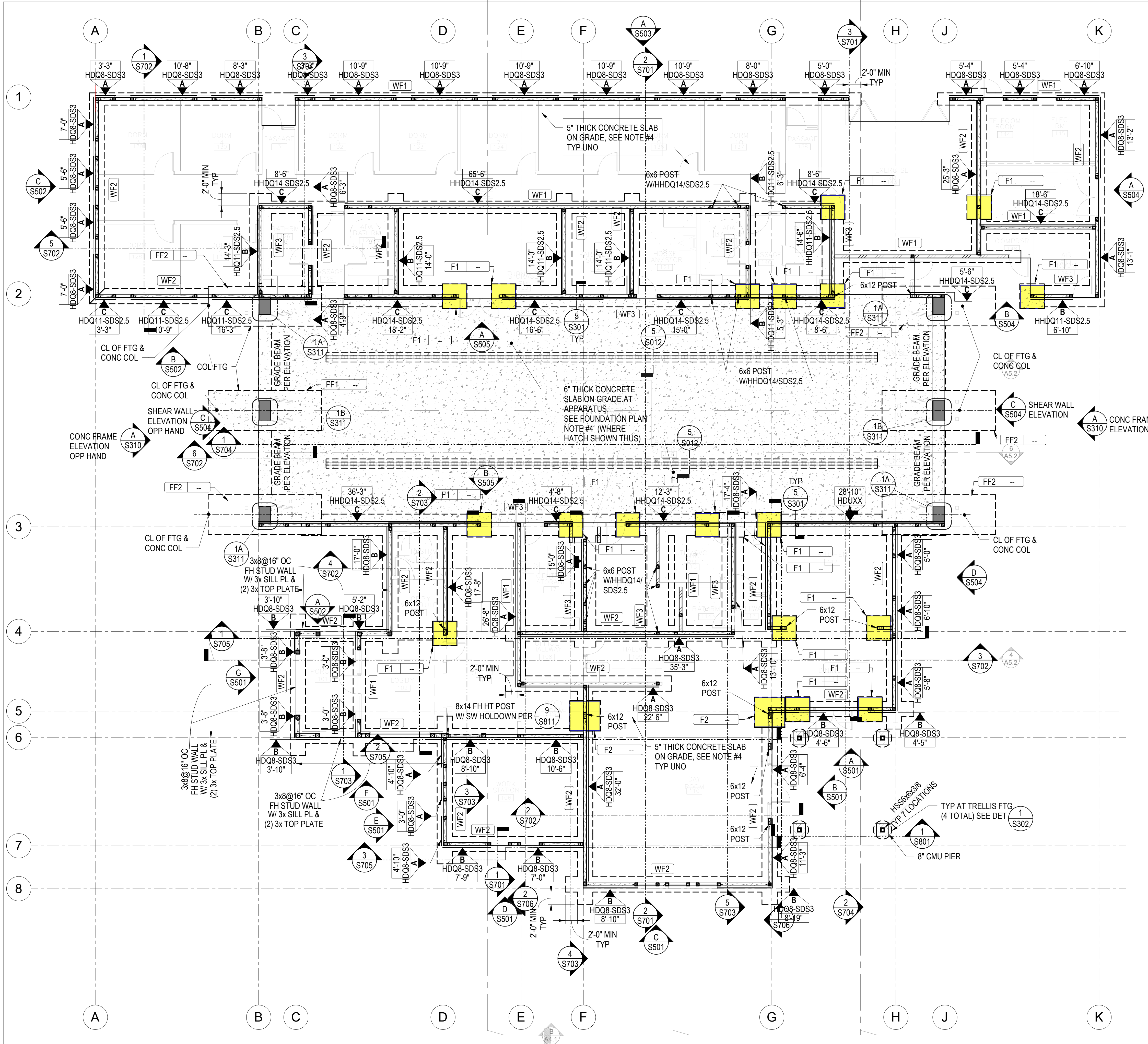


No.	P_u kip	M_{ux} k-ft	ϕP_n kip	ϕM_{nx} k-ft	Capacity Ratio
2	-16.4	145.8	-16.40	254.61	0.57
1	13.1	-145.8	13.10	-284.67	0.51

Max. Capacity Ratio: 0.57

3 FOUNDATION DESIGN

3.1 GRAVITY FOOTING DESIGN



GRAVITY WOOD POST FOOTING KEY MAP

FOUNDATION PLAN NOTES

- FOR GENERAL NOTES SEE S0.0 SERIES AND TYPICAL DETAILS SEE S0 SERIES SHEETS.
- VERIFY ALL DIMENSIONS PRIOR TO START OF WORK. SEE ARCHITECTURAL DRAWINGS FOR REMAINDER OF DIMENSIONS NOT SHOWN ON THIS PLAN.
- SEE ARCHITECTURAL DRAWINGS FOR FLOOR ELEVATIONS, DEPRESSIONS, SLOPES, OPENINGS, CURBS, DRAINS, SLAB EDGE LOCATIONS, ETC., AND FOR WALL OVERALL DIMENSIONS, LOCATIONS OF OPENINGS, ETC., NOT INDICATED ON STRUCTURAL DRAWINGS.
- TYPICAL SLAB ON GRADE SHALL BE AS FOLLOWS UNLESS NOTED OTHERWISE:
 - 5" CONCRETE SLAB W/ #4@18"OC EACH WAY AT CENTER OF SLAB OVER
 - SLAB SHALL BE PLACED OVER 20 MIL VAPOR BARRIER UNDERLAIN AND OVER 4 INCHES OF CLEAN COARSE SAND AND OVERLAIN BY 2 INCHES OF CLEAN COARSE SAND, SEE SOIL REPORT.
- INDICATES 6" THICK CONCRETE SLAB ON GRADE
 - PROVIDE #4@12" EW AT MID-HEIGHT OF SLAB.
 - SLAB SHALL BE PLACED OVER 20 MIL VAPOR BARRIER UNDRALAIN AND OVERLAIN BY 4 INCHES OF CLEAN COARSE SAND AND OVERLAIN BY 2 INCHES OF CLEAN COARSE SAND, SEE SOIL REPORT
- PROVIDE CONSTRUCTION JOINTS AND WEAKENED PLANE JOINTS (INDICATED AS CJ AND PW, RESPECTIVELY) IN SLAB ON GRADE AS SHOWN ON DETAIL (LOCATE PER ARCHITECTURAL DRAWINGS).
- INDICATES SHEAR WALL MARK FOR PLYWOOD SHEAR WALLS:
 - INDICATES SHEAR WALL TYPE PER SCHEDULE
 - INDICATES HOLDOWN TYPE EACH END OF SHEAR WALL PANEL SEE SCHEDULE (S039) FOR HD HOLDOWNS. OMIT HOLDOWNS WHERE HSS COL OCCURS AT END OF WALL.
 - INDICATES SHEAR WALL DESIGN LENGTH
- ☒ INDICATES HOLDOWN LOCATION. PROVIDE POST FOR HOLDOWN PER (S039)
- TYPICAL WOOD STUDS SHALL BE AS FOLLOWS, UNLESS NOTED OTHERWISE ON PLAN
 - 2x6@16" AT ALL EXTERIOR WALL. UNO
 - 2x6 @16" AT INTERIOR WALL UNO
 - EXTERIOR WALL (EXCLUDING THE APPARATUS RESERVE BUILDING): 2x6@16" UNO
 - EXTERIOR WALL FOR APPARATUS RESERVE BUILDING: 2x8" @16" OC
 - INTERIOR BEARING WALL: 2x6@16" AND LARGER AS INDICATED BY ARCH
 - INTERIOR NON-BEARING WALL: 2x4 @16" FOR HEIGHTS UP TO 10' OR 2x6@16" AND LARGER AS INDICATED BY ARCH

FOUNDATION PLAN LEGEND

- WF# INDICATES CONTINUOUS FOOTING SIZE. SEE PLAN (S301) FOR REINFORCING AND INFORMATION.
- FX X'-XX" INDICATES SPREAD FOOTING MARK SEE FOOTING SCHEDULE (S301)
- INDICATES TOP OF FOOTING ELEVATION SHALL BE MIN -1'-0" FROM TOP OF FINISHED GRADE OR LOWEST ADJACENT GRADE

ALL HARDWARE IS BY "SIMPSON" TYPICAL OR APPROVED EQUAL.

- ☒ OR POST INDICATES POST SHALL BE AS FOLLOWS:
4x4 WOOD POST AT 4" INTERIOR STUD WALLS, UNO ON PLAN
6x6 WOOD POST AT 6" EXTERIOR & INTERIOR STUD WALLS, UNO ON PLAN
WHERE WOOD POST ARE ALSO USED AS SHEAR WALL HOLDOWN SEE DETAIL (S039) & (S038) AND USE THE LARGER POST
- INDICATES SLOPED SLAB ON GRADE REFER TO ARCH DWGS
- S- - - S INDICATES STEP FOOTING, SEE DETAIL (S302)
- INDICATES CMU WALL, SEE DETAILS ON SHEETS S021 & S021
- INDICATES RAISED OR DEPRESSED SLAB, SEE DETAILS COORDINATE W/ MECH/ELECT DWGS/ARCH A1.0 (S011) OR (S013)
- GB INDICATES GRADE BEAM, SEE DETAIL (S311)

SHEAR WALL ELEVATION ON GRID 6

SCALE: 1/8" = 1'-0"

1

WILLIAM LOYD JONES
ARCHITECT

9415 culver boulevard
culver city, california
90232

TEL 310 392 3995

so
structural engineers

725 S. Figueroa St.,
3rd floor
Los Angeles, CA 90017
www.safibouquet.com
(213) 315-2277
Project #25534

FOUNDATION PLAN

FIRE STATION 46

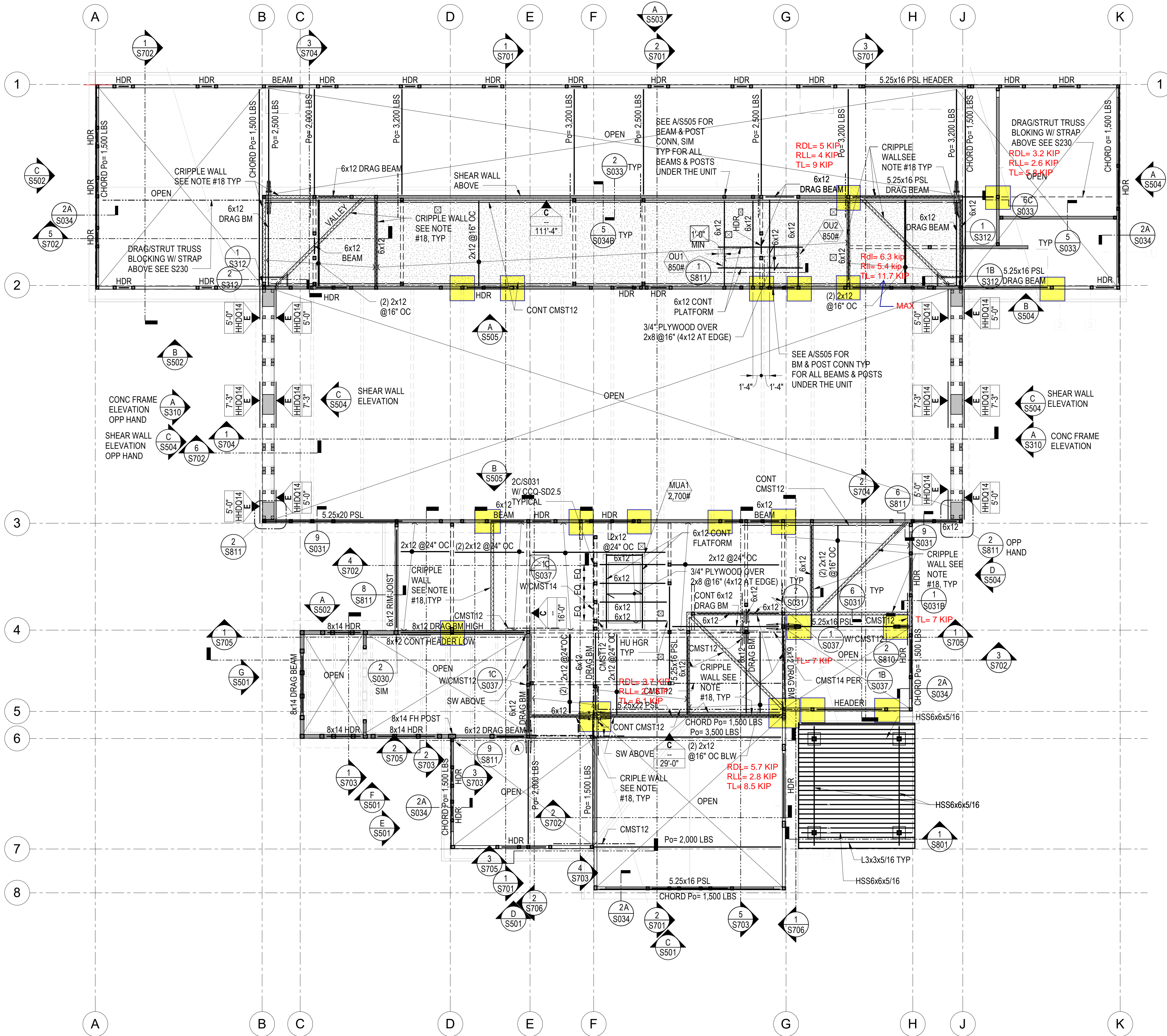
MISSION VILLAGE
COUNTY OF LOS ANGELES FIRE DEPARTMENT
VALENCIA, CALIFORNIA



THE ABOVE DRAWINGS AND SPECIFICATIONS AND ALL DESIGN AND CONSTRUCTION DETAILS ARE THE PROPERTY OF WILLIAM LOYD JONES ARCHITECT AND SHALL REMAIN THE PROPERTY OF THE ARCHITECT. NO PART HEREOF SHALL BE COPIED, REPRODUCED, OR USED IN CONNECTION WITH ANY WORK OR PROJECT OTHER THAN THE SPECIFIC PROJECT FOR WHICH THEY HAVE BEEN PREPARED. ANY REUSE OR MODIFICATION OF THESE DRAWINGS WITHOUT THE WRITTEN CONSENT OF THE ARCHITECT SHALL BE AT THE USER'S SOLE RISK AND WITHOUT LIABILITY TO THE ARCHITECT. WRITTEN OWNERSHIP OF THESE DRAWINGS SHALL HAVE PRECEDENCE OVER ANY OTHER CLAIMS TO INTELLECTUAL PROPERTY. CONTRACTORS SHALL VERIFY AND BE RESPONSIBLE FOR ALL DIMENSIONS AND CONDITIONS ON THE JOB AND THE OFFICE MUST BE NOTIFIED OF ANY VARIATIONS FROM THE DIMENSIONS AND CONDITIONS SHOWN IN THESE DRAWINGS. SHOP DETAILS MUST BE SUBMITTED TO THE OFFICE FOR APPROVAL, BEFORE PROCEEDING WITH FABRICATION.

Date	Issue Date
Drawn	
Checked	
Scale	AS NOTED
Job. No.	Project Number

S210



GRAVITY LOADS ON WOOD POST & FOOTING KEY MAP

FRAMING PLAN NOTES

- FOR GENERAL NOTES SEE S0.0 SERIES AND TYPICAL DETAILS SEE S0 SERIES SHEETS.
- VERIFY ALL DIMENSIONS PRIOR TO START OF WORK. SEE ARCHITECTURAL DRAWINGS FOR REMAINDER OF DIMENSIONS NOT SHOWN ON THIS PLAN.
- WHERE A SHEAR WALL IS SHOWN AS PORTION OF A WALL, THE REMAINDER OF THE WALL (INCLUDING ABOVE AND BELOW OPENINGS, PARAPETS, ETC) SHALL BE SHEATHED WITH THE SAME SHEATHING AND NAILING SCHEDULE ALONG THE ENTIRE WALL. IN ADDITION, CORNERS OF ALL OPENINGS IN SHEAR WALL SHALL BE STRAPPED.
- ALL SHEAR WALLS AND SHEAR TRANSFER NAILS SHALL BE COMMON WIRE NAILS. SINKER AND BOX NAILS ARE NOT PERMITTED.
- HOLDOWN CONNECTORS SHALL BE TIGHTENED JUST PRIOR TO COVERING THE WALL FRAMING.
- INDICATES BEARING AND/OR SHEAR WALL BELOW.
- ROOF DIAPHRAGM SHEATHING AND NAILING SHALL BE PER SHEET S240
- TYPICAL WOOD STUD WALL: SEE NOTE #8 ON SHEET S210
- ALL HARDWARE ARE BY "SIMPSON" TYPICAL OR APPROVED EQUAL.
- ALL SAWN WOOD FRAMING EXPOSED TO WEATHER SHALL BE P.T.D.F.
- HDR INDICATES HEADER PER SCHEDULE SEE DETAIL S030 UNO.
- INDICATES ROOF JOIST
- ROOF DIAPHRAGM NAILING TO BE INSPECTED BEFORE COVERING, FACE GRAIN OF PLYWOOD SHALL BE PERPENDICULAR TO SUPPORT.
- ALL NAILS ARE COMMON NAILS, UNO.
- CMST INDICATES SIMPSON CMST14 STRAP, UNO
- INDICATES SHEAR WALL TYPE PER SCHEDULE
- INDICATES HOLDOWN TYPE EACH END OF SHEAR WALL PANEL SEE SCHEDULE S030 FOR HD HOLDOWNS S030
- INDICATES SHEAR WALL DESIGN LENGTH
- SEE MECH DWGS FOR EQUIPMENT MOUNTED ON FLAT ROOFS, SEE S036 FOR TYPICAL MECHANICAL UNIT PLATFORM.
- INDICATES 2x6 STUD WALL @16"OC (CRIPPLE WALL) WITH PLYWOOD TYPE C PER SCHEDULE S031 MIN SEE DETAILS S031 & S037 FOR S030
- BOTTOM CONNECTION. SEE DETAIL S031 FOR TOP CONNECTION
- Po= INDICATES AXIAL SEISMIC DRAG STRUT LOAD (SERVICE) IN LBS. CONTRACTOR SHALL DESIGN TRUSSES TO RESIST THESE LOADS IN COMBINATION WITH GRAVITY LOADS IN ACCORDANCE WITH APPLICABLE BUILDING CODE AND ALL LISTED CRITERIA. WHERE REQUIRED BY ANALYSIS, PROVIDE DOUBLE OR MORE TRUSSES (PROVIDE MIN DOUBLE TRUSS), SEE S034 UNO
- INDICATES MECHANICAL UNIT TYPE
- INDICATES MECHANICAL UNIT OPERATING WT IN LBS
- T1 INDICATES ROOF WOOD TRUSSES AER DESIGN-BUILT. REFER TO S0.38 SERIES FOR TRUSS PROFILE AND GENERAL NOTES FOR DESIGN CRITERIA, TYP.
- SOLAR PANEL, FRAMING & THEIR CONNECTIONS TO STRUCTURAL MEMBERS BY OTHERS
- INDICATES DRAG/STRUT STRAP AT DRAG BEAM / TRUSS OR BLOCKS AND EXTENT PER PLAN, S037 AT BEAMS & S034 AT TRUSSES CMST14 UNO
- INDICATES CHORD STRAP. PROVIDE MIN 4x4 BLOCKING UNDER STRAP FULL LENGTH OF THE STRAP CMST14 UNO

STRAP NAILING SCHEDULE

- CMST14 - USE (2) ROWS OF 10d NAILS @ 3 1/2" OC UNO
CMST12 - USE (2) ROWS OF 10d NAILS @ 3 1/2" OC UNO
CMSTC16 - USE (2) ROWS OF 16d SINKERS @ 3" OC UNO
SEE S037 FOR SPLICE DETAIL WHERE LENGTH SPECIFIED ON PLAN EXCEEDS AVAILABLE STRAP LENGTH PER MANUFACTURER

- TPS-X INDICATES TOPPLATE SPLICE @ TOP OF WALL ABOVE PER USE TPS-3 AT ALL SHEAR AND BEARING WALLS UNLESS NOTED OTHERWISE

- DRAG BEAM INDICATES DRAG BEAM TO SHEAR WALL PER S037 WITH CMST14 UNO
- DRAG BEAM INDICATES DRAG BEAM TO DRAG BEAM PER WITH (2) HDU11-SDS2.5 EA SIDE (TOTAL 4) UNO
- INDICATES DRAG BEAM TO DRAG TRUSS PER WITH (2) HD7B EA SIDE (TOTAL 4) UNO

FLAT ROOF FRAMING PLAN

SCALE: 1/8" = 1'-0"

1

WILLIAM LOYD JONES
ARCHITECT

9415 culver boulevard
culver city, california
90232

TEL 310 392 3995

S0
saiful-bouquet
structural engineers

725 S. Figueroa St.,
3rd floor
Los Angeles, CA 90017
www.saifulbouquet.com
(213) 315-2277
Project #25534

LOW ROOF FRAMING PLAN

FIRE STATION 46

MISSION VILLAGE
COUNTY OF LOS ANGELES FIRE DEPARTMENT
VALENCIA, CALIFORNIA

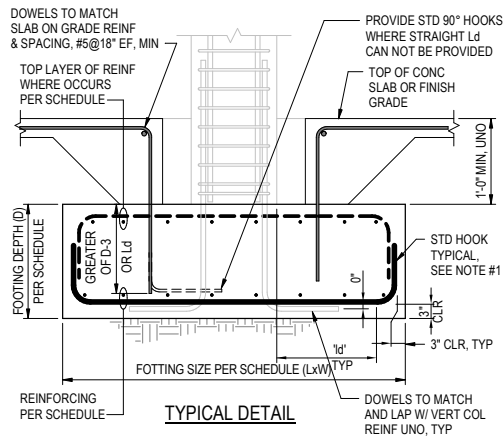


THE ABOVE DRAWINGS AND SPECIFICATIONS AND DESIGN AND CONSTRUCTION OF THE PROJECT SHALL BE THE RESPONSIBILITY OF THE ARCHITECT AND NO PART THEREOF SHALL BE COPIED, REPRODUCED, OR USED IN CONNECTION WITH ANY WORK OR PROJECT OTHER THAN THE SPECIFIC PROJECT FOR WHICH THEY HAVE BEEN PREPARED. ANY REUSE OR MODIFICATION OF THESE DRAWINGS WITHOUT THE WRITTEN CONSENT OF THE ARCHITECT SHALL BE AT THE USER'S SOLE RISK AND WITHOUT LIABILITY TO THE ARCHITECT. THE ARCHITECT'S OFFICE SHALL BE NOTIFIED OF ANY VARIATIONS FROM THE DIMENSIONS AND CONDITIONS SHOWN IN THESE DRAWINGS. SHOP DETAILS MUST BE SUBMITTED TO THE ARCHITECT FOR APPROVAL BEFORE PROCEEDING WITH FABRICATION.

Date	Issue Date
Drawn	
Checked	
Scale	AS NOTED
Job No.	Project Number

S220

APPENDIX 5



SPREAD FOOTING SCHEDULE							
MARK	DEPTH (D)	SIZE (LxW)	REINFORCING				REMARKS
			BOTTOM		TOP		
			SHORT	LONG	SHORT	LONG	
F1	18"	4'-0"x4'-0"	(4)#6(B) EW	(4)#6(B) EW	-	-	
F2	18"	5'-0"x5'-0"	(6)#6(B) EW	(6)#6(B) EW	-	-	
FF1	36"	18'-0"x6'-6"	#8@12"	#8@12"	#8@12"	#8@12"	
FF2	36"	12'-0"x6'-6"	#8@12"	#8@12"	#8@12"	#8@12"	

NOTES:

1. "ld"=STRAIGHT DEVELOPMENT LENGTH PER "TYPICAL STRAIGHT AND HOOKED DEVELOPMENT LENGTH SCHEDULE".
2. PROVIDE STANDARD HOOK WHERE A STRAIGHT BAR WITH "ld" CANNOT BE PROVIDED.

TYPICAL SPREAD FOOTING SCHEDULE AND DETAIL

SCALE: NTS

2

**POST FTG DESIGN
FOR MAX LOAD**

**RdI= 6.3 kip
RII= 5.4 kip
TL= 11.7 KIP**



**F1 4'X4'X18" W/ 4 #5
EA WAY**

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: FS 46 enercalc.ec6

LIC#: KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: ISOLATE POST FOOTING

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16

Load Combinations Used : ASCE 7-16

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	3.0 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing depth

Footing base depth below soil surface	=	ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

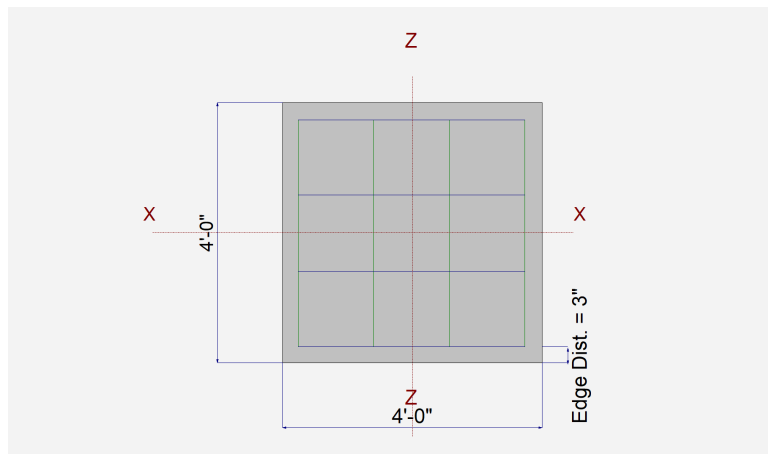
Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

Width parallel to X-X Axis	=	4.0 ft
Length parallel to Z-Z Axis	=	4.0 ft
Footing Thickness	=	18.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	in
pz : parallel to Z-Z Axis	=	in
Height	=	in
Rebar Centerline to Edge of Concrete...	=	3.0 in
at Bottom of footing	=	



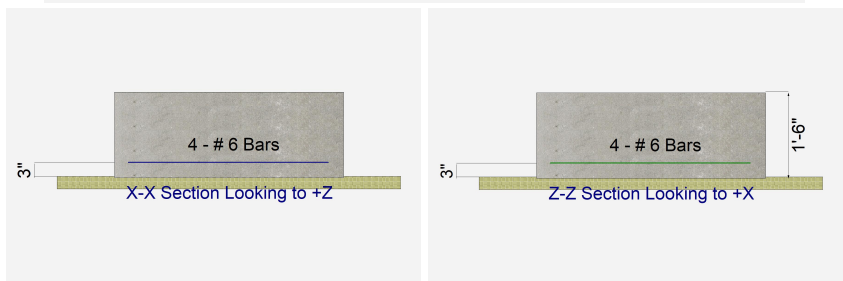
Reinforcing

Bars parallel to X-X Axis	=	4.0
Number of Bars	=	# 6
Reinforcing Bar Size	=	

Bars parallel to Z-Z Axis	=	4
Number of Bars	=	# 6
Reinforcing Bar Size	=	

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a



Applied Loads

	D	Lr	L	S	W	E	H
P : Column Load	=	6.30	0.0	5.60			k
OB : Overburden	=						ksf
M-xx	=						k-ft
M-zz	=						k-ft
V-x	=						k
V-z	=						k

General Footing

Project File: FS 46 enercalc.ec6

LIC#: KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: ISOLATE POST FOOTING

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.3204	Soil Bearing	0.9613 ksf	3.0 ksf	+D+L about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.07159	Z Flexure (+X)	2.065 k-ft/ft	28.846 k-ft/ft	+1.20D+1.60L
PASS	0.07159	Z Flexure (-X)	2.065 k-ft/ft	28.846 k-ft/ft	+1.20D+1.60L
PASS	0.07159	X Flexure (+Z)	2.065 k-ft/ft	28.846 k-ft/ft	+1.20D+1.60L
PASS	0.07159	X Flexure (-Z)	2.065 k-ft/ft	28.846 k-ft/ft	+1.20D+1.60L
PASS	0.05306	1-way Shear (+X)	4.359 psi	82.158 psi	+1.20D+1.60L
PASS	0.05306	1-way Shear (-X)	4.359 psi	82.158 psi	+1.20D+1.60L
PASS	0.05306	1-way Shear (+Z)	4.359 psi	82.158 psi	+1.20D+1.60L
PASS	0.05306	1-way Shear (-Z)	4.359 psi	82.158 psi	+1.20D+1.60L
PASS	0.1003	2-way Punching	16.476 psi	164.317 psi	+1.20D+1.60L

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc		Zecc		Actual Soil Bearing Stress @ Location				Actual / Allow Ratio
				(in)		Bottom, -Z	Top, +Z	Left, -X	Right, +X	
X-X, D Only	3.0	n/a	0.0			0.6113	0.6113	n/a	n/a	0.204
X-X, +D+L	3.0	n/a	0.0			0.9613	0.9613	n/a	n/a	0.320
X-X, +D+0.750L	3.0	n/a	0.0			0.8738	0.8738	n/a	n/a	0.291
X-X, +0.60D	3.0	n/a	0.0			0.3668	0.3668	n/a	n/a	0.122
Z-Z, D Only	3.0	0.0	n/a			n/a	n/a	0.6113	0.6113	0.204
Z-Z, +D+L	3.0	0.0	n/a			n/a	n/a	0.9613	0.9613	0.320
Z-Z, +D+0.750L	3.0	0.0	n/a			n/a	n/a	0.8738	0.8738	0.291
Z-Z, +0.60D	3.0	0.0	n/a			n/a	n/a	0.3668	0.3668	0.122

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	1.103	+Z	Bottom	0.3888	AsMin	0.440	28.846	OK
X-X, +1.40D	1.103	-Z	Bottom	0.3888	AsMin	0.440	28.846	OK
X-X, +1.20D+1.60L	2.065	+Z	Bottom	0.3888	AsMin	0.440	28.846	OK
X-X, +1.20D+1.60L	2.065	-Z	Bottom	0.3888	AsMin	0.440	28.846	OK
X-X, +1.20D+L	1.645	+Z	Bottom	0.3888	AsMin	0.440	28.846	OK
X-X, +1.20D+L	1.645	-Z	Bottom	0.3888	AsMin	0.440	28.846	OK
X-X, +1.20D	0.9450	+Z	Bottom	0.3888	AsMin	0.440	28.846	OK
X-X, +1.20D	0.9450	-Z	Bottom	0.3888	AsMin	0.440	28.846	OK
X-X, +0.90D	0.7088	+Z	Bottom	0.3888	AsMin	0.440	28.846	OK
X-X, +0.90D	0.7088	-Z	Bottom	0.3888	AsMin	0.440	28.846	OK
Z-Z, +1.40D	1.103	-X	Bottom	0.3888	AsMin	0.440	28.846	OK
Z-Z, +1.40D	1.103	+X	Bottom	0.3888	AsMin	0.440	28.846	OK
Z-Z, +1.20D+1.60L	2.065	-X	Bottom	0.3888	AsMin	0.440	28.846	OK
Z-Z, +1.20D+1.60L	2.065	+X	Bottom	0.3888	AsMin	0.440	28.846	OK
Z-Z, +1.20D+L	1.645	-X	Bottom	0.3888	AsMin	0.440	28.846	OK
Z-Z, +1.20D+L	1.645	+X	Bottom	0.3888	AsMin	0.440	28.846	OK
Z-Z, +1.20D	0.9450	-X	Bottom	0.3888	AsMin	0.440	28.846	OK

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: ISOLATE POST FOOTING

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.20D	0.9450	+X	Bottom	0.3888	AsMin	0.440	28.846	OK
Z-Z, +0.90D	0.7088	-X	Bottom	0.3888	AsMin	0.440	28.846	OK
Z-Z, +0.90D	0.7088	+X	Bottom	0.3888	AsMin	0.440	28.846	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	2.33 psi	2.33 psi	2.33 psi	2.33 psi	2.33 psi	82.16 psi	0.03	OK
+1.20D+1.60L	4.36 psi	4.36 psi	4.36 psi	4.36 psi	4.36 psi	82.16 psi	0.05	OK
+1.20D+L	3.47 psi	3.47 psi	3.47 psi	3.47 psi	3.47 psi	82.16 psi	0.04	OK
+1.20D	2.00 psi	2.00 psi	2.00 psi	2.00 psi	2.00 psi	82.16 psi	0.02	OK
+0.90D	1.50 psi	1.50 psi	1.50 psi	1.50 psi	1.50 psi	82.16 psi	0.02	OK

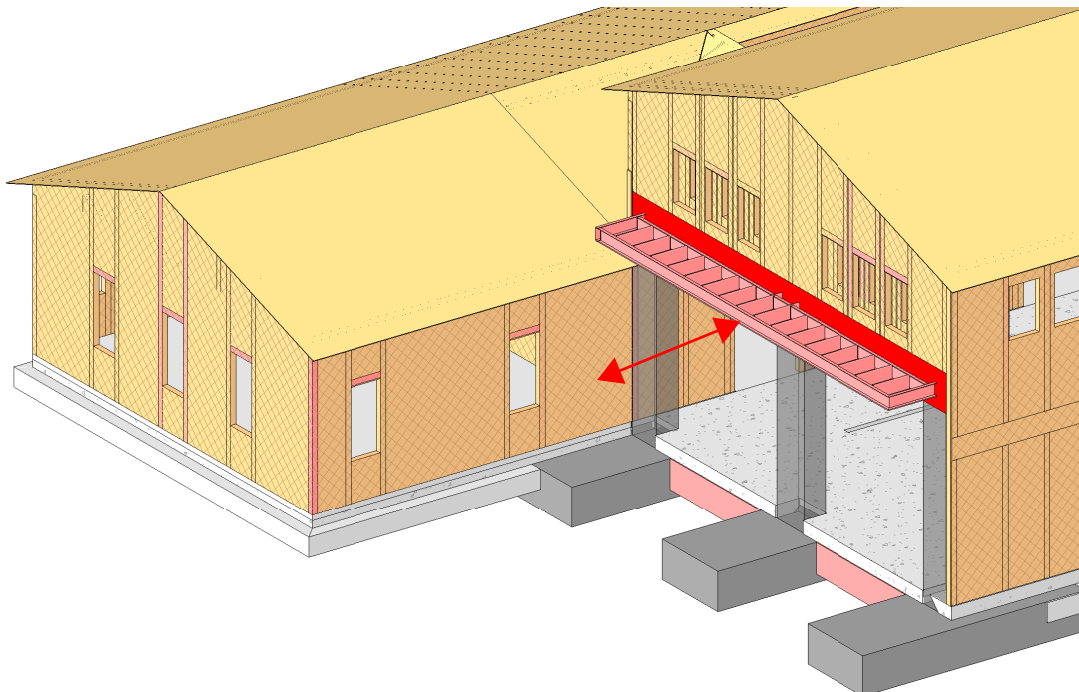
Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	8.80 psi	164.32psi	0.05353	OK
+1.20D+1.60L	16.48 psi	164.32psi	0.1003	OK
+1.20D+L	13.13 psi	164.32psi	0.07988	OK
+1.20D	7.54 psi	164.32psi	0.04589	OK
+0.90D	5.66 psi	164.32psi	0.03442	OK

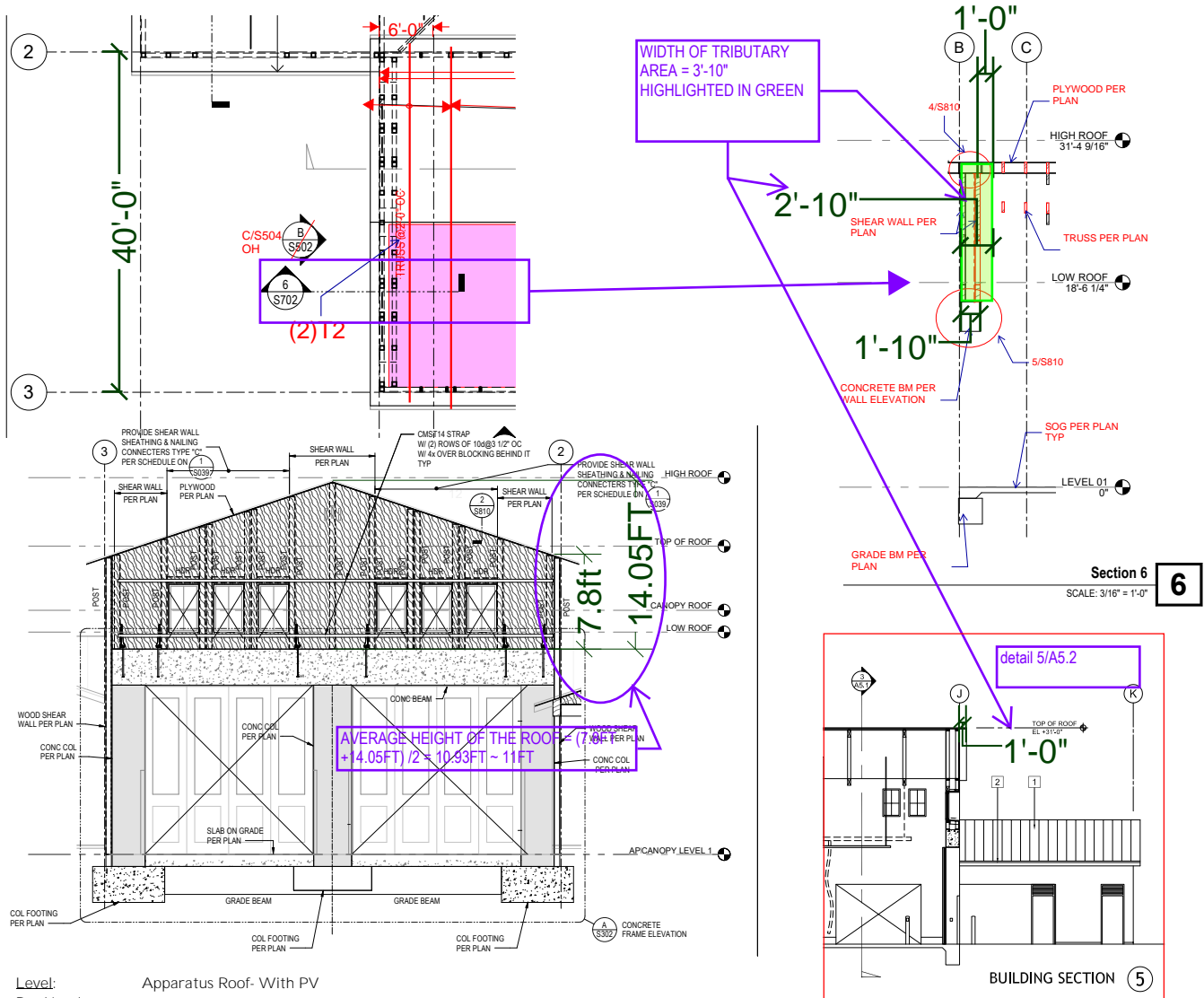
3.2 MOMENT FRAME FOOTING DESIGN

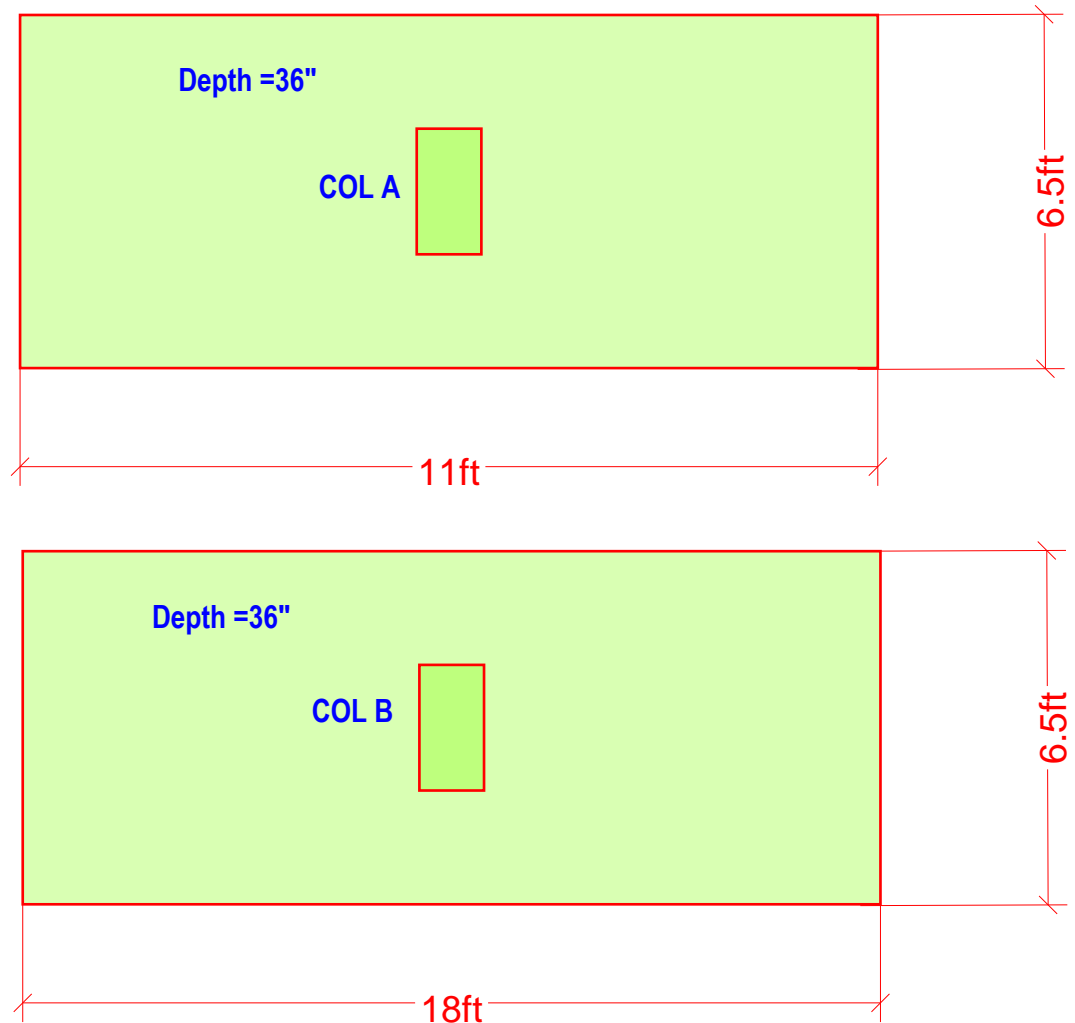
OUT OF PLANE DESIGN



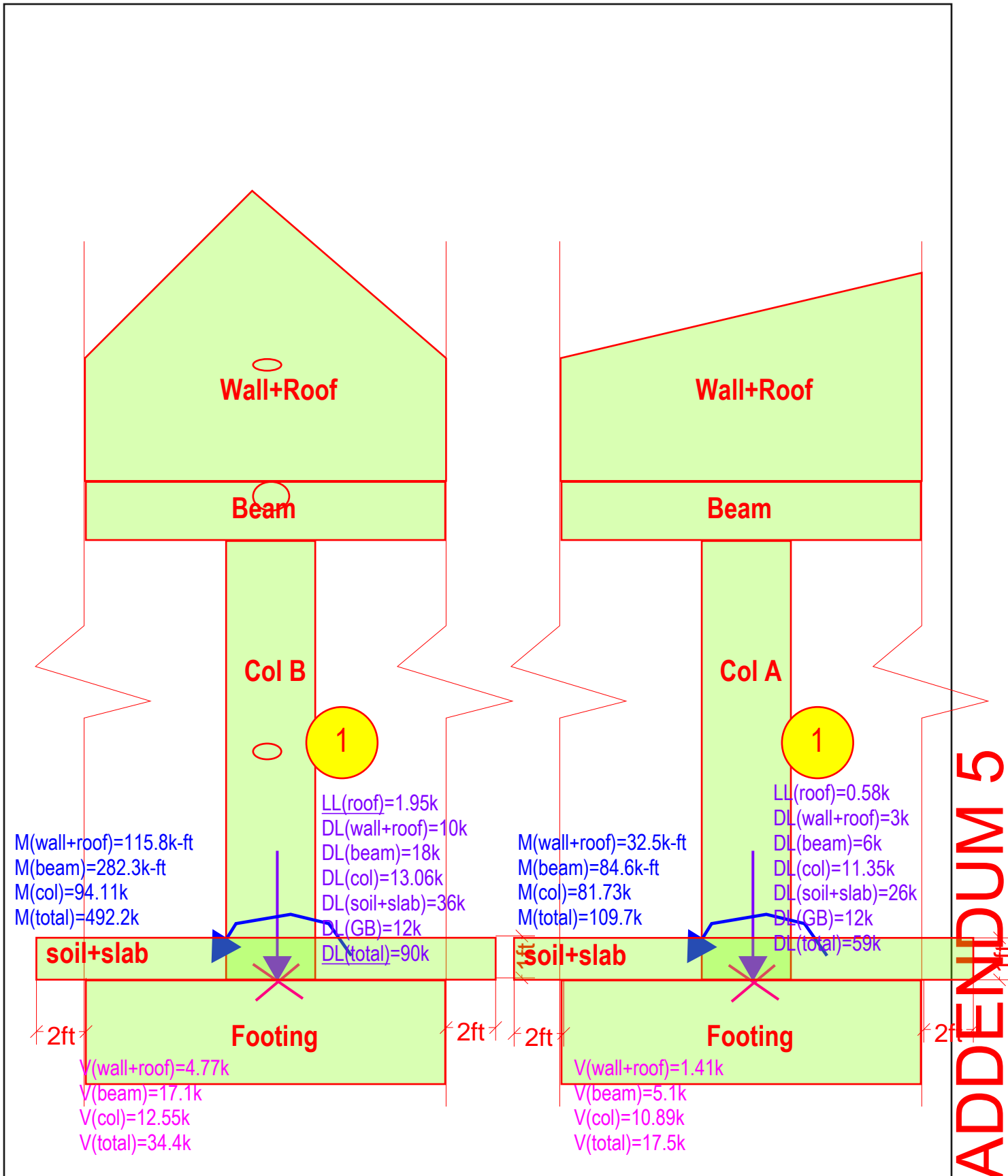
Columns & Footings are designed for OOP their self weights and half of wood wall self weights above.

Column & footing is designed as a cantilever members





SPREAD FOOTING SCHEDULE							
MARK	DEPTH (D)	SIZE (LxW)	REINFORCING				REMARKS
			BOTTOM		TOP		
			SHORT	LONG	SHORT	LONG	
F1	18"	3'-6"x3'-6"	(4)#6(B) EW	(4)#8(B) EW	-	-	
FF1	36"	18'-0"x6'-6"	#8@12"	#8@12"	#8@12"	#8@12"	
FF2	36"	11'-0"x6'-6"	#8@12"	#8@12"	#8@12"	#8@12"	



Interior

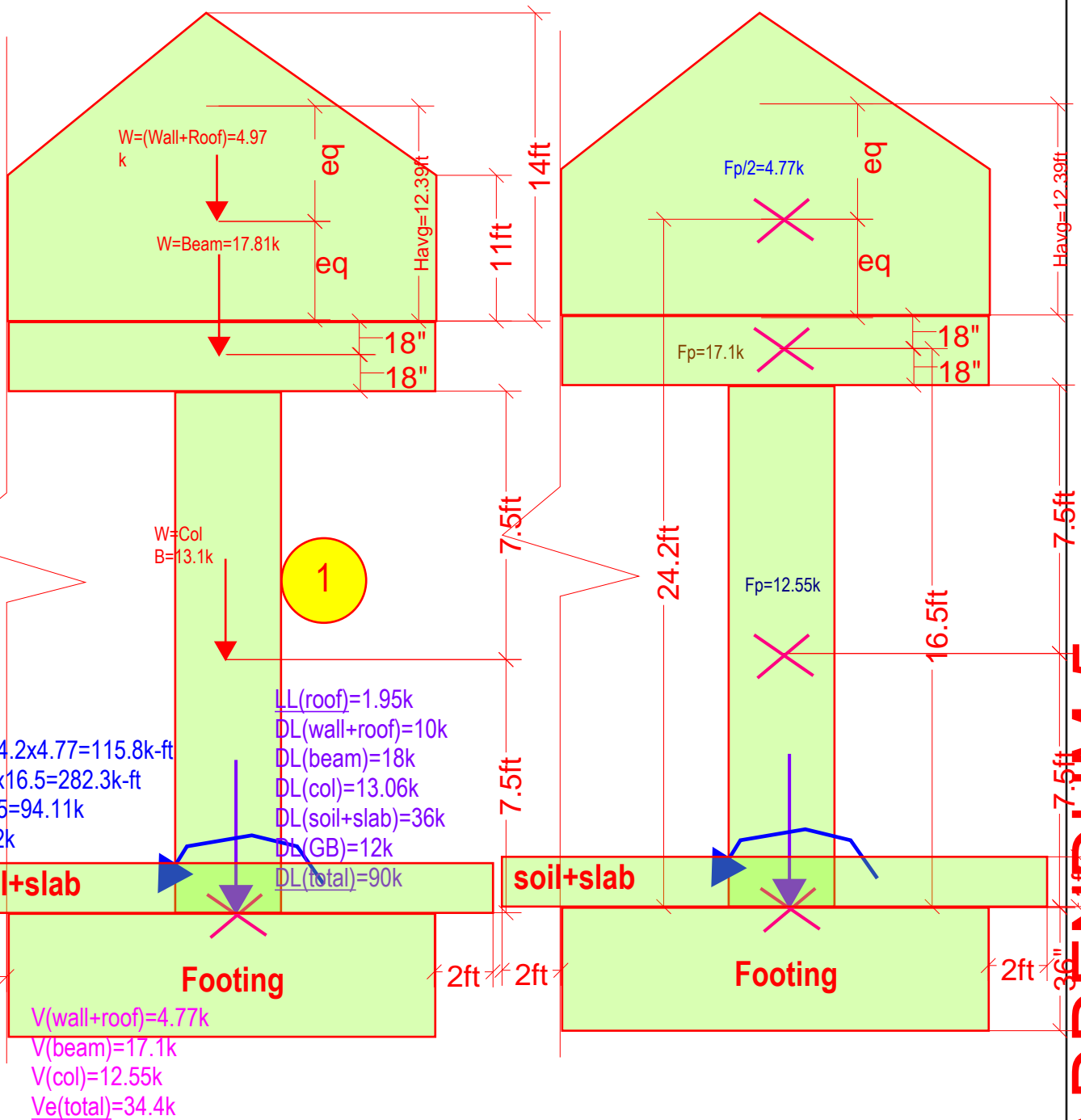
Fp=0.961W

All EQ loads Are strength Level

$$Sds := 1.601$$

$$Ie := 1.5$$

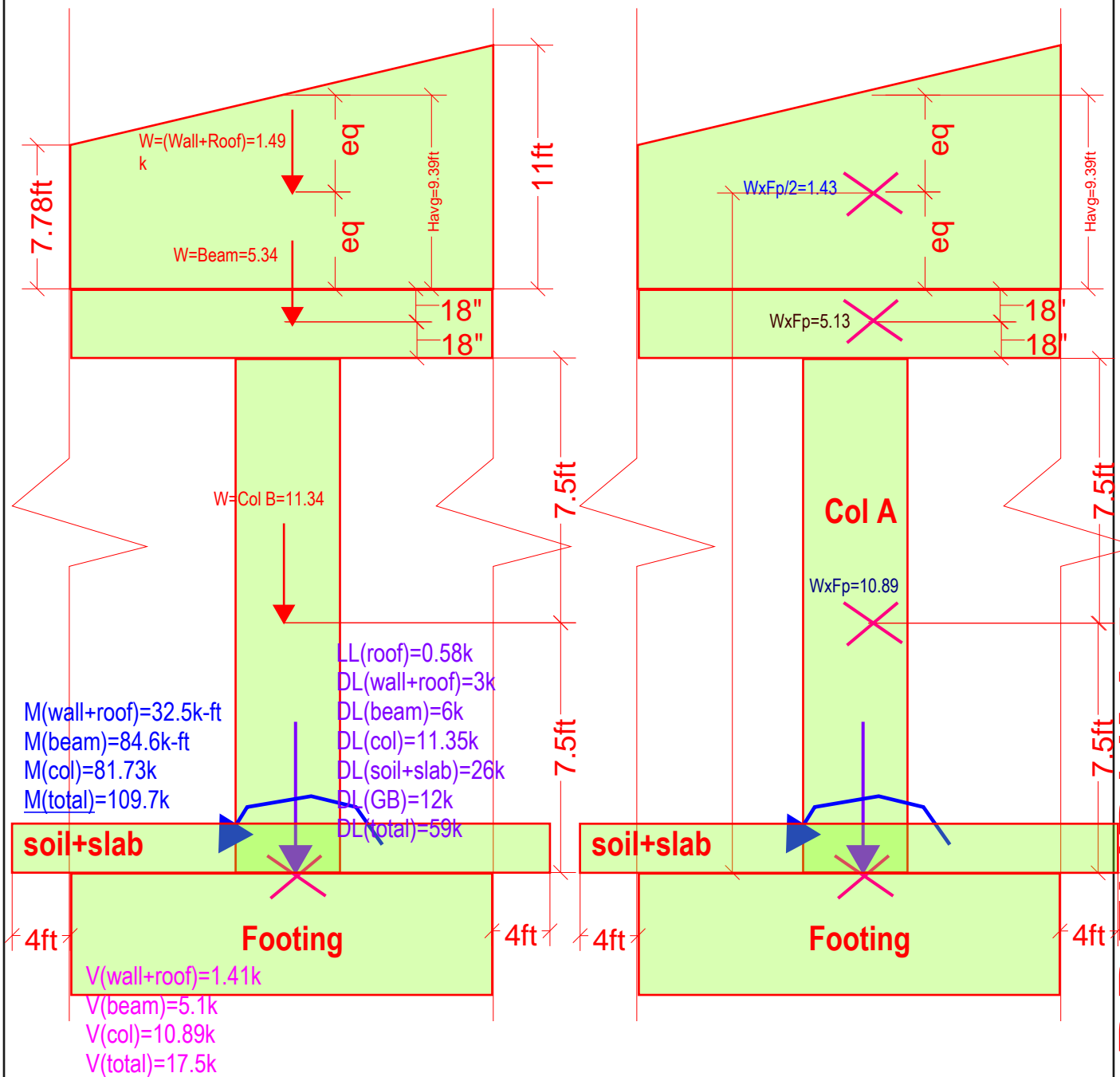
$$FpBeam := 0.4 Sds \cdot Ie = 0.961$$

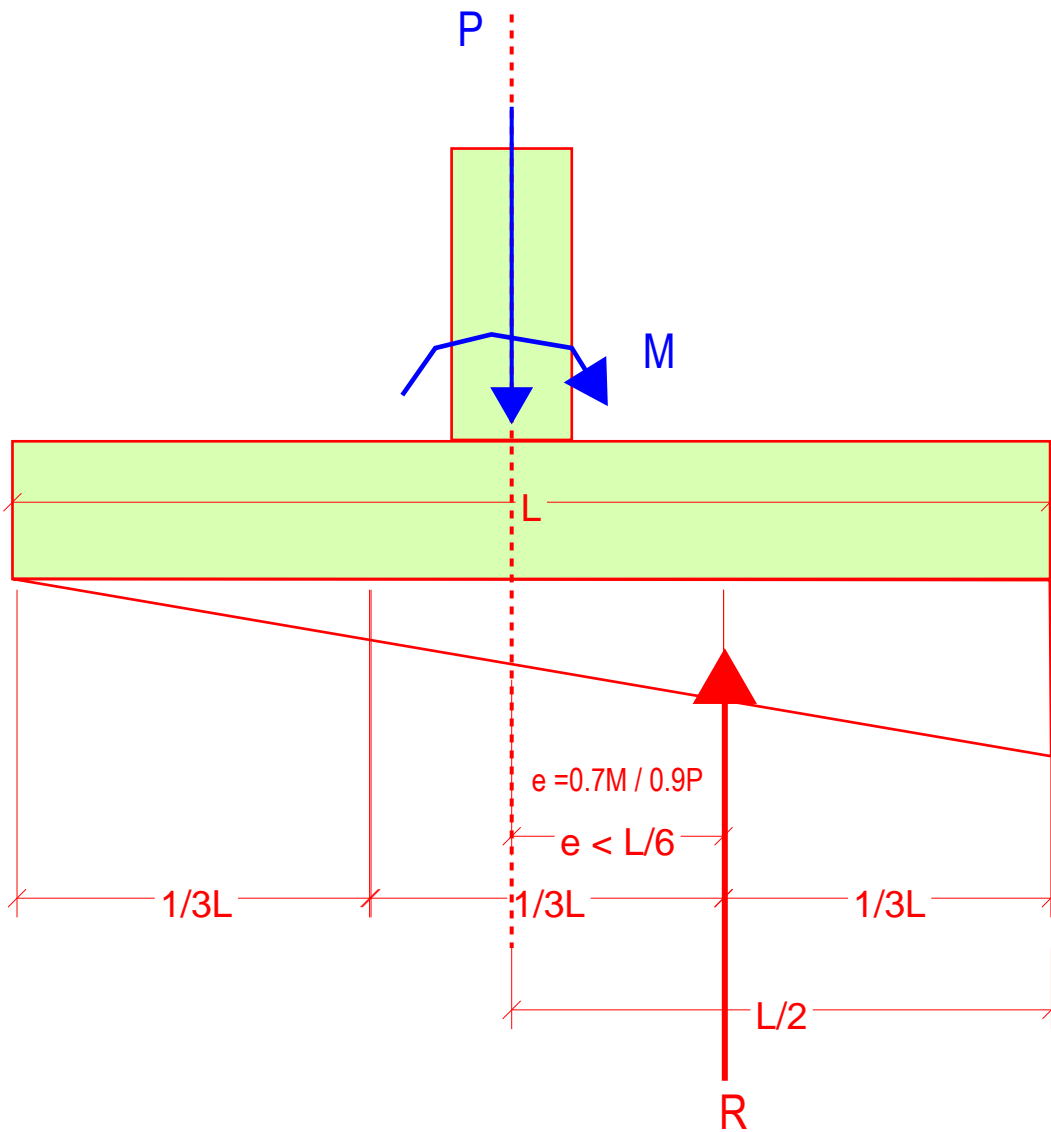


Exterior

$F_p=0.961W$

All EQ loads Are strength Level





COL A

$$e = 0.7M / 0.9D = 0.7 \times (109.7 + 17.5 \times 3) / 0.9 \times (59 + 3 \times 6.5 \times 11 \times 150 / 1000) = 1.3 < L/6 = 11/6 = 1.8 \quad \text{OK NO UPLIFT}$$

COL B

$$e = 0.7M / 0.9D = 0.7 \times (492.2 + 34.4 \times 3) / 0.9 \times (90 + 3 \times 6.5 \times 18 \times 150 / 1000) = 2.9 = L/6 = 18/6 = 3 \quad \text{OK NO UPLIFT}$$



Apparatus Roof- With PV

Item	Roof	Joint	Truss	Seismic
Color	4.0			4.0
Standing Seam (22 G)	4.0			4.0
1/4" Gypsum	1.0			1.0
6" Rigid Insulation	1.5			1.5
19/32" Plywood Diaphragm	2.0			2.0
Ceiling/MEP	5.0			5.0
Sprinkler	3.0			3.0
Misc.	5.0			3.0
Wood Truss @ 2'-0" o.c.		4.0		4.0
	27.5 psf	27.5 psf	31.5 psf	29.5 psf
				(Reducible)

Live Load Roof

20.0 psf

(Reducible)

#1

ROOF WEIGHT

$$Twidth := 3.83 \text{ ft}$$

Tributary width

$$Tlength := 2 \cdot 20.96 \text{ ft} = 41.92 \text{ ft}$$

Tributary length

$$Tarea := Twidth \cdot Tlength = 160.554 \text{ ft}^2 \quad \text{Tributary area}$$

$$RoofDL := 29.5 \cdot \text{psf}$$

$$RoofLL := 20 \text{ psf}$$

$$RoofW := RoofDL \cdot Tarea = 4736.331 \text{ lbf}$$

$$RoofLiveLoad := Tarea \cdot RoofLL = 3211.072 \text{ lbf} \quad \text{Total Roof Live Load}$$

Distributed Roof Live Load

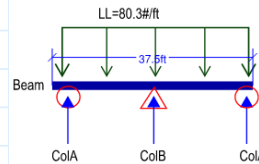
$$RoofDistLiveLoad := \frac{RoofLiveLoad}{40 \text{ ft}} = 80.277 \frac{\text{lbf}}{\text{ft}}$$

$$RoofLiveLoadColA := 0.58 \text{ kip}$$

$$RoofLiveLoadColB := 1.95 \text{ kip}$$

Extreme Reactions (service, kips)

	D
ColA Support #1	0.58
ColB Support #2	1.95
Support #3	0.58



WALL Self Weight

$$WallDL := 30 \text{ psf}$$

$$WallArea := 407.11 \text{ ft}^2$$

$$WallWeightTotal := WallDL \cdot WallArea = 12213.3 \text{ lbf}$$

Total Weight of the wall

$$WallDistWeight := \frac{WallWeightTotal}{40 \text{ ft}} = 305.333 \frac{\text{lbf}}{\text{ft}}$$

WALL + Roof Weight on the Column

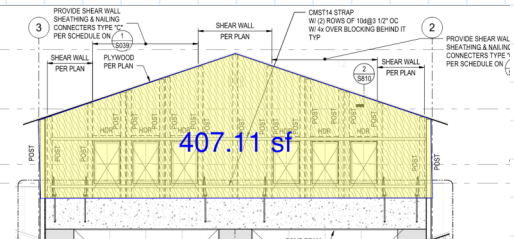
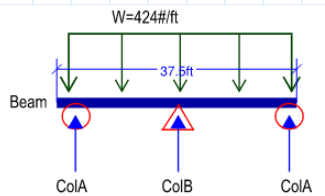
$$TotalDistDeadLoad := \frac{RoofW + WallWeightTotal}{40 \text{ ft}} = 423.741 \frac{\text{lbf}}{\text{ft}}$$

$$RoofandWallWeightColA := 2.98 \text{ kip}$$

$$RoofandWallWeightColB := 9.94 \text{ kip}$$

Extreme Reactions (service, kip)

	D
ColA Support #1	2.98
ColB Support #2	9.94
Support #3	2.98



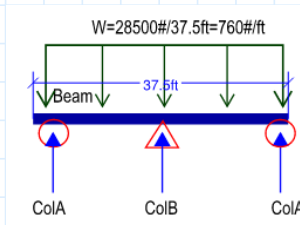
#1

Concrete Beam Self Weight

$$\text{BeamWeight} := 28500 \text{ lbf}$$

$$\text{BeamWeightColA} := 5.34 \text{ kip}$$

$$\text{BeamWeightColB} := 17.81 \text{ kip}$$



Concrete Beam self Weight

Extreme Reactions (service, kips)

	D	Lr
ColASupport #1	5.34	
ColBSupport #2	17.81	
Support #3	5.34	

#1

Concrete Column Self Weight

$$\text{ColHeight} := 15 \text{ ft}$$

$$\text{ColAarea} := 22 \text{ in} \cdot 33 \text{ in} = 5.042 \text{ ft}^2$$

$$\text{ColBarea} := 22 \text{ in} \cdot 38 \text{ in} = 5.806 \text{ ft}^2$$

$$\text{ConcDensity} := 150 \text{ pcf}$$

$$\text{ColA} := \text{ColAarea} \cdot \text{ConcDensity} = 756.25 \frac{\text{lbf}}{\text{ft}}$$

$$\text{ColB} := \text{ColBarea} \cdot \text{ConcDensity} = 870.833 \frac{\text{lbf}}{\text{ft}}$$

$$\text{ColAWeight} := \text{ColA} \cdot \text{ColHeight} = 11343.75 \text{ lbf}$$

$$\text{ColBWeight} := \text{ColB} \cdot \text{ColHeight} = 13062.5 \text{ lbf}$$

Concrete Col A height

Concrete Col A section area

Concrete Col B section area

Concrete Col A Weight/ft

Concrete Col B Weight/ft

Concrete Col A Weight

Concrete Col B Weight

Dead Load from Soil + Slab on the footing

$$\text{WeightofSoilColA} := ((6.5 \text{ ft} + 8 \text{ ft}) \cdot (11 \text{ ft} + 8 \text{ ft}) \text{ ConcDensity} \cdot 6 \text{ in}) + (6.5 \text{ ft} \cdot 11 \text{ ft}) \left(125 \frac{\text{lbf}}{\text{ft}^3} \cdot 6 \text{ in} \right) = 25131.25 \text{ lbf}$$

$$\text{WeightofGradeBeamColA} := \text{ConcDensity} \cdot \left(\frac{34 \text{ ft}}{2} - 6.5 \text{ ft} \right) \cdot (28 \text{ in} + 12 \text{ in}) \cdot 26 \text{ in} = 11375 \text{ lbf}$$

$$\text{WeightofSoilColB} := ((6.5 \text{ ft} + 8 \text{ ft}) \cdot (18 \text{ ft} + 8 \text{ ft}) \text{ ConcDensity} \cdot 6 \text{ in}) + (6.5 \text{ ft} \cdot 18 \text{ ft}) \left(125 \frac{\text{lbf}}{\text{ft}^3} \cdot 6 \text{ in} \right) = 35587.5 \text{ lbf}$$

$$\text{WeightofGradeBeamColB} := \text{ConcDensity} \cdot \left(\frac{34 \text{ ft}}{2} - 6.5 \text{ ft} \right) \cdot (28 \text{ in} + 12 \text{ in}) \cdot 26 \text{ in} = 11375 \text{ lbf}$$

Total Dead Load at the base of the Column

$$\text{TotalDadLoadColA} := \text{ColAWeight} + \text{BeamWeightColA} + \text{RoofandWallWeightColA} + \text{WeightofSoilColA} + \text{WeightofGradeBeamColA} = 56170 \text{ lbf}$$

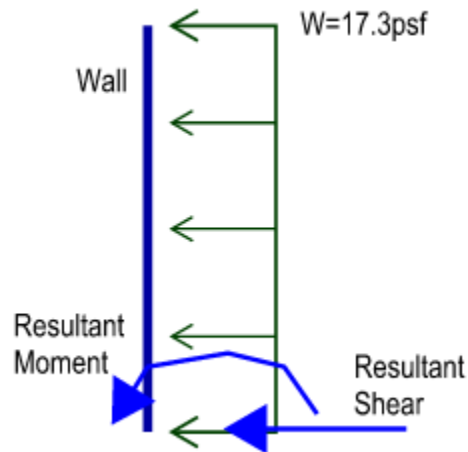
$$\text{TotalDadLoadColB} := \text{ColBWeight} + \text{BeamWeightColB} + \text{RoofandWallWeightColB} + \text{WeightofGradeBeamColB} + \text{WeightofSoilColB} = 87775 \text{ lbf}$$

Total Live Load at the base of the Column

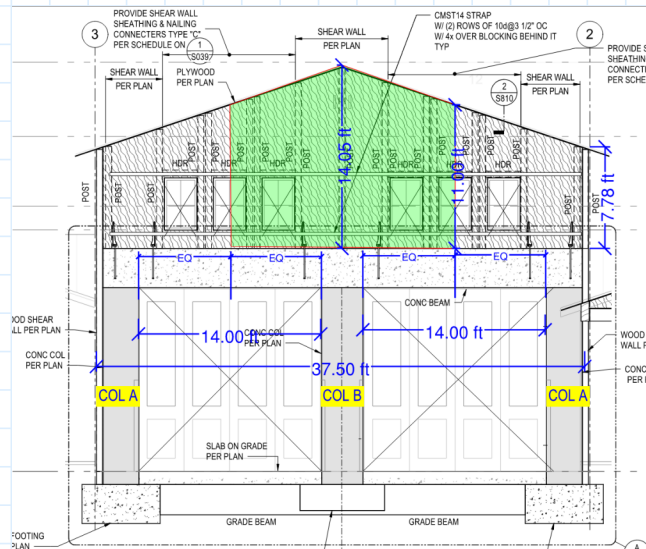
$$\text{TotalLiveLoadColA} := \text{RoofLiveLoadColA} = 580 \text{ lbf}$$

$$\text{TotalLiveLoadColB} := \text{RoofLiveLoadColB} = 1950 \text{ lbf}$$

Out of Plane Wind Load on the wall



	EFFECTIVE AREA	GENERAL (ZONE 4)	CORNER (ZONE 5)
WALL SURFACE	50 SF 100 SF	15.6 PSF 18.4 PSF	17.3 PSF 16.0 PSF



$$W_{\text{pressure}} := 17.3 \text{ psf}$$

$$\text{WallAH}_{\text{avg}} := \frac{11 \text{ ft} + 7.78 \text{ ft}}{2} = 9.39 \text{ ft}$$

$$\text{WallBH}_{\text{avg}} := \frac{11 \text{ ft} + 14.05 \text{ ft}}{2} = 12.525 \text{ ft}$$

$$\text{TribWidthColA} := \frac{14 \text{ ft} + 0 \text{ ft}}{2} = 7 \text{ ft}$$

$$\text{TribWidthColB} := \frac{14 \text{ ft} + 14 \text{ ft}}{2} = 14 \text{ ft}$$

Average Tributary height for column A

Average Tributary height for column B

Tributary width of column A

Tributary width of column B

$$\text{WallWindColA} := W_{\text{pressure}} \cdot \text{TribWidthColA} = 121.1 \frac{\text{lbf}}{\text{ft}}$$

distributed Out of plane Wind load/ft on wall for col A

$$\text{WallWindColB} := W_{\text{pressure}} \cdot \text{TribWidthColB} = 242.2 \frac{\text{lbf}}{\text{ft}}$$

distributed out of plane Wind load/ft on wall for col B

$$\text{WallWindShearColA} := \text{WallWindColA} \cdot \text{WallAH}_{\text{avg}} = 1137.129 \text{ lbf}$$

Resultant Out of plane Wind load/ft from wall for col A

$$\text{WallWindShearColB} := \text{WallWindColB} \cdot \text{WallBH}_{\text{avg}} = 3033.555 \text{ lbf}$$

Resultant Out of plane Wind load/ft from wall for col B

$$\text{WallWindMomentColA} := \text{WallWindColA} \cdot \text{WallAH}_{\text{avg}} \cdot \frac{\text{WallAH}_{\text{avg}}}{2} + \text{WallWindShearColA} \cdot \text{ColHeight} = 22395.756 \text{ lbf} \cdot \text{ft}$$

Resultant moment from wall wind at bttm of Col A

$$\text{WallWindMomentColB} := \text{WallWindColB} \cdot \text{WallBH}_{\text{avg}} \cdot \frac{\text{WallBH}_{\text{avg}}}{2} + \text{WallWindShearColB} \cdot \text{ColHeight} = 64500.963 \text{ lbf} \cdot \text{ft}$$

Resultant moment from

Out of Plane Wind Load on the Beam

$BeamDepth := 36 \text{ in}$

$BeamDistWind := W_{pressure} \cdot BeamDepth = 51.9 \frac{lb}{ft}$

distributed Out of plane Wind
load/ft on the beam

$ColAReaction := 0.37 \text{ kip}$

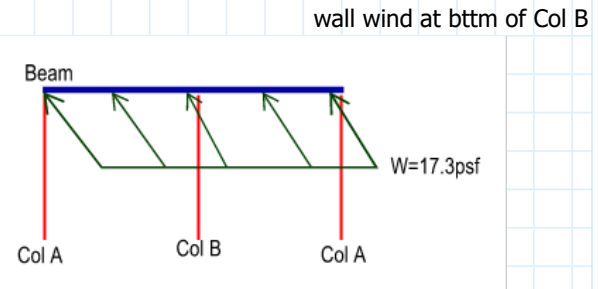
$ColBReaction := 1.22 \text{ kip}$

$BeamWindShearColA := ColAReaction$

$BeamWindShearColB := ColBReaction$

$BeamWindMomentColA := ColAReaction \cdot ColHeight = 5550 \text{ lb} \cdot \text{ft}$

$BeamWindMomentColB := ColBReaction \cdot ColHeight = 18300 \text{ lb} \cdot \text{ft}$



Beam Out of plane Wind load
moment at the bttm of column A

Beam Out of plane Wind load
moment at the bttm of column B

Out of Plane Wind Load on the Column

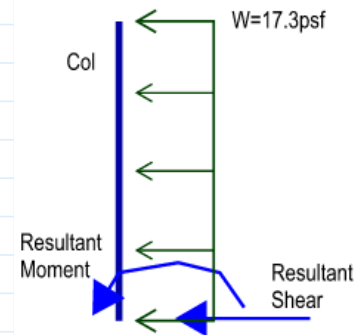
$$ColAwidth := 33 \text{ in}$$

$$TribWidthColA = 7 \text{ ft}$$

$$ColBwidth := 38 \text{ in}$$

$$TribWidthColB = 14 \text{ ft}$$

$$ColHeight := 15 \text{ ft}$$



$$ColADistWind := W_{pressure} \cdot TribWidthColA = 121.1 \frac{\text{lb}}{\text{ft}}$$

distributed out of plane Wind load/ft col A

$$ColBDistWind := W_{pressure} \cdot TribWidthColB = 242.2 \frac{\text{lb}}{\text{ft}}$$

distributed out of plane Wind load/ft col B

$$ColAWindShear := ColADistWind \cdot ColHeight = 1816.5 \text{ lb}$$

Resultant Out of plane Wind load/ft from wall for col A

$$ColBWindShear := ColBDistWind \cdot ColHeight = 3633 \text{ lb}$$

Resultant Out of plane Wind load/ft from wall for col A

$$ColAWindMoment := ColAWindShear \cdot \frac{ColHeight}{2} = 13623.75 \text{ lb} \cdot \text{ft}$$

Resultant moment from wind at bttm of Col A

$$ColBWindMoment := ColBWindShear \cdot \frac{ColHeight}{2} = 27247.5 \text{ lb} \cdot \text{ft}$$

Resultant moment from wind at bttm of Col B

Total Out of Plane Wind Load @ the base of Columns

$$TotalWindShearColA := ColAWindShear + WallWindShearColA + BeamWindShearColA = 3323.629 \text{ lb}$$

$$TotalWindShearColB := ColBWindShear + WallWindShearColB + BeamWindShearColB = 7886.555 \text{ lb}$$

$$TotalWindMomentColA := ColAWindMoment + WallWindMomentColA + BeamWindMomentColA = 41569.506 \text{ lb} \cdot \text{ft}$$

$$TotalWindMomentColB := ColBWindMoment + WallWindMomentColB + BeamWindMomentColB = 110048.463 \text{ lb} \cdot \text{ft}$$

Out of Plane Seismic Load Calculation of Wall&Roof

12.11 STRUCTURAL WALLS AND THEIR ANCHORAGE

12.11.1 Design for Out-of-Plane Forces. Structural walls shall be designed for a force normal to the surface equal to $F_p = 0.4 S_{DS} I_e$ times the weight of the structural wall with a minimum force of 10% of the weight of the structural wall.

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

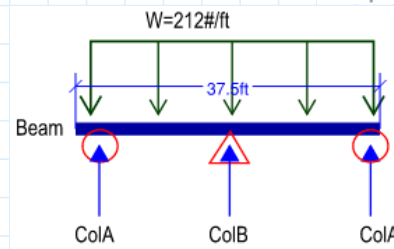
Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p , applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

$$TotalDistDeadLoad := \frac{0.5 \cdot RoofW + 0.5 \cdot WallWeightTotal}{40 \text{ ft}} = 211.87 \frac{\text{lb}}{\text{ft}}$$

$$RoofandWallWeightColA := 1.49 \text{ kip}$$

$$RoofandWallWeightColB := 4.97 \text{ kip}$$



Extreme Reactions (service, kips)

	D
ColA _{Support #1}	1.49
ColB _{Support #2}	4.97
Support #3	1.49

$$S_{DS} := 1.601$$

$$I_e := 1.5$$

out of plane seismic coefficient of wall

$$F_p := 0.4 S_{DS} \cdot I_e = 0.961$$

$$F_p WallColA := F_p \cdot RoofandWallWeightColA = 1431.294 \text{ lb}$$

Total out of plane seismic load of wall and roof for ColA

$$F_p WallColB := F_p \cdot RoofandWallWeightColB = 4774.182 \text{ lb}$$

Total out of plane seismic load of wall and roof for ColB

$$WallSeismicShearColA := F_p WallColA = 1431.294 \text{ lb}$$

Resultant out of plane seismic Shear from wall at bttm of Col A

$$WallSeismicShearColB := F_p WallColB = 4774.182 \text{ lb}$$

Resultant out of plane seismic Shear from wall at bttm of Col B

$$WallSeismicMomentColA := F_p WallColA \cdot \left(ColHeight + BeamDepth + \frac{WallAH_{avg}}{2} \right) = 32483.217 \text{ lb} \cdot \text{ft}$$

Resultant out of plane seismic moment from wall at bttm of Col A

$$WallSeismicMomentColB := F_p WallColB \cdot \left(ColHeight + BeamDepth + \frac{WallBH_{avg}}{2} \right) = 115833.591 \text{ lb} \cdot \text{ft}$$

Resultant out of plane seismic moment from wall at bttm of Col B

Out of Plane Seismic Load Calculation of Beam

$$BeamWeight = 28500 \text{ lbf}$$

$$BeamWeight := 28500 \text{ lbf}$$

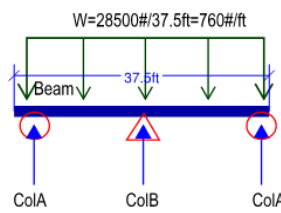
$$BeamWeightColA := 5.34 \text{ kip}$$

$$BeamWeightColB := 17.81 \text{ kip}$$

$$Sds := 1.601$$

$$I_e := 1.5$$

$$FpBeam := 0.4 Sds \cdot I_e = 0.961$$



Extreme Reactions (service, kips)

	D	Lr
ColASupport #1	5.34	
ColBSupport #2	17.81	
Support #3	5.34	

Total out of plane seismic load of wall

$$BeamSeismicShearColA := BeamWeightColA \cdot FpBeam = 5129.604 \text{ lbf}$$

Resultant out of plane seismic Shear from Beam at bttm of Col A

$$BeamSeismicShearColB := BeamWeightColB \cdot FpBeam = 17108.286 \text{ lbf}$$

Resultant out of plane seismic Shear from Beam at bttm of Col B

$$BeamSeismicMomentColA := BeamWeightColA \cdot FpBeam \cdot \left(ColHeight + \frac{BeamDepth}{2} \right) = 84638.466 \text{ lbf} \cdot \text{ft}$$

Resultant out of plane seismic moment from Beam at bttm of Col A

$$BeamSeismicMomentColB := BeamWeightColB \cdot FpBeam \cdot \left(ColHeight + \frac{BeamDepth}{2} \right) = 282286.719 \text{ lbf} \cdot \text{ft}$$

Resultant out of plane seismic moment from Beam at bttm of Col B

12.11 STRUCTURAL WALLS AND THEIR ANCHORAGE

12.11.1 Design for Out-of-Plane Forces. Structural walls shall be designed for a force normal to the surface equal to $F_p = 0.4 S_{DS} I_e$ times the weight of the structural wall with a minimum force of 10% of the weight of the structural wall.

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p , applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

Out of Plane Seismic Load Calculation of Column

$$ColHeight = 15 \text{ ft}$$

$$Sds := 1.601$$

$$I_e := 1.5$$

$$ColAWeight = 11343.75 \text{ lbf}$$

$$ColBWeight = 13062.5 \text{ lbf}$$

$$FpColA := 0.4 Sds \cdot I_e \cdot ColAWeight = 10896.806 \text{ lbf}$$

$$FpColB := 0.4 Sds \cdot I_e \cdot ColBWeight = 12547.838 \text{ lbf}$$

$$ColASeismicShear := FpColA = 10896.806 \text{ lbf}$$

$$ColBSeismicShear := FpColB = 12547.838 \text{ lbf}$$

$$ColASeismicMoment := FpColA \cdot \frac{ColHeight}{2} = 81726.047 \text{ lbf} \cdot \text{ft}$$

$$ColBSeismicMoment := FpColB \cdot \frac{ColHeight}{2} = 94108.781 \text{ lbf} \cdot \text{ft}$$

Total Out of Plane Seismic Load @ the base of Columns

$$TotalSeismicShearColA := ColASeismicShear + WallSeismicShearColA + BeamSeismicShearColA = 17457.704 \text{ lbf}$$

$$TotalWindShearColB := ColBSeismicShear + WallSeismicShearColB + BeamSeismicShearColB = 34430.306 \text{ lbf}$$

$$TotalWindMomentColA := ColASeismicMoment + WallWindMomentColA + BeamWindMomentColA = 109671.803 \text{ lbf} \cdot \text{ft}$$

$$TotalWindMomentColB := ColBSeismicMoment + WallSeismicMomentColB + BeamSeismicMomentColB = 492229.091 \text{ lbf} \cdot \text{ft}$$

Comparing the Wind Load vs Seismic, Seismic Load is Governing and will be used for design of Footings

12.11 STRUCTURAL WALLS AND THEIR ANCHORAGE

12.11.1 Design for Out-of-Plane Forces. Structural walls shall be designed for a force normal to the surface equal to $F_p = 0.4 S_{DS} I_e$ times the weight of the structural wall with a minimum force of 10% of the weight of the structural wall.

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p , applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

Total out of plane seismic load of ColA

Total out of plane seismic load of ColB

Resultant out of plane seismic Shear from Col at bttm of Col A

Resultant out of plane seismic Shear from Col at bttm of Col B

Resultant out of plane seismic moment from Col at bttm of Col A

Resultant out of plane seismic moment from Col at bttm of Col B

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: frame footing.ec6

LIC# : KW-06016422, Build:20.23.08.01

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2023

DESCRIPTION: Footing under Column A

Code References

Calculations per ACI 318-19, IBC 2021, ASCE 7-16

Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	5.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	150.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	3.990 ksf
Soil Density	=	125.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	465.50 pcf
Soil/Concrete Friction Coeff.	=	0.4650

Increases based on footing Depth

Footing base depth below soil surface	=	1.50 ft
Allow press. increase per foot of depth when footing base is below	=	ksf ft

Increases based on footing plan dimension

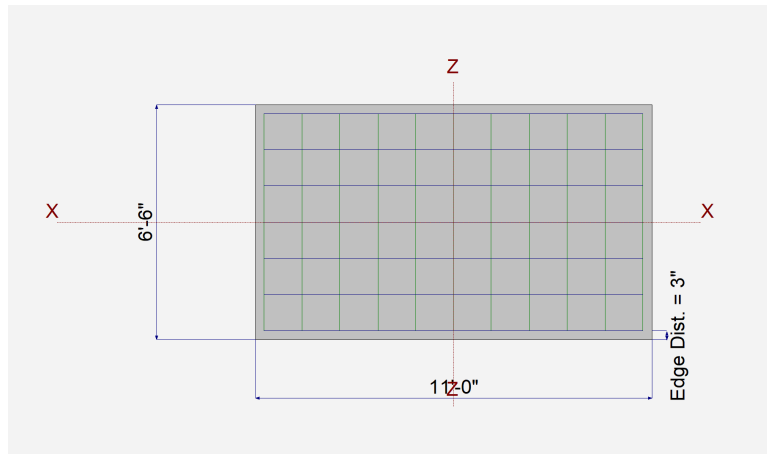
Allowable pressure increase per foot of depth when max. length or width is greater than	=	ksf ft
---	---	--------

Dimensions

Width parallel to X-X Axis	=	11.0 ft
Length parallel to Z-Z Axis	=	6.50 ft
Footing Thickness	=	36.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	in
pz : parallel to Z-Z Axis	=	in
Height	=	in
Rebar Centerline to Edge of Concrete... at Bottom of footing	=	3.0 in



Reinforcing

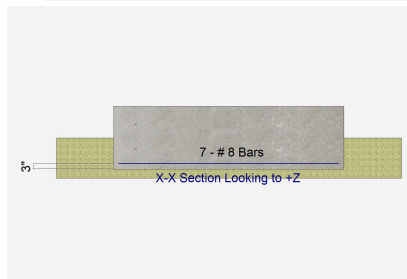
Bars parallel to X-X Axis	=	
Number of Bars	=	7.0
Reinforcing Bar Size	=	# 8
Bars parallel to Z-Z Axis	=	
Number of Bars	=	11.0
Reinforcing Bar Size	=	# 8

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation

Bars along Z-Z Axis

# Bars required within zone	74.3 %
# Bars required on each side of zone	25.7 %



Applied Loads

	D	Lr	L	S	W	E	H	
P : Column Load	=	61.0	0.580					k
OB : Overburden	=							ksf
M-xx	=							k-ft
M-zz	=				61.970	109.70		k-ft
V-x	=				4.10	17.50		k
V-z	=							k

General Footing

Project File: frame footing.ec6

LIC#: KW-06016422, Build:20.23.08.01

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2023

DESCRIPTION: Footing under Column A

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.5416	Soil Bearing	2.161 ksf	3.990 ksf	+D+0.70E about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	4.062	Overturning - Z-Z	113.540 k-ft	461.216 k-ft	+0.90D+0.70E
PASS	3.461	Sliding - X-X	12.250 k	42.398 k	+0.90D+0.70E
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.2579	Z Flexure (+X)	32.092 k-ft/ft	124.423 k-ft/ft	+1.520D+E
PASS	0.1452	Z Flexure (-X)	18.065 k-ft/ft	124.423 k-ft/ft	+1.40D
PASS	0.05922	X Flexure (+Z)	6.850 k-ft/ft	115.663 k-ft/ft	+1.520D+E
PASS	0.05922	X Flexure (-Z)	6.850 k-ft/ft	115.663 k-ft/ft	+1.520D+E
PASS	0.1457	1-way Shear (+X)	15.451 psi	106.066 psi	+1.520D+E
PASS	0.07820	1-way Shear (-X)	8.294 psi	106.066 psi	+1.40D
PASS	0.01606	1-way Shear (+Z)	1.703 psi	106.066 psi	+1.520D+E
PASS	0.01606	1-way Shear (-Z)	1.703 psi	106.066 psi	+1.520D+E
PASS	0.08940	2-way Punching	18.964 psi	212.132 psi	+1.520D+E



Top reinforcing mat required (see 'Bending' tab).

Hand check required for anchor pullout.

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Zecc (in)	Actual Soil Bearing Stress @ Location				Actual / Allow Ratio
				Bottom, -Z	Top, +Z	Left, -X	Right, +X	
X-X, D Only	3.990	n/a	0.0	1.303	1.303	n/a	n/a	0.327
X-X, +D+Lr	3.990	n/a	0.0	1.311	1.311	n/a	n/a	0.329
X-X, +D+0.750Lr	3.990	n/a	0.0	1.309	1.309	n/a	n/a	0.328
X-X, +D+0.60W	3.990	n/a	0.0	1.303	1.303	n/a	n/a	0.327
X-X, +D+0.70E	3.990	n/a	0.0	1.303	1.303	n/a	n/a	0.327
X-X, +D+0.750Lr+0.450W	3.990	n/a	0.0	1.309	1.309	n/a	n/a	0.328
X-X, +D+0.450W	3.990	n/a	0.0	1.303	1.303	n/a	n/a	0.327
X-X, +D+0.5250E	3.990	n/a	0.0	1.303	1.303	n/a	n/a	0.327
X-X, +0.90D+0.60W	3.990	n/a	0.0	1.173	1.173	n/a	n/a	0.294
X-X, +0.90D+0.70E	3.990	n/a	0.0	1.173	1.173	n/a	n/a	0.294
Z-Z, D Only	3.990	0.0	n/a	n/a	n/a	1.303	1.303	0.327
Z-Z, +D+Lr	3.990	0.0	n/a	n/a	n/a	1.311	1.311	0.329
Z-Z, +D+0.750Lr	3.990	0.0	n/a	n/a	n/a	1.309	1.309	0.328
Z-Z, +D+0.60W	3.990	5.739	n/a	n/a	n/a	0.9666	1.640	0.411
Z-Z, +D+0.70E	3.990	14.623	n/a	n/a	n/a	0.4456	2.161	0.542
Z-Z, +D+0.750Lr+0.450W	3.990	4.284	n/a	n/a	n/a	1.057	1.562	0.392
Z-Z, +D+0.450W	3.990	4.304	n/a	n/a	n/a	1.051	1.556	0.390
Z-Z, +D+0.5250E	3.990	10.967	n/a	n/a	n/a	0.660	1.946	0.488
Z-Z, +0.90D+0.60W	3.990	6.377	n/a	n/a	n/a	0.8363	1.509	0.378
Z-Z, +0.90D+0.70E	3.990	16.248	n/a	n/a	n/a	0.3153	2.030	0.509

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D Only	None	0.0 k-ft	Infinity	OK
X-X, +D+Lr	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr	None	0.0 k-ft	Infinity	OK
X-X, +D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.70E	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.5250E	None	0.0 k-ft	Infinity	OK
X-X, +0.90D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +0.90D+0.70E	None	0.0 k-ft	Infinity	OK
Z-Z, D Only	None	0.0 k-ft	Infinity	OK
Z-Z, +D+Lr	None	0.0 k-ft	Infinity	OK

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: frame footing.ec6

LIC#: KW-06016422, Build:20.23.08.01

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2023

DESCRIPTION: Footing under Column A

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Z-Z, +D+0.750Lr	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.60W	44.562 k-ft	512.46 k-ft	11.50	OK
Z-Z, +D+0.70E	113.540 k-ft	512.46 k-ft	4.514	OK
Z-Z, +D+0.750Lr+0.450W	33.422 k-ft	514.86 k-ft	15.405	OK
Z-Z, +D+0.450W	33.422 k-ft	512.46 k-ft	15.333	OK
Z-Z, +D+0.5250E	85.155 k-ft	512.46 k-ft	6.018	OK
Z-Z, +0.90D+0.60W	44.562 k-ft	461.216 k-ft	10.350	OK
Z-Z, +0.90D+0.70E	113.540 k-ft	461.216 k-ft	4.062	OK

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
X-X, D Only	0.0 k	46.730 k	No Sliding	OK
X-X, +D+Lr	0.0 k	47.0 k	No Sliding	OK
X-X, +D+0.750Lr	0.0 k	46.933 k	No Sliding	OK
X-X, +D+0.60W	2.460 k	46.730 k	18.996	OK
X-X, +D+0.70E	12.250 k	46.730 k	3.815	OK
X-X, +D+0.750Lr+0.450W	1.845 k	46.933 k	25.438	OK
X-X, +D+0.450W	1.845 k	46.730 k	25.328	OK
X-X, +D+0.5250E	9.188 k	46.730 k	5.086	OK
X-X, +0.90D+0.60W	2.460 k	42.398 k	17.235	OK
X-X, +0.90D+0.70E	12.250 k	42.398 k	3.461	OK
Z-Z, D Only	0.0 k	49.087 k	No Sliding	OK
Z-Z, +D+Lr	0.0 k	49.357 k	No Sliding	OK
Z-Z, +D+0.750Lr	0.0 k	49.289 k	No Sliding	OK
Z-Z, +D+0.60W	0.0 k	49.087 k	No Sliding	OK
Z-Z, +D+0.5250E	0.0 k	49.087 k	No Sliding	OK
Z-Z, +0.90D+0.60W	0.0 k	44.754 k	No Sliding	OK
Z-Z, +0.90D+0.70E	0.0 k	44.754 k	No Sliding	OK
Z-Z, +D+0.70E	0.0 k	49.087 k	No Sliding	OK
Z-Z, +D+0.750Lr+0.450W	0.0 k	49.289 k	No Sliding	OK
Z-Z, +D+0.450W	0.0 k	49.087 k	No Sliding	OK

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	6.308	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.40D	6.308	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+0.50Lr	5.428	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+0.50Lr	5.428	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D	5.407	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D	5.407	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+1.60Lr	5.475	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+1.60Lr	5.475	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+1.60Lr+0.50W	5.475	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+1.60Lr+0.50W	5.475	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+0.50W	5.407	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+0.50W	5.407	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+0.50Lr+W	5.428	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+0.50Lr+W	5.428	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+W	5.407	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.20D+W	5.407	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.520D+E	6.850	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.520D+E	6.850	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +0.90D+W	4.055	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +0.90D+W	4.055	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +0.5798D+E	2.612	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +0.5798D+E	2.612	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
Z-Z, +1.40D	18.065	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.40D	18.065	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+0.50Lr	15.546	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+0.50Lr	15.546	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: frame footing.ec6

LIC# : KW-06016422, Build:20.23.08.01

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2023

DESCRIPTION: Footing under Column A

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.20D	15.485	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D	15.485	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+1.60Lr	15.681	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+1.60Lr	15.681	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+1.60Lr+0.50W	12.825	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+1.60Lr+0.50W	18.537	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+0.50W	12.628	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+0.50W	18.341	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+0.50Lr+W	9.833	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+0.50Lr+W	21.258	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+W	9.772	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.20D+W	21.197	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.520D+E	7.141	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.520D+E	32.092	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +0.90D+W	5.901	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +0.90D+W	17.326	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +0.5798D+E	3.555	-X	Top	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +0.5798D+E	21.396	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	8.29 psi	8.29 psi	1.57 psi	1.57 psi	8.29 psi	106.07 psi	0.08	OK
+1.20D+0.50Lr	7.14 psi	7.14 psi	1.35 psi	1.35 psi	7.14 psi	106.07 psi	0.07	OK
+1.20D	7.11 psi	7.11 psi	1.34 psi	1.34 psi	7.11 psi	106.07 psi	0.07	OK
+1.20D+1.60Lr	7.20 psi	7.20 psi	1.36 psi	1.36 psi	7.20 psi	106.07 psi	0.07	OK
+1.20D+1.60Lr+0.50W	5.72 psi	8.68 psi	1.36 psi	1.36 psi	8.68 psi	106.07 psi	0.08	OK
+1.20D+0.50W	5.63 psi	8.59 psi	1.34 psi	1.34 psi	8.59 psi	106.07 psi	0.08	OK
+1.20D+0.50Lr+W	4.19 psi	10.09 psi	1.35 psi	1.35 psi	10.09 psi	106.07 psi	0.10	OK
+1.20D+W	4.16 psi	10.06 psi	1.34 psi	1.34 psi	10.06 psi	106.07 psi	0.09	OK
+1.520D+E	2.56 psi	15.45 psi	1.70 psi	1.70 psi	15.45 psi	106.07 psi	0.15	OK
+0.90D+W	2.38 psi	8.28 psi	1.01 psi	1.01 psi	8.28 psi	106.07 psi	0.08	OK
+0.5798D+E	1.81 psi	10.77 psi	0.65 psi	0.65 psi	10.77 psi	106.07 psi	0.10	OK

All units k

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	17.46 psi	212.13psi	0.08233	OK
+1.20D+0.50Lr	15.03 psi	212.13psi	0.07085	OK
+1.20D	14.97 psi	212.13psi	0.07057	OK
+1.20D+1.60Lr	15.16 psi	212.13psi	0.07146	OK
+1.20D+1.60Lr+0.50W	15.16 psi	212.13psi	0.07146	OK
+1.20D+0.50W	14.97 psi	212.13psi	0.07057	OK
+1.20D+0.50Lr+W	15.03 psi	212.13psi	0.07085	OK
+1.20D+W	14.97 psi	212.13psi	0.07057	OK
+1.520D+E	18.96 psi	212.13psi	0.0894	OK
+0.90D+W	11.23 psi	212.13psi	0.05293	OK
+0.5798D+E	7.52 psi	212.13psi	0.03547	OK

ADDENDUM 5

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: frame footing.ec6

LIC# : KW-06016422, Build:20.23.08.01

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2023

DESCRIPTION: Footing under Column B

Code References

Calculations per ACI 318-19, IBC 2021, ASCE 7-16

Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	5.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	150.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	3.990 ksf
Soil Density	=	125.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	465.50 pcf
Soil/Concrete Friction Coeff.	=	0.4650

Increases based on footing Depth

Footing base depth below soil surface	=	4.0 ft
Allow press. increase per foot of depth when footing base is below	=	ksf ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth when max. length or width is greater than	=	ksf ft
---	---	--------

Dimensions

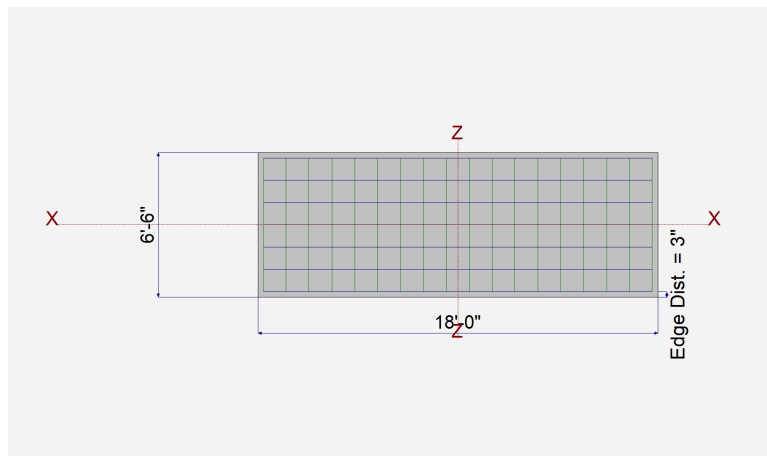
Width parallel to X-X Axis	=	18.0 ft
Length parallel to Z-Z Axis	=	6.50 ft
Footing Thickness	=	36.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	in
pz : parallel to Z-Z Axis	=	in
Height	=	in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in
----------------------	---	--------



Reinforcing

Bars parallel to X-X Axis

Number of Bars	=	7.0
Reinforcing Bar Size	=	# 8

Bars parallel to Z-Z Axis

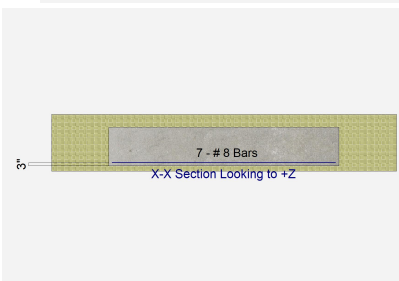
Number of Bars	=	18.0
Reinforcing Bar Size	=	# 8

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation

Bars along Z-Z Axis

# Bars required within zone	53.1 %
# Bars required on each side of zone	46.9 %



Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	93.0	1.950						k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=					109.150	492.20		k-ft
V-x	=					7.850	34.40		k
V-z	=								k

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: frame footing.ec6

LIC# : KW-06016422, Build:20.23.08.01

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2023

DESCRIPTION: Footing under Column B

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.6065	Soil Bearing	2.420 ksf	3.990 ksf	+D+0.70E about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	2.831	Overturning - Z-Z	416.780 k-ft	1,179.77 k-ft	+0.90D+0.70E
PASS	3.474	Sliding - X-X	24.080 k	83.648 k	+0.90D+0.70E
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.3622	Z Flexure (+X)	45.069 k-ft/ft	124.423 k-ft/ft	+1.40D
PASS	0.3622	Z Flexure (-X)	45.069 k-ft/ft	124.423 k-ft/ft	+1.40D
PASS	0.05081	X Flexure (+Z)	5.877 k-ft/ft	115.663 k-ft/ft	+1.40D
PASS	0.05081	X Flexure (-Z)	5.877 k-ft/ft	115.663 k-ft/ft	+1.40D
PASS	0.1669	1-way Shear (+X)	17.704 psi	106.066 psi	+1.40D
PASS	0.1669	1-way Shear (-X)	17.704 psi	106.066 psi	+1.40D
PASS	0.01378	1-way Shear (+Z)	1.461 psi	106.066 psi	+1.40D
PASS	0.01378	1-way Shear (-Z)	1.461 psi	106.066 psi	+1.40D
PASS	0.1314	2-way Punching	27.881 psi	212.132 psi	+1.40D

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc		Actual Soil Bearing Stress @ Location				Actual / Allow Ratio
			Zecc (in)	Bottom, -Z	Top, +Z	Left, -X	Right, +X	
X-X, D Only	3.990	n/a	0.0	1.245	1.245	n/a	n/a	0.312
X-X, +D+Lr	3.990	n/a	0.0	1.262	1.262	n/a	n/a	0.316
X-X, +D+0.750Lr	3.990	n/a	0.0	1.257	1.257	n/a	n/a	0.315
X-X, +D+0.60W	3.990	n/a	0.0	1.245	1.245	n/a	n/a	0.312
X-X, +D+0.70E	3.990	n/a	0.0	1.245	1.245	n/a	n/a	0.312
X-X, +D+0.750Lr+0.450W	3.990	n/a	0.0	1.257	1.257	n/a	n/a	0.315
X-X, +D+0.450W	3.990	n/a	0.0	1.245	1.245	n/a	n/a	0.312
X-X, +D+0.5250E	3.990	n/a	0.0	1.245	1.245	n/a	n/a	0.312
X-X, +0.90D+0.60W	3.990	n/a	0.0	1.120	1.120	n/a	n/a	0.281
X-X, +0.90D+0.70E	3.990	n/a	0.0	1.120	1.120	n/a	n/a	0.281
Z-Z, D Only	3.990	0.0	n/a	n/a	n/a	1.245	1.245	0.312
Z-Z, +D+Lr	3.990	0.0	n/a	n/a	n/a	1.262	1.262	0.316
Z-Z, +D+0.750Lr	3.990	0.0	n/a	n/a	n/a	1.257	1.257	0.315
Z-Z, +D+0.60W	3.990	6.560	n/a	n/a	n/a	1.020	1.469	0.368
Z-Z, +D+0.70E	3.990	34.338	n/a	n/a	n/a	0.06934	2.420	0.607
Z-Z, +D+0.750Lr+0.450W	3.990	4.871	n/a	n/a	n/a	1.089	1.426	0.357
Z-Z, +D+0.450W	3.990	4.920	n/a	n/a	n/a	1.076	1.413	0.354
Z-Z, +D+0.5250E	3.990	25.754	n/a	n/a	n/a	0.3632	2.127	0.533
Z-Z, +0.90D+0.60W	3.990	7.289	n/a	n/a	n/a	0.8958	1.345	0.337
Z-Z, +0.90D+0.70E	3.990	38.154	n/a	n/a	n/a	0.0	2.298	0.576

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D Only	None	0.0 k-ft	Infinity	OK
X-X, +D+Lr	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr	None	0.0 k-ft	Infinity	OK
X-X, +D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.70E	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.5250E	None	0.0 k-ft	Infinity	OK
X-X, +0.90D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +0.90D+0.70E	None	0.0 k-ft	Infinity	OK
Z-Z, D Only	None	0.0 k-ft	Infinity	OK
Z-Z, +D+Lr	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.750Lr	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.60W	79.620 k-ft	1,310.85 k-ft	16.464	OK
Z-Z, +D+0.70E	416.780 k-ft	1,310.85 k-ft	3.145	OK
Z-Z, +D+0.750Lr+0.450W	59.715 k-ft	1,324.01 k-ft	22.172	OK

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: frame footing.ec6

LIC# : KW-06016422, Build:20.23.08.01

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2023

DESCRIPTION: Footing under Column B

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Z-Z, +D+0.450W	59.715 k-ft	1,310.85 k-ft	21.952	OK
Z-Z, +D+0.5250E	312.585 k-ft	1,310.85 k-ft	4.194	OK
Z-Z, +0.90D+0.60W	79.620 k-ft	1,179.77 k-ft	14.817	OK
Z-Z, +0.90D+0.70E	416.780 k-ft	1,179.77 k-ft	2.831	OK

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
X-X, D Only	0.0 k	90.420 k	No Sliding	OK
X-X, +D+Lr	0.0 k	91.327 k	No Sliding	OK
X-X, +D+0.750Lr	0.0 k	91.10 k	No Sliding	OK
X-X, +D+0.60W	4.710 k	90.420 k	19.198	OK
X-X, +D+0.70E	24.080 k	90.420 k	3.755	OK
X-X, +D+0.750Lr+0.450W	3.533 k	91.10 k	25.789	OK
X-X, +D+0.450W	3.533 k	90.420 k	25.597	OK
X-X, +D+0.5250E	18.060 k	90.420 k	5.007	OK
X-X, +0.90D+0.60W	4.710 k	83.648 k	17.760	OK
X-X, +0.90D+0.70E	24.080 k	83.648 k	3.474	OK
Z-Z, D Only	0.0 k	130.570 k	No Sliding	OK
Z-Z, +D+Lr	0.0 k	131.477 k	No Sliding	OK
Z-Z, +D+0.750Lr	0.0 k	131.250 k	No Sliding	OK
Z-Z, +D+0.60W	0.0 k	130.570 k	No Sliding	OK
Z-Z, +D+0.5250E	0.0 k	130.570 k	No Sliding	OK
Z-Z, +0.90D+0.60W	0.0 k	123.797 k	No Sliding	OK
Z-Z, +0.90D+0.70E	0.0 k	123.797 k	No Sliding	OK
Z-Z, +D+0.70E	0.0 k	130.570 k	No Sliding	OK
Z-Z, +D+0.750Lr+0.450W	0.0 k	131.250 k	No Sliding	OK
Z-Z, +D+0.450W	0.0 k	130.570 k	No Sliding	OK

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	5.877	+Z	Bottom	0.7776	AsMin	0.790	115.663	OK
X-X, +1.40D	5.877	-Z	Bottom	0.7776	AsMin	0.790	115.663	OK
Z-Z, +1.40D	45.069	-X	Bottom	0.7776	AsMin	0.8508	124.423	OK
Z-Z, +1.40D	45.069	+X	Bottom	0.7776	AsMin	0.8508	124.423	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	17.70 psi	17.70 psi	1.46 psi	1.46 psi	17.70 psi	106.07 psi	0.17	OK

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	27.88 psi	212.13psi	0.1314	OK

ADDENDUM 5

3.3 SHEAR WALL FOOTING DESIGN

Typical Shear walls footing at line 3

$$T2/C2 = F \cdot H/L = V_s \cdot L \cdot H/L = V_s \cdot H = 420 \text{ lb/ft} \cdot (27\text{ft} - 12\text{ft}) = 6300 \text{ lb}$$

$$T/C = (F2 \cdot H2 + F1 \cdot H1)/L = (V_{s2} \cdot L \cdot H2 + V_{s1} \cdot L \cdot H1)/L = V_{s2} \cdot H2 + V_{s1} \cdot H1$$

$$= 420 \text{ lb/ft} \cdot 27 \text{ ft} + 94 \text{ lb/ft} \cdot 15 \text{ ft} = 11340 \text{ lb} + 1410 \text{ lb} = 12750 \text{ lb}$$

$$\text{HDQ14-SDS2.5 with 6x6 } T_a = 13710 \text{ lb} > T = 12750 \text{ lb}$$

$$F2 = L \cdot V_{s2} = 17 \text{ ft} \cdot 420 \text{ lb/ft} \cdot 1.4$$

$$= 9996 \text{ lb (LRFD)}$$

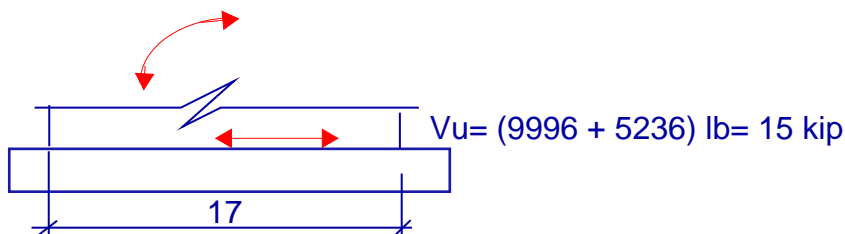
$$F1 = L \cdot V_{s1} = 17 \text{ ft} \cdot 94 \text{ lb/ft} \cdot 1.4$$

$$= 2237 \text{ lb (LRFD)}$$

$$P_{dl} = 1 \text{ kip/ft} \cdot 22 \text{ ft} = 22 \text{ kip}$$

$$P_{rl} = 0.73 \text{ kip/ft} \cdot 22 \text{ ft} = 16 \text{ kip}$$

$$M_u = 394 \text{ kipft}$$



$$M_u = (9996 \text{ lb} \cdot (27) \text{ ft} + 2237 \text{ lb} \cdot (15) \text{ ft}) = 299880 \text{ lbft} + 33555 \text{ lbft} = 333 \text{ kipft}$$

$$P_{dl} = P_{\text{high-roof}} + P_{\text{low-roof}} + P_w = 27 \text{ psf} \cdot 38\text{ft}/2 + 19 \text{ psf} \cdot 14 \text{ ft}/2 + 15 \text{ psf} \cdot 24 \text{ ft} = 513 \text{ lb/ft} + 133 \text{ lb/ft} + 360 \text{ lb/ft} = 1006 \text{ lb/ft}$$

$$P_{ll} = P_{\text{high-roof}} + P_{\text{low-roof}} = 20 \text{ psf} \cdot 38\text{ft}/2 + 50 \text{ psf} \cdot 14 \text{ ft}/2 = 380 \text{ lb/ft} + 350 \text{ lb/ft} = 730 \text{ lb/ft}$$

$V = 514 \text{ lb/ft (ASD)}$ $k = \text{stiffness of the anchorage} = F / \delta \text{ (deflection / elongation)}$

Shear Walls in a Line N/S Direction Between E & F

(bending) (shear) (wall anchorage slip)

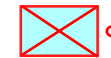
$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h}{b}\Delta_a \quad (\text{C4.3.2-2})$$

δ

(1) HDQ11-SDS2.5

$k = (13710 \text{ lb}/0.107") = 128,130 \text{ lb/in}$

$$V = \frac{8h^3}{EAb_{sw1}} + \frac{h}{1000G_a} + \frac{h^2}{kb_{sw1}}$$



End Post A = $5.5 \times 7.5 = 41.25 \text{ in}^2$

$$V = \frac{8(10')^3}{(1,400,000)(16.5)(4')} + \frac{(10')}{1000(14,000)} + \frac{(10')^2}{(64,924)(4')}$$

δ

0.000023 0.0000007 0.00011

1600000 41.25 in² 14 ft 128130 lb/in 14 ft

374 lb/ft $V = \frac{\delta}{0.00013} \Rightarrow$

$$\delta = 0.00013 \times 374 \text{ lb/ft} = 0.048"$$

$$\text{Drift} = 0.048 \times \frac{4}{1.5} = 0.22" \leq 0.375 = 3/8"$$

ADDENDUM 5

TABLE 7.1.1-1: Minimum Footing Dimensions

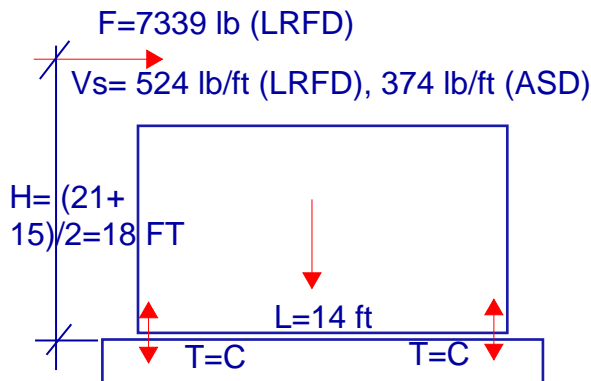
FOOTING TYPE	*MINIMUM DEPTH (inches)	MINIMUM WIDTH (inches)
Continuous	18	12
Isolated	18	18

*Below lowest adjacent pad grade

Minimum footing depths shown above are taken from lowest adjacent pad grade. Design foundations recommended above for a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) for dead-plus-live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading.

Typical SW footing Design in N/S direction

Between E & F



$$M = 7339 \text{ lb} \cdot 18 \text{ ft} = 132 \text{ kipft (LRFD)}$$

$$P_{dl \text{ wall}} = 10 \text{ psf} \cdot 14 \text{ ft} \cdot 14 \text{ ft} = 2 \text{ kip}$$

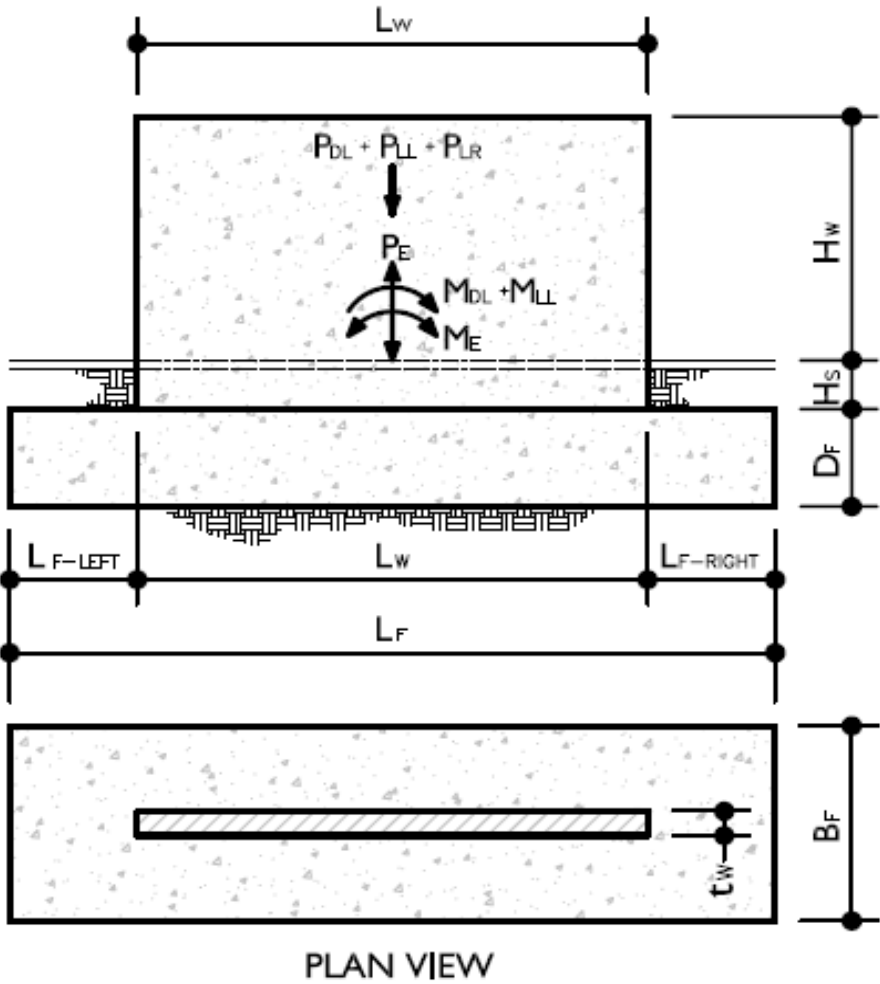
$$P_{\text{slab}} = 5/12 \cdot 150 \text{ pcf} \cdot 18 \text{ ft} \cdot 6 \text{ ft} = 6.7 \text{ kip}$$

Footing Type W2 is OK

SHEAR WALL FOOTING DESIGN

Project : _____
Project # : _____ Engineer : _____
Shear Wall : _____ Shear Wall Location : **Grid 3**

Sheet : _____
Date : **6/10/2025**
Checked : _____
Wall Type : **Concrete**



STEP 1: GENERAL INPUT DATA

Footing Properties:		
Redundancy Factor, ρ	1.00	Concrete Strength (footing), f'_c
S_{ds}	1.601 g	Footing Width, B_f
Allowable Soil Pressure q_A	3.00 ksf	Footing Depth, D_f
1/3 Increase Allowed?	YES	Footing Length Left, L_{f-LEFT}
Net Allowable Pressure?	YES	Footing Length Right, $L_{f-RIGHT}$
Wall Properties:		
Shear Wall Height, H_w	24.00 ft	Total Footing Length, L_f
Shear Wall Length, L_w	22.00 ft	Area of Footing, A_{FTG}
Shear Wall Thickness, t_w	6.00 in	Section Modulus of Footing, S_{FTG}
Unit Weight of Wall, γ_w	0 pcf	Unit Weight of Footing, γ_f
Wall Weight, W_w	0.0 k	Footing Weight, W_f
Ω_o	2.50	Soil Above Footing, H_s
Omega?	NO	Unit Weight of Soil, γ_s
		Weight of Soil, W_s
		10% Reduction?
		25% Reduction?

STEP 2: APPLIED LOADS (At the Center of Wall)

	Seismic (E)	Dead (DL)	Live (LL)	Roof Live (LR)	
P =	0.0	22.0	0.0	16.0	k
M =	333.0	0.0	0.0	0.0	ft-k
V =	15.0	0.0	0.0	0.0	k
Mtot =	355.5	= M + V x Footing Depth			

STEP 3: FOOTING SIZE (Service Loads)

Comb. 1: D + L + Lr		[CBC Eq. 16-16]				Allowable**	
$P_1 = (W_w + W_f + W_s + P_D) + P_L + P_{LR} =$		54.9	k	$P/A + M/S$			
$M_1 = (W_w + P_D + P_L + P_{LR})(L_{f-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} + M_{RL} =$		0.0	ft-k	1.06 ksf		< 3.25 ksf	
$M_1 NET = (W_w + P_D + P_L + P_{LR})(L_{f-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) =$		N/A	ft-k	$P/A - M/S$		< O.K. >	
$X_1 =$		N/A	ft	1.06 ksf			
Comb. 2: D + L + $\rho Q_E/1.4$ ($E_v = 0$)*		[CBC Eq. 16-20]					
$P_2 = (W_w + W_f + W_s + P_D) + P_L + \rho P_E/1.4 =$		38.9	k	$2P/(XB_f)$			
$M_2 = [(W_w + P_D) + P_L + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} + \rho M_E/1.4 =$		253.9	ft-k	2.00 ksf		< 4.33 ksf	
$M_2 NET = [(W_w + P_D) + P_L + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) + M_{DL} + M_{LL} + \rho M_E/1.4 =$		759.6	ft-k	$P/A - M/S$		< O.K. >	
O.K.		$a_2 =$	19.53 ft	0.00 ksf			
$X_2 =$		19.42	ft				
Comb. 3: D + L - $\rho Q_E/1.4$ ($E_v = 0$)*		[CBC Eq. 16-20]					
$P_3 = (W_w + W_f + W_s + P_D) - P_L + \rho P_E/1.4 =$		38.9	k	$2P/(XB_f)$			
$M_2 = [(W_w + P_D) + P_L - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} - \rho M_E/1.4 =$		(253.9)	ft-k	2.00 ksf		< 4.33 ksf	
$M_2 NET = [(W_w + P_D) + P_L - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) + M_{DL} + M_{LL} - \rho M_E/1.4 =$		251.8	ft-k	$P/A - M/S$		< O.K. >	
O.K.		$a_3 =$	6.47 ft	0.00 ksf			
$X_3 =$		19.42	ft				
Comb. 4: 0.9D + $\rho Q_E/1.4$ ($E_v = 0$)*		[CBC Eq. 16-21]					
$P_4 = (0.9)(W_w + W_f + W_s + P_D) + \rho P_E/1.4 =$		35.0	k	$2P/(XB_f)$			
$M_4 = [(0.9)(W_w + P_D) + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + (0.9)M_{DL} + \rho M_E/1.4 =$		253.9	ft-k	2.03 ksf		< 4.33 ksf	
$M_4 NET = [(0.9)(W_w + P_D) + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (0.9)(W_f + W_s)(L_f/2) + (0.9)M_{DL} + \rho M_E/1.4 =$		709.1	ft-k	$P/A - M/S$		< O.K. >	
O.K.		$a_4 =$	20.25 ft	0.00 ksf			
$X_4 =$		17.24	ft				
Comb. 5: 0.9D - $\rho Q_E/1.4$ ($E_v = 0$)*		[CBC Eq. 16-21]					
$P_5 = (0.9)(W_w + W_f + W_s + P_D) - \rho P_E/1.4 =$		35.0	k	$2P/(XB_f)$			
$M_5 = [(0.9)(W_w + P_D) - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + (0.9)M_{DL} - \rho M_E/1.4 =$		(253.9)	ft-k	2.03 ksf		< 4.33 ksf	
$M_5 NET = [(0.9)(W_w + P_D) - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (0.9)(W_f + W_s)(L_f/2) + (0.9)M_{DL} - \rho M_E/1.4 =$		201.2	ft-k	$P/A - M/S$		< O.K. >	
O.K.		$a_5 =$	5.75 ft	0.00 ksf			
$X_5 =$		17.24	ft				



SHEAR WALL FOOTING DESIGN

Project : _____
Project # : _____ Engineer : _____

Sheet : _____
Date : 6/10/2025
Checked : _____
Wall Type: Concrete

Shear Wall : WALL 2 Shear Wall Location : Grid 3

STEP 4: DESIGN FOR FLEXURE

A. Factored Soil Pressure

f₁ = 0.5

Comb. 6: 1.4D

P6 = 54.5 k

M6 = 0.0 ft-k

M6net = N/A ft-k

a6 = N/A ft

X6 = N/A ft

[CBC Eq. 16-1]

P/A + M/S = 1.05 ksf

P/A - M/S = 1.05 ksf

p1 = 1.047 ksf

p2 = 1.047 ksf

p3 = 1.047 ksf

p4 = 1.047 ksf

Comb. 7: 1.2D + 1.6L + 0.5Lr

P7 = 54.7 k

M7 = 0.0 ft-k

M7net = N/A ft-k

a7 = N/A ft

X7 = N/A ft

[CBC Eq. 16-2]

P/A + M/S = 1.05 ksf

P/A - M/S = 1.05 ksf

p1 = 1.052 ksf

p2 = 1.052 ksf

p3 = 1.052 ksf

p4 = 1.052 ksf

Comb. 8: 1.2D + f₁L + 1.6Lr

P8 = 72.3 k

M8 = 0.0 ft-k

M8net = N/A ft-k

a8 = N/A ft

X8 = N/A ft

[CBC Eq. 16-3]

P/A + M/S = 1.39 ksf

P/A - M/S = 1.39 ksf

p1 = 1.390 ksf

p2 = 1.390 ksf

p3 = 1.390 ksf

p4 = 1.390 ksf

Comb.9: (1.2 + 0.2S_{DS})D + f₁L + ρQ_E

P9 = 59.1 k

M9 = 355.5 ft-k

M9net = 1124.3 ft-k

a9 = 19.01 ft

X9 = 20.97 ft

[CBC Eq. 16-5]

2P/(XBf) = 2.82 ksf

P/A-M/S or 0 = 0.00 ksf

p1 = 0.000 ksf

p2 = 0.000 ksf

p3 = 2.552 ksf

p4 = 2.821 ksf

O.K.

Comb. 10: (1.2 + 0.2S_{DS})D + f₁L - ρQ_E

P10 = 59.1 k

M10 = (355.5) ft-k

M10net = 413.3 ft-k

a10 = 6.99 ft

X10 = 20.97 ft

[CBC Eq. 16-5]

2P/(XBf) = 2.82 ksf

P/A-M/S or 0 = 0.00 ksf

p1 = 2.821 ksf

p2 = 2.552 ksf

p3 = 0.000 ksf

p4 = 0.000 ksf

Comb. 11: 0.9D + ρQ_E

P11 = 35.0 k

M11 = 355.5 ft-k

M11net = 810.6 ft-k

a11 = 23.15 ft

X11 = 8.54 ft

[CBC Eq. 16-6]

2P/(XBf) = 4.10 ksf

P/A-M/S or 0 = 0.00 ksf

p1 = 0.000 ksf

p2 = 0.000 ksf

p3 = 3.140 ksf

p4 = 4.101 ksf

O.K.

Comb. 12: 0.9D - ρQ_E

P11 = 35.0 k

M11 = (355.5) ft-k

M11net = 99.6 ft-k

a11 = 2.85 ft

X11 = 8.54 ft

[CBC Eq. 16-6]

2P/(XBf) = 4.10 ksf

P/A-M/S or 0 = 0.00 ksf

p1 = 4.101 ksf

p2 = 3.140 ksf

p3 = 0.000 ksf

p4 = 0.000 ksf



SHEAR WALL FOOTING DESIGN

Sheet : _____

Project : _____

Date : 6/10/2025

Project # : _____

Engineer : _____

Checked : _____

Shear Wall : WALL 2

Shear Wall Location : Grid 3

Wall Type: Concrete

B. Longitudinal Reinforcing [Bottom]

Rebar clearance from the bottom : 3.00 in
d_t = D_F - clear - 1" = 14 in

Assuming tension-controlled section
φ : 0.90

Steel Strength (rebar), f_y : 60.00
β₁ : 0.85
ρ_{MIN} : 0.0033333

Left	M _u @ p2 (ft-k)	Soil Pressure
Comb. 6:	2.4	Case 1
Comb. 7:	2.6	Case 1
Comb. 8:	4.0	Case 1
Comb. 9:	0.0	Case 4B or 5B
Comb. 10:	8.9	Case 3A or 4A
Comb. 11:	0.0	Case 4B or 5B
Comb. 12:	14.0	Case 3A or 4A

Right	M _u @ p3 (ft-k)	Soil Pressure
Comb. 6:	2.4	Case 1
Comb. 7:	2.6	Case 1
Comb. 8:	4.0	Case 1
Comb. 9:	8.9	Case 3B or 4B
Comb. 10:	0.0	Case 4A or 5A
Comb. 11:	14.0	Case 3B or 4B
Comb. 12:	0.0	Case 4A or 5A

ρ = 0.00066 Temp. Steel
A_{S REQ'D} = 0.39 in²
Rebars provided 2- #6
A_{S prov.} = 0.88 in² < O.K. >
c = 1.02 in
c / d_t = 0.073
φ = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90
Tension Control Assumption is Correct.
Spacing = 16.00 in

ρ = 0.00066 Temp. Steel
A_{S REQ'D} = 0.39 in²
Rebars provided 2- #6
A_{S prov.} = 0.88 in² < O.K. >
c = 1.02 in
c / d_t = 0.073
φ = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90
Tension Control Assumption is Correct.
Spacing = 16.00 in

C. Longitudinal Reinforcing [Top]

[Moments are due to the weight of footing and soil]

Left
Mu @ p2 = 1.82 ft-k
ρ = 0.00009 Temp. Steel
A_{S REQ'D} = 0.39 in²
Rebars provided 2- #6
A_{S prov.} = 0.88 in² < O.K. >
c = 1.02 in
c / d_t = 0.073
φ = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90
Tension Control Assumption is Correct.
Spacing = 16.00 in

Right
Mu @ p3 = 1.82 ft-k
ρ = 0.00009 Temp. Steel
A_{S REQ'D} = 0.39 in²
Rebars provided 2- #6
A_{S prov.} = 0.88 in² < O.K. >
c = 1.02 in
c / d_t = 0.073
φ = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90
Tension Control Assumption is Correct.
Spacing = 16.00 in

D. Transverse Reinforcing [Bottom]

d_t = D_F - clear - 2" = 13 in

b = 12 in

Left
Mu @ p2 = 0.80 ft-k / ft
ρ = 0.00009 Temp. Steel
A_{S REQ'D} = 0.19 in²
Rebars provided #5 @ 18 o.c. < O.K. >
A_{S prov.} = 0.20 in² < O.K. >
c = 0.24 in
c / d_t = 0.018
φ = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90
Tension Control Assumption is Correct.

Right
Mu @ p3 = 0.80 ft-k
ρ = 0.00009 Temp. Steel
A_{S REQ'D} = 0.19 in²
Rebars provided #5 @ 18 o.c. < O.K. >
A_{S prov.} = 0.20 in² < O.K. >
c = 0.24 in
c / d_t = 0.018
φ = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90
Tension Control Assumption is Correct.

E. Transverse Reinforcing [Top]

b = 12 in

Mu = 0.13 ft-k
ρ = 0.00001 Temp. Steel
A_{S REQ'D} = 0.19 in²
Rebars provided #5 @ 18 o.c. < O.K. >

A_{S prov.} = 0.20 in² < O.K. >
c = 0.24 in
c / d_t = 0.018
φ = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90
Tension Control Assumption is Correct.



SHEAR WALL FOOTING DESIGN

Project : _____
Project # : _____ Engineer : _____
Shear Wall : WALL 2 Shear Wall Location : Grid 3

Sheet : _____
Date : 6/10/2025
Checked : _____
Wall Type: Concrete

STEP 5: CHECK FOR SHEAR

A. Longitudinal [The critital location for flexural shear is a distance "d" from the end of wall]

Steel Strength (rebar), f_y : 60.00

Rebar clearance from the bottom : 3.00 in ϕ : 0.75
 $d_t = D_F - \text{clear} - 1" = 14$ in

Left	V_{u12} (kips)	M_{u12} (ft-k)	V_{ud}/M_u	ϕV_c (kips)	
Comb. 6:	1.0	0.4	1.000	27.9	< O.K. >
Comb. 7:	1.1	0.5	1.000	27.9	< O.K. >
Comb. 8:	1.7	0.7	1.000	27.9	< O.K. >
Comb. 9:	(0.8)	0.0	N/A	27.6	< O.K. >
Comb. 10:	3.8	1.6	1.000	27.9	< O.K. >
Comb. 11:	(0.5)	0.0	N/A	27.6	< O.K. >
Comb. 12:	6.0	2.6	1.000	27.9	< O.K. >

Right	V_{u34} (kips)	M_{u34} (ft-k)	V_{ud}/M_u	ϕV_c (kips)	
Comb. 6:	1.0	0.4	1.000	27.9	< O.K. >
Comb. 7:	1.1	0.5	1.000	27.9	< O.K. >
Comb. 8:	1.7	0.7	1.000	27.9	< O.K. >
Comb. 9:	3.8	1.6	1.000	27.9	< O.K. >
Comb. 10:	(0.8)	0.0	N/A	27.6	< O.K. >
Comb. 11:	6.0	2.6	1.000	27.9	< O.K. >
Comb. 12:	(0.5)	0.0	N/A	27.6	< O.K. >

*Design shear, V_u , is N/A if $d > L_F\text{-Left}$ or $L_F\text{-Right}$

Shear Capacity

Larger of
$$\left\{ \begin{aligned} \phi V_c &= \phi 2 \sqrt{f'_c} b d = 27.6 \text{ k} & b &= B_F & \text{(ACI Eq. 11-3)} \\ \phi V_c &= \phi \left(1.9 \sqrt{f'_c} + 2500 \rho_w \frac{V_u d}{M_u} \right) b_w d < 3.5 \sqrt{f'_c} b_w d & & & \text{(ACI Eq. 11-5)} \end{aligned} \right.$$

V_{ud}/M_u shall not be greather than 1.0

B. Transverse [The critital location for flexural shear is a distance "d" from the face of wall]

$d_t = D_F - \text{clear} - 2" = 13$ in $b = 12$ in

Shear Capacity
$$\phi V_c = \phi 2 \sqrt{f'_c} b d = 13.8 \text{ k}$$

Left $V_u @ p2 = \text{N/A k}$ < O.K. > Right $M_u @ p3 = \text{N/A k}$ < O.K. >

STEP 6: SUMMARY

FOOTING SIZE			LONGITUDINAL REINFORCEMENT			
Footing Width =	2.0	ft	Left		Right	
Footing Depth =	1.5	ft	Bottom:	2- #6	Bottom:	2- #6
Total Footing Length =	26.0	ft	Top:	2- #6	Top:	2- #6
Footing Length (Left) =	2.0	ft	TRANSVERSE REINFORCEMENT			
Footing Length (Right) =	2.0	ft	Left		Right	
Shear Wall Length, L_w =	22.0	ft	Bottom:	#5 @ 18 o.c.	Bottom:	#5 @ 18 o.c.
			Top:	#5 @ 18 o.c.	Top:	#5 @ 18 o.c.

SHEAR WALL FOOTING DESIGN

Project : FS- Main Building

Project # : 25534

Shear Wall : D/S502

Engineer : _____

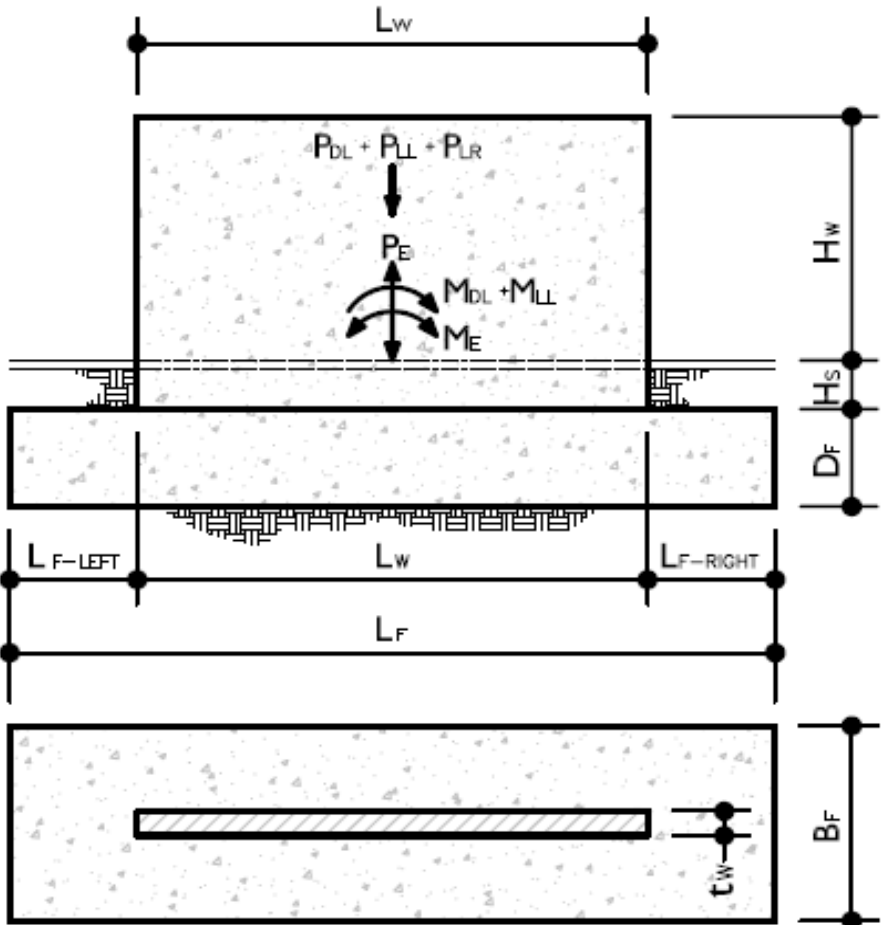
Shear Wall Location : C &D

Sheet : _____

Date : 7/3/2025

Checked : _____

Wall Type: Concrete



PLAN VIEW

STEP 1: GENERAL INPUT DATA

		Footing Properties:	
Redundancy Factor, ρ :	1.30	Concrete Strength (footing), f'_c :	3.00
S_{ds} :	1.601 g	Footing Width, B_f :	3.00
Allowable Soil Pressure q_A :	3.00 ksf	Footing Depth, D_f :	1.50
1/3 Increase Allowed?	YES	Footing Length Left, L_{F-LEFT} :	3.50
Net Allowable Pressure?	YES	Footing Length Right, $L_{F-RIGHT}$:	3.50
Wall Properties:		Total Footing Length, L_F :	10.0
Shear Wall Height, H_w :	10.00 ft	Area of Footing, A_{FTG} :	30.0
Shear Wall Length, L_w :	3.00 ft	Section Modulus of Footing, S_{FTG} :	50.0
Shear Wall Thickness, t_w :	8.00 in	Unit Weight of Footing, γ_F :	150
Unit Weight of Wall, γ_w :	0 pcf	Footing Weight, W_F :	6.8
Wall Weight, W_w :	0.0 k	Soil Above Footing, H_s :	1.00
Ω_o =	2.50	Unit Weight of Soil, γ_s :	100
Omega?	NO	Weight of Soil, W_s :	3.0
		10% Reduction?	NO
		25% Reduction?	NO

STEP 2: APPLIED LOADS (At the Center of Wall)

	Seismic (E)	Dead (DL)	Live (LL)	Roof Live (LR)	
P =	0.0	6.0	0.0	0.0	k
M =	47.6	0.0	0.0	0.0	ft-k
V =	2.4	0.0	0.0	0.0	k
Mtot =	51.2	= M + V x Footing Depth			

STEP 3: FOOTING SIZE (Service Loads)

Comb. 1: D + L + Lr

[CBC Eq. 16-16]

$$\begin{aligned} P_1 &= (W_w + W_f + W_s + P_D) + P_L + P_{LR} = 15.8 \text{ k} \\ M_1 &= (W_w + P_D + P_L + P_{LR})(L_{F-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} + M_{RL} = 0.0 \text{ ft-k} \\ M_1 \text{ NET} &= (W_w + P_D + P_L + P_{LR})(L_{F-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) = \text{N/A} \text{ ft-k} \\ X_1 &= \text{N/A} \text{ ft} \end{aligned}$$

P/A + M/S
0.53 ksf
P/A - M/S
0.53 ksf

Allowable**

<

3.25 ksf

< O.K. >

Comb. 2: D + L + $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-20]

$$\begin{aligned} P_2 &= (W_w + W_f + W_s + P_D) + P_L + \rho P_E/1.4 = 15.8 \text{ k} \\ M_2 &= [(W_w + P_D) + P_L + (\rho P_E/1.4)](L_{F-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} + \rho M_E/1.4 = 47.5 \text{ ft-k} \\ M_2 \text{ NET} &= [(W_w + P_D) + P_L + (\rho P_E/1.4)](L_{F-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) + M_{DL} + M_{LL} + \rho M_E/1.4 = 126.3 \text{ ft-k} \\ a_2 &= 8.02 \text{ ft} \\ X_2 &= 5.94 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)
1.77 ksf
P/A-M/S
0.00 ksf

<

4.33 ksf

< O.K. >

Comb. 3: D + L - $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-20]

$$\begin{aligned} P_3 &= (W_w + W_f + W_s + P_D) - P_L + \rho P_E/1.4 = 15.8 \text{ k} \\ M_2 &= [(W_w + P_D) + P_L - (\rho P_E/1.4)](L_{F-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} - \rho M_E/1.4 = (47.5) \text{ ft-k} \\ M_2 \text{ NET} &= [(W_w + P_D) + P_L - (\rho P_E/1.4)](L_{F-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) + M_{DL} + M_{LL} - \rho M_E/1.4 = 31.2 \text{ ft-k} \\ a_3 &= 1.98 \text{ ft} \\ X_3 &= 5.94 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)
1.77 ksf
P/A-M/S
0.00 ksf

<

4.33 ksf

< O.K. >

Comb. 4: 0.9D + $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-21]

$$\begin{aligned} P_4 &= (0.9)(W_w + W_f + W_s + P_D) + \rho P_E/1.4 = 14.2 \text{ k} \\ M_4 &= [(0.9)(W_w + P_D) + (\rho P_E/1.4)](L_{F-LEFT} + L_w/2 - L_f/2) + (0.9)M_{DL} + \rho M_E/1.4 = 47.5 \text{ ft-k} \\ M_4 \text{ NET} &= [(0.9)(W_w + P_D) + (\rho P_E/1.4)](L_{F-LEFT} + L_w/2) + (0.9)(W_f + W_s)(L_f/2) + (0.9)M_{DL} + \rho M_E/1.4 = 118.4 \text{ ft-k} \\ a_4 &= 8.35 \text{ ft} \\ X_4 &= 4.94 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)
1.91 ksf
P/A-M/S
0.00 ksf

<

4.33 ksf

< O.K. >

Comb. 5: 0.9D - $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-21]

$$\begin{aligned} P_5 &= (0.9)(W_w + W_f + W_s + P_D) - \rho P_E/1.4 = 14.2 \text{ k} \\ M_5 &= [(0.9)(W_w + P_D) - (\rho P_E/1.4)](L_{F-LEFT} + L_w/2 - L_f/2) + (0.9)M_{DL} - \rho M_E/1.4 = (47.5) \text{ ft-k} \\ M_5 \text{ NET} &= [(0.9)(W_w + P_D) - (\rho P_E/1.4)](L_{F-LEFT} + L_w/2) + (0.9)(W_f + W_s)(L_f/2) + (0.9)M_{DL} - \rho M_E/1.4 = 23.3 \text{ ft-k} \\ a_5 &= 1.65 \text{ ft} \\ X_5 &= 4.94 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)
1.91 ksf
P/A-M/S
0.00 ksf

<

4.33 ksf

< O.K. >



SHEAR WALL FOOTING DESIGN

Project : FS- Main Building
Project # : 25534 Engineer : _____
Shear Wall : D/S502 Shear Wall Location : C &D

Sheet : _____
Date : 7/3/2025
Checked : _____
Wall Type : Concrete

STEP 4: DESIGN FOR FLEXURE

A. Factored Soil Pressure

Comb. 6: 1.4D			[CBC Eq. 16-1]		$f_1 = 0.5$		
P6 =	22.1	k	P/A + M/S =	0.74 ksf	p1 =	0.735 ksf	
M6 =	0.0	ft-k	P/A - M/S =	0.74 ksf	p2 =	0.735 ksf	
M6net =	N/A	ft-k			p3 =	0.735 ksf	
a6 =	N/A	ft			p4 =	0.735 ksf	
X6 =	N/A	ft					
Comb. 7: 1.2D + 1.6L + 0.5Lr			[CBC Eq. 16-2]				
P7 =	18.9	k	P/A + M/S =	0.63 ksf	p1 =	0.630 ksf	
M7 =	0.0	ft-k	P/A - M/S =	0.63 ksf	p2 =	0.630 ksf	
M7net =	N/A	ft-k			p3 =	0.630 ksf	
a7 =	N/A	ft			p4 =	0.630 ksf	
X7 =	N/A	ft					
Comb. 8: 1.2D + f1L + 1.6Lr			[CBC Eq. 16-3]				
P8 =	18.9	k	P/A + M/S =	0.63 ksf	p1 =	0.630 ksf	
M8 =	0.0	ft-k	P/A - M/S =	0.63 ksf	p2 =	0.630 ksf	
M8net =	N/A	ft-k			p3 =	0.630 ksf	
a8 =	N/A	ft			p4 =	0.630 ksf	
X8 =	N/A	ft					
Comb.9: (1.2 + 0.2SDS)D + f1L + pQE			[CBC Eq. 16-5]				
P9 =	23.9	k	2P/(XBf) =	2.40 ksf	p1 =	0.000 ksf	
M9 =	66.6	ft-k	P/A-M/S or 0 =	0.00 ksf	p2 =	0.058 ksf	
M9net =	186.3	ft-k			p3 =	1.137 ksf	
a9 =	7.78	ft			p4 =	2.397 ksf	
X9 =	6.66	ft					
Comb. 10: (1.2 + 0.2SDS)D + f1L - pQE			[CBC Eq. 16-5]				
P10 =	23.9	k	2P/(XBf) =	2.40 ksf	p1 =	2.397 ksf	
M10 =	(66.6)	ft-k	P/A-M/S or 0 =	0.00 ksf	p2 =	1.137 ksf	
M10net =	53.2	ft-k			p3 =	0.058 ksf	
a10 =	2.22	ft			p4 =	0.000 ksf	
X10 =	6.66	ft					
Comb. 11: 0.9D + pQE			[CBC Eq. 16-6]				
P11 =	14.2	k	2P/(XBf) =	10.35 ksf	p1 =	0.000 ksf	
M11 =	66.6	ft-k	P/A-M/S or 0 =	0.00 ksf	p2 =	0.000 ksf	
M11net =	137.4	ft-k			p3 =	0.000 ksf	
a11 =	9.70	ft			p4 =	10.348 ksf	
X11 =	0.91	ft					
Comb. 12: 0.9D - pQE			[CBC Eq. 16-6]				
P11 =	14.2	k	2P/(XBf) =	10.35 ksf	p1 =	10.348 ksf	
M11 =	(66.6)	ft-k	P/A-M/S or 0 =	0.00 ksf	p2 =	0.000 ksf	
M11net =	4.3	ft-k			p3 =	0.000 ksf	
a11 =	0.30	ft			p4 =	0.000 ksf	
X11 =	0.91	ft					



SHEAR WALL FOOTING DESIGN

Project : FS- Main Building

Project # : 25534

Shear Wall : D/S502

Engineer :

Shear Wall Location : C &D

Sheet :

Date : 7/3/2025

Checked :

Wall Type : Concrete

B. Longitudinal Reinforcing [Bottom]

Rebar clearance from the bottom : 3.00 in

$d_t = D_F - \text{clear} - 1" = 14 \text{ in}$

Assuming tension-controlled section

$\phi : 0.90$

Steel Strength (rebar), f_y : 60.00

$\beta_1 : 0.85$

$\rho_{MIN} : 0.0033333$

Left	Mu @ p2 (ft-k)	Soil Pressure
Comb. 6:	5.1	Case 1
Comb. 7:	4.4	Case 1
Comb. 8:	4.4	Case 1
Comb. 9:	0.0	Case 3B
Comb. 10:	27.2	Case 3A or 4A
Comb. 11:	0.0	Case 4B or 5B
Comb. 12:	39.9	Case 5A

Right	Mu @ p3 (ft-k)	Soil Pressure
Comb. 6:	5.1	Case 1
Comb. 7:	4.4	Case 1
Comb. 8:	4.4	Case 1
Comb. 9:	27.2	Case 3B or 4B
Comb. 10:	0.0	Case 3A
Comb. 11:	39.9	Case 5B
Comb. 12:	0.0	Case 4A or 5A

$\rho = 0.00128$ ACI 10.5.3

$AS_{REQ'D} = 0.86 \text{ in}^2$

Rebars provided 3- #7

$AS_{prov.} = 1.80 \text{ in}^2 < O.K. >$

$c = 1.39 \text{ in}$

$c / d_t = 0.099$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Spacing = 14.00 in

$\rho = 0.00128$ ACI 10.5.3

$AS_{REQ'D} = 0.86 \text{ in}^2$

Rebars provided 3- #7

$AS_{prov.} = 1.80 \text{ in}^2 < O.K. >$

$c = 1.39 \text{ in}$

$c / d_t = 0.099$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Spacing = 14.00 in

C. Longitudinal Reinforcing [Top] [Moments are due to the weight of footing and soil]

Left

Mu @ p2 = 8.36 ft-k

$\rho = 0.00026$ Temp. Steel

$AS_{REQ'D} = 0.58 \text{ in}^2$

Rebars provided 3- #7

$AS_{prov.} = 1.80 \text{ in}^2 < O.K. >$

$c = 1.39 \text{ in}$

$c / d_t = 0.099$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Spacing = 14.00 in

Right

Mu @ p3 = 8.36 ft-k

$\rho = 0.00026$ Temp. Steel

$AS_{REQ'D} = 0.58 \text{ in}^2$

Rebars provided 3- #7

$AS_{prov.} = 1.80 \text{ in}^2 < O.K. >$

$c = 1.39 \text{ in}$

$c / d_t = 0.099$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Spacing = 14.00 in

D. Transverse Reinforcing [Bottom]

$d_t = D_F - \text{clear} - 2" = 13 \text{ in}$

b = 12 in

Left

Mu @ p2 = 0.57 ft-k / ft

$\rho = 0.00006$ Temp. Steel

$AS_{REQ'D} = 0.19 \text{ in}^2$

Rebars provided #5 @ 18 o.c. < O.K. >

$AS_{prov.} = 0.20 \text{ in}^2 < O.K. >$

$c = 0.16 \text{ in}$

$c / d_t = 0.012$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Right

Mu @ p3 = 0.57 ft-k

$\rho = 0.00006$ Temp. Steel

$AS_{REQ'D} = 0.19 \text{ in}^2$

Rebars provided #5 @ 18 o.c. < O.K. >

$AS_{prov.} = 0.20 \text{ in}^2 < O.K. >$

$c = 0.16 \text{ in}$

$c / d_t = 0.012$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

E. Transverse Reinforcing [Top]

b = 12 in

Mu = 0.31 ft-k

$\rho = 0.00003$ Temp. Steel

$AS_{REQ'D} = 0.19 \text{ in}^2$

Rebars provided #5 @ 18 o.c. < O.K. >

As prov. = 0.20 in² < O.K. >

c = 0.16 in

c / d_t = 0.012

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.



SHEAR WALL FOOTING DESIGN

Project : FS- Main Building
Project # : 25534
Engineer : _____
Shear Wall : D/S502
Shear Wall Location : C &D

Sheet : _____
Date : 7/3/2025
Checked : _____
Wall Type: Concrete

STEP 5: CHECK FOR SHEAR

A. Longitudinal [The critital location for flexural shear is a distance "d" from the end of wall]

Steel Strength (rebar), *f_y* : 60.00

Rebar clearance from the bottom : 3.00 in
d_t = *D_F* - clear - 1" = 14 in
φ : 0.75

Left	<i>V_{u12}</i> (kips)	<i>M_{u12}</i> (ft-k)	<i>V_{ud}/M_u</i>	<i>φV_c</i> (kips)	
Comb. 6:	2.0	2.3	1.000	42.7	< O.K. >
Comb. 7:	1.7	2.0	1.000	42.7	< O.K. >
Comb. 8:	1.7	2.0	1.000	42.7	< O.K. >
Comb. 9:	(3.5)	0.0	N/A	41.4	< O.K. >
Comb. 10:	10.4	13.3	0.914	42.4	< O.K. >
Comb. 11:	(2.0)	0.0	N/A	41.4	< O.K. >
Comb. 12:	12.1	26.4	0.537	41.4	< O.K. >

Right	<i>V_{u34}</i> (kips)	<i>M_{u34}</i> (ft-k)	<i>V_{ud}/M_u</i>	<i>φV_c</i> (kips)	
Comb. 6:	2.0	2.3	1.000	42.7	< O.K. >
Comb. 7:	1.7	2.0	1.000	42.7	< O.K. >
Comb. 8:	1.7	2.0	1.000	42.7	< O.K. >
Comb. 9:	10.4	13.3	0.914	42.4	< O.K. >
Comb. 10:	(3.5)	0.0	N/A	41.4	< O.K. >
Comb. 11:	12.1	26.4	0.537	41.4	< O.K. >
Comb. 12:	(2.0)	0.0	N/A	41.4	< O.K. >

*Design shear, *V_u*, is N/A if *d* > *L_{F-Left}* or *L_{F-Right}*

Shear Capacity

Larger of
$$\left\{ \begin{aligned} \phi V_c &= \phi 2 \sqrt{f'_c} b d = 41.4 \text{ k} & b &= B_F & (\text{ACI Eq. 11-3}) \\ \phi V_c &= \phi \left(1.9 \sqrt{f'_c} + 2500 \rho_w \frac{V_u d}{M_u} \right) b_w d < 3.5 \sqrt{f'_c} b_w d & & & (\text{ACI Eq. 11-5}) \end{aligned} \right.$$

V_{ud}/M_u shall not be greather than 1.0

B. Transverse [The critital location for flexural shear is a distance "d" from the face of wall]

d_t = *D_F* - clear - 2" = 13 in
b = 12 in

Shear Capacity
$$\phi V_c = \phi 2 \sqrt{f'_c} b d = 13.8 \text{ k}$$

Left
V_u @ *p*2 = 0.05 k < O.K. >

Right
M_u @ *p*3 = 0.05 k < O.K. >

STEP 6: SUMMARY

FOOTING SIZE			LONGITUDINAL REINFORCEMENT			
Footing Width =	3.0	ft	Left		Right	
Footing Depth =	1.5	ft	Bottom:	3- #7	Bottom:	3- #7
Total Footing Length =	10.0	ft	Top:	3- #7	Top:	3- #7
Footing Length (Left) =	3.5	ft	TRANSVERSE REINFORCEMENT			
Footing Length (Right) =	3.5	ft	Left		Right	
Shear Wall Length, <i>L_w</i> =	3.0	ft	Bottom:	#5 @ 18 o.c.	Bottom:	#5 @ 18 o.c.
			Top:	#5 @ 18 o.c.	Top:	#5 @ 18 o.c.

SHEAR WALL FOOTING DESIGN

Project : FS- Main Building

Project # : 25534

Shear Wall : N/S

Engineer :

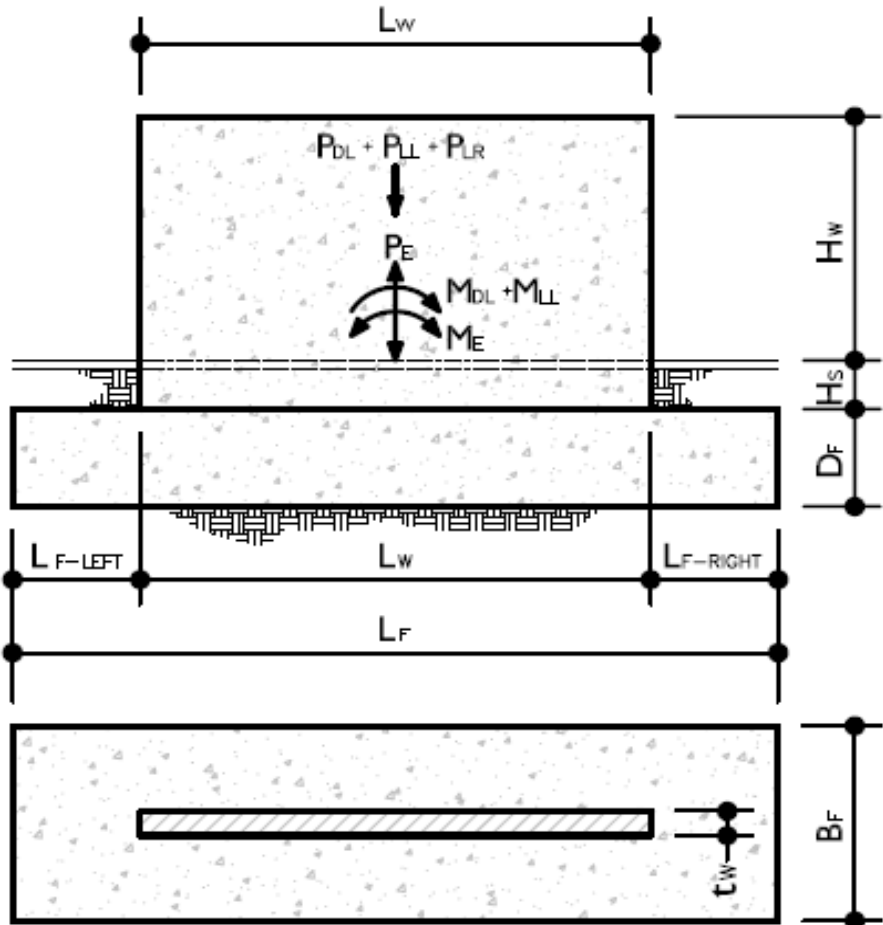
Shear Wall Location : E & F

Sheet :

Date : 7/3/2025

Checked :

Wall Type: Concrete



PLAN VIEW

STEP 1: GENERAL INPUT DATA

		Footing Properties:	
Redundancy Factor, ρ	1.30	Concrete Strength (footing), f'_c	3.00
S_{ds}	1.601 g	Footing Width, B_f	3.00
Allowable Soil Pressure q_a	3.00 ksf	Footing Depth, D_f	1.50
1/3 Increase Allowed?	YES	Footing Length Left, L_{f-left}	2.00
Net Allowable Pressure?	YES	Footing Length Right, $L_{f-right}$	2.00
Wall Properties:		Total Footing Length, L_f	18.0
Shear Wall Height, H_w	14.00 ft	Area of Footing, A_{FTG}	54.0
Shear Wall Length, L_w	14.00 ft	Section Modulus of Footing, S_{FTG}	162.0
Shear Wall Thickness, t_w	8.00 in	Unit Weight of Footing, γ_f	150
Unit Weight of Wall, γ_w	0 pcf	Footing Weight, W_f	12.2
Wall Weight, W_w	0.0 k	Soil Above Footing, H_s	1.00
Ω_o	2.50	Unit Weight of Soil, γ_s	100
Omega?	NO	Weight of Soil, W_s	5.4
		10% Reduction?	NO
		25% Reduction?	NO

STEP 2: APPLIED LOADS (At the Center of Wall)

	Seismic (E)	Dead (DL)	Live (LL)	Roof Live (LR)	
P =	0.0	8.7	0.0	0.0	k
M =	132.0	0.0	0.0	0.0	ft-k
V =	7.3	0.0	0.0	0.0	k
Mtot =	143.0	= M + V x Footing Depth			

STEP 3: FOOTING SIZE (Service Loads)

Comb. 1: D + L + Lr

[CBC Eq. 16-16]

$$\begin{aligned} P_1 &= (W_w + W_f + W_s + P_D) + P_L + P_{LR} = 26.3 \text{ k} \\ M_1 &= (W_w + P_D + P_L + P_{LR})(L_{f-left} + L_w/2 - L_f/2) + M_{DL} + M_{LL} + M_{RL} = 0.0 \text{ ft-k} \\ M_1 \text{ NET} &= (W_w + P_D + P_L + P_{LR})(L_{f-left} + L_w/2) + (W_f + W_s)(L_f/2) = \text{N/A} \text{ ft-k} \\ X_1 &= \text{N/A} \text{ ft} \end{aligned}$$

Allowable**	
P/A + M/S	0.49 ksf
P/A - M/S	0.49 ksf

<

3.25 ksf

< O.K. >

Comb. 2: D + L + $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-20]

$$\begin{aligned} P_2 &= (W_w + W_f + W_s + P_D) + P_L + \rho P_E/1.4 = 26.3 \text{ k} \\ M_2 &= [(W_w + P_D) + P_L + (\rho P_E/1.4)](L_{f-left} + L_w/2 - L_f/2) + M_{DL} + M_{LL} + \rho M_E/1.4 = 132.7 \text{ ft-k} \\ M_2 \text{ NET} &= [(W_w + P_D) + P_L + (\rho P_E/1.4)](L_{f-left} + L_w/2) + (W_f + W_s)(L_f/2) + M_{DL} + M_{LL} + \rho M_E/1.4 = 369.0 \text{ ft-k} \\ a_2 &= 14.06 \text{ ft} \\ X_2 &= 11.83 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)	1.48 ksf
P/A - M/S	0.00 ksf

<

4.33 ksf

< O.K. >

Comb. 3: D + L - $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-20]

$$\begin{aligned} P_3 &= (W_w + W_f + W_s + P_D) - P_L + \rho P_E/1.4 = 26.3 \text{ k} \\ M_2 &= [(W_w + P_D) + P_L - (\rho P_E/1.4)](L_{f-left} + L_w/2 - L_f/2) + M_{DL} + M_{LL} - \rho M_E/1.4 = (132.7) \text{ ft-k} \\ M_2 \text{ NET} &= [(W_w + P_D) + P_L - (\rho P_E/1.4)](L_{f-left} + L_w/2) + (W_f + W_s)(L_f/2) + M_{DL} + M_{LL} - \rho M_E/1.4 = 103.5 \text{ ft-k} \\ a_3 &= 3.94 \text{ ft} \\ X_3 &= 11.83 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)	1.48 ksf
P/A - M/S	0.00 ksf

<

4.33 ksf

< O.K. >

Comb. 4: 0.9D + $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-21]

$$\begin{aligned} P_4 &= (0.9)(W_w + W_f + W_s + P_D) + \rho P_E/1.4 = 23.6 \text{ k} \\ M_4 &= [(0.9)(W_w + P_D) + (\rho P_E/1.4)](L_{f-left} + L_w/2 - L_f/2) + (0.9)M_{DL} + \rho M_E/1.4 = 132.7 \text{ ft-k} \\ M_4 \text{ NET} &= [(0.9)(W_w + P_D) + (\rho P_E/1.4)](L_{f-left} + L_w/2) + (0.9)(W_f + W_s)(L_f/2) + (0.9)M_{DL} + \rho M_E/1.4 = 345.4 \text{ ft-k} \\ a_4 &= 14.62 \text{ ft} \\ X_4 &= 10.14 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)	1.55 ksf
P/A - M/S	0.00 ksf

<

4.33 ksf

< O.K. >

Comb. 5: 0.9D - $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-21]

$$\begin{aligned} P_5 &= (0.9)(W_w + W_f + W_s + P_D) - \rho P_E/1.4 = 23.6 \text{ k} \\ M_5 &= [(0.9)(W_w + P_D) - (\rho P_E/1.4)](L_{f-left} + L_w/2 - L_f/2) + (0.9)M_{DL} - \rho M_E/1.4 = (132.7) \text{ ft-k} \\ M_5 \text{ NET} &= [(0.9)(W_w + P_D) - (\rho P_E/1.4)](L_{f-left} + L_w/2) + (0.9)(W_f + W_s)(L_f/2) + (0.9)M_{DL} - \rho M_E/1.4 = 79.9 \text{ ft-k} \\ a_5 &= 3.38 \text{ ft} \\ X_5 &= 10.14 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)	1.55 ksf
P/A - M/S	0.00 ksf

<

4.33 ksf

< O.K. >



SHEAR WALL FOOTING DESIGN

Project : FS- Main Building
Project # : 25534 Engineer : _____
Shear Wall : N/S Shear Wall Location : E & F

Sheet : _____
Date : 7/3/2025
Checked : _____
Wall Type: Concrete

STEP 4: DESIGN FOR FLEXURE

A. Factored Soil Pressure

f₁ = 0.5

Comb. 6: 1.4D			[CBC Eq. 16-1]						
P6 =	36.8	k	P/A + M/S =	0.68	ksf	p1 =	0.681	ksf	
M6 =	0.0	ft-k	P/A - M/S =	0.68	ksf	p2 =	0.681	ksf	
M6net =	N/A	ft-k				p3 =	0.681	ksf	
a6 =	N/A	ft				p4 =	0.681	ksf	
X6 =	N/A	ft							

Comb. 7: 1.2D + 1.6L + 0.5Lr			[CBC Eq. 16-2]						
P7 =	31.5	k	P/A + M/S =	0.58	ksf	p1 =	0.583	ksf	
M7 =	0.0	ft-k	P/A - M/S =	0.58	ksf	p2 =	0.583	ksf	
M7net =	N/A	ft-k				p3 =	0.583	ksf	
a7 =	N/A	ft				p4 =	0.583	ksf	
X7 =	N/A	ft							

Comb. 8: 1.2D + f ₁ L + 1.6Lr			[CBC Eq. 16-3]						
P8 =	31.5	k	P/A + M/S =	0.58	ksf	p1 =	0.583	ksf	
M8 =	0.0	ft-k	P/A - M/S =	0.58	ksf	p2 =	0.583	ksf	
M8net =	N/A	ft-k				p3 =	0.583	ksf	
a8 =	N/A	ft				p4 =	0.583	ksf	
X8 =	N/A	ft							

Comb.9: $(1.2 + 0.2S_{DS})D + f_1L + \rho Q_E$		[CBC Eq. 16-5]					
P9 =	39.9	k	$2P/(XBf) =$	2.04	ksf		p1 = 0.000 ksf
M9 =	185.8	ft-k	$P/A-M/S \text{ or } 0 =$	0.00	ksf		p2 = 0.000 ksf
M9net =	545.0	ft-k					p3 = 1.728 ksf
a9 =	13.66	ft					p4 = 2.042 ksf
X9 =	13.03	ft					

O.K.

O.K.

Comb. 10: $(1.2 + 0.2S_{DS})D + f_1L - \rho Q_E$		[CBC Eq. 16-5]									
	P10 =	39.9	k		2P/(XBf) =	2.04	ksf		p1 =	2.042	ksf
	M10 =	(185.8)	ft-k		P/A-M/S or 0 =	0.00	ksf		p2 =	1.728	ksf
	M10net =	173.3	ft-k						p3 =	0.000	ksf
	a10 =	4.34	ft						p4 =	0.000	ksf
	X10 =	13.03	ft								

Comb. 11: 0.9D + p_{QE}			[CBC Eq. 16-6]						
P11 =	23.6	k	2P/(XBf) =	4.63	ksf	p1 =	0.000	ksf	
M11 =	185.8	ft-k	P/A-M/S or 0 =	0.00	ksf	p2 =	0.000	ksf	
M11net =	398.5	ft-k				p3 =	1.908	ksf	
<div>O.K.</div> a11 =	16.87	ft				p4 =	4.630	ksf	
X11 =	3.40	ft							

O.K.

Comb. 12: 0.9D - ρQ_E		[CBC Eq. 16-6]					
P11 =	23.6	k	2P/(XBf) =	4.63	ksf		p1 = 4.630 ksf
M11 =	(185.8)	ft-k	P/A-M/S or 0 =	0.00	ksf		p2 = 1.908 ksf
M11net =	26.8	ft-k					p3 = 0.000 ksf
<div></div> a11 =	1.13	ft					p4 = 0.000 ksf
X11 =	3.40	ft					



SHEAR WALL FOOTING DESIGN

Project : FS- Main Building
Project # : 25534
Engineer : _____
Shear Wall : N/S
Shear Wall Location : E &F

Sheet : _____
Date : 7/3/2025
Checked : _____
Wall Type: Concrete

B. Longitudinal Reinforcing [Bottom]

Rebar clearance from the bottom : 3.00 in
 $d_t = D_F - \text{clear} - 1" = 14$ in

Assuming tension-controlled section
 ϕ : 0.90

Steel Strength (rebar), f_y : 60.00
 β_1 : 0.85
 ρ_{MIN} : 0.0033333

Left	Mu @ p2 (ft-k)	Soil Pressure
Comb. 6:	1.4	Case 1
Comb. 7:	1.2	Case 1
Comb. 8:	1.2	Case 1
Comb. 9:	0.0	Case 4B or 5B
Comb. 10:	8.7	Case 3A or 4A
Comb. 11:	0.0	Case 4B or 5B
Comb. 12:	20.6	Case 3A or 4A

Right	Mu @ p3 (ft-k)	Soil Pressure
Comb. 6:	1.4	Case 1
Comb. 7:	1.2	Case 1
Comb. 8:	1.2	Case 1
Comb. 9:	8.7	Case 3B or 4B
Comb. 10:	0.0	Case 4A or 5A
Comb. 11:	20.6	Case 3B or 4B
Comb. 12:	0.0	Case 4A or 5A

$\rho = 0.00065$ Temp. Steel
 $A_{S\ REQ'D} = 0.58$ in²
Rebars provided 3- #7
 $A_{S\ prov.} = 1.80$ in² < O.K. >
 $c = 1.39$ in
 $c / d_t = 0.099$
 $\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$
Tension Control Assumption is Correct.
Spacing = 14.00 in

$\rho = 0.00065$ Temp. Steel
 $A_{S\ REQ'D} = 0.58$ in²
Rebars provided 3- #7
 $A_{S\ prov.} = 1.80$ in² < O.K. >
 $c = 1.39$ in
 $c / d_t = 0.099$
 $\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$
Tension Control Assumption is Correct.
Spacing = 14.00 in

C. Longitudinal Reinforcing [Top] [Moments are due to the weight of footing and soil]

Left
 $M_u @ p2 = 2.73$ ft-k
 $\rho = 0.00009$ Temp. Steel
 $A_{S\ REQ'D} = 0.58$ in²
Rebars provided 3- #7
 $A_{S\ prov.} = 1.80$ in² < O.K. >
 $c = 1.39$ in
 $c / d_t = 0.099$
 $\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$
Tension Control Assumption is Correct.
Spacing = 14.00 in

Right
 $M_u @ p3 = 2.73$ ft-k
 $\rho = 0.00009$ Temp. Steel
 $A_{S\ REQ'D} = 0.58$ in²
Rebars provided 3- #7
 $A_{S\ prov.} = 1.80$ in² < O.K. >
 $c = 1.39$ in
 $c / d_t = 0.099$
 $\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$
Tension Control Assumption is Correct.
Spacing = 14.00 in

D. Transverse Reinforcing [Bottom]

$d_t = D_F - \text{clear} - 2" = 13$ in

$b = 12$ in

Left
 $M_u @ p2 = 1.10$ ft-k / ft
 $\rho = 0.00012$ Temp. Steel
 $A_{S\ REQ'D} = 0.19$ in²
Rebars provided #5 @ 18 o.c. < O.K. >
 $A_{S\ prov.} = 0.20$ in² < O.K. >
 $c = 0.16$ in
 $c / d_t = 0.012$
 $\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$
Tension Control Assumption is Correct.

Right
 $M_u @ p3 = 1.10$ ft-k
 $\rho = 0.00012$ Temp. Steel
 $A_{S\ REQ'D} = 0.19$ in²
Rebars provided #5 @ 18 o.c. < O.K. >
 $A_{S\ prov.} = 0.20$ in² < O.K. >
 $c = 0.16$ in
 $c / d_t = 0.012$
 $\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$
Tension Control Assumption is Correct.

E. Transverse Reinforcing [Top]

$b = 12$ in

$M_u = 0.31$ ft-k
 $\rho = 0.00003$ Temp. Steel
 $A_{S\ REQ'D} = 0.19$ in²
Rebars provided #5 @ 18 o.c. < O.K. >

$A_{S\ prov.} = 0.20$ in² < O.K. >
 $c = 0.16$ in
 $c / d_t = 0.012$
 $\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$
Tension Control Assumption is Correct.



SHEAR WALL FOOTING DESIGN

Project : FS- Main Building
Project # : 25534
Engineer : _____
Shear Wall : N/S
Shear Wall Location : E &F

Sheet : _____
Date : 7/3/2025
Checked : _____
Wall Type: Concrete

STEP 5: CHECK FOR SHEAR

A. Longitudinal [The critital location for flexural shear is a distance "d" from the end of wall]

Steel Strength (rebar), f_y : 60.00

Rebar clearance from the bottom : 3.00 in
 $d_t = D_F - \text{clear} - 1" = 14$ in
 ϕ : 0.75

Left	V_{u12} (kips)	M_{u12} (ft-k)	V_{ud}/M_u	ϕV_c (kips)	
Comb. 6:	0.6	0.2	1.000	42.7	< O.K. >
Comb. 7:	0.5	0.2	1.000	42.7	< O.K. >
Comb. 8:	0.5	0.2	1.000	42.7	< O.K. >
Comb. 9:	(1.2)	0.0	N/A	41.4	< O.K. >
Comb. 10:	3.7	1.6	1.000	42.7	< O.K. >
Comb. 11:	(0.7)	0.0	N/A	41.4	< O.K. >
Comb. 12:	9.4	4.1	1.000	42.7	< O.K. >

Right	V_{u34} (kips)	M_{u34} (ft-k)	V_{ud}/M_u	ϕV_c (kips)	
Comb. 6:	0.6	0.2	1.000	42.7	< O.K. >
Comb. 7:	0.5	0.2	1.000	42.7	< O.K. >
Comb. 8:	0.5	0.2	1.000	42.7	< O.K. >
Comb. 9:	3.7	1.6	1.000	42.7	< O.K. >
Comb. 10:	(1.2)	0.0	N/A	41.4	< O.K. >
Comb. 11:	9.4	4.1	1.000	42.7	< O.K. >
Comb. 12:	(0.7)	0.0	N/A	41.4	< O.K. >

*Design shear, V_u , is N/A if $d > L_F\text{-Left}$ or $L_F\text{-Right}$

Shear Capacity

Larger of
$$\left\{ \begin{aligned} \phi V_c &= \phi 2 \sqrt{f'_c} b d = 41.4 \text{ k} & b &= B_F & \text{(ACI Eq. 11-3)} \\ \phi V_c &= \phi \left(1.9 \sqrt{f'_c} + 2500 \rho_w \frac{V_u d}{M_u} \right) b_w d < 3.5 \sqrt{f'_c} b_w d & & & \text{(ACI Eq. 11-5)} \end{aligned} \right.$$

V_{ud}/M_u shall not be greather than 1.0

B. Transverse [The critital location for flexural shear is a distance "d" from the face of wall]

$d_t = D_F - \text{clear} - 2" = 13$ in
 $b = 12$ in

Shear Capacity
$$\phi V_c = \phi 2 \sqrt{f'_c} b d = 13.8 \text{ k}$$

Left
 $V_u @ p2 = 0.13 \text{ k}$ < O.K. >

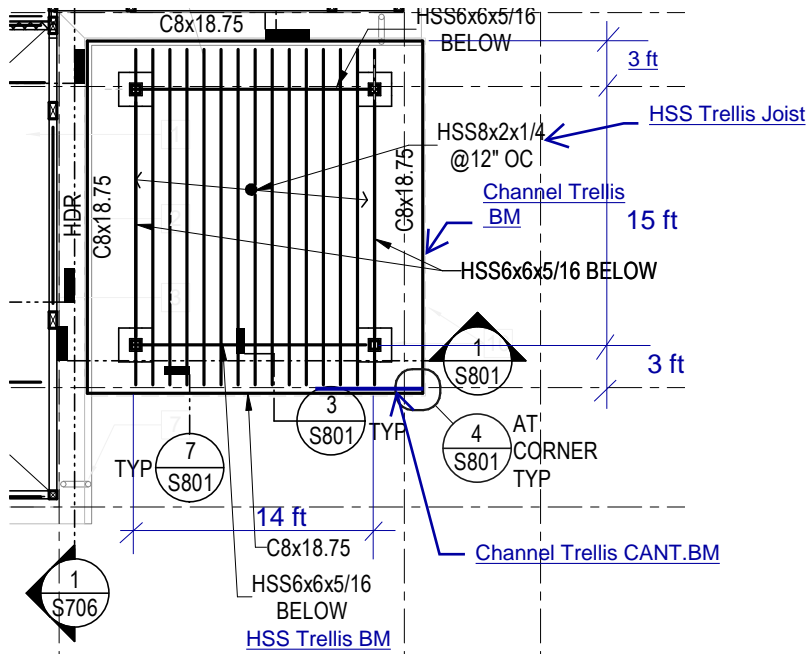
Right
 $M_u @ p3 = 0.13 \text{ k}$ < O.K. >

STEP 6: SUMMARY

FOOTING SIZE			LONGITUDINAL REINFORCEMENT			
Footing Width =	3.0	ft	<u>Left</u>		<u>Right</u>	
Footing Depth =	1.5	ft	Bottom:	3- #7	Bottom:	3- #7
Total Footing Length =	18.0	ft	Top:	3- #7	Top:	3- #7
Footing Length (Left) =	2.0	ft	<u>TRANSVERSE REINFORCEMENT</u>			
Footing Length (Right) =	2.0	ft				
Shear Wall Length, $L_w = 14.0$ ft			<u>Left</u>		<u>Right</u>	
			Bottom:	#5 @ 18 o.c.	Bottom:	#5 @ 18 o.c.
			Top:	#5 @ 18 o.c.	Top:	#5 @ 18 o.c.

4 MISCELLANEOUS

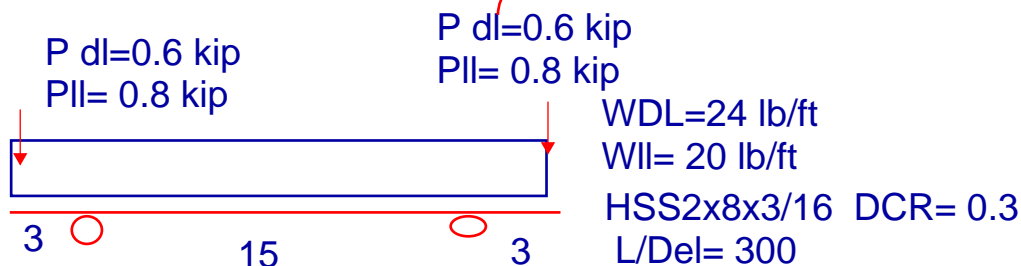
4.1 CANOPY DESIGN



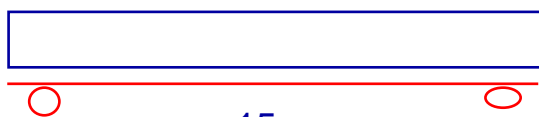
CANOPY JOISTS & BEAM KEY MAP

HSS 2x8 @ 12" o.c 24 psf
HSS 6x6 @ 14'-0" o.c $38/14 = 2.8$ psf
C8x @ 21'-0" o.c 1 psf
LL= 20 psf (conservatively)

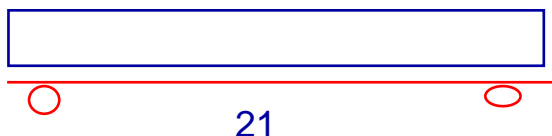
HSS Trellis Joist



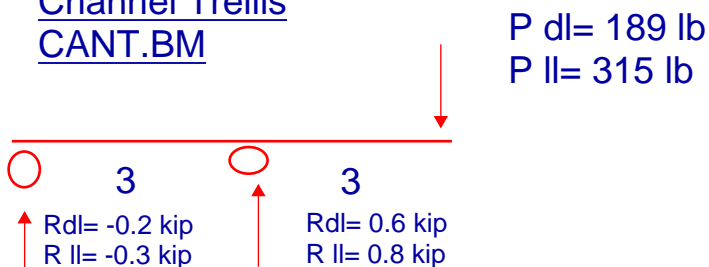
HSS Trellis BM



Channel Trellis BM



Channel Trellis CANT.BM



Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: trellis joist with point load from Cx

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

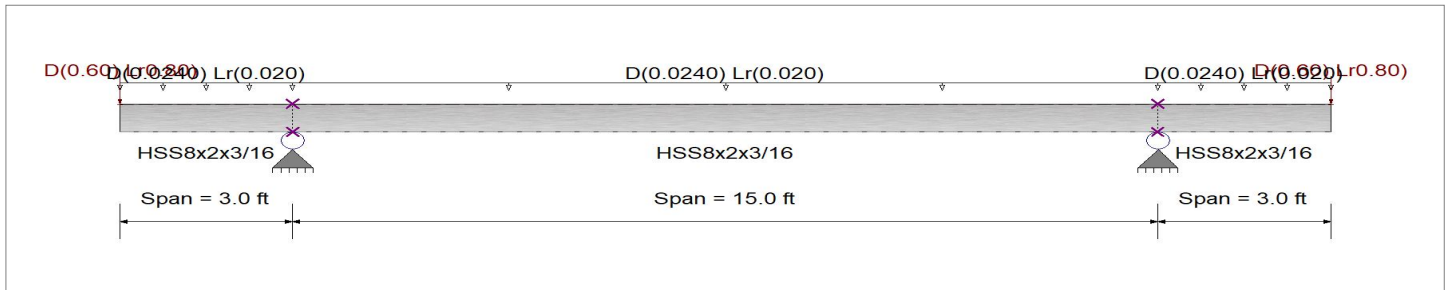
Analysis Method : Allowable Strength Design

Beam Bracing : Completely Unbraced

Bending Axis : Major Axis Bending

Fy : Steel Yield : 46.0 ksi

E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading

Load for Span Number 1

Uniform Load : D = 0.0240, Lr = 0.020 k/ft, Tributary Width = 1.0 ft

Point Load : D = 0.60, Lr = 0.80 k @ 0.0 ft

Load for Span Number 2

Uniform Load : D = 0.0240, Lr = 0.020 k/ft, Tributary Width = 1.0 ft

Load for Span Number 3

Uniform Load : D = 0.0240, Lr = 0.020 k/ft, Tributary Width = 1.0 ft

Point Load : D = 0.60, Lr = 0.80 k @ 3.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.258 : 1	Maximum Shear Stress Ratio =		0.036 : 1
Section used for this span		HSS8x2x3/16	Section used for this span		HSS8x2x3/16
Ma : Applied		4.452 k-ft	Va : Applied		1.568 k
Mn / Omega : Allowable		17.239 k-ft	Vn/Omega : Allowable		43.009 k
Load Combination		+D+Lr	Load Combination		+D+Lr
Span # where maximum occurs		Span # 2	Location of maximum on span		15.000 ft
Span # where maximum occurs		Span # 2	Span # where maximum occurs		Span # 2
Maximum Deflection					
Max Downward Transient Deflection		0.146 in	Ratio =	491	>=360
Max Upward Transient Deflection		-0.153 in	Ratio =	1,173	>=360
Max Downward Total Deflection		0.239 in	Ratio =	301	>=180
Max Upward Total Deflection		-0.238 in	Ratio =	755	>=180
			Span: 3 : Lr Only		
			Span: 3 : Lr Only		
			Span: 3 : +D+Lr		
			Span: 3 : +D+Lr		

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
D Only													
Dsgn. L = 3.00 ft	1	0.114	0.016		-1.96	1.96	28.79	17.24	1.00	1.00	0.71	71.82	43.01
Dsgn. L = 15.00 ft	2	0.114	0.016	-0.00	-1.96	1.96	28.79	17.24	1.54	1.00	0.71	71.82	43.01
Dsgn. L = 3.00 ft	3	0.114	0.016		-1.96	1.96	28.79	17.24	1.00	1.00	0.71	71.82	43.01
+D+Lr													
Dsgn. L = 3.00 ft	1	0.258	0.036		-4.45	4.45	28.79	17.24	1.00	1.00	1.57	71.82	43.01
Dsgn. L = 15.00 ft	2	0.258	0.036	-0.00	-4.45	4.45	28.79	17.24	1.32	1.00	1.57	71.82	43.01
Dsgn. L = 3.00 ft	3	0.258	0.036		-4.45	4.45	28.79	17.24	1.00	1.00	1.57	71.82	43.01
+D+0.750Lr													

ADDENDUM 5

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: trellis joist with point load from Cx

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
Dsgn. L = 3.00 ft	1		0.222	0.031		-3.83	3.83	28.79	17.24	1.00	1.00	1.35	71.82	43.01
Dsgn. L = 15.00 ft	2		0.222	0.031	-0.00	-3.83	3.83	28.79	17.24	1.34	1.00	1.35	71.82	43.01
Dsgn. L = 3.00 ft	3		0.222	0.031		-3.83	3.83	28.79	17.24	1.00	1.00	1.35	71.82	43.01
+0.60D														
Dsgn. L = 3.00 ft	1		0.068	0.010		-1.18	1.18	28.79	17.24	1.00	1.00	0.42	71.82	43.01
Dsgn. L = 15.00 ft	2		0.068	0.010	-0.00	-1.18	1.18	28.79	17.24	1.54	1.00	0.42	71.82	43.01
Dsgn. L = 3.00 ft	3		0.068	0.010		-1.18	1.18	28.79	17.24	1.00	1.00	0.42	71.82	43.01

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.2388	0.000		0.0000	0.000
	2	0.0000	0.000	+D+Lr	-0.2383	7.500
+D+Lr	3	0.2385	3.000		0.0000	7.500

Vertical Reactions

Load Combination	Support notation : Far left is #1				Values in KIPS
	Support 1	Support 2	Support 3	Support 4	
Max Upward from all Load Conditions		1.988	1.988		
Max Upward from Load Combinations		1.988	1.988		
Max Upward from Load Cases		1.010	1.010		
D Only		0.978	0.978		
+D+Lr		1.988	1.988		
+D+0.750Lr		1.735	1.735		
+0.60D		0.587	0.587		
Lr Only		1.010	1.010		

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: trellis beam

CODE REFERENCES

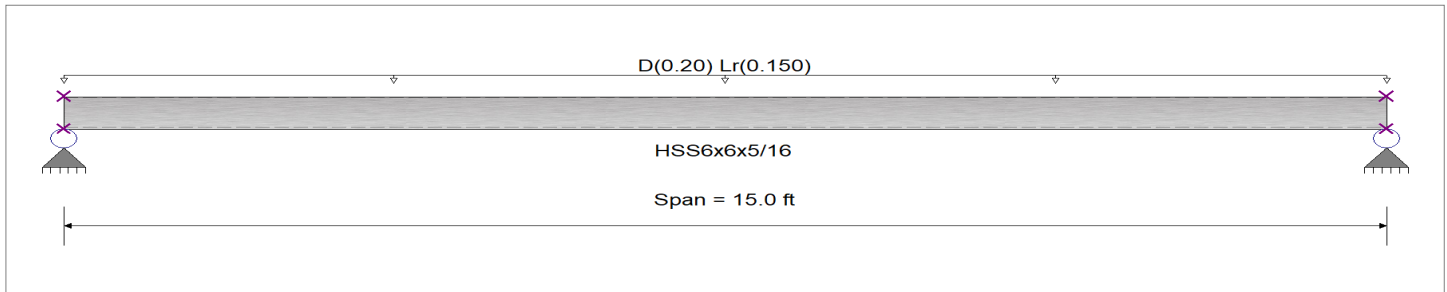
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 46.0 ksi
E : Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.20, Lr = 0.150 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.336 : 1	Maximum Shear Stress Ratio =		0.057 : 1
Section used for this span		HSS6x6x5/16	Section used for this span		HSS6x6x5/16
Ma : Applied		10.500 k-ft	Va : Applied		2.80 k
Mn / Omega : Allowable		31.218 k-ft	Vn/Omega : Allowable		49.315 k
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span		0.000 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		0.172 in	Ratio =		1,043 >=360
Max Upward Transient Deflection		0.000 in	Ratio =		0 <360
Max Downward Total Deflection		0.429 in	Ratio =		419 >=180
Max Upward Total Deflection		0.000 in	Ratio =		0 <180

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
D Only													
Dsgn. L = 15.00 ft	1	0.201	0.034	6.28		6.28	52.13	31.22	1.14	1.00	1.68	82.36	49.31
+D+Lr													
Dsgn. L = 15.00 ft	1	0.336	0.057	10.50		10.50	52.13	31.22	1.14	1.00	2.80	82.36	49.31
+D+0.750Lr													
Dsgn. L = 15.00 ft	1	0.303	0.051	9.45		9.45	52.13	31.22	1.14	1.00	2.52	82.36	49.31
+0.60D													
Dsgn. L = 15.00 ft	1	0.121	0.020	3.77		3.77	52.13	31.22	1.14	1.00	1.01	82.36	49.31

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.4295	7.543		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.800	2.800
Max Upward from Load Combinations	2.800	2.800
Max Upward from Load Cases	1.675	1.675
D Only	1.675	1.675
+D+Lr	2.800	2.800
+D+0.750Lr	2.519	2.519
+0.60D	1.005	1.005

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: trellis beam

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Lr Only	1.125	1.125

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercalc.ec6

LIC#: KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Trellis -Channel

CODE REFERENCES

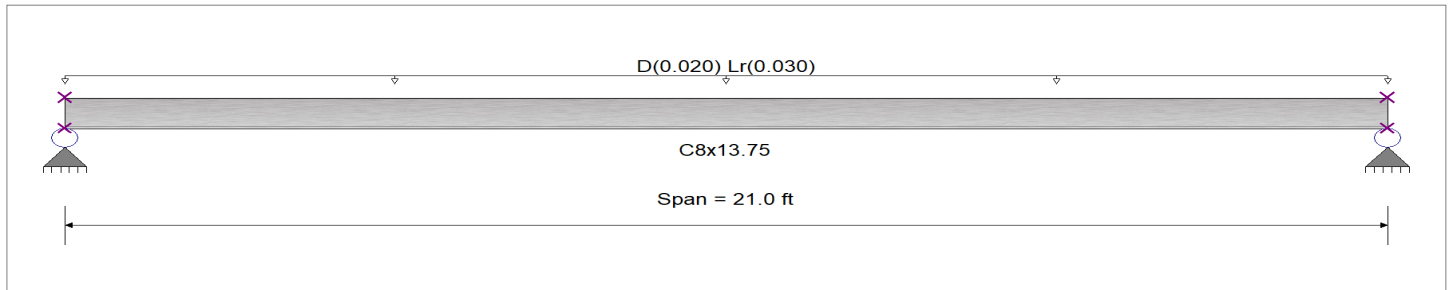
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 36.0 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 = 0.020, Lr = 0.030 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.413 : 1	Maximum Shear Stress Ratio =	0.017 : 1
Section used for this span	C8x13.75	Section used for this span	C8x13.75
Ma : Applied	2.756 k-ft	Va : Applied	0.5250 k
Mn / Omega : Allowable	6.678 k-ft	Vn / Omega : Allowable	31.352 k
Load Combination	+D+Lr	Load Combination	+D+Lr
Span # where maximum occurs	Span # 1	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	0.126 in Ratio = 2,000	>=360	
Max Upward Transient Deflection	0.000 in Ratio = 0	<360	Span: 1 : Lr Only
Max Downward Total Deflection	0.210 in Ratio = 1200	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0.000 in Ratio = 0	<180	

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
D Only													
Dsgn. L = 21.00 ft	1	0.165	0.007	1.10		1.10	11.15	6.68	1.14	1.00	0.21	52.36	31.35
+D+Lr													
Dsgn. L = 21.00 ft	1	0.413	0.017	2.76		2.76	11.15	6.68	1.14	1.00	0.53	52.36	31.35
+D+0.750Lr													
Dsgn. L = 21.00 ft	1	0.351	0.014	2.34		2.34	11.15	6.68	1.14	1.00	0.45	52.36	31.35
+0.60D													
Dsgn. L = 21.00 ft	1	0.099	0.004	0.66		0.66	11.15	6.68	1.14	1.00	0.13	52.36	31.35

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.2099	10.560		0.0000	0.000

Vertical Reactions

Support notation : Far left is #

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.525	0.525
Max Upward from Load Combinations	0.525	0.525
Max Upward from Load Cases	0.315	0.315
Max Downward from all Load Conditions (Resis		0.210
Max Downward from Load Combinations (Resis		0.210
Max Downward from Load Cases (Resisting Up		0.210
D Only	0.210	0.210

ADDENDUM 5

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Trellis -Channel

Load Combination	Support notation : Far left is #			Values in KIPS
	Support 1	Support 2		
+D+Lr	0.525	0.525	0.210	
+D+0.750Lr	0.446	0.446	0.210	
+0.60D	0.126	0.126	0.210	
Lr Only	0.315	0.315	0.210	

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: CANT Trellis -Channel

CODE REFERENCES

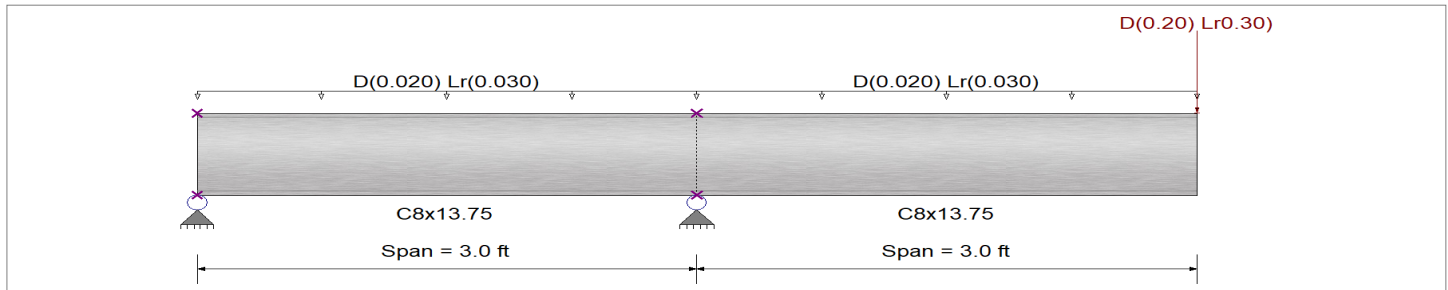
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 36.0 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 for Span Number 1

Uniform Load : D = 0.020, Lr = 0.030 k/ft, Tributary Width = 1.0 ft

Load for Span Number 2

Uniform Load : D = 0.020, Lr = 0.030 k/ft, Tributary Width = 1.0 ft

Point Load : D = 0.20, Lr = 0.30 k @ 3.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.089 : 1	Maximum Shear Stress Ratio =		0.021 : 1
Section used for this span		C8x13.75	Section used for this span		C8x13.75
Ma : Applied		1.725 k-ft	Va : Applied		0.650 k
Mn / Omega : Allowable		19.335 k-ft	Vn/Omega : Allowable		31.352 k
Load Combination		+D+Lr	Load Combination		+D+Lr
Span # where maximum occurs		Span # 2	Location of maximum on span		3.000 ft
Span # where maximum occurs		Span # 2	Span # where maximum occurs		Span # 1
Maximum Deflection					
Max Downward Transient Deflection		0.010 in	Ratio =	7,271	>=360
Max Upward Transient Deflection		0.000 in	Ratio =	0	<360
Max Downward Total Deflection		0.017 in	Ratio =	4363	>=180
Max Upward Total Deflection		-0.002 in	Ratio =	22967	>=180
			Span: 2 : Lr Only		
			Span: 2 : Lr Only		
			Span: 2 : +D+Lr		
			Span: 2 : +D+Lr		

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	
D Only													
Dsgn. L = 3.00 ft	1	0.035	0.008		-0.69	0.69	33.00	19.76	1.73	1.00	0.26	52.36	31.35
Dsgn. L = 3.00 ft	2	0.036	0.008		-0.69	0.69	32.29	19.34	1.00	1.00	0.26	52.36	31.35
+D+Lr													
Dsgn. L = 3.00 ft	1	0.087	0.021		-1.73	1.73	33.00	19.76	1.73	1.00	0.65	52.36	31.35
Dsgn. L = 3.00 ft	2	0.089	0.021		-1.73	1.73	32.29	19.34	1.00	1.00	0.65	52.36	31.35
+D+0.750Lr													
Dsgn. L = 3.00 ft	1	0.074	0.018		-1.47	1.47	33.00	19.76	1.73	1.00	0.55	52.36	31.35
Dsgn. L = 3.00 ft	2	0.076	0.018		-1.47	1.47	32.29	19.34	1.00	1.00	0.55	52.36	31.35
+0.60D													
Dsgn. L = 3.00 ft	1	0.021	0.005		-0.41	0.41	33.00	19.76	1.73	1.00	0.16	52.36	31.35
Dsgn. L = 3.00 ft	2	0.021	0.005		-0.41	0.41	32.29	19.34	1.00	1.00	0.16	52.36	31.35

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: CANT Trellis -Channel

Overall Maximum Deflections

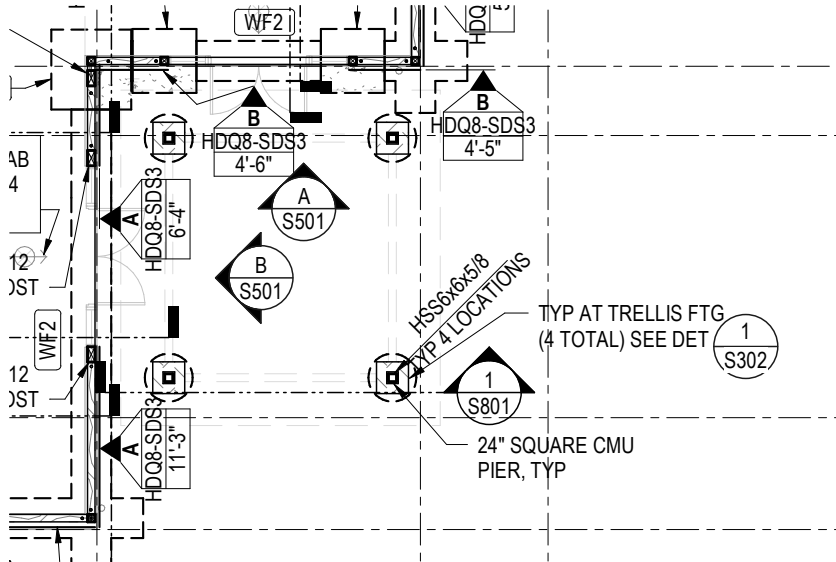
Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+Lr	-0.0016	1.752
+D+Lr	2	0.0165	3.000		0.0000	1.752

Vertical Reactions

Support notation : Far left is #'

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions		1.300	
Max Upward from Load Combinations		1.300	
Max Upward from Load Cases		0.780	
Max Downward from all Load Conditions (Resis	-0.500		
Max Downward from Load Combinations (Resi:	-0.500		
Max Downward from Load Cases (Resisting Up	-0.300		
D Only	-0.200	0.520	
+D+Lr	-0.500	1.300	
+D+0.750Lr	-0.425	1.105	
+0.60D	-0.120	0.312	
Lr Only	-0.300	0.780	



CANOPY HSS COL & FOOTING KEY MAP

3.) Determine Weight of Structure:

$$\text{HSS } 8 \times 2 \times 1/4: 15.6 \text{ PLF } (21 \text{ ft}) (15) = 4914 \text{ \#}$$

$$\text{HSS } 6 \times 6 \times 5/16: 23.34 \text{ PLF } [(15 \text{ ft}) (4)] = 1400 \text{ lb}$$

$$\text{C8x18.75} : 18.75 \text{ PLF } (21 + 20) (2) = 1537 \text{ \#}$$

$$\text{HSS } 6 \times 6 \times 5/8: 42 \text{ lb/ft } * (5 \text{ ft}) * 4 = 840 \text{ lb}$$

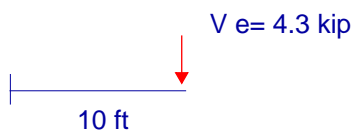
$$\Sigma \text{DL} = 8691 \text{ \#}$$

$$\delta_{\text{DL}} = \Sigma \text{DL} / (20 \text{ ft} * 21 \text{ ft}) = 20.6 \text{ psf}$$

4.) Design Steel Column:

$$V = 2.0 * 8691 \text{ lb} = 17382 \text{ lb}$$

$$V \text{ per col} = 17382 \text{ lb} / 4 = 4.3 \text{ kip}$$



$$\delta_x = \frac{C_d \delta_{xe}}{I_e} = 1.25 * 1.4" / 1.5 = 1.16" < 0.015 * 10 * 12 = 1.8" \text{ OK}$$

HSS 6x6x5/8 ok

Steel Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Trellis Cant Col

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

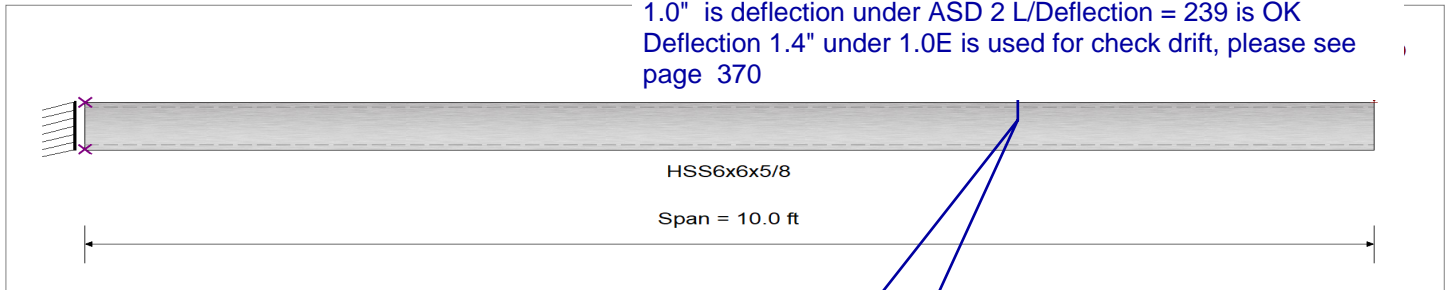
Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 46.0 ksi
 E : Modulus : 29,000.0 ksi

1.4 " is deflection under strength level 1.0E
 1.0" is deflection under ASD 2 L/Deflection = 239 is OK
 Deflection 1.4" under 1.0E is used for check drift, please see page 370



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 = 4.0 k @ 10.0 ft

DESIGN SUMMARY

Maximum Bending Stress Ratio =

Section used for this span

Ma : Applied

Mn / Omega : Allowable

Load Combination

Span # where maximum occurs

HSS6x6x5/8

28.000 k-ft

53.253 k-ft

E Only * 0.70

Span # 1

Maximum Shear Stress Ratio =

Section used for this span

Va : Applied

Vn/Omega : Allowable

Load Combination

Location of maximum on span

Span # where maximum occurs

Design N.G.

0.034 : 1

HSS6x6x5/8

2.80 k

81.753 k

E Only * 0.70

0.000 ft

Span # 1

Maximum Deflection

Max Downward Transient Deflection

Max Upward Transient Deflection

Max Downward Total Deflection

Max Upward Total Deflection

1.430 in Ratio =

0.000 in Ratio =

1.005 in Ratio =

0.000 in Ratio =

167 < 360

0 < 360

239 >= 180

0 < 180

SEE PAGE 370 FOR CHECK DRIFT

Span: 1 : E Only

Span: 1 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
			M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 10.00 ft		1		0.000				88.93	53.25	1.00	1.00	-0.00	136.53	81.75
E Only * 0.70														
Dsgn. L = 10.00 ft		1	0.526	0.034		-28.00	28.00	88.93	53.25	1.00	1.00	2.80	136.53	81.75
E Only * 0.5250														
Dsgn. L = 10.00 ft		1	0.394	0.026		-21.00	21.00	88.93	53.25	1.00	1.00	2.10	136.53	81.75

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
E Only	1	1.4362	10.000		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	4.000	1.010
Max Upward from Load Combinations		1.010
Max Upward from Load Cases	4.000	1.010
Max Downward from all Load Conditions (Resist)		1.010
Max Downward from Load Combinations (Resist)		1.010
Max Downward from Load Cases (Resisting Up)		1.010
E Only * 0.70	2.800	371.010

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Trellis Cant Col

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
E Only * 0.5250	2.100	1.010
E Only	4.000	1.010

BASE SHEAR

2022 CBC / ASCE 7-16 Equivalent Lateral Force Procedure

Project: **Fire Station**
SBI Job No.: **14608**

Building: **Entry Canopy**
LFRS: **OCCS**
Direction: **X**

Building Data

Occupancy Category = **IV** Table 1604.5, 2013 CBC Importance Factor, I_e = **1.50** Table 1.5-2

Seismic Ground Motion Values Section 11.4

S_s = **2.608** From geotech or Figs.22-1 to 11
 S_1 = **0.986** From geotech or Figs.22-1 to 11
Site Class = **D** From geotech or Table 20.3-1
 T_L = **8 sec** Figs.22-12 to 16

F_a = **1.00** Table 11.4-1
 F_v = **1.50** Table 11.4-2
 $S_{MS} = F_a S_s$ = **2.608** Eq. 11.4-1
 $S_{M1} = F_v S_1$ = **1.479** Eq. 11.4-2
 $S_{DS} = (2/3) S_{MS}$ = **1.739** Eq. 11.4-3
 $S_{D1} = (2/3) S_{M1}$ = **0.986** Eq. 11.4-4

Seismic Design Category

F Tables 11.6-1 & 2

Building Period Section 12.8.2

C_t = **0.02** Table 12.8-2
 α = **0.75** Table 12.8-2
 h_n = **11.00 ft** Height of Building
 T_b = **0.000 sec** From Analysis
(Input zero to use T_a)

$T_a = C_t h_n^\alpha$ = **0.121 sec** Eq. 12.8-7
 C_u = **1.40** Table 12.8-1
 $T_{a, max} = C_u T_a$ = **0.169 sec** Section. 12.8.2

Period = **0.121 sec** «-- used for design
0.121 sec «-- used for drift
Section. 12.8.6.2

Base Shear Section 12.8

W = **6 kips** Total Structure Weight
 R = **1.25** Table 12.2-1
 C_d = **1.25** Table 12.2-1

For Design Only

For Design

with $S_{ds} = 1.6$, $C_s = 1.92$

For Drift Only

C_s = $S_{DS} / (R/I_e) = 2.086$ Eq. 12.8-2
 $C_{s, max}$ = $S_{D1} / [T (R/I_e)] = 9.795$ Eq. 12.8-3, for $T \leq T_L$
 $C_{s, max}$ = $S_{D1} T_L / [T^2 (R/I_e)] = N/A$ Eq. 12.8-4, for $T > T_L$
 $C_{s, min}$ = $\max\{0.01, 0.044 S_{DS} I_e\} = N/A$ Eq. 12.8-5, if $S_1 < 0.6g$
 $C_{s, min}$ = $\max\{0.01, 0.044 S_{DS} I_e, 0.5 S_1 / (R/I_e)\} = 0.592$ Eq. 12.8-5 or 12.8-6, if $S_1 \geq 0.6g$

$C_{s, max} = 2.086$
 $C_s = 9.795$
 $C_s = N/A$
 $C_{s, min} = N/A$
 $C_{s, min} = 0.592$

Use, $C_s = 2.086$

Use, $C_s = 2.086$

$V_{design} = C_s W = 13 \text{ kips}$ Eq. 12.8-1

$V_{drift} = C_s W = 13 \text{ kips}$
Allowable Drift = **0.010** h_{sx}
Table 12.12-1

* Note: All references are from ASCE 7-10 unless noted otherwise.

ADDENDUM 5

Steel Base Plate

Project File: FS 46 enercalc.ec6
(c) ENERCALC INC 1983-2022

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

Code Reference

Calculations per AISC Design Guide # 1, IBC 2018, CBC 2019, ASCE 7-16, AISC 360-16
Load Combination Set : IBC 2021

General Information

Material Properties

AISC Design Method	Load Resistance Factor Design	ϕ_c : LRFD Resistance Factor	0.65
Steel Plate Fy	= 50.0 ksi		
Concrete Support f'c	= 3 ksi		
Assumed Bearing Area	Full Bearing	Nominal Bearing Fp per J8	2.550 ksi

Column & Plate

Column Properties

Steel Section	HSS6x6x5/8		
Depth	6 in	Area	11.7 in ²
Width	6 in	Ixx	55.2 in ⁴
Flange Thickness	0.581 in	Iyy	55.2 in ⁴
Web Thickness	0 in		

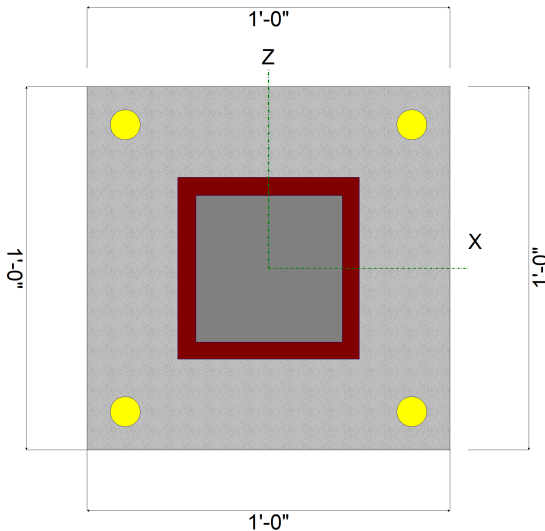
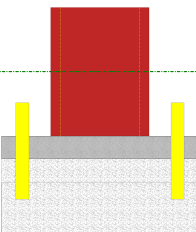
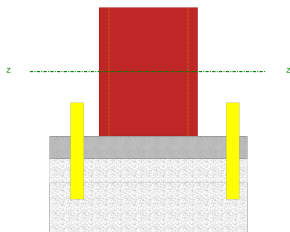
Plate Dimensions

N : Length	12 in
B : Width	12 in
Thickness	1.375 in

Support Dimensions

Width along "X"	12 in
Length along "Z"	12 in

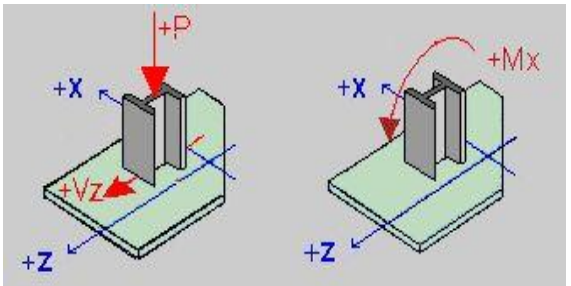
Column assumed welded to base plate.



Applied Loads

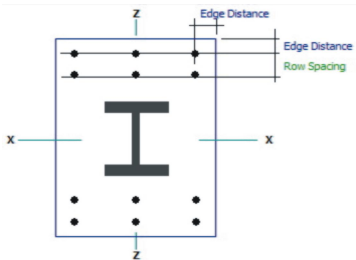
	P-Y	V-Z	M-X
D : Dead Load	2.0 k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	2.0 k	k	k-ft
S : Snow	k	k	k-ft
W : Wind	k	k	k-ft
E : Earthquake	k	4.0 k	40.0 k-ft
H : Lateral Earth	k	k	k-ft

" 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	1 1/2"
Max of Tension or Pullout Capacity.....	k
Shear Capacity.....	k
Edge distance : bolt to plate.....	1.25 in
Number of Bolts in each Row.....	2
Number of Bolt Rows.....	1



Project Title:
Engineer:
Project ID:
Project Descr:

Steel Base Plate

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method **Load Resistance Factor Design**
Governing Load Combination **+1.20D+E**
Governing Load Case Type **Axial + Moment, L/2 < Eccentricity, Tension <**
Governing STRESS RATIO **1.0**
Design Plate Size **1'-0" x 1'-0" x 1'-3/8"**
Pu : Axial 0.000 k
Mu : Moment 0.000 k-ft

Mu : Max. Moment 13.254 k-in
fb : Max. Bending Stress 42.061 ksi
Fb : Allowable : 45.000 ksi
Fy * Phi
Bending Stress Ratio **0.935**
Bending Stress OK
fu : Max. Plate Bearing Stress 1.658 ksi
Fp : Allowable : 1.658 ksi
Bearing Stress Ratio **1.000**
Bearing Stress OK

Load Comb. : +1.40D

Axial Load Only, No Moment

Loading

Pu : Axial 2.800 k
Design Plate Height 12.000 in
Design Plate Width 12.000 in
Will be different from entry if partial bearing used.
A1 : Plate Area 144.000 in^2
A2: Support Area 144.000 in^2
sqrt(A2/A1) 1.000

Distance for Moment Calculation

" m " 3.150 in
" n " 3.150 in
X 0.000 in^2
Lambda 0.000
n' 0.000 in
n' * Lambda 0.000 in
L = max(m, n, n") 3.150 in

Bearing Stresses

Fp : Allowable 1.658 ksi
fu : Max. Bearing Pressure 0.019 ksi
Stress Ratio 0.012

Plate Bending Stresses

Mmax = Fu * L^2 / 2 0.096 k-in on 1" strip)
fb : Actual 0.204 ksi
Fb : Allowable 45.000 ksi
Stress Ratio 0.005

Load Comb. : +1.20D+0.50Lr

Axial Load Only, No Moment

Loading

Pu : Axial 3.400 k
Design Plate Height 12.000 in
Design Plate Width 12.000 in
Will be different from entry if partial bearing used.
A1 : Plate Area 144.000 in^2
A2: Support Area 144.000 in^2
sqrt(A2/A1) 1.000

Distance for Moment Calculation

" m " 3.150 in
" n " 3.150 in
X 0.000 in^2
Lambda 0.000
n' 0.000 in
n' * Lambda 0.000 in
L = max(m, n, n") 3.150 in

Bearing Stresses

Fp : Allowable 1.658 ksi
fu : Max. Bearing Pressure 0.024 ksi
Stress Ratio 0.014

Plate Bending Stresses

Mmax = Fu * L^2 / 2 0.117 k-in on 1" strip)
fb : Actual 0.248 ksi
Fb : Allowable 45.000 ksi
Stress Ratio 0.006

ADDENDUM 5

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Base Plate

Project File: FS 46 enercalc.ec6

LIC#: KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

Load Comb. : +1.20D

Axial Load Only, No Moment

Loading

Pu : Axial 2.400 k
Design Plate Height 12.000 in
Design Plate Width 12.000 in
Will be different from entry if partial bearing used.
A1 : Plate Area 144.000 in²
A2: Support Area 144.000 in²
sqrt(A2/A1) 1.000

Distance for Moment Calculation

" m " 3.150 in
" n " 3.150 in
X 0.000 in²
Lambda 0.000
n' 0.000 in
n' * Lambda 0.000 in
L = max(m, n, n") 3.150 in

Bearing Stresses

Fp : Allowable 1.658 ksi
fu : Max. Bearing Pressure 0.017 ksi
Stress Ratio **0.010**

Plate Bending Stresses

Mmax = Fu * L² / 2 0.083 k-in on 1" strip)
fb : Actual 0.175 ksi
Fb : Allowable 45.000 ksi
Stress Ratio **0.004**

Load Comb. : +1.20D+1.60Lr

Axial Load Only, No Moment

Loading

Pu : Axial 5.600 k
Design Plate Height 12.000 in
Design Plate Width 12.000 in
Will be different from entry if partial bearing used.
A1 : Plate Area 144.000 in²
A2: Support Area 144.000 in²
sqrt(A2/A1) 1.000

Distance for Moment Calculation

" m " 3.150 in
" n " 3.150 in
X 0.000 in²
Lambda 0.000
n' 0.000 in
n' * Lambda 0.000 in
L = max(m, n, n") 3.150 in

Bearing Stresses

Fp : Allowable 1.658 ksi
fu : Max. Bearing Pressure 0.039 ksi
Stress Ratio **0.023**

Plate Bending Stresses

Mmax = Fu * L² / 2 0.193 k-in on 1" strip)
fb : Actual 0.408 ksi
Fb : Allowable 45.000 ksi
Stress Ratio **0.009**

Load Comb. : +1.20D+E

Axial Load + Moment, Ecc. > L/2

Loading

Pu : Axial 2.400 k
Mu : Moment 40.000 k-ft
Eccentricity 200.000 in
A1 : Plate Area 144.000 in²
A2 : Support Area 144.000 in²
sqrt(A2/A1) 1.000

Calculate plate moment from bearing . . .

max(m, n) 3.600 in
"A" : Bearing Length 5.552 in
Mpl : Plate Moment 0.702 k-in

Calculate plate moment from bolt tension . . .

Tension per Bolt 26.409 k
Tension : Allowable 0.000 k
Stress Ratio **0.000**
Dist. from Bolt to Col. Edge 1.900 in
Effective Bolt Width for Bending 7.600 in
Plate Moment from Bolt Tension 13.205 k-in

Bearing Stresses

Fp : Allowable 1.658 ksi
fu : Max. Bearing Pressure (set equal to Fp)
Stress Ratio **1.000**

Plate Bending Stresses

Mmax 13.205 k-in on 1" strip)
fb : Actual 41.906 ksi
Fb : Allowable 45.000 ksi
Stress Ratio **0.931**

Steel Base Plate

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

Load Comb. : +0.90D

Axial Load Only, No Moment

Loading

Pu : Axial	1.800 k
Design Plate Height	12.000 in
Design Plate Width	12.000 in
<i>Will be different from entry if partial bearing used.</i>	
A1 : Plate Area	144.000 in ²
A2: Support Area	144.000 in ²
sqrt(A2/A1)	1.000

Distance for Moment Calculation

" m "	3.150 in
" n "	3.150 in
X	0.000 in ²
Lambda	0.000
n'	0.000 in
n' * Lambda	0.000 in
L = max(m, n, n")	3.150 in

Bearing Stresses

Fp : Allowable	1.658 ksi
fu : Max. Bearing Pressure	0.013 ksi
Stress Ratio	0.008

Plate Bending Stresses

Mmax = Fu * L ² / 2	0.062 k-in on 1" strip)
fb : Actual	0.131 ksi
Fb : Allowable	45.000 ksi
Stress Ratio	0.003

Load Comb. : +0.90D+E

Axial Load + Moment, Ecc. > L/2

Loading

Pu : Axial	1.800 k
Mu : Moment	40.000 k-ft
Eccentricity	266.667 in
A1 : Plate Area	144.000 in ²
A2 : Support Area	144.000 in ²
sqrt(A2/A1)	1.000

Calculate plate moment from bearing ...

max(m, n)	3.600 in
"A" : Bearing Length	5.512 in
Mpl : Plate Moment	0.700 k-in

Calculate plate moment from bolt tension ...

Tension per Bolt	26.507 k
Tension : Allowable	0.000 k
Stress Ratio	0.000
Dist. from Bolt to Col. Edge	1.900 in
Effective Bolt Width for Bending	7.600 in
Plate Moment from Bolt Tension	13.254 k-in

Bearing Stresses

Fp : Allowable	1.658 ksi
fu : Max. Bearing Pressure	(set equal to Fp)
Stress Ratio	1.000

Plate Bending Stresses

Mmax	13.254 k-in on 1" strip)
fb : Actual	42.061 ksi
Fb : Allowable	45.000 ksi
Stress Ratio	0.935

Pole Footing Embedded in Soil

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Trellis Column Pole Footing

Code References

Calculations per IBC 2018 1807.3, CBC 2019, ASCE 7-16

Load Combinations Used : IBC 2021

General Information

Pole Footing Shape Circular
 Pole Footing Diameter 36.0 in
 Calculate Min. Depth for Allowable Pressures
 No Lateral Restraint at Ground Surface
 Allow Passive 350.0 pcf
 Max Passive 1,500.0 psf

Controlling Values

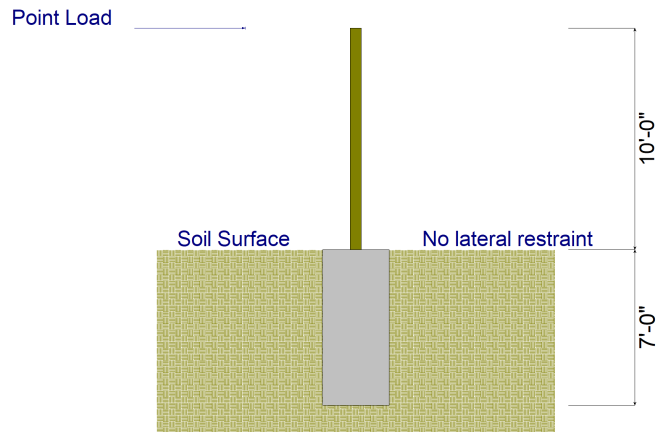
Governing Load Combination E Only * 0.70
 Lateral Load 2.80 k
 Moment 28.0 k-ft

NO Ground Surface Restraint

Pressures at 1/3 Depth
 Actual 809.09 psf
 Allowable 809.59 psf

Minimum Required Depth 7.0 ft

Footing Base Area 7.069 ft²
 Maximum Soil Pressure 0.0 ksf



Applied Loads

Lateral Concentrated Load (k)		Lateral Distributed Loads (klf)			Vertical Load (k)
D : Dead Load	0.0 k	0.0	0.0	0.0 k/ft	0.0 k
Lr : Roof Live	0.0 k	0.0	0.0	0.0 k/ft	0.0 k
L : Live	0.0 k	0.0	0.0	0.0 k/ft	0.0 k
S : Snow	0.0 k	0.0	0.0	0.0 k/ft	0.0 k
W : Wind	0.0 k	0.0	0.0	0.0 k/ft	0.0 k
E : Earthquake	4.0 k	0.0	0.0	0.0 k/ft	0.0 k
H : Lateral Earth	0.0 k	0.0	0.0	0.0 k/ft	0.0 k
Load distance above ground surface	10.0 ft	TOP of Load above ground surface			
		0.0	0.0	0.0 ft	
		BOTTOM of Load above ground surface			
		0.0	0.0	0.0 ft	

Load Combination Results

Load Combination	Forces @ Ground Surface		Required Depth - (ft)	Pressure at 1/3 Depth		Soil Increase Factor
	Loads - (k)	Moments - (ft-k)		Actual - (psf)	Allow - (psf)	
	0.000	0.000	0.13	0.0	0.0	1.000
E Only * 0.70	2.800	28.000	7.00	809.1	809.6	1.000
E Only * 0.5250	2.100	21.000	6.25	725.3	725.4	1.000

FOOTING REINFORCING

$$\Omega_o = 1.25$$

$$(1.2 + 0.2S_{DS})D + \Omega_o Q_E + 0.5L$$

$$P_u = 1.548(1.1k) + 0.5(1.6) = 2.5k$$

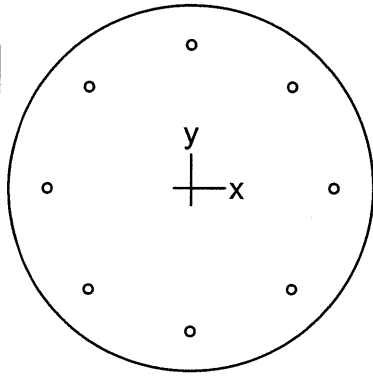
$$M_u = 2.3k \times 1.25 \times 10.5ft = 30.2k \cdot ft$$

$$(0.9 - 0.2S_{DS})D + \Omega_o Q_E$$

$$P_u = 0.552(1.1) = 0.60k$$

$$M_u = 30.2k \cdot ft$$

SEE SP COLUMN RESULTS



36 in diam.

Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

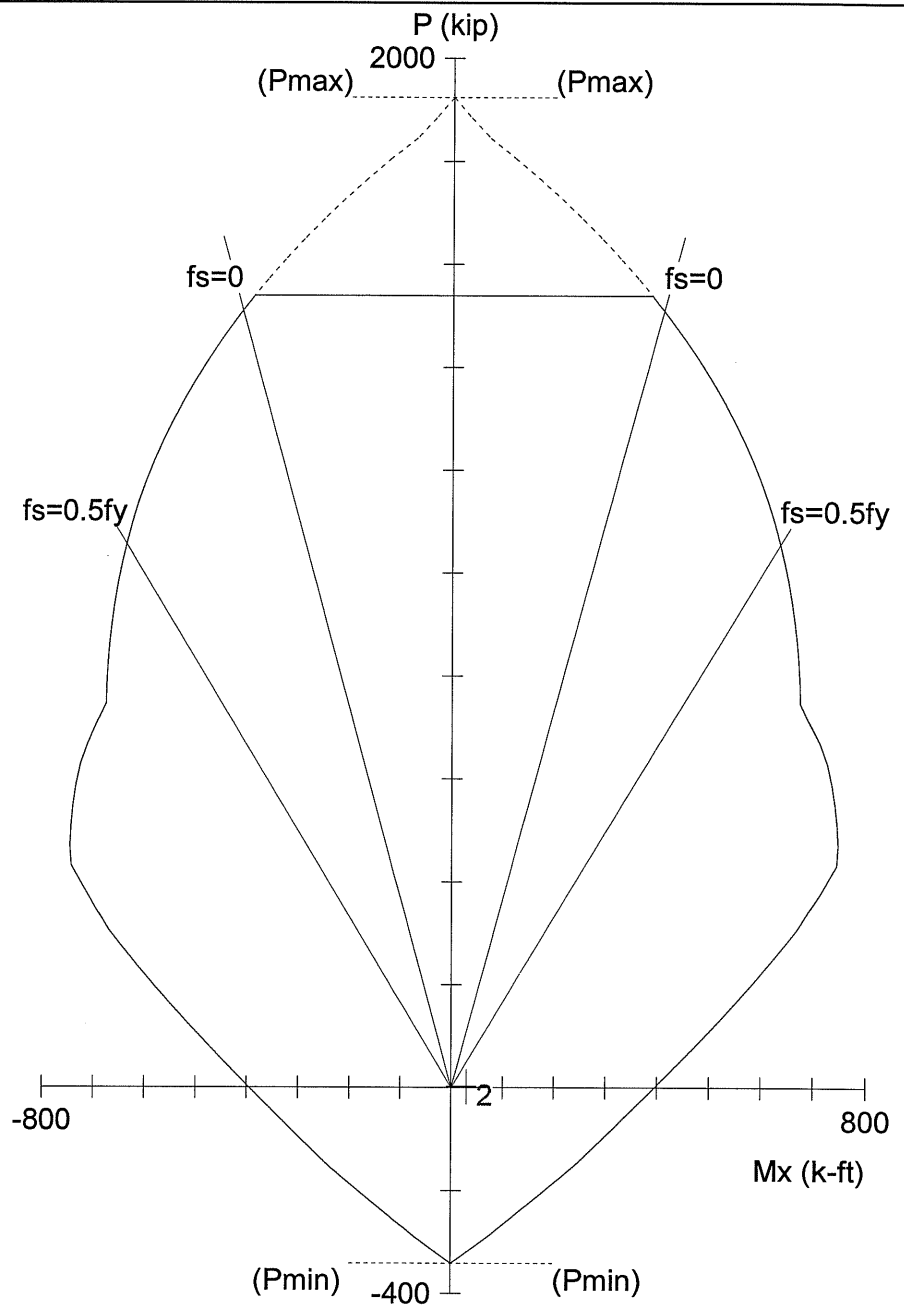
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 05/20/15

Time: 17:19:49



spColumn v4.81. Licensed to: Saiful/Bouquet, Inc. (SBI). License ID: 59950-1034111-2-27A36-25BCA

File: W:\14608_F.S. 143\Engineering\spColumn\Entry Canopy Pole Footing (Between F and G).col

Project: Fire Station 143

Column: Entry Trelli

Engineer: NH

$f'_c = 3$ ksi

$f_y = 60$ ksi

$A_g = 1017.88$ in²

8 #8 bars

$E_c = 3122$ ksi

$E_s = 29000$ ksi

$A_s = 6.32$ in²

$\rho = 0.62\%$

$f_c = 2.55$ ksi

$X_o = 0.00$ in

$I_x = 82448$ in⁴

$e_u = 0.003$ in/in

$Y_o = 0.00$ in

$I_y = 82448$ in⁴

Beta1 = 0.85

Min clear spacing = 9.81 in

Clear cover = 3.38 in

Confinement: Tied

$\phi(a) = 0.8$, $\phi(b) = 0.9$, $\phi(c) = 0.65$

```

          oooooo          o
          oo   oo          oo
    ooooo  oooooo  oo          ooooo  oo          oo  oo  o ooooooo  o ooooo
    oo  o  oo  oo  oo  oo          oo  oo  oo          oo  oo  oo  oo  oo  oo  oo
    oo          oo  oo  oo          oo  oo  oo          oo  oo  oo  oo  oo  oo
    ooooo  oo  oo  oo          oo  oo  oo          oo  oo  oo  oo  oo  oo
          oo  oooooo  oo          oo  oo  oo          oo  oo  oo  oo  oo  oo
    o  oo  oo          oo  oo          oo  oo  oo  o  oo  oo  oo  oo  oo  oo
    ooooo  oo          oooooo  ooooo  ooo  ooooo  o  oo  oo  oo  oo  oo  oo (TM)
    
```

```

=====
                        spColumn v4.81 (TM)
    Computer program for the Strength Design of Reinforced Concrete Sections
    Copyright © 1988-2012, STRUCTUREPOINT, LLC.
                        All rights reserved
=====
    
```

Licensee stated above acknowledges that STRUCTUREPOINT (SP) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the spColumn computer program. Furthermore, STRUCTUREPOINT neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the spColumn program. Although STRUCTUREPOINT has endeavored to produce spColumn error free the program is not and cannot be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensee's. Accordingly, STRUCTUREPOINT disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the spColumn program.

General Information:

File Name: W:\14608_F.S. 143\Engineering\spColumn\Entry Canopy Pole Footing (Between F and G).col
 Project: Fire Station 143
 Column: Entry Trelli Engineer: NH
 Code: ACI 318-11 Units: English
 Run Option: Investigation Slenderness: Not considered
 Run Axis: X-axis Column Type: Structural

Material Properties:

f'c = 3 ksi fy = 60 ksi
 Ec = 3122.02 ksi Es = 29000 ksi
 Ultimate strain = 0.003 in/in
 Beta1 = 0.85

Section:

Circular: Diameter = 36 in
 Gross section area, Ag = 1017.88 in^2
 Ix = 82448 in^4 Iy = 82448 in^4
 rx = 9 in ry = 9 in
 Xo = 0 in Yo = 0 in

Reinforcement:

Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
 phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
 Pattern: All Sides Equal (Cover to transverse reinforcement)
 Total steel area: As = 6.32 in^2 at rho = 0.62% (Note: rho < 1.0%)
 Minimum clear spacing = 9.81 in

8 #8 Cover = 3 in

Factored Loads and Moments with Corresponding Capacities:

No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu NA	depth in	Dt depth in	eps_t	Phi
1	2.50	30.20	397.44	13.160	6.43	32.13	0.01199	0.900
2	0.60	30.20	395.56	13.098	6.40	32.13	0.01205	0.900

*** End of output ***

4.2 TRASH ENCLOSURE DESIGN



Project: Fire Station
SBI Job No.:
Description: Loading Criteria
Date

Level: Trash Enclosure

Dead Load:

Item	Gravity	Joist	Truss	Seismic
Roofing	3.0			3.0
Framing (Seismic)	0.0			6.3
Misc	4.0			4.0
	7.0 psf	7.0 psf	7.0 psf	13.3 psf

Live Load: Roof 20.0 psf (Reducible)

TRASH ENCLOSURE FRAMING

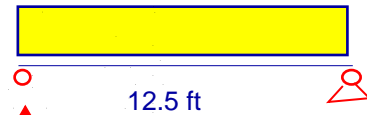
DL = 7 PSF LL = 20 PSF

SPAN : 12.5 ft SPACING : ~ 4'-0"

For HSS 4x2x1/4 AT 12.5 ft SPAN,

$$W_D = (7)(4'-0") = 28 \text{ PLF}$$

$$W_L = (20)(4'-0") = 80 \text{ PLF}$$



R_{DL} = 0.23 kip (including self weight)
R_{LL} = 0.5 kip

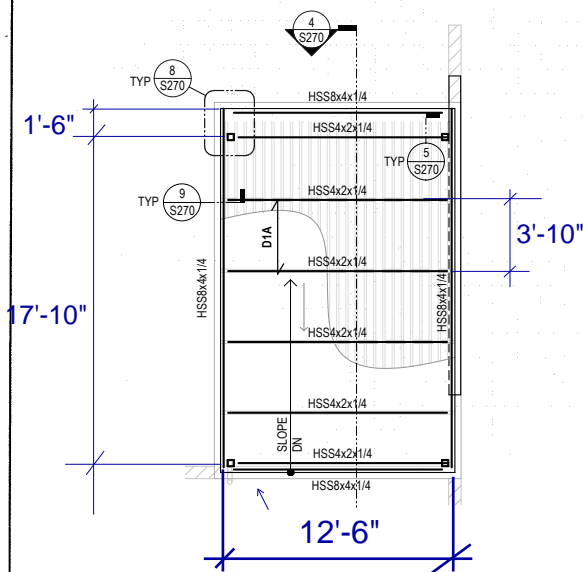
For HSS 8x4x1/4 AT 12.5 ft SPAN,

$$W_D = 0.23 \text{ kip/4'-0"} = 0.0575 \text{ kip/ft}$$

$$W_L = 0.5 \text{ kip/4'-0"} = 0.125 \text{ kip/ft}$$



USE HSS 8x4x1/4 GIRDERS & EDGE BEAMS
HSS 4x2x1/4 BEAMS (TYP. INT.)



Steel Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Trash Enclosure Joist

CODE REFERENCES

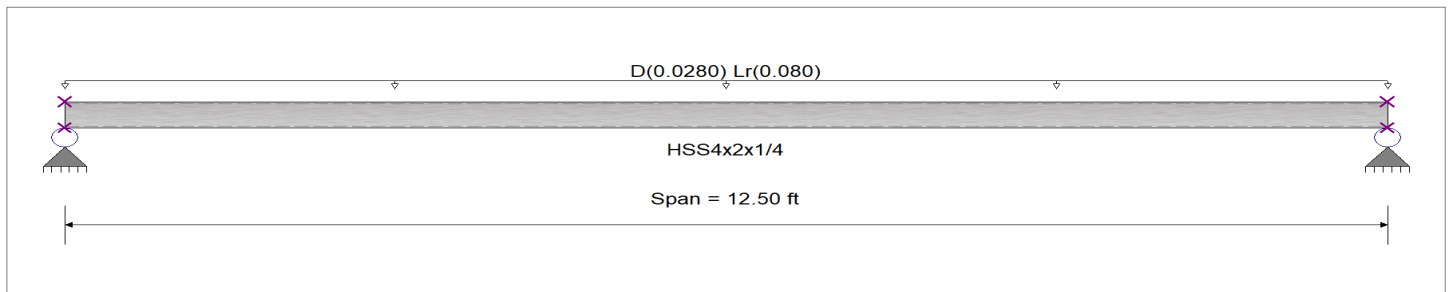
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 46.0 ksi
 E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.0280, Lr = 0.080 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.338 : 1	Maximum Shear Stress Ratio =		0.029 : 1
Section used for this span		HSS4x2x1/4	Section used for this span		HSS4x2x1/4
Ma : Applied		2.281 k-ft	Va : Applied		0.7301 k
Mn / Omega : Allowable		6.749 k-ft	Vn/Omega : Allowable		25.423 k
Load Combination		+D+Lr	Load Combination		+D+Lr
Span # where maximum occurs		Span # 1	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		0.339 in	Ratio =		442 >=360
Max Upward Transient Deflection		0.000 in	Ratio =		0 <360
Max Downward Total Deflection		0.495 in	Ratio =		303 >=240.
Max Upward Total Deflection		0.000 in	Ratio =		0 <240.0
			Span: 1 : Lr Only		
			Span: 1 : +D+Lr		

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
D Only													
Dsgn. L = 12.50 ft	1	0.107	0.009	0.72		0.72	11.27	6.75	1.14	1.00	0.23	42.46	25.42
+D+Lr													
Dsgn. L = 12.50 ft	1	0.338	0.029	2.28		2.28	11.27	6.75	1.14	1.00	0.73	42.46	25.42
+D+0.750Lr													
Dsgn. L = 12.50 ft	1	0.280	0.024	1.89		1.89	11.27	6.75	1.14	1.00	0.61	42.46	25.42
+0.60D													
Dsgn. L = 12.50 ft	1	0.064	0.005	0.43		0.43	11.27	6.75	1.14	1.00	0.14	42.46	25.42

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.4950	6.286		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.730	0.730
Max Upward from Load Combinations	0.730	0.730
Max Upward from Load Cases	0.500	0.500
D Only	0.230	0.230
+D+Lr	0.730	0.730
+D+0.750Lr	0.605	0.605
+0.60D	0.138	0.138

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Trash Enclosure Joist

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Lr Only	0.500	0.500

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercalc.ec6

LIC#: KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Trash Enclosure Edge Beam

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

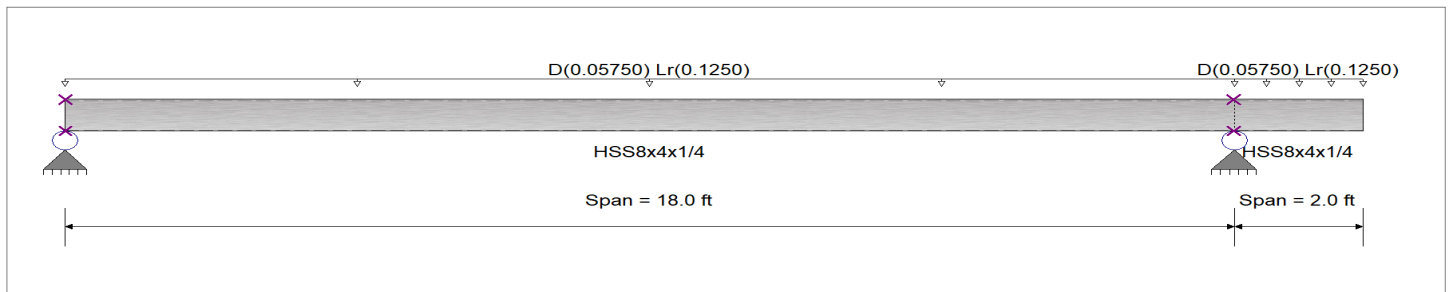
Analysis Method : Allowable Strength Design

Beam Bracing : Completely Unbraced

Bending Axis : Major Axis Bending

Fy : Steel Yield : 46.0 ksi

E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading

Load for Span Number 1

Uniform Load : D = 0.05750, Lr = 0.1250 k/ft, Tributary Width = 1.0 ft

Load for Span Number 2

Uniform Load : D = 0.05750, Lr = 0.1250 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.261 : 1	Maximum Shear Stress Ratio =	0.033 : 1
Section used for this span	HSS8x4x1/4	Section used for this span	HSS8x4x1/4
Ma : Applied	7.961 k-ft	Va : Applied	1.836 k
Mn / Omega : Allowable	30.529 k-ft	Vn/Omega : Allowable	56.229 k
Load Combination	+D+Lr	Load Combination	+D+Lr
Span # where maximum occurs	Span # 1	Location of maximum on span	18.000 ft
		Span # where maximum occurs	Span # 1
Maximum Deflection			
Max Downward Transient Deflection	0.234 in Ratio = 923	>=360	Span: 2 : Lr Only
Max Upward Transient Deflection	-0.081 in Ratio = 595	>=360	Span: 2 : Lr Only
Max Downward Total Deflection	0.377 in Ratio = 573	>=240.	Span: 2 : +D+Lr
Max Upward Total Deflection	-0.130 in Ratio = 369	>=240.	Span: 2 : +D+Lr

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
			M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only														
Dsgn. L =	18.00 ft	1	0.099	0.012	3.02	-0.15	3.02	50.98	30.53	1.14	1.00	0.70	93.90	56.23
Dsgn. L =	2.00 ft	2	0.005	0.003		-0.15	0.15	50.98	30.53	1.00	1.00	0.15	93.90	56.23
+D+Lr														
Dsgn. L =	18.00 ft	1	0.261	0.033	7.96	-0.40	7.96	50.98	30.53	1.14	1.00	1.84	93.90	56.23
Dsgn. L =	2.00 ft	2	0.013	0.007		-0.40	0.40	50.98	30.53	1.00	1.00	0.40	93.90	56.23
+D+0.750Lr														
Dsgn. L =	18.00 ft	1	0.220	0.028	6.73	-0.34	6.73	50.98	30.53	1.14	1.00	1.55	93.90	56.23
Dsgn. L =	2.00 ft	2	0.011	0.006		-0.34	0.34	50.98	30.53	1.00	1.00	0.34	93.90	56.23
+0.60D														
Dsgn. L =	18.00 ft	1	0.059	0.007	1.81	-0.09	1.81	50.98	30.53	1.14	1.00	0.42	93.90	56.23
Dsgn. L =	2.00 ft	2	0.003	0.002		-0.09	0.09	50.98	30.53	1.00	1.00	0.09	93.90	56.23

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.3772	9.000		0.0000	0.000
	2	0.0000	9.000	+D+Lr	-0.1300	2.000

ADDENDUM 5

Project Title:
Engineer:
Project ID:
Project Descr:

DESCRIPTION: Trash Enclosure Edge Beam

Vertical Reactions	Support notation : Far left is #1			Values in KIPS
	Support 1	Support 2	Support 3	
Load Combination				
Max Upward from all Load Conditions	1.791	2.239		
Max Upward from Load Combinations	1.791	2.239		
Max Upward from Load Cases	1.111	1.389		
D Only	0.680	0.850		
+D+Lr	1.791	2.239		
+D+0.750Lr	1.514	1.892		
+0.60D	0.408	0.510		
Lr Only	1.111	1.389		

BASE SHEAR

Equivalent Lateral Force Procedure

Project: **Fire Station**
SBI Job No.: _____

Building: **Trash Enclosure**
LFRS: **Ordinary Cantilever Column**
Direction: **X**

Building Data

Occupancy Category = **IV** Table 1604.5, 2013 CBC Importance Factor, $I_e = 1.50$ Table 1.5-2

Seismic Ground Motion Values Section 11.4

$S_S = 2.608$ From geotech or Figs.22-1 to 11
 $S_I = 0.986$ From geotech or Figs.22-1 to 11
Site Class = **D** From geotech or Table 20.3-1
 $T_L = 8 \text{ sec}$ Figs.22-12 to 16

$F_a = 1.00$ Table 11.4-1
 $F_v = 1.50$ Table 11.4-2
 $S_{MS} = F_a S_S = 2.608$ Eq. 11.4-1
 $S_{M1} = F_v S_I = 1.479$ Eq. 11.4-2
 $S_{DS} = (2/3) S_{MS} = 1.739$ Eq. 11.4-3
 $S_{D1} = (2/3) S_{M1} = 0.986$ Eq. 11.4-4

Seismic Design Category

F Tables 11.6-1 & 2

Building Period Section 12.8.2

$C_t = 0.02$ Table 12.8-2
 $\alpha = 0.75$ Table 12.8-2
 $h_n = 13.00 \text{ ft}$ Height of Building
 $T_b = 0.000 \text{ sec}$ From Analysis
(Input zero to use T_a)

$T_a = C_t h_n^\alpha = 0.137 \text{ sec}$ Eq. 12.8-7
 $C_u = 1.40$ Table 12.8-1
 $T_{a, \max} = C_u T_a = 0.192 \text{ sec}$ Section. 12.8.2

Period = **0.137 sec** <-- used for design
0.137 sec <-- used for drift
Section. 12.8.6.2

Base Shear Section 12.8

$W = 4 \text{ kips}$ Total Structure Weight
 $R = 1.25$ Table 12.2-1
 $C_d = 1.25$ Table 12.2-1

For Design Only

$C_s = S_{DS} / (R/I_e) = 2.086$ Eq. 12.8-2
 $C_{s, \max} = S_{D1} / [T (R/I_e)] = 8.641$ Eq. 12.8-3, for $T \leq T_L$
 $C_{s, \max} = S_{D1} T_L / [T^2 (R/I_e)] = \text{N/A}$ Eq. 12.8-4, for $T > T_L$
 $C_{s, \min} = \max\{0.01, 0.044 S_{DS} I_e\} = \text{N/A}$ Eq. 12.8-5, if $S_I < 0.6g$
 $C_{s, \min} = \max\{0.01, 0.044 S_{DS} I_e, 0.5 S_I / (R/I_e)\} = 0.592$ Eq. 12.8-5 and 12.8-6, if $S_I \geq 0.6g$

Use, $C_s = 2.086$

$V_{\text{design}} = C_s W = 9 \text{ kips}$ Eq. 12.8-1

Conservative number

For Drift Only

$C_{s, \max} = 2.086$
 $C_s = 8.641$
 $C_s = \text{N/A}$
 $C_{s, \min} = \text{N/A}$
 $C_{s, \min} = 0.592$

Use, $C_s = 2.086$

$V_{\text{drift}} = C_s W = 9 \text{ kips}$
Allowable Drift = **0.015** h_{sx}
Table 12.12-1

* Note: All references are from ASCE 7-10 unless noted otherwise.

ADDENDUM 5

TRASH ENCLOSURE COLUMNS

DL: 3 PSF DECK
4 PSF MISC

FRAMING: HSS 4x2x1/4 8.81 PLF
HSS 8x4x1/4 19.02 PLF
HSS 4x4x1/4 12.21 PLF

TYP BEAMS : 5 (11 FT) (8.81 PLF) = 485 lb

GIRDERS & EDGE: (19.02 PLF) (25 FT (2) + 15 FT + 10 FT) = 1427 lb

COLUMN: $[2(4.83 \text{ FT}/2) + 2(2 \text{ FT}/2)] (12.21 \text{ PLF}) = 84 \text{ lb}$

SEISMIC MASS

TOTAL = 1996 lb

$$1996 \text{ lb} / A_{\text{ROOF}} = 1996 / 316 = 6.32 \text{ PSF}$$

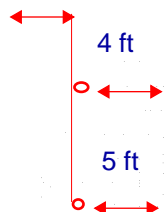
$$\Sigma \text{DL} = 13.3 \text{ PSF} \quad W = 13.3 \text{ psf} * (A) = 13.3 \text{ psf} * 12.5 \text{ ft} * 20 \text{ ft} = 3.3 \text{ kip}$$

$$C_s = 2.086$$

$$V_b = 6.6 \text{ kip}$$

$$6.6 \text{ kip} / 4 = 1.65 \text{ kip/col}$$

1.65 kip/col



HSS 4x4x1/2 is OK

Re= 2.9 kip, provide (4) AB to CMU per 7/S270, by inspection is OK

Re= 1.3 kip, provide (2) AB to FTG per detail 6/S270, by inspection is OK

Steel Beam

Project File: FS 46 enercal.ec6

LIC#: KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Trash Enclosure Column

CODE REFERENCES

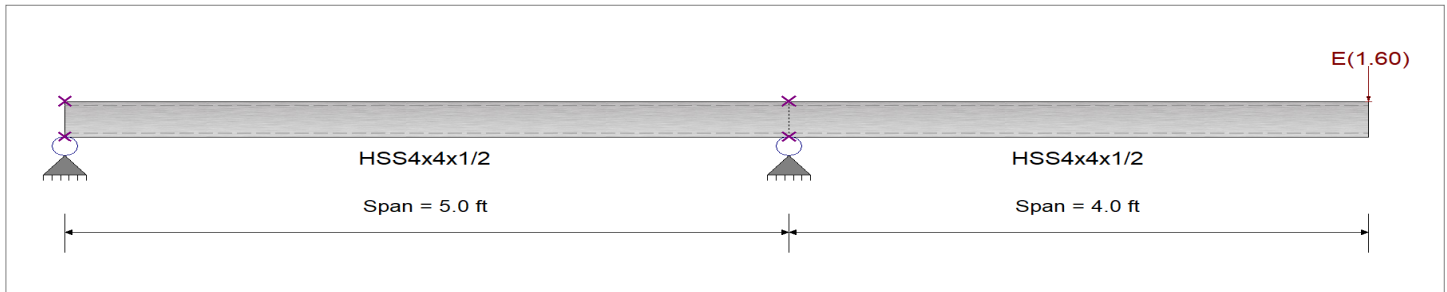
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 46.0 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 2

Point Load : E = 1.60 k @ 4.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.253 : 1	Maximum Shear Stress Ratio =		0.028 : 1
Section used for this span		HSS4x4x1/2	Section used for this span		HSS4x4x1/2
Ma : Applied		4.480 k-ft	Va : Applied		1.120 k
Mn / Omega : Allowable		17.675 k-ft	Vn/Omega : Allowable		40.039 k
Load Combination		E Only * 0.70	Load Combination		E Only * 0.70
Span # where maximum occurs		Span # 1	Location of maximum on span		5.000 ft
			Span # where maximum occurs		Span # 1
Maximum Deflection					
Max Downward Transient Deflection		0.384 in	Ratio =	249	>=240. Span: 2 : E Only
Max Upward Transient Deflection		-0.052 in	Ratio =	1,161	>=240. Span: 2 : E Only
Max Downward Total Deflection		0.269 in	Ratio =	357	>=240. Span: 2 : E Only * 0.70
Max Upward Total Deflection		-0.036 in	Ratio =	1659	>=240. Span: 2 : E Only * 0.70

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
Dsgn. L = 5.00 ft	1		0.000				29.52	17.67	1.00	1.00	-0.00	66.87	40.04
Dsgn. L = 4.00 ft	2		0.000				29.52	17.67	1.00	1.00	-0.00	66.87	40.04
E Only * 0.70													
Dsgn. L = 5.00 ft	1	0.253	0.028		-4.48	4.48	29.52	17.67	1.67	1.00	1.12	66.87	40.04
Dsgn. L = 4.00 ft	2	0.253	0.028		-4.48	4.48	29.52	17.67	1.00	1.00	1.12	66.87	40.04
E Only * 0.5250													
Dsgn. L = 5.00 ft	1	0.190	0.021		-3.36	3.36	29.52	17.67	1.67	1.00	0.84	66.87	40.04
Dsgn. L = 4.00 ft	2	0.190	0.021		-3.36	3.36	29.52	17.67	1.00	1.00	0.84	66.87	40.04

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	E Only	-0.0517	2.900
E Only	2	0.3840	4.000		0.0000	2.900

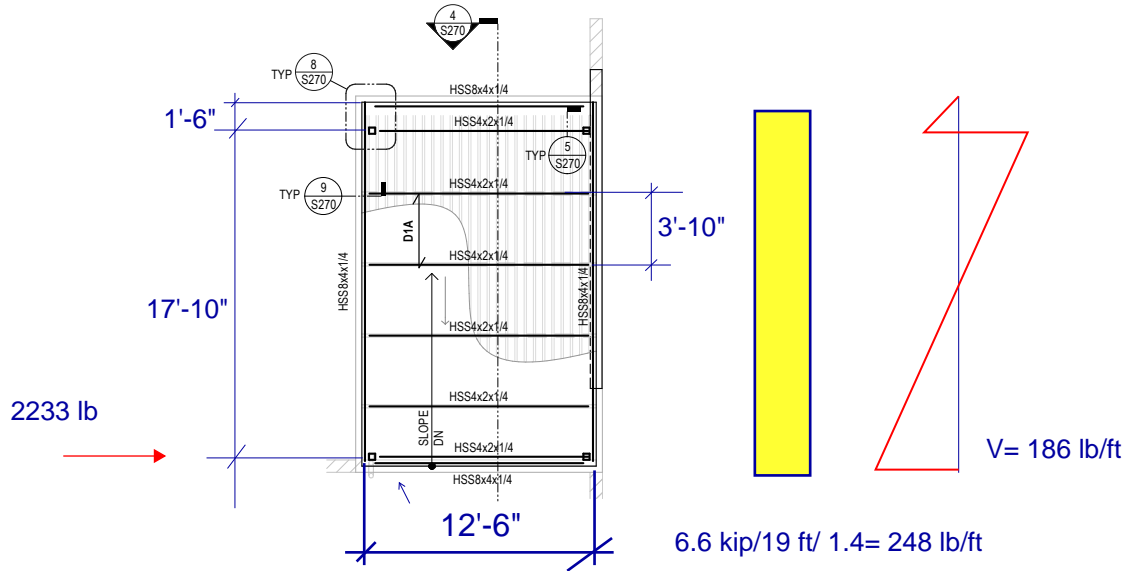
Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions		2.880	
Max Upward from Load Cases		2.880	
Max Downward from all Load Conditions (Resist	-1.280		
Max Downward from Load Cases (Resisting Upl	-1.280		

DIAPHRAGM



DECK SPAN: ~ 5'-0"

VERCO HSB-36 SS PANELS, 18 GAGE
36/7 SCREWS, #10 @ 18" O.C.

SHEAR STRENGTH = 551 PLF FOR 6'-0" SPANS

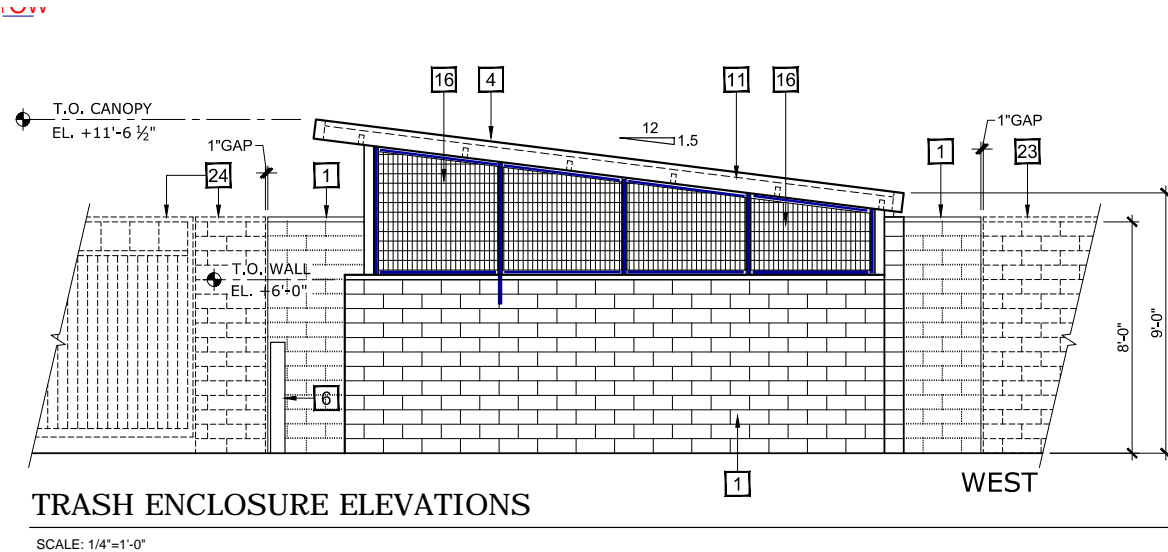
TABLE 29 - ALLOWABLE DIAPHRAGM SHEAR STRENGTH, q (plf), AND FLEXIBILITY FACTORS, F, FOR
HSB®-36-SS DECK PANELS ATTACHED WITH SDI RECOGNIZED #12 OR #14 SCREWS TO
SUPPORTS 0.0385" AND THICKER AND SIDELAPS FASTENED WITH #10 SCREWS^{1,2,3,4,5,6,7,8} (Cont'd.)

DECK GAGE	SIDELAP ATTACH- MENT	SPAN (ft-in.)									
		2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	
36/7/4 ATTACHMENT PATTERN FOR SDI RECOGNIZED SCREWS											
22	#10 @ 24"	q	471	421	343	318	263	257	223	224	202
		F	4.5+59R	6.2+39R	8.4+28R	8.7+22R	10.2+18R	10.1+15R	11.3+13R	11+11R	12+10R
	#10 @ 18"	q	562	421	396	364	302	291	282	250	249
		F	3.4+60R	6.2+39R	7.1+29R	7.7+23R	8.9+18R	9.1+16R	9.2+14R	10+12R	10+11R
	#10 @ 12"	q	562	486	445	405	377	357	341	328	319
		F	3.4+60R	5.3+39R	6.3+29R	6.9+23R	7.4+19R	7.7+16R	8+14R	8.2+13R	8.3+11R
	#10 @ 8"	q	643	603	539	520	476	471	444	443	424
		F	2.8+60R	4.2+40R	5.2+30R	5.6+24R	6.1+20R	6.2+17R	6.6+15R	6.6+13R	6.8+12R
	#10 @ 6"	q	717	656	623	590	567	550	537	527	519
		F	2.4+60R	3.9+40R	4.6+30R	5.1+24R	5.4+20R	5.6+17R	5.8+15R	5.9+13R	6+12R
	#10 @ 4"	q	838	791	765	739	721	707	697	689	682
		F	1.8+61R	3.2+40R	3.9+30R	4.3+24R	4.6+20R	4.8+17R	5+15R	5.1+13R	5.2+12R
20	#10 @ 24"	q	576	521	425	400	331	326	283	284	256
		F	4.8+37R	6+24R	7.8+17R	8+13R	9.3+10R	9.2+9R	10.2+7R	10+6R	10.8+5R
	#10 @ 18"	q	694	521	492	456	382	370	360	319	317
		F	3.8+37R	6+24R	6.7+18R	7.1+14R	8.2+11R	8.3+9R	8.3+8R	9.1+7R	9+6R
	#10 @ 12"	q	694	605	557	509	477	453	435	420	409
		F	3.8+37R	5.2+25R	5.9+18R	6.4+14R	6.7+12R	7+10R	7.2+9R	7.3+8R	7.5+7R
	#10 @ 8"	q	799	755	677	657	604	600	566	567	542
		F	3.3+38R	4.2+25R	5+19R	5.2+15R	5.6+12R	5.7+10R	5.9+9R	5.9+8R	6.1+7R
	#10 @ 6"	q	891	821	784	746	719	700	685	673	664
		F	2.9+38R	3.9+25R	4.4+19R	4.7+15R	4.9+12R	5.1+11R	5.2+9R	5.3+8R	5.4+7R
	#10 @ 4"	q	1041	989	960	931	911	896	884	875	868
		F	2.4+38R	3.3+25R	3.8+19R	4.1+15R	4.2+13R	4.4+11R	4.5+10R	4.5+8R	4.6+8R
18	#10 @ 24"	q	792	729	594	568	475	472	411	416	373
		F	4.7+17R	5.4+11R	6.8+7R	6.9+6R	8+4R	7.8+3R	8.7+3R	8.5+2R	9.2+2R
	#10 @ 18"	q	969	729	696	651	551	538	529	469	468
		F	3.8+18R	5.4+11R	5.8+8R	6.1+6R	7+5R	7+4R	7+3R	7.7+3R	7.6+2R
	#10 @ 12"	q	969	854	793	731	688	658	634	616	601
		F	3.8+18R	4.7+11R	5.2+8R	5.5+7R	5.8+5R	5.9+4R	6.1+4R	6.2+3R	6.3+3R
	#10 @ 8"	q	1122	1074	969	949	877	875	829	833	799
		F	3.3+18R	3.8+12R	4.4+9R	4.5+7R	4.8+6R	4.8+5R	5+4R	4.9+4R	5.1+3R
	#10 @ 6"	q	1254	1169	1123	1076	1043	1019	1001	986	975
		F	3+18R	3.6+12R	3.9+9R	4.1+7R	4.2+6R	4.3+5R	4.4+4R	4.4+4R	4.5+4R
	#10 @ 4"	q	1461	1400	1367	1333	1309	1291	1278	1267	1259
		F	2.6+19R	3.1+12R	3.3+9R	3.5+7R	3.6+6R	3.6+5R	3.7+5R	3.7+4R	3.8+4R
16	#10 @ 24"	q	1019	950	774	748	631	633	552	562	504
		F	4.3+9R	4.9+6R	6.1+3R	6.1+3R	7+2R	6.9+1R	7.6+1R	7.4+1R	8+0R
	#10 @ 18"	q	1260	950	914	862	731	719	709	635	636
		F	3.6+10R	4.9+6R	5.2+4R	5.4+3R	6.1+2R	6.1+2R	6.1+2R	6.7+1R	6.6+1R
	#10 @ 12"	q	1260	1121	1046	971	920	882	853	831	813
		F	3.6+10R	4.2+6R	4.6+4R	4.8+3R	5+3R	5.2+2R	5.3+2R	5.4+2R	5.4+1R
	#10 @ 8"	q	1466	1416	1284	1265	1175	1176	1117	1124	1081
		F	3.1+10R	3.4+7R	3.8+5R	3.9+4R	4.1+3R	4.1+3R	4.3+2R	4.3+2R	4.4+2R
	#10 @ 6"	q	1639	1540	1486	1431	1393	1366	1344	1327	1314
		F	2.8+10R	3.2+7R	3.4+5R	3.5+4R	3.6+3R	3.7+3R	3.7+2R	3.8+2R	3.8+2R
	#10 @ 4"	q	1901	1833	1796	1758	1732	1712	1697	1686	1676
		F	2.5+11R	2.8+7R	2.9+5R	3+4R	3.1+3R	3.1+3R	3.2+3R	3.2+2R	3.2+2R

See Pages 111 for footnotes.

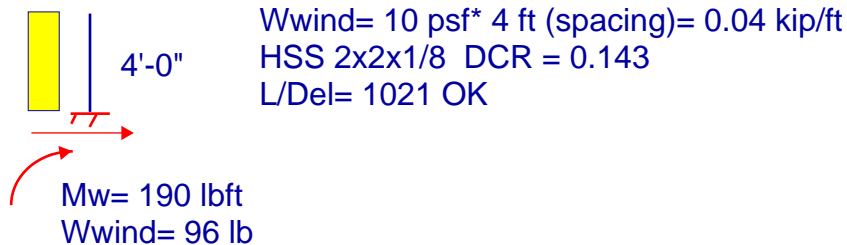
(continued)

FENCE FRAMING DESIGN

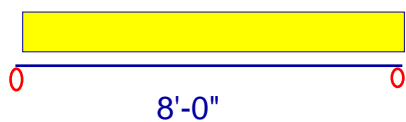


Assume 10 psf wind at fence

Fence post



Fence horizontal HSS



$W_{wind} = 10 \text{ psf} * 4 \text{ ft (spacing)} = 0.04 \text{ kip/ft}$
HSS 2x2x1/8 DCR = 0.143
L/Del = 1021 OK

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Fence Post at Trash Enclosure

CODE REFERENCES

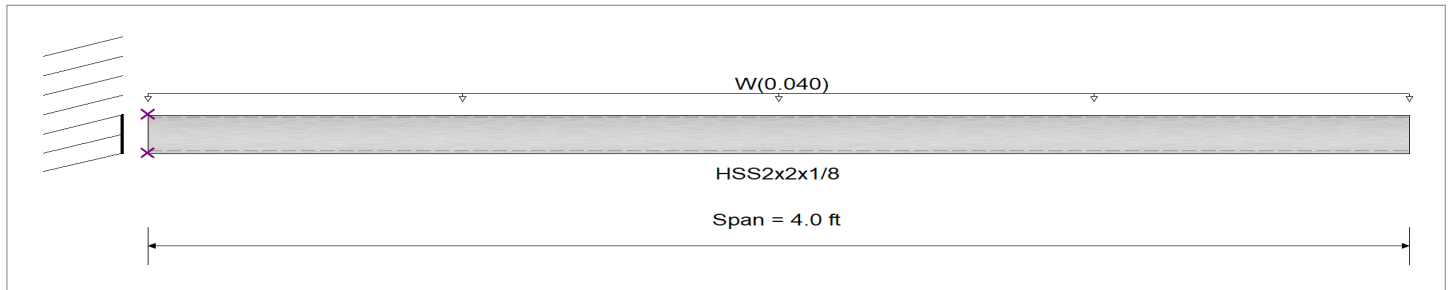
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 46.0 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 : W = 0.040 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.143 : 1		Maximum Shear Stress Ratio =		0.015 : 1	
Section used for this span		HSS2x2x1/8		Section used for this span		HSS2x2x1/8	
Ma : Applied		0.192 k-ft		Va : Applied		0.0960 k	
Mn / Omega : Allowable		1.341 k-ft		Vn/Omega : Allowable		6.334 k	
Load Combination		+0.60W		Load Combination		+0.60W	
Span # where maximum occurs		Span # 1		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		0.157 in Ratio =		612		>=360	
Max Upward Transient Deflection		0.000 in Ratio =		0		<360 Span: 1 : W Only	
Max Downward Total Deflection		0.094 in Ratio =		1021		>=240. Span: 1 : +0.60W	
Max Upward Total Deflection		0.000 in Ratio =		0		<240.0	

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	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 4.00 ft	1		0.000				2.24	1.34	1.00	1.00	-0.00	10.58	6.33
+0.60W													
Dsgn. L = 4.00 ft	1	0.143	0.015		-0.19	0.19	2.24	1.34	1.00	1.00	0.10	10.58	6.33
+0.450W													
Dsgn. L = 4.00 ft	1	0.107	0.011		-0.14	0.14	2.24	1.34	1.00	1.00	0.07	10.58	6.33

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.1566	4.000		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.160	
Max Upward from Load Combinations	0.096	
Max Upward from Load Cases	0.160	
+0.60W	0.096	
+0.450W	0.072	
W Only	0.160	

Steel Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Fence Beam at Trash Enclosure

CODE REFERENCES

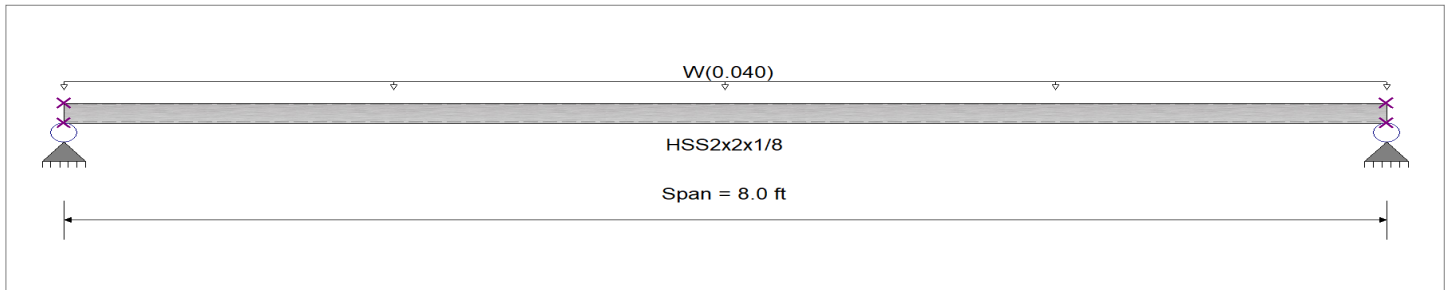
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 46.0 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 : W = 0.040 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.143 : 1		Maximum Shear Stress Ratio =		0.015 : 1	
Section used for this span		HSS2x2x1/8		Section used for this span		HSS2x2x1/8	
Ma : Applied		0.192 k-ft		Va : Applied		0.0960 k	
Mn / Omega : Allowable		1.341 k-ft		Vn/Omega : Allowable		6.334 k	
Load Combination		+0.60W		Load Combination		+0.60W	
Span # where maximum occurs		Span # 1		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		0.262 in Ratio =		365 >=360			
Max Upward Transient Deflection		0.000 in Ratio =		0 <360		Span: 1 : W Only	
Max Downward Total Deflection		0.158 in Ratio =		609 >=240.		Span: 1 : +0.60W	
Max Upward Total Deflection		0.000 in Ratio =		0 <240.0			

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	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 8.00 ft	1		0.000				2.24	1.34	1.00	1.00	-0.00	10.58	6.33
+0.60W													
Dsgn. L = 8.00 ft	1	0.143	0.015	0.19		0.19	2.24	1.34	1.14	1.00	0.10	10.58	6.33
+0.450W													
Dsgn. L = 8.00 ft	1	0.107	0.011	0.14		0.14	2.24	1.34	1.14	1.00	0.07	10.58	6.33

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.2628	4.023		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.160	0.160
Max Upward from Load Combinations	0.096	0.096
Max Upward from Load Cases	0.160	0.160
+0.60W	0.096	0.096
+0.450W	0.072	0.072
W Only	0.160	0.160

4.3 SITE WALL DESIGN

CMU SITE WALL DESIGN:

1.) Seismic Forces:

$$F_p = \frac{0.4 a_p S_{DS} I_p}{R_p} (1 + 2 z/h)$$

$$= \frac{0.4 (2.5) (1.739) (1.5)}{2.5} (1 + 0) = 1.04 W_p$$

$$F_p = 1.04 (80 \text{ PSF}) = \underline{\underline{83.5 \text{ PSF}}}$$

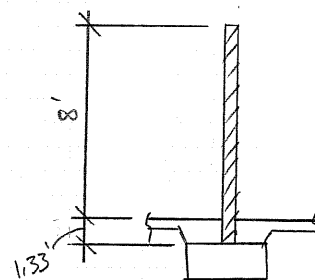
By inspection, seismic will govern over wind

2.) Design CMU Wall & Ftg: Typical 8'-0" Site Wall

Wall Ht = 8'-0"

⇒ See Attached CMD Analysis for
Design of Stem

⇒ See Attached EnerCalc Analysis for
Design of Footing



∴ USE 8" CMU w/ #6 @ 16" OC

WIND LOADS

CMU WALL (SOUTH END)

EXPOSURE B

$$L = 272 \text{ FT} \quad V = 115 \text{ MPH}$$

$$h = 8 \text{ FT}$$

$$F = q_h G C_f A_s \quad (29.4-1)$$

$$q_h = 0.00256 K_h K_{ht} K_d V^2 \\ = 0.00256 (0.85) (1.0) (0.85) (115)^2 = 24.5 \text{ PSF}$$

$$G = 0.85 \quad \text{RIGID STRUCTURE}$$

$$C_f = 3.36 \quad (\text{WORST CASE})$$

$$A_s = 2176 \text{ FT}^2$$

69.972 PSF ACTING AT 0.05h ABOVE WALL CENTER

70 PSF

SEISMIC GOVERNS

FORCE COEFFICIENT, C_f

$$B = 272 \text{ FT}$$

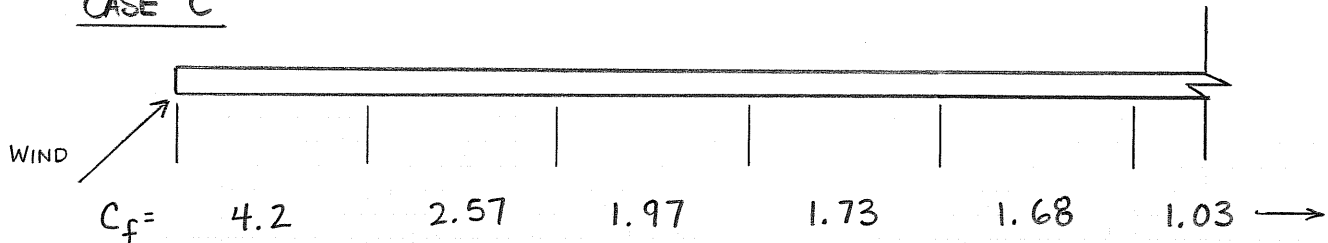
$$S = 8 \text{ FT}$$

$$S/h = 1 \quad B/S = 34$$

CASE A & CASE B

$$C_f = 1.30$$

CASE C



WORST CASE, NO RETURN CORNER

CASE C REDUCTION FACTOR, $(1.8 - S/h) = 0.8$

$$C_f = 4.2(0.8) = 3.36$$

Saiful Bouquet

333 City Blvd West, Suite 710

Orange, CA 928686

T: 714-464-1117

F:

Project: FS

Number:

Date:

By:

Sheet

of

CMD12.01.01

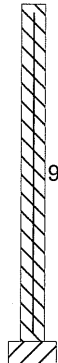
Filename: typical 8ft cmu site wall.dat

Reinforced Concrete Masonry Out-Of-Plane Loaded Wall per 2013 CBC Section 2108 (2011 MSJC Section 3.3.5)

- Ultimate Strength Design of Typ 8' Site Wall -

Input:

7.625 in.



9.3 ft. # 6 @ 16 inches on center
at 3.813 inches from the loaded face

Support Condition: Free Top, Fixed Bottom

Wall Section

Wall Geometry:

9.33' high, 8" nominal thickness CMU wall

Cells Grouted at 8" o.c.

Wall Material Properties:

$f'_m = 1,500$ psi

$f_y = 60,000$ psi

$E_m = 1,350$ ksi

$E_s = 29,000$ ksi

$n = 21.48$

Applied Loads:

	P plf	M plf-in	Wall Weight psf
Dead Load	0	0	80
Floor Live Load	0	0	
Roof Live Load	0	0	
Snow Load	0	0	
Horizontal Seismic Load	0	0	
Vertical Seismic Load	0	0	28
Vertical Wind Load	0	0	

Lateral Loading:

#	Load Condition	Load Type	Load Start Location	Value	Load End Location	Value
	E,W,H ¹	P,T,M ²	ft	plf,psf,plf-in	ft	psf
1	E	T	0.0	83	9.3	83

Seismic Performance Category: F

Output:

Input Reinforcing:

6 @ 16 inches o.c., $A_s = 0.33$ in²/ft, $\rho = 0.0072$
maximum $\rho = 0.0072$

Wall Design Geometry, Gross Moment of Inertia and Section Modulus:

Positive Moments: $b = 16.00$ in $d = 3.81$ in

Negative Moments: $b = 16.00$ in $d = 3.81$ in

Gross Section: $b = 12.00$ in

$I_g = b \cdot t^3 / 12 = 443$ in⁴/ft

$S_g = I_g / (t/2) = 116$ in³/ft

Allowable Stresses and Cracking Moment:

$h'/t = 14.68 \leq 30$ - $F_{a \max} = 0.2 f'_m$

Maximum Axial Stress for $h'/t > 30 = 0.05 f'_m = 75$ psi

Maximum Axial Stress for $h'/t \leq 30 = 0.2 f'_m = 300$ psi

f_r (per MSJC Table 3.1.7.2.1) = 163.0 psi

$M_{cr} = S_g \cdot f_r / 1000 = 19.0$ kip-in/ft

Saiful Bouquet

333 City Blvd West, Suite 710

Orange, CA 928686

T: 714-464-1117

F:

Project: FS

Number:

Date:

By:

Sheet

of

CMD12.01.01

Filename: typical 8ft cmu site wall.dat

Reinforced Concrete Masonry Out-Of-Plane Loaded Wall per 2013 CBC Section 2108 (2011 MSJC Section 3.3.5)

- Ultimate Strength Design of Typ 8' Site Wall -

Factored Input Out-Of-Plane Loads and Moments:

	Load	Load	P_u	M_u	W_u start	h_{start}	W_u end	h_{end}
	Type	Factor	klf	klf-in	ksf	ft	ksf	ft
Load 1	E	1.0			0.084	0.00	0.084	9.33

Ultimate Axial Loads and Stress Checks:

Load Case	$P_{u0} + P_{uw}$	@ h	$P_{u0} + P_{uw}/A_g$	Stress	
	lbs	ft	psi	Ratio	
1.4D	1,045.0	0.00	11.42	0.038	OK
1.2D+1.0E+0.5L+0.2S	1,156.9	0.00	12.64	0.042	OK
1.2D-1.0E+0.5L+0.2S	1,156.9	0.00	12.64	0.042	OK
1.2D+1.0E	1,156.9	0.00	12.64	0.042	OK
1.2D-1.0E	1,156.9	0.00	12.64	0.042	OK

Service Load Moments and Maximum Deflections:- Maximum Allowable Deflection, $\Delta_{s \max} = 0.007h = 0.784$ in

Equivalent Unfactored Load Case	P_s	M_{s0}	$M_s P_\Delta$	@ h	Δ_s	@ h	
	kips	kip-in	kip-in	ft	in	ft	
D+0.75[L+S+0.7E]	0.88	-22.90	-23.02	0.00	0.2764	9.33	$\leq \Delta_{\max}$ - OK
D+0.7E	0.93	-30.53	-30.81	0.00	0.6151	9.33	$\leq \Delta_{\max}$ - OK
D-0.7E	0.93	30.53	30.81	0.00	-0.6148	9.33	$\leq \Delta_{\max}$ - OK
0.6D+0.7E	0.26	-30.53	-30.61	0.00	0.6203	9.33	$\leq \Delta_{\max}$ - OK
0.6D-0.7E	0.26	30.53	30.61	0.00	-0.6200	9.33	$\leq \Delta_{\max}$ - OK

Ultimate and Nominal Shear:

Load Case	Governing Location	V_u	ϕV_n	$\phi = 0.80$ Strength Ratio	
		kips	kips		
1.2D+1.0E+0.5L+0.2S	Bottom	0.78	6.61	0.12	OK
1.2D-1.0E+0.5L+0.2S	Bottom	0.78	6.61	0.12	OK
1.2D+1.0E	Bottom	0.78	6.61	0.12	OK
0.9D+1.0E	Bottom	0.78	6.46	0.12	OK
0.9D-1.0E	Bottom	0.78	6.46	0.12	OK

Saiful Bouquet

333 City Blvd West, Suite 710

Orange, CA 928686

T: 714-464-1117

F:

Project: FS

Number:

Date:

By:

Sheet

of

CMD12.01.01

Filename: typical 8ft cmu site wall.dat

Reinforced Concrete Masonry Out-Of-Plane Loaded Wall per 2013 CBC Section 2108 (2011 MSJC Section 3.3.5)

- Ultimate Strength Design of Typ 8' Site Wall -

Ultimate Axial Load and Data for Ultimate Positive Moment Calculations:

Load Case	P_u kips	@ h ft	A_{se} in ²	c in	I_{cr} in ⁴
1.4D	1.04	0.00	0.35	1.82	53.7
1.2D+1.6L+0.5Lr	0.90	0.00	0.35	1.81	53.3
1.2D-1.0E+0.5L+0.2S	1.16	0.00	0.35	1.83	53.9
1.2D-1.0E	1.16	0.00	0.35	1.83	53.9
0.9D-1.0E	0.41	0.00	0.34	1.76	52.3

Ultimate Positive Moment and Deflection:

Load Case	M_n kip-in	ϕM_n kip-in	M_{u0} kip-in	$M_{uP\Delta}$ kip-in	$\phi = 0.90$ Strength Ratio	Δ_u in	
1.4D	64.7	58.2	0.00	0.00	0.00	0.000	OK
1.2D+1.6L+0.5Lr	64.3	57.9	0.00	0.00	0.00	0.000	OK
1.2D-1.0E+0.5L+0.2S	65.0	58.5	43.61	44.30	0.76	1.191	OK
1.2D-1.0E	65.0	58.5	43.61	44.30	0.76	1.191	OK
0.9D-1.0E	63.0	56.7	43.61	43.86	0.77	1.205	OK

Ultimate Axial Load and Data for Ultimate Negative Moment Calculations:

Load Case	P_u kips	@ h ft	A_{se} in ²	c in	I_{cr} in ⁴
1.4D	1.04	0.00	0.35	1.82	53.7
1.2D+1.6L+0.5Lr	0.90	0.00	0.35	1.81	53.3
1.2D+1.0E+0.5L+0.2S	1.16	0.00	0.35	1.83	53.9
1.2D+1.0E	1.16	0.00	0.35	1.83	53.9
0.9D+1.0E	0.41	0.00	0.34	1.76	52.2

Ultimate Negative Moment and Deflection:

Load Case	M_n kip-in	ϕM_n kip-in	M_{u0} kip-in	$M_{uP\Delta}$ kip-in	$\phi = 0.90$ Strength Ratio	Δ_u in	
1.4D	-64.6	-58.2	0.00	0.00	0.00	0.000	OK
1.2D+1.6L+0.5Lr	-64.3	-57.8	0.00	0.00	0.00	0.000	OK
1.2D+1.0E+0.5L+0.2S	-64.9	-58.4	-43.61	-44.30	0.76	-1.191	OK
1.2D+1.0E	-64.9	-58.4	-43.61	-44.30	0.76	-1.191	OK
0.9D+1.0E	-63.0	-56.7	-43.61	-43.86	0.77	-1.206	OK

ADDENDUM 5

Cantilevered Retaining Wall

File = c:\SHERV\1\MISC-1.PRO\3333SB~1\3333SB~1.EC6
ENERCALC, INC. 1983-2014, Build:6.14.9.18, Ver:6.14.9.18

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: Typical 8'-0" CMU Site Wall Ftg Type A (Ftg Design Only)

Calculations per ACI 318-11, ACI 530-11, IBC 2012, CBC 2013, ASCE 7-10

Criteria		
Retained Height	=	2.00 ft
Wall height above soil	=	8.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft
Vertical component of active		
Lateral soil pressure options:		
NOT USED for Soil Pressure.		
NOT USED for Sliding Resistance.		
NOT USED for Overturning Resistance.		

Soil Data		
Allow Soil Bearing	=	3,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	36.0 psf/ft
Toe Active Pressure	=	36.0 psf/ft
Passive Pressure	=	350.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.400
Soil height to ignore		
for passive pressure		
	=	12.00 in

Surcharge Loads		
Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem		
Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem		
Lateral Load	=	58.5 plf
...Height to Top	=	10.00 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load		
Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type	Line Load	
Base Above/Below Soil	=	0.0 ft
at Back of Wall	=	
Poisson's Ratio	=	0.300

Wind on Exposed Stem = 0.0 psf

Design Summary

Wall Stability Ratios		
Overturning	=	1.71 OK
Sliding	=	2.58 OK
Total Bearing Load	=	2,495 lbs
...resultant ecc.	=	16.90 in

Soil Pressure @ Toe	=	1,523 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	1,828 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	11.7 psi OK
Footing Shear @ Heel	=	6.7 psi OK
Allowable	=	82.2 psi

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	687.0 lbs
less 100% Passive Force	= -	777.8 lbs
less 100% Friction Force	= -	998.0 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

Design Height Above Ftg

Wall Material Above "Ht"	=	Masonry
Thickness	=	8.00 in
Rebar Size	=	# 6
Rebar Spacing	=	16.00 in
Rebar Placed at	=	Center

Design Data

fb/FB + fa/Fa	=	0.000
Total Force @ Section	lbs =	0.0
Moment....Actual	ft-l =	0.0
Moment....Allowable	ft-l =	1,270.8
Shear....Actual	psi =	0.0
Shear....Allowable	psi =	38.7
Wall Weight	psf =	78.0
Rebar Depth 'd'	in =	3.75
Lap splice if above	in =	36.00
Lap splice if below	in =	36.00
Hook embed into footing	in =	36.00

Masonry Data

f'm	psi =	1,500
Fs	psi =	24,000
Solid Grouting	=	Yes

Modular Ratio 'n'	=	21.48
Short Term Factor	=	1.000
Equiv. Solid Thick.	in =	7.60
Masonry Block Type	=	2
Masonry Design Method	=	ASD

	Top Stem	2nd
Stem OK	Ratio > 1.0	
ft = 10.00	0.00	
Masonry	Masonry	
Thickness	8.00	8.00
Rebar Size	# 6	# 6
Rebar Spacing	16.00	16.00
Rebar Placed at	Center	Center
fb/FB + fa/Fa	0.000	2.335
Total Force @ Section	lbs = 0.0	639.0
Moment....Actual	ft-l = 0.0	2,967.0
Moment....Allowable	ft-l = 1,270.8	1,270.8
Shear....Actual	psi = 0.0	14.2
Shear....Allowable	psi = 38.7	38.7
Wall Weight	psf = 78.0	78.0
Rebar Depth 'd'	in = 3.75	3.75
Lap splice if above	in = 36.00	54.00
Lap splice if below	in = 36.00	11.50
Hook embed into footing	in = 36.00	11.50

f'm	psi = 1,500	1,500
Fs	psi = 24,000	24,000
Solid Grouting	= Yes	Yes
Modular Ratio 'n'	= 21.48	21.48
Short Term Factor	= 1.000	1.000
Equiv. Solid Thick.	in = 7.60	7.60

ADDENDUM 5

Cantilevered Retaining Wall

File = c:\SHERVI~1\MISC~1\PROJ\3333SB~1\3333SB~1.EC6
ENERCALC, INC. 1983-2014, Build:6.14.9.18, Ver:6.14.9.18

Lic. #: KW-06003631

Licensee : SAIFUL - BOUQUET CONSULTING ENGINEERS

Description : Typical 8'-0" CMU Site Wall Ftg Type A (Ftg Design Only)

Footing Dimensions & Strengths

Toe Width	=	2.17 ft
Heel Width	=	2.83
Total Footing Width	=	5.00
Footing Thickness	=	16.00 in
Key Width	=	12.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 ft
f'c =	3,000 psi	Fy = 60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top	2.00	@ Btm. = 3.00 in

Footing Design Results

	Toe	Heel
Factored Pressure	= 1,828	0 psf
Mu' : Upward	= 3,795	0 ft-lb
Mu' : Downward	= 1,013	1,372 ft-lb
Mu: Design	= 2,782	1,372 ft-lb
Actual 1-Way Shear	= 11.66	6.74 psi
Allow 1-Way Shear	= 82.16	82.16 psi
Toe Reinforcing	= # 7 @ 16.00 in	
Heel Reinforcing	= # 6 @ 16.00 in	
Key Reinforcing	= None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, $M_u < S * F_r$
Heel: Not req'd, $M_u < S * F_r$
Key: Not req'd, $M_u < S * F_r$

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....			RESISTING.....							
Item		Force lbs	Distance ft	Moment ft-lb		Force lbs	Distance ft	Moment ft-lb			
Heel Active Pressure	=	200.0	1.11	222.2	Soil Over Heel	=	476.7	3.92	1,866.9		
Surcharge over Heel	=				Sloped Soil Over Heel	=					
Toe Active Pressure	=	-98.0	0.78	-76.2	Surcharge Over Heel	=					
Surcharge Over Toe	=				Adjacent Footing Load	=					
Adjacent Footing Load	=				Axial Dead Load on Stem	=					
Added Lateral Load	=	585.0	6.33	3,705.0	* Axial Live Load on Stem	=					
Load @ Stem Above Soil	=				Soil Over Toe	=	238.3	1.08	258.2		
					Surcharge Over Toe	=					
					Stem Weight(s)	=	780.0	2.50	1,950.0		
					Earth @ Stem Transitions	=					
					Footing Weight	=	1,000.0	2.50	2,500.0		
					Key Weight	=		2.50			
					Vert. Component	=					
Total	=	687.0	O.T.M.	=	3,851.0	Total	=	2,495.0 lbs	R.M.	=	6,575.1
Resisting/Overturning Ratio				=	1.71						
Vertical Loads used for Soil Pressure =			2,495.0	lbs							

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

ADDENDUM 5

Cantilevered Retaining Wall

File = c:\SHERV\1\MISC-1.PRO\3333SB-1\3333SB-1.EC6
ENERCALC, INC. 1983-2014, Build:6.14.9.18, Ver:6.14.9.18

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: Typical 8'-0" CMU Site Wall Ftg Type B (Ftg Design Only)

Calculations per ACI 318-11, ACI 530-11, IBC 2012, CBC 2013, ASCE 7-10

Criteria		
Retained Height	=	2.00 ft
Wall height above soil	=	8.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft
Vertical component of active Lateral soil pressure options:		
NOT USED for Soil Pressure.		
NOT USED for Sliding Resistance.		
NOT USED for Overturning Resistance.		

Soil Data		
Allow Soil Bearing	=	3,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	36.0 psf/ft
Toe Active Pressure	=	36.0 psf/ft
Passive Pressure	=	350.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.400
Soil height to ignore for passive pressure	=	12.00 in

Surcharge Loads		
Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem		
Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem		
Lateral Load	=	58.5 plf
...Height to Top	=	10.00 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load		
Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type	=	Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios		
Overturning	=	1.70 OK
Sliding	=	2.85 OK
Total Bearing Load	=	2,943 lbs
...resultant ecc.	=	21.94 in

Soil Pressure @ Toe	=	2,129 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	2,555 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	15.0 psi OK
Allowable	=	82.2 psi

Sliding Calcs (Vertical Component NOT Used)		
Lateral Sliding Force	=	687.0 lbs
less 100% Passive Force	= -	777.8 lbs
less 100% Friction Force	= -	1,170.0 lbs
Added Force Req'd	=	0.0 lbs OK
...for 1.5 : 1 Stability	=	0.0 lbs OK

Load Factors		
Dead Load	=	1.200
Live Load	=	1.600
Earth, H	=	1.600
Wind, W	=	1.600
Seismic, E	=	1.000

Stem Construction

Design Height Above Ftg		
Wall Material Above "Ht"	=	Masonry
Thickness	=	8.00 in
Rebar Size	=	# 6
Rebar Spacing	=	16.00 in
Rebar Placed at	=	Center

Design Data		
fb/FB + fa/Fa	=	0.000
Total Force @ Section	=	0.0 lbs
Moment....Actual	=	0.0 ft-lb
Moment.....Allowable	=	1,270.8 ft-lb
Shear.....Actual	=	0.0 psi
Shear.....Allowable	=	38.7 psi
Wall Weight	=	78.0 psf
Rebar Depth 'd'	=	3.75 in
Lap splice if above	=	36.00 in
Lap splice if below	=	36.00 in
Hook embed into footing	=	36.00 in

Masonry Data		
f'm	=	1,500 psi
Fs	=	24,000 psi
Solid Grouting	=	Yes

Modular Ratio 'n'	=	21.48
Short Term Factor	=	1.000
Equiv. Solid Thick.	=	7.60 in
Masonry Block Type	=	2
Masonry Design Method	=	ASD

	Top Stem	2nd
Stem OK	Ratio > 1.0	
ft = 10.00	0.00	
Masonry	Masonry	
Thickness	8.00	8.00
Rebar Size	# 6	# 6
Rebar Spacing	16.00	16.00
Rebar Placed at	Center	Center
fb/FB + fa/Fa	0.000	2.335
Total Force @ Section	0.0 lbs	639.0 lbs
Moment....Actual	0.0 ft-lb	2,967.0 ft-lb
Moment.....Allowable	1,270.8 ft-lb	1,270.8 ft-lb
Shear.....Actual	0.0 psi	14.2 psi
Shear.....Allowable	38.7 psi	38.7 psi
Wall Weight	78.0 psf	78.0 psf
Rebar Depth 'd'	3.75 in	3.75 in
Lap splice if above	36.00 in	54.00 in
Lap splice if below	36.00 in	11.50 in
Hook embed into footing	36.00 in	11.50 in
f'm	1,500 psi	1,500 psi
Fs	24,000 psi	24,000 psi
Solid Grouting	Yes	Yes
Modular Ratio 'n'	21.48	21.48
Short Term Factor	1.000	1.000
Equiv. Solid Thick.	7.60 in	7.60 in
Masonry Block Type	2	
Masonry Design Method	ASD	

ADDENDUM 5

Cantilevered Retaining Wall

File = c:\SHERV\1\MISC-1.PRO\3333SB-1\3333SB-1.EC6
ENERCALC, INC. 1983-2014, Build:6.14.9.18, Ver:6.14.9.18

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: Typical 8'-0" CMU Site Wall Ftg Type B (Ftg Design Only)

Footing Dimensions & Strengths

Toe Width	=	0.00 ft
Heel Width	=	5.50
Total Footing Width	=	5.50
Footing Thickness	=	16.00 in
Key Width	=	12.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 ft
f'c =	3,000 psi	Fy = 60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top	2.00	@ Btm. = 3.00 in

Footing Design Results

	Toe	Heel
Factored Pressure	= 2,555	0 psf
Mu' : Upward	= 0	0 ft-lb
Mu' : Downward	= 0	0 ft-lb
Mu: Design	= 0	4,747 ft-lb
Actual 1-Way Shear	= 0.00	15.04 psi
Allow 1-Way Shear	= 0.00	82.16 psi
Toe Reinforcing	= # 7 @ 16.00 in	
Heel Reinforcing	= # 6 @ 16.00 in	
Key Reinforcing	= None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr

Heel: Not req'd, Mu < S * Fr

Key: Not req'd, Mu < S * Fr

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....			RESISTING.....					
Item		Force lbs	Distance ft	Moment ft-lb		Force lbs	Distance ft	Moment ft-lb	
Heel Active Pressure	=	200.0	1.11	222.2	Soil Over Heel	=	1,063.3	3.08	3,278.6
Surcharge over Heel	=				Sloped Soil Over Heel	=			
Toe Active Pressure	=	-98.0	0.78	-76.2	Surcharge Over Heel	=			
Surcharge Over Toe	=				Adjacent Footing Load	=			
Adjacent Footing Load	=				Axial Dead Load on Stem	=			
Added Lateral Load	=	585.0	6.33	3,705.0	* Axial Live Load on Stem	=			
Load @ Stem Above Soil	=				Soil Over Toe	=			
					Surcharge Over Toe	=			
					Stem Weight(s)	=	780.0	0.33	260.0
					Earth @ Stem Transitions	=			
					Footing Weight	=	1,100.0	2.75	3,025.0
					Key Weight	=		2.50	
					Vert. Component	=			

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

ADDENDUM 5

10'-8" CMU SITE WALL (10" CMU)

SEISMIC :

$$F_p = \frac{0.4 a_p S_{DS} W}{R_p / I_p} (1 + 2 \frac{z}{h})$$

$$= \frac{0.4 (2.5) (1.739) (100)}{(2.5 / 1.5)} (1 + 0)$$

$$F_p = 1.04 (100 \text{ PSF}) = \underline{\underline{104 \text{ PSF}}}$$

$$E_v = 0.2 S_{DS} W$$

$$= 0.2 (1.739) (100) = \underline{\underline{35 \text{ PSF}}}$$

WIND : SEE PREVIOUS CALC

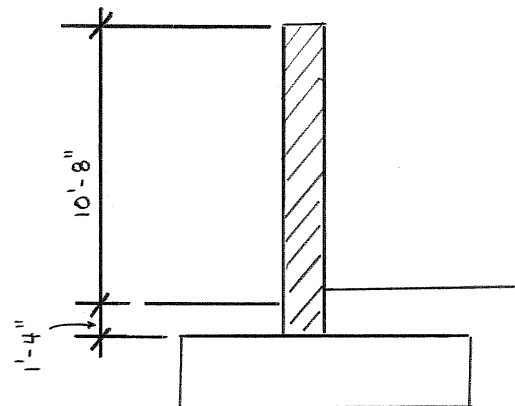
SEISMIC GOVERNS

DESIGN OF CMU WALL & FOOTING

WALL HEIGHT = 8'-0"

SEE ATTACHED CMD ANALYSIS FOR
DESIGN OF STEM

SEE ATTACHED ENERCALC ANALYSIS FOR
DESIGN OF FOOTING



∴ USE 10" CMU w/#6 @ 16" o.c.
EA. FACE

Saiful Bouquet

333 City Blvd West, Suite 710
Orange, CA 92868
T: 714-464-1117
F:

Project:

Number:

Date:

By:

Sheet

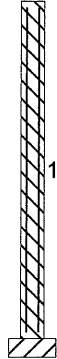
of

CMD12.01.01

Reinforced Concrete Masonry Out-Of-Plane Loaded Wall per 2013 CBC Section 2108 (2011 MSJC Section 3.3.5)
- Ultimate Strength Design -

Input:

9.625 in.



1st Curtain: # 6 @ 16 inches on center
at 7.25 inches from the loaded face
2nd Curtain: # 6 @ 16 inches on center
at 2.375 inches from the loaded face

Support Condition: Free Top, Fixed Bottom

Wall Section

Wall Geometry:

12.00' high, 10" nominal thickness CMU wall
Cells Grouted at 8" o.c.

Wall Material Properties:

$f'_m = 1,500$ psi
 $f_y = 60,000$ psi
 $E_m = 1,350$ ksi
 $E_s = 29,000$ ksi
 $n = 21.48$

Applied Loads:

	P plf	M plf-in	Wall Weight psf
Dead Load	0	0	100
Floor Live Load	0	0	
Roof Live Load	0	0	
Snow Load	0	0	
Horizontal Seismic Load	0	0	
Vertical Seismic Load	0	0	35
Vertical Wind Load	0	0	

Lateral Loading:

#	Load Condition	Load Type	Load Start Location	Value	Load End Location	Value
	E,W,H ¹	P,T,M ²	ft	plf,psf,plf-in	ft	psf
1	E	T	0.0	104	12.0	104

Seismic Performance Category: F

Output:

Input Reinforcing:

1st Curtain: # 6 @ 16 inches o.c., $A_s = 0.33$ in²/ft, $\rho = 0.0038$
2nd Curtain: # 6 @ 16 inches o.c., $A_s = 0.33$ in²/ft, $\rho = 0.0038$
maximum $\rho = 0.0038$

Wall Design Geometry, Gross Moment of Inertia and Section Modulus:

Positive Moments: $b = 16.00$ in $d = 7.25$ in
Negative Moments: $b = 16.00$ in $d = 7.25$ in
Gross Section: $b = 12.00$ in
 $I_g = b^3 t / 12 = 892$ in⁴/ft
 $S_g = I_g / (t/2) = 185$ in³/ft

Allowable Stresses and Cracking Moment:

$h'/t = 14.96 \leq 30$ $-F_{a \max} = 0.2 f'_m$
Maximum Axial Stress for $h'/t > 30 = 0.05 f'_m = 75$ psi
Maximum Axial Stress for $h'/t \leq 30 = 0.2 f'_m = 300$ psi
 f_r (per MSJC Table 3.1.7.2.1) = 163.0 psi
 $M_{cr} = S_g f_r / 1000 = 30.2$ kip-in/ft

Saiful Bouquet

333 City Blvd West, Suite 710

Orange, CA 92868

T: 714-464-1117

F:

Project: Fire Station

Number:

Date:

By:

Sheet

of

CMD12.01.01

Filename: 10ft-8in CMU Site Wall.dat

Reinforced Concrete Masonry Out-Of-Plane Loaded Wall per 2013 CBC Section 2108 (2011 MSJC Section 3.3.5)

- Ultimate Strength Design -

Factored Input Out-Of-Plane Loads and Moments:

	Load	Load	P_u	M_u	W_u start	h_{start}	W_u end	h_{end}
	Type	Factor	klf	klf-in	ksf	ft	ksf	ft
Load 1	E	1.0			0.104	0.00	0.104	12.00

Ultimate Axial Loads and Stress Checks:

Load Case	$P_{u0} + P_{uw}$	@ h	$P_{u0} + P_{uw}/A_g$	Stress Ratio	
	lbs	ft	psi		
1.4D	1,680.0	0.00	14.55	0.048	OK
1.2D+1.0E+0.5L+0.2S	1,860.0	0.00	16.10	0.054	OK
1.2D-1.0E+0.5L+0.2S	1,860.0	0.00	16.10	0.054	OK
1.2D+1.0E	1,860.0	0.00	16.10	0.054	OK
1.2D-1.0E	1,860.0	0.00	16.10	0.054	OK

Service Load Moments and Maximum Deflections:- Maximum Allowable Deflection, $\Delta_{s \max} = 0.007h = 1.008$ in

Equivalent Unfactored Load Case	P_s	M_{s0}	$M_{sP\Delta}$	@ h	Δ_s	@ h	
	kips	kip-in	kip-in	ft	in	ft	
D+0.75[L+S+0.7E]	1.42	-47.17	-47.46	0.00	0.4037	12.00	$\leq \Delta_{\max}$ - OK
D+0.7E	1.49	-62.90	-63.39	0.00	0.6557	12.00	$\leq \Delta_{\max}$ - OK
D-0.7E	1.49	62.90	63.39	0.00	-0.6557	12.00	$\leq \Delta_{\max}$ - OK
0.6D+0.7E	0.43	-62.90	-63.04	0.00	0.6583	12.00	$\leq \Delta_{\max}$ - OK
0.6D-0.7E	0.43	62.90	63.04	0.00	-0.6583	12.00	$\leq \Delta_{\max}$ - OK

Ultimate and Nominal Shear:

Load Case	Governing Location	V_u	ϕV_n	$\phi = 0.80$ Strength Ratio	
		kips	kips		
1.2D+1.0E+0.5L+0.2S	Bottom	1.25	8.42	0.15	OK
1.2D-1.0E+0.5L+0.2S	Bottom	1.25	8.42	0.15	OK
1.2D+1.0E	Bottom	1.25	8.42	0.15	OK
0.9D+1.0E	Bottom	1.25	8.18	0.15	OK
0.9D-1.0E	Bottom	1.25	8.18	0.15	OK

ADDENDUM 5

Saiful Bouquet

333 City Blvd West, Suite 710

Orange, CA 92868

T: 714-464-1117

F:

Project:

Number:

Date:

By:

Sheet

of

CMD12.01.01

Filename: 10ft-8in CMU Site Wall.dat

Reinforced Concrete Masonry Out-Of-Plane Loaded Wall per 2013 CBC Section 2108 (2011 MSJC Section 3.3.5)

- Ultimate Strength Design -

Ultimate Axial Load and Data for Ultimate Positive Moment Calculations:

Load Case	P_u kips	@ h ft	A_{se} in ²	c in	I_{cr} in ⁴
1.4D	1.68	0.00	0.36	1.88	243.1
1.2D+1.6L+0.5Lr	1.44	0.00	0.36	1.86	242.3
1.2D-1.0E+0.5L+0.2S	1.86	0.00	0.36	1.90	243.7
1.2D-1.0E	1.86	0.00	0.36	1.90	243.7
0.9D-1.0E	0.66	0.00	0.34	1.78	239.5

Ultimate Positive Moment and Deflection:

Load Case	M_n kip-in	ϕM_n kip-in	M_{u0} kip-in	$M_{uP\Delta}$ kip-in	$\phi = 0.90$ Strength Ratio	Δ_u in	
1.4D	152.3	137.1	0.00	0.00	0.00	0.000	OK
1.2D+1.6L+0.5Lr	151.4	136.3	0.00	0.00	0.00	0.000	OK
1.2D-1.0E+0.5L+0.2S	153.0	137.7	89.86	90.87	0.66	1.086	OK
1.2D-1.0E	153.0	137.7	89.86	90.87	0.66	1.086	OK
0.9D-1.0E	148.5	133.7	89.86	90.22	0.67	1.092	OK

Ultimate Axial Load and Data for Ultimate Negative Moment Calculations:

Load Case	P_u kips	@ h ft	A_{se} in ²	c in	I_{cr} in ⁴
1.4D	1.68	0.00	0.36	1.88	243.1
1.2D+1.6L+0.5Lr	1.44	0.00	0.36	1.86	242.3
1.2D+1.0E+0.5L+0.2S	1.86	0.00	0.36	1.90	243.7
1.2D+1.0E	1.86	0.00	0.36	1.90	243.7
0.9D+1.0E	0.66	0.00	0.34	1.78	239.5

Ultimate Negative Moment and Deflection:

Load Case	M_n kip-in	ϕM_n kip-in	M_{u0} kip-in	$M_{uP\Delta}$ kip-in	$\phi = 0.90$ Strength Ratio	Δ_u in	
1.4D	-152.3	-137.1	0.00	0.00	0.00	0.000	OK
1.2D+1.6L+0.5Lr	-151.4	-136.3	0.00	0.00	0.00	0.000	OK
1.2D+1.0E+0.5L+0.2S	-153.0	-137.7	-89.86	-90.87	0.66	-1.086	OK
1.2D+1.0E	-153.0	-137.7	-89.86	-90.87	0.66	-1.086	OK
0.9D+1.0E	-148.5	-133.7	-89.86	-90.22	0.67	-1.092	OK

ADDENDUM 5

Cantilevered Retaining Wall

File = W:\14608_~1.143\ENGINE~1\FIREST~1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19
Licensee : SAIFUL - BOUQUET CONSULTING ENGINEERS

Lic. #: KW-06003631

Description : 10'-8" CMU Site Wall (Footing Design ONLY)

Calculations per ACI 318-11, ACI 530-11, IBC 2012,
CBC 2013, ASCE 7-10

Criteria

Retained Height	=	2.00 ft
Wall height above soil	=	10.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	16.00 in
Water height over heel	=	0.0 ft
Vertical component of active Lateral soil pressure options:		
NOT USED for Soil Pressure.		
NOT USED for Sliding Resistance.		
NOT USED for Overturning Resistance.		

Soil Data

Allow Soil Bearing	=	4,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	36.0 psf/ft
Toe Active Pressure	=	36.0 psf/ft
Passive Pressure	=	350.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.400
Soil height to ignore for passive pressure	=	12.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	104.0 plf
...Height to Top	=	12.00 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Wind on Exposed Stem = 0.0 psf

Design Summary

Wall Stability Ratios

Overturning	=	1.32 Ratio < 1.5!
Sliding	=	2.07 OK

Total Bearing Load	=	4,143 lbs
...resultant ecc.	=	36.41 in

Soil Pressure @ Toe	=	3,858 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	4,500 psf
Soil Pressure Less Than Allowable		

ACI Factored @ Toe	=	4,629 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	20.7 psi OK
Allowable	=	82.2 psi

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	1,320.0 lbs
less 100% Passive Force	=	- 1,069.4 lbs
less 100% Friction Force	=	- 1,650.0 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Load Factors

Dead Load		1.200
Live Load		1.600
Earth, H		1.600
Wind, W		1.600
Seismic, E		1.000

Stem Construction

Design Height Above Ftg

Wall Material Above "Ht"	=	Masonry
Thickness	in =	10.00
Rebar Size	=	# 6
Rebar Spacing	in =	16.00
Rebar Placed at	=	Edge

Design Data

fb/FB + fa/Fa	=	0.769
Total Force @ Section	lbs =	1,312.0
Moment.....Actual	ft-l =	7,542.0
Moment.....Allowable	ft-l =	9,805.4
Shear.....Actual	psi =	15.1
Shear.....Allowable	psi =	69.7
Wall Weight	psf =	98.0
Rebar Depth 'd'	in =	7.25
Lap splice if above	in =	54.00
Lap splice if below	in =	7.67
Hook embed into footing	in =	7.67

Masonry Data

f'm	psi =	1,500
Fs	psi =	20,000
Solid Grouting	=	Yes

Modular Ratio 'n'	=	21.48
Short Term Factor	=	1.000
Equiv. Solid Thick.	in =	9.60
Masonry Block Type	=	2
Masonry Design Method	=	LRFD

ADDENDUM 5

Cantilevered Retaining Wall

File = W:\14608 -1.143\ENGINE-1\FIREST-1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: 10'-8" CMU Site Wall (Footing Design ONLY)

Footing Dimensions & Strengths

Toe Width	=	0.00 ft
Heel Width	=	7.50
Total Footing Width	=	7.50
Footing Thickness	=	16.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c =	3,000 psi	Fy = 60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top	2.00	@ Btm. = 3.00 in

Footing Design Results

	Toe	Heel
Factored Pressure	= 4,629	0 psf
Mu' : Upward	= 0	0 ft-lb
Mu' : Downward	= 0	11,911 ft-lb
Mu: Design	= 0	11,911 ft-lb
Actual 1-Way Shear	= 0.00	20.74 psi
Allow 1-Way Shear	= 0.00	82.16 psi
Toe Reinforcing	= # 7 @ 16.00 in	
Heel Reinforcing	= # 6 @ 16.00 in	
Key Reinforcing	= None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr

Heel: #4@ 8.25 in, #5@ 13.00 in, #6@ 18.25 in, #7@ 24.75 in, #8@ 32.75 in, #9@ 41

Key: No key defined

Summary of Overturning & Resisting Forces & Moments

ItemOVERTURNING.....			RESISTING.....		
	Force lbs	Distance ft	Moment ft-lb		Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	= 200.0	1.11	222.2	Soil Over Heel	= 1,466.7	4.17	6,111.1
Surcharge over Heel	=			Sloped Soil Over Heel	=		
Toe Active Pressure	= -128.0	0.89	-113.8	Surcharge Over Heel	=		
Surcharge Over Toe	=			Adjacent Footing Load	=		
Adjacent Footing Load	=			Axial Dead Load on Stem	=		
Added Lateral Load	= 1,248.0	7.33	9,152.0	* Axial Live Load on Stem	=		
Load @ Stem Above Soil	=			Soil Over Toe	=		
				Surcharge Over Toe	=		
				Stem Weight(s)	= 1,176.0	0.42	490.0
				Earth @ Stem Transitions	=		
				Footing Weight	= 1,500.0	3.75	5,625.0
				Key Weight	=		
				Vert. Component	=		
Total	= 1,320.0	O.T.M. =	9,260.4	Total =	4,142.7 lbs	R.M. =	12,226.1
Resisting/Overturning Ratio	= 1.32						
Vertical Loads used for Soil Pressure =	4,142.7 lbs						

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

ADDENDUM 5

4.4 FUEL DISPENSING AREA DESIGN



Project: Fire Station
SBI Job No.:
Description: Loading Criteria
Date

Level: Fuel Dispensing Area (Generator & Hose Storage)

Dead Load:

Item	Gravity	Joist	Truss	Seismic
Roofing	5.0			5.0
1/4" Densdeck	1.0			1.0
6" Rigid Insulation	1.5			1.5
1 9/32" Plywood Diaphragm	2.0			2.0
Ceiling	3.0			3.0
MEP/Sprinkler	3.0			3.0
Misc	2.5			2.5
Joist Framing		2.0		2.0
Partitions (Seismic Only)	0.0			5.0
	18.0 psf	20.0 psf	20.0 psf	25.0 psf

Live Load: Roof **20.0 psf** (Reducible)

Ext Wall Loads: Stucco **18.0 psf**

FUEL DISPENSING AREA - FLAT ROOF - GENERATOR

$DL = 18 \text{ PSF}$ $L_R = 20 \text{ PSF}$

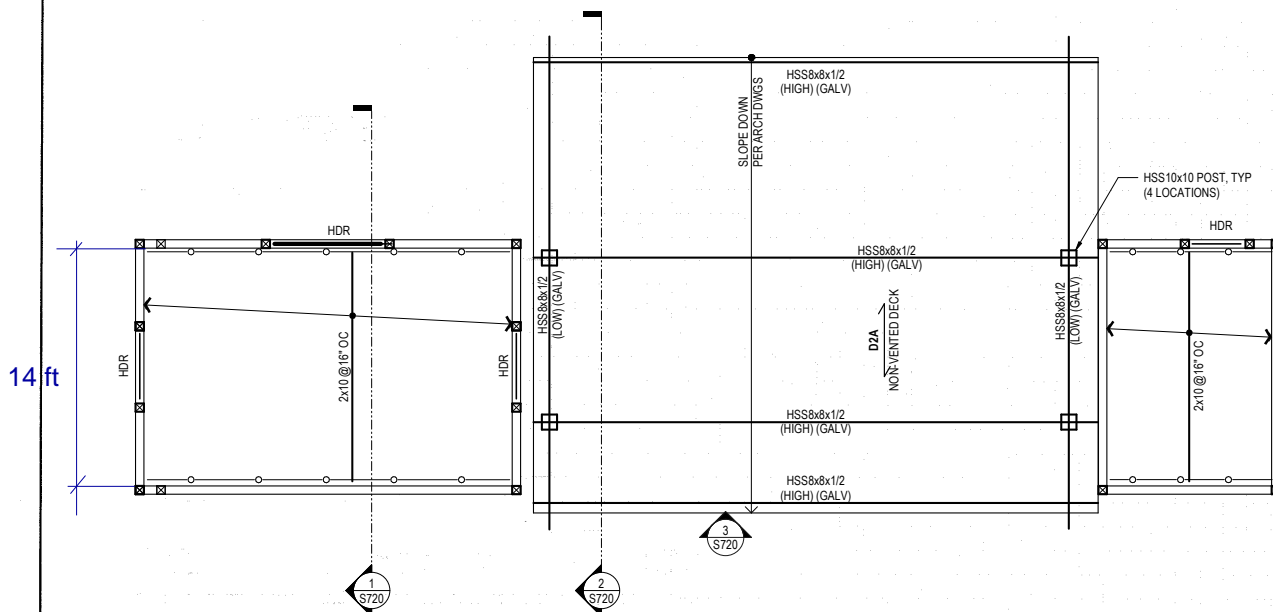
SPAN = 14 FT

$w_D = 18 \text{ PSF} (1.33 \text{ FT}) = 24 \text{ lb/ft}$

$w_L = 20 \text{ PSF} (1.33 \text{ FT}) = 26.7 \text{ lb/ft}$

SPACING: 16" O.C.

USE 2x10 @ 16" O.C.



Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Fuel Dispensing- Flat Roof Joist

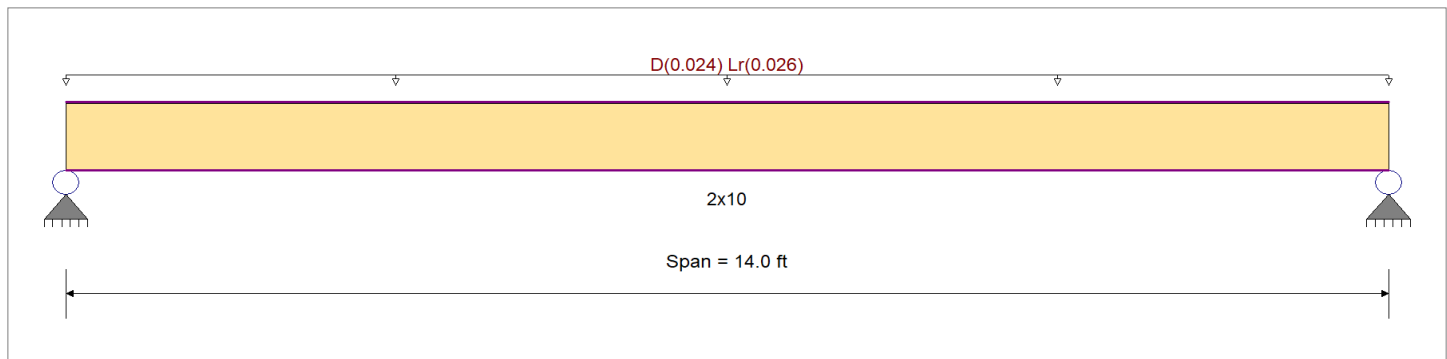
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design	Fb +	1000 psi	E : Modulus of Elasticity	
Load Combination : IBC 2021	Fb -	1000 psi	Ebend- xx	1700ksi
	Fc - Prll	1500 psi	Eminbend - xx	620ksi
Wood Species : Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade : No.1	Fv	180 psi		
	Ft	675 psi	Density	31.21 pcf
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.0240, Lr = 0.0260 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.461 : 1		Section used for this span	=	0.159 : 1	
fb: Actual	=	728.55 psi		fv: Actual	=	35.72 psi	
F'b	=	1,581.25 psi		F'v	=	225.00 psi	
Load Combination	=	+D+Lr		Load Combination	=	+D+Lr	
Location of maximum on span	=	7.000 ft		Location of maximum on span	=	13.234 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.134 in	Ratio =	1249 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.274 in	Ratio =	613 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 14.0 ft	1	0.326	0.112	0.90	1.00	1.00	1.00	1.100	1.00	1.00	1.15	0.66	371.2	1,138.5	0.17	18.2	162.0
+D+Lr					1.00	1.00	1.00	1.100	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 14.0 ft	1	0.461	0.159	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.15	1.30	728.5	1,581.3	0.33	35.7	225.0
+D+0.750Lr					1.00	1.00	1.00	1.100	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 14.0 ft	1	0.404	0.139	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.15	1.14	639.2	1,581.3	0.29	31.3	225.0
+0.60D					1.00	1.00	1.00	1.100	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 14.0 ft	1	0.110	0.038	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.15	0.40	222.7	2,024.0	0.10	10.9	288.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Fuel Dispensing- Flat Roof Joist

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.2740	7.051		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.371	0.371
Max Upward from Load Combinations	0.371	0.371
Max Upward from Load Cases	0.189	0.189
D Only	0.189	0.189
+D+Lr	0.371	0.371
+D+0.750Lr	0.326	0.326
+0.60D	0.113	0.113
Lr Only	0.182	0.182

FUEL DISPENSING AREA - HEADER (HOSE STORAGE)

$$DL = 20 \text{ PSF} \quad LL = 20 \text{ PSF}$$

$$TW = 7'-0" \quad \text{SPAN} : 3'-8"$$

$$W_D = 20 \text{ PSF} (7 \text{ FT}) + 18 \text{ PSF} (3.67 \text{ FT}) = 206 \text{ PLF}$$

$$W_L = 20 \text{ PSF} (7 \text{ FT}) = 140 \text{ PLF}$$

HEADER (GENERATOR)

$$TW = 7 \text{ FT} \quad \text{SPAN} : 6'-8"$$

$$W_D = 20 \text{ PSF} (7 \text{ FT}) + 18 \text{ PSF} (4.167 \text{ FT}) = 215 \text{ PLF}$$

$$W_L = 20 \text{ PSF} (7 \text{ FT}) = 140 \text{ PLF}$$

Multiple Simple Beam

File = W:\14608_~1\143ENGINE~1\FUELDI~1.EC6

ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Lic. #: KW-06003631

Licensee : SAIFUL - BOUQUET CONSULTING ENGINEERS

Description : 4" & 6" Headers (Fuel Dispensing Area)

Wood Beam Design : 3'-8" Span, 4x6

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **4x6, Sawn, Fully Unbraced**

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir - Larch

Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Beam self weight calculated and added to loads

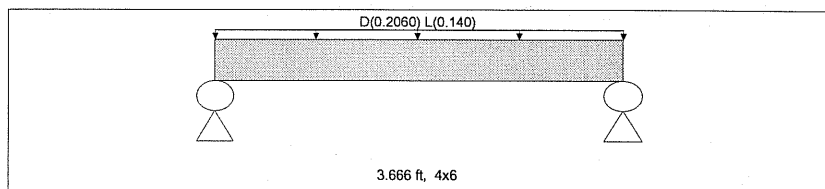
Unif Load: D = 0.2060, L = 0.140 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.216 : 1**
fb : Actual : 544.22 psi at 1.833 ft in Span # 1
Fb : Allowable : 2,523.17 psi
Load Comb : +1.20D+0.50Lr+1.60L+1.60H

Max fv/FvRatio = **0.132 : 1**
fv : Actual : 51.26 psi at 3.214 ft in Span # 1
Fv : Allowable : 388.80 psi
Load Comb : +1.20D+0.50Lr+1.60L+1.60H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.39	0.26					
Right Support	0.39	0.26					



Max Deflections				
Downward L+Lr+S	0.007 in	Downward Total	0.018 in	
Upward L+Lr+S	0.000 in	Upward Total	0.000 in	
Live Load Defl Ratio	5971 >360	Total Defl Ratio	2386 >180	

Wood Beam Design : 6'-8" Span, 4x10

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **4x10, Sawn, Fully Unbraced**

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir - Larch

Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Beam self weight calculated and added to loads

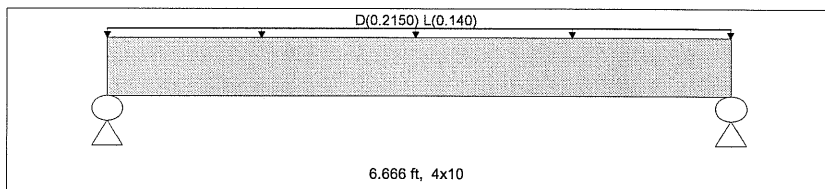
Unif Load: D = 0.2150, L = 0.140 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.282 : 1**
fb : Actual : 655.28 psi at 3.333 ft in Span # 1
Fb : Allowable : 2,323.98 psi
Load Comb : +1.20D+0.50Lr+1.60L+1.60H

Max fv/FvRatio = **0.151 : 1**
fv : Actual : 58.60 psi at 5.911 ft in Span # 1
Fv : Allowable : 388.80 psi
Load Comb : +1.20D+0.50Lr+1.60L+1.60H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.74	0.47					
Right Support	0.74	0.47					



Max Deflections				
Downward L+Lr+S	0.017 in	Downward Total	0.044 in	
Upward L+Lr+S	0.000 in	Upward Total	0.000 in	
Live Load Defl Ratio	4724 >360	Total Defl Ratio	1826 >180	

Wood Beam Design : 3'-8" Span, 6x6

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x6, Sawn, Fully Unbraced**

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir - Larch

Wood Grade : No.1

Fb - Tension	1,000.0 psi	Fc - Prll	1,500.0 psi	Fv	180.0 psi	Ebend- xx	1,700.0 ksi	Density	32.210 pcf
Fb - Compr	1,000.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	620.0 ksi		

Applied Loads

Beam self weight calculated and added to loads

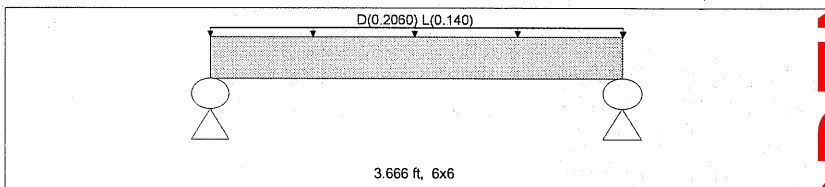
Unif Load: D = 0.2060, L = 0.140 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.161 : 1**
fb : Actual : 348.47 psi at 1.833 ft in Span # 1
Fb : Allowable : 2,159.00 psi
Load Comb : +1.20D+0.50Lr+1.60L+1.60H

Max fv/FvRatio = **0.084 : 1**
fv : Actual : 32.82 psi at 3.214 ft in Span # 1
Fv : Allowable : 388.80 psi
Load Comb : +1.20D+0.50Lr+1.60L+1.60H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.39	0.26					
Right Support	0.39	0.26					



Max Deflections				
Downward L+Lr+S	0.004 in	Downward Total	0.011 in	
Upward L+Lr+S	0.000 in	Upward Total	0.000 in	
Live Load Defl Ratio	9970 >360	Total Defl Ratio	3956 >180	

Multiple Simple Beam

File = W:\14608_~1\143\ENGINE~1\FUELDI~1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Lic. #: KW-06003631

Licensee : SAIFUL - BOUQUET CONSULTING ENGINEERS

Wood Beam Design : 6'-8" Span, 6x10

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x10, Sawn, Fully Unbraced**

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir - Larch

Wood Grade : No.1

Fb - Tension	1,000.0 psi	Fc - Prll	1,500.0 psi	Fv	180.0 psi	Ebend- xx	1,700.0 ksi	Density	32.210 pcf
Fb - Compr	1,000.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	620.0 ksi		

Applied Loads

Beam self weight calculated and added to loads

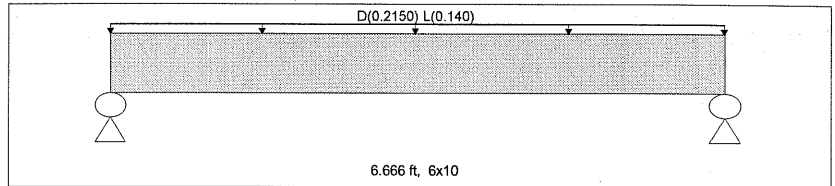
Unif Load: D = 0.2150, L = 0.140 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.185 : 1**
fb : Actual : 399.64 psi at 3.333 ft in Span # 1
Fb : Allowable : 2,156.51 psi
Load Comb : +1.20D+0.50Lr+1.60L+1.60H

Max fv/FvRatio = **0.094 : 1**
fv : Actual : 36.39 psi at 5.888 ft in Span # 1
Fv : Allowable : 388.80 psi
Load Comb : +1.20D+0.50Lr+1.60L+1.60H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.76	0.47					
Right Support	0.76	0.47					



Max Deflections			
Downward L+Lr+S	0.009 in	Downward Total	0.025 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	8546 >360	Total Defl Ratio	3262 >180

P - REDUNDANCY FACTOR

GENERATOR

IN N-S DIRECTION,

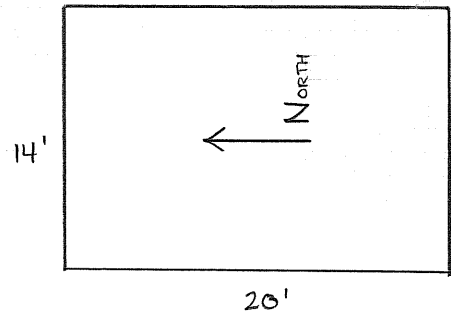
ELIMINATING A 6'-10" SHEAR WALL (WEST WALL)
DECREASES STORY STRENGTH ABOUT
25%, WHICH IS LESS THAN 33% (12.3-3)

$$P = 1.0$$

IN W-E DIRECTION

ELIMINATING EITHER THE 4'-6" OR 5'-0" WALL (SOUTH WALL)
DECREASES STORY STRENGTH ABOUT 25%, LESS THAN 33%. (12.3-3)

$$P = 1.0$$



HOSE STORAGE

IN N-S DIRECTION

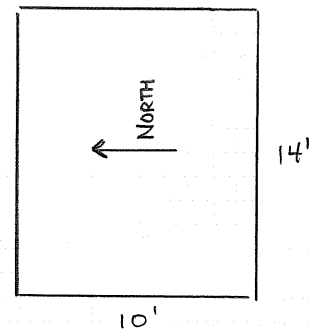
ELIMINATING THE 4'-8" WALL ON THE WEST
FACE REDUCES STRENGTH BY 50%, WHICH
IS GREATER THAN 33%.

$$P = 1.3$$

IN W-E DIRECTION

SHEAR WALLS HAVE HEIGHT:LENGTH RATIOS < 1.0

$$P = 1.0$$



SHEAR WALLS - GENERATOR

ROOF LOADS

25 PSF

$$(25 \text{ PSF})(14 \text{ FT})(20 \text{ FT}) = 7.0 \text{ k}$$

WALLS

18 PSF

$$(18 \text{ PSF})(20 \text{ FT})(13 \text{ FT})(2) + (18 \text{ PSF})(14 \text{ FT})(13 \text{ FT}/2)(2) = 12.64 \text{ k}$$

$$\begin{aligned} \text{TOTAL WEIGHT} &= 7.0 + 12.64 \\ &= 19.6 \text{ k} \end{aligned}$$

$$C_s = 0.401$$

$$\text{BASE SHEAR, } V_b = C_s W = 0.401(19.6) = 7.9 \text{ k}$$

$$2 \text{ WALLS } \therefore 7.9 / 2 = 3.94 \text{ k EACH SIDE}$$

SOUTH WALL ($p=1.0$)

4'-6" AND 5'-0" WALL SECTIONS

$$\text{DEMAND: } 3.94 \text{ k} / (4.5 + 5) = 415 \text{ lb/ft} \quad \text{To ASD, } 415 / 1.4 = 296 \text{ lb/ft}$$

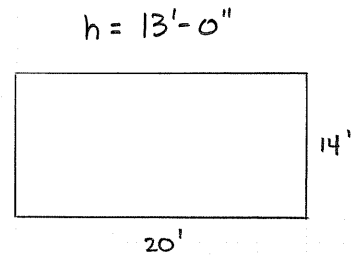
$$\text{CAPACITY: } 510 \text{ lb/ft} \left(\frac{2(4.5 \text{ FT})}{(13 \text{ FT})} \right) = 353 \text{ lb/ft} \quad \underline{\text{TYPE B}}$$

WEST WALL ($p=1.0$)

2 - 6'-10" WALLS

$$\text{DEMAND: } 3.94 \text{ k} / (6.83)(2) = 289 \text{ lb/ft} \quad \text{To ASD, } 289 / 1.4 = 206 \text{ lb/ft}$$

$$\text{CAPACITY: } 340 \text{ lb/ft} \quad \underline{\text{TYPE A}}$$



SHEAR WALLS - HOSE STORAGE

ROOF LOADS

25 PSF

$$(25)(14)(10)(1.01) = 3.54 \text{ k}$$

↳ SLOPE

WALLS

18 PSF

$$(2)(18 \text{ PSF}) \left[(14 \text{ FT})(13 \text{ FT}) + (10 \text{ FT})(13 \text{ FT}/2) \right] = 8.9 \text{ k}$$

$$\text{TOTAL WT} = 3.54 + 8.9 = 12.44 \text{ k}$$

$$C_s = 0.401$$

$$V_b = C_s W = 0.401 (12.44 \text{ k}) = 4.99 \text{ k}$$

2 WALLS, EACH WALL SEES 2.5 k

WEST WALL ($p=1.3$)

4'-8" SECTION

$$\text{DEMAND: } (2.5 \text{ k}/4.67)(1.3) = 696 \text{ lb/ft} \quad \text{TO ASD, } 696/1.4 = 497 \text{ lb/ft}$$

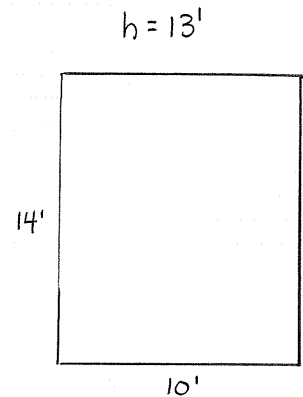
$$\text{CAPACITY: } 870 \text{ lb/ft} \left(\frac{2(4.67 \text{ FT})}{(13 \text{ FT})} \right) = 625 \text{ lb/ft} \quad \underline{\text{TYPE D}}$$

NORTH WALL ($p=1.0$)

14'-0" SECTION

$$\text{DEMAND: } 2.5 \text{ k}/14 \text{ FT} = 179 \text{ lb/ft} \quad \text{TO ASD, } 179/1.4 = 128 \text{ lb/ft}$$

$$\text{CAPACITY: } 340 \text{ lb/ft} \quad \underline{\text{TYPE A}}$$



HOLD DOWNS

GENERATOR

$$DL = 18 \text{ PSF}$$

SOUTH WALL

4'-6" AND 5'-0" SECTIONS

4'-6" SECTION

$$V = (296 \text{ lb/ft})(4.5 \text{ FT}) = 1.332 \text{ k}$$

$$M_{OT} = Vh = 1.332 \text{ k} (13 \text{ FT}) = 17.3 \text{ k} \cdot \text{ft}$$

$$M_{RES} = \left[(18 \text{ PSF})(13 \text{ FT}) \right] (4.5 \text{ FT})^2 / 2 = 2.37 \text{ k} \cdot \text{ft}$$

$$T = \frac{M_{OT} - 0.9 M_{RES}}{(4.5 - 0.5) \text{ FT}} = \frac{17.3 - 0.9 (2.37)}{4.0} = 3.8 \text{ k}$$

USE HDU5

CAPACITY: 4.52 k

5'-0" SECTION

$$\text{SIM. } T = 3.69 \text{ k}$$

USE HDU5

WEST WALL

6'-10" SECTION

$$V = (206 \text{ lb/ft})(6.83 \text{ FT}) = 1.41 \text{ k}$$

$$M_{OT} = Vh = 1.41 \text{ k} (13 \text{ FT}) = 18.3 \text{ k} \cdot \text{ft}$$

$$M_{RES} = \left[(18 \text{ PSF})(13 \text{ FT}) + (18 \text{ PSF})(7 \text{ FT}) \right] (6.83 \text{ FT})^2 / 2 = 8.4 \text{ k} \cdot \text{ft}$$

$$T = \frac{M_{OT} - 0.9 M_{RES}}{(6.83 - 0.5) \text{ FT}} = 1.59 \text{ k}$$

USE HDU4

CAPACITY: 3.65 k

20'-0" SECTION

$$T = -0.57 \text{ k}$$

USE HDU4

HOSE STORAGE

$$DL = 18 \text{ PSF}$$

WEST WALL

4'-8" SECTION

$$V = 497 \text{ lb/ft} (4.67 \text{ FT}) = 2.32 \text{ k}$$

$$M_{OT} = Vh = 2.32 \text{ k} (13 \text{ FT}) = 30.2 \text{ k}\cdot\text{ft}$$

$$M_{RES} = \left[(18 \text{ PSF})(13 \text{ FT}) + (18 \text{ PSF})(7 \text{ FT}) \right] (4.67 \text{ FT})^2 / 2 = 3.92 \text{ k}\cdot\text{ft}$$

$$T = \frac{M_{OT} - 0.9 M_{RES}}{(4.67 - 0.5) \text{ FT}} = 6.4 \text{ k}$$

USE HDU 11 CAPACITY: 8.94k

10'-0" SECTION

$$T = 5.1 \text{ k}$$

USE HDU 8 CAPACITY: 6.92k

NORTH WALL

14'-0" SECTION

$$V = 128 \text{ lb/ft} (14 \text{ FT}) = 1.8 \text{ k}$$

$$M_{OT} = Vh = 1.8 \text{ k} (13 \text{ FT}) = 23.3 \text{ k}\cdot\text{ft}$$

$$M_{RES} = \left[(18 \text{ PSF})(13 \text{ FT}) \right] (14 \text{ FT})^2 / 2 = 22.9 \text{ k}\cdot\text{ft}$$

$$T = \frac{M_{OT} - 0.9 M_{RES}}{(14 - 0.5) \text{ FT}} = 0.20 \text{ k}$$

HDU 4 CAPACITY: 3.65k

DIAPHRAGM - GENERATOR (SPANS W TO E)

$$W_{px} = (25 \text{ PSF})(14 \text{ FT})(20 \text{ FT}) + (20 \text{ FT})(13 \text{ FT}/2)(18 \text{ PSF}) = 9.34 \text{ k}$$

$$F_{px} = \frac{V}{W_{px}} W_{px} = V = 7.9 \text{ k} \quad (12.10-1)$$

$$F_{px, \min} = 0.2 S_{DS} I_e W_{px} = (0.2)(1.739)(1.5)(9.34) = 4.87 \text{ k}$$

$$F_{px, \max} = 0.4 S_{DS} I_e W_{px} = (0.4)(1.739)(1.5)(9.34) = 9.75 \text{ k}$$

$$w = F_{px}/L = 7.9 \text{ k}/20 \text{ FT} = 0.395 \text{ k/ft}$$

$$V_{\max} = \frac{wL}{2} = 3.95 \text{ k}$$

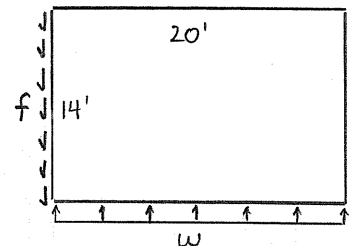
$$f = \frac{V_{\max}}{14 \text{ FT}} = 282 \text{ lb/ft}$$

CASE 1

$$V_{S, ASD} = 282/1.4 = \underline{\underline{201.5 \text{ lb/ft}}}$$

19/32" THICK PLYWOOD (BLOCKED DIAPHRAGM)

10d @ 6" B.N. , 10d @ 6" E.N.



$$V_{S, NOM} = 720 \text{ lb/ft} \quad (\text{TABLE 4.2A})$$

$$\text{CAPACITY: } 720/2.0 = 360 \text{ lb/ft} \quad \checkmark$$

GENERATOR (SPANS N TO S)

$$W_{px} = (25 \text{ PSF})(14 \text{ FT})(20 \text{ FT}) + (18 \text{ PSF})(14 \text{ FT})(13 \text{ FT}/2)$$

$$= 8.64 \text{ k}$$

$$F_{px} = \frac{V}{W_{px}} W_{px} = V = 7.9 \text{ k}$$

$$F_{px, \min} = 0.2 S_{DS} I_e W_{px} = 0.2 (1.739) (1.5) (8.64) = 4.51 \text{ k}$$

$$F_{px, \max} = 0.4 S_{DS} I_e W_{px} = 9.02 \text{ k}$$

$$w = F_{px}/L = 7.9 \text{ k}/14 \text{ FT} = 564 \text{ lb/ft}$$

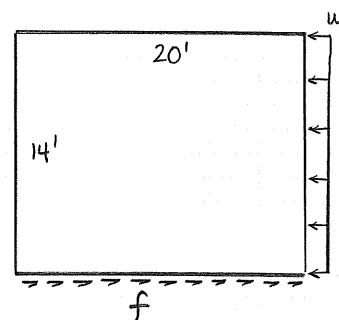
$$V_{\max} = \frac{wL}{2} = 3.95 \text{ k}$$

$$f = \frac{V_{\max}}{20 \text{ FT}} = 198 \text{ lb/ft}$$

CASE 3

$$V_{S, ASD} = 198/1.4 = \underline{\underline{141 \text{ lb/ft}}}$$

19/32" THICK, 10d @ 6" SIMILAR TO PREVIOUS



HOSE STORAGE (N TO S)

$$W_{px} = (25 \text{ PSF})(14 \text{ FT})(10 \text{ FT}) + (18 \text{ PSF})(13 \text{ FT}/2)(14 \text{ FT})$$

$$= 5.14 \text{ k}$$

$$F_{px} = 5 \text{ k}$$

$$F_{px, \min} = 0.2(1.739)(1.5)(5.14) = 2.68 \text{ k}$$

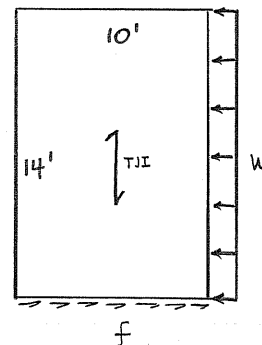
$$F_{px, \max} = 0.4(1.739)(1.5)(5.14) = 5.36 \text{ k}$$

$$w = F_{px}/L = 5/14 = 357 \text{ lb/ft}$$

$$V_{\max} = \frac{wL}{2} = 2.5 \text{ k}$$

$$f = V_{\max}/10 \text{ FT} = 250 \text{ lb/ft}$$

$$V_{s, \text{ASD}} = 250/1.4 = \underline{\underline{179 \text{ lb/ft}}}$$



CASE 3

SIMILAR TO GENERATOR DIAPHRAGM

19/32" THICK, 10d @ 6"

W TO E

N TO S CONDITION GOVERNS

CHORD AND COLLECTOR FORCES

GENERATOR

WEST WALL

$$W = F_{px} / L = 7.9 \text{ k} / 20 \text{ FT} = 395 \text{ PLF}$$

$$M_{\max} = \frac{WL^2}{8} = 395(20)^2 / 8 = 19.75 \text{ K} \cdot \text{FT}$$

$$\text{CHORD FORCE} = \frac{M_{\max}}{14 \text{ FT}} = 1.41 \text{ K}$$

$$W = F_{px} / L = 7.9 / 14 = 564 \text{ PLF}$$

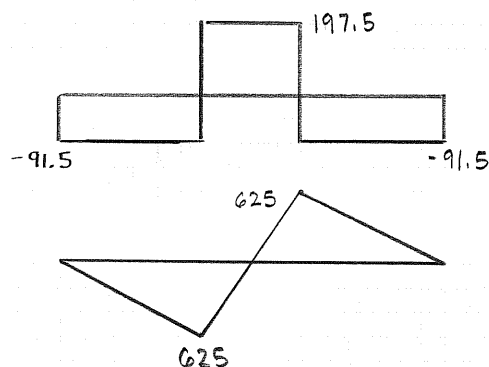
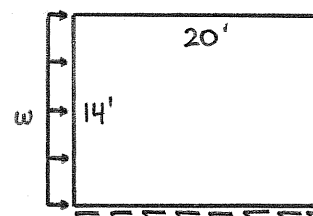
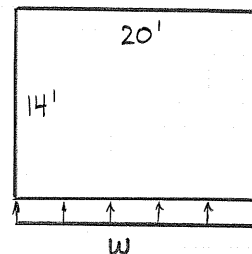
$$V_{\max} = \frac{WL}{2} = 3.95 \text{ K}$$

$$\text{WALL LENGTH AVAILABLE} = 2(6'-10") = 13'-8"$$

$$f = V_{\max} / 13.67 \text{ FT} = 289 \text{ PLF}$$

$$\text{COLLECTOR FORCE} = 625 \text{ lb}$$

$$\text{DESIGN FORCE} = \underline{1.41 \text{ K}}$$



$$f_t = \frac{P}{A_n} \leq F_t'$$

$$A_{n_{2 \times 6}} = 8.25 \text{ IN}^2$$

$$f_t = \frac{1.41 \text{ K}}{8.25 \text{ IN}^2} = 171 \text{ PSI}$$

$$F_t' = F_t (C_d)(C_M)(C_t)(C_F)(C_i)$$

$$= (675 \text{ PSI})(1.6)(1.0)(1.0)(1.3)(1.0) = 1404 \text{ PSI} \quad \checkmark$$

USE 2x6 STANDARD TOP PL AS CHORD

SOUTH WALL

$$W = F_{px} / L = 7.9k / 14_{FT} = 564 \text{ PLF}$$

$$M_{max} = \frac{WL^2}{8} = 564(14)^2 / 8 = 13.82 \text{ k} \cdot \text{FT}$$

$$\text{CHORD FORCE} = M_{max} / 20_{FT} = 691 \text{ lb}$$

$$W = F_{px} / L = 7.9 / 20 = 395 \text{ PLF}$$

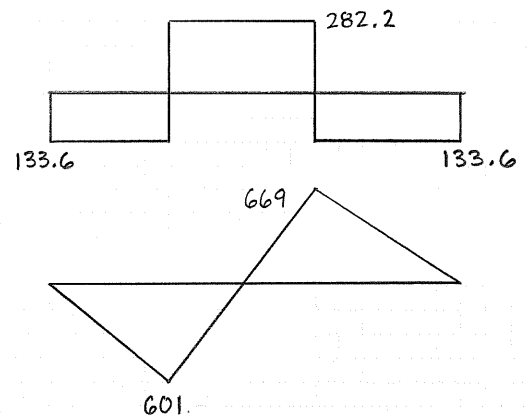
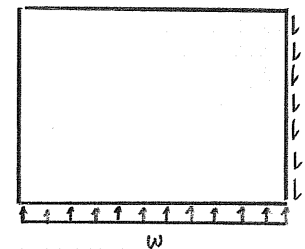
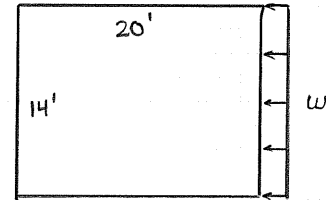
$$V_{max} = \frac{WL}{2} = 3.95 \text{ k}$$

$$\text{WALL LENGTH AVAILABLE} = 9' - 6"$$

$$f = V_{max} / 9.5_{FT} = 415.8 \text{ PLF}$$

$$\text{COLLECTOR FORCE} = 669 \text{ lb}$$

$$\text{DESIGN FORCE} = \underline{\underline{691 \text{ lb}}}$$



HOSE STORAGE

WEST WALL

$$w = F_{px}/L = 5/10 = 500 \text{ PLF}$$

$$M_{max} = \frac{wL^2}{8} = 500(10)^2/8 = 6.25 \text{ k}\cdot\text{FT}$$

$$\text{CHORD FORCE} = M_{max}/14 \text{ FT} = 447 \text{ lb}$$

$$w = F_{px}/L = 5/14 = 357 \text{ PLF}$$

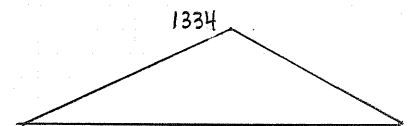
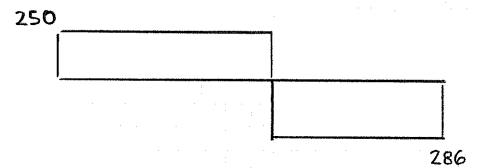
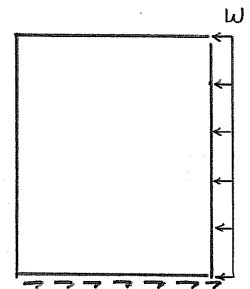
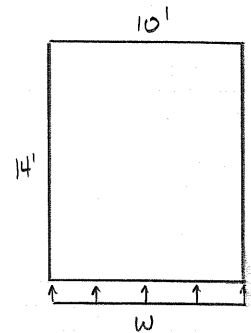
$$V_{max} = \frac{wL}{2} = 2.5 \text{ k}$$

$$\text{WALL LENGTH AVAILABLE} = 4' - 8"$$

$$f = V_{max}/4.67 \text{ FT} = 536 \text{ PLF}$$

$$\text{COLLECTOR FORCE} = 1334 \text{ lb}$$

$$\text{DESIGN FORCE} = \underline{1.33 \text{ k}}$$



4.5 FUEL DISPENSING CANOPY DESIGN



Project: Fire Station
SBI Job No.:
Description: Loading Criteria
Date

Level: Fuel Dispensing Area (Fuel Tank Canopy)

Dead Load:

Item	Gravity	Joist	Truss	Seismic
Roofing	3.0			3.0
Framing (Seismic)	0.0			14.0
Misc	3.0			3.0
	6.0 psf	6.0 psf	6.0 psf	20.0 psf

Live Load: Roof

20.0 psf

(Reducible)

FUEL TANK CANOPY

DEAD LOAD: 3 PSF DECK
3 PSF MISC

LIVE LOAD: 20 PSF

WIND LOAD: $P = q_h G C_N$ (27.4-3)
(MWFRS)
 $q_h = 0.00256 K_h K_{zt} K_d V^2$
 $= 25.89 \text{ PSF}$

$K_h = 0.90$ EXPOSURE C (27.3-1)
 $K_{zt} = 1.0$
 $K_d = 0.85$ (26.6-1)
 $V = 115$ (26.5-1B)

$$T = C_t h_n^x \quad (12.8.2.1)$$

$$= 0.02 (20)^{0.75} = 0.189$$

$$f = \frac{1}{T} = 5.29$$

IF $f > 1$, RIGID

$$\text{RIGID} \rightarrow G = 0.85 \quad (26.9.1)$$

$$C_N: \theta = \tan^{-1} \left(\frac{1.5}{12} \right) = 7.13^\circ$$

$$C_N = 1.2 \text{ OR } -1.1 \quad (27.4-4)$$

$$P = (25.9)(0.85)(1.2) = 26.4 \text{ PSF}$$

WIND LOAD: $P = q_h G C_N$ (OPEN BUILDINGS)
(C & C)

$$q_h = 25.9 \text{ PSF}$$

$$G = 0.85$$

$$C_N = 1.6 \text{ OR } -1.4$$

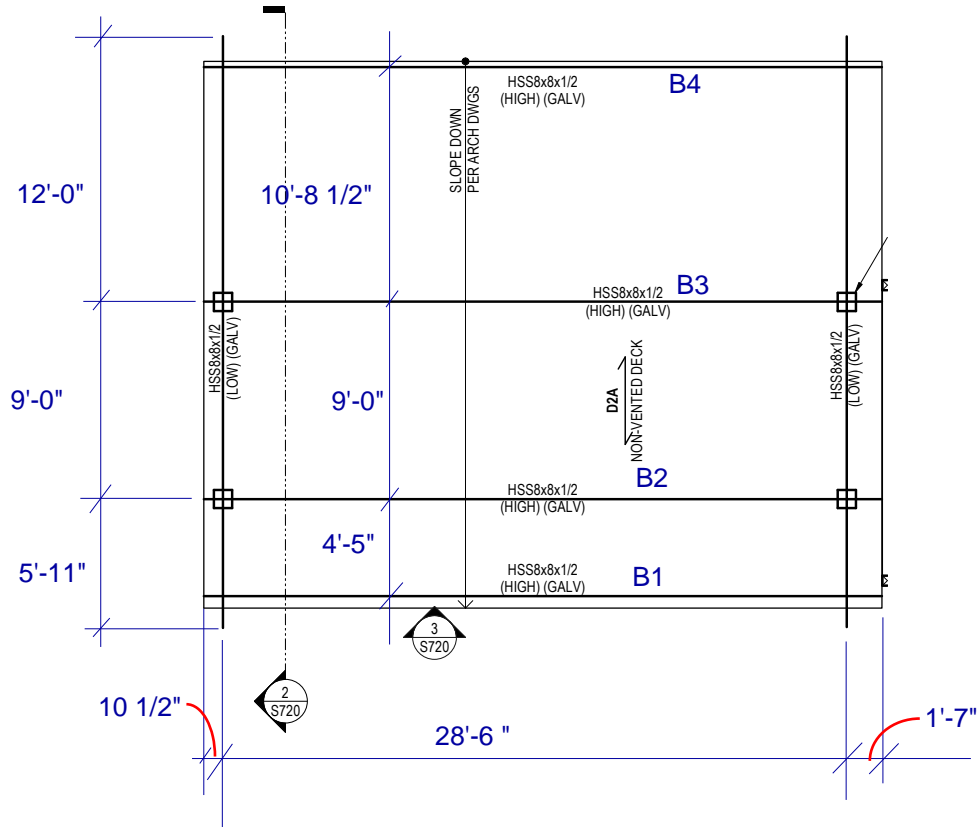
$$P = 35.2 \text{ PSF OR } -30.8 \text{ PSF}$$

$$h = 18.65 \text{ FT}$$

$$L = 27 \text{ FT}$$

$$h/L = 0.69$$

$$a = 3 \text{ FT}$$



CANOPY BEAM & GIRDER DESIGN

$$DL = 6 \text{ PSF} \quad LL = 20 \text{ PSF} \quad W = \pm 26.4 \text{ PSF}$$

B1

$$W_D = (6 \text{ PSF})(4.83 \text{ FT}/2 + 1 \text{ FT}) = 20.5 \text{ PLF}$$

$$W_L = (20 \text{ PSF})(3.415 \text{ FT}) = 68.3 \text{ PLF}$$

$$W_W = (35.2 \text{ PSF})(3.415 \text{ FT}) = 120.2 \text{ PLF}$$

B2

$$W_D = (6 \text{ PSF})(9 \text{ FT}/2 + 4.83 \text{ FT}/2) = 41.5 \text{ PLF}$$

$$W_L = (20 \text{ PSF})(6.917 \text{ FT}) = 138 \text{ PLF}$$

$$W_W = (35.2 \text{ PSF})(6.917 \text{ FT}) = 243 \text{ PLF}$$

B3

$$W_D = (6 \text{ PSF})(11.17 \text{ FT}/2 + 9 \text{ FT}/2) = 60.5 \text{ PLF}$$

$$W_L = (20 \text{ PSF})(10.08 \text{ FT}) = 202 \text{ PLF}$$

$$W_W = (35.2 \text{ PSF})(10.08 \text{ FT}) = 355 \text{ PLF}$$

B4

$$W_D = (6 \text{ PSF})(11.17 \text{ FT}/2 + 1 \text{ FT}) = 39.5 \text{ PLF}$$

$$W_L = (20 \text{ PSF})(6.58 \text{ FT}) = 132 \text{ PLF}$$

$$W_W = (35.2 \text{ PSF})(6.58 \text{ FT}) = 232 \text{ PLF}$$

GIRDER

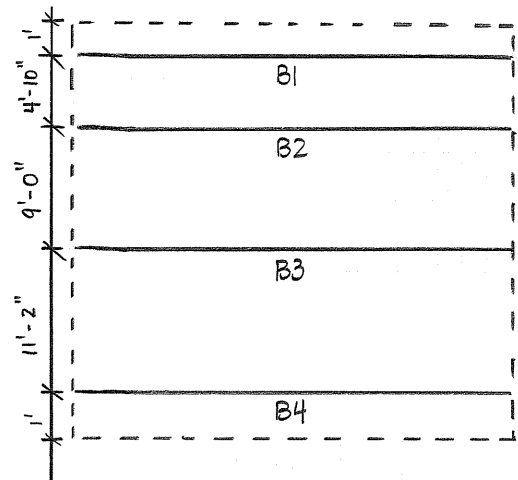
$$\text{TRIBUTARY WIDTH} = 15' - 6"$$

INCREASE LOADS FOR SLOPED ROOF

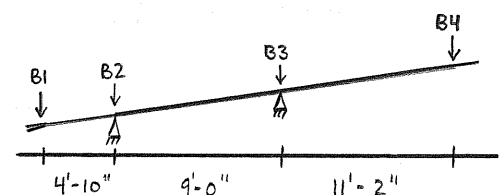
$$\text{USE FACTOR OF } \left(\frac{27.21}{27} \right) = 1.0078$$

USE BEAM REACTIONS AS LOADS

USE HSS 8 x 8 x 1/2 GIRDERS



USE HSS 8 x 8 x 1/2
FOR ALL BEAMS



Multiple Simple Beam

Lic. #: KW-06003631

File = W:\14608_~1.143\ENGINE~1\FUELDI~1.EC6

ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: Canopy Beams

Steel Beam Design: B1 Down

Calculations per AISC 360-10, IBC 20012, CBC 2013, ASCE 7-10

STEEL Section: HSS8x8x1/2, Fully Braced

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Fy = 46.0 ksi E = 29,000.0 ksi

Applied Loads

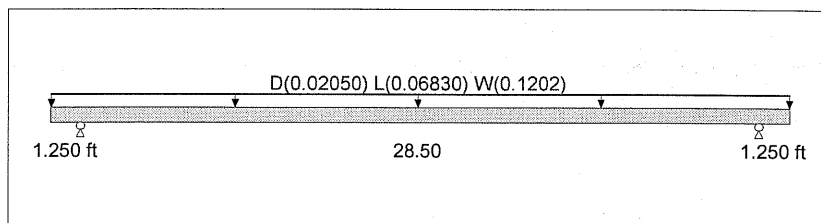
Beam self weight calculated and added to loads

Unif Load: D = 0.02050, L = 0.06830, W = 0.1202 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.185 : 1
Mu : Applied 23.920 k-ft at 14.250 ft in Span # 2
Mn * Phi : Allow 129.375 k-ft
Load Comb : +1.20D+0.50Lr+0.50L+W+1.60H
Max fv/FvRatio = 0.022 : 1
Vu : Applied 3.383 k at 1.250 ft in Span # 1
Vn * Phi : Allow 152.583 k
Load Comb : +1.20D+0.50Lr+0.50L+W+1.60H

Max Reactions (k)	D	L	Lr	S	W	E
Left Support	1.07	1.06			1.86	
Right Support	1.07	1.06			1.86	



H	Max Deflections		
	Downward L+Lr+S	0.493 in	Downward Total 0.716 in
	Upward L+Lr+S	-0.068 in	Upward Total -0.099 in
	Live Load Defl Ratio	438	Total Defl Ratio 302

Steel Beam Design: B1 Up (Unbraced)

Calculations per AISC 360-10, IBC 20012, CBC 2013, ASCE 7-10

STEEL Section: HSS8x8x1/2, Fully Unbraced

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Fy = 46.0 ksi E = 29,000.0 ksi

Applied Loads

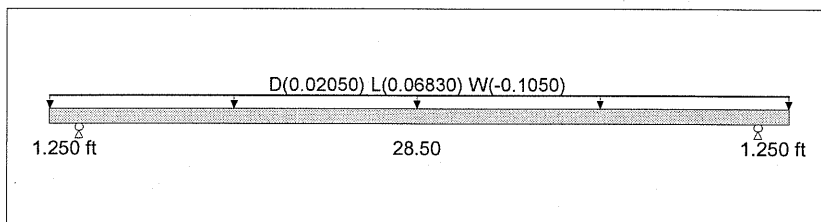
Beam self weight calculated and added to loads

Unif Load: D = 0.02050, L = 0.06830, W = -0.1050 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.150 : 1
Mu : Applied 19.379 k-ft at 14.250 ft in Span # 2
Mn * Phi : Allow 129.375 k-ft
Load Comb : +1.20D+0.50Lr+1.60L+1.60H
Max fv/FvRatio = 0.018 : 1
Vu : Applied 2.741 k at 1.250 ft in Span # 1
Vn * Phi : Allow 152.583 k
Load Comb : +1.20D+0.50Lr+1.60L+1.60H

Max Reactions (k)	D	L	Lr	S	W	E
Left Support	1.07	1.06			-1.63	
Right Support	1.07	1.06			-1.63	



H	Max Deflections		
	Downward L+Lr+S	0.280 in	Downward Total 0.564 in
	Upward L+Lr+S	-0.431 in	Upward Total -0.088 in
	Live Load Defl Ratio	502	Total Defl Ratio 384

Steel Beam Design: B2 Down

Calculations per AISC 360-10, IBC 20012, CBC 2013, ASCE 7-10

STEEL Section: HSS8x8x1/2, Fully Braced

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Fy = 46.0 ksi E = 29,000.0 ksi

Applied Loads

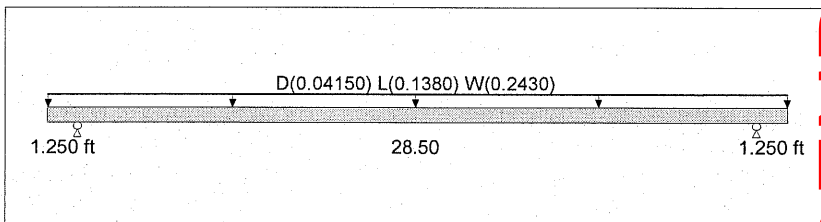
Beam self weight calculated and added to loads

Unif Load: D = 0.04150, L = 0.1380, W = 0.2430 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.327 : 1
Mu : Applied 42.342 k-ft at 14.250 ft in Span # 2
Mn * Phi : Allow 129.375 k-ft
Load Comb : +1.20D+0.50Lr+0.50L+W+1.60H
Max fv/FvRatio = 0.039 : 1
Vu : Applied 5.989 k at 1.250 ft in Span # 1
Vn * Phi : Allow 152.583 k
Load Comb : +1.20D+0.50Lr+0.50L+W+1.60H

Max Reactions (k)	D	L	Lr	S	W	E
Left Support	1.40	2.14			3.77	
Right Support	1.40	2.14			3.77	



H	Max Deflections		
	Downward L+Lr+S	0.996 in	Downward Total 1.243 in
	Upward L+Lr+S	-0.138 in	Upward Total -0.172 in
	Live Load Defl Ratio	216	Total Defl Ratio 174 < 180

Multiple Simple Beam

Lic. #: KW-06003631

File = W:\14608_~1\143\ENGINE~1\FUELDI~1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Steel Beam Design : B2 Up (Unbraced)

Calculations per AISC 360-10, IBC 20012, CBC 2013, ASCE 7-10

STEEL Section : HSS8x8x1/2, Fully Unbraced

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Fy = 46.0 ksi E = 29,000.0 ksi

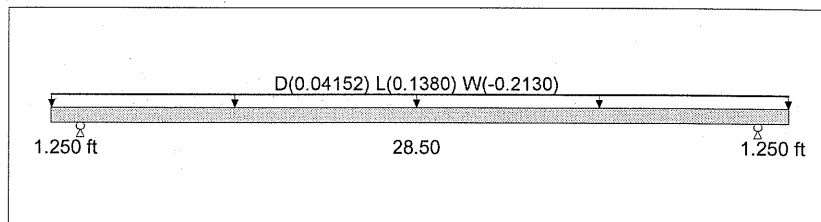
Applied Loads

Beam self weight calculated and added to loads

Unif Load: D = 0.04152, L = 0.1380, W = -0.2130 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.256 : 1**
Mu : Applied 33.156 k-ft at 14.250 ft in Span # 2
Mn * Phi : Allow 129.375 k-ft
Load Comb : +1.20D+0.50Lr+1.60L+1.60H
Max fv/FvRatio = **0.031 : 1**
Vu : Applied 4.690 k at 1.250 ft in Span # 1
Vn * Phi : Allow 152.583 k
Load Comb : +1.20D+0.50Lr+1.60L+1.60H



Max Reactions (k)	D	L	Lr	S	W	E
Left Support	1.40	2.14			-3.30	
Right Support	1.40	2.14			-3.30	

Max Deflections			
Downward L+Lr+S	0.566 in	Downward Total	0.936 in
Upward L+Lr+S	-0.873 in	Upward Total	-0.302 in
Live Load Defl Ratio	246	Total Defl Ratio	230

Steel Beam Design : B3 Down

Calculations per AISC 360-10, IBC 20012, CBC 2013, ASCE 7-10

STEEL Section : HSS8x8x1/2, Fully Braced

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Fy = 46.0 ksi E = 29,000.0 ksi

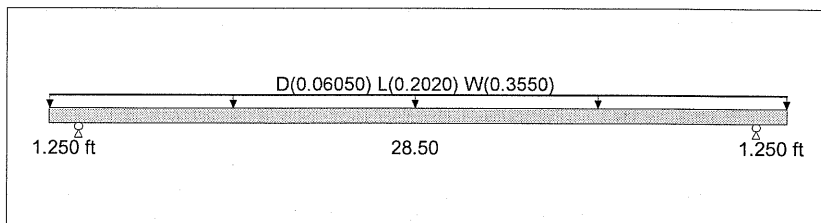
Applied Loads

Beam self weight calculated and added to loads

Unif Load: D = 0.06050, L = 0.2020, W = 0.3550 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.457 : 1**
Mu : Applied 59.147 k-ft at 14.250 ft in Span # 2
Mn * Phi : Allow 129.375 k-ft
Load Comb : +1.20D+0.50Lr+0.50L+W+1.60H
Max fv/FvRatio = **0.055 : 1**
Vu : Applied 8.366 k at 1.250 ft in Span # 1
Vn * Phi : Allow 152.583 k
Load Comb : +1.20D+0.50Lr+0.50L+W+1.60H



Max Reactions (k)	D	L	Lr	S	W	E
Left Support	1.69	3.13			5.50	
Right Support	1.69	3.13			5.50	

Max Deflections			
Downward L+Lr+S	1.456 in	Downward Total	1.724 in
Upward L+Lr+S	-0.202 in	Upward Total	-0.239 in
Live Load Defl Ratio	148 < 180	Total Defl Ratio	124 < 180

Steel Beam Design : B3 Up (Unbraced)

Calculations per AISC 360-10, IBC 20012, CBC 2013, ASCE 7-10

STEEL Section : HSS8x8x1/2, Fully Unbraced

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Fy = 46.0 ksi E = 29,000.0 ksi

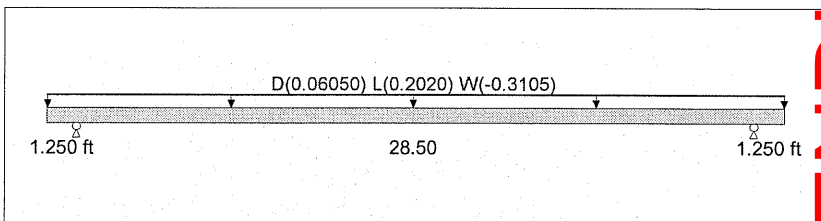
Applied Loads

Beam self weight calculated and added to loads

Unif Load: D = 0.06050, L = 0.2020, W = -0.3105 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.354 : 1**
Mu : Applied 45.767 k-ft at 14.250 ft in Span # 2
Mn * Phi : Allow 129.375 k-ft
Load Comb : +1.20D+0.50Lr+1.60L+1.60H
Max fv/FvRatio = **0.042 : 1**
Vu : Applied 6.473 k at 1.250 ft in Span # 1
Vn * Phi : Allow 152.583 k
Load Comb : +1.20D+0.50Lr+1.60L+1.60H



Max Reactions (k)	D	L	Lr	S	W	E
Left Support	1.69	3.13			-4.81	
Right Support	1.69	3.13			-4.81	

Max Deflections			
Downward L+Lr+S	0.828 in	Downward Total	1.276 in
Upward L+Lr+S	-1.273 in	Upward Total	-0.495 in
Live Load Defl Ratio	170 < 180	Total Defl Ratio	168 < 180

Multiple Simple Beam

File = W:\14608_~1.143\ENGINE~1\FUELDI~1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Steel Beam Design : B4 Down

Calculations per AISC 360-10, IBC 20012, CBC 2013, ASCE 7-10

STEEL Section : HSS8x8x1/2, Fully Braced

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Fy = 46.0 ksi E = 29,000.0 ksi

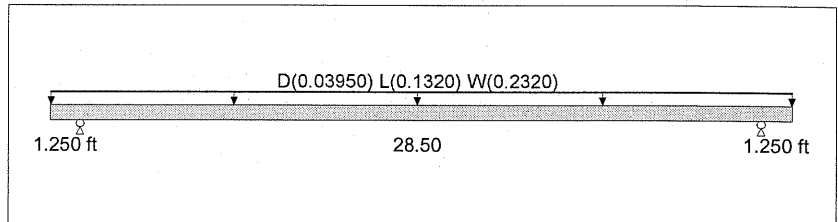
Applied Loads

Beam self weight calculated and added to loads

Unif Load: D = 0.03950, L = 0.1320, W = 0.2320 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.315 : 1
Mu : Applied 40.689 k-ft at 14.250 ft in Span # 2
Mn * Phi : Allow 129.375 k-ft
Load Comb : +1.20D+0.50Lr+0.50L+W+1.60H
Max fv/FvRatio = 0.038 : 1
Vu : Applied 5.755 k at 1.250 ft in Span # 1
Vn * Phi : Allow 152.583 k
Load Comb : +1.20D+0.50Lr+0.50L+W+1.60H



Max Reactions (k) D L Lr S W E
Left Support 1.37 2.05 3.60
Right Support 1.37 2.05 3.60

Max Deflections
Downward L+Lr+S 0.951 in Downward Total 1.196 in
Upward L+Lr+S -0.132 in Upward Total -0.166 in
Live Load Defl Ratio 226 Total Defl Ratio 180

Steel Beam Design : B4 Up (Unbraced)

Calculations per AISC 360-10, IBC 20012, CBC 2013, ASCE 7-10

STEEL Section : HSS8x8x1/2, Fully Unbraced

Using Load Resistance Factor Design with IBC 2012 Load Combinations, Major Axis Bending

Fy = 46.0 ksi E = 29,000.0 ksi

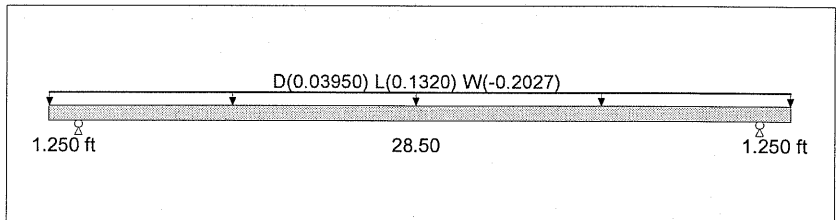
Applied Loads

Beam self weight calculated and added to loads

Unif Load: D = 0.03950, L = 0.1320, W = -0.2027 k/ft, Trib = 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.247 : 1
Mu : Applied 31.944 k-ft at 14.250 ft in Span # 2
Mn * Phi : Allow 129.375 k-ft
Load Comb : +1.20D+0.50Lr+1.60L+1.60H
Max fv/FvRatio = 0.030 : 1
Vu : Applied 4.518 k at 1.250 ft in Span # 1
Vn * Phi : Allow 152.583 k
Load Comb : +1.20D+0.50Lr+1.60L+1.60H



Max Reactions (k) D L Lr S W E
Left Support 1.37 2.05 -3.14
Right Support 1.37 2.05 -3.14

Max Deflections
Downward L+Lr+S 0.541 in Downward Total 0.903 in
Upward L+Lr+S -0.831 in Upward Total -0.282 in
Live Load Defl Ratio 260 Total Defl Ratio 238

Steel Beam

File = W:\14608_~1\143\ENGINE~1\FUELDI~1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: Canopy Girder

CODE REFERENCES

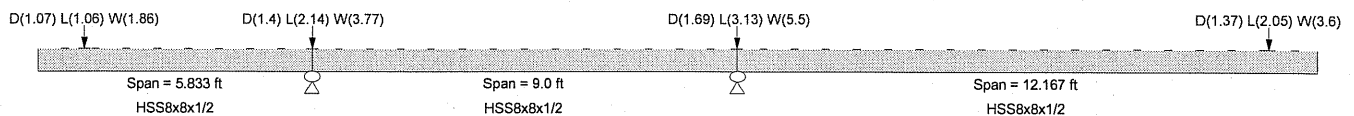
Calculations per AISC 360-10, IBC 2012, ASCE 7-10

Load Combination Set: IBC 2012

Material Properties

Analysis Method: Load Resistance Factor Design
Beam Bracing: Beam is Fully Braced against lateral-torsional buckling
Bending Axis: Major Axis Bending
Load Combination: IBC 2012

Fy: Steel Yield: 46.0 ksi
E: Modulus: 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading

Load(s) for Span Number 1

Point Load: D = 1.070, L = 1.060, W = 1.860 k @ 1.0 ft, (B1 Down)

Load(s) for Span Number 2

Point Load: D = 1.40, L = 2.140, W = 3.770 k @ 0.0 ft, (B2 Down)

Point Load: D = 1.690, L = 3.130, W = 5.50 k @ 9.0 ft, (B3 Down)

Load(s) for Span Number 3

Point Load: D = 1.370, L = 2.050, W = 3.60 k @ 11.167 ft, (B4 Down)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio =	0.575 : 1	Maximum Shear Stress Ratio =	0.102 : 1
Section used for this span	HSS8x8x1/2	Section used for this span	HSS8x8x1/2
Mu : Applied	74.331 k-ft	Vu : Applied	15.532 k
Mn * Phi : Allowable	129.375 k-ft	Vn * Phi : Allowable	152.583 k
Load Combination	+1.20D+0.50Lr+0.50L+W+1.60H	Load Combination	+1.20D+0.50Lr+0.50L+W+1.60H
Location of maximum on span	9.000 ft	Location of maximum on span	9.000 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2
Maximum Deflection			
Max Downward Transient Deflection	1.673 in	Ratio =	174 < 180
Max Upward Transient Deflection	-0.122 in	Ratio =	882
Max Downward Total Deflection	2.244 in	Ratio =	130 < 180
Max Upward Total Deflection	-0.167 in	Ratio =	646

STRUCTURE WEIGHT (SEISMIC MASS)

ROOF :

DECK : 3 PSF

MISC : 3 PSF

FRAMING :

HSS 8x8x1/2 : 48.72 PLF

COLUMNS : 62.46 PLF

4 BEAMS @ 31 FT AND 2 GIRDERS @ 27 FT

8.67k

4 COLUMNS (2 @ 17.77 FT, 2 @ 18.895 FT)

2.3k

$$\frac{8.67 + 2.3}{(25 \times 31)} = 14 \text{ PSF}$$

$$\Sigma DL = 20 \text{ PSF}$$

$$W = 20 \text{ PSF} \times A = 20 \text{ PSF} \times (25 \text{ FT} \times 31 \text{ FT})$$

$$W = 15.5 \text{ k}$$

BASE SHEAR

Equivalent Lateral Force Procedure

Project: **Fire Station**
SBI Job No.:

Building: **Fuel Canopy**
LFRS: **Ordinary Cantilever Column**
Direction: **X**

Building Data

Occupancy Category = **IV** Table 1604.5, 2013 CBC Importance Factor, I_e = **1.50** Table 1.5-2

Seismic Ground Motion Values

Section 11.4

S_S = **2.608** From geotech or Figs.22-1 to 11
 S_I = **0.986** From geotech or Figs.22-1 to 11
Site Class = **D** From geotech or Table 20.3-1
 T_L = **8 sec** Figs.22-12 to 16

F_a = **1.00** Table 11.4-1
 F_v = **1.50** Table 11.4-2
 $S_{MS} = F_a S_S$ = **2.608** Eq. 11.4-1
 $S_{MI} = F_v S_I$ = **1.479** Eq. 11.4-2
 $S_{DS} = (2/3) S_{MS}$ = **1.739** Eq. 11.4-3
 $S_{DI} = (2/3) S_{MI}$ = **0.986** Eq. 11.4-4

Seismic Design Category

F Tables 11.6-1 & 2

Building Period

Section 12.8.2

C_t = **0.02** Table 12.8-2
 α = **0.75** Table 12.8-2
 h_n = **20.33 ft** Height of Building
 T_b = **0.000 sec** From Analysis
(Input zero to use T_a)

$T_a = C_t h_n^\alpha$ = **0.191 sec** Eq. 12.8-7
 C_u = **1.40** Table 12.8-1
 $T_{a, max} = C_u T_a$ = **0.268 sec** Section. 12.8.2

Period = **0.191 sec** «-- used for design
0.191 sec «-- used for drift
Section. 12.8.6.2

Base Shear

Section 12.8

W = **16 kips** Total Structure Weight
 R = **1.25** Table 12.2-1
 C_d = **1.25** Table 12.2-1

For Design Only

C_s = $S_{DS} / (R/I_e) = 2.086$ Eq. 12.8-2
 $C_{s, max}$ = $S_{DI} / [T (R/I_e)] = 6.179$ Eq. 12.8-3, for $T \leq T_L$
 $C_{s, max}$ = $S_{DI} T_L / [T^2 (R/I_e)] = N/A$ Eq. 12.8-4, for $T > T_L$
 $C_{s, min}$ = $\max\{0.01, 0.044 S_{DS} I_e\} = N/A$ Eq. 12.8-5, if $S_I < 0.6g$
 $C_{s, min}$ = $\max\{0.01, 0.044 S_{DS} I_e, 0.5 S_I / (R/I_e)\} = 0.592$ Eq. 12.8-5 and 12.8-6, if $S_I \geq 0.6g$

Use, $C_s = 2.086$

$V_{design} = C_s W = 32$ kips Eq. 12.8-1

For Drift Only

$C_{s, max}$ = 2.086
 C_s = 6.179
 C_s = N/A
 $C_{s, min}$ = N/A
 $C_{s, min}$ = 0.592

Use, $C_s = 2.086$

$V_{drift} = C_s W = 32$ kips
Allowable Drift = **0.015** h_{sx}
Table 12.12-1

* Note: All references are from ASCE 7-10 unless noted otherwise.

ADDENDUM 5

Steel Column

File = W:\14608_1-1.143\ENGINE-1\FUELDI-1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: Canopy Cantilever Column (18'-11")

Code References

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10

Load Combinations Used: IBC 2012

General Information

Steel Section Name: **HSS10x10x5/8**
Analysis Method: Load Resistance Factor
Steel Stress Grade
Fy: Steel Yield 46.0 ksi
E: Elastic Bending Modulus 29,000.0 ksi
Load Combination: IBC 2012

Overall Column Height 18.895 ft
Top & Bottom Fixity Top Free, Bottom Fixed
Brace condition for deflection (buckling) along columns:
X-X (width) axis:
Unbraced Length for X-X Axis buckling = 18.895 ft, K = 2.1
Y-Y (depth) axis:
Unbraced Length for Y-Y Axis buckling = 10 ft, K = 2.1

Applied Loads

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

Column self weight included: 1,438.38 lbs * Dead Load Factor

AXIAL LOADS ...

Girder Reaction: Axial Load at 18.895 ft, D = 5.310, L = 7.150, W = 12.570 k

BENDING LOADS ...

Base Shear: Lat. Point Load at 17.770 ft creating My-y, E = 10.0 k

E = 31% OF BASE SHEAR (32k)

TALL COLUMN HAS GREATER TRIB. WIDTH
THAN SHORT COL.

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = 0.9028 : 1
Load Combination +1.548D+0.50L+0.70S+1.250E+1.6
Location of max. above base 0.0 ft
At maximum location values are ...
Pu 14.020 k
0.9 * Pn 302.156 k
Mu-x 0.0 k-ft
0.9 * Mn-x: 252.540 k-ft
Mu-y -222.125 k-ft
0.9 * Mn-y: 252.540 k-ft

Maximum SERVICE Load Reactions ..

Top along X-X 0.0 k
Bottom along X-X 12.50 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 4.993 in at 18.895 ft above base
for load combination: +1.250E

PASS Maximum Shear Stress Ratio = 0.06294 : 1
Load Combination +1.548D+0.50L+0.70S+1.250E+1.6
Location of max. above base 0.0 ft
At maximum location values are ...
Vu: Applied 12.50 k
Vn * Phi: Allowable 198.609 k

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} = 1.25 * \delta_{xe} / 1.5 = 4.99" / 1.5 = 3.3" < 0.015 * 18.9' * 12 = 3.4" \text{ OK}$$

HSS 10x10x5/8 ok

ADDENDUM 5

FUEL AREA DIAPHRAGM

$$W_{px} = (20 \text{ PSF})(25 \text{ FT})(31 \text{ FT}) = 15.5 \text{ k}$$

$$F_{px} = V_b = 32 \text{ k}$$

$$F_{px, \min} = 0.2(1.739)(1.5)(15.5) = 8.1 \text{ k}$$

$$F_{px, \max} = 16.2 \text{ k}$$

$$W = F_{px}/L = 16.2 \text{ k} / 31 \text{ FT} = 523 \text{ PLF}$$

$$V_{\max} = \frac{WL}{2} = 523(31)/2 = 8.1 \text{ k}$$

$$f = V_{\max} / 25 \text{ FT} = \underline{\underline{324 \text{ PLF}}}$$

IN OPPOSITE DIRECTION THAN SHOWN,

$$f = V_{\max} / 31 \text{ FT} = 261 \text{ PLF}$$

PREVIOUS CASE GOVERNS

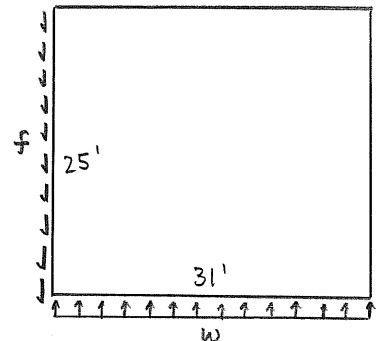
USE VERCO HSN3-NS 16 GAGE DECK

32/7 SCREW PATTERN, SIDELAPS SCREWED

CAPACITY: 12'-0" SPAN,

SCREWS @ 12" OC,

SHEAR STRENGTH = 850 PLF ✓



LONGEST SPAN = 11'-0"

TABLE 37 - ALLOWABLE DIAPHRAGM SHEAR STRENGTH, q (plf), AND FLEXIBILITY FACTORS, F , FOR HSN3™-NS DECK PANELS ATTACHED WITH SDI RECOGNIZED #12 OR #14 SCREWS TO SUPPORTS 0.0385" AND THICKER AND SIDELAPS FASTENED WITH #10 SCREWS^{1,2,3,4,5,6,7,8,9} (Cont'd.)

DECK GAGE	SIDELAP ATTACH- MENT	SPAN (ft.-in.)									
		2'-0"	4'-0"	6'-0"	8'-0"	10'-0"	12'-0"	14'-0"	16'-0"	18'-0"	
32/7 ATTACHMENT PATTERN FOR SDI RECOGNIZED SCREWS											
22	#10 @ 24"	q	643	461	345	288	254	231	215	203	193
		F	-0.9+140R	5.4+69R	8.2+45R	9.8+33R	11+26R	11.8+21R	12.5+18R	13+16R	13.4+14R
	#10 @ 18"	q	697	503	384	347	301	270	265	247	232
		F	-1.5+140R	4.5+69R	7.2+46R	8.2+34R	9.4+27R	10.2+22R	10.4+19R	10.9+16R	11.4+14R
	#10 @ 12"	q	697	542	451	401	369	347	331	318	309
		F	-1.5+140R	3.9+70R	6+46R	7.1+34R	7.8+27R	8.3+23R	8.7+19R	9+17R	9.2+15R
	#10 @ 8"	q	742	610	532	488	460	440	426	415	406
		F	-2+141R	3.1+70R	4.9+46R	5.9+35R	6.4+28R	6.8+23R	7.1+20R	7.3+17R	7.5+15R
	#10 @ 6"	q	780	669	602	564	539	522	510	500	493
		F	-2.3+141R	2.6+70R	4.3+47R	5.1+35R	5.6+28R	6+23R	6.3+20R	6.4+17R	6.6+15R
#10 @ 4"	q	841	761	713	685	667	655	645	638	633	
	F	-2.7+141R	1.9+70R	3.5+47R	4.3+35R	4.8+28R	5.1+23R	5.3+20R	5.5+18R	5.6+16R	
20	#10 @ 24"	q	779	565	428	359	318	291	272	257	246
		F	1.2+88R	5.8+43R	7.9+28R	9.2+20R	10.1+16R	10.8+13R	11.4+11R	11.8+9R	12.1+8R
	#10 @ 18"	q	848	618	479	437	380	343	338	315	297
		F	0.7+89R	5+43R	7.1+28R	7.7+21R	8.7+17R	9.4+14R	9.4+12R	9.9+10R	10.3+9R
	#10 @ 12"	q	848	667	562	502	465	439	420	405	394
		F	0.7+89R	4.4+44R	5.9+29R	6.8+21R	7.3+17R	7.6+14R	7.9+12R	8.1+11R	8.3+9R
	#10 @ 8"	q	905	754	664	613	581	558	542	529	519
		F	0.3+89R	3.7+44R	4.9+29R	5.6+22R	6+17R	6.3+15R	6.5+12R	6.6+11R	6.8+10R
	#10 @ 6"	q	953	827	752	709	681	662	647	636	628
		F	0+89R	3.2+44R	4.3+29R	4.9+22R	5.3+18R	5.5+15R	5.7+13R	5.8+11R	5.9+10R
#10 @ 4"	q	1026	939	886	856	837	823	813	805	799	
	F	-0.4+89R	2.6+44R	3.7+30R	4.2+22R	4.5+18R	4.7+15R	4.9+13R	5+11R	5.1+10R	
18	#10 @ 24"	q	1052	774	601	510	456	419	393	374	359
		F	2.7+43R	5.6+20R	7.1+13R	8.1+9R	8.8+7R	9.3+6R	9.7+5R	10+4R	10.3+3R
	#10 @ 18"	q	1154	854	677	625	550	498	495	463	438
		F	2.2+43R	4.9+21R	6.4+13R	6.8+10R	7.5+8R	8+6R	8+5R	8.4+4R	8.8+4R
	#10 @ 12"	q	1154	926	792	717	669	635	611	593	578
		F	2.2+43R	4.4+21R	5.4+14R	5.9+10R	6.3+8R	6.5+7R	6.7+6R	6.9+5R	7+4R
	#10 @ 8"	q	1236	1051	941	878	837	809	789	773	761
		F	1.9+43R	3.8+21R	4.5+14R	4.9+10R	5.2+8R	5.4+7R	5.5+6R	5.6+5R	5.6+5R
	#10 @ 6"	q	1302	1153	1063	1012	978	955	938	925	915
		F	1.6+43R	3.4+21R	4+14R	4.3+11R	4.6+9R	4.7+7R	4.8+6R	4.9+5R	4.9+5R
#10 @ 4"	q	1401	1302	1243	1209	1187	1172	1161	1152	1145	
	F	1.3+43R	2.9+22R	3.4+14R	3.7+11R	3.9+9R	4+7R	4.1+6R	4.1+5R	4.2+5R	
16	#10 @ 24"	q	1333	993	787	673	605	560	527	503	484
		F	3+24R	5.2+11R	6.4+7R	7.2+5R	7.7+4R	8.2+3R	8.5+2R	8.8+2R	9+1R
	#10 @ 18"	q	1470	1101	883	824	734	670	665	626	594
		F	2.6+24R	4.6+12R	5.7+7R	6+5R	6.6+4R	7.1+3R	7+3R	7.4+2R	7.7+2R
	#10 @ 12"	q	1470	1199	1038	948	890	850	820	798	781
		F	2.6+24R	4.1+12R	4.8+8R	5.3+6R	5.5+4R	5.7+4R	5.9+3R	6+3R	6.1+2R
	#10 @ 8"	q	1578	1363	1235	1161	1114	1081	1057	1039	1024
		F	2.3+24R	3.6+12R	4.1+8R	4.4+6R	4.5+5R	4.7+4R	4.7+3R	4.8+3R	4.9+3R
	#10 @ 6"	q	1663	1494	1392	1334	1296	1270	1250	1235	1224
		F	2.1+25R	3.2+12R	3.6+8R	3.8+6R	4+5R	4.1+4R	4.1+3R	4.2+3R	4.2+3R
#10 @ 4"	q	1786	1678	1615	1578	1555	1538	1526	1517	1509	
	F	1.8+25R	2.7+12R	3.1+8R	3.3+6R	3.4+5R	3.4+4R	3.5+3R	3.5+3R	3.6+3R	

¹ The dimension from the first and last sidelap connection within each span is to be no more than one-half of specified spacing.

² R is the ratio of vertical span (L_v) of the deck to the length (L_s) of the deck sheet: $R = L_v / L_s$

³ Interpolation of diaphragm shear strength between adjacent spans or sidelap spacings is permissible. For interpolation of the diaphragm flexibility factor between adjacent spans, use the flexibility factor for the closest adjacent span length.

⁴ Diaphragm shear values for side seam fasteners placed at spacings other than those in the table should be determined based on the number of fasteners in each span.

⁵ The allowable diaphragm shear values in the table utilize a factor of safety, $\Omega = 2.5$ (limited by connections), with the exception of the shaded table values, which utilize a factor of safety of $\Omega = 2.0$ (limited by panel buckling.)

⁶ See Table 16B page 57 or guide to proper selection of support fastening screws.

⁷ See Table 16C page 57 for adjustment factors when using generic screws and/or steel supports less than 0.0385 in. thick.

⁸ HSN3™-SS deck panels may be used in lieu of HSN3™-NS deck panels.

⁹ See Table 16F page 58 for adjustment factors when using acoustical deck profiles.

Pole Footing Embedded in Soil

File = W:\14608_~1\143\ENGINE~1\FUELDI~1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: Fuel Canopy Pole Footing

Code References

Calculations per IBC 2012 1807.3, CBC 2013, ASCE 7-10

Load Combinations Used: IBC 2012

General Information

Pole Footing Shape: Circular
Footing Diameter: 36.0 in
Calculate Min. Depth for Allowable Pressures
No Lateral Restraint at Ground Surface
Allow Passive: 465.0 pcf
Max Passive: 3,500.0 psf

Controlling Values

Governing Load Combination: +D+0.70E+H

Lateral Load: 7.0 k
Moment: 132.265 k-ft

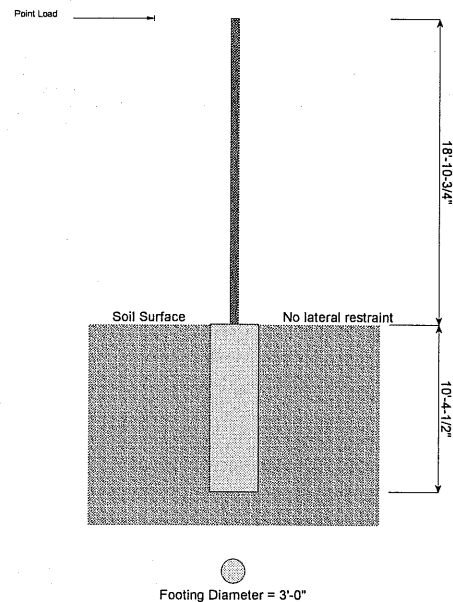
NO Ground Surface Restraint

Pressures at 1/3 Depth

Actual: 1,593.33 psf
Allowable: 1,594.53 psf

Minimum Required Depth: 10.375 ft

Footing Base Area: 7.069 ft²
Maximum Soil Pressure: 1.017 ksf



Applied Loads

Lateral Concentrated Load

D: Dead Load: 0.0 k
Lr: Roof Live: 0.0 k
L: Live: 0.0 k
S: Snow: 0.0 k
W: Wind: 0.0 k
E: Earthquake: 10.0 k
H: Lateral Earth: 0.0 k
Load distance above ground surface: 18.895 ft

Lateral Distributed Load

0.0 k/ft
0.0 k/ft
0.0 k/ft
0.0 k/ft
0.0 k/ft
0.0 k/ft
0.0 k/ft
TOP of Load above ground surface: 0.0 ft
BOTTOM of Load above ground surface: 0.0 ft

Vertical Load

7.192 k
0.0 k
0.0 k
0.0 k
0.0 k
0.0 k
0.0 k

Load Combination Results

Load Combination	Forces @ Ground Surface		Required Depth - (ft)	Pressure at 1/3 Depth		Soil Increase Factor
	Loads - (k)	Moments - (ft-k)		Actual - (psf)	Allow - (psf)	
+D+H	0.000	0.000	0.13	0.0	0.0	1.000
+D+L+H	0.000	0.000	0.13	0.0	0.0	1.000
+D+Lr+H	0.000	0.000	0.13	0.0	0.0	1.000
+D+S+H	0.000	0.000	0.13	0.0	0.0	1.000
+D+0.750Lr+0.750L+H	0.000	0.000	0.13	0.0	0.0	1.000
+D+0.750L+0.750S+H	0.000	0.000	0.13	0.0	0.0	1.000
+D+0.60W+H	0.000	0.000	0.13	0.0	0.0	1.000
+D+0.70E+H	7.000	132.265	10.38	1,593.3	1,594.5	1.000
+D+0.750Lr+0.750L+0.450W+H	0.000	0.000	0.13	0.0	0.0	1.000

Pole Footing Embedded in Soil

File = W:\14608_~1\143\ENGINE~1\FUELDI~1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: Fuel Canopy Pole Footing

+D+0.750L+0.750S+0.450W+H	0.000	0.000	0.13	0.0	0.0	1.000
+D+0.750L+0.750S+0.5250E+H	5.250	99.199	9.25	1,431.1	1,432.1	1.000
+0.60D+0.60W+0.60H	0.000	0.000	0.13	0.0	0.0	1.000
+0.60D+0.70E+0.60H	7.000	132.265	10.38	1,593.3	1,594.5	1.000

FOOTING REINFORCING DESIGN

OVERSTRENGTH COMBINATIONS

$\Omega_o = 1.25$ FOR ORDINARY CANTILEVER COLUMNS

$$(1.2 + 0.2 S_{DS})D + \Omega_o Q_E + L + 0.2 S$$

$$P_u = [1.2 + 0.2(1.739)](5.31) + 7.15$$

$$= 15.4 \text{ k}$$

$$M_u = 1.25(188.95) = 236.2 \text{ k}\cdot\text{FT}$$

$$D = 5.31 \text{ k}$$

$$L = 7.15 \text{ k}$$

$$Q_E = 10 \text{ k}(18.895 \text{ FT})$$

$$= 188.95 \text{ k}\cdot\text{FT}$$

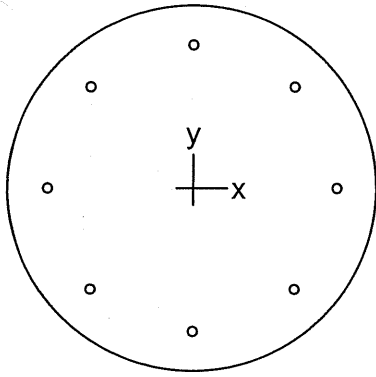
$$(0.9 - 0.2 S_{DS})D + \Omega_o Q_E + 1.6 H$$

$$P_u = [0.9 - 0.2(1.739)](5.31)$$

$$= 2.93 \text{ k}$$

$$M_u = 1.25(188.95) = 236.2 \text{ k}\cdot\text{FT}$$

SEE SP COLUMN RESULTS



36 in diam.

Code: ACI 318-08

Units: English

Run axis: About X-axis

Run option: Investigation

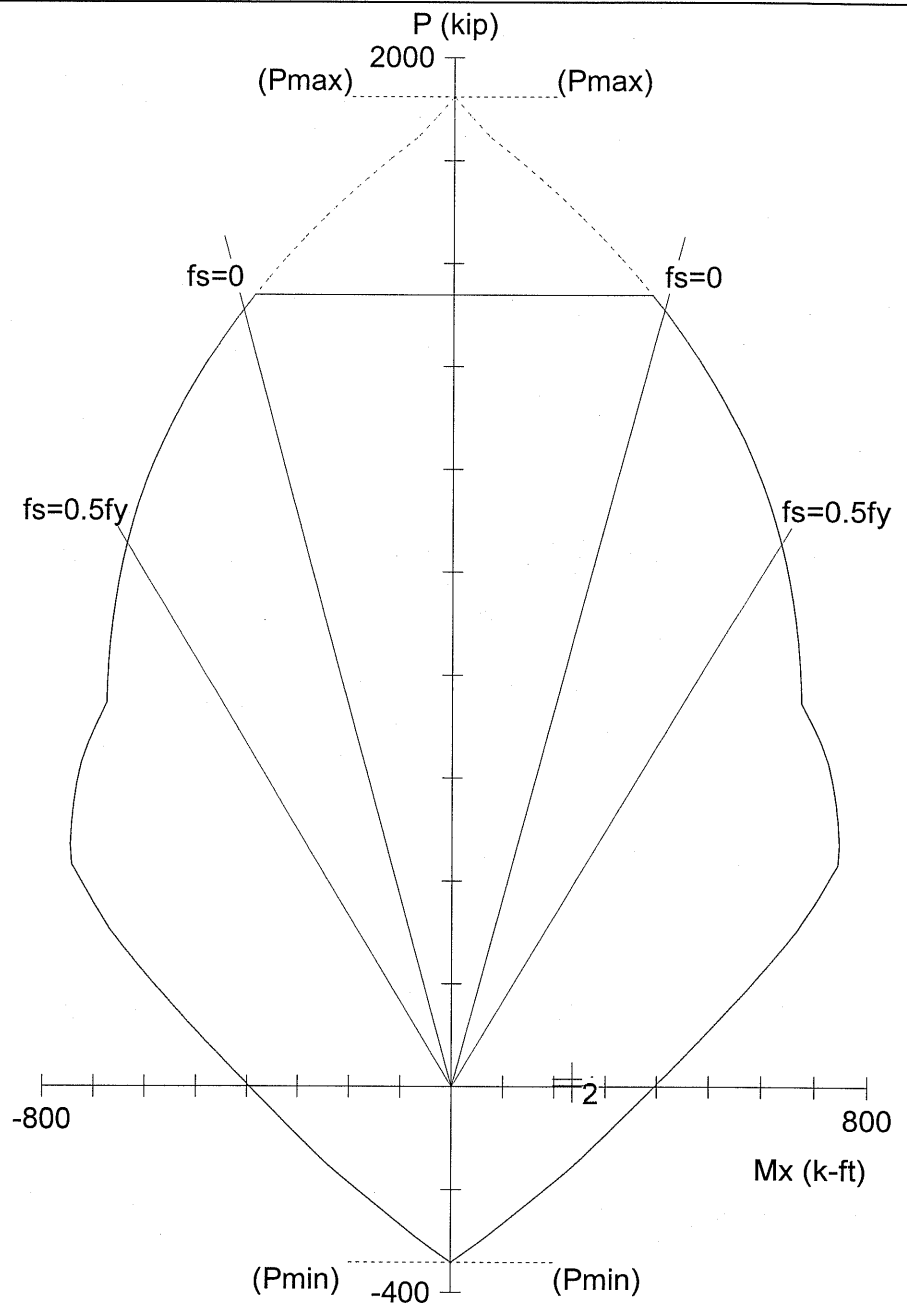
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 02/26/15

Time: 10:59:47



spColumn v4.60. Licensed to: Saiful/Bouquet, Inc. (SBI). License ID: 59950-1034111-2-27A36-2E93F

File: untitled.col

Project: Fire Station 143

Column: Fuel Canopy

$f'_c = 3$ ksi

$f_y = 60$ ksi

Engineer: NH

$A_g = 1017.88$ in²

8 #8 bars

$E_c = 3122$ ksi

$E_s = 29000$ ksi

$A_s = 6.32$ in²

$\rho = 0.62\%$

$f_c = 2.55$ ksi

$X_o = 0.00$ in

$I_x = 82448$ in⁴

$e_u = 0.003$ in/in

$Y_o = 0.00$ in

$I_y = 82448$ in⁴

Beta1 = 0.85

Min clear spacing = 9.81 in

Clear cover = 3.38 in

Confinement: Tied

$\phi(a) = 0.8$, $\phi(b) = 0.9$, $\phi(c) = 0.65$

General Information:

File Name: untitled.col
 Project: Fire Station 143
 Column: Fuel Canopy
 Code: ACI 318-08
 Engineer: NH
 Units: English
 Run Option: Investigation
 Run Axis: X-axis
 Slenderness: Not considered
 Column Type: Structural

Material Properties:

f'c = 3 ksi
 Ec = 3122.02 ksi
 Ultimate strain = 0.003 in/in
 Beta1 = 0.85
 fy = 60 ksi
 Es = 29000 ksi

Section:

Circular: Diameter = 36 in
 Gross section area, Ag = 1017.88 in²
 Ix = 82448 in⁴
 rx = 9 in
 Xo = 0 in
 Iy = 82448 in⁴
 ry = 9 in
 Yo = 0 in

Reinforcement:

Bar Set: ASTM A615

Size	Diam (in)	Area (in ²)	Size	Diam (in)	Area (in ²)	Size	Diam (in)	Area (in ²)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
 phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
 Pattern: All Sides Equal (Cover to transverse reinforcement)
 Total steel area: As = 6.32 in² at rho = 0.62% (Note: rho < 1.0%)
 Minimum clear spacing = 9.81 in

8 #8 Cover = 3 in

Factored Loads and Moments with Corresponding Capacities:

No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu NA	depth in	Dt in	depth in	eps_t	Phi
1	15.40	236.20	410.22	1.737	6.59	32.13	0.01162	0.900	
2	2.93	236.20	397.87	1.684	6.43	32.13	0.01198	0.900	

*** End of output ***

Steel Base Plate

File = W:\14608_~1\143\ENGINE~1\FUELDI~1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: Canopy Plate & Anchor Bolts

Code References

Calculations per AISC Design Guide # 1, IBC 2012, CBC 2013, ASCE 7-10

Load Combination Set: IBC 2012

General Information

Material Properties

AISC Design Method Load Resistance Factor Design
Steel Plate F_y = 50.0 ksi
Concrete Support f'_c = 3.0 ksi
Assumed Bearing Area: Full Bearing

Φ_c : LRFD Resistance Factor

0.60

Allowable Bearing F_p per J8

4.080 ksi

Column & Plate

Column Properties

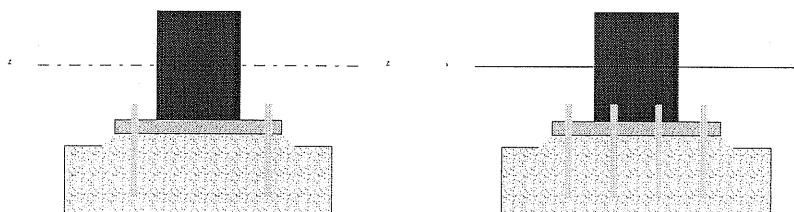
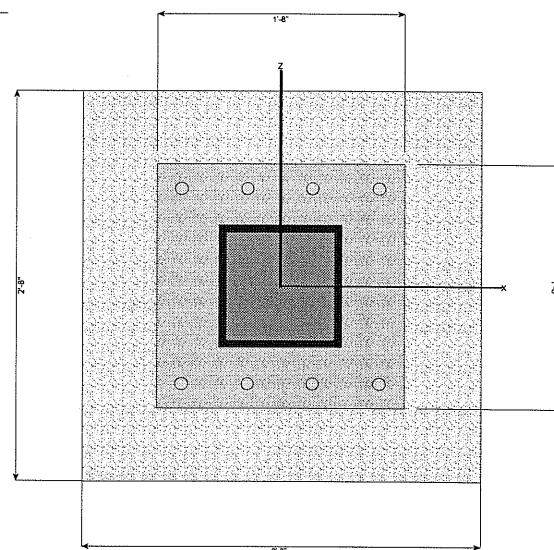
Steel Section: HSS10x10x5/8
Depth 10 in Area 21 in²
Width 10 in I_{xx} in⁴
Flange Thickness 0.581 in I_{yy} in⁴
Web Thickness in

Plate Dimensions

N: Length 20.0 in
B: Width 20.0 in
Thickness 1.625 in
Column assumed welded to base plate.

Support Dimensions

Width along "X" 32.0 in
Length along "Z" 32.0 in



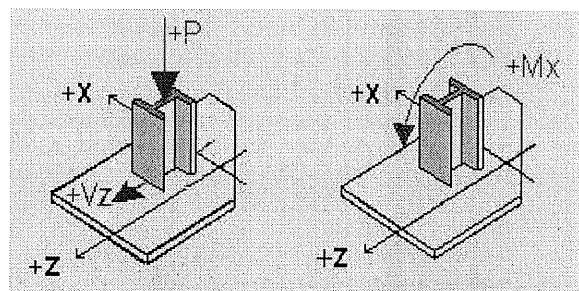
Applied Loads

	P-Y	V-Z	M-X
D: Dead Load	5.310 k	k	k-ft
L: Live	7.150 k	k	k-ft
Lr: Roof Live	k	k	k-ft
S: Snow	k	k	k-ft
W: Wind	12.570 k	k	k-ft
E: Earthquake	k	k	189.0 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.



ADDENDUM 5

Steel Base Plate

File = W:\14608_~1.143\ENGINE~1\FUELDI~1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.1.19, Ver:6.15.1.19

Lic. #: KW-06003631

Licensee: SAIFUL - BOUQUET CONSULTING ENGINEERS

Description: Canopy Plate & Anchor Bolts

GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method Load Resistance Factor Design
Governing Load Combination +0.5522D+1.250E+0.90H
Governing Load Case Type Axial + Moment, L/2 < Eccentricity, Tension on Bc
Design Plate Size **1'-8" x 1'-8" x 1 -5/8**
Pu : Axial 2.932 k
Mu : Moment 236.250 k-ft

Mu : Max. Moment 29.517 k-in
fb : Max. Bending Stress 44.712 ksi
Fb : Allowable : 45.000 ksi

Bending Stress Ratio 0.994

Bending Stress OK

fu : Max. Plate Bearing Stress 2.448 ksi
Fp : Allowable : 2.448 ksi

min(0.85*fc*sqrt(A2/A1), 1.7*fc)*Phi
Bearing Stress Ratio

1.000

Bearing Stress OK

Tension in each Bolt 45.411
Allowable Bolt Tension 54.000

Tension Stress Ratio 0.841

TENSION IN ANCHOR BOLTS

45.594 K IN EACH BOLT PER ENERCALC OUTPUT

$0.9 A_s F_y$ = TENSION CAPACITY OF REBAR

$$45.594 / (0.9)(60) = 0.844 \text{ IN}^2$$

#9 BARS HAVE $A_s = 1.0 \text{ IN}^2$

USE (12) - #9 THREADED REBAR

4.6 REVERSE APPARATUS DESIGN

Seismic Loads

A roof= 57 ft* 41 ft= 2337 sf => W roof= 2337 sf* 20.5 psf= 47.9 kip

W walls= (868 sf*2 *15 pdf + 988 sf*15 psf + 988sf* 8 psf)/2= (26 kip + 14 kip + 7.9kip)/2 = 48 kip/2= 24 kip

W total = W roof + W walls = 47.9 kip + 24 kip = 72 kip

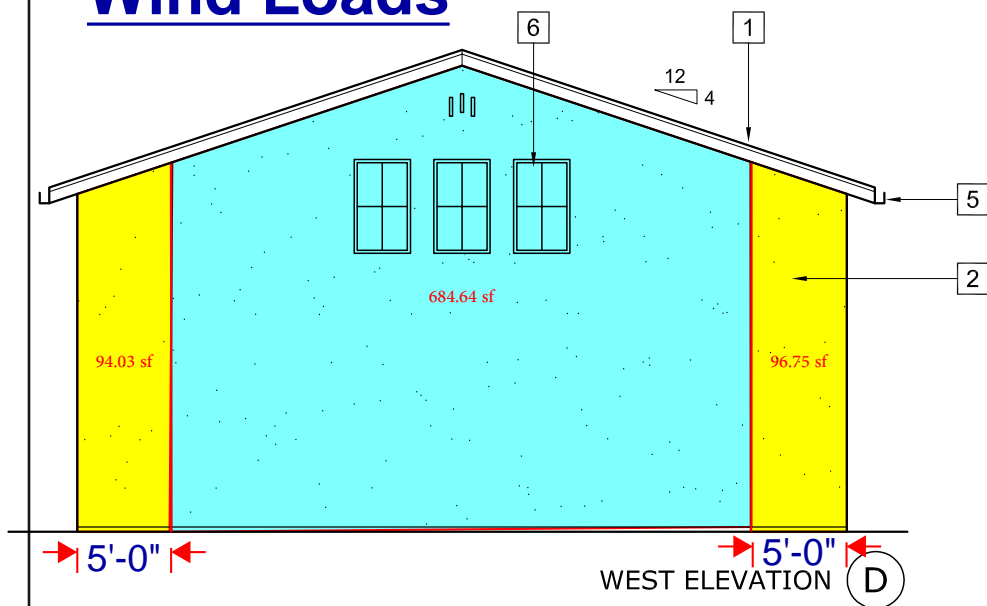
V= 0.369 * 72 kip= 26.5 kip Seismic Governed

Vh= 26.5 kip * 1.3 = 34.4 kip (LRFD, STRENGTH)= 34.4 kip*0.7= 24 kip(ASD)

Vh= 26.5 kip (LRFD, DRIFT), 26.5kip *0.7= 18.5 kip (ASD)

Fp dia= 0.48/0.369 * 18.5 kip= 24 kip

Wind Loads



$a=0.1 * 41 \text{ ft}= 4.1$

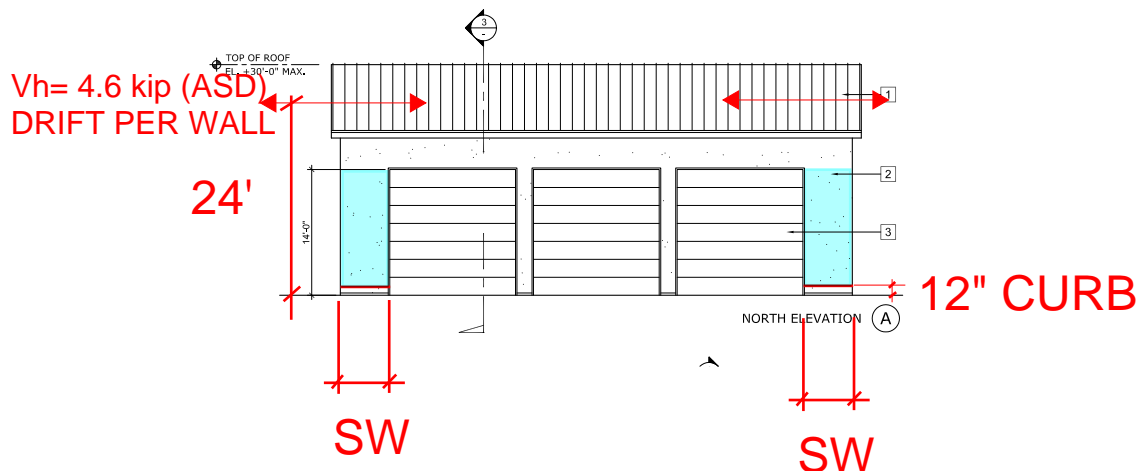
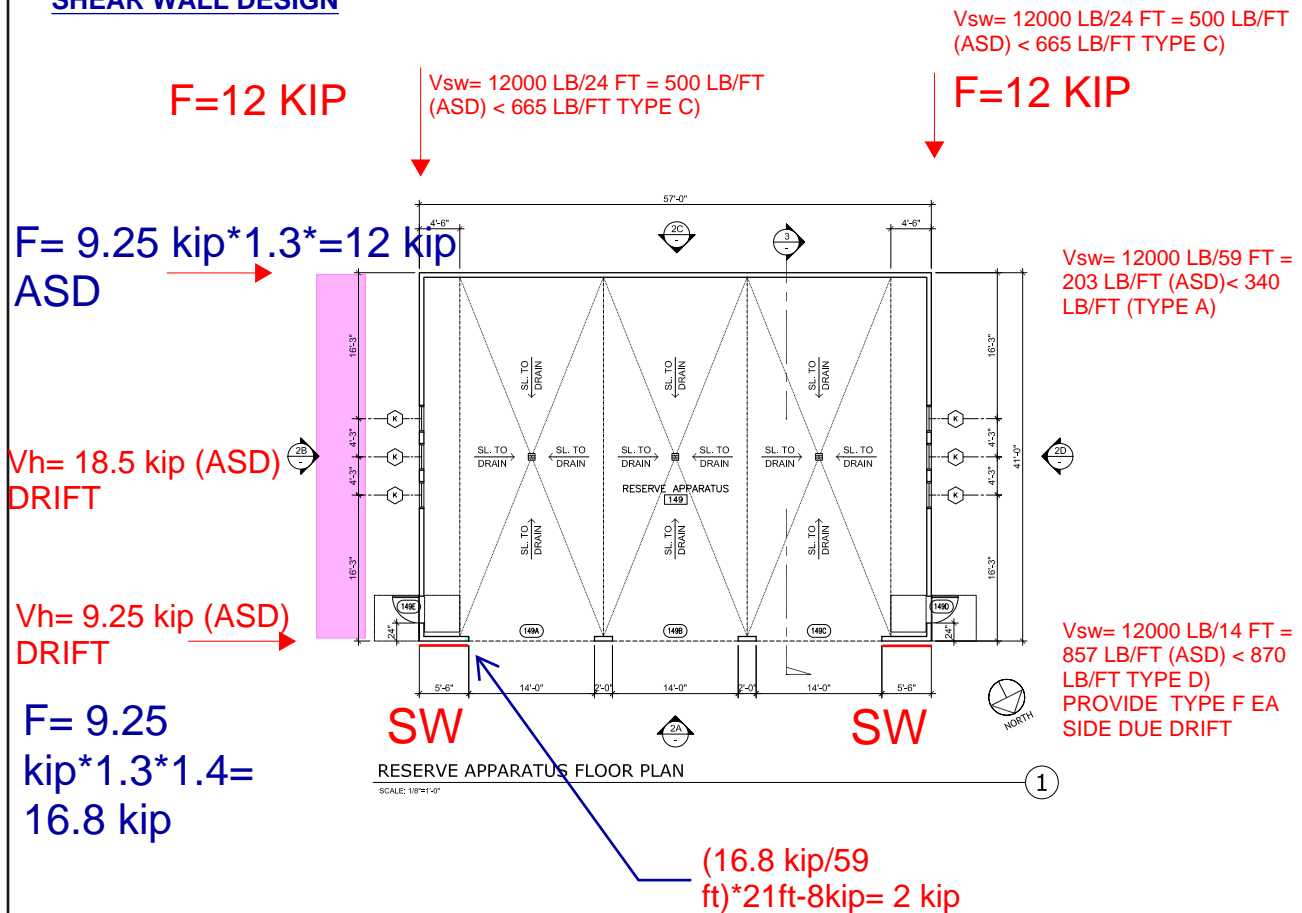
$0.4 * h= 0.4 * 26 \text{ ft}= 10.6 \text{ ft}$, $a= 4.1 \text{ ft}$

$0.04 * 41 = 1.6 \text{ ft}$
3 ft

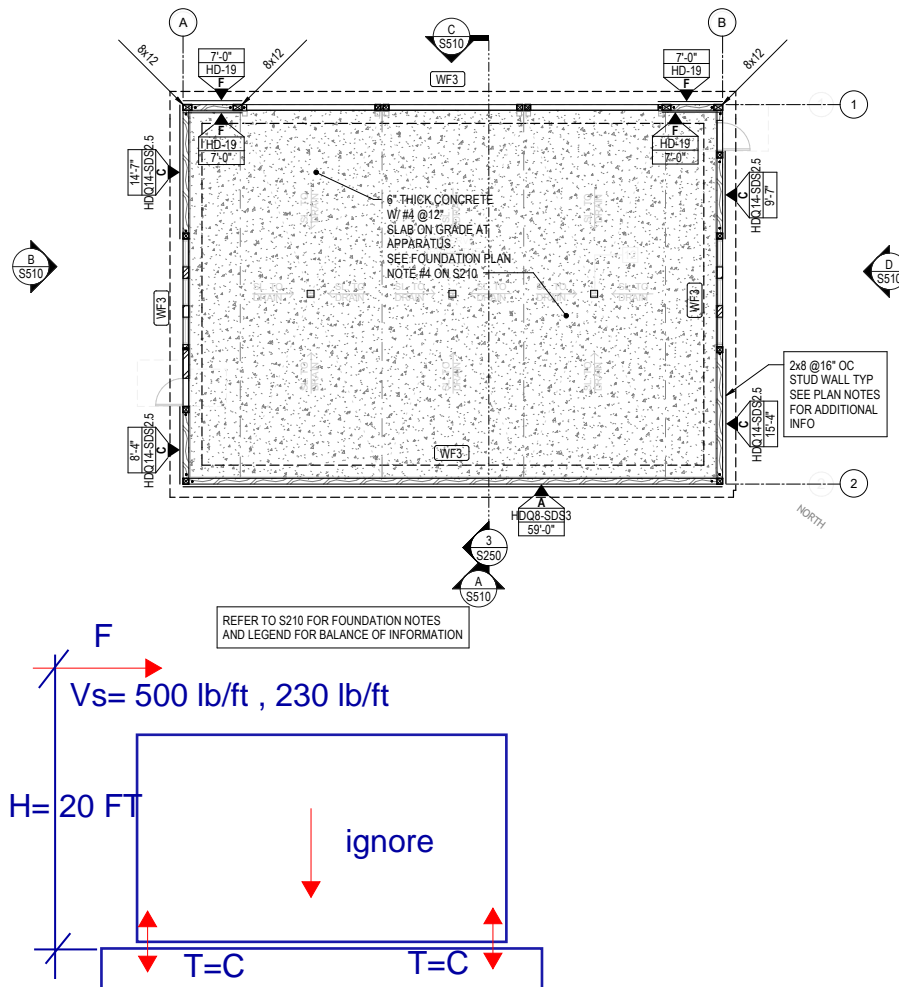
a=5 ft

V wind= (94 sf* 50 psf + 684 sf* 33 psf)/2 = 14 kip (**WIND IS NOT GOVERNED**)

SHEAR WALL DESIGN



Shear wall holdown Design



Line A & Line B

$T/C = F \cdot H/L = V_s \cdot L \cdot H/L = V_s \cdot H = 500 \text{ lb/ft} \cdot 20 \text{ ft} = 10,000 \text{ lb}$
 HDQ14-SDS2.5 with 6x 6 Ta= 13710 lb*0.75=10282 lb > T= 10,000 lb

Line 2

$T/C = F \cdot H/L = V_s \cdot L \cdot H/L = V_s \cdot H = 230 \text{ lb/ft} \cdot 24 \text{ ft} = 5,520 \text{ lb}$
 HDQ8-SDS3 with 6x 6 Ta= 9230 lb > T= 5520 lb

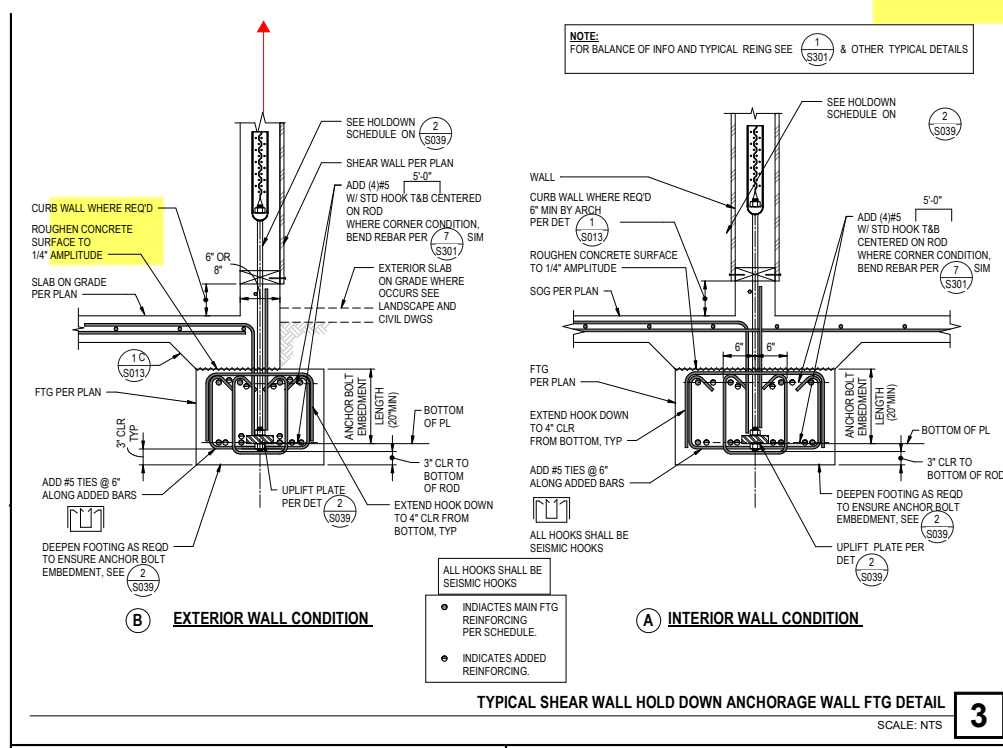
Line 1

$T/C = F \cdot H/L = V_s \cdot L \cdot H/L = V_s \cdot H = 857 \text{ lb/ft} \cdot 24 \text{ ft} = 20,568 \text{ lb}$
 (2) HD-19 with 6x 6 Ta= 2*16775 lb = 33,550 lb > T= 20,568 lb

Shear Wall Anchorage Design

Typical Anchorage For Shear at Reverse Building. Line 1

T = 20 kip/2 = 10 kip (ASD) max



Loading for Shear Wall anchorage Design:

1.25" Diameter $F_y=36$ ksi $T_u= 1.2 \cdot F_y \cdot A_g= 1.2 \cdot 36 \text{ ksi} \cdot 1.0 \text{ in}^2= 43 \text{ kip}$.

Wood Holdown HD19 $T_u= 19.0 \text{ kip} \cdot 1.4= 26.6 \text{ kip}$ (Design For this Failure first)

Omega Load= $(3.0-1/2) 10 \text{ kip} \cdot 1.4= 35 \text{ kip}$

SECTION 17.4.3 PULL OUT STRENGTH

$$N_{pn} = \psi_{cpn} N_p \quad (17.4.31)$$

$$\psi_{cp} = 1.0 \quad (\text{CRACK SERVICE}) \quad 17.4.36$$

$$N_p = 8 A_{brg} f'_c \quad (17.4.3.4)$$

$$N_p = 8 \times (4" \times 4") \times 3 \text{ ksi} = 384 \text{ kip}$$

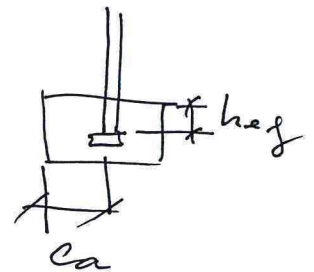
$$\rightarrow \phi N_{pn} = 0.75 \times 1.0 \times 384 = 288 \text{ kip} > T_u = 26.6 \text{ kip}$$

SECTION 17.4.4 CONCRETE SIDE FACE BLOW OUT

$$h_{ef} = (20" - 1.5") = 18.5"$$

$$C_a = \frac{24}{2} = 12"$$

$$2.5 \times C_a = 30" > h_{ef}$$



\hookrightarrow NO SIDE BLOW OUT OCCURS

STRENGTH OF ANCHOR REINF

REINF. EA. SIDE OF A.B. $0.5 h_{ef}$

$$(0.5 \times 18.5 \times 2) = 18.5"$$

#5 RE @ 6" o.c

$\square \leftarrow 2 \text{ leg}$

$$\left(\frac{18.5}{6} \times 2 \right) = 6$$

$$\phi N_u = \phi P_n = 0.75 \times 60 \text{ ksi} \times 6 \times 0.31 = 84 \text{ kip} > 26.5 \text{ kip}$$

ANCHOR . UPLIFT CHECK

$$T_u = 26 \text{ kip}$$

$$q_u = \frac{26}{(4 \times 4)} = 1.6 \text{ ksi}$$

$$M_u = \frac{q_u l^2}{2} = \frac{1.6 \times 2^2}{2} = 3.2 \text{ kip} \cdot \text{ft}$$

$$Z_{req} = \frac{3.2 \text{ kip} \cdot \text{ft}}{0.9 \times 36 \text{ ksi}} = 0.09 \text{ in}^3$$

$$Z = \frac{bd^2}{4} \rightarrow d = 0.31 \text{ in}$$

USE PL 3/8" x 4 x 4 x 3/8

$V = 836 \text{ lb/ft}$ (ASD) $k = \text{stiffness of the anchorage} = F / \delta$ (deflection / elongation)

Shear Walls in a Line

(bending) (shear) (wall anchorage slip)

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h}{b}\Delta_a \quad (\text{C4.3.2-2})$$

δ

(2) HD19

$$k = 2 * (19070 \text{ lb}/0.137") = 278,394 \text{ lb/in}$$

$$V = \frac{8h^3}{EAb_{sw1}} + \frac{h}{1000G_a} + \frac{h^2}{kb_{sw1}}$$

End Post A = $7.5 * 7.5 = 56.25 \text{ in}^2$



$$V = \frac{8(13')^3}{(1,400,000)(56.25 \text{ in}^2)(5.5 \text{ ft})} + \frac{(13')}{1000(14,000)} + \frac{(13')^2}{(64,924)(5.5 \text{ ft})}$$

δ

0.000033 0.00000065 0.00011

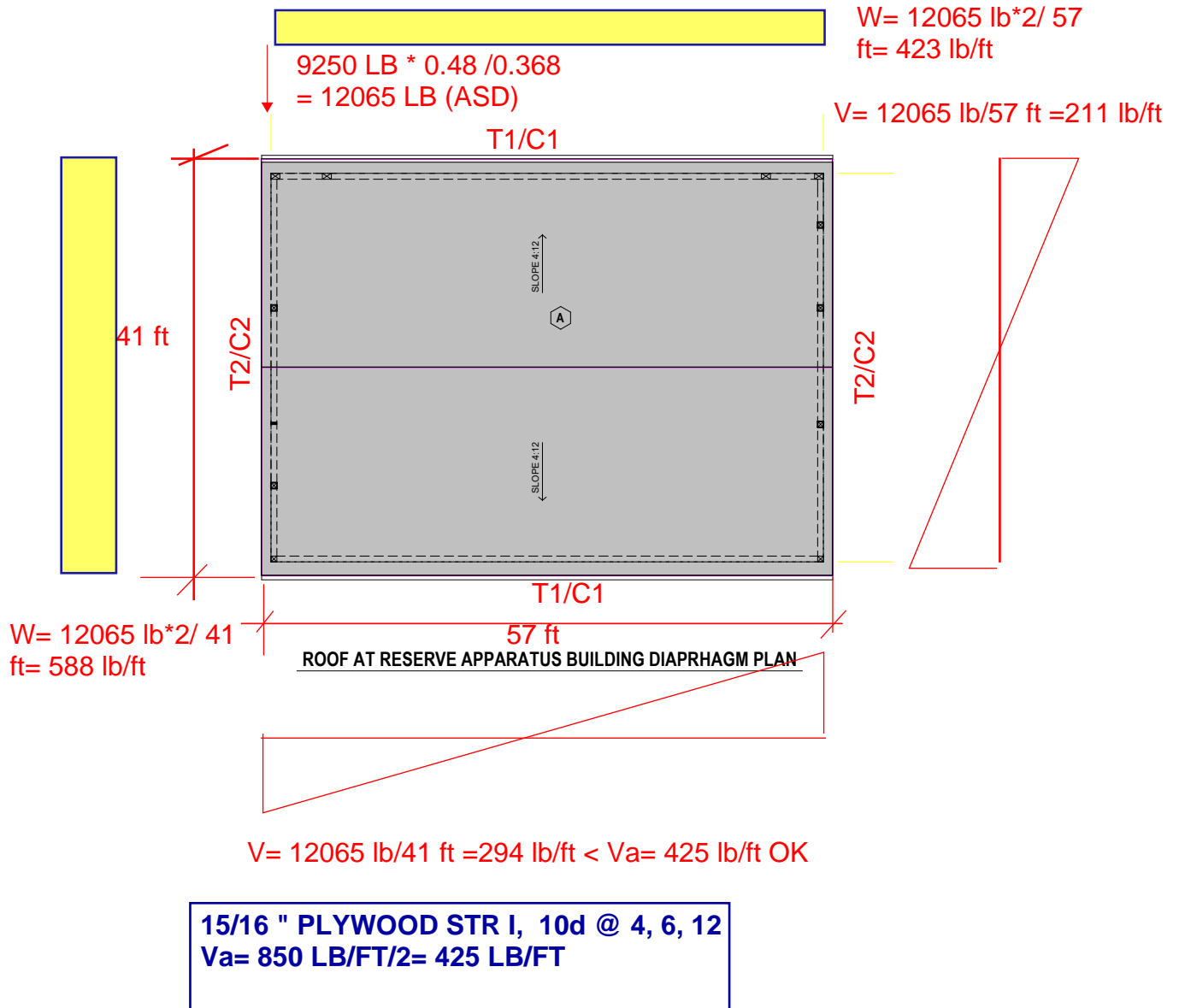
$$\frac{836 \text{ lb/ft}}{700 \text{ lb/ft}} = \frac{0.000144}{0.000144} \Rightarrow$$

$$\delta = 0.000144 * 836 \text{ lb/ft} = 0.120"$$

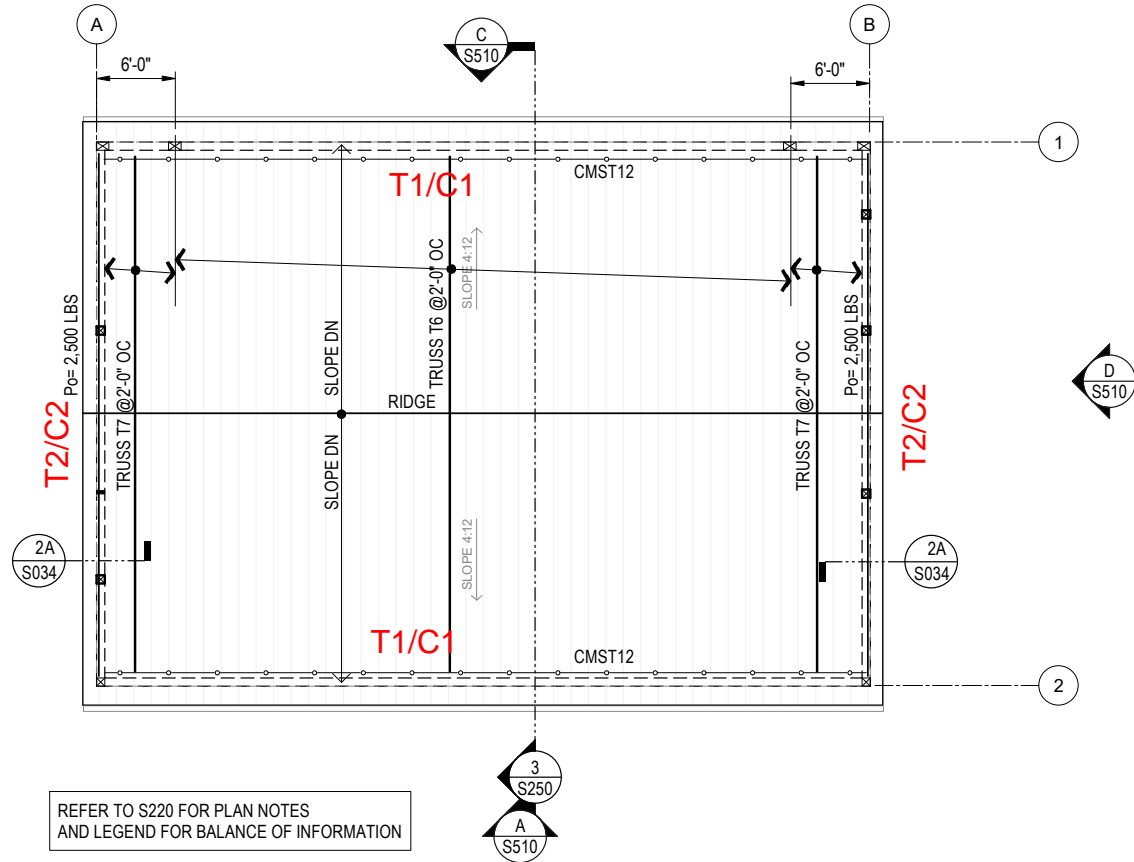
$$\text{Drift} = 0.120 * 4 / 1.5 = 0.32" \leq 0.375 = 3/8"$$

ADDENDUM 5

Diaphragm Design



Chord Design



CHORD DESIGN

$$T1/C1 = (423 \text{ LB/FT} \cdot 57^{2/8}) / 41 \text{ FT} = 4190 \text{ LB}$$

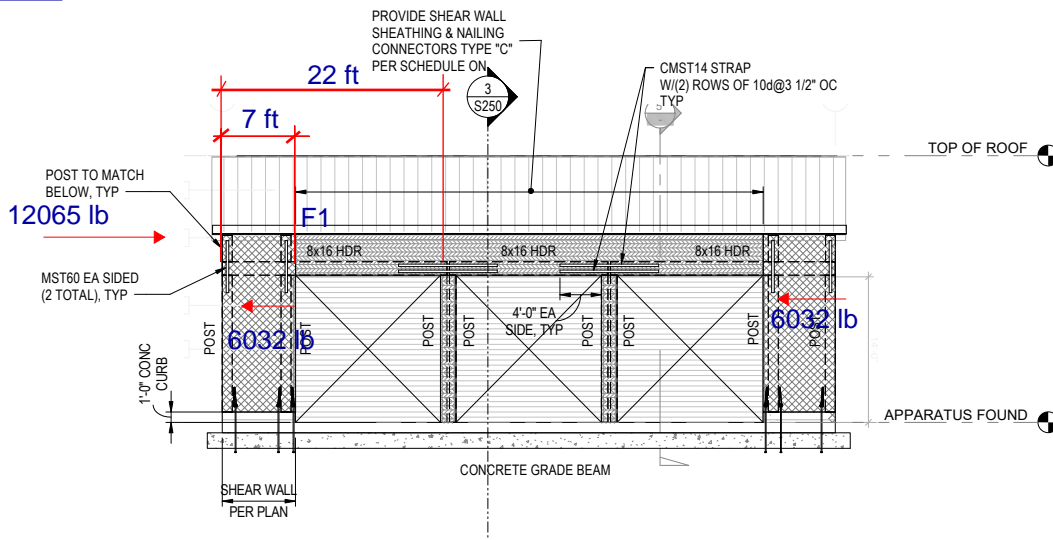
$$\text{CMST12 } T_a = 9215 \text{ LB} > T = 4190 \text{ LB OK}$$

$$T2/C2 = (588 \text{ LB/FT} \cdot 41^{2/8}) / 57 \text{ FT} = 2168 \text{ LB}$$

PROVIDE 2500 LB FOR TRUSS DESIGN

Drag Design

Line 1



$$F1 = 6032 \text{ lb} - 211 \text{ lb/ft} \times 7 \text{ ft} = 4555 \text{ lb}$$

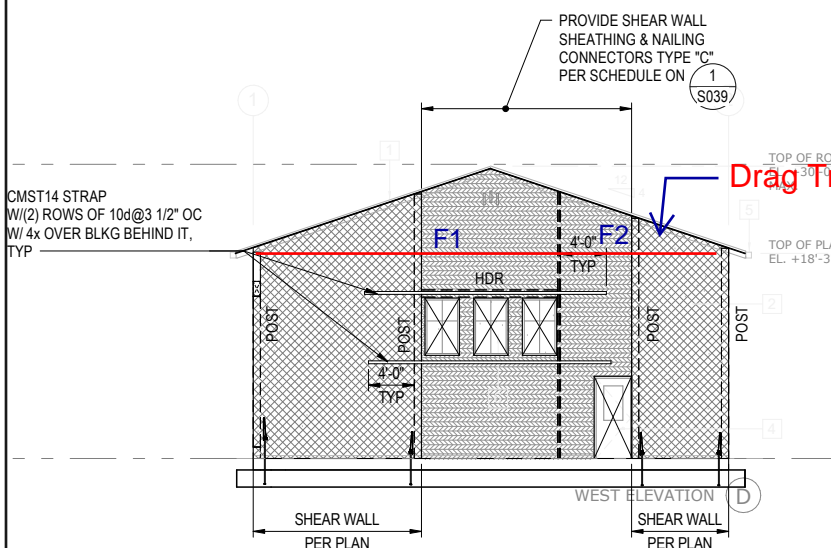
$$\text{CMST12 Ta} = 9215 \text{ lb} > 4555 \text{ lb OK}$$

$$\text{Roof Dia can take only} = 425 \text{ lb/ft} \times 7 \text{ ft} = 2975 \text{ lb}$$

$$\text{The difference } T = (4555 \text{ lb} - 2975 \text{ lb}) = 1580 \text{ lb}$$

use CMST 14 ea side Ta = 6475 lb > 1580 lb

Line A & B



$$L = 15'-4"$$

$$V = (12065 / (15.3 \text{ ft} + 9.6 \text{ ft})) \text{ lb/ft} \times$$

$$15.3 \text{ ft} = 484 \text{ lb/ft} \times 15.3 \text{ ft} = 7413 \text{ lb}$$

$$L = 9'-7"$$

$$V = 500 \text{ lb/ft} \times 15.3 \text{ ft} = 4792 \text{ lb}$$

Drag Truss

$$V = 12065 \text{ lb} / 41 \text{ ft} = 294 \text{ lb/ft}$$

$$F1 = 7413 \text{ lb} - 294 \text{ lb/ft} \times 15.3 \text{ ft} = 2915 \text{ lb}$$

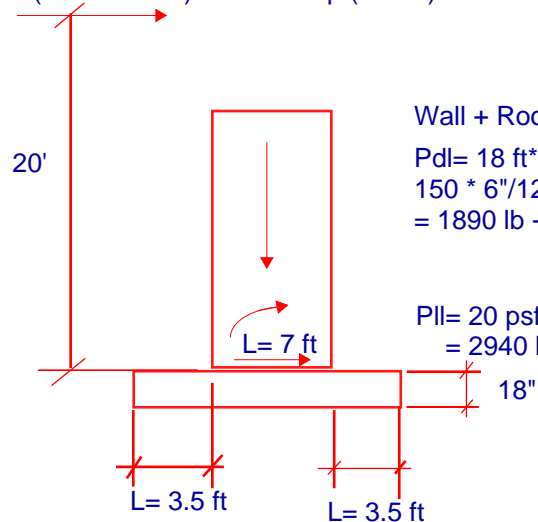
$$F2 = 7413 \text{ lb} - 294 \text{ lb/ft} \times 31.4 \text{ ft} = -1816 \text{ lb}$$

Drag Truss shall be designed for 3000 lb

Shear Wall Footing Design

SW FOOTING LINE 1

$$F = (857 \text{ lb/ft} \times 7 \text{ ft}) \times 1.4 = 8.4 \text{ kip (LRFD)}$$



Wall + Roof + Curb + SOG + Wall Header

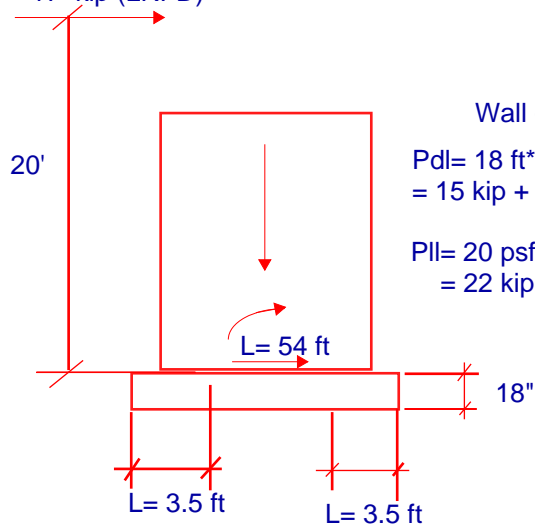
$$\begin{aligned} Pdl &= 18 \text{ ft} \times 7 \text{ ft} \times 15 \text{ psf} + 23.5 \text{ psf} \times 42 \text{ ft} \times 7 \text{ ft} + 150 \text{ pcf} \times 2' \times 7' \times 8/12 + \\ &150 \times 6''/12 \times 15 \text{ ft} \times 8 \text{ ft} + 7000 = \\ &= 1890 \text{ lb} + 3454 \text{ lb} + 1400 \text{ lb} + 9000 \text{ lb} + 7000 \text{ lb} = 22 \text{ kip} \end{aligned}$$

$$\begin{aligned} Pll &= 20 \text{ psf} \times 42 \text{ ft} \times 7 \text{ ft} \\ &= 2940 \text{ lb} \end{aligned}$$

3'-6" wide x 18" depth w/ 5 # 6 top & Bottom

SW FOOTING LINE 2

$$F = 17 \text{ kip (LRFD)}$$

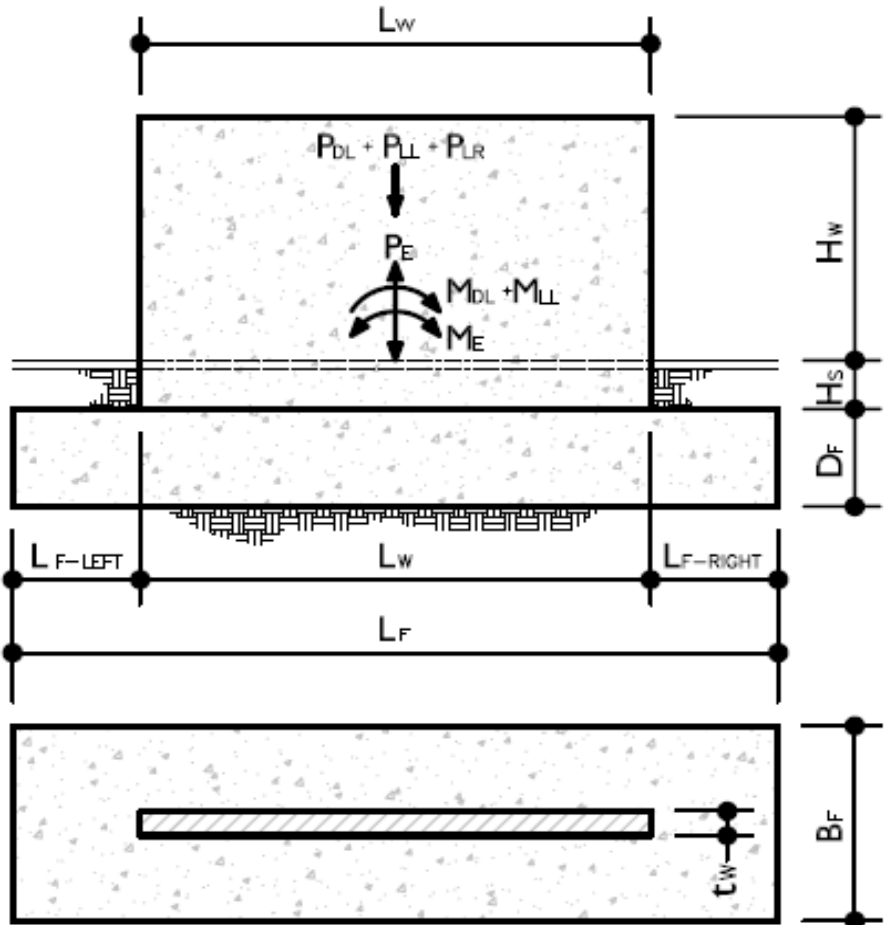


Wall + Roof + Curb + SOG

$$\begin{aligned} Pdl &= 18 \text{ ft} \times 54 \text{ ft} \times 15 \text{ psf} + 23.5 \text{ psf} \times 42 \text{ ft} \times 54 \text{ ft} + 150 \text{ pcf} \times 6/12 \times 54 \text{ ft} \times 4 \text{ ft} \\ &= 15 \text{ kip} + 26 \text{ lb} + 16 \text{ kip} = 57 \text{ kip} \end{aligned}$$

$$\begin{aligned} Pll &= 20 \text{ psf} \times 42 \text{ ft} \times 54 \text{ ft} \\ &= 22 \text{ kip} \end{aligned}$$

3'-6" wide x 18" depth w/ 4 # 7 top & Bottom



PLAN VIEW

STEP 1: GENERAL INPUT DATA

Redundancy Factor, ρ :		1.30	Concrete Strength (footing), f'_c :		3.00
S_{ds} :		1.601	Footing Width, B_f :		3.50
Allowable Soil Pressure q_a :		3.00	Footing Depth, D_f :		1.50
1/3 Increase Allowed?		YES	Footing Length Left, L_{f-LEFT} :		3.50
Net Allowable Pressure?		YES	Footing Length Right, $L_{f-RIGHT}$:		3.50
Wall Properties:			Total Footing Length, L_f :		14.0
Shear Wall Height, H_w :		18.00	Area of Footing, A_{FTG} :		49.0
Shear Wall Length, L_w :		7.00	Section Modulus of Footing, S_{FTG} :		114.3
Shear Wall Thickness, t_w :		8.00	Unit Weight of Footing, γ_f :		150
Unit Weight of Wall, γ_w :		0	Footing Weight, W_f :		11.0
Wall Weight, W_w :		0.0	Soil Above Footing, H_s :		1.00
Ω_o =		2.50	Unit Weight of Soil, γ_s :		100
Omega?		NO	Weight of Soil, W_s :		4.9
			10% Reduction?		NO
			25% Reduction?		NO

STEP 2: APPLIED LOADS (At the Center of Wall)

	Seismic (E)	Dead (DL)	Live (LL)	Roof Live (LR)	
P =	0.0	22.0	0.0	2.9	k
M =	160.0	0.0	0.0	0.0	ft-k
V =	8.4	0.0	0.0	0.0	k
Mtot =	172.6	= M + V x Footing Depth			

STEP 3: FOOTING SIZE (Service Loads)

Comb. 1: D + L + Lr

[CBC Eq. 16-16]

$$\begin{aligned} P_1 &= (W_w + W_f + W_s + P_D) + P_L + P_{LR} = 40.8 \text{ k} \\ M_1 &= (W_w + P_D + P_L + P_{LR})(L_{f-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} + M_{RL} = 0.0 \text{ ft-k} \\ M_1 \text{ NET} &= (W_w + P_D + P_L + P_{LR})(L_{f-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) = \text{N/A} \text{ ft-k} \\ X_1 &= \text{N/A} \text{ ft} \end{aligned}$$

P/A + M/S
0.83 ksf
P/A - M/S
0.83 ksf

Allowable**

<

3.25 ksf

< O.K. >

Comb. 2: D + L + $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-20]

$$\begin{aligned} P_2 &= (W_w + W_f + W_s + P_D) + P_L + \rho P_E/1.4 = 37.9 \text{ k} \\ M_2 &= [(W_w + P_D) + P_L + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} + \rho M_E/1.4 = 160.3 \text{ ft-k} \\ M_2 \text{ NET} &= [(W_w + P_D) + P_L + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) + M_{DL} + M_{LL} + \rho M_E/1.4 = 425.7 \text{ ft-k} \\ a_2 &= 11.23 \text{ ft} \\ X_2 &= 8.32 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)
2.60 ksf
P/A-M/S
0.00 ksf

<

4.33 ksf

< O.K. >

Comb. 3: D + L - $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-20]

$$\begin{aligned} P_3 &= (W_w + W_f + W_s + P_D) - P_L + \rho P_E/1.4 = 37.9 \text{ k} \\ M_2 &= [(W_w + P_D) - P_L - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} - \rho M_E/1.4 = (160.3) \text{ ft-k} \\ M_2 \text{ NET} &= [(W_w + P_D) - P_L - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) + M_{DL} + M_{LL} - \rho M_E/1.4 = 105.2 \text{ ft-k} \\ a_3 &= 2.77 \text{ ft} \\ X_3 &= 8.32 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)
2.60 ksf
P/A-M/S
0.00 ksf

<

4.33 ksf

< O.K. >

Comb. 4: 0.9D + $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-21]

$$\begin{aligned} P_4 &= (0.9)(W_w + W_f + W_s + P_D) + \rho P_E/1.4 = 34.1 \text{ k} \\ M_4 &= [(0.9)(W_w + P_D) + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + (0.9)M_{DL} + \rho M_E/1.4 = 160.3 \text{ ft-k} \\ M_4 \text{ NET} &= [(0.9)(W_w + P_D) + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (0.9)(W_f + W_s)(L_f/2) + (0.9)M_{DL} + \rho M_E/1.4 = 399.2 \text{ ft-k} \\ a_4 &= 11.70 \text{ ft} \\ X_4 &= 6.91 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)
2.82 ksf
P/A-M/S
0.00 ksf

<

4.33 ksf

< O.K. >

Comb. 5: 0.9D - $\rho Q_E/1.4$ ($E_v = 0$)*

[CBC Eq. 16-21]

$$\begin{aligned} P_5 &= (0.9)(W_w + W_f + W_s + P_D) - \rho P_E/1.4 = 34.1 \text{ k} \\ M_5 &= [(0.9)(W_w + P_D) - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + (0.9)M_{DL} - \rho M_E/1.4 = (160.3) \text{ ft-k} \\ M_5 \text{ NET} &= [(0.9)(W_w + P_D) - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (0.9)(W_f + W_s)(L_f/2) + (0.9)M_{DL} - \rho M_E/1.4 = 78.7 \text{ ft-k} \\ a_5 &= 2.30 \text{ ft} \\ X_5 &= 6.91 \text{ ft} \end{aligned}$$

O.K.

2P/(XBf)
2.82 ksf
P/A-M/S
0.00 ksf

<

4.33 ksf

< O.K. >



SHEAR WALL FOOTING DESIGN

Project : FS- Reverse Apparatus - Line

Project # : 25534

Engineer :

Sheet :

Date : 6/25/2025

Checked :

Shear Wall : WALL -1

Shear Wall Location : Grid 1

Wall Type: Concrete

STEP 4: DESIGN FOR FLEXURE

A. Factored Soil Pressure

f1 = 0.5

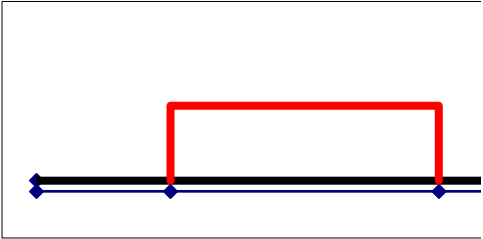
Comb. 6: 1.4D

[CBC Eq. 16-1]

P6 = 53.1 k
M6 = 0.0 ft-k
M6net = N/A ft-k
a6 = N/A ft
X6 = N/A ft

P/A + M/S = 1.08 ksf
P/A - M/S = 1.08 ksf

p1 = 1.084 ksf
p2 = 1.084 ksf
p3 = 1.084 ksf
p4 = 1.084 ksf



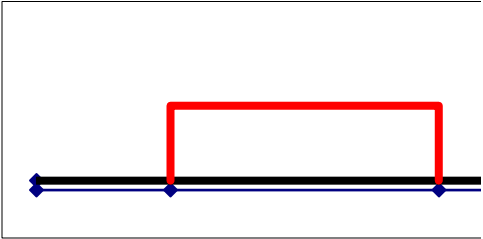
Comb. 7: 1.2D + 1.6L + 0.5Lr

[CBC Eq. 16-2]

P7 = 47.0 k
M7 = 0.0 ft-k
M7net = N/A ft-k
a7 = N/A ft
X7 = N/A ft

P/A + M/S = 0.96 ksf
P/A - M/S = 0.96 ksf

p1 = 0.958 ksf
p2 = 0.958 ksf
p3 = 0.958 ksf
p4 = 0.958 ksf



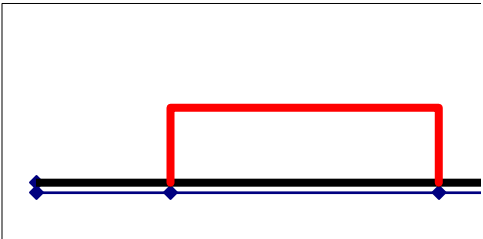
Comb. 8: 1.2D + f1L + 1.6Lr

[CBC Eq. 16-3]

P8 = 50.2 k
M8 = 0.0 ft-k
M8net = N/A ft-k
a8 = N/A ft
X8 = N/A ft

P/A + M/S = 1.02 ksf
P/A - M/S = 1.02 ksf

p1 = 1.023 ksf
p2 = 1.023 ksf
p3 = 1.023 ksf
p4 = 1.023 ksf



Comb.9: (1.2 + 0.2SDS)D + f1L + pQE

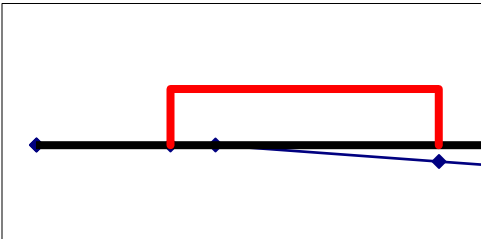
[CBC Eq. 16-5]

P9 = 57.7 k
M9 = 224.4 ft-k
M9net = 628.0 ft-k
a9 = 10.89 ft
X9 = 9.32 ft

2P/(XBf) = 3.53 ksf
P/A-M/S or 0 = 0.00 ksf

p1 = 0.000 ksf
p2 = 0.000 ksf
p3 = 2.207 ksf
p4 = 3.533 ksf

O.K.



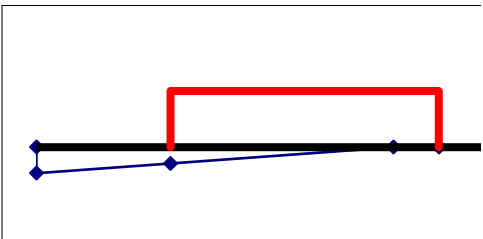
Comb. 10: (1.2 + 0.2SDS)D + f1L - pQE

[CBC Eq. 16-5]

P10 = 57.7 k
M10 = (224.4) ft-k
M10net = 179.2 ft-k
a10 = 3.11 ft
X10 = 9.32 ft

2P/(XBf) = 3.53 ksf
P/A-M/S or 0 = 0.00 ksf

p1 = 3.533 ksf
p2 = 2.207 ksf
p3 = 0.000 ksf
p4 = 0.000 ksf



Comb. 11: 0.9D + pQE

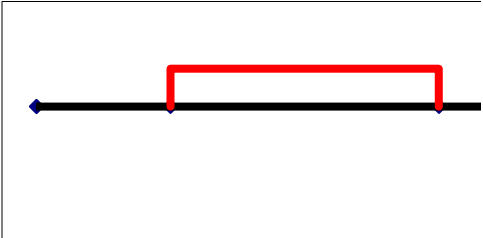
[CBC Eq. 16-6]

P11 = 34.1 k
M11 = 224.4 ft-k
M11net = 463.3 ft-k
a11 = 13.57 ft
X11 = 1.28 ft

2P/(XBf) = 15.25 ksf
P/A-M/S or 0 = 0.00 ksf

p1 = 0.000 ksf
p2 = 0.000 ksf
p3 = 0.000 ksf
p4 = 15.254 ksf

O.K.



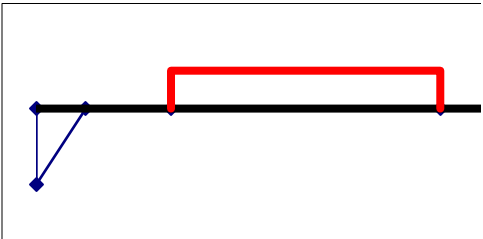
Comb. 12: 0.9D - pQE

[CBC Eq. 16-6]

P11 = 34.1 k
M11 = (224.4) ft-k
M11net = 14.5 ft-k
a11 = 0.43 ft
X11 = 1.28 ft

2P/(XBf) = 15.25 ksf
P/A-M/S or 0 = 0.00 ksf

p1 = 15.254 ksf
p2 = 0.000 ksf
p3 = 0.000 ksf
p4 = 0.000 ksf





SHEAR WALL FOOTING DESIGN

Project : FS- Reverse Apparatus - Line

Project # : 25534

Shear Wall : WALL -1

Engineer :

Shear Wall Location : Grid 1

Sheet :

Date : 6/25/2025

Checked :

Wall Type : Concrete

B. Longitudinal Reinforcing [Bottom]

Rebar clearance from the bottom : 3.00 in

$d_t = D_F - \text{clear} - 1" = 14 \text{ in}$

Assuming tension-controlled section

$\phi : 0.90$

Steel Strength (rebar), f_y : 60.00

$\beta_1 : 0.85$

$\rho_{MIN} : 0.0033333$

Left	Mu @ p2 (ft-k)	Soil Pressure
Comb. 6:	13.5	Case 1
Comb. 7:	12.2	Case 1
Comb. 8:	13.6	Case 1
Comb. 9:	0.0	Case 4B or 5B
Comb. 10:	55.7	Case 3A or 4A
Comb. 11:	0.0	Case 4B or 5B
Comb. 12:	98.6	Case 5A

$\rho = 0.00275$ ACI 10.5.3
 $A_{S \text{ REQ'D}} = 1.96 \text{ in}^2$
Rebars provided 4- #7
 $A_{S \text{ prov.}} = 2.41 \text{ in}^2 < \text{O.K.} >$
 $c = 1.59 \text{ in}$
 $c / d_t = 0.113$
 $\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$
Tension Control Assumption is Correct.
Spacing = 11.33 in

Right	Mu @ p3 (ft-k)	Soil Pressure
Comb. 6:	13.5	Case 1
Comb. 7:	12.2	Case 1
Comb. 8:	13.6	Case 1
Comb. 9:	55.7	Case 3B or 4B
Comb. 10:	0.0	Case 4A or 5A
Comb. 11:	98.6	Case 5B
Comb. 12:	0.0	Case 4A or 5A

$\rho = 0.00275$ ACI 10.5.3
 $A_{S \text{ REQ'D}} = 1.96 \text{ in}^2$
Rebars provided 4- #7
 $A_{S \text{ prov.}} = 2.41 \text{ in}^2 < \text{O.K.} >$
 $c = 1.59 \text{ in}$
 $c / d_t = 0.113$
 $\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$
Tension Control Assumption is Correct.
Spacing = 11.33 in

C. Longitudinal Reinforcing [Top] [Moments are due to the weight of footing and soil]

Left

Mu @ p2 = 9.75 ft-k

$\rho = 0.00026$ Temp. Steel

$A_{S \text{ REQ'D}} = 0.68 \text{ in}^2$

Rebars provided 4- #6

$A_{S \text{ prov.}} = 1.77 \text{ in}^2 < \text{O.K.} >$

$c = 1.16 \text{ in}$

$c / d_t = 0.083$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Spacing = 11.33 in

Right

Mu @ p3 = 9.75 ft-k

$\rho = 0.00026$ Temp. Steel

$A_{S \text{ REQ'D}} = 0.68 \text{ in}^2$

Rebars provided 4- #6

$A_{S \text{ prov.}} = 1.77 \text{ in}^2 < \text{O.K.} >$

$c = 1.16 \text{ in}$

$c / d_t = 0.083$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Spacing = 11.33 in

D. Transverse Reinforcing [Bottom]

$d_t = D_F - \text{clear} - 2" = 13 \text{ in}$

Left

Mu @ p2 = 1.92 ft-k / ft

$\rho = 0.00021$ Temp. Steel

$A_{S \text{ REQ'D}} = 0.19 \text{ in}^2$

Rebars provided #5 @ 18 o.c. < O.K. >

$A_{S \text{ prov.}} = 0.20 \text{ in}^2 < \text{O.K.} >$

$c = 0.13 \text{ in}$

$c / d_t = 0.010$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Right

Mu @ p3 = 1.92 ft-k

$\rho = 0.00021$ Temp. Steel

$A_{S \text{ REQ'D}} = 0.19 \text{ in}^2$

Rebars provided #5 @ 18 o.c. < O.K. >

$A_{S \text{ prov.}} = 0.20 \text{ in}^2 < \text{O.K.} >$

$c = 0.13 \text{ in}$

$c / d_t = 0.010$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

E. Transverse Reinforcing [Top]

$b = 12 \text{ in}$

Mu = 0.46 ft-k

$\rho = 0.00005$ Temp. Steel

$A_{S \text{ REQ'D}} = 0.19 \text{ in}^2$

Rebars provided #5 @ 18 o.c. < O.K. >

$b = 12 \text{ in}$

$A_{S \text{ prov.}} = 0.20 \text{ in}^2 < \text{O.K.} >$

$c = 0.13 \text{ in}$

$c / d_t = 0.010$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.



SHEAR WALL FOOTING DESIGN

Project : FS- Reverse Apparatus - Line
Project # : 25534 Engineer : _____
Shear Wall : WALL -1 Shear Wall Location : Grid 1

Sheet : _____
Date : 6/25/2025
Checked : _____
Wall Type: Concrete

STEP 5: CHECK FOR SHEAR

A. Longitudinal [The critital location for flexural shear is a distance "d" from the end of wall]

Steel Strength (rebar), *f_y* : 60.00

Rebar clearance from the bottom : 3.00 in ϕ : 0.75
d_t = *D_F* - clear - 1" = 14 in

Left	<i>V_{u12}</i> (kips)	<i>M_{u12}</i> (ft-k)	<i>V_u</i> / <i>M_u</i>	ϕV_c (kips)	
Comb. 6:	5.1	6.0	1.000	50.4	< O.K. >
Comb. 7:	4.6	5.4	1.000	50.4	< O.K. >
Comb. 8:	5.2	6.0	1.000	50.4	< O.K. >
Comb. 9:	(4.0)	0.0	N/A	48.3	< O.K. >
Comb. 10:	21.2	26.1	0.946	50.2	< O.K. >
Comb. 11:	(2.4)	0.0	N/A	48.3	< O.K. >
Comb. 12:	31.7	62.3	0.594	48.6	< O.K. >

Right	<i>V_{u34}</i> (kips)	<i>M_{u34}</i> (ft-k)	<i>V_u</i> / <i>M_u</i>	ϕV_c (kips)	
Comb. 6:	5.1	6.0	1.000	50.4	< O.K. >
Comb. 7:	4.6	5.4	1.000	50.4	< O.K. >
Comb. 8:	5.2	6.0	1.000	50.4	< O.K. >
Comb. 9:	21.2	26.1	0.946	50.2	< O.K. >
Comb. 10:	(4.0)	0.0	N/A	48.3	< O.K. >
Comb. 11:	31.7	62.3	0.594	48.6	< O.K. >
Comb. 12:	(2.4)	0.0	N/A	48.3	< O.K. >

*Design shear, *V_u*, is N/A if *d* > *L_F*-Left or *L_F*-Right

Shear Capacity

Larger of
$$\left\{ \begin{aligned} \phi V_c &= \phi 2 \sqrt{f'_c} b d = 48.3 \text{ k} & b &= B_F & \text{(ACI Eq. 11-3)} \\ \phi V_c &= \phi \left(1.9 \sqrt{f'_c} + 2500 \rho_w \frac{V_u d}{M_u} \right) b_w d < 3.5 \sqrt{f'_c} b_w d & & & \text{(ACI Eq. 11-5)} \end{aligned} \right.$$

V_u/*M_u* shall not be greather than 1.0

B. Transverse [The critital location for flexural shear is a distance "d" from the face of wall]

d_t = *D_F* - clear - 2" = 13 in *b* = 12 in

Shear Capacity
$$\phi V_c = \phi 2 \sqrt{f'_c} b d = 13.8 \text{ k}$$

Left *V_u* @ *p*2 = 0.57 k < O.K. > Right *M_u* @ *p*3 = 0.57 k < O.K. >

STEP 6: SUMMARY

FOOTING SIZE			LONGITUDINAL REINFORCEMENT			
Footing Width =	3.5	ft	Left		Right	
Footing Depth =	1.5	ft	Bottom:	4- #7	Bottom:	4- #7
Total Footing Length =	14.0	ft	Top:	4- #6	Top:	4- #6
Footing Length (Left) =	3.5	ft	TRANSVERSE REINFORCEMENT			
Footing Length (Right) =	3.5	ft	Left		Right	
Shear Wall Length, <i>L_w</i> =	7.0	ft	Bottom:	#5 @ 18 o.c.	Bottom:	#5 @ 18 o.c.
			Top:	#5 @ 18 o.c.	Top:	#5 @ 18 o.c.

SHEAR WALL FOOTING DESIGN

Project : FS- Reverse Apparatus - Line

Project # : 25534

Shear Wall : WALL -2

Engineer :

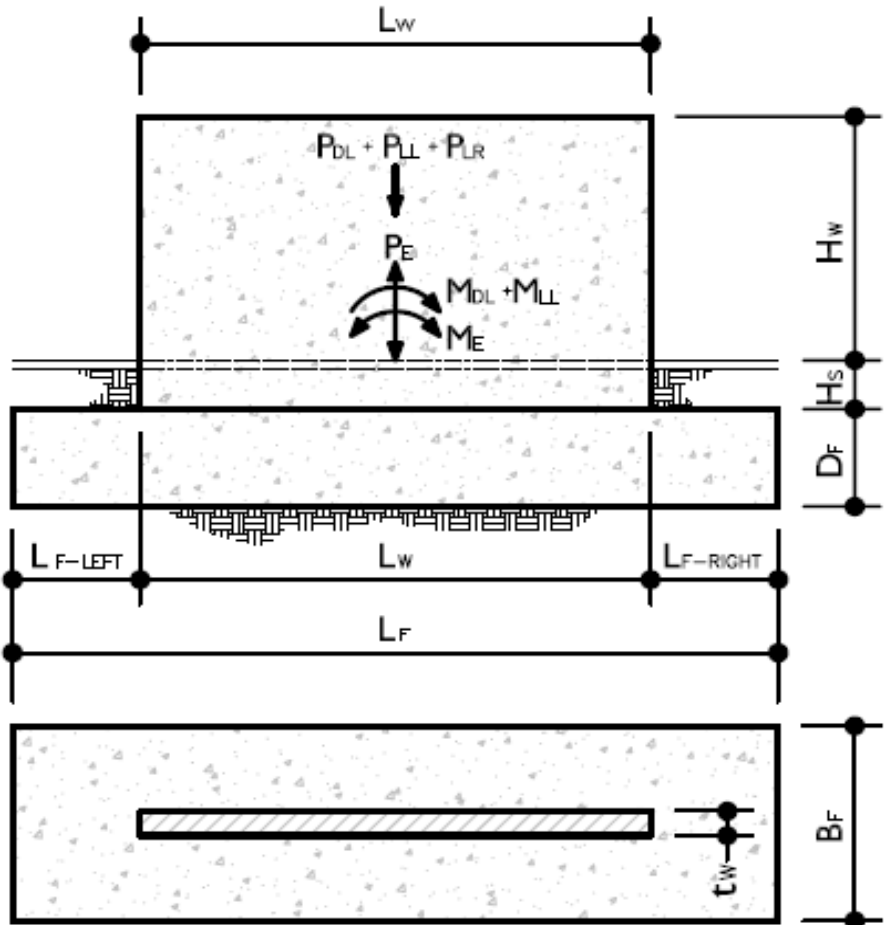
Shear Wall Location : Grid 2

Sheet :

Date : 6/25/2025

Checked :

Wall Type: Concrete



PLAN VIEW

STEP 1: GENERAL INPUT DATA

Redundancy Factor, ρ :		1.30	Concrete Strength (footing), f'_c :		3.00
S_{ds} :		1.601	Footing Width, B_f :		3.50
Allowable Soil Pressure q_a :		3.00	Footing Depth, D_f :		1.50
1/3 Increase Allowed?		YES	Footing Length Left, L_{f-LEFT} :		1.00
Net Allowable Pressure?		YES	Footing Length Right, $L_{f-RIGHT}$:		1.00
Wall Properties:			Total Footing Length, L_f :		56.0
Shear Wall Height, H_w :		18.00	Area of Footing, A_{FTG} :		196.0
Shear Wall Length, L_w :		54.00	Section Modulus of Footing, S_{FTG} :		1829.3
Shear Wall Thickness, t_w :		8.00	Unit Weight of Footing, γ_f :		150
Unit Weight of Wall, γ_w :		0	Footing Weight, W_f :		44.1
Wall Weight, W_w :		0.0	Soil Above Footing, H_s :		1.00
Ω_o =		2.50	Unit Weight of Soil, γ_s :		100
Omega?		NO	Weight of Soil, W_s :		19.6
			10% Reduction?		NO
			25% Reduction?		NO

STEP 2: APPLIED LOADS (At the Center of Wall)

	Seismic (E)	Dead (DL)	Live (LL)	Roof Live (LR)	
P =	0.0	57.0	0.0	22.0	k
M =	340.0	0.0	0.0	0.0	ft-k
V =	17.0	0.0	0.0	0.0	k
Mtot =	365.5	= M + V x Footing Depth			

STEP 3: FOOTING SIZE (Service Loads)

Comb. 1: D + L + Lr	[CBC Eq. 16-16]	$P_1 = (W_w + W_f + W_s + P_D) + P_L + P_{LR} = 142.7$ k	$\frac{P}{A} + \frac{M}{S}$		Allowable**
		$M_1 = (W_w + P_D + P_L + P_{LR})(L_{f-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} + M_{RL} = 0.0$ ft-k	0.73 ksf	<	3.25 ksf
		$M_1 NET = (W_w + P_D + P_L + P_{LR})(L_{f-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) = N/A$ ft-k	$\frac{P}{A} - \frac{M}{S}$		< O.K. >
		$X_1 = N/A$ ft	0.73 ksf		
Comb. 2: D + L + $\rho Q_E/1.4$ ($E_v = 0$)*	[CBC Eq. 16-20]	$P_2 = (W_w + W_f + W_s + P_D) + P_L + \rho P_E/1.4 = 120.7$ k	$\frac{P}{A} + \frac{M}{S}$		
		$M_2 = [(W_w + P_D) + P_L + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} + \rho M_E/1.4 = 339.4$ ft-k	0.80 ksf	<	4.33 ksf
		$M_2 NET = [(W_w + P_D) + P_L + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) + M_{DL} + M_{LL} + \rho M_E/1.4 = N/A$ ft-k	$\frac{P}{A} - \frac{M}{S}$		< O.K. >
		$a_2 = N/A$ ft	0.43 ksf		
		$X_2 = N/A$ ft			
Comb. 3: D + L - $\rho Q_E/1.4$ ($E_v = 0$)*	[CBC Eq. 16-20]	$P_3 = (W_w + W_f + W_s + P_D) - P_L + \rho P_E/1.4 = 120.7$ k	$\frac{P}{A} + \frac{M}{S}$		
		$M_2 = [(W_w + P_D) + P_L - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + M_{DL} + M_{LL} - \rho M_E/1.4 = (339.4)$ ft-k	0.80 ksf	<	4.33 ksf
		$M_2 NET = [(W_w + P_D) + P_L - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (W_f + W_s)(L_f/2) + M_{DL} + M_{LL} - \rho M_E/1.4 = N/A$ ft-k	$\frac{P}{A} - \frac{M}{S}$		< O.K. >
		$a_3 = N/A$ ft	0.43 ksf		
		$X_3 = N/A$ ft			
Comb. 4: 0.9D + $\rho Q_E/1.4$ ($E_v = 0$)*	[CBC Eq. 16-21]	$P_4 = (0.9)(W_w + W_f + W_s + P_D) + \rho P_E/1.4 = 108.6$ k	$\frac{P}{A} + \frac{M}{S}$		
		$M_4 = [(0.9)(W_w + P_D) + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + (0.9)M_{DL} + \rho M_E/1.4 = 339.4$ ft-k	0.74 ksf	<	4.33 ksf
		$M_4 NET = [(0.9)(W_w + P_D) + (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (0.9)(W_f + W_s)(L_f/2) + (0.9)M_{DL} + \rho M_E/1.4 = N/A$ ft-k	$\frac{P}{A} - \frac{M}{S}$		< O.K. >
		$a_4 = N/A$ ft	0.37 ksf		
		$X_4 = N/A$ ft			
Comb. 5: 0.9D - $\rho Q_E/1.4$ ($E_v = 0$)*	[CBC Eq. 16-21]	$P_5 = (0.9)(W_w + W_f + W_s + P_D) - \rho P_E/1.4 = 108.6$ k	$\frac{P}{A} + \frac{M}{S}$		
		$M_5 = [(0.9)(W_w + P_D) - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2 - L_f/2) + (0.9)M_{DL} - \rho M_E/1.4 = (339.4)$ ft-k	0.74 ksf	<	4.33 ksf
		$M_5 NET = [(0.9)(W_w + P_D) - (\rho P_E/1.4)](L_{f-LEFT} + L_w/2) + (0.9)(W_f + W_s)(L_f/2) + (0.9)M_{DL} - \rho M_E/1.4 = N/A$ ft-k	$\frac{P}{A} - \frac{M}{S}$		< O.K. >
		$a_5 = N/A$ ft	0.37 ksf		
		$X_5 = N/A$ ft			



SHEAR WALL FOOTING DESIGN

Project : FS- Reverse Apparatus - Line

Project # : 25534

Engineer : _____

Sheet : _____

Date : 6/25/2025

Checked : _____

Shear Wall : WALL -2

Shear Wall Location : Grid 2

Wall Type: Concrete

STEP 4: DESIGN FOR FLEXURE

A. Factored Soil Pressure

f₁ = 0.5

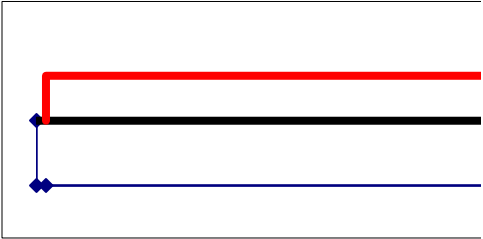
Comb. 6: 1.4D

[CBC Eq. 16-1]

P6 = 169.0 k
M6 = 0.0 ft-k
M6net = N/A ft-k
a6 = N/A ft
X6 = N/A ft

P/A + M/S = 0.86 ksf
P/A - M/S = 0.86 ksf

p1 = 0.862 ksf
p2 = 0.862 ksf
p3 = 0.862 ksf
p4 = 0.862 ksf



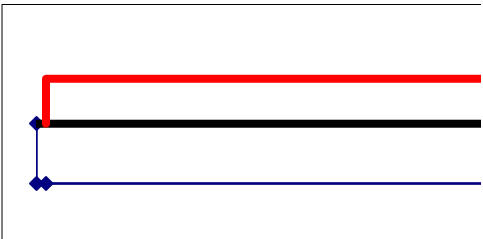
Comb. 7: 1.2D + 1.6L + 0.5Lr

[CBC Eq. 16-2]

P7 = 155.8 k
M7 = 0.0 ft-k
M7net = N/A ft-k
a7 = N/A ft
X7 = N/A ft

P/A + M/S = 0.80 ksf
P/A - M/S = 0.80 ksf

p1 = 0.795 ksf
p2 = 0.795 ksf
p3 = 0.795 ksf
p4 = 0.795 ksf



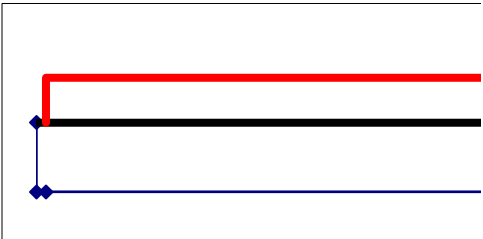
Comb. 8: 1.2D + f₁L + 1.6Lr

[CBC Eq. 16-3]

P8 = 180.0 k
M8 = 0.0 ft-k
M8net = N/A ft-k
a8 = N/A ft
X8 = N/A ft

P/A + M/S = 0.92 ksf
P/A - M/S = 0.92 ksf

p1 = 0.919 ksf
p2 = 0.919 ksf
p3 = 0.919 ksf
p4 = 0.919 ksf



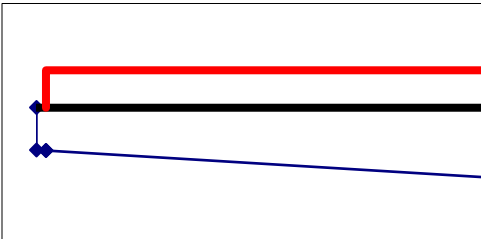
Comb.9: (1.2 + 0.2S_{DS})D + f₁L + pQ_E

[CBC Eq. 16-5]

P9 = 183.5 k
M9 = 475.2 ft-k
M9net = 5612.8 ft-k
a9 = 30.59 ft
X9 = 56.00 ft

P/A + M/S = 1.20 ksf
P/A-M/S or 0 = 0.68 ksf

p1 = 0.676 ksf
p2 = 0.686 ksf
p3 = 1.187 ksf
p4 = 1.196 ksf



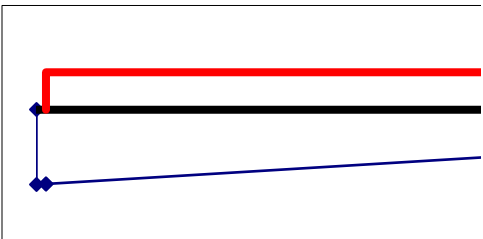
Comb. 10: (1.2 + 0.2S_{DS})D + f₁L - pQ_E

[CBC Eq. 16-5]

P10 = 183.5 k
M10 = (475.2) ft-k
M10net = 4662.5 ft-k
a10 = 25.41 ft
X10 = 56.00 ft

P/A + M/S = 1.20 ksf
P/A-M/S or 0 = 0.68 ksf

p1 = 1.196 ksf
p2 = 1.187 ksf
p3 = 0.686 ksf
p4 = 0.676 ksf



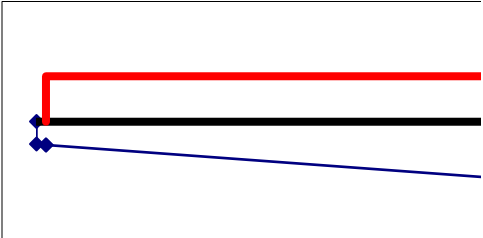
Comb. 11: 0.9D + pQ_E

[CBC Eq. 16-6]

P11 = 108.6 k
M11 = 475.2 ft-k
M11net = 3516.8 ft-k
a11 = 32.37 ft
X11 = 56.00 ft

P/A + M/S = 0.81 ksf
P/A-M/S or 0 = 0.29 ksf

p1 = 0.294 ksf
p2 = 0.304 ksf
p3 = 0.805 ksf
p4 = 0.814 ksf



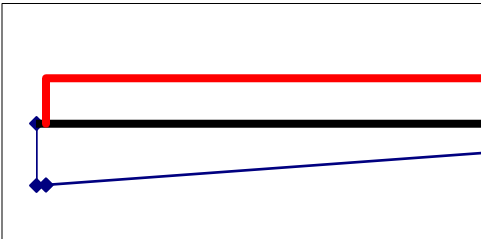
Comb. 12: 0.9D - pQ_E

[CBC Eq. 16-6]

P11 = 108.6 k
M11 = (475.2) ft-k
M11net = 2566.5 ft-k
a11 = 23.63 ft
X11 = 56.00 ft

P/A + M/S = 0.81 ksf
P/A-M/S or 0 = 0.29 ksf

p1 = 0.814 ksf
p2 = 0.805 ksf
p3 = 0.304 ksf
p4 = 0.294 ksf





SHEAR WALL FOOTING DESIGN

Project : FS- Reverse Apparatus - Line

Project # : 25534

Shear Wall : WALL -2

Engineer :

Shear Wall Location : Grid 2

Sheet :

Date : 6/25/2025

Checked :

Wall Type: Concrete

B. Longitudinal Reinforcing [Bottom]

Rebar clearance from the bottom : 3.00 in

Assuming tension-controlled section

Steel Strength (rebar), f_y : 60.00

$d_t = D_F - \text{clear} - 1" = 14$ in

ϕ : 0.90

β_1 : 0.85

ρ_{MIN} : 0.0033333

Left	Mu @ p2 (ft-k)	Soil Pressure
Comb. 6:	0.7	Case 1
Comb. 7:	0.7	Case 1
Comb. 8:	0.9	Case 1
Comb. 9:	0.3	Case 2B
Comb. 10:	1.2	Case 2A
Comb. 11:	0.0	Case 2B
Comb. 12:	0.9	Case 2A

Right	Mu @ p3 (ft-k)	Soil Pressure
Comb. 6:	0.7	Case 1
Comb. 7:	0.7	Case 1
Comb. 8:	0.9	Case 1
Comb. 9:	1.2	Case 2B
Comb. 10:	0.3	Case 2A
Comb. 11:	0.9	Case 2B
Comb. 12:	0.0	Case 2A

$\rho = 0.00003$ Temp. Steel

AS REQ'D = 0.68 in²

Rebars provided 4- #6

As prov.= 1.77 in² < O.K. >

c = 1.16 in

c / d_t = 0.083

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Spacing = 11.33 in

$\rho = 0.00003$ Temp. Steel

AS REQ'D = 0.68 in²

Rebars provided 4- #6

As prov.= 1.77 in² < O.K. >

c = 1.16 in

c / d_t = 0.083

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Spacing = 11.33 in

C. Longitudinal Reinforcing [Top] [Moments are due to the weight of footing and soil]

Left

Right

Mu @ p2 = 0.80 ft-k

Mu @ p3 = 0.80 ft-k

$\rho = 0.00002$ Temp. Steel

$\rho = 0.00002$ Temp. Steel

AS REQ'D = 0.68 in²

AS REQ'D = 0.68 in²

Rebars provided 4- #6

Rebars provided 4- #6

As prov.= 1.77 in² < O.K. >

As prov.= 1.77 in² < O.K. >

c = 1.16 in

c = 1.16 in

c / d_t = 0.083

c / d_t = 0.083

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Tension Control Assumption is Correct.

Spacing = 11.33 in

Spacing = 11.33 in

D. Transverse Reinforcing [Bottom]

$d_t = D_F - \text{clear} - 2" = 13$ in

b = 12 in

Left

Right

Mu @ p2 = 0.69 ft-k / ft

Mu @ p3 = 0.69 ft-k

$\rho = 0.00008$ Temp. Steel

$\rho = 0.00008$ Temp. Steel

AS REQ'D = 0.19 in²

AS REQ'D = 0.19 in²

Rebars provided #5 @ 18 o.c. < O.K. >

Rebars provided #5 @ 18 o.c. < O.K. >

As prov.= 0.20 in² < O.K. >

As prov.= 0.20 in² < O.K. >

c = 0.13 in

c = 0.13 in

c / d_t = 0.010

c / d_t = 0.010

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.

Tension Control Assumption is Correct.

E. Transverse Reinforcing [Top]

b = 12 in

Mu = 0.46 ft-k

As prov.= 0.20 in² < O.K. >

$\rho = 0.00005$ Temp. Steel

c = 0.13 in

AS REQ'D = 0.19 in²

c / d_t = 0.010

Rebars provided #5 @ 18 o.c. < O.K. >

$\phi = 0.65 + 0.25 [(1 / (c / d_t)) - 5/3] = 0.90$

Tension Control Assumption is Correct.



SHEAR WALL FOOTING DESIGN

Project : FS- Reverse Apparatus - Line
Project # : 25534
Engineer :
Shear Wall : WALL -2
Shear Wall Location : Grid 2

Sheet :
Date : 6/25/2025
Checked :
Wall Type: Concrete

STEP 5: CHECK FOR SHEAR

A. Longitudinal [The critital location for flexural shear is a distance "d" from the end of wall]

Steel Strength (rebar), f_y : 60.00

Rebar clearance from the bottom : 3.00 in
 $d_t = D_F - \text{clear} - 1" = 14$ in
 ϕ : 0.75

Left	V_{u12} (kips)	M_{u12} (ft-k)	V_{ud}/M_u	ϕV_c (kips)	
Comb. 6:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 7:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 8:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 9:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 10:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 11:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 12:	N/A	N/A	N/A	48.3	< O.K. >

Right	V_{u34} (kips)	M_{u34} (ft-k)	V_{ud}/M_u	ϕV_c (kips)	
Comb. 6:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 7:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 8:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 9:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 10:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 11:	N/A	N/A	N/A	48.3	< O.K. >
Comb. 12:	N/A	N/A	N/A	48.3	< O.K. >

*Design shear, V_u , is N/A if $d > L_F\text{-Left}$ or $L_F\text{-Right}$

Shear Capacity

Larger of
$$\left\{ \begin{aligned} \phi V_c &= \phi 2 \sqrt{f'_c} b d = 48.3 \text{ k} & b &= B_F & (\text{ACI Eq. 11-3}) \\ \phi V_c &= \phi \left(1.9 \sqrt{f'_c} + 2500 \rho_w \frac{V_u d}{M_u} \right) b_w d < 3.5 \sqrt{f'_c} b_w d & & & (\text{ACI Eq. 11-5}) \end{aligned} \right.$$

V_{ud}/M_u shall not be greather than 1.0

B. Transverse [The critital location for flexural shear is a distance "d" from the face of wall]

$d_t = D_F - \text{clear} - 2" = 13$ in
 $b = 12$ in

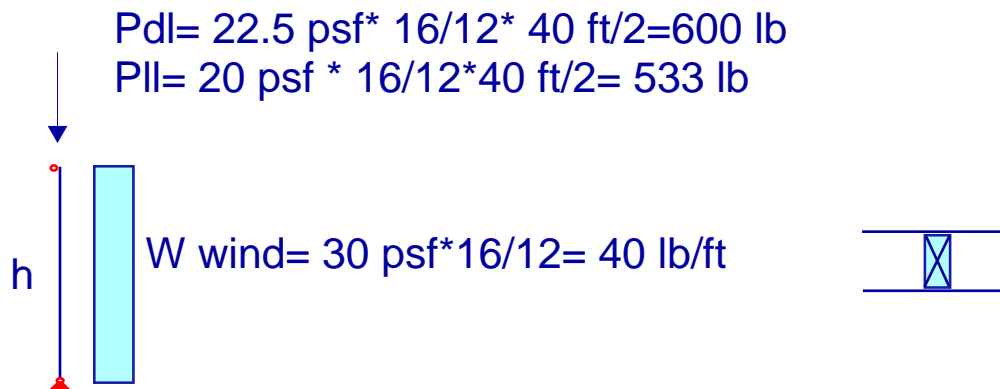
Shear Capacity
$$\phi V_c = \phi 2 \sqrt{f'_c} b d = 13.8 \text{ k}$$

Left $V_u @ p2 = 0.23 \text{ k}$ < O.K. > Right $M_u @ p3 = 0.23 \text{ k}$ < O.K. >

STEP 6: SUMMARY

FOOTING SIZE			LONGITUDINAL REINFORCEMENT			
Footing Width =	3.5	ft	Left		Right	
Footing Depth =	1.5	ft	Bottom:	4- #6	Bottom:	4- #6
Total Footing Length =	56.0	ft	Top:	4- #6	Top:	4- #6
Footing Length (Left) =	1.0	ft	TRANSVERSE REINFORCEMENT			
Footing Length (Right) =	1.0	ft	Left		Right	
Shear Wall Length, L_w =	54.0	ft	Bottom:	#5 @ 18 o.c.	Bottom:	#5 @ 18 o.c.
			Top:	#5 @ 18 o.c.	Top:	#5 @ 18 o.c.

Stud wall design at Reserve Apparatus



$$h = 18'-3" - (0'-3" + 2.5" + 0'-6") = 17'-3 \frac{1}{2}"$$

$$2 \times 6 @ 16" \text{ DCR} = 0.92$$

$$\Delta_{el} = 2.3" \times 0.42 = 0.96" \quad L/\Delta_{el} = 17.25 \text{ ft} \times 12/0.97" = 214 < 360 \text{ NG}$$

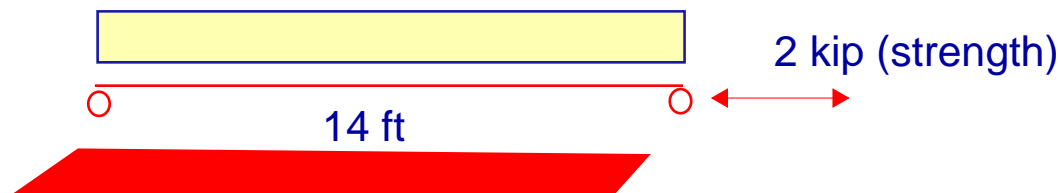
$$2 \times 8 @ 16" \text{ DCR} = 0.5$$

$$\Delta_{el} = 1" \times 0.42 = 0.42" \quad L/\Delta_{el} = 17.25 \text{ ft} \times 12/0.42" = 494 > 360$$

Provide 2x8 @ 16" o.c OK

$$W_{dl} = 24 \text{ psf} \times 42 \text{ ft}/2 = 0.5 \text{ kip/ft}$$

$$W_{ll} = 20 \text{ psf} \times 42 \text{ ft}/2 = 0.42 \text{ kip/ft}$$



$$W_{\text{wind}} = 30 \text{ psf} \times 18 \text{ ft}/2 = 0.27 \text{ kip/ft}$$

(strength)

8x14 beam is OK

Wood Column

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Reserve Apparatus Header- Axial and Bending

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
 Load Combinations Used : ASCE 7-16

General Information

Analysis Method		Allowable Stress Design		Wood Section Name		8x14	
End Fixities		Top & Bottom Pinned		Wood Grading/Manuf.		Graded Lumber	
Overall Column Height		14 ft		Wood Member Type		Sawn	
(Used for non-slender calculations)							
Wood Species		Douglas Fir-Larch		Exact Width		7.50 in	
Wood Grade		No.1		Exact Depth		13.50 in	
Fb +		1,350.0 psi	Fv	170.0 psi	Area		101.250 in^2
Fb -		1,350.0 psi	Ft	675.0 psi	Ix		1,537.73 in^4
Fc - Prll		925.0 psi	Density	31.210 pcf	Iy		474.609 in^4
Fc - Perp		625.0 psi					
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial			
Basic		1,600.0	1,600.0	1,600.0 ksi		Cf or Cv for Bending 0.9870	
Minimum		580.0	580.0			Cf or Cv for Compression0.9870	
						Cf or Cv for Tension 0.9870	
						Cm : Wet Use Factor 1.0	
						Ct : Temperature Fact 1.0	
						Cfu : Flat Use Factor 1.0	
						Kf : Built-up columns 1.0	
						Use Cr : Repetitive ? No	
Brace condition for deflection (buckling) along columns :							
				X-X (width) axis : Unbraced Length for buckling ABOUT Y-Y Axis = 14 ft, k			
				Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 14 ft, k			

Applied Loads

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

Column self weight included : 307.223 lbs * Dead Load Factor

AXIAL LOADS . . .

Axial Load at 14.0 ft, E = 2.0 k

BENDING LOADS . . .

Lat. Uniform Load creating Mx-x, D = 0.50, LR = 0.40 k/ft

Lat. Uniform Load creating My-y, W = 0.270 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.7056 : 1	Maximum SERVICE Lateral Load Reactions . .			
Load Combination	+D+Lr	Top along Y-Y	6.30 k	Bottom along Y-Y	6.30 k
Governing NDS Formula Comp + Mxx, NDS Eq. 3.9-3		Top along X-X	1.890 k	Bottom along X-X	1.890 k
Location of max.above base	6.953 ft	Maximum SERVICE Load Lateral Deflections . . .			
At maximum location values are .		Along Y-Y	0.3196 in at 7.047 ft above base		
Applied Axial	0.3072 k	for load combination : +D+Lr			
Applied Mx	22.049 k-ft	Along X-X	0.3106 in at 7.047 ft above base		
Applied My	0.0 k-ft	for load combination : W Only			
Fc : Allowable	712.88 psi	Other Factors used to calculate allowable stresses . . .			
PASS Maximum Shear Stress Ratio =	0.4392 : 1			<u>Bending</u>	<u>Compression</u>
Load Combination	+D+Lr				<u>Tension</u>
Location of max.above base	14.0 ft				
Applied Design Shear	93.333 psi				
Allowable Shear	212.50 psi				

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.739	0.5426	PASS	6.953 ft	0.3389	PASS	14.0 ft
+D+Lr	1.250	0.625	0.7056	PASS	6.953 ft	0.4392	PASS	14.0 ft
+D+0.750Lr	1.250	0.625	0.6272	PASS	6.953 ft	0.3904	PASS	0.0 ft
+D+0.60W	1.600	0.531	0.4854	PASS	6.953 ft	0.1906	PASS	14.0 ft
+D+0.750Lr+0.450W	1.600	0.531	0.6264	PASS	7.047 ft	0.3050	PASS	0.0 ft
+D+0.450W	1.600	0.531	0.4409	PASS	6.953 ft	0.1906	PASS	14.0 ft
+0.60D+0.60W	1.600	0.531	0.3615	PASS	6.953 ft	0.1144	PASS	14.0 ft
+D+0.70E	1.600	0.531	0.3093	PASS	6.953 ft	0.1906	PASS	14.0 ft

Wood Column

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Reserve Apparatus Header- Axial and Bending

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
+D+0.5250E	1.600	0.531	0.3088	PASS	6.953 ft	0.1906	PASS	14.0 ft
+0.60D+0.70E	1.600	0.531	0.1856	PASS	6.953 ft	0.1144	PASS	14.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top		@ Base	@ Top		@ Base	@ Top
D Only				3.500	3.500	0.307					
+D+Lr				6.300	6.300	0.307					
+D+0.750Lr				5.600	5.600	0.307					
+D+0.60W	1.134	1.134		3.500	3.500	0.307					
+D+0.750Lr+0.450W	0.851	0.851		5.600	5.600	0.307					
+D+0.450W	0.851	0.851		3.500	3.500	0.307					
+0.60D+0.60W	1.134	1.134		2.100	2.100	0.184					
+D+0.70E				3.500	3.500	1.707					
+D+0.5250E				3.500	3.500	1.357					
+0.60D+0.70E				2.100	2.100	1.584					
Lr Only				2.800	2.800						
W Only	1.890	1.890									
E Only						2.000					

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance	Max. Y-Y Deflection		Distance
	in	ft		in	ft	
D Only	0.0000	0.0000	0.000ft	0.178	7.047	ft
+D+Lr	0.0000	0.0000	0.000ft	0.320	7.047	ft
+D+0.750Lr	0.0000	0.0000	0.000ft	0.284	7.047	ft
+D+0.60W	0.1864	7.047ft		0.178	7.047	ft
+D+0.750Lr+0.450W	0.1398	7.047ft		0.284	7.047	ft
+D+0.450W	0.1398	7.047ft		0.178	7.047	ft
+0.60D+0.60W	0.1864	7.047ft		0.107	7.047	ft
+D+0.70E	0.0000	0.000ft		0.178	7.047	ft
+D+0.5250E	0.0000	0.000ft		0.178	7.047	ft
+0.60D+0.70E	0.0000	0.000ft		0.107	7.047	ft
Lr Only	0.0000	0.000ft		0.142	7.047	ft
W Only	0.3106	7.047ft		0.000	0.000	ft
E Only	0.0000	0.000ft		0.000	0.000	ft

Wood Column

Project File: FS 46 enercal.ec6

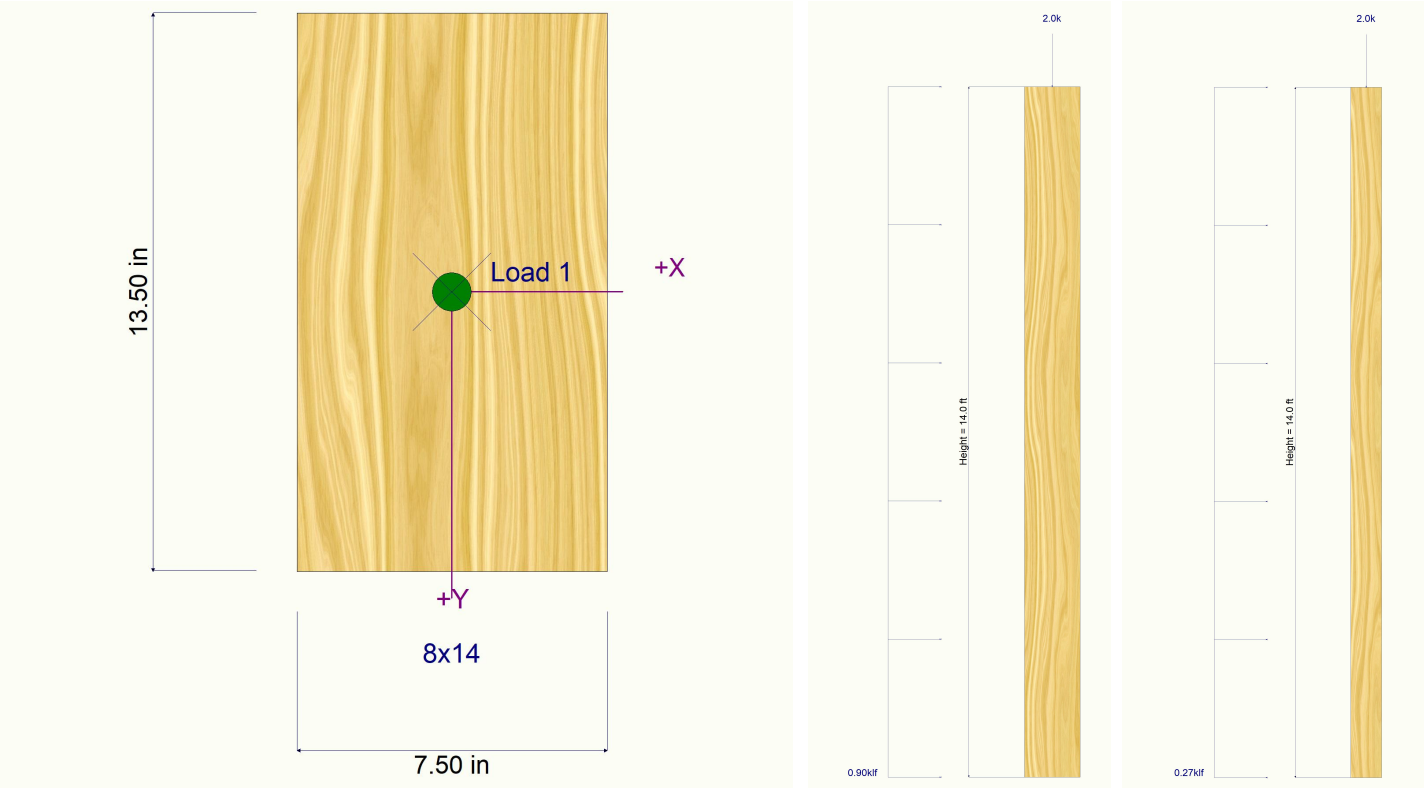
LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Reserve Apparatus Header- Axial and Bending

Sketches



Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Header Beam-Reserve Apparatus

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

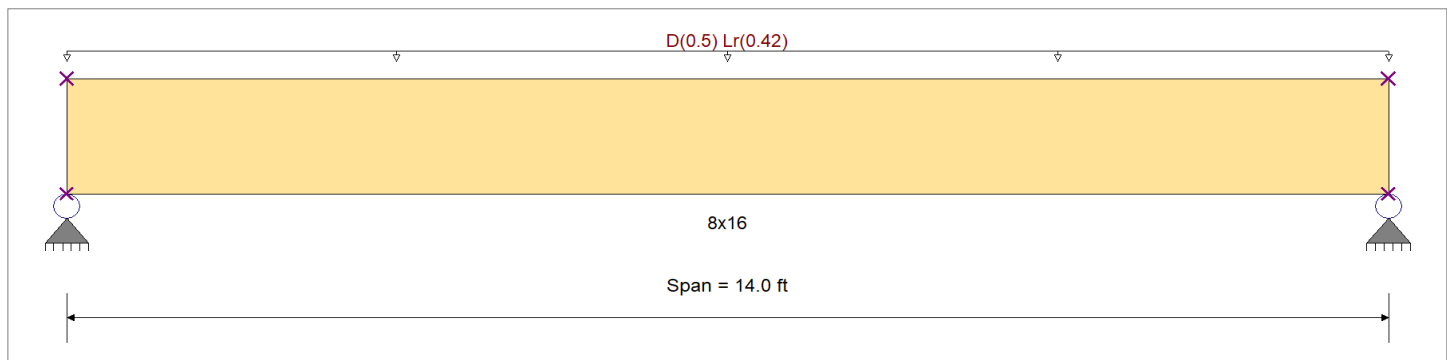
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
 Wood Grade : No.1

Beam Bracing : Completely Unbraced

Fb +	1,350.0 psi	E : Modulus of Elasticity	
Fb -	1,350.0 psi	Ebend- xx	1,600.0 ksi
Fc - Prll	925.0 psi	Eminbend - xx	580.0 ksi
Fc - Perp	625.0 psi		
Fv	170.0 psi		
Ft	675.0 psi	Density	31.210 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.50, Lr = 0.420 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.556	1	Section used for this span	=	0.320	1
fb: Actual	=	900.66	psi	fv: Actual	=	67.93	psi
F'b	=	1,619.26	psi	F'v	=	212.50	psi
Load Combination		+D+Lr		Load Combination		+D+Lr	
Location of maximum on span	=	7.000	ft	Location of maximum on span	=	12.723	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.098 in	Ratio =	1713 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.215 in	Ratio =	782 >=180	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0			
Length = 14.0 ft	1	0.418	0.241	0.90	1.00	1.00	0.99	0.972	1.00	1.00	1.00	12.25	489.5	1,170.7	2.86	36.9	153.0
+D+Lr														0.0			
Length = 14.0 ft	1	0.556	0.320	1.25	1.00	1.00	0.99	0.972	1.00	1.00	1.00	22.54	900.7	1,619.3	5.26	67.9	212.5
+D+0.750Lr														0.0			
Length = 14.0 ft	1	0.493	0.283	1.25	1.00	1.00	0.99	0.972	1.00	1.00	1.00	19.97	797.9	1,619.3	4.66	60.2	212.5
+0.60D														0.0			
Length = 14.0 ft	1	0.142	0.081	1.60	1.00	1.00	0.98	0.972	1.00	1.00	1.00	7.35	293.7	2,062.9	1.72	22.2	272.0

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Header Beam-Reserve Apparatus

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.2148	7.051		0.0000	0.000

Vertical Reactions

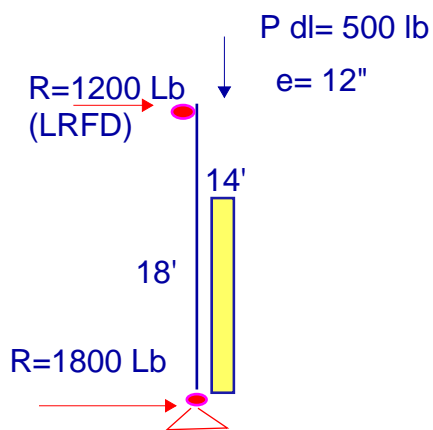
Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	6.440	6.440
Max Upward from Load Combinations	6.440	6.440
Max Upward from Load Cases	3.500	3.500
D Only	3.500	3.500
+D+Lr	6.440	6.440
+D+0.750Lr	5.705	5.705
+0.60D	2.100	2.100
Lr Only	2.940	2.940

HSS support Roll up door:

Wind= $30 \text{ psf} \times 14 \text{ ft} / 2 = 210 \text{ lb/ft}$



Door weight:

$1.25 \text{ psf} \times 14' \times 14' = 245 \text{ lb}$, design for 1000 lb (including Motor)

HSS 6x6x3/8

DCR= 0.13

$\Delta e = 0.6 \times 0.388 = 0.23 \text{ ''}$

$L/\Delta e = 18 \times 12 / 0.23 = 939 >> 360 \text{ OK}$

15/16 '' PLYWOOD STR I, 10d @ 4, 6, 12
 $V_a = 850 \text{ LB/FT} / 2 = 425 \text{ LB/FT}$

1. Single HSS $R = 1200 \text{ lb} / 1.6 = 750 \text{ lb}$

Try 3/4'' DIA MB

$P_a = 1560 \text{ lb} \times 1.6 = 2496 \text{ lb}$ / bolt provide 2 bolts

6x6 length = $750 \text{ lb} / 425 \text{ lb/ft} = 1.8 \text{ ft}$ provide 3'-0"

1. Double HSS $R = 2400 \text{ lb} / 1.6 = 1500 \text{ lb}$

Try 3/4'' DIA MB

$P_a = 1560 \text{ lb} \times 1.6 = 2496 \text{ lb}$ / bolt provide 3 bolts

6x6 length = $1500 \text{ lb} / 425 \text{ lb/ft} = 3.5 \text{ ft}$ provide 6'-0"

Steel Column

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HSS column at Rollup Door

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
 Load Combinations Used : ASCE 7-16

General Information

Steel Section Name :	HSS6x6x3/8	Overall Column Height	18 ft
Analysis Method :	Allowable Strength	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade		Brace condition for deflection (buckling) along columns :	
Fy : Steel Yield	46.0 ksi	X-X (width) axis :	
E : Elastic Bending Modulus	29,000.0 ksi	Unbraced Length for buckling ABOUT Y-Y Axis = 18 ft, K = 1.0	
		Y-Y (depth) axis :	
		Unbraced Length for buckling ABOUT X-X Axis = 18 ft, K = 1.0	

Applied Loads

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

Column self weight included : 493.304 lbs * Dead Load Factor
 AXIAL LOADS . . .
 Axial Load at 18.0 ft, Xecc = 12.0 in, Yecc = 12.0 in, D = 0.50 k
 BENDING LOADS . . .
 Lat. Uniform Load from 0.0-->14.0 ft creating Mx-x, W = 0.210 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.1315 : 1	Maximum Load Reactions . .	
Load Combination	+D+0.60W	Top along X-X	0.02778 k
Location of max.above base	8.577 ft	Bottom along X-X	0.02778 k
At maximum location values are . . .		Top along Y-Y	1.143 k
Pa : Axial	0.9933 k	Bottom along Y-Y	1.797 k
Pn / Omega : Allowable	114.160 k	Maximum Load Deflections . . .	
Ma-x : Applied	4.373 k-ft	Along Y-Y	0.3877 in at 8.940ft above base
Mn-x / Omega : Allowable	36.267 k-ft	for load combination :W Only	
Ma-y : Applied	-0.2383 k-ft	Along X-X	-0.01581 in at 10.510ft above base
Mn-y / Omega : Allowable	36.267 k-ft	for load combination :D Only	
PASS Maximum Shear Stress Ratio	0.01858 : 1		
Load Combination	+0.60D+0.60W		
Location of max.above base	0.0 ft		
At maximum location values are . . .			
Va : Applied	1.061 k		
Vn / Omega : Allowable	57.137 k		

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios				Maximum Shear Ratios					
	Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Stress Ratio	Status	Location
D Only	0.032	PASS	17.88 ft	1.17	1.66	94.74	94.74	0.000	PASS	0.00 ft
+D+0.60W	0.132	PASS	8.58 ft	1.17	1.66	94.74	94.74	0.018	PASS	0.00 ft
+D+0.450W	0.100	PASS	8.58 ft	1.17	1.66	94.74	94.74	0.014	PASS	0.00 ft
+0.60D+0.60W	0.130	PASS	8.58 ft	1.17	1.66	94.74	94.74	0.019	PASS	0.00 ft
+0.60D	0.019	PASS	17.88 ft	1.17	1.66	94.74	94.74	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction			k	Y-Y Axis Reaction		Mx - End Moments		My - End Moments	
	@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
D Only	0.993	0.028	0.028		-0.028	0.028				
+D+0.60W	0.993	0.028	0.028		1.050	0.714				
+D+0.450W	0.993	0.028	0.028		0.781	0.542				
+0.60D+0.60W	0.596	0.017	0.017		1.061	0.703				
+0.60D	0.596	0.017	0.017		-0.017	0.017				
W Only					1.797	1.143				

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Column

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: HSS column at Rollup Door

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base @ Top	k	Y-Y Axis Reaction @ Base @ Top	Mx - End Moments @ Base @ Top	k-ft	My - End Moments @ Base @ Top
Axial @ Base	Maximum	0.993	0.028	0.028	-0.028	0.028		
"	Minimum				1.797	1.143		
Reaction, X-X Axis Base	Maximum	0.993	0.028	0.028	-0.028	0.028		
"	Minimum				1.797	1.143		
Reaction, Y-Y Axis Base	Maximum				1.797	1.143		
"	Minimum	0.993	0.028	0.028	-0.028	0.028		
Reaction, X-X Axis Top	Maximum	0.993	0.028	0.028	-0.028	0.028		
"	Minimum				1.797	1.143		
Reaction, Y-Y Axis Top	Maximum	0.993	0.028	0.028	-0.028	0.028		
"	Minimum				1.797	1.143		
Moment, X-X Axis Base	Maximum	0.993		0.028	-0.028	0.028		
"	Minimum	0.993		0.028	-0.028	0.028		
Moment, Y-Y Axis Base	Maximum	0.993	0.028	0.028	-0.028	0.028		
"	Minimum	0.993	0.028	0.028	-0.028	0.028		
Moment, X-X Axis Top	Maximum	0.993	0.028	0.028	-0.028	0.028		
"	Minimum	0.993	0.028	0.028	-0.028	0.028		
Moment, Y-Y Axis Top	Maximum	0.993	0.028	0.028	-0.028	0.028		
"	Minimum	0.993	0.028	0.028	-0.028	0.028		

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	-0.0158 in	10.510 ft	-0.016 in	10.510 ft
+D+0.60W	-0.0158 in	10.510 ft	0.217 in	8.819 ft
+D+0.450W	-0.0158 in	10.510 ft	0.159 in	8.819 ft
+0.60D+0.60W	-0.0095 in	10.510 ft	0.223 in	8.819 ft
+0.60D	-0.0095 in	10.510 ft	-0.009 in	10.510 ft
W Only	0.0000 in	0.000 ft	0.388 in	8.940 ft

Steel Section Properties : HSS6x6x3/8

Depth	=	6.000 in	I xx	=	39.50 in^4	J	=	64.600 in^4
Design Thick	=	0.349 in	S xx	=	13.20 in^3			
Width	=	6.000 in	R xx	=	2.280 in			
Wall Thick	=	0.375 in	Zx	=	15.800 in^3			
Area	=	7.580 in^2	I yy	=	39.500 in^4	C	=	22.100 in^3
Weight	=	27.406 plf	S yy	=	13.200 in^3			
			R yy	=	2.280 in			

Ycg = 0.000 in

ADDENDUM 5

Steel Column

Project File: FS 46 enercal.ec6

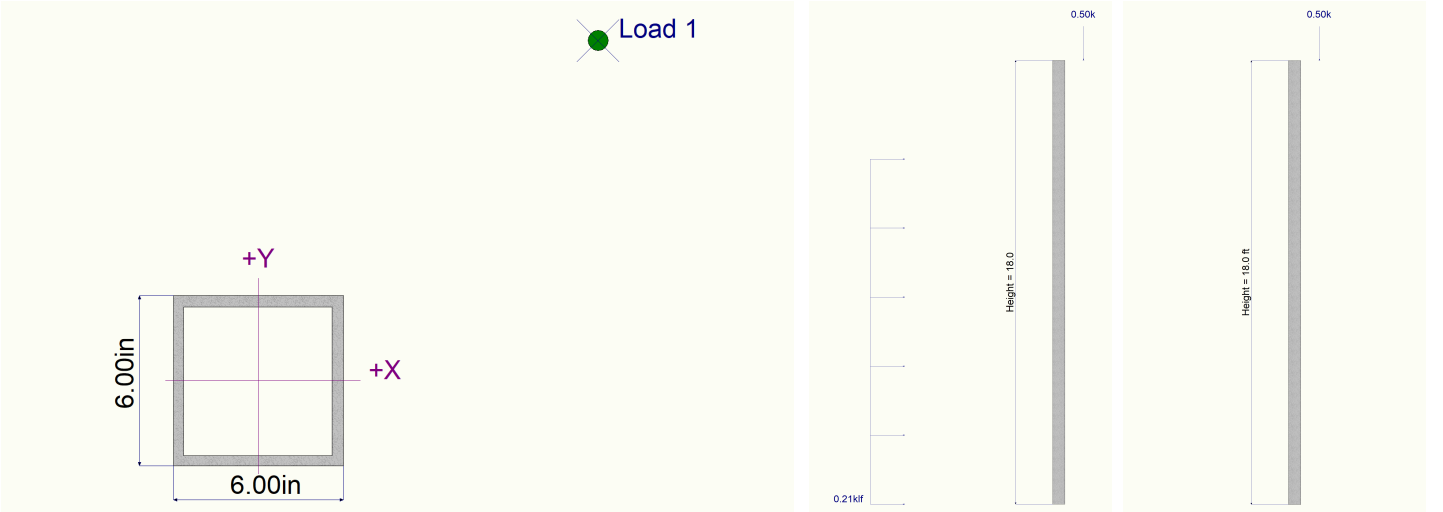
LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

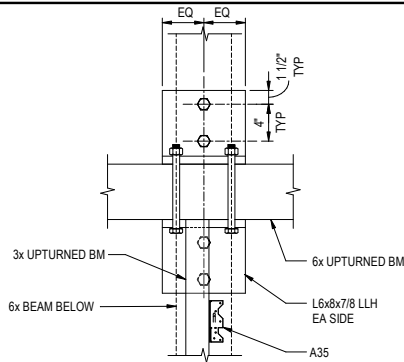
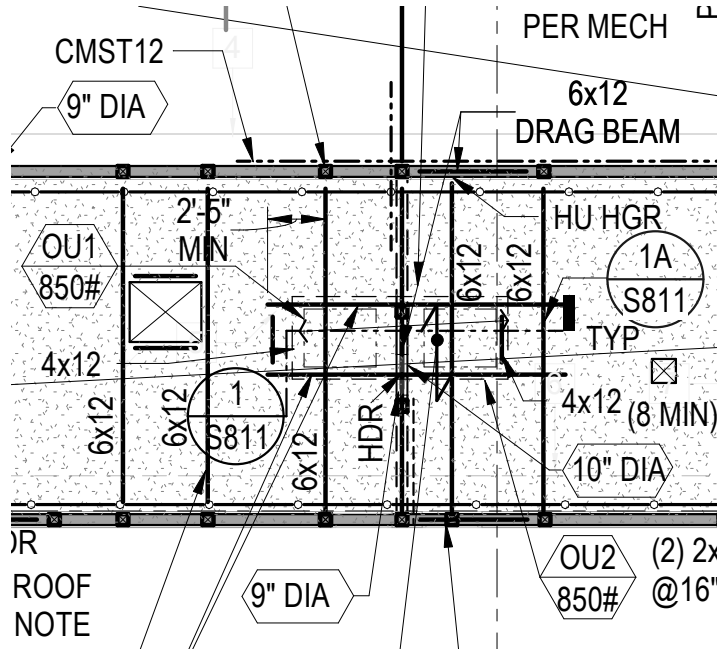
(c) ENERCALC INC 1983-2022

DESCRIPTION: HSS column at Rollup Door

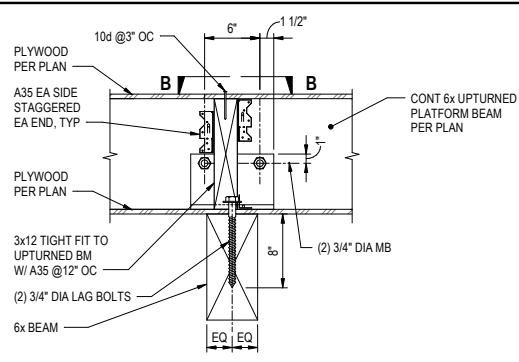
Sketches



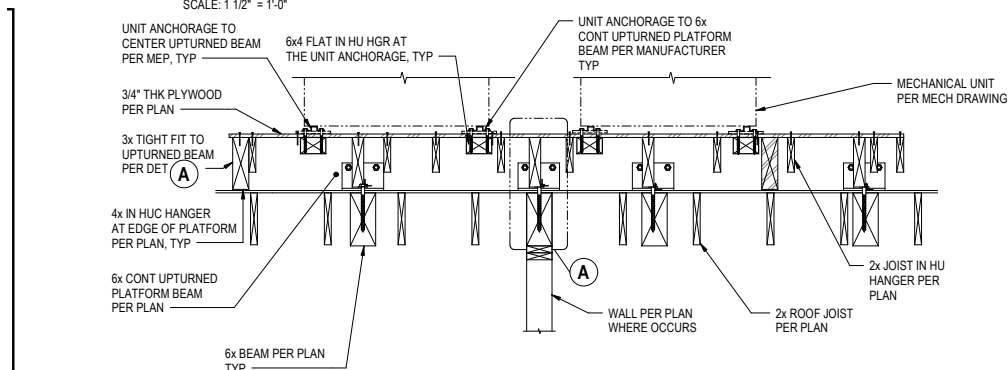
4.7 UNIT



DETAIL B-B
SCALE: 1 1/2" = 1'-0"



DETAIL A
SCALE: 1 1/2" = 1'-0"



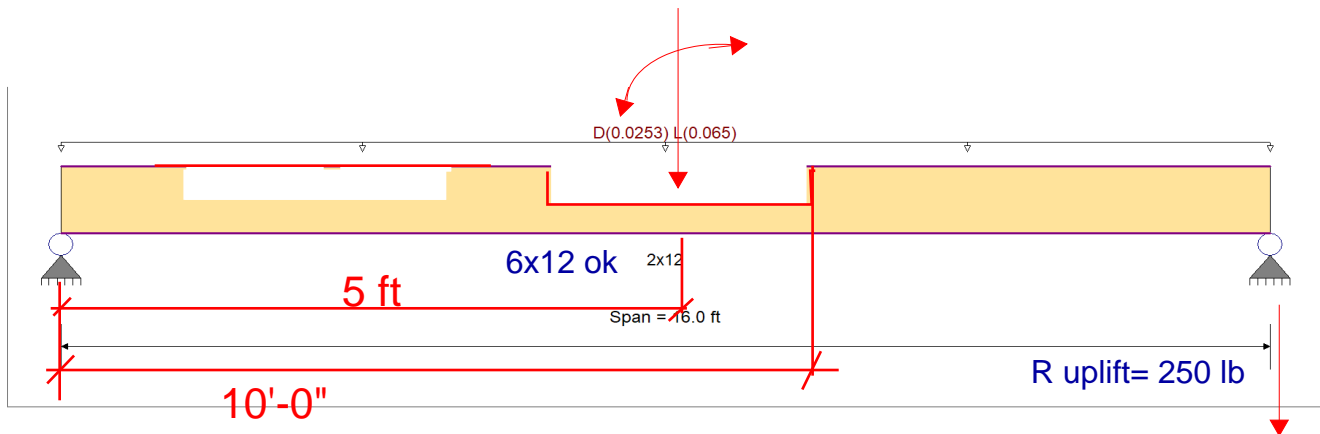
SECTION AT MECHANICAL UNIT
SCALE: 3/4" = 1'-0"

1

6x12 support OC units

$$P_{unit} = 1000 \text{ lb}/2 = 500 \text{ lb}$$

$$M = 9084 \text{ lbft}/2 = 4542 \text{ lbft}$$



6x12 support Mua

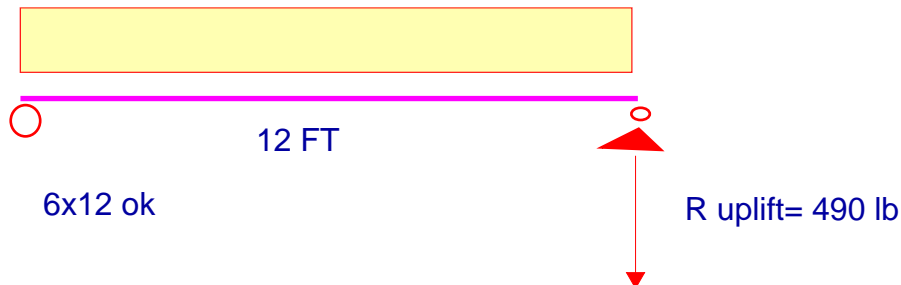
$$DL = 19 \text{ psf} \times 4 \text{ ft} = 76 \text{ lb/ft}$$

$$LL = 50 \text{ psf} \times 4 \text{ ft} = 200 \text{ lb/ft}$$

$$\text{unit} = 3000$$

$$\text{lb}/6\text{ft} \times 12\text{ft} = 42 \text{ psf}$$

$$< 50 \text{ psf}$$



$$=1.6*1.601*1.5*950 \text{ lb} = 3650 \text{ lb}$$

CALCULATIONS FOR VIBREX TYPE RMU-EQ-SH VIBRATION ISOLATORS/SEISMIC RESTRAINTS - 2021 IBC/2022 CBC, L

CALCULATES WORST CASE LOADING FOR (4) to (14) ISOLATORS. (UNITS: LBS & INCHES)

USING COMBINED LOAD EQUATIONS 16-5: $U = 1.2D + 1.0E$ & 16-7: $U = 0.9D + 1.0E$

SEISMIC/WIND FORCES - CH. 13, 26, & 29 OF ASCE 7

13.3-2: $F_p = 1.6 * S_{DS} * I_p * W_p$ (MAX)	=	3876.00 lbs
13.3-1: $F_p = (0.4 * a_p * S_{DS} / (R_p / I_p)) * (1 + 2z / h_r) * W_p$	=	3633.75 lbs
13.3-3: $F_p = 0.3 * S_{DS} * I_p * W_p$ (MIN)	=	726.75 lbs
CONTROLLING HOR. SEISMIC FORCE: F_p	=	3633.75 lbs
OVERTURNING MOMENT: $M = F_p * H_{C.G.}$	=	109012.50 lb-in
* SUPP. VERT. SEISM. FORCE: $F_{pv} = 0.2 S_{DS} W_p$	=	323.00 lbs
DESIGN SPECTRAL RESP. ACCELERATION: S_{DS}	=	1.700
IMPORTANCE FACTOR: I_p	=	1.50
COMPONENT AMPLIFICATION FACTOR: a_p	=	2.50
COMP. RESPONSE MODIFICATION FACTOR: R_p	=	2.00
EQUIPMENT ELEVATION / ROOF ELEVATION: z / h_r	=	1.00

(WITH RESPECT TO GRADE)

UNIT LOCATED INDOORS OR OUTDOORS	=	OUTDOORS
F_h FROM PREVIOUS CALCULATION (IF APPLICABLE)	=	879.75 lbs
F_v FROM PREVIOUS CALCULATION (IF APPLICABLE)	=	282.96 lbs
WIND OVERTURNING MOMENT: $M_w = F_p H_w$	=	30791.25 lb-in

EQUIPMENT WEIGHT & GEOMETRY

EQUIPMENT WEIGHT: W_p (UNIT + RAIL)	=	950.00 lbs
L:	=	22.88 in
B:	=	18.00 in
E_x :	=	2.00 in
E_y :	=	1.00 in
HEIGHT TO CG.: $H_{C.G.}$	=	30.00 in
CNTR OF AREA OF WINDWARD FACE: H_w	=	35.00 in
MAX SPRING REACTION	=	300.00 lbs
NO. OF ISOLATORS (N_i)	=	4.00

ISOLATOR FORCES

MOMENT OF INERTIA ABOUT X AXIS: I_x	=	1296.00 in ⁴
MOMENT OF INERTIA ABOUT Z AXIS: I_z	=	2093.06 in ⁴
γ (DIR. OF Vrot-CRIT. ANGLE): $\tan^{-1}(B/L)$	=	38.20
α (DIR. OF Vdir-CRIT. ANGLE): $\tan^{-1}(E_y/E_x)$	=	26.57
β : $\gamma - \alpha$	=	168.37
θ (DIR. OF F_{ph} FOR MAX UPLIFT): $\tan^{-1}((L * I_x) / (B * I_y))$	=	38.20

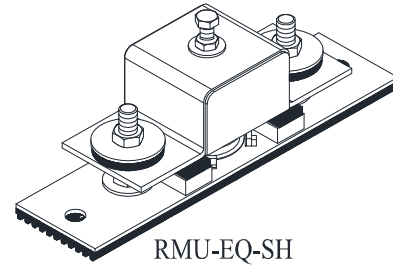
FROM SEISMIC LOADS (LOAD APPLIED TO CRITICAL ANGLE)

VERTICAL REACTIONS (WITH 1.2D OR 0.9D)

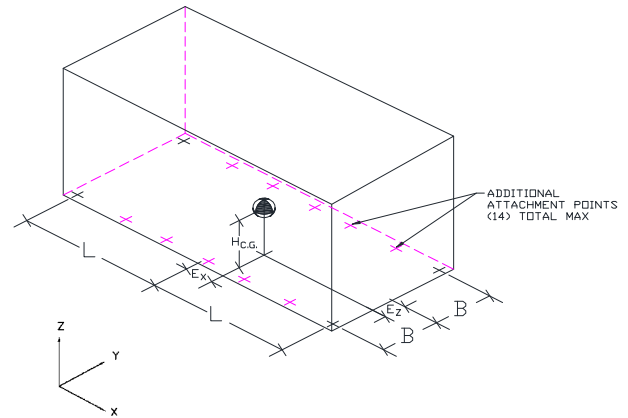
R_m (DUE TO OVERTURNING MOMENT)	=	1926.60 lbs
R_e MAX (DUE TO ECCENTRICITY):	=	52.30 lbs
R_e MIN (DUE TO ECCENTRICITY):	=	19.02 lbs
R_v MAX (DUE TO VERTICAL LOADS):	=	365.75 lbs
R_v MIN (DUE TO VERTICAL LOADS):	=	133.00 lbs
P_{MAX} (MAX DOWNWARD REACTION): $R_m + R_e MAX + R_v MAX$	=	2344.65 lbs
P_{MIN} (MAX UPLIFT/ANCH. POINT - IF POS.): $R_m + R_e MIN - R_v MIN$	=	1812.62 lbs

HORIZONTAL REACTIONS:

V_{rot} (SHEAR DUE TO ECCENTRICITY):	=	69.79 lbs
V_{dir} (DIRECT SHEAR):	=	908.44 lbs
V_{MAX} (TOTAL SHEAR/ISOLATOR):	=	976.89 lbs



9084 lbft



FROM WIND LOADS (LOAD PERPENDICULAR TO UNIT LENGTH)

VERTICAL REACTIONS (WITH 1.2D OR 0.9D)

R_m (DUE TO OVERTURNING MOMENT)	=	427.66 lbs
R_e MAX (DUE TO ECCENTRICITY):	=	52.30 lbs
R_e MIN (DUE TO ECCENTRICITY):	=	40.75 lbs
R_v MAX (DUE TO VERTICAL LOADS):	=	285.00 lbs
R_v MIN (DUE TO VERTICAL LOADS):	=	143.01 lbs
P_{MAX} (MAX DOWNWARD REACTION): $R_m + R_e MAX +$	=	764.95 lbs
P_{MIN} : $R_m + R_e MIN - R_v MIN$ (0 IF NEGATIVE)	=	325.40 lbs

HORIZONTAL REACTIONS:

$V_{MAX} = V_{dir}$	=	219.94 lbs
---------------------	---	------------

SELECT ISOLATOR → RMUH-EQ-SH-2

SEISMIC LOADS GOVERN THE DESIGN OF THE ISOLATOR!!

ISOLATOR CAPACITIES: V_C	=	3000.00	T_C (ADJUSTED) =	3600.00
$V_{MAX} / V_C + P_{MIN} / T_C$	=	0.83	\leq	1.00

LAG BOLT FORCES

SEISMIC LOADS GOVERN THE DESIGN OF THE BOLTS!!

P_{MIN} (MAX UPLIFT/ANCH. POINT - IF POS.): $\Omega_0 R_m + R_e MIN - R_v MIN$	=	1812.62 lbs
V_{MAX} (TOTAL SHEAR/ANCH. POINT):	=	976.89 lbs
# OF BOLTS: n	=	2
BOLT TENSION DUE TO UPLIFT: T_1	=	906.31 lbs
BOLT TENSION DUE TO ISO. OVERTURNING: T_2	=	341.91 lbs
COMBINED BOLT TENSION: $T_u = (T_1 + T_2)$	=	1248.22 lbs
BOLT SHEAR: $V_u = V_{MAX} / n$	=	488.45 lbs

LAG BOLT DIAMETER: = 5/8
LAG BOLT INTO DOUGLAS FIR WOOD. SEE THE FOLLOWING CALCULATIONS FOR DESIGN AND COMBINED LOAD CHECK PER CHAPTER 12 OF THE 2018 NDS.

JOB NAME: FIRE STATION 46	M.W. SAUSSE' & CO., INC.
CUST.:	PREPARED BY : PDG
MECH. ENG.: KHALIFEH	DATE : 06-May-25
MARK: OU-1, 2	SHEET NO. : P19461-S6104-C1.1

CALCULATIONS FOR VIBREX TYPE RMU-EQ-SH VIBRATION ISOLATORS/SEISMIC RESTRAINTS - 2021 IBC/2022 CBC, L

CALCULATES WORST CASE LOADING FOR (4) to (14) ISOLATORS. (UNITS: LBS & INCHES)

USING COMBINED LOAD EQUATIONS 16-5: $U = 1.2D + 1.0E$ & 16-7: $U = 0.9D + 1.0E$

SEISMIC/WIND FORCES - CH. 13, 26, & 29 OF ASCE 7

13.3-2: $F_p = 1.6 \cdot S_{DS} \cdot I_p \cdot W_p$ (MAX)	=	7727.52 lbs
13.3-1: $F_p = (0.4 \cdot a_p \cdot S_{DS} / (R_p / I_p)) \cdot (1 + 2z/h_r) \cdot W_p$	=	7244.55 lbs
13.3-3: $F_p = 0.3 \cdot S_{DS} \cdot I_p \cdot W_p$ (MIN)	=	1448.01 lbs
CONTROLLING HOR. SEISMIC FORCE: F_p	=	7244.55 lbs
OVERTURNING MOMENT: $M = F_p \cdot H_{C.G.}$	=	152135.55 lb-in
* SUPP. VERT. SEISM. FORCE: $F_{pv} = 0.2 S_{DS} W_p$	=	643.96 lbs
DESIGN SPECTRAL RESP. ACCELERATION: S_{DS}	=	1.700
IMPORTANCE FACTOR: I_p	=	1.50
COMPONENT AMPLIFICATION FACTOR: a_p	=	2.50
COMP. RESPONSE MODIFICATION FACTOR: R_p	=	2.00
EQUIPMENT ELEVATION / ROOF ELEVATION: z/h_r	=	1.00

(WITH RESPECT TO GRADE)

UNIT LOCATED INDOORS OR OUTDOORS	=	OUTDOORS
F_h FROM PREVIOUS CALCULATION (IF APPLICABLE)	=	286.73 lbs
F_v FROM PREVIOUS CALCULATION (IF APPLICABLE)	=	197.56 lbs
WIND OVERTURNING MOMENT: $M_w = F_p \cdot H_w$	=	6308.13 lb-in

EQUIPMENT WEIGHT & GEOMETRY

EQUIPMENT WEIGHT: W_p (UNIT + RAIL)	=	1894.00 lbs
L:	=	51.50 in
B:	=	32.25 in
E_x :	=	2.00 in
E_y :	=	1.00 in
HEIGHT TO CG.: $H_{C.G.}$	=	21.00 in
CNTR OF AREA OF WINDWARD FACE: H_w	=	22.00 in
MAX SPRING REACTION	=	450.00 lbs
NO. OF ISOLATORS (N_i)	=	6.00

ISOLATOR FORCES

MOMENT OF INERTIA ABOUT X AXIS: I_x	=	6240.38 in ⁴
MOMENT OF INERTIA ABOUT Z AXIS: I_z	=	15913.50 in ⁴
γ (DIR. OF Vrot-CRIT. ANGLE): $\tan^{-1}(B/L)$	=	32.06
α (DIR. OF Vdir-CRIT. ANGLE): $\tan^{-1}(E_y/E_x)$	=	26.57
β : $\gamma - \alpha$	=	174.51
θ (DIR. OF F_{ph} FOR MAX UPLIFT): $\tan^{-1}((L \cdot I_x)/(B \cdot I_y))$	=	32.06

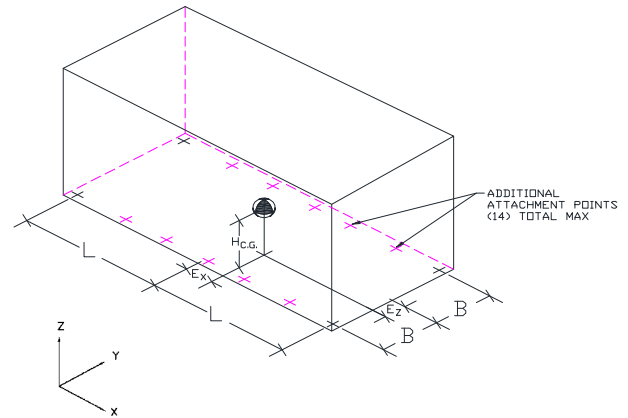
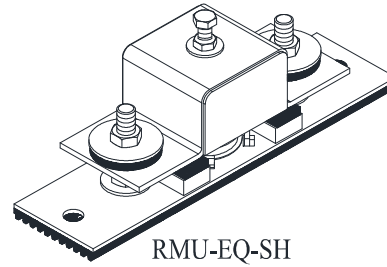
FROM SEISMIC LOADS (LOAD APPLIED TO CRITICAL ANGLE)

VERTICAL REACTIONS (WITH 1.2D OR 0.9D)

R_m (DUE TO OVERTURNING MOMENT)	=	927.67 lbs
R_e MAX (DUE TO ECCENTRICITY):	=	33.95 lbs
R_e MIN (DUE TO ECCENTRICITY):	=	12.35 lbs
R_v MAX (DUE TO VERTICAL LOADS):	=	486.13 lbs
R_v MIN (DUE TO VERTICAL LOADS):	=	176.77 lbs
P_{MAX} (MAX DOWNWARD REACTION): $R_m + R_e MAX + R_v MAX$	=	1447.75 lbs
P_{MIN} (MAX UPLIFT/ANCH. POINT - IF POS.): $R_m + R_e MIN + R_v MIN$	=	763.24 lbs

HORIZONTAL REACTIONS:

V_{rot} (SHEAR DUE TO ECCENTRICITY):	=	66.65 lbs
V_{dir} (DIRECT SHEAR):	=	1207.43 lbs
V_{MAX} (TOTAL SHEAR/ISOLATOR):	=	1273.78 lbs



FROM WIND LOADS (LOAD PERPENDICULAR TO UNIT LENGTH)

VERTICAL REACTIONS (WITH 1.2D OR 0.9D)

R_m (DUE TO OVERTURNING MOMENT)	=	32.60 lbs
R_e MAX (DUE TO ECCENTRICITY):	=	33.95 lbs
R_e MIN (DUE TO ECCENTRICITY):	=	26.46 lbs
R_v MAX (DUE TO VERTICAL LOADS):	=	378.80 lbs
R_v MIN (DUE TO VERTICAL LOADS):	=	251.17 lbs
P_{MAX} (MAX DOWNWARD REACTION): $R_m + R_e MAX +$	=	445.35 lbs
P_{MIN} : $R_m + R_e MIN - R_v MIN$ (0 IF NEGATIVE)	=	0.00 lbs

HORIZONTAL REACTIONS:

$V_{MAX} = V_{dir}$	=	47.79 lbs
---------------------	---	-----------

SELECT ISOLATOR → RMU-EQ-SH-2

SEISMIC LOADS GOVERN THE DESIGN OF THE ISOLATOR!!

ISOLATOR CAPACITIES: V_C	=	2200.00	T_C (ADJUSTED) =	2370.00
$V_{MAX}/V_C + P_{MIN}/T_C$	=	0.90	\leq	1.00

LAG BOLT FORCES

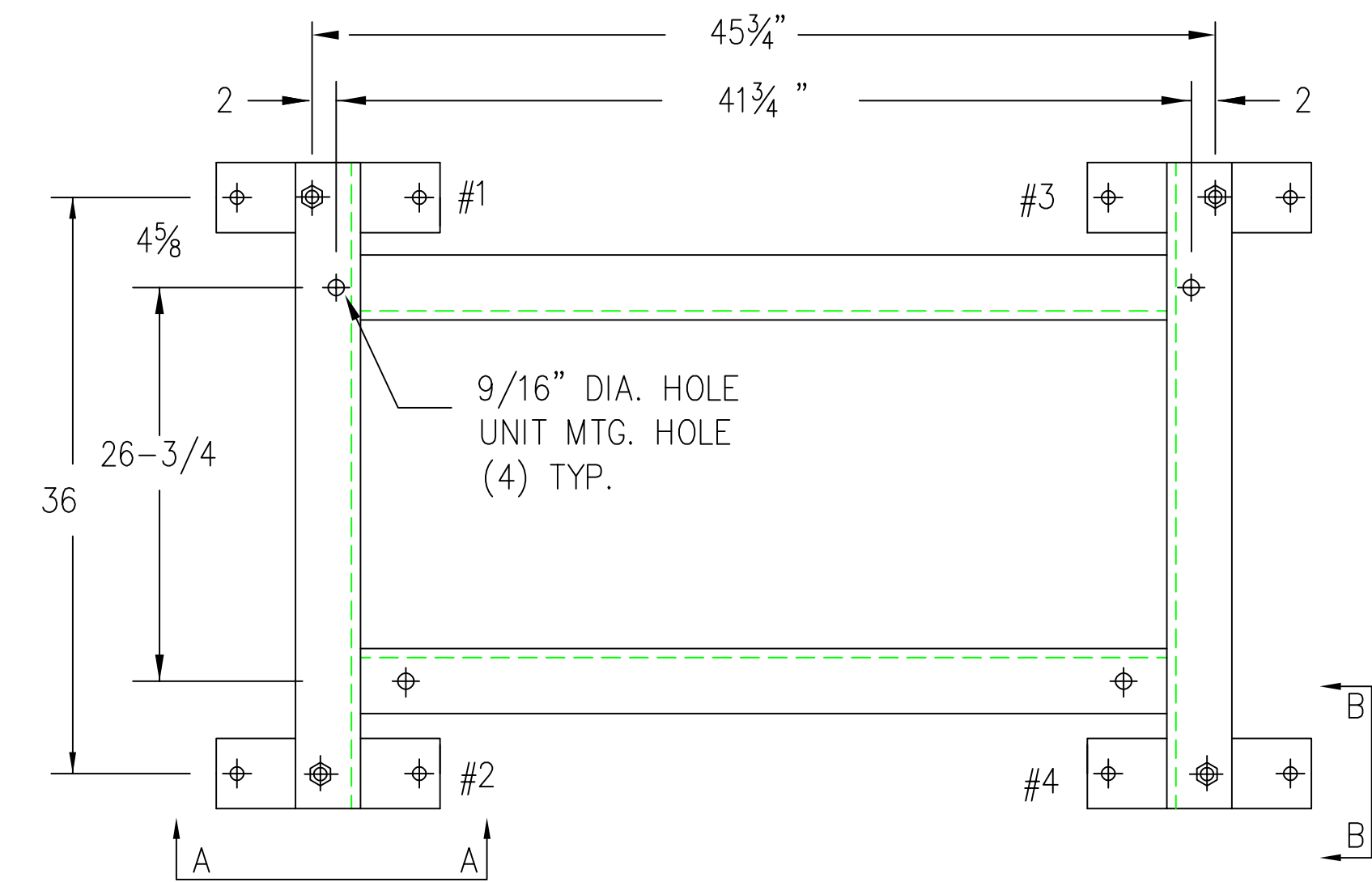
SEISMIC LOADS GOVERN THE DESIGN OF THE BOLTS!!

P_{MIN} (MAX UPLIFT/ANCH. POINT - IF POS.): $\Omega_0 R_m + R_e MIN - R_v MIN$	=	763.24 lbs
V_{MAX} (TOTAL SHEAR/ANCH. POINT):	=	1273.78 lbs
# OF BOLTS: n	=	2
BOLT TENSION DUE TO UPLIFT: T_1	=	381.62 lbs
BOLT TENSION DUE TO ISO. OVERTURNING: T_2	=	445.82 lbs
COMBINED BOLT TENSION: $T_u = (T_1 + T_2)$	=	827.44 lbs
BOLT SHEAR: $V_u = V_{MAX}/n$	=	636.89 lbs

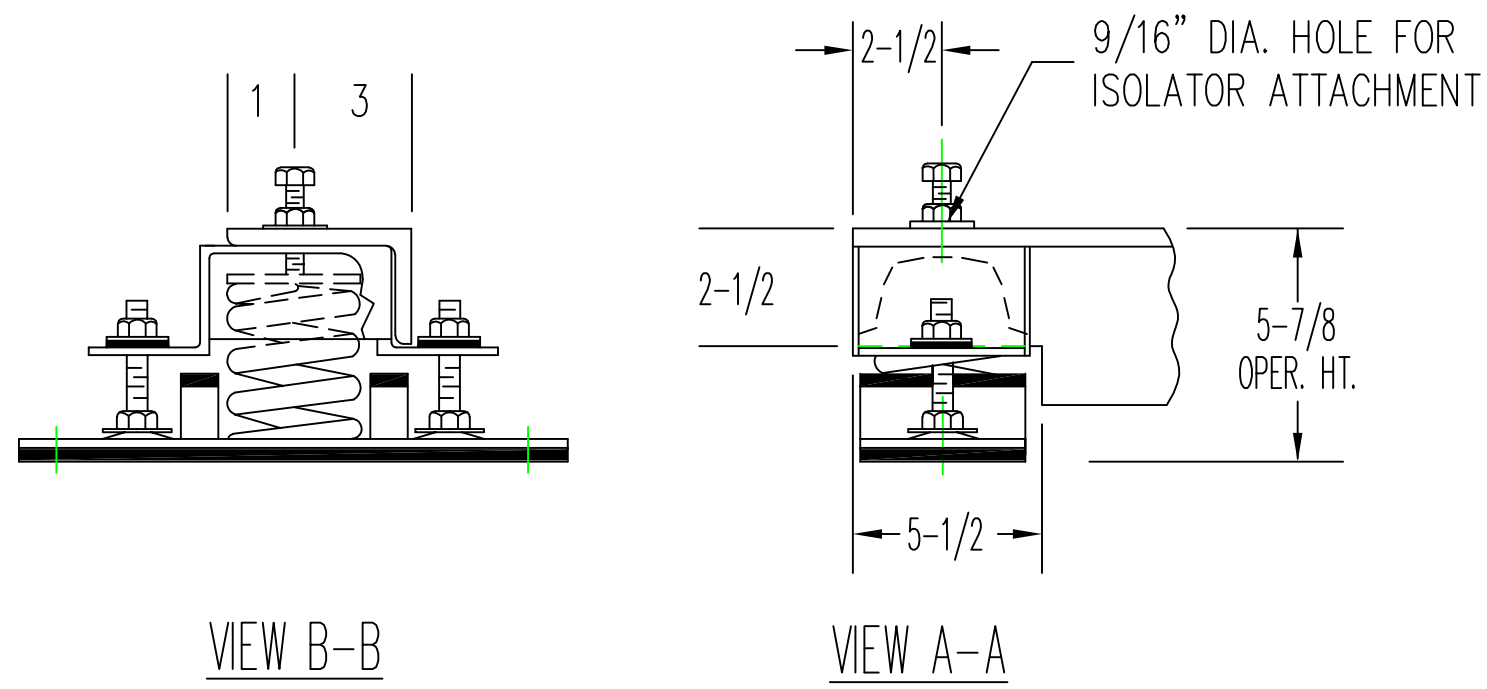
LAG BOLT DIAMETER: = 5/8
LAG BOLT INTO DOUGLAS FIR WOOD. SEE THE FOLLOWING CALCULATIONS FOR DESIGN AND COMBINED LOAD CHECK PER CHAPTER 12 OF THE 2018 NDS.

JOB NAME: FIRE STATION 46	M.W. SAUSSE' & CO., INC.
CUST.:	PREPARED BY : TDT
MECH. ENG.: KHALIFEH	DATE : 15-May-25
MARK: MUA-1	SHEET NO. : -C3

MARK	MAKE	MODEL	STEEL SIZE
OU-1	MITSUBISHI	PURY-ED144	L4 X 3 X 1/4
OU-2	MITSUBISHI	PURY-EP120	

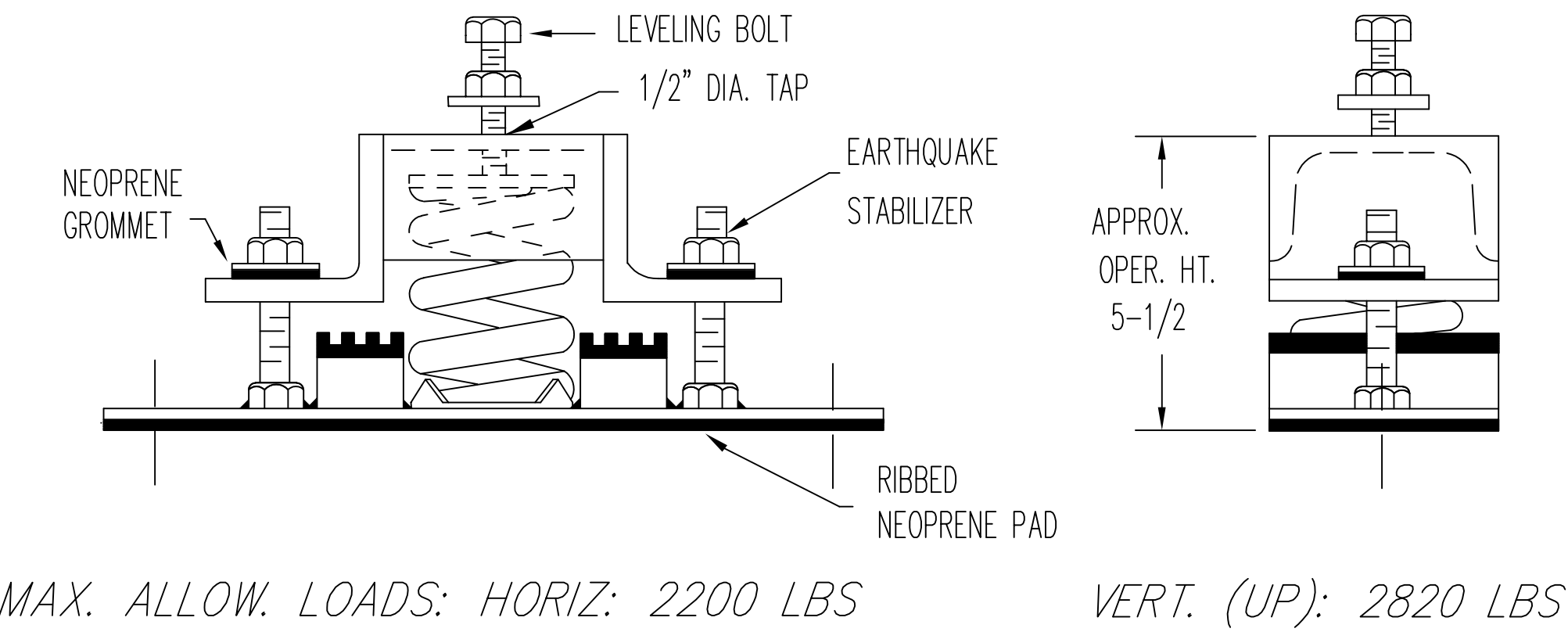
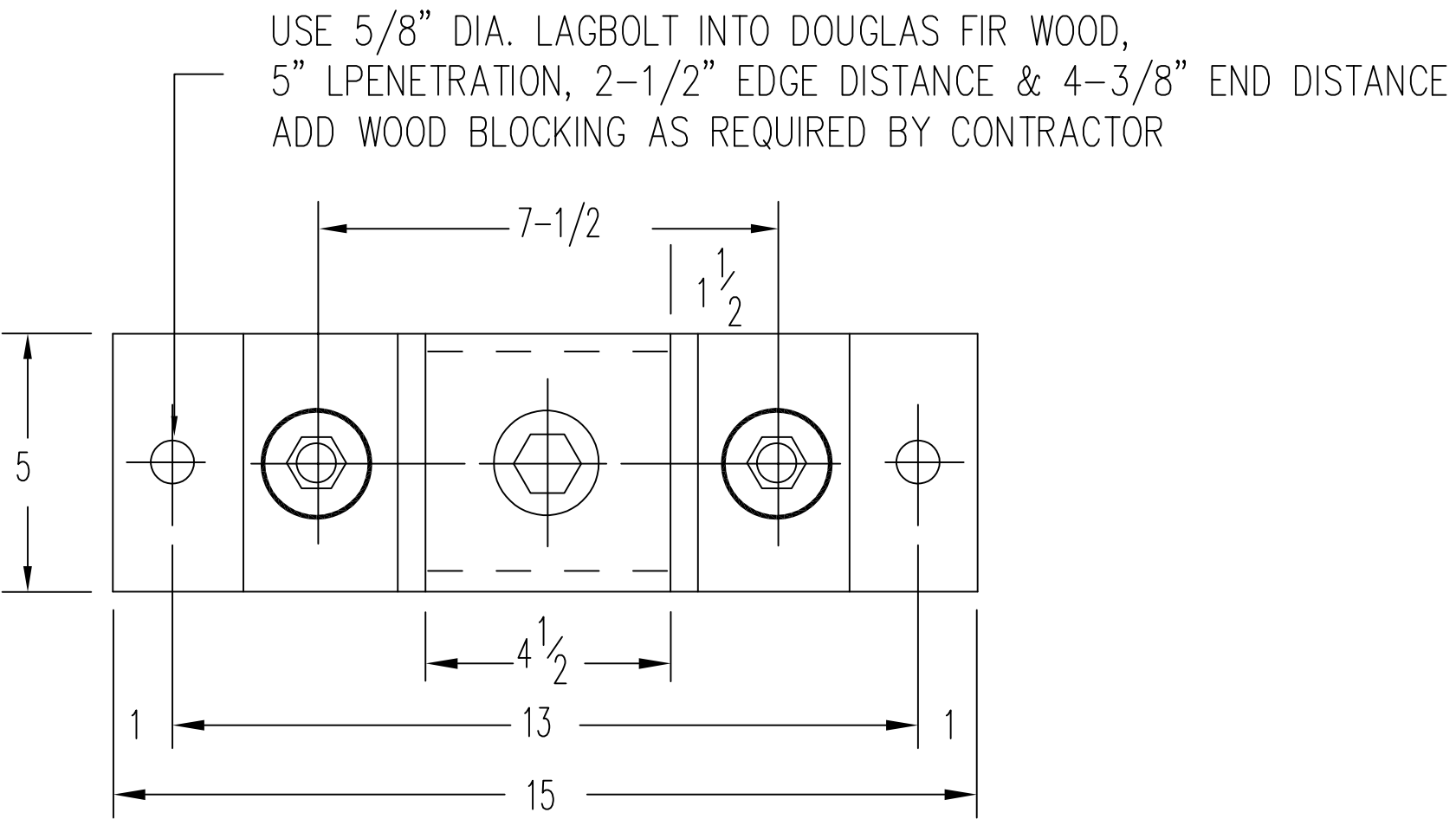


- NOTES:
1. APPROXIMATE STEEL WEIGHT & ISOLATORS = 250 LBS
 2. FOR ISOLATOR DETAILS, SEE DWG-1.1.
 3. M.W. SAUSSE & CO. INC. IS NOT RESPONSIBLE FOR THE STRUCTURAL INTEGRITY OF THE EQUIPMENT WHEN ANCHORED AS SHOWN.
 4. ALL DIMENSIONS TO BE FIELD VERIFIED BY CONTRACTOR BEFORE CONSTRUCTION.



DETAIL-1

DETAIL-3



M. W. SAUSSE & CO., INC. 28744 Whitherspoon Pkwy. Valencia, CA 91355 Phone: (661) 257-3311 Fax: (661) 257-7673  	JOB NAME: FIRE STATION 46	REVISIONS:	DRN: TDT
	CUST.:	A:	DATE: 5-6-25
	CUST. P.O.:	B:	DRAWING NO.:
	MECH. ENGR.: KHALIFEH	C:	-1
	MARK: OU-1,2	D:	

CALCULATIONS FOR RIGID ANCHORAGE - WIND LOADING (ASCE 7-16)

CALCULATE HORIZONTAL WIND FORCE (F IN LBS., DIMS. IN FT):

EXPOSURE CATEGORY: C

BUILDING HEIGHT (h_f): 35

FIND VELOCITY PRESSURE (q_h) = $0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2$ (LBS/FT²)

VELOCITY EXPOSURE FACTOR (K_z) (TABLE 29.3-1): 1.01

OPOGRAPHIC FACTOR $K_{zt} = (1 + K_1 K_2 K_3)^2$ (EQ. 26.8-1)

K_1, K_2 AND K_3 FROM (FIGURE 26.8-1):

IF GROUND IS FLAT ENTER
THE VALUE OF ZERO FOR
ALL

$K_1 =$	0
$K_2 =$	0
$K_3 =$	0
$K_{zt} =$	1

K_d (DIRECTIONALITY FACTOR, TABLE 26.10-1) = 0.85

OCCUPANCY/RISK CATEGORY = IV

*(FOR WIND SPEED MAP SELECTION - SEE FIG. 26.5-1A TO 1C)

WIND VELOCITY (V IN MPH) = 102

VELOCITY PRESSURE (q_h) = 22.9

HORIZONTAL AND VERTICAL WIND LOADS FROM SECTION 29.5.1

HORIZONTAL LOAD:

(GC_r) = 1.9 (REDUCEABLE, SEE SECTION 29.4.1)

A_r = 20.25 ft² (VERTICAL FACE OF EQUIPMENT NORMAL TO HORIZ. WIND PRESSURE)

(29.4-2) $F_h = q_h(GC_r)A_r = 879.749922$ lbs

VERTICAL LOAD:

(GC_r) = 1.5 (REDUCEABLE, SEE SECTION 29.4.1)

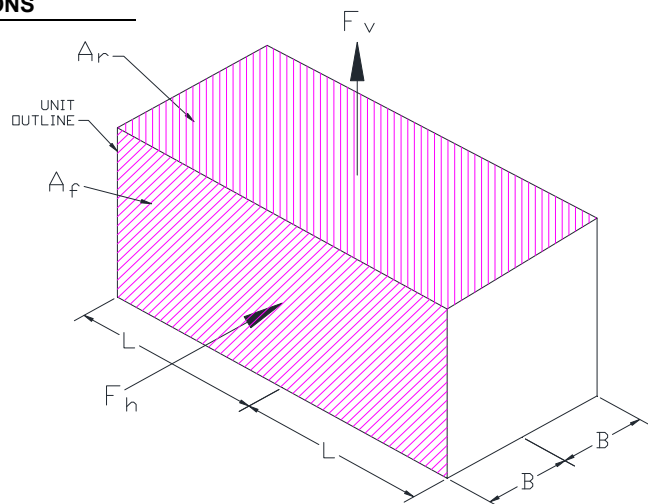
A_r = 8.25 ft² (HORIZONTAL FACE OF EQUIPMENT NORMAL TO VERT. WIND PRESSURE)

(29.4-3) $F_v = q_h(GC_r)A_r = 282.960501$ lbs

DESIGN LOADS FOR ANCHORAGE CALCULATIONS

$F_h = 879.7499$ lbs

$F_v = 282.9605$ lbs



JOB NAME: FIRE STATION 46	M. W. SAUSSE' & CO., INC.	
CUST.:	PREPARED BY:	TDT
MECH. ENG.: KHALIFEH	DATE :	5-6-25
MARK: OU-1, 2	SHEET NO.:	-C1.1

Wood Beam

Project File: FS 46 enercalc.ec6

LIC#: KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Beam at OU unit at Flat Roof L= 16 ft

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

Material Properties

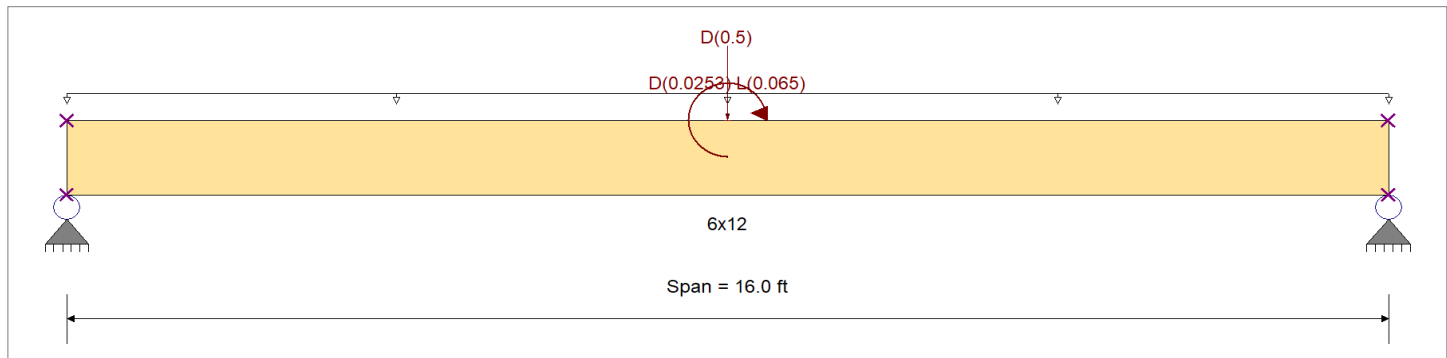
Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
 Wood Grade : No.1

Beam Bracing : Completely Unbraced

Fb + 1,350.0 psi
 Fb - 1,350.0 psi
 Fc - Prll 925.0 psi
 Fc - Perp 625.0 psi
 Fv 170.0 psi
 Ft 675.0 psi

E : Modulus of Elasticity
 Ebend- xx 1,600.0 ksi
 Eminbend - xx 580.0 ksi
 Density 31.210 pcf
 Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.02530, L = 0.0650, Tributary Width = 1.0 ft

Point Load : D = 0.50 k @ 8.0 ft

Moment : E = 4.50 k-ft, Location = 8.0 ft from left end of this span

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.347	1	Section used for this span	=	0.137	1
fb: Actual	=	527.42	psi	fv: Actual	=	23.36	psi
F'b	=	1,520.91	psi	F'v	=	170.00	psi
Load Combination	=	+D+L		Load Combination	=	+D+L	
Location of maximum on span	=	8.000	ft	Location of maximum on span	=	15.066	ft
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.086 in	Ratio =	2221 >= 360	Span: 1 : L Only			
Max Upward Transient Deflection	-0.014 in	Ratio =	13289 >= 360	Span: 1 : E Only			
Max Downward Total Deflection	0.205 in	Ratio =	937 >= 240	Span: 1 : +D+L			
Max Upward Total Deflection	-0.000 in	Ratio =	562070 >= 240	Span: 1 : +0.60D+0.70E			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0			
Length = 16.0 ft	1	0.234	0.081	0.90	1.00	1.00	0.98	1.000	1.00	1.00	1.15	3.25	321.5	1,372.5	0.53	12.5	153.0
+D+L														0.0			
Length = 16.0 ft	1	0.347	0.137	1.00	1.00	1.00	0.98	1.000	1.00	1.00	1.15	5.33	527.4	1,520.9	0.98	23.4	170.0
+D+0.750L														0.0			
Length = 16.0 ft	1	0.252	0.097	1.25	1.00	1.00	0.97	1.000	1.00	1.00	1.15	4.81	476.0	1,886.6	0.87	20.6	212.5
+0.60D														0.0			
Length = 16.0 ft	1	0.081	0.027	1.60	1.00	1.00	0.96	1.000	1.00	1.00	1.15	1.95	192.9	2,382.6	0.32	7.5	272.0
+D+0.70E														0.0			
														0.0			

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Beam at OU unit at Flat Roof L= 16 ft

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 16.0 ft	1		0.200	0.063	1.60	1.00	1.00	0.96	1.000	1.00	1.00	1.15	4.82	477.4	2,382.6	0.72	17.1	272.0
+D+0.750L+0.5250E							1.00	1.00	0.96	1.000	1.00	1.00	1.15		0.0	0.00	0.0	0.0
Length = 16.0 ft	1		0.249	0.089	1.60	1.00	1.00	0.96	1.000	1.00	1.00	1.15	5.99	592.9	2,382.6	1.02	24.1	272.0
+0.60D+0.70E							1.00	1.00	0.96	1.000	1.00	1.00	1.15		0.0	0.00	0.0	0.0
Length = 16.0 ft	1		0.146	0.045	1.60	1.00	1.00	0.96	1.000	1.00	1.00	1.15	3.52	348.8	2,382.6	0.51	12.1	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.2048	8.058		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.082	1.100
Max Upward from Load Combinations	1.082	1.100
Max Upward from Load Cases	0.562	0.562
Max Downward from all Load Conditio	-0.281	
Max Downward from Load Cases (Resis	-0.281	
D Only	0.562	0.562
+D+L	1.082	1.082
+D+0.750L	0.952	0.952
+0.60D	0.337	0.337
+D+0.70E	0.365	0.759
+D+0.750L+0.5250E	0.804	1.100
+0.60D+0.70E	0.140	0.534
L Only	0.520	0.520
E Only	-0.281	0.281

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Typical Wood Beam at Flat Roof _ Mua L= 12 ft Max @ 48"

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

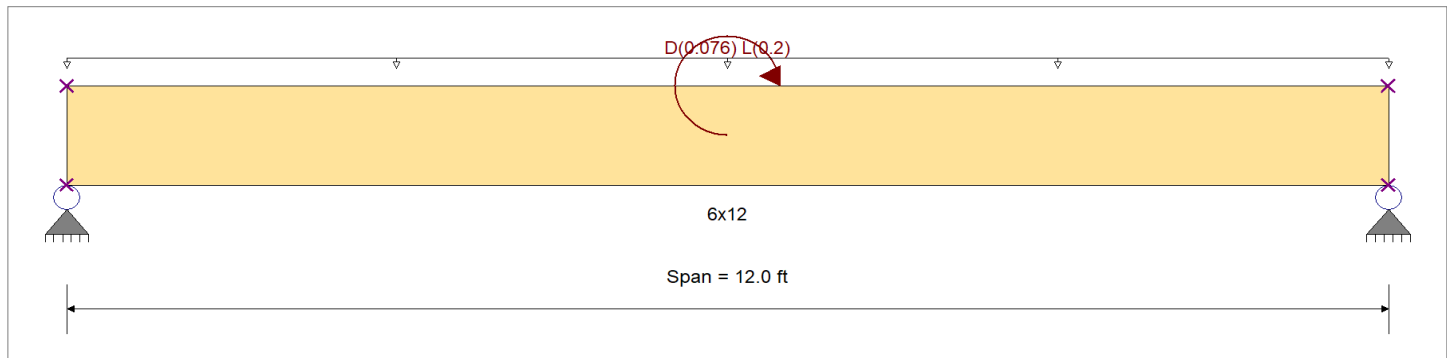
Material Properties

Analysis Method : Allowable Stress Design
 Load Combination : ASCE 7-16

Wood Species : Douglas Fir-Larch
 Wood Grade : No.1

Beam Bracing : Completely Unbraced

Fb +	1350 psi	E : Modulus of Elasticity	
Fb -	1350 psi	Ebend- xx	1600ksi
Fc - Prll	925 psi	Eminbend - xx	580ksi
Fc - Perp	625 psi		
Fv	170 psi		
Ft	675 psi	Density	31.21 pcf
		Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.0760, L = 0.20 , Tributary Width = 1.0 ft

Moment : E = 5.90 k-ft, Location = 6.0 ft from left end of this span

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.321	1	Section used for this span	=	0.196	1
fb: Actual	=	491.76psi		fv: Actual	=	33.25 psi	
F'b	=	1,530.28psi		F'v	=	170.00 psi	
Load Combination	=	+D+L		Load Combination	=	+D+L	
Location of maximum on span	=	6.000ft		Location of maximum on span	=	0.000ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.084 in	Ratio =	1711 >=360	Span: 1 : L Only			
Max Upward Transient Deflection	-0.011 in	Ratio =	13514 >=360	Span: 1 : E Only			
Max Downward Total Deflection	0.116 in	Ratio =	1239 >=240	Span: 1 : +D+L			
Max Upward Total Deflection	-0.001 in	Ratio =	233726 >=240	Span: 1 : +0.60D+0.70E			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 12.0 ft	1	0.098	0.060	0.90	1.00	1.00	0.99	1.000	1.00	1.00	1.15	1.37	135.4	1,379.7	0.39	9.2	153.0
+D+L					1.00	1.00	0.99	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 12.0 ft	1	0.321	0.196	1.00	1.00	1.00	0.99	1.000	1.00	1.00	1.15	4.97	491.8	1,530.3	1.40	33.3	170.0
+D+0.750L					1.00	1.00	0.99	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 12.0 ft	1	0.212	0.128	1.25	1.00	1.00	0.98	1.000	1.00	1.00	1.15	4.07	402.7	1,903.6	1.15	27.2	212.5
+0.60D					1.00	1.00	0.98	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 12.0 ft	1	0.034	0.020	1.60	1.00	1.00	0.97	1.000	1.00	1.00	1.15	0.82	81.2	2,417.3	0.23	5.5	272.0
+D+0.70E					1.00	1.00	0.97	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Typical Wood Beam at Flat Roof _ Mua L= 12 ft Max @ 48"

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+0.750L+0.5250E	Length = 12.0 ft	1	0.141	0.064	1.60	1.00	1.00	0.97	1.000	1.00	1.00	1.15	3.43	339.8	2,417.3	0.73	17.3	272.0
							1.00	1.00	0.97	1.000	1.00	1.00	1.15		0.0		0.00	0.0
+0.60D+0.70E	Length = 12.0 ft	1	0.230	0.123	1.60	1.00	1.00	0.97	1.000	1.00	1.00	1.15	5.62	556.0	2,417.3	1.41	33.4	272.0
							1.00	1.00	0.97	1.000	1.00	1.00	1.15		0.0		0.00	0.0
	Length = 12.0 ft	1	0.118	0.050	1.60	1.00	1.00	0.97	1.000	1.00	1.00	1.15	2.89	285.7	2,417.3	0.58	13.7	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.1161	6.044		0.0000	0.000

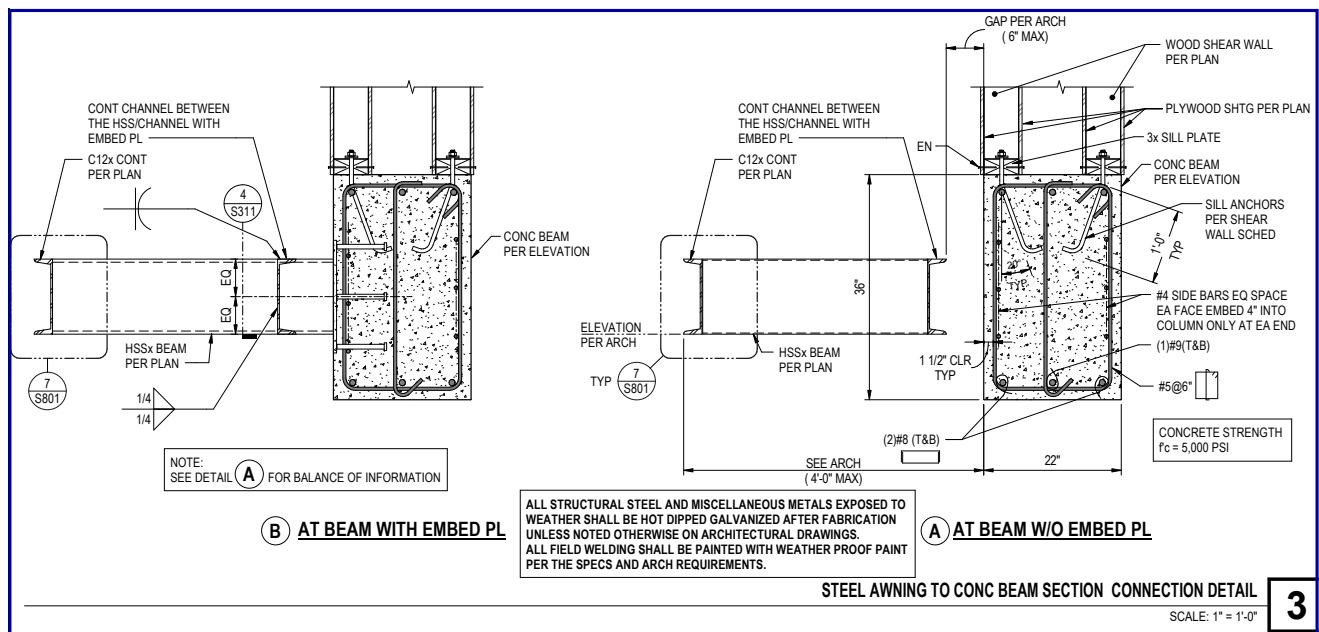
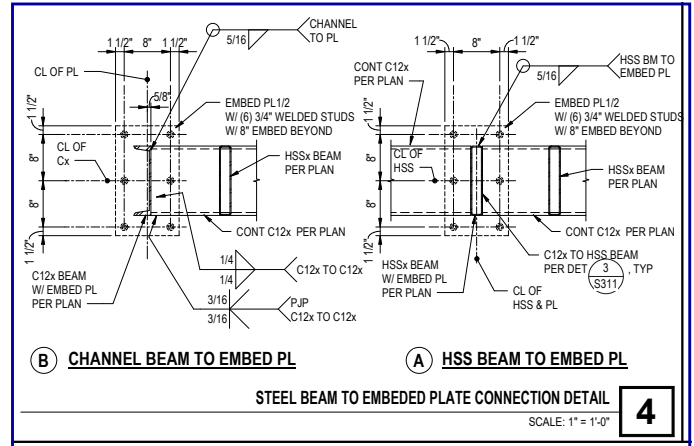
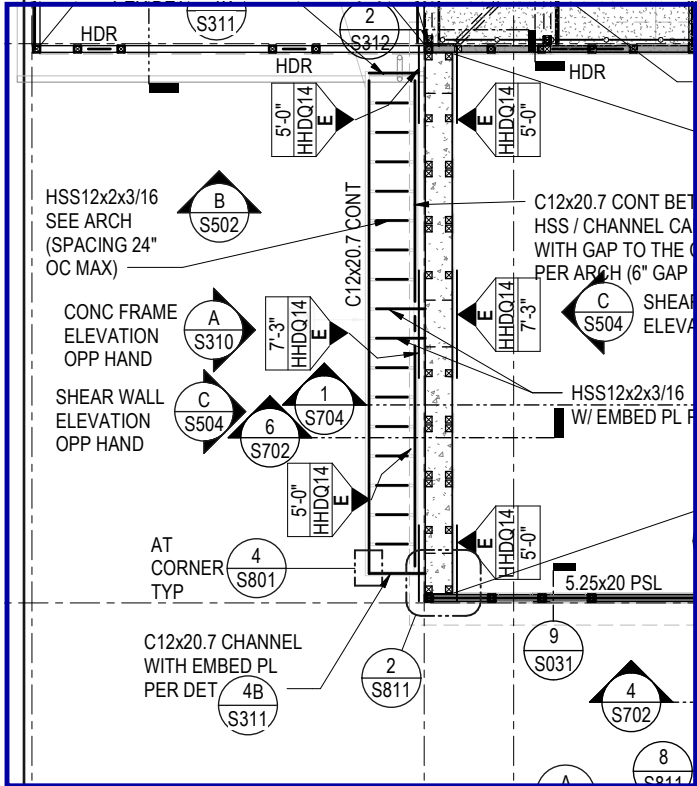
Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.656	1.656
Max Upward from Load Combinations	1.656	1.656
Max Upward from Load Cases	1.200	1.200
Max Downward from all Load Conditio	-0.492	
Max Downward from Load Combinations	-0.071	
Max Downward from Load Cases (Resis	-0.492	
D Only	0.456	0.456
+D+L	1.656	1.656
+D+0.750L	1.356	1.356
+0.60D	0.274	0.274
+D+0.70E	0.112	0.800
+D+0.750L+0.5250E	1.098	1.614
+0.60D+0.70E	-0.071	0.618
L Only	1.200	1.200
E Only	-0.492	0.492

4.8 AWNING



HSS 12x2 @ 24" oc. 28 lb/2 ft = 14 psf

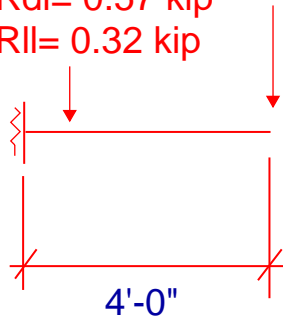
C12x 30 lb/4 ft = 7.5 psf

Total DL = 21.5 psf

DL = 25 psf

LL = 20 psf

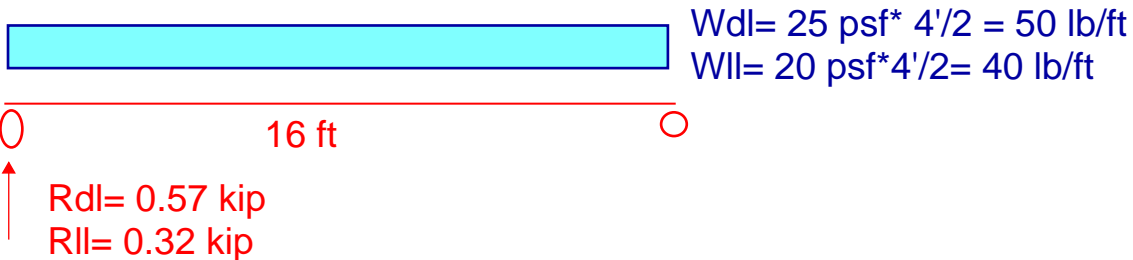
R_{dl} = 0.57 kip
R_{ll} = 0.32 kip



P_{dl} = 25 psf * 4'/2 * 16 ft/2 = 400 lb

P_{ll} = 20 psf * 4'/2 * 16 ft/2 = 320 lb

R_u = 1.2 * (0.57 + 0.4) kip + 1.6 * (0.32 + 0.32) kip = 2.4 kip
M_u = 0.68 * (4 + 0.5) ft + 0.5 kip * (4 + 0.5) ft = 3 kipft + 2.25 kipft = 5.5 kipft = 66000 lb"



W_{dl} = 25 psf * 4'/2 = 50 lb/ft

W_{ll} = 20 psf * 4'/2 = 40 lb/ft

HSS 12x2 @ 24" oc. 28 lb/2 ft= 14 psf

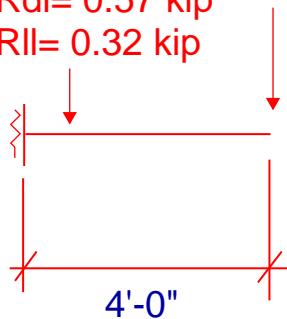
C12x 30 lb/4 ft = 7.5 psf

Total DL= 21.5 psf

DL= 25 psf

LL= 20 psf

$R_{dl} = 0.57 \text{ kip}$
 $R_{ll} = 0.32 \text{ kip}$



$P_{dl} = 25 \text{ psf} \times 4' / 2 \times 16 \text{ ft} / 2 = 400 \text{ lb}$
 $P_{ll} = 20 \text{ psf} \times 4' / 2 \times 16 \text{ ft} / 2 = 320 \text{ lb}$

$R_u = 1.2 \times (0.57 + 0.4) \text{ kip} + 1.6 \times (0.32 + 0.32) \text{ kip} = 2.4 \text{ kip}$
 $M_u = 0.68 \times (4 + 0.5) \text{ ft} + 0.5 \text{ kip} \times (4 + 0.5) \text{ ft} = 3 \text{ kipft} + 2.25 \text{ kipft} = 5.5 \text{ kipft} = 66000 \text{ lb"}\mathbf{}$



$W_{dl} = 25 \text{ psf} \times 4' / 2 = 50 \text{ lb/ft}$
 $W_{ll} = 20 \text{ psf} \times 4' / 2 = 40 \text{ lb/ft}$

Steel Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Awning Channel Cant

CODE REFERENCES

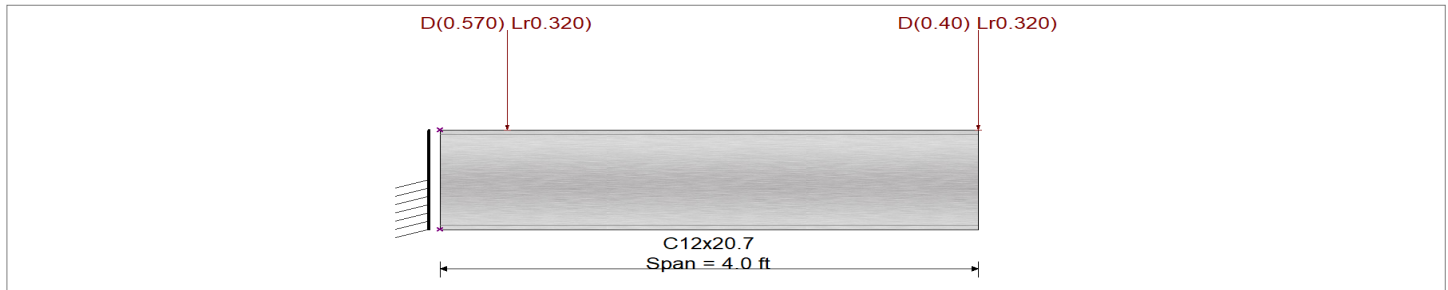
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 36.0 ksi
 E : Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading

Load(s) for Span Number 1

Point Load : D = 0.40, Lr = 0.320 k @ 4.0 ft

Point Load : D = 0.570, Lr = 0.320 k @ 0.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.078 : 1	Maximum Shear Stress Ratio =		0.039 : 1
Section used for this span		C12x20.7	Section used for this span		C12x20.7
Ma : Applied		3.491 k-ft	Va : Applied		1.693 k
Mn / Omega : Allowable		44.485 k-ft	Vn/Omega : Allowable		43.769 k
Load Combination		+D+Lr	Load Combination		+D+Lr
Span # where maximum occurs		Span # 1	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		0.003 in	Ratio =	29,838	>=360
Max Upward Transient Deflection		0.000 in	Ratio =	0	<360
					Span: 1 : Lr Only
Max Downward Total Deflection		0.008 in	Ratio =	12662	>=240.
					Span: 1 : +D+Lr
Max Upward Total Deflection		0.000 in	Ratio =	0	<240.0

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	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only														
Dsgn. L = 4.00 ft	1		0.046	0.024		-2.05	2.05	74.29	44.48	1.00	1.00	1.05	73.09	43.77
+D+Lr														
Dsgn. L = 4.00 ft	1		0.078	0.039		-3.49	3.49	74.29	44.48	1.00	1.00	1.69	73.09	43.77
+D+0.750Lr														
Dsgn. L = 4.00 ft	1		0.070	0.035		-3.13	3.13	74.29	44.48	1.00	1.00	1.53	73.09	43.77
+0.60D														
Dsgn. L = 4.00 ft	1		0.028	0.014		-1.23	1.23	74.29	44.48	1.00	1.00	0.63	73.09	43.77

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0076	4.000		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.693	
Max Upward from Load Combinations	1.693	
Max Upward from Load Cases	1.053	
D Only	1.053	

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28 SAIFUL - BOUQUET CONSULTING ENGINEERS (c) ENERCALC INC 1983-2022

DESCRIPTION: Awning Channel Cant

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+Lr	1.693	
+D+0.750Lr	1.533	
+0.60D	0.632	
Lr Only	0.640	

Steel Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Awning HSS 12x2 Cant

CODE REFERENCES

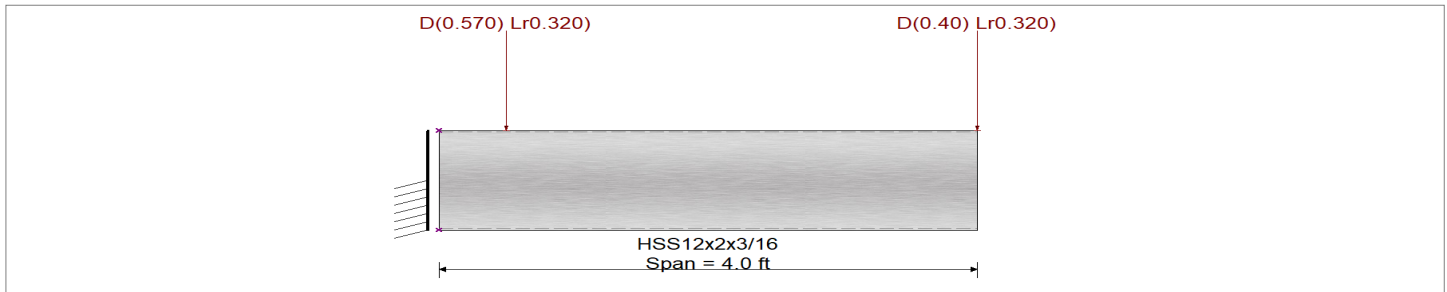
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 46.0 ksi
 E : Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading

Load(s) for Span Number 1

Point Load : D = 0.40, Lr = 0.320 k @ 4.0 ft

Point Load : D = 0.570, Lr = 0.320 k @ 0.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.099 : 1	Maximum Shear Stress Ratio =		0.027 : 1
Section used for this span		HSS12x2x3/16	Section used for this span		HSS12x2x3/16
Ma : Applied		3.462 k-ft	Va : Applied		1.678 k
Mn / Omega : Allowable		34.954 k-ft	Vn/Omega : Allowable		61.804 k
Load Combination		+D+Lr	Load Combination		+D+Lr
Span # where maximum occurs		Span # 1	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		0.006 in	Ratio =	15,590	>=360
Max Upward Transient Deflection		0.000 in	Ratio =	0	<360
Max Downward Total Deflection		0.014 in	Ratio =	6663	>=240.
Max Upward Total Deflection		0.000 in	Ratio =	0	<240.0

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	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 4.00 ft	1	0.058	0.017		-2.02	2.02	58.37	34.95	1.00	1.00	1.04	103.21	61.80
+D+Lr													
Dsgn. L = 4.00 ft	1	0.099	0.027		-3.46	3.46	58.37	34.95	1.00	1.00	1.68	103.21	61.80
+D+0.750Lr													
Dsgn. L = 4.00 ft	1	0.089	0.025		-3.10	3.10	58.37	34.95	1.00	1.00	1.52	103.21	61.80
+0.60D													
Dsgn. L = 4.00 ft	1	0.035	0.010		-1.21	1.21	58.37	34.95	1.00	1.00	0.62	103.21	61.80

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0144	4.000		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.678	
Max Upward from Load Combinations	1.678	
Max Upward from Load Cases	1.038	
D Only	1.038	

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28 SAIFUL - BOUQUET CONSULTING ENGINEERS (c) ENERCALC INC 1983-2022

DESCRIPTION: Awning HSS 12x2 Cant

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+Lr	1.678	
+D+0.750Lr	1.518	
+0.60D	0.623	
Lr Only	0.640	

Steel Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Awning back Channel

CODE REFERENCES

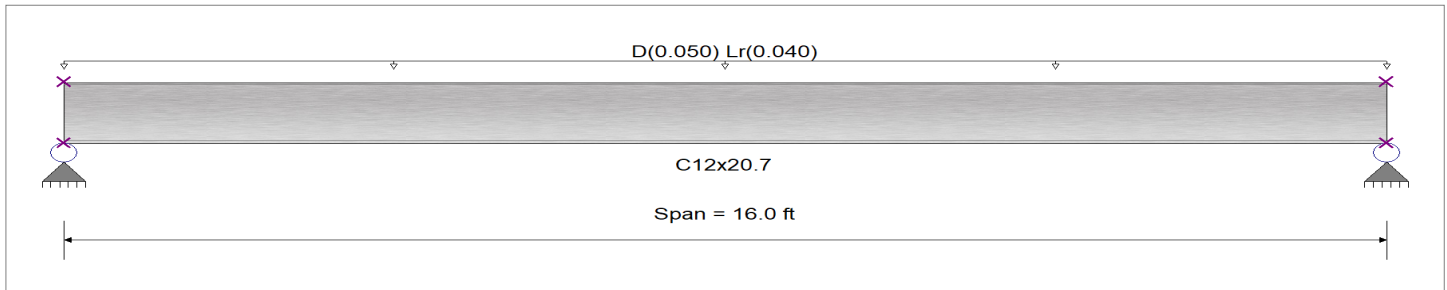
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

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

Fy : Steel Yield : 36.0 ksi
 E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.050, Lr = 0.040 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.166 : 1	Maximum Shear Stress Ratio =		0.020 : 1
Section used for this span		C12x20.7	Section used for this span		C12x20.7
Ma : Applied		3.542 k-ft	Va : Applied		0.8856 k
Mn / Omega : Allowable		21.340 k-ft	Vn/Omega : Allowable		43.769 k
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span		0.000 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		0.016 in	Ratio = 12,122		>=360
Max Upward Transient Deflection		0.000 in	Ratio = 0		<360
Max Downward Total Deflection		0.044 in	Ratio = 4380		>=240.
Max Upward Total Deflection		0.000 in	Ratio = 0		<240.0

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	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 16.00 ft	1	0.106	0.013	2.26		2.26	35.64	21.34	1.14	1.00	0.57	73.09	43.77
+D+Lr													
Dsgn. L = 16.00 ft	1	0.166	0.020	3.54		3.54	35.64	21.34	1.14	1.00	0.89	73.09	43.77
+D+0.750Lr													
Dsgn. L = 16.00 ft	1	0.151	0.018	3.22		3.22	35.64	21.34	1.14	1.00	0.81	73.09	43.77
+0.60D													
Dsgn. L = 16.00 ft	1	0.064	0.008	1.36		1.36	35.64	21.34	1.14	1.00	0.34	73.09	43.77

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0438	8.046		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.886	0.886
Max Upward from Load Combinations	0.886	0.886
Max Upward from Load Cases	0.566	0.566
D Only	0.566	0.566
+D+Lr	0.886	0.886
+D+0.750Lr	0.806	0.806
+0.60D	0.339	0.339

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28 SAIFUL - BOUQUET CONSULTING ENGINEERS (c) ENERCALC INC 1983-2022

DESCRIPTION: Awning back Channel

Vertical Reactions


Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Lr Only	0.320	0.320

www.hilti.com		Page:	1
Company:		Specifier:	
Address:		E-Mail:	
Phone Fax:		Date:	7/30/2025
Design:	Concrete - Jul 29, 2025 (1)		
Fastening point:			

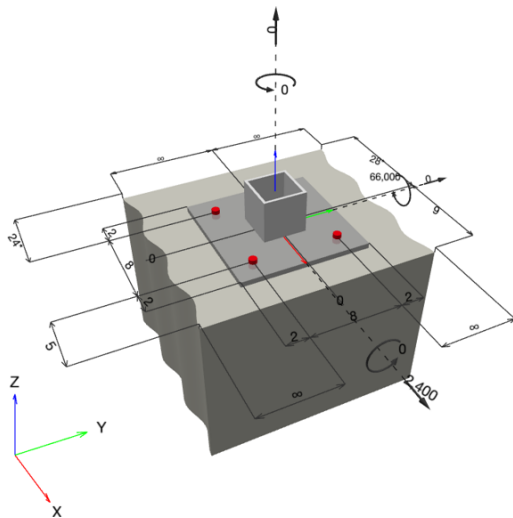
Specifier's comments:

1 Input data

Anchor type and diameter:	Hex Head ASTM F 1554 GR. 36 3/4	
Item number:	not available	
Specification text:	Ø 3/4 in Hex Head ASTM F 1554 GR. 36 with 8 in nominal embedment depth per Technical data , cast in place installation per MPII,	
Effective embedment depth:	$h_{ef} = 8.000$ in.	
Material:	ASTM F 1554	
Evaluation Service Report:	Hilti Technical Data	
Issued Valid:	- -	
Proof:	Design Method ACI 318-19 / CIP	
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)	
Stand-off installation:	$e_b = 0.000$ in. (no stand-off); $t = 0.500$ in.	
Anchor plate ^R :	$l_x \times l_y \times t = 12.000$ in. x 12.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)	
Profile:	Square HSS (AISC), HSS4X4X.25; $(L \times W \times T) = 4.000$ in. x 4.000 in. x 0.250 in.	
Base material:	cracked concrete, 4000, $f'_c = 4,000$ psi; $h = 22.000$ in.	
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar	

^R - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



www.hilti.com

Company:
Address:
Phone | Fax: |
Design: Concrete - Jul 29, 2025 (1)
Fastening point:

Page: 2
Specifier:
E-Mail:
Date: 7/30/2025

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V _x = 2,400; V _y = 0; M _x = 0; M _y = 66,000; M _z = 0;	no	36

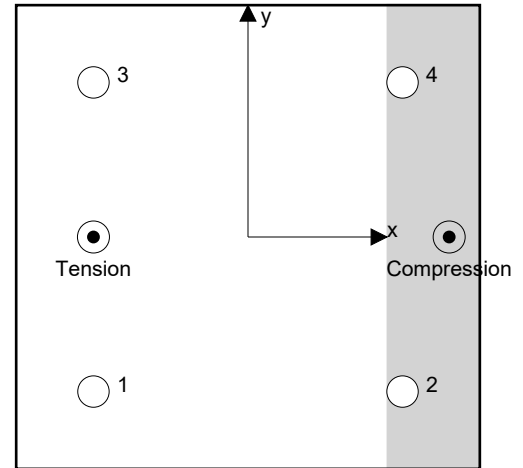
2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	3,584	600	600	0
2	0	600	600	0
3	3,584	600	600	0
4	0	600	600	0

Max. concrete compressive strain: 0.12 [‰]
Max. concrete compressive stress: 502 [psi]
Resulting tension force in (x/y)=(-4.000/0.000): 7,168 [lb]
Resulting compression force in (x/y)=(5.207/0.000): 7,168 [lb]



Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N _{ua} [lb]	Capacity ϕ N _n [lb]	Utilization $\beta_N = N_{ua} / \phi N_n$	Status
Steel Strength*	3,584	14,529	25	OK
Pullout Strength*	3,584	14,650	25	OK
Concrete Breakout Failure**	7,168	32,056	23	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)



www.hilti.com

Company:		Page:	3
Address:		Specifier:	
Phone Fax:		E-Mail:	
Design:	Concrete - Jul 29, 2025 (1)	Date:	7/30/2025
Fastening point:			

3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
 $\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in. ²]	f_{uta} [psi]
0.33	58,000

Calculations

N_{sa} [lb]
19,372

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
19,372	0.750	14,529	3,584

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
 $N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
 $\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in. ²]	λ_a	f'_c [psi]
1.000	0.65	1.000	4,000

Calculations

N_p [lb]
20,928

Results

N_{pn} [lb]	$\phi_{concrete}$	ϕN_{pn} [lb]	N_{ua} [lb]
20,928	0.700	14,650	3,584

www.hilti.com

Company:

Address:

Phone | Fax:

Design:

Fastening point:

Concrete - Jul 29, 2025 (1)

Page:

Specifier:

E-Mail:

Date:

4

7/30/2025

3.3 Concrete Breakout Failure

$$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318-19 Eq. (17.6.2.1b)}$$

$$\phi N_{cbg} \geq N_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

$$A_{Nc} \text{ see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)}$$

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-19 Eq. (17.6.2.1.4)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.3.1)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.4.1b)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.6.1b)}$$

$$N_b = k_c \lambda_a \sqrt{f_c} h_{ef}^{1.5} \quad \text{ACI 318-19 Eq. (17.6.2.2.1)}$$

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
8.000	0.000	0.000	13.000	1.000
c_{ac} [in.]	k_c	λ_a	f_c [psi]	
-	24	1.000	4,000	

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
768.00	576.00	1.000	1.000	1.000	1.000	34,346

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
45,795	0.700	32,056	7,168

www.hilti.com

Company:		Page:	5
Address:		Specifier:	
Phone Fax:		E-Mail:	
Design:	Concrete - Jul 29, 2025 (1)	Date:	7/30/2025
Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_v = V_{ua}/\phi V_n$	Status
Steel Strength*	600	7,555	8	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	2,400	55,097	5	OK
Concrete edge failure in direction x+**	2,400	6,831	36	OK

* highest loaded anchor **anchor group (relevant anchors)

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

4.1 Steel Strength

$$V_{sa} = 0.6 A_{se,V} f_{uta} \quad \text{ACI 318-19 Eq. (17.7.1.2b)}$$

$$\phi V_{steel} \geq V_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

Variables

$A_{se,V}$ [in. ²]	f_{uta} [psi]
0.33	58,000

Calculations

V_{sa} [lb]
11,623

Results

V_{sa} [lb]	ϕ_{steel}	ϕV_{sa} [lb]	V_{ua} [lb]
11,623	0.650	7,555	600

www.hilti.com

Company:

Address:

Phone | Fax:

Design:

Fastening point:

Concrete - Jul 29, 2025 (1)

Page:

Specifier:

E-Mail:

Date:

6

7/30/2025

4.2 Pryout Strength

$$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right] \quad \text{ACI 318-19 Eq. (17.7.3.1b)}$$

$$\phi V_{cp} \geq V_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

$$A_{Nc} \text{ see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)}$$

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-19 Eq. (17.6.2.1.4)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.3.1)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.4.1b)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.6.1b)}$$

$$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} \quad \text{ACI 318-19 Eq. (17.6.2.2.1)}$$

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	8.000	0.000	0.000	5.000
$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
800.00	576.00	1.000	1.000	0.825	1.000	34,346

Results

V_{cp} [lb]	$\phi_{concrete}$	ϕV_{cp} [lb]	V_{ua} [lb]
78,710	0.700	55,097	2,400

www.hilti.com

Company:
Address:
Phone | Fax: |
Design: Concrete - Jul 29, 2025 (1)
Fastening point:

Page: 7
Specifier:
E-Mail:
Date: 7/30/2025

4.3 Concrete edge failure in direction x+

$$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}} \right) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$$

ACI 318-19 Eq. (17.7.2.1b)

$$\phi V_{cbg} \geq V_{ua}$$

ACI 318-19 Table 17.5.2

$$A_{Vc} \text{ see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*}$$

$$A_{Vc0} = 4.5 c_{a1}^2$$

ACI 318-19 Eq. (17.7.2.1.3)

$$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e_v}{1.5c_{a1}}} \right) \leq 1.0$$

ACI 318-19 Eq. (17.7.2.3.1)

$$\psi_{ed,V} = 0.7 + 0.3 \left(\frac{c_{a2}}{1.5c_{a1}} \right) \leq 1.0$$

ACI 318-19 Eq. (17.7.2.4.1b)

$$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$$

ACI 318-19 Eq. (17.7.2.6.1)

$$V_b = 9 \lambda_a \sqrt{f_c} c_{a1}^{1.5}$$

ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
5.000	-	0.000	1.000	22.000
l_e [in.]	λ_a	d_a [in.]	f_c [psi]	$\psi_{parallel,V}$
6.000	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in. ²]	A_{Vc0} [in. ²]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
172.50	112.50	1.000	1.000	1.000	6,364

Results

V_{cbg} [lb]	$\phi_{concrete}$	ϕV_{cbg} [lb]	V_{ua} [lb]
9,758	0.700	6,831	2,400

*Anchor row defined by: Anchor 2, 4; Case 3 controls

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization β_{NV} [%]	Status
0.247	0.351	5/3	28	OK

$$\beta_{NV} = \beta_N^\zeta + \beta_V^\zeta \leq 1$$



www.hilti.com

Company:		Page:	8
Address:		Specifier:	
Phone Fax:		E-Mail:	
Design:	Concrete - Jul 29, 2025 (1)	Date:	7/30/2025
Fastening point:			

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to <https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/>

Fastening meets the design criteria!

www.hilti.com

Company:
Address:
Phone | Fax: |
Design: Concrete - Jul 29, 2025 (1)
Fastening point:

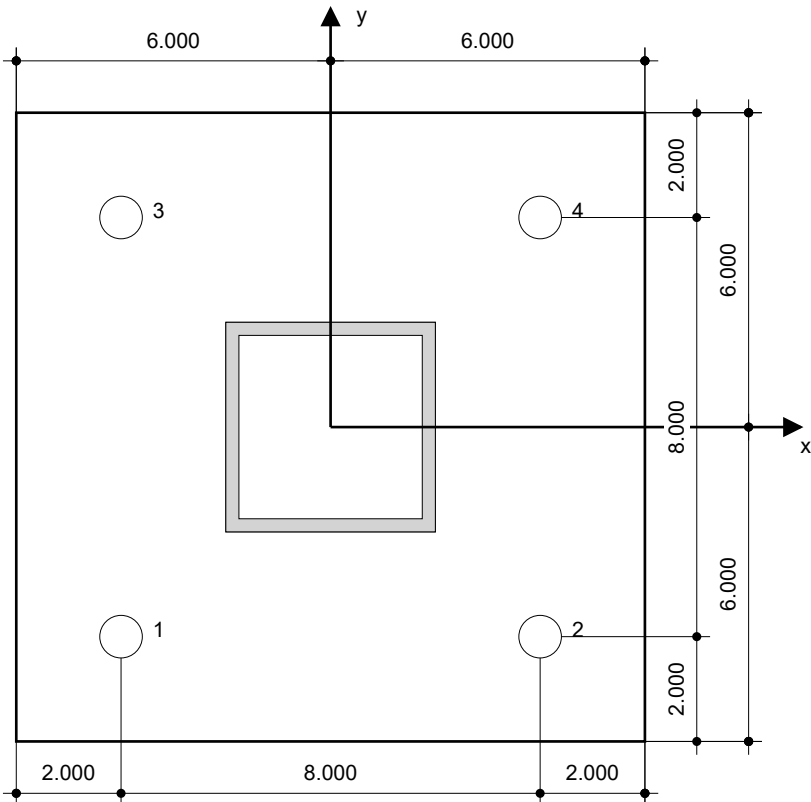
Page: 9
Specifier:
E-Mail:
Date: 7/30/2025

7 Installation data

Profile: Square HSS (AISC), HSS4X4X.25; (L x W x T) = 4.000 in. x 4.000 in. x 0.250 in.
Hole diameter in the fixture: $d_f = 0.812$ in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 3/4
Item number: not available
Maximum installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 8.000 in.
Minimum thickness of the base material: 9.000 in.

Ø 3/4 in Hex Head ASTM F 1554 GR. 36 with 8 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c _{-x}	c _{+x}	c _{-y}	c _{+y}
1	-4.000	-4.000	24.000	13.000	-	-
2	4.000	-4.000	32.000	5.000	-	-
3	-4.000	4.000	24.000	13.000	-	-
4	4.000	4.000	32.000	5.000	-	-



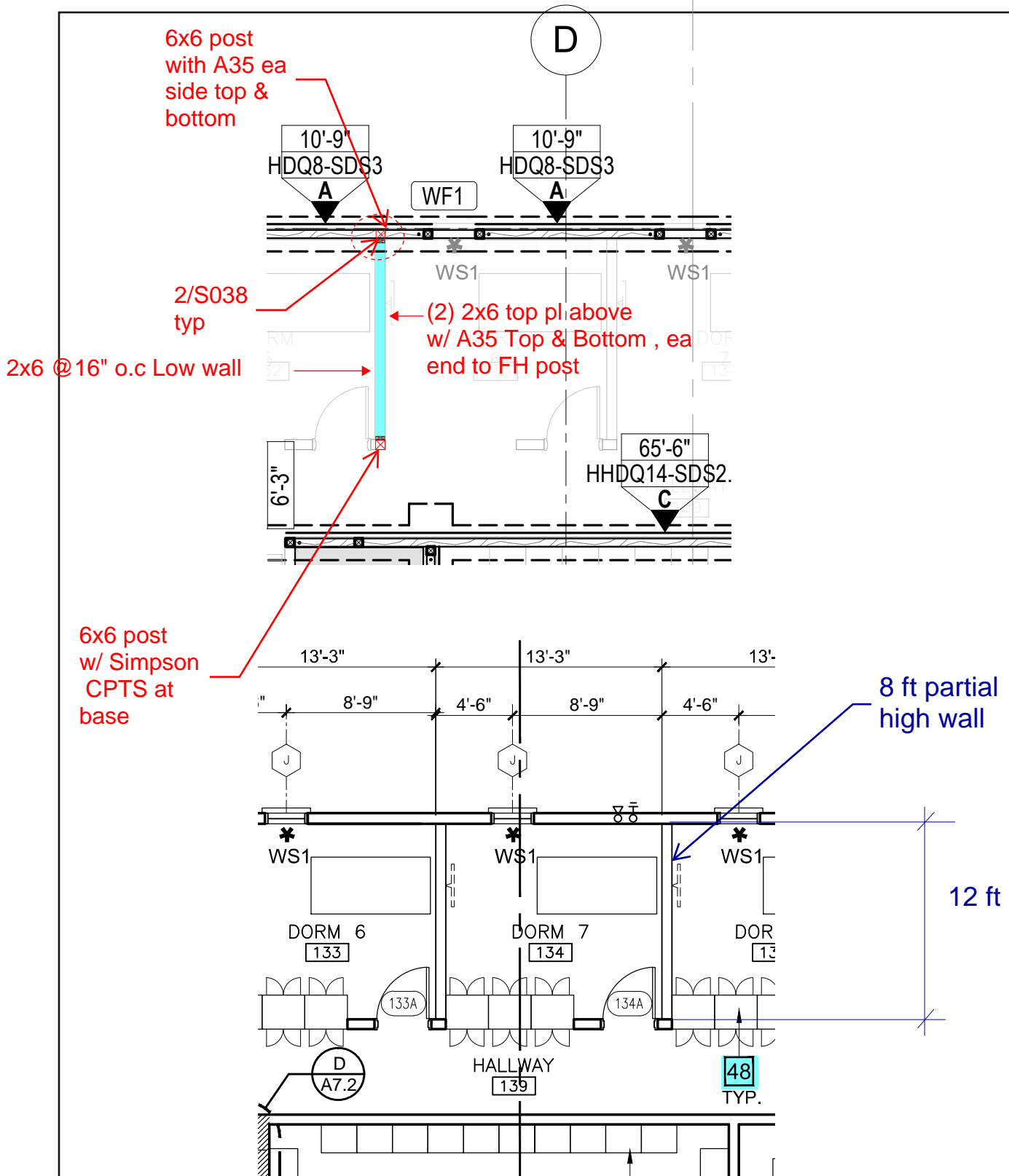
www.hilti.com

Company:		Page:	10
Address:		Specifier:	
Phone Fax:		E-Mail:	
Design:	Concrete - Jul 29, 2025 (1)	Date:	7/30/2025
Fastening point:			

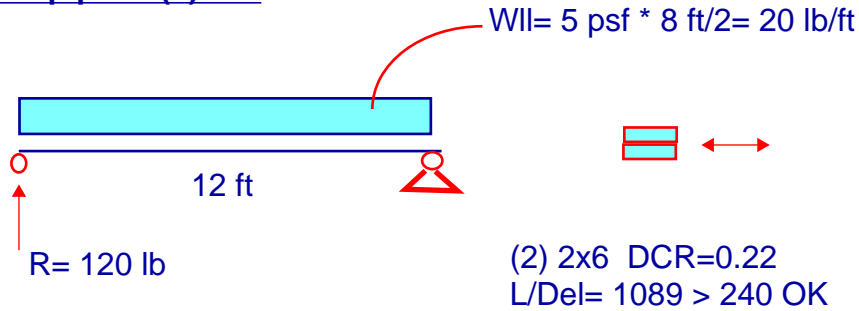
8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

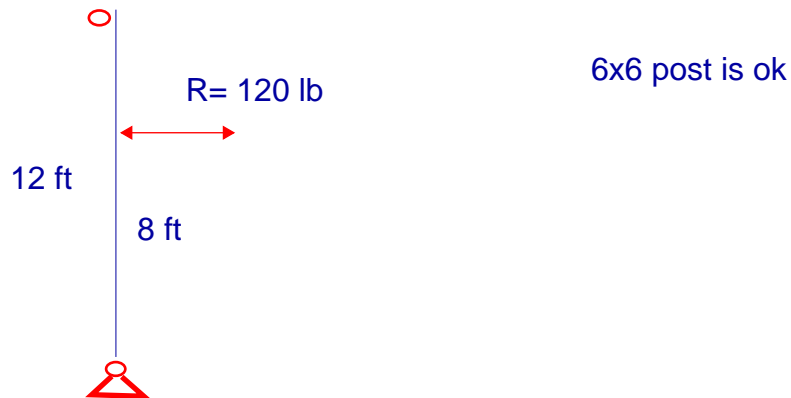
4.9 LOW WALL DESIGN



Cont. top plate (2) 2x6



Post support (2) 2x6



Wood Beam

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Cont (2) 2x6 top plates at low wall

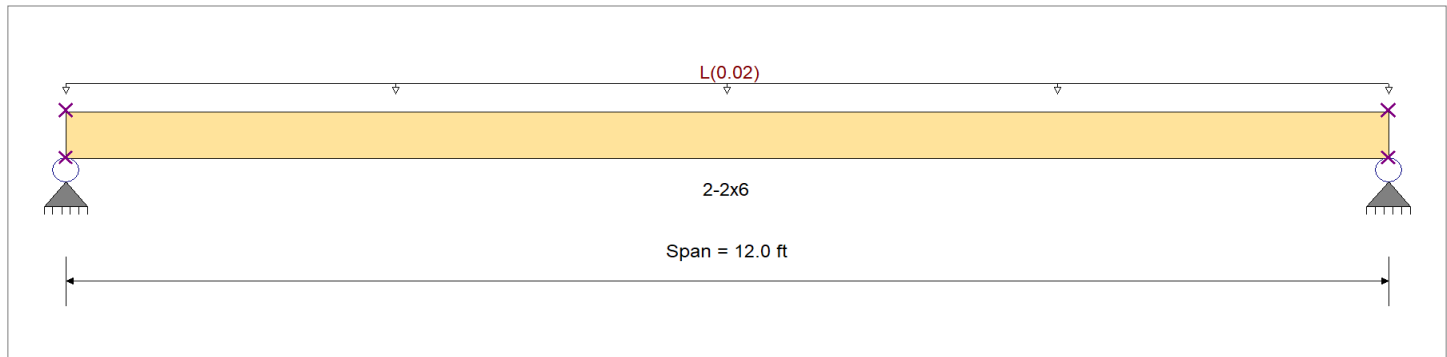
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design	Fb +	1000 psi	E : Modulus of Elasticity	
Load Combination : IBC 2021	Fb -	1000 psi	Ebend- xx	1700ksi
	Fc - Prll	1500 psi	Eminbend - xx	620ksi
Wood Species : Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade : No.1	Fv	180 psi		
	Ft	675 psi	Density	31.21 pcf
Beam Bracing : Completely Unbraced				



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : L = 0.020 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.224 : 1	Maximum Shear Stress Ratio	=	0.056 : 1
Section used for this span		2-2x6	Section used for this span		2-2x6
fb: Actual	=	285.62psi	fv: Actual	=	10.11 psi
F'b	=	1,275.49psi	F'v	=	180.00 psi
Load Combination		L Only	Load Combination		L Only
Location of maximum on span	=	6.000ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection	0 in	Ratio =	0 < 1084	n/a	
Max Upward Transient Deflection	0 in	Ratio =	0 < 1084	n/a	
Max Downward Total Deflection	0.133 in	Ratio =	1084 >= 240	Span: 1 : L Only	
Max Upward Total Deflection	0 in	Ratio =	0 < 240	n/a	

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
	Length = 12.0 ft	1			0.90	1.00	1.00	0.98	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
L Only	Length = 12.0 ft	1				1.00	1.00	0.98	1.300	1.00	1.00	1.00			1,150.8	0.00	0.0	162.0
	Length = 12.0 ft	1	0.224	0.056	1.00	1.00	1.00	0.98	1.300	1.00	1.00	1.00	0.36	285.6	1,275.5	0.11	10.1	180.0
+0.750L	Length = 12.0 ft	1				1.00	1.00	0.98	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
	Length = 12.0 ft	1	0.135	0.034	1.25	1.00	1.00	0.97	1.300	1.00	1.00	1.00	0.27	214.2	1,583.4	0.08	7.6	225.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
L Only	1	0.1327	6.044		0.0000	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Cont (2) 2x6 top plates at low wall

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.120	0.120
Max Upward from Load Combinations	0.090	0.090
Max Upward from Load Cases	0.120	0.120
L Only	0.120	0.120
+0.750L	0.090	0.090

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Column

Project File: FS 46 enercalc.ec6

LIC# : KW-06016422, Build:20.22.12.28

SAIFUL - BOUQUET CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2022

DESCRIPTION: Post Support (2) 2x6 top plates

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
 Load Combinations Used : IBC 2021

General Information

Analysis Method :	Allowable Stress Design			Wood Section Name	6x6		
End Fixities	Top & Bottom Pinned			Wood Grading/Manuf.	Graded Lumber		
Overall Column Height	12 ft			Wood Member Type	Sawn		
(Used for non-slender calculations)							
Wood Species	Douglas Fir-Larch			Exact Width	5.50 in	Allow Stress Modification Factors	
Wood Grade	No.1			Exact Depth	5.50 in	Cf or Cv for Bending	1.0
Fb +	1200 psi	Fv	170 psi	Area	30.250 in^2	Cf or Cv for Compression	1.0
Fb -	1200 psi	Ft	825 psi	Ix	76.255 in^4	Cf or Cv for Tension	1.0
Fc - Prll	1000 psi	Density	31.21 pcf	Iy	76.255 in^4	Cm : Wet Use Factor	1.0
Fc - Perp	625 psi					Ct : Temperature Factor	1.0
E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial			Cfu : Flat Use Factor	1.0
	Basic	1600	1600	1600 ksi		Kf : Built-up columns	1.0 NDS 15.3.2
	Minimum	580	580			Use Cr : Repetitive ?	No
Brace condition for deflection (buckling) along columns :							
				X-X (width) axis :	Unbraced Length for buckling ABOUT Y-Y Axis = 12 ft, K =		
				Y-Y (depth) axis :	Unbraced Length for buckling ABOUT X-X Axis = 12 ft, K =		

Applied Loads

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

Column self weight included : 78.675 lbs * Dead Load Factor

BENDING LOADS . . .

Lat. Point Load at 8.0 ft creating Mx-x, L = 0.120 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.1155 : 1**
 Load Combination +D+L
 Governing NDS Formula Comp + Mxx, NDS Eq. 3.9-3
 Location of max.above base 7.973 ft
 At maximum location values are . . .
 Applied Axial 0.07868 k
 Applied Mx 0.3189 k-ft
 Applied My 0.0 k-ft
 Fc : Allowable 556.14 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.080 k	Bottom along Y-Y	0.040 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y	0.05312 in	at	6.604 ft	above base
for load combination : +D+L				
Along X-X	0.0 in	at	0.0 ft	above base
for load combination : n/a				

Other Factors used to calculate allowable stresses . . .

<u>Bending</u>	<u>Compression</u>	<u>Tension</u>
----------------	--------------------	----------------

PASS Maximum Shear Stress Ratio = **0.02334 : 1**
 Load Combination +D+L
 Location of max.above base 12.0 ft
 Applied Design Shear 3.967 psi
 Allowable Shear 170.0 psi

Load Combination Results

Load Combination	C _D	C _P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.596	0.004845	PASS	0.0 ft	0.0	PASS	12.0 ft
+D+L	1.000	0.556	0.1155	PASS	7.973 ft	0.02334	PASS	12.0 ft
+D+0.750L	1.250	0.472	0.06929	PASS	7.973 ft	0.0140	PASS	12.0 ft
+0.60D	1.600	0.386	0.002526	PASS	0.0 ft	0.0	PASS	12.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		Y-Y Axis Reaction		Axial Reaction	My - End Moments		Mx - End Moments	
	@ Base	@ Top	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top
D Only					0.079				
+D+L			0.040	0.080	0.079				
+D+0.750L			0.030	0.060	0.079				

Wood Column

Project File: FS 46 enercal.ec6

LIC# : KW-06016422, Build:20.22.12.28 SAIFUL - BOUQUET CONSULTING ENGINEERS (c) ENERCALC INC 1983-2022

DESCRIPTION: Post Support (2) 2x6 top plates

Maximum Reactions

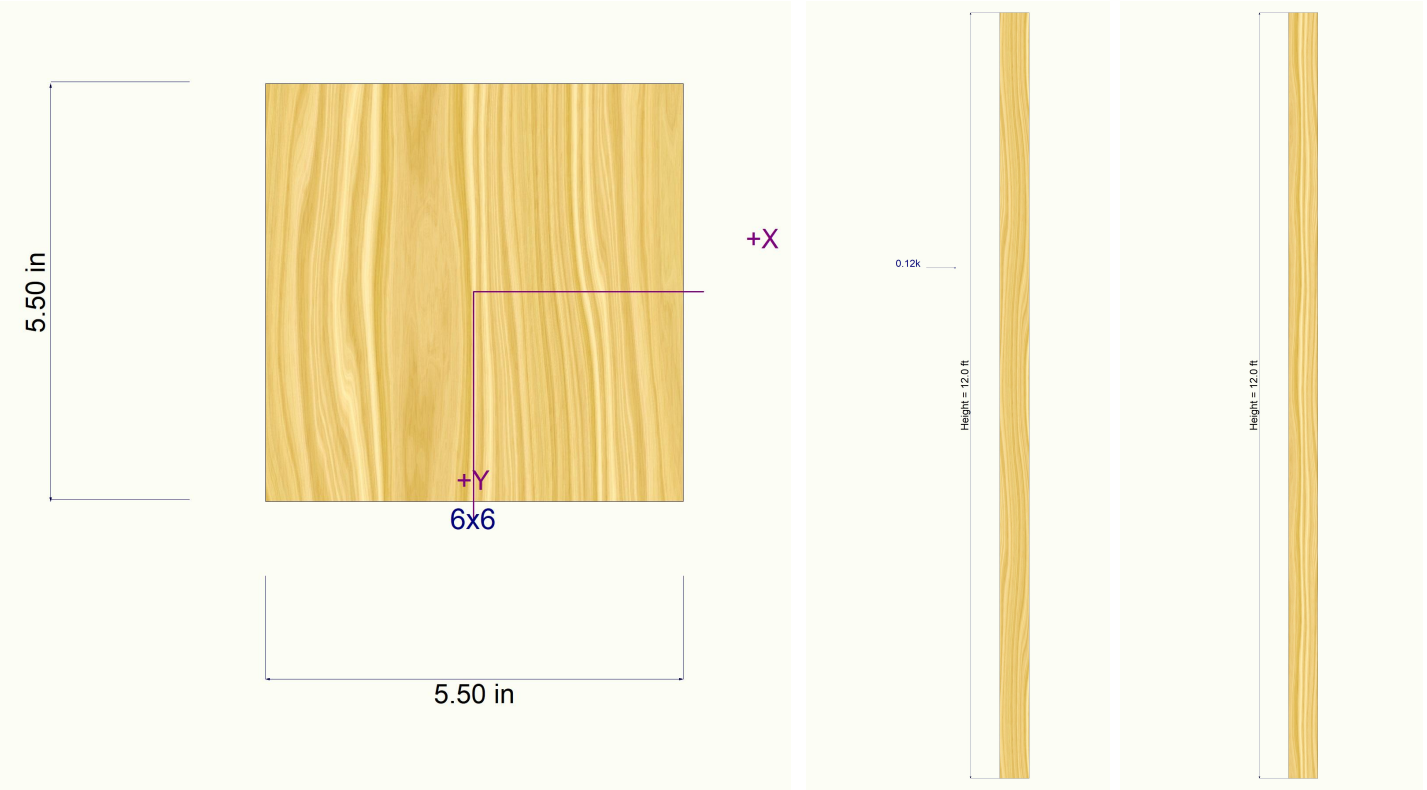
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
+0.60D						0.047					
L Only				0.040	0.080						

Maximum Deflections for Load Combinations

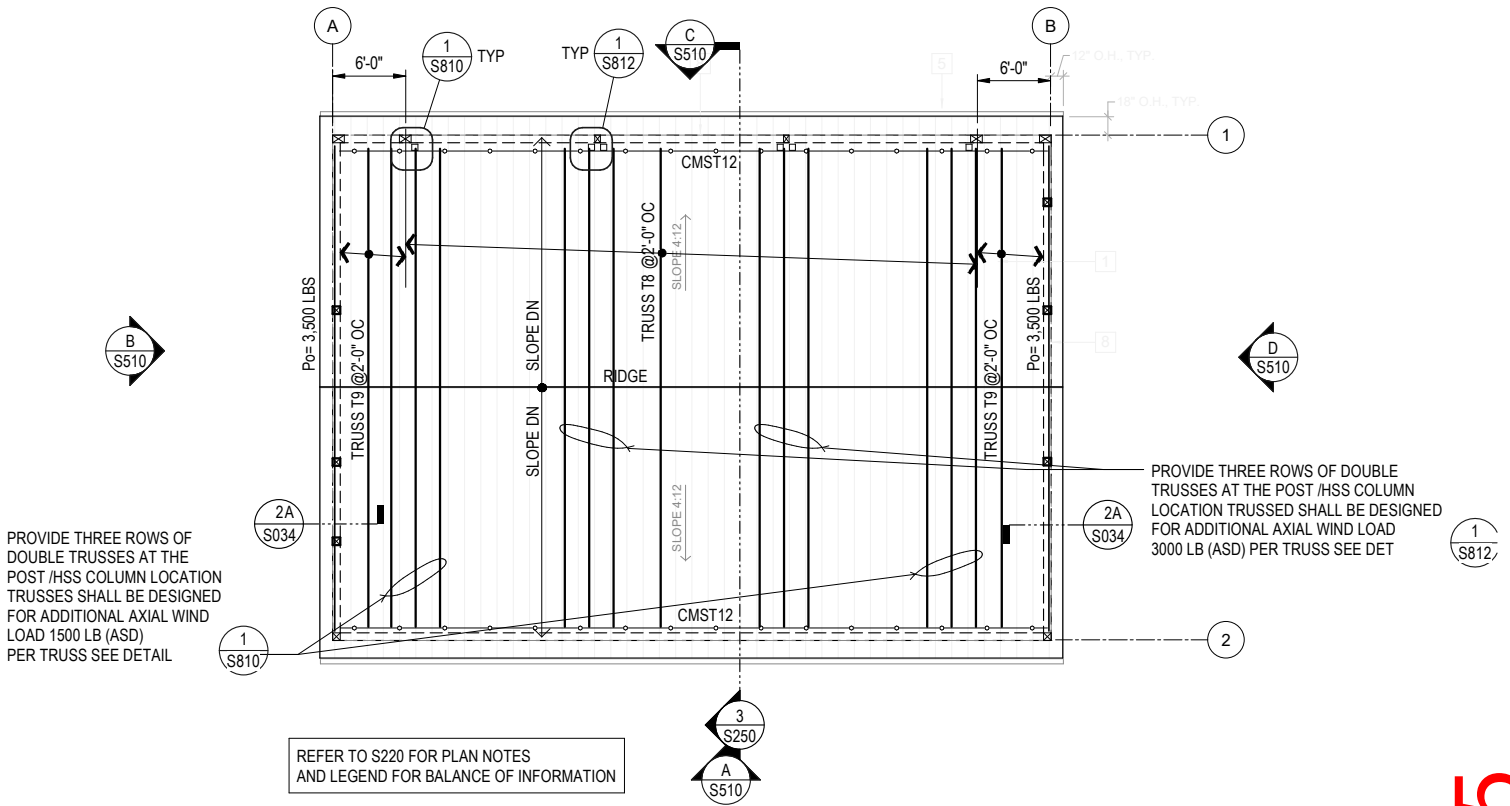
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000ft	0.053 in	6.604 ft
+D+0.750L	0.0000 in	0.000ft	0.040 in	6.604 ft
+0.60D	0.0000 in	0.000ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000ft	0.053 in	6.604 ft

Sketches



4.10 LOADING ON TRUSS

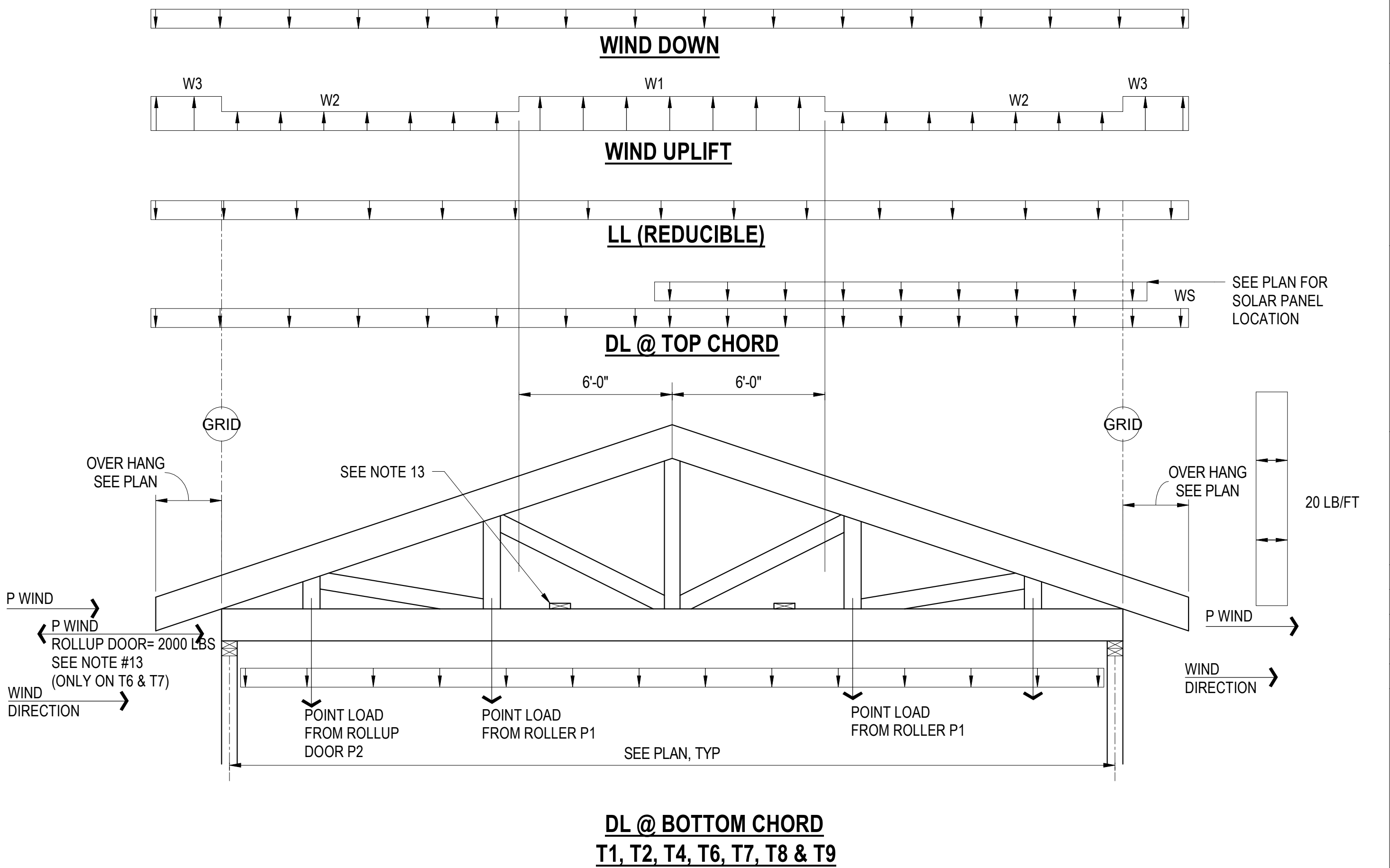
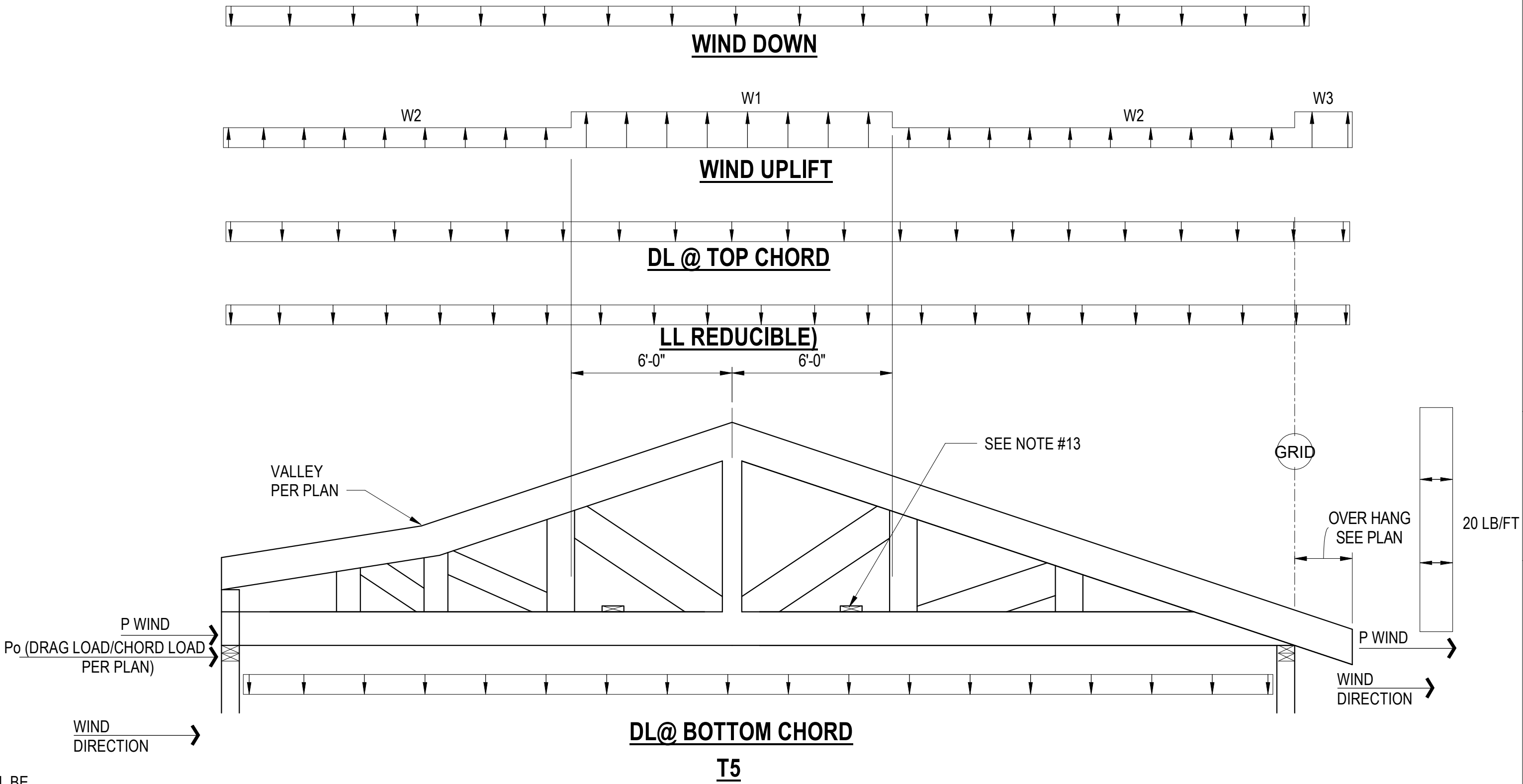
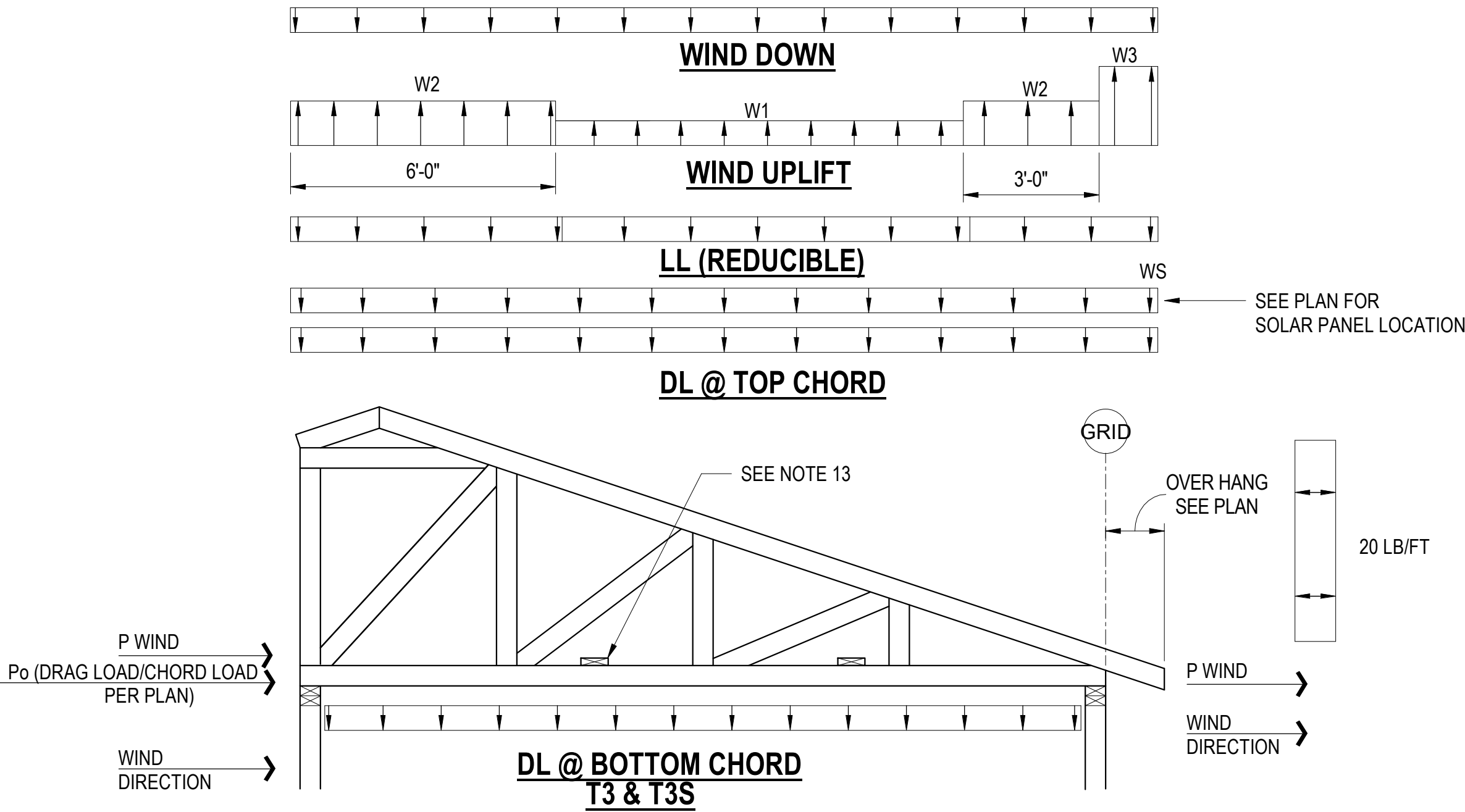
ADDENDUM 5



			TOP CHORD							BOTTOM CHORD			TOP OF WALL REACTION AT BOTTOM OF TRUSS (NOTE 7)	
MAIN BUILDING	TRUSS (NOTE 1)	CASE	DL (LB/FT) (SEE NOTES 8, 9 & 10)		LL (LB/FT) (NOTE 6)	WIND UP (LB/FT) (NOTE 5)			WIND DOWN (LB/FT) (NOTE 5)	DL (LB/FT) (NOTE 9 & 8)	P DL (LB) (NOTE 11)		P WIND (LB)	
			ROOF	SOLAR		W1	W2	W3			P1	P2		
	T1	1	17	12	40	58	28	65	15	26	200	--	250	
		2	33	12	40	58	28	65	15	10	200	--	250	
	T2	1	17	12	40	66	58	65	15	26	200	--	250	
		2	33	12	40	66	58	65	15	10	200	--	250	
	T3	1	26	--	40	32	36	65	10	10	--	--	250	
		2	17	--	40	32	36	65	10	19	--	--	250	
	T3S	1	26	12	40	32	36	65	10	10	--	--	250	
		2	17	12	40	32	36	65	10	19	--	--	250	
	T4	1	26	12	40	58	50	65	14	10	--	--	250	
		2	17	12	40	58	50	65	14	19	--	--	250	
T5	1	26	--	40	58	50	65	14	10	--	--	250		
	2	17	--	40	58	50	65	14	19	--	--	250		
T6	1	17	--	40	58	28	65	15	22	--	300	360		
	2	29	--	40	58	28	65	15	10	--	300	360		
T7	1	17	--	40	66	58	65	15	22	--	300	360		
	2	29	--	40	66	58	65	15	10	--	300	360		
RESERVE APPARATUS BUILDING	T8	1	17	--	40	58	28	65	15	22	--	300	450	
		2	29	--	40	58	28	65	15	10	--	300	450	
	T9	1	17	--	40	66	58	65	15	22	--	300	450	
		2	29	--	40	66	58	65	15	10	--	300	450	

NOTES:

- LOADING SHOWN IS BASED ON TRUSS SPACING OF 24" OC. TRUSS SPACING SHALL NOT EXCEED 24" OC. SEE FRAMING PLAN FOR TRUSS LOCATION
- TRUSS DESIGNER/CONTRACTOR SHALL CONSIDER ALL LOADING THAT OCCURS ON THE TRUSSES AND TRUSS BLOCKING INCLUDING THOSE INDICATED ON ALL THE DETAILS AND PLANS IN ADDITION TO TABLE ABOVE. IN ADDITION TO ABOVE, LOADING DUE TO MECHANICAL, ELECTRICAL, AND PLUMBING EQUIPMENT HUNG FROM THE TRUSSES SHALL BE CONSIDERED, CORRDINATE W/ MEP DRAWINGS. SEE DETAIL 8/S036 CONNECTION DETAIL. TRUSS DESIGNER/CONTRACTOR SHALL COORDINATE W/ DESIGN DOCUMENTS WHERE DIFFERENT TRUSS CONDITIONS OCCUR. AND SUBMIT ALL TRUSS CONDITIONS TO ENGINEER AND ARCH FOR REVIEW AND APPROVAL PRIOR TO FABRICATION
- TRUSS DESIGNER SHALL COORDINATE IN PLANE LOADINGS FOR THE TRUSS COLLECTORS AND TRUSS BLOCKING WITH DESIGN DRAWINGS AND THE EOR THE INPLANE LOADING CAPACITY OF TRUSS COLLECTORS SHALL MATCH TOTAL SHEAR WALL CAPACITY IN LINE OR BELOW THEM AND LOADING INDICATED ON PLANS/DETAILS WHICHEVER IS LARGER FOR COLLECTOR TRUSSES MIN TRANSFER LOAD FROM BOTTOM CHORD TO TOP CHORD OF TRUSS SHALL BE 250 LB/FT ALONG LENGTH OF TRUSS. THE INPLANE LOADING CAPACITY OF TRUSS BLK'G SHALL MATCH CAPACITY OF SHEAR WALL BELOW OR IN-LINE WITH THEM. THE MINIMUM LOAD TRANSFER FROM BOTTOM TO TOP OF TRUSS BLK'G SHALL BE 1060 LBS PER TRUSS BLOCKING OR 530 PLF WHICHEVER IS LARGER. WHERE INDICATED ON PLAN, Po IS SEISMIC DRAG LOADS (ASD). TRUSS SHALL BE DESIGNED TO CARRY THIS LOAD FROM TOP CHORD WHERE ROOF DIAPHRAGM OCCURS TO BOTTOM CHORD WHERE SHEAR WALLS OCCURS.
- SEE PLANS AND TYPICAL ROOF TRUSS DETAILS FOR CONDITION WHERE TRUSS COLLECTOR OCCURS.
- WIND OR SEISMIC LOADING INDICATED IS ASD LEVEL LOADS.
- REDUCE ROOF LIVE LOADS FOR SLOPE PER CBC 2022, SECTION 167.14.2.
- TRUSS SHALL BE DESIGNED TO CARRY THE REACTION FROM TOP OF WALL BELOW THE TRUSS DUE TO WIND LOADING TO TOP TRUSS CHORD WHERE ROOF DIAPHRAGM OCCURS.
- ADD 10 PSF DEAD LOAD AT THE TRUSS OVERHANG FOR STUCCO SOFFIT
- DL INDICATED IN THE TABLE ABOVE DOES NOT INCLUDE SELF-WEIGHT OF TRUSSES. ALL DEAD LOADS MUST BE INCREASED TO ACCOUNT FOR ROOF SLOPE PER ARCH.
- SOLAR PANELS INCLUDING CONNECTIONS SHALL BE DESIGNED BY OTHER. SOLAR PANEL WITH 6 PSF MAX. SEE PLAN FOR SOLAR PANEL LOCATION.
- ALL CONCENTRATE LOADS SHALL BE AT PANEL POINT (12" MAX AWAY PANEL POINT). THE CONCENTRATE LOADS CAN NOT EXCEED THE UNIFORM LOAD TIMES THE DISTANCE BETWEEN PANEL POINTS.
- CONTINUOUS MEMBERS TO BRACE BOTTOM CHORDS FOR STABILITY OR OTHER REQUIREMENTS ARE PER TRUSS MANUFACTURER.
- ADDITIONAL WIND LOAD FROM THE ROLL-UP DOOR FRAMING TO THE TRUSSES AT RESERVE APPARATUS BUILDING SEE PLAN AND DETAILS
- BOTTOM CHORD OF THE TRUSSES SHALL MINIMUM 2X6. WHERE LARGER MEMBERS ARE REQUIRED IT SHALL BE BROUGHT TO THE ATTENTION AOR/EOR AND APPROVAL OBTAINED PRIOR TO FABRICATION.



TYPICAL DESIGN CRITERIA FOR ROOF TRUSS AND TRUSS BLOCKING LOADING

SCALE: NTS

1

WILLIAM LOYD JONES
ARCHITECT

9415 culver boulevard
culver city, california
90230
90233
90232
90231

TEL 310 392 3995

S

o

sciful-bouquet

structural engineers

726 S. Figueroa St.,
37th floor
Los Angeles, CA 90017
www.sciful-bouquet.com
(213) 315-2277
Project #25534

TYPICAL WOOD DETAILS

FIRE STATION 46
MISSION VILLAGE
COUNTY OF LOS ANGELES FIRE DEPARTMENT
VALENCIA, CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
No. S3289
Exp. 12/31/25
WEIGHTMAN FOUR FANTASY
STRUCTURAL
STATE OF CALIFORNIA

THE ABOVE DRAWINGS AND SPECIFICATIONS AND DESIGN AND ARRANGEMENTS REPRESENTED THEREBY ARE AND SHALL REMAIN THE PROPERTY OF THE ARCHITECT AND NO PART THEREOF SHALL BE COPIED, DISCLOSED TO OTHERS OR USED IN CONNECTION WITH ANY WORK OR PROJECT OTHER THAN THE SPECIFIC PROJECT FOR WHICH THEY HAVE BEEN PREPARED, AND DEVELOPED WITHOUT THE WRITTEN CONSENT OF THE ARCHITECT. VERBAL CONTACT WITH THESE DRAWINGS OR SPECIFICATIONS SHALL CONSTITUTE CONCLUSIVE EVIDENCE OF ACCEPTANCE OF THESE RESTRICTIONS.

WRITTEN DIMENSIONS ON THESE DRAWINGS SHALL HAVE PRECEDENCE OVER SCALED DIMENSIONS. CONTRACTORS SHALL VERIFY AND BE RESPONSIBLE FOR ALL DIMENSIONS AND CONDITIONS ON THE JOB AND THIS OFFICE MUST BE NOTIFIED OF ANY VARIATIONS FROM THE DIMENSIONS AND CONDITIONS SHOWN IN THESE DRAWINGS. SHOP DETAILS MUST BE SUBMITTED TO THIS OFFICE FOR APPROVAL, BEFORE PROCEEDING WITH FABRICATION.
Copyright 2025

Date

Issue Date

Drawn

Checked

Scale

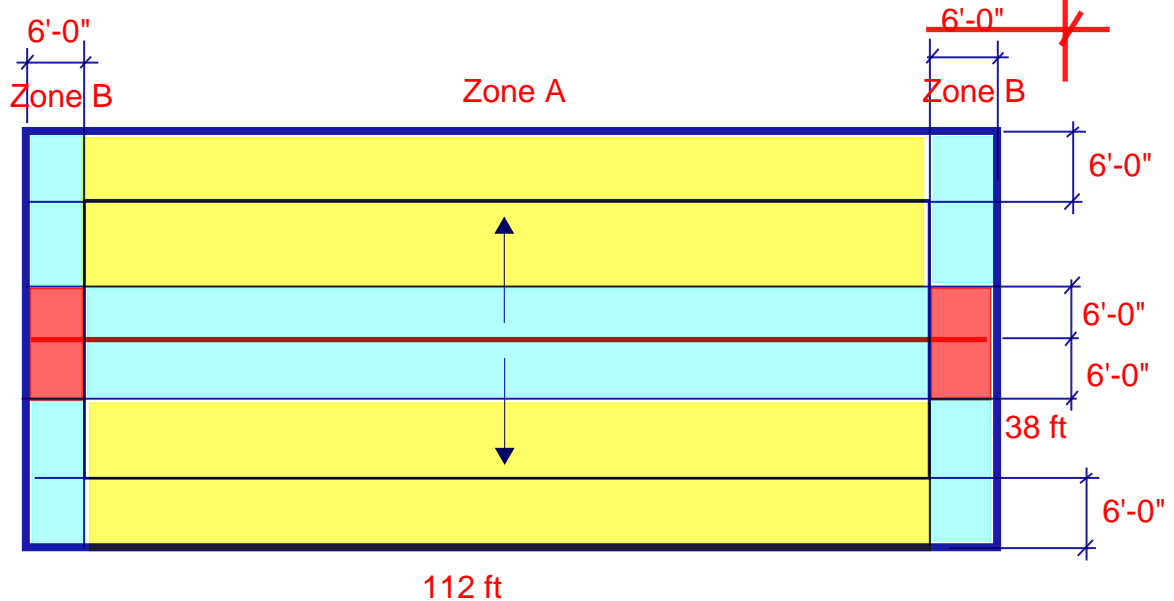
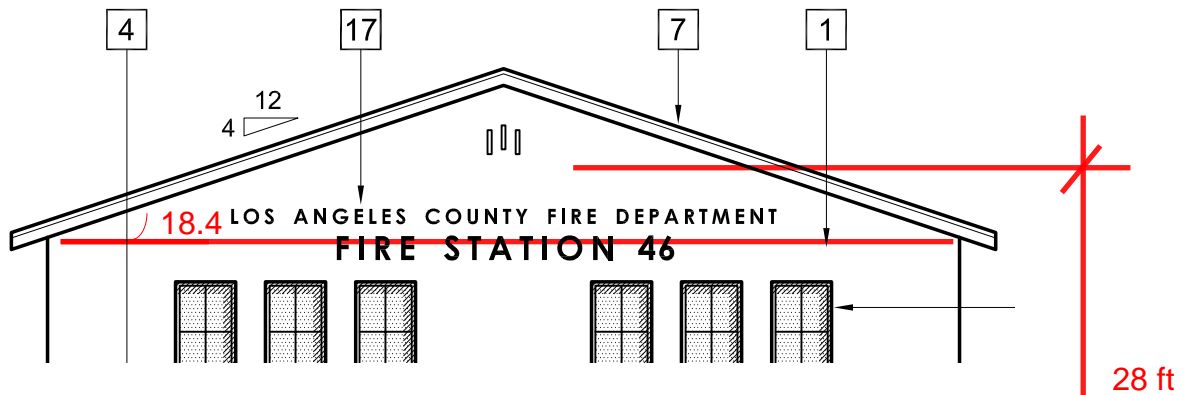
AS NOTED

Job. No.

Project Number

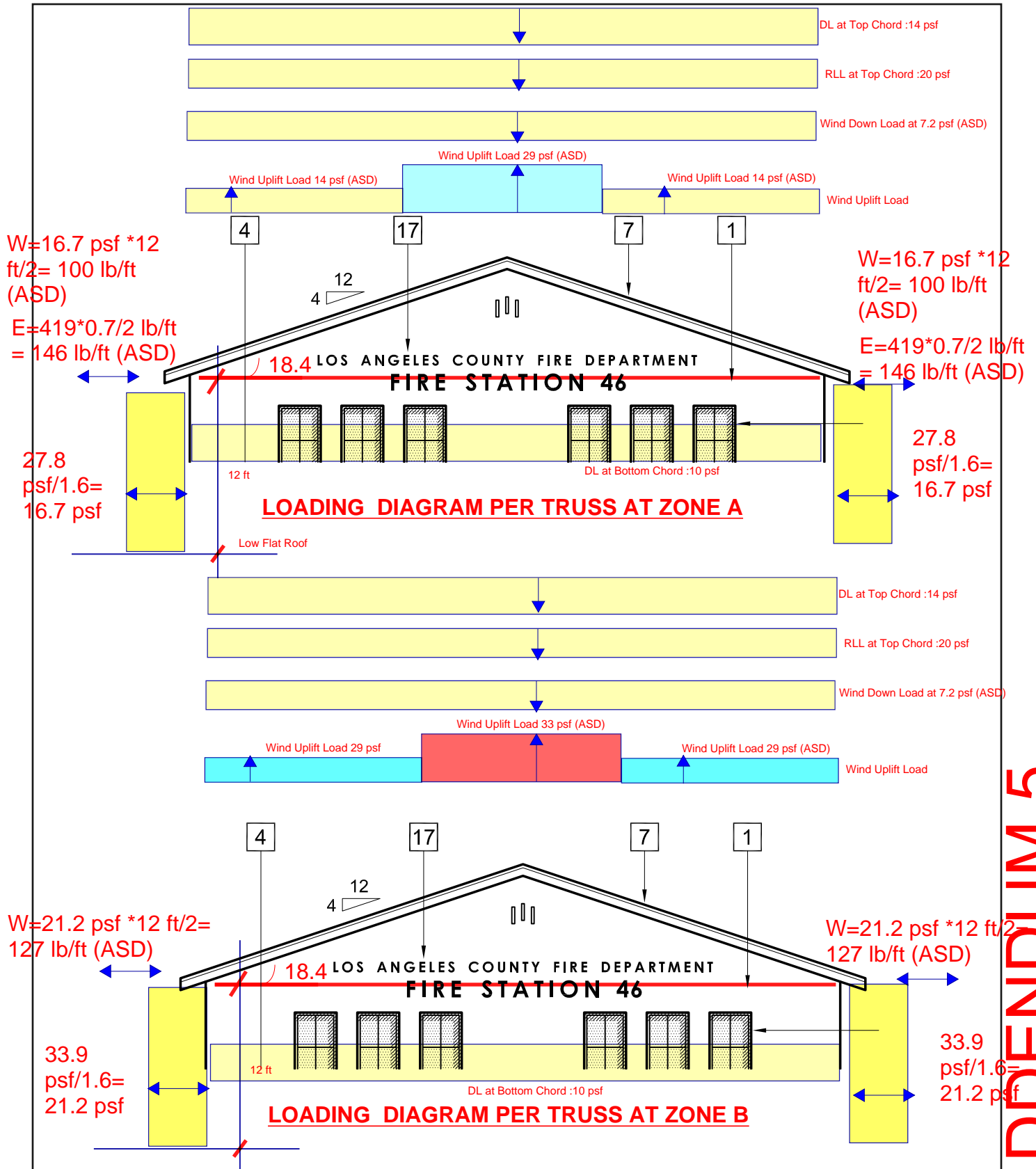
10% DESIGN DEVELOPMENT SUBMITTAL

S035



- Indicates area with wind uplift load 14 psf (ASD)
- Indicates area with wind uplift load 29 psf (ASD)
- Indicates area with wind uplift load 33 psf (ASD)

1. Wind down load of 7.2 psf (ASD) for the whole roof



ASCE 7-10 Wind Calculation - Components & Cladding

Code: ASCE 7-10

Assumptions

- 1) All Section, Figure, and Table references are to ASCE 7-10, unless noted otherwise.
- 2) Building is enclosed. (See Section 26.2 for definition of Enclosed Building) This assumption has 2 impacts:
 - a. Value of GC_{pi} is dependent on type of building enclosure.
 - b. $q_i = q_h$ in equation 30.6-1 for positive internal pressure evaluation (for $h > 60'$).
- 3) Reduction factor for large volumes is ignored. (Section 26.11.1.1)
- 3) Roof Angle, $\theta \leq 7^\circ$, Fig 30.4-2A is used for Roof Pressures, 10% reduction in GC_p values for Fig. 30.4-1.

Building Parameters

Roof Mean Height, $h = 30.0$ ft

Number of Stories = 1

Story Information:					
Story	Story Ht.	z	K_z	K_{zt}	q_z
Roof	-	30	0.98	1.00	23.5
Ground	30.0 ft	0	0.85	1.00	20.4

Topographic Effects: Section 26.8

Conditions of Section 26.8.1 Satisfied: No (Section 26.8.1)

Topographic Factor, $K_{zt} = 1.00$ (Section 26.8.2)

Wind Parameters

Risk Category, IV (CBC 2013 Table 1604.5)

Ultimate Wind Speed, $V = 105$ mph (Figure 26.5-1)

Directionality Factor, $K_d = 0.85$ (Table 26.6-1)

Exposure Category = C (Section 26.7)

Topographic Factor at Roof, $K_{zt,h} = 1.00$ (Section 26.8.2)

Roof Velocity Pressure Coefficient, $K_z = 0.98$ (Table 30.3-1)

Roof Velocity Pressure, $q_h = 23.55$ psf (Equation 30.3-1)

Internal Pressure Coefficient, $GC_{pi} = \pm 0.18$ (Table 26.11-1)

Roof Angle (where applicable), $\theta = 0^\circ$ (Figure 30.4)

Parapet Parameters

For Parapet Design with $h < 160'$, refer to Section 30.7.1.2

Parapet Height Above Roof, $h_p = 0.00$ ft

Topographic Factor at Roof, $K_{zt,p} = 1.00$ (Section 26.8.2)

Parapet Velocity Pressure Coeff, $K_{z,p} = 0.98$ (Table 30.3-1)

Parapet Velocity Pressure, $q_p = 23.5$ psf (Equation 30.3-1)

Parapet Internal Pressure Coef, $GC_{pi} = \pm 0.00$ (Table 26.11-1)

Mean Roof Height, h , is ≤ 60 ft, use Figure 30.4-1 to determine Wall External Pressure Coefficients

Figure 30.4-1 - External Pressure Coefficients (GC_p), $h \leq 60$ ft.

Wind Effective Area, A	Region 4 (Fig. 30.4-1)		Region 5 (Fig. 30.4-1)	
	GC_{p+}	GC_{p-}	GC_{p+}	GC_{p-}
10.0 ft	0.900	-0.999	0.900	-1.260
50.0 ft	0.791	-0.881	0.791	-1.046
100.0 ft	0.743	-0.833	0.743	-0.951

ASCE 7-10 Wind Calculation

Section 30.4 - Wall Cladding Design

Wind Pressures, $h \leq 60$ ft

Project: Fire Station #46

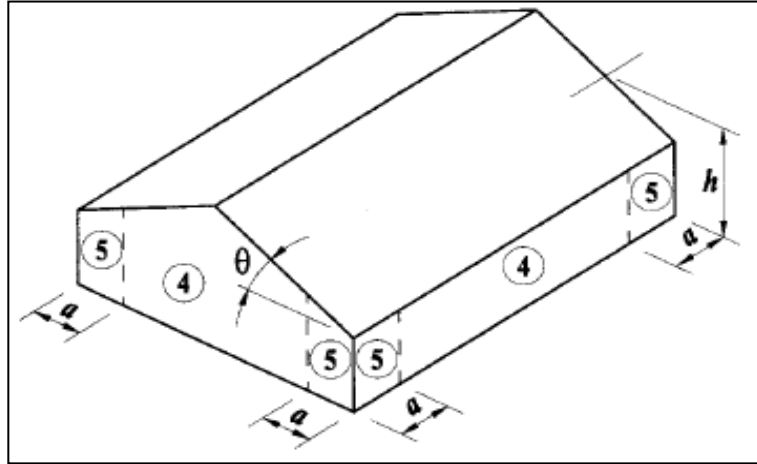
Building: All Buildings

Date: 3/13/2025

Project #: 08534

Engineer :

Code: ASCE 7-10



For Buildings with $h \leq 60$ ft, design wind pressure, p , for components and cladding is independent from height above ground, z .

Region 4		
Typical for all levels	p_{max}	p_{min}
	psf	psf
	(Eqn. 30.4-1)	
Wind Effective Area, $A = 10$ sf	25.4	-27.8
Wind Effective Area, $A = 50$ sf	22.9	-25.0
Wind Effective Area, $A = 100$ sf	21.7	-23.9

Region 5		
Typical for all levels	p_{max}	p_{min}
	psf	psf
	(Eqn. 30.4-1)	
Wind Effective Area, $A = 10$ sf	25.4	-33.9
Wind Effective Area, $A = 50$ sf	22.9	-28.9
Wind Effective Area, $A = 100$ sf	21.7	-26.6

Notes:

1. Positive pressures act towards surface, negative pressures act away from surface. Cladding elements and fasteners shall be designed for both positive and negative pressures
2. For $\theta \leq 10^\circ$, GC_p values are to be reduced by 10% per note 5 on Fig. 30.4-1
3. Minimum design wind pressure is 16 psf per Section 30.2.2

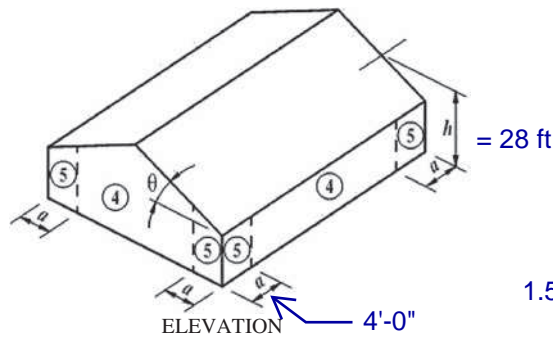
Fig 30.4-1 (Part 1)

$10\% \ 38 \text{ ft} = 3.8 \text{ ft}$

Notation

$a = 10\%$ of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

$0.4 * 28 \text{ ft} = 11.2$



1.52 ft

WIND LOAD CALCULATION FOR ROOF TRUSS DESIGN

30.3.2 Design Wind Pressures. Design wind pressures on C&C elements of low-rise buildings and buildings with $h \leq 60$ ft ($h \leq 18.3$ m) shall be determined from the following equation:

$$p = q_h[(GC_p) - (GC_{pi})](\text{lb/ft}^2) \quad (30.3-1)$$

$$p = q_h[(GC_p) - (GC_{pi})](\text{N/m}^2) \quad (30.3-1.\text{si})$$

where

q_h = velocity pressure evaluated at mean roof height h as defined in Section 26.10;

(GC_p) = external pressure coefficients given in:

- Fig. 30.3-1 (walls),
- Figs. 30.3-2A-I (flat roofs, gable roofs and hip roofs),
- Fig. 30.3-3 (stepped roofs),
- Fig. 30.3-4 (multispan gable roofs),
- Figs. 30.3-5A-B (monoslope roofs),
- Fig. 30.3-6 (sawtooth roofs),
- Fig. 30.3-7 (domed roofs),
- Fig. 27.3-3, Note 4 (arched roofs);

(GC_{pi}) = internal pressure coefficient given in Table 26.13-1.

Table 26.13-1 Main Wind Force Resisting System and Components and Cladding (All Heights): Internal Pressure Coefficient, (GC_{pi}) , for Enclosed, Partially Enclosed, Partially Open, and Open Buildings (Walls and Roof)

Enclosure Classification	Criteria for Enclosure Classification	Internal Pressure	Internal Pressure Coefficient, (GC_{pi})
Enclosed buildings	A_o is less than the smaller of $0.01A_g$ or 4 sq ft (0.37 m) and $A_{oi}/A_{gi} \leq 0.2$	Moderate	+0.18 -0.18
Partially enclosed buildings	$A_o > 1.1A_{oi}$ and $A_o >$ the lesser of $0.01A_g$ or 4 sq ft (0.37 m) and $A_{oi}/A_{gi} \leq 0.2$	High	+0.55 -0.55
Partially open buildings	A building that does not comply with Enclosed, Partially Enclosed, or Open classifications	Moderate	+0.18 -0.18
Open buildings	Each wall is at least 80% open	Negligible	0.00

Notes

1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of (GC_{pi}) shall be used with q_e or q_h as specified.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:
 - a. A positive value of (GC_{pi}) applied to all internal surfaces, or
 - b. A negative value of (GC_{pi}) applied to all internal surfaces.

BUILDING, ENCLOSED: A building that has the total area of openings in each wall, that receives positive external pressure, less than or equal to 4 sq ft (0.37 m²) or 1% of the area of that wall, whichever is smaller. This condition is expressed for each wall by the following equation:

$$A_o < 0.01A_g, \text{ or } 4 \text{ sq ft (0.37 m}^2\text{), whichever is smaller,}$$

where A_o and A_g are as defined for Open Buildings.

BUILDING, LOW-RISE: Enclosed or partially enclosed building that complies with the following conditions:

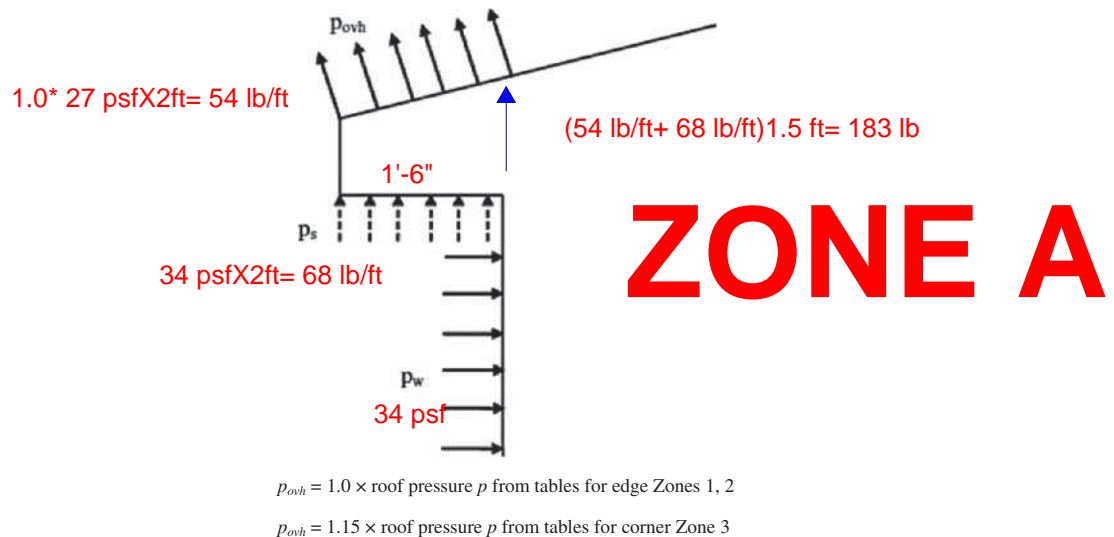
1. Mean roof height h less than or equal to 60 ft (18 m).
2. Mean roof height h does not exceed least horizontal dimension.

ADDENDUM 5

337

WIND LOAD CALCULATION FOR ROOF TRUSS DESIGN CONT

Diagram

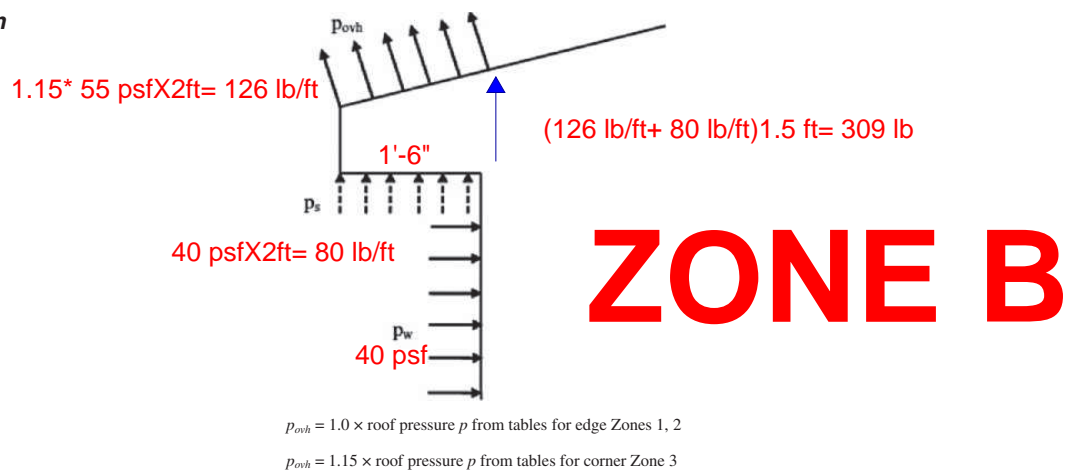


Notes

1. p_{ovh} = Roof pressure at overhang for interior, edge, or corner zone as applicable from figures in roof pressure table.
2. p_{ovh} from figures includes load from top and bottom surface of overhang.
3. Pressure p_s at soffit of overhang shall be taken as equal to the wall pressure p_w .

FIGURE 30.6-2 Components and Cladding, Part 4 [$h \leq 160$ ft ($h \leq 18.3$ m)]: Roof Overhang Wind Loads for Enclosed Simple Diaphragm Buildings—Application of Roof Overhang Wind Loads

Diagram



Notes

1. p_{ovh} = Roof pressure at overhang for interior, edge, or corner zone as applicable from figures in roof pressure table.
2. p_{ovh} from figures includes load from top and bottom surface of overhang.
3. Pressure p_s at soffit of overhang shall be taken as equal to the wall pressure p_w .

FIGURE 30.6-2 Components and Cladding, Part 4 [$h \leq 160$ ft ($h \leq 18.3$ m)]: Roof Overhang Wind Loads for Enclosed Simple Diaphragm Buildings—Application of Roof Overhang Wind Loads

4.11 CHECK MEMBERS FOR 4" DIAMETER SPRINKLER LINES

STRUCTURAL CALCULATIONS

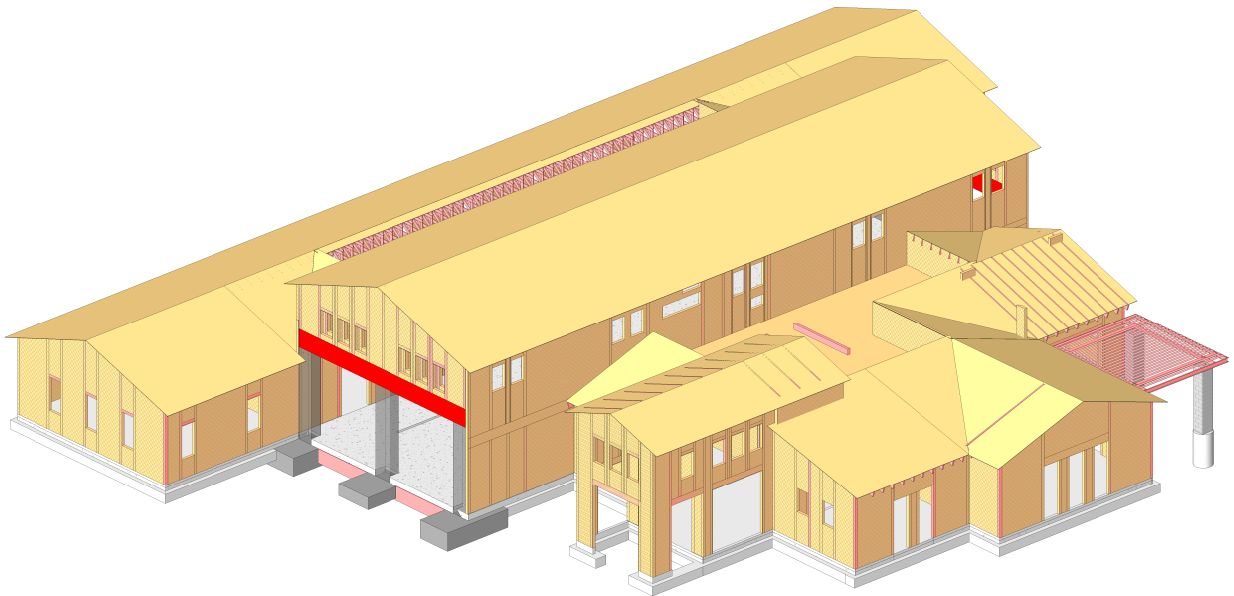
for

FIRE STATION 46

Valencia, CA

100% CD SUBMITTAL

**CHECK STRUCTURAL MEMBERS FOR 4" DIAMETER
FIRE SPRINKLER LINE**



Prepared by:



725 S. Figueroa St.
Los Angeles, California 90017
(213) 315-2277

SBI Job # 25534

May 05, 2026



ADDENDUM 5

TOLBrace™ Seismic Bracing Calculations

VB.8.144

Project Address: FIRE STATION 46

26720 BOMBEROA LANE

VALENCIA, CA

Job #

Contractor: Address:

Phone:

License:

EAT-ON

Powering Building Worldwide

Calculations based on 2020 NFPA Pamphlet #13

Brace Information

Maximum Brace Length 7' 0" (2.134 m)

Diameter of Brace 1"

Type of Brace Sch.40

Angle of Brace 75° Min.

Least Rad. of Gyration 0.42" (11 mm)

L/R Value 200

Max Horizontal Load 1604 lbs (728 kg)

TOLCO™ Brace Components

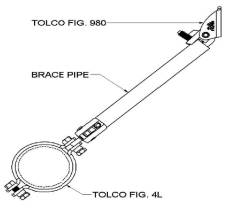
TOLCO™ Component	Listed Load	Adjusted Load
Fig. 4L Clamp	1000 lbs (454 kg)	966 lbs (438 kg)
Fig. 980 - 5/8" Universal Swivel	2100 lbs (953 kg)	2028 lbs (920 kg)

See Fastener Information

*Calculation Based on CONCENTRIC Loading

*Please Note: These calculations are for TOLCO™ components only. Use of any other components voids these calculations and the listing of the assembly.

Seismic Brace Assembly Detail



Brace Identification on Plans 4" BR-1

Brace Type Lateral [X] Longitudinal [] 4-Way []

Fastener Information

Orientation to Connecting Surface NFPA Type F

Fastener

Type 5/8in. x 5 1/2in. Thru Bolt

Diameter 5/8in.

Length 5 1/2in.

Maximum Load 960 lbs (435 kg)

Prying Factor N/A

Sprinkler System Load Calculation (Fpw = CpWp)

Cp = 1.27

Diameter	Type	Length	Total Length	Weight Per Unit Length	Total Weight
4" (100 mm)	Sch. 10	25 ft (7.6 m)	25 ft (7.6 m)	11.78 lb/ft (1.753 kg/m)	294 lbs (133 kg)
1.25" (32 mm)	Sch. 40	80 ft (24.4 m)	80 ft (24.4 m)	2.93 lb/ft (.436 kg/m)	234 lbs (106 kg)

Subtotal Weight 528 lbs (240 kg)

Wp (incl. 15%) 607 lbs (275 kg)

Total (Fpw) 771 lbs (350 kg)

Main Size 4" Type/Sch. Sch. 10 Spacing (ft) 25

Maximum Fpw per 18.5.52 (if applicable) 793 lb (359 kg)

TOLBrace™ Version 8)

Use of TOLBrace™ is subject to terms and conditions per the end user license agreement

TOLBrace™ Seismic Bracing Calculations

VB.8.144

Project Address: FIRE STATION 46

26720 BOMBEROA LANE

VALENCIA, CA

Job #

Contractor: Address:

Phone:

License:

EAT-ON

Powering Building Worldwide

Calculations based on 2020 NFPA Pamphlet #13

Brace Information

Maximum Brace Length 7' 0" (2.134 m)

Diameter of Brace 1"

Type of Brace Sch.40

Angle of Brace 75° Min.

Least Rad. of Gyration 0.42" (11 mm)

L/R Value 200

Max Horizontal Load 1604 lbs (728 kg)

TOLCO™ Brace Components

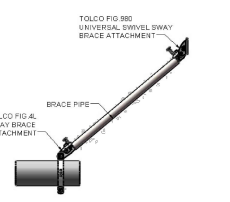
TOLCO™ Component	Listed Load	Adjusted Load
Fig. 4L Clamp	2000 lbs (907 kg)	1932 lbs (876 kg)
Fig. 980 - 5/8" Universal Swivel	2100 lbs (953 kg)	2028 lbs (920 kg)

See Fastener Information

*Calculation Based on CONCENTRIC Loading

*Please Note: These calculations are for TOLCO™ components only. Use of any other components voids these calculations and the listing of the assembly.

Seismic Brace Assembly Detail



Brace Identification on Plans 4" BR-2

Brace Type Lateral [] Longitudinal [X] 4-Way []

Fastener Information

Orientation to Connecting Surface NFPA Type F

Fastener

Type 5/8in. x 5 1/2in. Thru Bolt

Diameter 5/8in.

Length 5 1/2in.

Maximum Load 960 lbs (435 kg)

Prying Factor N/A

Sprinkler System Load Calculation (Fpw = CpWp)

Cp = 1.27

Diameter	Type	Length	Total Length	Weight Per Unit Length	Total Weight
4" (100 mm)	Sch. 10	40 ft (12.2 m)	40 ft (12.2 m)	11.78 lb/ft (1.753 kg/m)	471 lbs (214 kg)

Subtotal Weight 471 lbs (214 kg)

Wp (incl. 15%) 542 lbs (246 kg)

Total (Fpw) 688 lbs (312 kg)

Main Size 4" Type/Sch. Sch. 10 Spacing (ft) 40

Maximum Fpw per 18.5.52 (if applicable) N/A

TOLBrace™ Version 8)

Use of TOLBrace™ is subject to terms and conditions per the end user license agreement

FOR 4" DIAMETER PIPE LINE , HANGER SPACING SHALL NOT EXCEED 8'-0" O.C TYP

FIRE SPRINKLER FLOOR PLAN

SCALE: 1/8"=1'-0"



F5.1

F2.0



ADDENDUM 5

Check wood truss for hanger load.

Pipe parallel at bottom chord:

truss load on S0.35  exclude gyp ceiling per loading criteria
pipe with 12 lb/ft < (26 lb/ft - 3 psf * 2) = 20 lb/ft OK  MEP load truss design

Pipe parallel at top chord

truss load on S0.35  Exclude plywood & roofing per loading criteria
pipe with 12 lb/ft < (33 lb/ft - (8 psf * 2)) = 17 lb/ft OK  MEP load truss design

Pipe perpendicular to truss


Pipe hanger @ 8'-0"

P hanger = 12 lb/ft * 8 = 96 lb < Pmax = 200 lb (truss load on S0.35) OK


or P hanger / (distance between panel point) = 96 lb / 8 ft = 12 lb/ft < 17 lb/ft per note #11 on sheet S0.35

Check wood truss for brace load.

Pipe brace at bottom chord:

truss live load on S0.35  MEP load truss design see above
Pipe brace with 960 lb / 25 ft = 38 lb/ft < (40 lb/ft + 20 lb/ft) = 60 lb/ft OK

Pipe brace at top chord

truss live load on S0.35  MEP load truss design see above
Pipe brace with 960 lb / 25 ft = 38 lb/ft < (40 lb/ft + 17 lb/ft) = 47 lb/ft OK

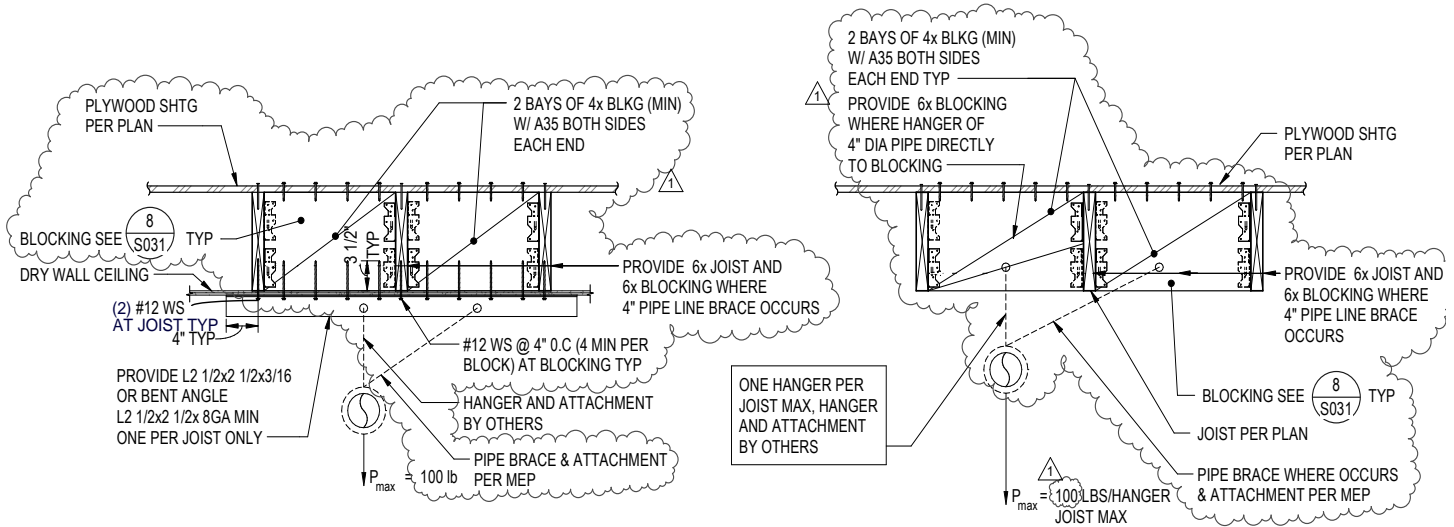
Check connection.

Pmax = 960 lb ,

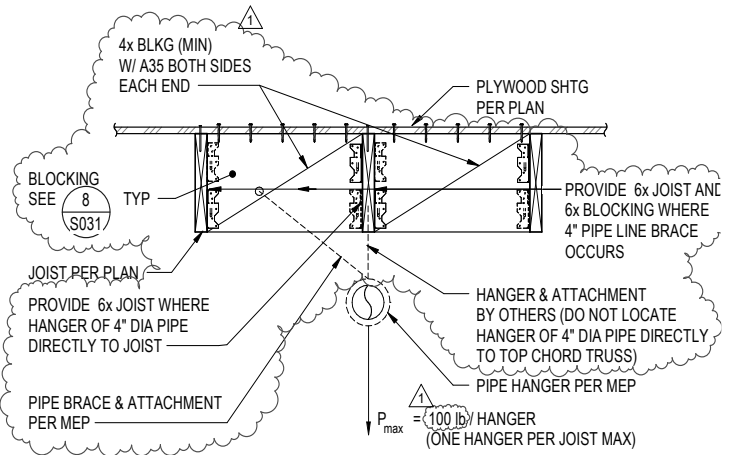
Shear Check: number of #12 WS = 960 lb / 162 lb * 1.6 = 3.7

Tension Check : Number of #12 WS = 960 lb / 157 lb * 3" * 1.6 = 1.7

screws per detail 8 is OK



ALTERNATE ATTACHMENT OPTION



CONDITION AT SAWN LUMBER JOIST OR TOP CHORD TRUSS FRAMING

TYPICAL PIPE HANGER DETAIL

SCALE: NTS

7

Check wood joist for hanger load.

Pipe parallel to joist

MEP design load



pipe with 12 lb/ft < (13 psf-3 psf)* 2= 20 lb/ft

Pipe perpendicular to joist

Pipe hanger @ 8'-0"

$P = 12 \text{ lb/ft} \times 8 \text{ ft} = 96 \text{ lb}$

$96 \text{ lb} / 16 \text{ ft} = 6 \text{ lb/ft} < 20 \text{ lb/ft}$ OK

Check wood joists for brace load.

Pipe brace P= 960 lb (seismic) (25 ft of spacing braces)

Typical joist 2x12 @ 24" o.c span 12' long

Design Roof LL = 20 psf

Degin_DL (MEP + Sprinkler) = 5 psf + 4.5 psf= 9.5 psf

Total design load=(20 psf + 9.5 psf) = 29.5 psf

Total designed load per joist

$29.5 \text{ psf} \times 2 \text{ ft} \times 12 \text{ ft} = 708 \text{ lb} < P = 960 \text{ lb}$

2x12 is NG at brace, increasing to 6x12 at brace only OK

Check connection.

$P_{\max} = 960 \text{ lb}$,

Shear Check: number of #12 WS = $960 \text{ lb} / 162 \text{ lb} \times 1.6 = 3.7$

Tension Check : Number of #12 WS= $960 \text{ lb} / 157 \text{ lb} / \text{in} \times 3 \text{ in} \times 1.6 = 1.7$

screws per detail 7 is OK