SFA Design Group, LLC

STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

Portland, OR | Seattle, WA

p: (503) 641-8311 www.sfadg.com

PRCTI20242007

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

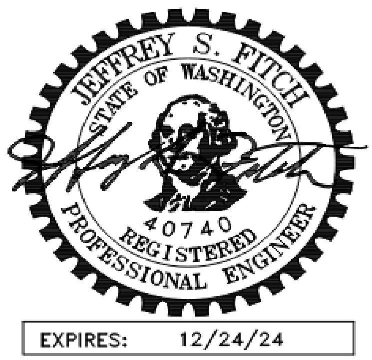
Building	Planning
Engineering	Public Works
Fire	Traffic

These calculations must be on site and made available by the Permittee for all inspections.

STRUCTURAL CALCULATIONS

Don's Drive-In Repair

925 S Meridian, Puyallup, WA 98371



LIMITATIONS

ENGINEER WAS RETAINED IN A LIMITED CAPACITY FOR THIS PROJECT. DESIGN IS BASED UPON INFORMATION PROVIDED BY THE CLIENT WHO IS SOLELY RESPONSIBLE FOR ACCURACY OF SAME. NO RESPONSIBILITY AND/OR LIABILITY IS ASSUMED BY, OR IS TO BE ASSIGNED TO THE ENGINEER FOR ITEMS BEYOND THAT SHOWN ON THESE SHEETS.

Project No. 24-007
May 20, 2024

SFA Design Group, LLC
STRUCTURAL | CIVIL | LAND USE PLANNING

PROJECT NO.	SHEET NO.
24-007	

PROJECT	DATE
Don's Drive-In Repair	5/20/2024
SUBJECT	BY
Table of Contents	CVG

SFA Design Group, LLC
STRUCTURAL | CIVIL | LAND USE PLANNING

PROJECT		PROJECT NO.	SHEET NO.
		24-007	
PROJECT: Don's Drive-In Repair		DATE: 4/17/2024	
SUBJECT: Design Criteria		BY: CVG	

Structural Narrative

The existing structure is a one-story restaurant. The gravity system consists of 2x10 wood joists and sistered 2x12 wood beams over wood columns, and 4x8x16 unreinforced CMU block walls. The foundation system consists of slab on grade and continuous footing is assumed at the perimeter of the structure. The lateral system consists of wood roof diaphragm (in the form of diagonal decking) over unreinforced CMU shearwalls. A car collided with the north facing wall, and damaged 11'-9" length of wall. Scope of work consists of replacing damaged wall to pre-damaged condition (per WSEBC section 405.2.1), and support of new hood, make-up air unit and exhaust fan.

General

Building Department	City of Puyallup
Building Code	2021 IBC
Risk Category	II
Project Site Latitude/Longitude	47.18330/-122.29320

Dead Loads

(E) Roof Dead Load

Roofing	4.0 psf
Diagonal Decking	2.0 psf
Loose Insulation	1.0 psf
2x10 at 16" oc	2.5 psf
Electrical and Mechanical	1.0 psf
5/8" Gyp Ceiling	2.7 psf
Miscellaneous	1.8 psf
Total	**15.0 psf**

Live Loads

Roof Snow Load	25 psf

Deflections

Total Load Deflection Limit	L/240
Live Load Deflection Limit	L/360

SFA Design Group, LLC
STRUCTURAL | CIVIL | LAND USE PLANNING

PROJECT NO.	SHEET NO.
24-007	

PROJECT Don's Drive-In Repair	DATE 4/17/2024
SUBJECT Design Criteria	BY CVG

Wind Loads

Wind Speed (3-Second Gust)	97.0 mph
Exposure	B
Elevation	44 ft
Topography Factor (K_{zt})	1.0 (Flat)

Seismic Loads

Seismic Design Category	D
Site Soil Classification	D
Seismic Importance Factor	1.0
Response Modification Coefficient	1.5
Basic Seismic Force Resisting System	Ordinary plain masonry shearwalls
Mapped MCE Spectral Response Acceleration (Short Periods, S_S)	126.900%
Mapped MCE Spectral Response Acceleration (1 Second, S_1)	43.700%
Design Spectral Response Acceleration (Short Periods, S_{DS})	101.500%
Seismic Response Coefficient, C_s	0.677
Design Base Shear	30320 lbs
Analysis Procedure Used	Equivalent Lateral Force
Special Seismic Ordinances/Notes:	None

Soil Parameters

Allowable Soil Bearing Pressure	1500 psf
1/3 Increase for Wind/Earthquake forces?	Yes
Minimum Frost Depth	12 in
Coefficient of Friction	0.30

Additional Ordinances/Notes:	None

SFA Design Group, LLC
STRUCTURAL | CIVIL | LAND USE PLANNING

PROJECT NO.	SHEET NO.
24-001	

PROJECT: DON'S DRIVE-IN
DATE: 4/2024
SUBJECT: BUILDING EVALUATION
BY: CNG

DAMAGED WALL LENGTH:

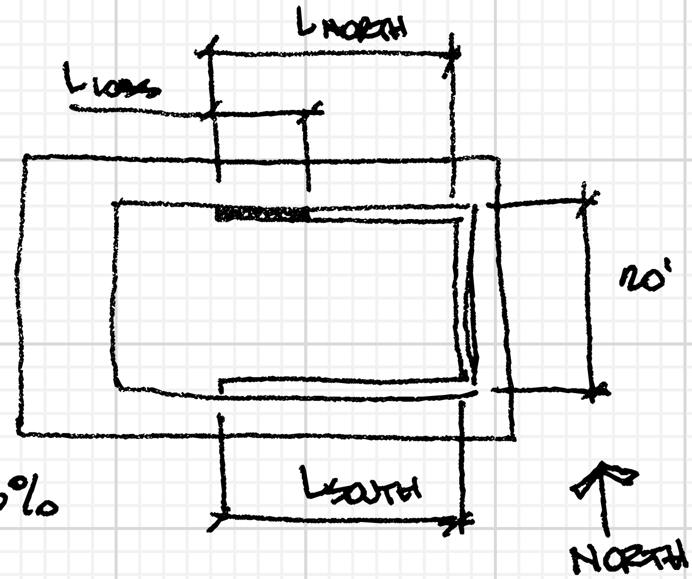
$L_{WALL\ LOSS} = 11'\text{-}9''$ DUE TO CAR COLLISION TO NORTH-FACING WALL

PREDAMAGED WALL LENGTH:

$L_{NORTH} = 27'\text{-}0''$

$L_{SOUTH} = 27'\text{-}9''$

$\overline{54'\text{-}9''}$



LATERAL CAPACITY:

$$\frac{11.75}{54.75} = 21.5\% < 33\%$$

∴ NOT A "SUBSTANTIAL STRUCTURAL DAMAGE" AS DEFINED IN WASHINGTON STATE EXISTING BUILDING CODE

∴ DAMAGED WALL CAN BE RESTORED TO PRE-DAMAGED CONDITION PER WASHINGTON STATE EXISTING BUILDING CODE SECTION 405.2.1

Wood Beam

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

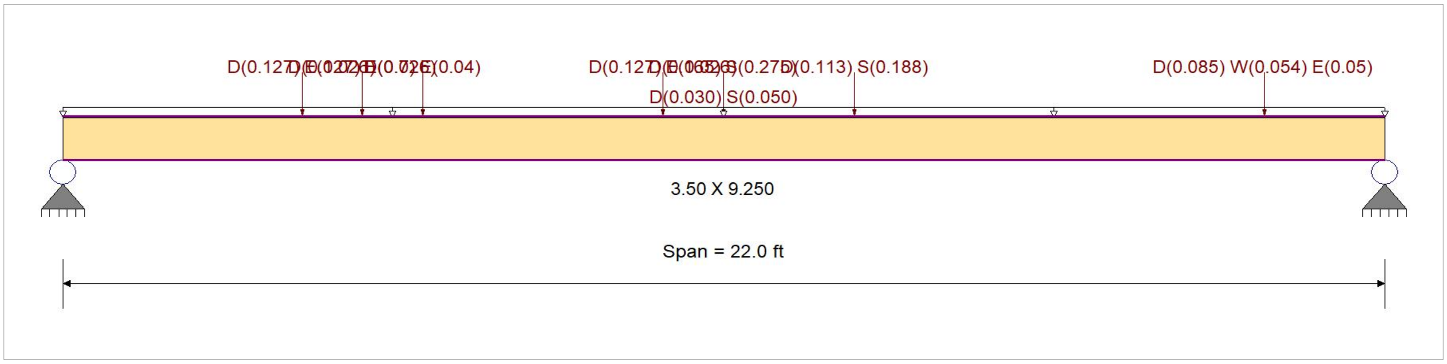
DESCRIPTION: (E) Roof Joist with Mech Unit Loads

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 +	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	2,600.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade :	MicroLam LVL 2.0 E	Fv	285.0 psi		
		Ft	1,555.0 psi	Density	42.010 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 calculated and added to loading
Loads on all spans...
  Uniform Load on ALL spans : D = 0.0150, S = 0.0250 ksf, Tributary Width = 2.0 ft
  Point Load : D = 0.70, E = 0.040 k @ 6.0 ft, (MUA load)
  Point Load : D = 0.1270, E = 0.0260 k @ 4.0 ft, (hood)
  Point Load : D = 0.1270, E = 0.0260 k @ 5.0 ft, (hood)
  Point Load : D = 0.1270, E = 0.0260 k @ 10.0 ft, (hood)
  Point Load : D = 0.1650, S = 0.2750 k @ 11.0 ft, (roof beam reaction at opening)
  Point Load : D = 0.1130, S = 0.1880 k @ 13.170 ft, (roof beam reaction at opening)
  Point Load : D = 0.0850, W = 0.0540, E = 0.050 k @ 20.0 ft, (1/2 hood)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.974** : 1	Maximum Shear Stress Ratio	=	**0.290** : 1
Section used for this span		**3.50 X 9.250**	Section used for this span		**3.50 X 9.250**
fb: Actual	=	3,017.78 psi	fv: Actual	=	94.90 psi
F'b	=	3,097.74 psi	F'v	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	11.000 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	0.952 in	Ratio =	277 >=240	Span: 1 : S Only
Max Upward Transient Deflection	-0.083 in	Ratio =	3186 >=240	Span: 1 : E Only * -1.0
Max Downward Total Deflection	1.361 in	Ratio =	194 >=180	Span: 1 : D Only
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.350	0.962
Max Upward from Load Cases	1.350	0.962
D Only	1.350	0.962
S Only	0.763	0.800
W Only	0.005	0.049
E Only	0.089	0.079

Wood Beam

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC (c) ENERCALC INC 1983-2023

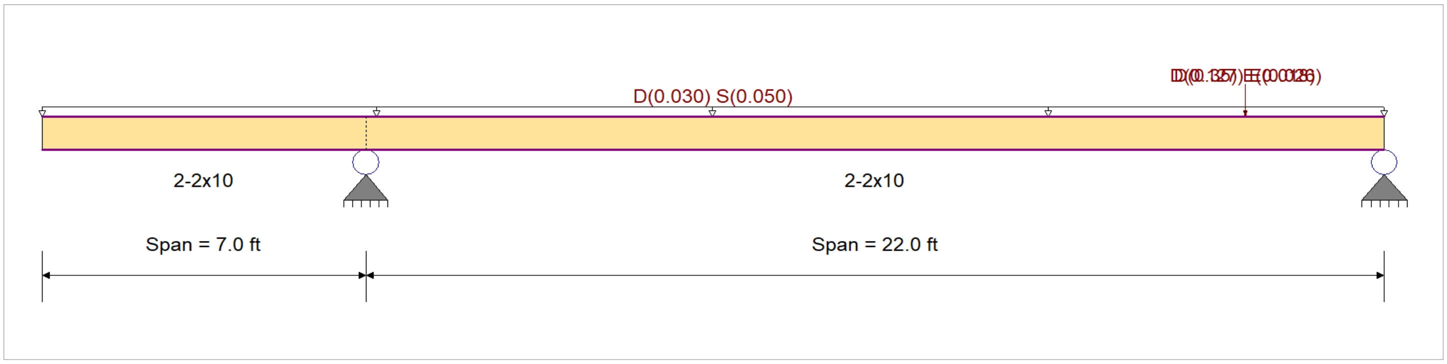
DESCRIPTION: (E) Cant Roof Joist with Mech Unit Loads

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 +	1,200.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,200.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,550.0 psi	Eminbend - xx	660.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1 & Better	Fv	180.0 psi		
		Ft	800.0 psi	Density	31.210 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
Loads on all spans...
Uniform Load on ALL spans : D = 0.0150, S = 0.0250 ksf, Tributary Width = 2.0 ft
Load for Span Number 2
Point Load : D = 0.1270, E = 0.0260 k @ 19.0 ft, (hood)
Point Load : D = 0.350, E = 0.0180 k @ 19.0 ft, (50% of MUA load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.806** : 1	Maximum Shear Stress Ratio	=	**0.313** : 1
Section used for this span		**2-2x10**	Section used for this span		**2-2x10**
fb: Actual	=	1,406.97 psi	fv: Actual	=	64.81 psi
F'b	=	1,745.70 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	12.905 ft	Location of maximum on span	=	21.263 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.565 in	Ratio =	466	>=360	Span: 2 : S Only
Max Upward Transient Deflection	-0.375 in	Ratio =	446	>=360	Span: 2 : E Only * -1.0
Max Downward Total Deflection	0.617 in	Ratio =	428	>=240	Span: 2 : D Only
Max Upward Total Deflection	-0.445 in	Ratio =	376	>=240	Span: 1 : D Only

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		0.956	0.768
Max Upward from Load Cases		0.956	0.768
D Only		0.753	0.768
S Only		0.956	0.494
E Only		0.006	0.038

Wood Beam

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 | SFA ENGINEERING LLC | (c) ENERCALC INC 1983-2023

DESCRIPTION: (E) Dropped Beam with Mech Units

CODE REFERENCES

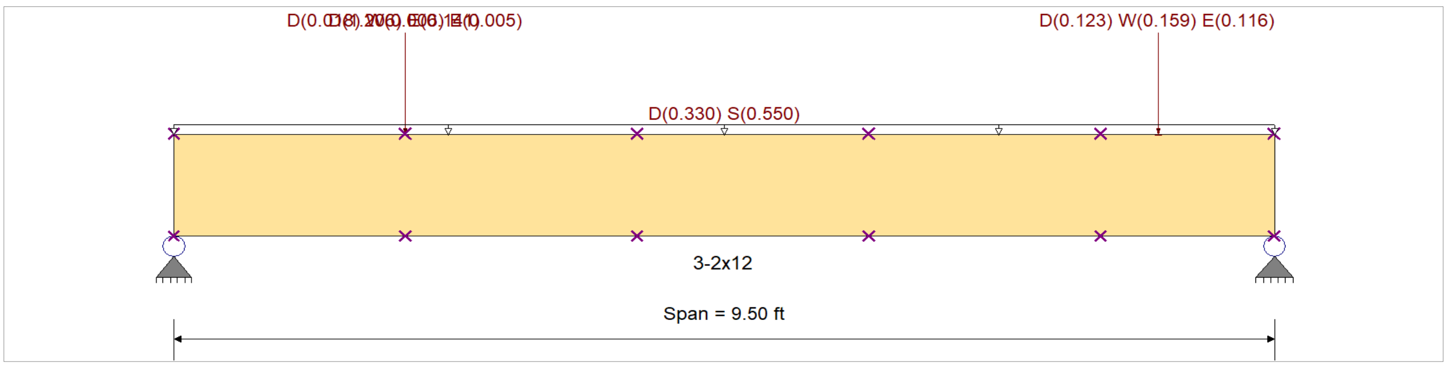
Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,200.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,200.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,550.0 psi	Eminbend - xx	660.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1 & Better	Fv	180.0 psi		
		Ft	800.0 psi	Density	31.210 pcf
Beam Bracing :	Beam bracing is defined as a set spacing over all spans				

Unbraced Lengths

First Brace starts at 2.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 2.0 ft



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0150, S = 0.0250 ksf, Tributary Width = 22.0 ft
Point Load : D = 1.206, E = 0.1410 k @ 2.0 ft, (MUA and hood)
Point Load : D = 0.1230, W = 0.1590, E = 0.1160 k @ 8.50 ft, (27% HVAC unit (vert loads) & OT)
Point Load : D = 0.0180, W = 0.0060, E = 0.0050 k @ 2.0 ft, (10% condenser unit)

Sistered member is 2.0E LVL, therefore OK by inspection

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	**1.033** : 1	Maximum Shear Stress Ratio	=	**0.621** : 1
Section used for this span		**3-2x12**	Section used for this span		**3-2x12**
fb: Actual	=	1,421.85 psi	fv: Actual	=	128.46 psi
F'b	=	1,376.53 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.473 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	0.105 in	Ratio =	1080	>=240	Span: 1 : S Only
Max Upward Transient Deflection	-0.004 in	Ratio =	28621	>=240	Span: 1 : E Only * -1.0
Max Downward Total Deflection	0.088 in	Ratio =	1293	>=180	Span: 1 : D Only
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.613	2.613
Max Upward from Load Cases	2.613	2.613
D Only	2.547	1.935
S Only	2.613	2.613
W Only	0.021	0.144
E Only	0.127	0.135

Wood Beam

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC (c) ENERCALC INC 1983-2023

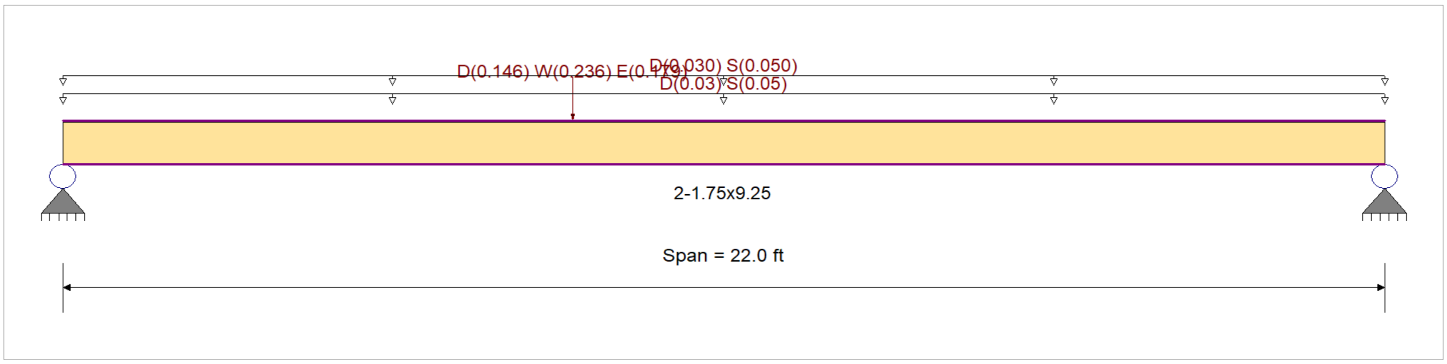
DESCRIPTION: (E) Roof Joist with HVAC (Supported by ((5) Joists Min)

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 +	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	2,600.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade :	MicroLam LVL 2.0 E	Fv	285.0 psi		
		Ft	1,555.0 psi	Density	42.010 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 NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0150, S = 0.0250 ksf, Tributary Width = 2.0 ft
Point Load : D = 0.1460, W = 0.2360, E = 0.1790 k @ 8.50 ft, (HVAC Unit)
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	**0.769**: 1	Maximum Shear Stress Ratio	=	**0.245** : 1
Section used for this span		**2-1.75x9.25**	Section used for this span		**2-1.75x9.25**
fb: Actual	=	2,478.88 psi	fv: Actual	=	80.34 psi
F'b	=	3,221.65 psi	F'v	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	10.679 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	1.148 in	Ratio =	229 <240	Span: 1 : S Only
Max Upward Transient Deflection	-0.184 in	Ratio =	1434 >=240	Span: 1 : -W
Max Downward Total Deflection	0.802 in	Ratio =	329 >=180	Span: 1 : D Only
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.850	1.816
Max Upward from Load Combinations	1.850	1.816
Max Upward from Load Cases	1.100	1.100
D Only	0.750	0.716
+D+S	1.850	1.816
+D+0.750S	1.575	1.541
+D+0.60W	0.836	0.771
+D+0.450W	0.815	0.757
+D+0.750S+0.450W	1.640	1.582
+0.60D+0.60W	0.537	0.485
+D+0.70E	0.826	0.765

Wood Beam

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: (E) Roof Joist with HVAC (Supported by ((5) Joists Min)

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750S+0.5250E	1.632	1.578
+0.60D+0.70E	0.527	0.478
S Only	1.100	1.100
W Only	0.145	0.091
E Only	0.110	0.069

Wood Beam

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: (E) Dropped Beam [For Load Generation Purposes Only]

CODE REFERENCES

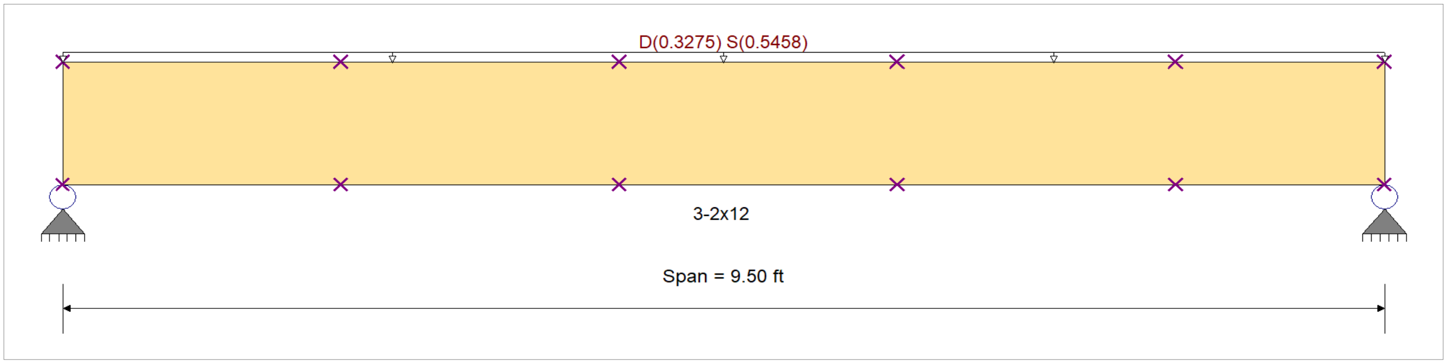
Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,200.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,200.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,550.0 psi	Eminbend - xx	660.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1 & Better	Fv	180.0 psi		
		Ft	800.0 psi	Density	31.210 pcf
Beam Bracing :	Beam bracing is defined as a set spacing over all spans				

Unbraced Lengths

First Brace starts at 2.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 2.0 ft



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.0150, S = 0.0250 ksf, Tributary Width = 21.830 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.916** : 1	Maximum Shear Stress Ratio	=	**0.483** : 1
Section used for this span		**3-2x12**	Section used for this span		**3-2x12**
fb: Actual	=	1,260.98 psi	fv: Actual	=	99.91 psi
F'b	=	1,376.53 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.750 ft	Location of maximum on span	=	8.564 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.105 in	Ratio =	1089 >=240	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <240	n/a
Max Downward Total Deflection	0.065 in	Ratio =	1756 >=180	Span: 1 : D Only
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	4.200	4.200
Max Upward from Load Combinations	4.200	4.200
Max Upward from Load Cases	2.592	2.592
D Only	1.608	1.608
+D+S	4.200	4.200
+D+0.750S	3.552	3.552
S Only	2.592	2.592

Wood Beam

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: (E) Dropped Beam w/ HVAC Loads

CODE REFERENCES

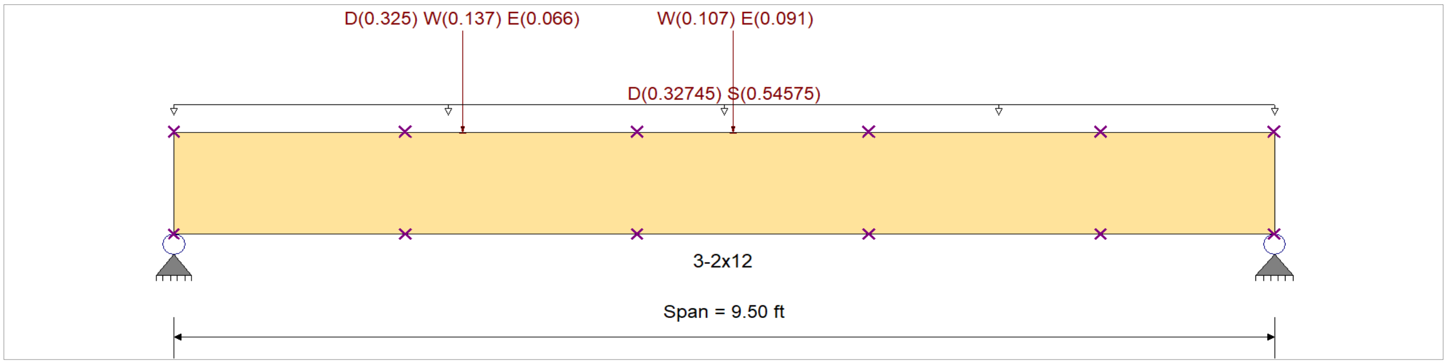
Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,200.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,200.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,550.0 psi	Eminbend - xx	660.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1 & Better	Fv	180.0 psi		
		Ft	800.0 psi	Density	31.210 pcf
Beam Bracing :	Beam bracing is defined as a set spacing over all spans				

Unbraced Lengths

First Brace starts at 2.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 2.0 ft



Applied Loads

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

Beam self weight NOT internally calculated and added
Point Load : D = 0.3250, W = 0.1370, E = 0.0660 k @ 2.50 ft
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 21.830 ft, (existing roof)
Point Load : W = 0.1070, E = 0.0910 k @ 4.830 ft, (due to OT)

use 1.75x9.25 2.0E LVL sistered to (E) dropped beam

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.942** : 1	Maximum Shear Stress Ratio	=	**0.511** : 1
Section used for this span		**3-2x12**	Section used for this span		**3-2x12**
fb: Actual	=	1,297.22 psi	fv: Actual	=	105.77 psi
F'b	=	1,376.53 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.646 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	0.105 in	Ratio =	1089 >=240	Span: 1 : S Only
Max Upward Transient Deflection	-0.007 in	Ratio =	17153 >=240	Span: 1 : -W
Max Downward Total Deflection	0.070 in	Ratio =	1620 >=180	Span: 1 : D Only
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	4.387	4.233
Max Upward from Load Combinations	4.387	4.233
Max Upward from Load Cases	2.592	2.592
D Only	1.795	1.641
+D+S	4.387	4.233
+D+0.750S	3.739	3.585
S Only	2.592	2.592
W Only	0.154	0.090
E Only	0.093	0.064

comparison with existing loads, new loads are less than 5% over, therefore existing beam connections OK

Masonry Column

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: Column Load at CMU Wall

Code References

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Material Properties

F'm	=	1,500.0 psi
Fr - Rupture	=	75.0 psi
Em = f'm *	=	900.0
Column Density	=	130.0 pcf
Rebar Grade	=	Grade 60
Fy - Yield	=	60000 psi
Fs - Allowable	=	32,000.0 psi
E - Rebar	=	29,000.0 ksi

Column Data

Column width along X-X	=	7.625 in
Column depth along Y-Y	=	7.625 in
Longitudinal Bar Size	=	# 4.0
Bars per side at +Y & -Y	=	2
Bars per side at +X & -X	=	2
Cover from ties	=	3.50 in
Actual Edge to Bar Center	=	4.125 in

Analysis Settings

Analysis Method	=	**Allowable Design**
End Fixity Condition	=	Top Pinned, Bottom Pinned
Overall Column Height	=	9.0 ft
Construction Type		Solid Grouted Hollow Concrete Masor
Tie Bar Size	=	# 3.0
Tie Bar Spacing	=	7.625 in

Brace condition for deflection (buckling) along columns :

X-X (width) axis : Unbraced Length for buckling ABOUT Y-Y Axis = 9.0 ft, K =
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 9.0 ft, K =

Applied Loads

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

Column self weight included : 472.393 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = 4.0 in, D = 2.589, S = 2.613, E = 0.1110 k
BENDING LOADS . . .

DESIGN SUMMARY

Bending Check Results

PASS **Maximum Bending Stress Ratio =** **0.844** : 1

Load Combination	+D+S
Location of max.above base	8.940 ft
At maximum location values are . . .	
Axial - Applied	5.674 k
Axial - Allowable	6.733 k
Moment - Applied	-1.722 k-ft
Moment - Allowable	2.014 k-ft

PASS **Reinforcing Area Check** *(TMS 402-16, Sec 5.3.1.3*

As : Actual Reinforcement	0.800
Min: 0.0025 * An	0.145
Max: 0.04 * An	2.326

Dimensional Checks

Min. Side Dim. >= 8" *(TMS 402-16, Sec. 5.3.1.*
PASS Governing K * Lu / Dimension <= *(TMS 402-16, Sec. 5.3.1.*

Maximum SERVICE Load Reactions . .

Top along X-X	0.193 k
Bottom along X-X	0.193 k

Maximum SERVICE Load Deflections . . .

Along x-x 0.041 in at 5.255 ft above base
for load combination : +D+S

Compressive Strength 33.468 k *(TMS 402-16, Sec. 9.3.4.*
Pa = (0.25 f'm An + 0.65 Ast Fs) * [1-(h/(140*r))^2]

PASS **Check Column Ties** *(TMS 402-16, Sec 5.3.1.4)*

Min. Tie Dia. = 1/4", # 3 bar provided
Max Tie Spacing = 7.63 in, Provided = 7.63 in

Load Combination Results

Load Combination	Maximum Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Axial Load: Actual	Allow	Maximum Moments: Actual	Allow
D Only	0.4169	PASS	8.940 ft	3.061 k	7.346 k	0.8572 k-ft	2.045 k-ft
+D+S	0.8438	PASS	8.940 ft	5.674 k	6.733 k	1.722 k-ft	2.014 k-ft
+D+0.750S	0.7459	PASS	8.940 ft	5.021 k	6.733 k	1.506 k-ft	2.014 k-ft
+0.60D	0.2614	PASS	0.0 ft	1.837 k	6.733 k	0.0 k-ft	2.014 k-ft
+D+0.70E	0.4276	PASS	8.940 ft	3.139 k	7.346 k	0.8829 k-ft	2.045 k-ft
+D+0.750S+0.5250E	0.7546	PASS	8.940 ft	5.079 k	6.733 k	1.525 k-ft	2.014 k-ft
+0.60D+0.70E	0.2724	PASS	0.0 ft	1.915 k	6.733 k	0.0 k-ft	2.014 k-ft

Masonry Column

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: Column Load at CMU Wall

Maximum Reactions

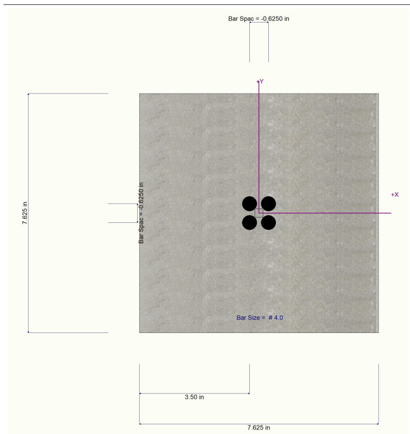
Note: Only non-zero reactions are listed.

Load Combination	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base
D Only	-0.096 k	0.096 k	3.061 k
+D+S	-0.193 k	0.193 k	5.674 k
+D+0.750S	-0.168 k	0.168 k	5.021 k
+0.60D	-0.058 k	0.058 k	1.837 k
+D+0.70E	-0.099 k	0.099 k	3.139 k
+D+0.750S+0.5250E	-0.171 k	0.171 k	5.079 k
+0.60D+0.70E	-0.060 k	0.060 k	1.915 k
S Only	-0.097 k	0.097 k	2.613 k
E Only	-0.004 k	0.004 k	0.111 k

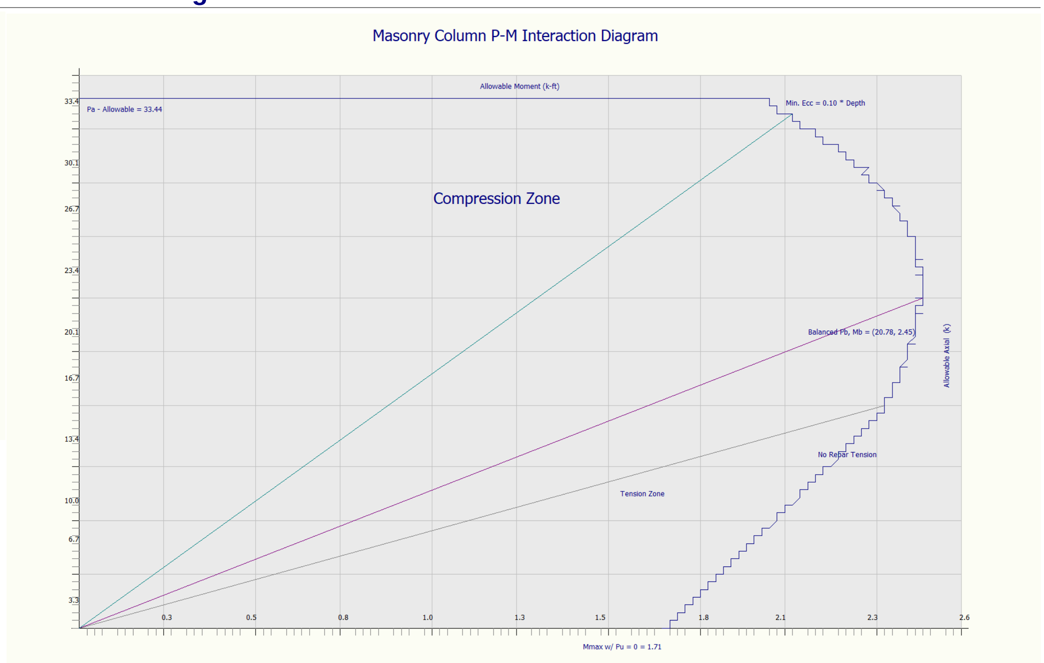
Maximum Deflections for Load Combinations

Load Combination	Max. Y-Y Deflection	Distance
D Only	0.0206 in	5.255 ft
+D+S	0.0413 in	5.255 ft
+D+0.750S	0.0361 in	5.255 ft
+0.60D	0.0123 in	5.255 ft
+D+0.70E	0.0212 in	5.255 ft
+D+0.750S+0.5250E	0.0366 in	5.255 ft
+0.60D+0.70E	0.0129 in	5.255 ft
S Only	0.0207 in	5.255 ft
E Only	0.0009 in	5.255 ft

Cross Section



Interaction Diagram



Wood Column

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: (E) Column

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				
End Fixities	Top & Bottom Pinned				
Overall Column Height			9 ft		
(Used for non-slender calculations)					
Wood Species	Douglas Fir-Larch				
Wood Grade	No.1 & Better				
Fb +	1,200.0 psi	Fv	180.0 psi		
Fb -	1,200.0 psi	Ft	800.0 psi		
Fc - Prll	1,550.0 psi	Density	31.210 pcf		
Fc - Perp	625.0 psi				
E : Modulus of Elasticity . . .		x-x Bending	y-y Bending	Axial	
	Basic	1,800.0	1,800.0	1,800.0 ksi	
	Minimum	660.0	660.0		

Wood Section Name	**4x6**
Wood Grading/Manuf.	Graded Lumber
Wood Member Type	Sawn
Exact Width	**3.50** in
Exact Depth	**5.50** in
Area	19.250 in^2
Ix	48.526 in^4
Iy	19.651 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compressio	1.10
Cf or Cv for Tension	1.30
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

Column Buckling Condition:
ABOUT X-X Axis: Lux = 9 ft, Kx = 1.0
ABOUT Y-Y Axis: Luy = 9 ft, Ky = 1.0

Applied Loads

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

Column self weight included : 37.550 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = 0.50 in, D = 3.730, S = 5.205, W = 0.2980, E = 0.2280 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.9848 : 1**
Load Combination	+D+S
Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base	8.940 ft
At maximum location values are .	
Applied Axial	8.973 k
Applied Mx	-0.3698 k-ft
Applied My	0.0 k-ft
Fc : Allowable	530.43 psi
PASS Maximum Shear Stress Ratio =	**0.01557 : 1**
Load Combination	+D+S
Location of max.above base	9.0 ft
Applied Design Shear	4.835 psi
Allowable Shear	207.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.04137 k	Bottom along Y-Y	0.04137 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y -0.03860 in at 5.255 ft above base
for load combination : +D+S
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

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.337	0.3786	PASS	8.940 ft	0.008306	PASS	9.0 ft
+D+S	1.150	0.271	0.9848	PASS	8.940 ft	0.01557	PASS	9.0 ft
+D+0.750S	1.150	0.271	0.7520	PASS	8.940 ft	0.01330	PASS	9.0 ft
+D+0.60W	1.600	0.199	0.3779	PASS	8.940 ft	0.004896	PASS	9.0 ft
+D+0.450W	1.600	0.199	0.3736	PASS	8.940 ft	0.004840	PASS	9.0 ft
+D+0.750S+0.450W	1.600	0.199	0.7477	PASS	8.940 ft	0.009730	PASS	9.0 ft
+0.60D+0.60W	1.600	0.199	0.2335	PASS	8.940 ft	0.003027	PASS	9.0 ft
+D+0.70E	1.600	0.199	0.3760	PASS	8.940 ft	0.004872	PASS	9.0 ft
+D+0.750S+0.5250E	1.600	0.199	0.7464	PASS	8.940 ft	0.009712	PASS	9.0 ft
+0.60D+0.70E	1.600	0.199	0.2317	PASS	8.940 ft	0.003003	PASS	9.0 ft

Wood Column

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: (E) Column

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	k-ft	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only				-0.017	0.017	3.768					
+D+S				-0.041	0.041	8.973					
+D+0.750S				-0.035	0.035	7.671					
+D+0.60W				-0.018	0.018	3.946					
+D+0.450W				-0.018	0.018	3.902					
+D+0.750S+0.450W				-0.036	0.036	7.805					
+0.60D+0.60W				-0.011	0.011	2.439					
+D+0.70E				-0.018	0.018	3.927					
+D+0.750S+0.5250E				-0.036	0.036	7.791					
+0.60D+0.70E				-0.011	0.011	2.420					
S Only				-0.024	0.024	5.205					
W Only				-0.001	0.001	0.298					
E Only				-0.001	0.001	0.228					

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	-0.016 in	5.255 ft
+D+S	0.0000 in	0.000 ft	-0.039 in	5.255 ft
+D+0.750S	0.0000 in	0.000 ft	-0.033 in	5.255 ft
+D+0.60W	0.0000 in	0.000 ft	-0.017 in	5.255 ft
+D+0.450W	0.0000 in	0.000 ft	-0.017 in	5.255 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	-0.034 in	5.255 ft
+0.60D+0.60W	0.0000 in	0.000 ft	-0.010 in	5.255 ft
+D+0.70E	0.0000 in	0.000 ft	-0.017 in	5.255 ft
+D+0.750S+0.5250E	0.0000 in	0.000 ft	-0.033 in	5.255 ft
+0.60D+0.70E	0.0000 in	0.000 ft	-0.010 in	5.255 ft
S Only	0.0000 in	0.000 ft	-0.022 in	5.255 ft
W Only	0.0000 in	0.000 ft	-0.001 in	5.255 ft
E Only	0.0000 in	0.000 ft	-0.001 in	5.255 ft

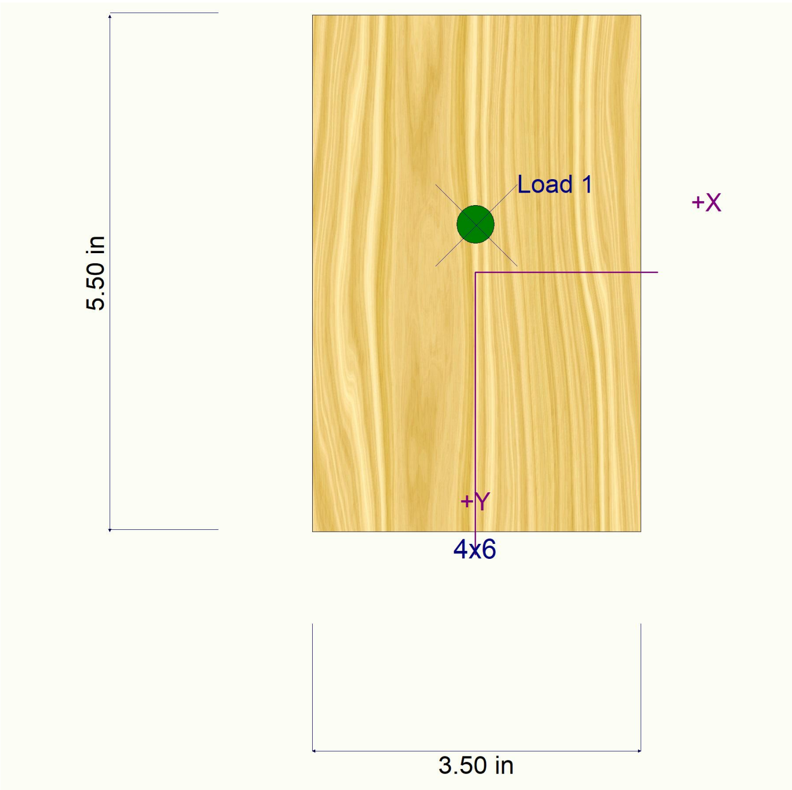
Wood Column

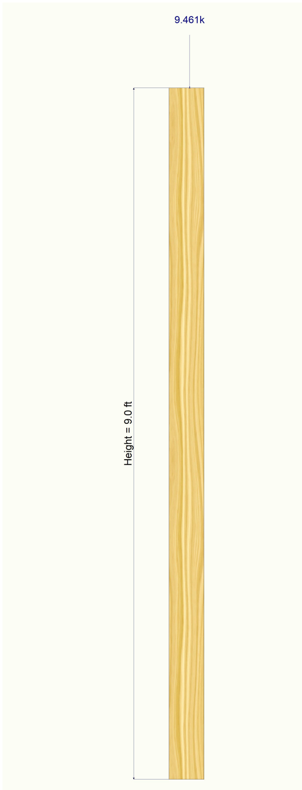
Project File: Dons Drivein calcs.ec6

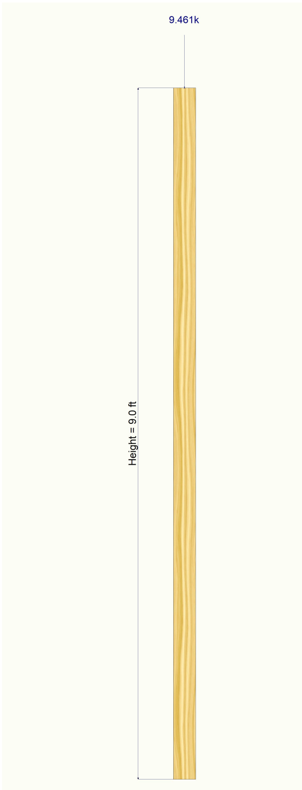
LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: (E) Column

Sketches







General Footing

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: (E) Footing Check

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	=	2.50	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	=	1.50	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

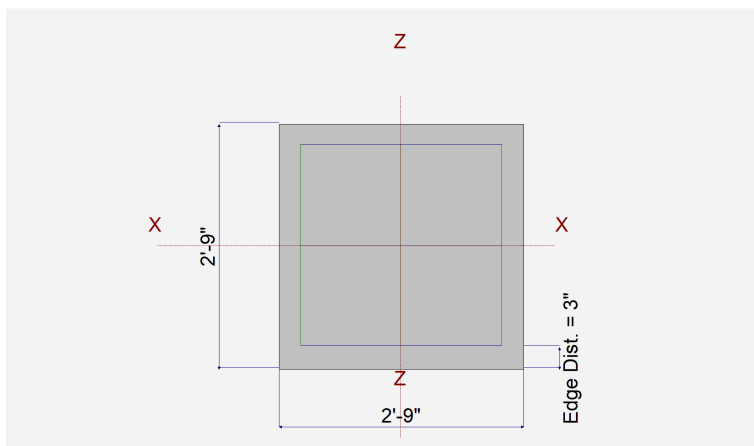
Footing base depth below soil surface	=		ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

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	=	2.750	ft
Length parallel to Z-Z Axis	=	2.750	ft
Footing Thickness	=	8.0	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...

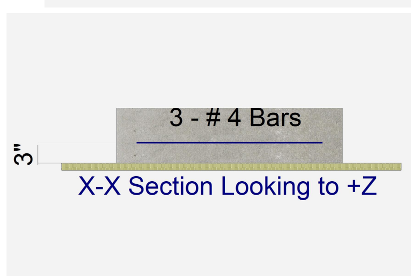
at Bottom of footing	=	3.0	in

Reinforcing

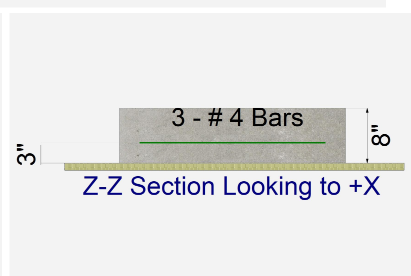
Bars parallel to X-X Axis			
Number of Bars	=		3.0
Reinforcing Bar Size	=	#	4
Bars parallel to Z-Z Axis			
Number of Bars	=		3.0
Reinforcing Bar Size	=	#	4

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



X-X Section Looking to +Z



Z-Z Section Looking to +X

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	3.730			5.205	0.2980	0.2280		k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: (E) Footing Check

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8520	Soil Bearing	1.278 ksf	1.50 ksf	+D+S 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	0.3477	Z Flexure (+X)	1.619 k-ft/ft	4.657 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.3477	Z Flexure (-X)	1.619 k-ft/ft	4.657 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.3477	X Flexure (+Z)	1.619 k-ft/ft	4.657 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.3477	X Flexure (-Z)	1.619 k-ft/ft	4.657 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.3663	1-way Shear (+X)	27.476 psi	75.0 psi	+1.20D+1.60S+0.50W
PASS	0.3663	1-way Shear (-X)	27.476 psi	75.0 psi	+1.20D+1.60S+0.50W
PASS	0.3663	1-way Shear (+Z)	27.476 psi	75.0 psi	+1.20D+1.60S+0.50W
PASS	0.3663	1-way Shear (-Z)	27.476 psi	75.0 psi	+1.20D+1.60S+0.50W
PASS	0.8414	2-way Punching	126.214 psi	150.0 psi	+1.20D+1.60S+0.50W

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	1.50	n/a	0.0	0.5899	0.5899	n/a	n/a	0.393
X-X, +D+S	1.50	n/a	0.0	1.278	1.278	n/a	n/a	0.852
X-X, +D+0.750S	1.50	n/a	0.0	1.106	1.106	n/a	n/a	0.737
X-X, +D+0.60W	1.50	n/a	0.0	0.6135	0.6135	n/a	n/a	0.409
X-X, +D+0.450W	1.50	n/a	0.0	0.6076	0.6076	n/a	n/a	0.405
X-X, +D+0.750S+0.450W	1.50	n/a	0.0	1.124	1.124	n/a	n/a	0.749
X-X, +0.60D+0.60W	1.50	n/a	0.0	0.3776	0.3776	n/a	n/a	0.252
X-X, +D+0.70E	1.50	n/a	0.0	0.6110	0.6110	n/a	n/a	0.407
X-X, +D+0.750S+0.5250E	1.50	n/a	0.0	1.122	1.122	n/a	n/a	0.748
X-X, +0.60D+0.70E	1.50	n/a	0.0	0.3750	0.3750	n/a	n/a	0.250
Z-Z, D Only	1.50	0.0	n/a	n/a	n/a	0.5899	0.5899	0.393
Z-Z, +D+S	1.50	0.0	n/a	n/a	n/a	1.278	1.278	0.852
Z-Z, +D+0.750S	1.50	0.0	n/a	n/a	n/a	1.106	1.106	0.737
Z-Z, +D+0.60W	1.50	0.0	n/a	n/a	n/a	0.6135	0.6135	0.409
Z-Z, +D+0.450W	1.50	0.0	n/a	n/a	n/a	0.6076	0.6076	0.405
Z-Z, +D+0.750S+0.450W	1.50	0.0	n/a	n/a	n/a	1.124	1.124	0.749
Z-Z, +0.60D+0.60W	1.50	0.0	n/a	n/a	n/a	0.3776	0.3776	0.252
Z-Z, +D+0.70E	1.50	0.0	n/a	n/a	n/a	0.6110	0.6110	0.407
Z-Z, +D+0.750S+0.5250E	1.50	0.0	n/a	n/a	n/a	1.122	1.122	0.748
Z-Z, +0.60D+0.70E	1.50	0.0	n/a	n/a	n/a	0.3750	0.3750	0.250

Seismic Design

Job Number: 24-007

ASCE 7-16 Chapters 11 & 12

Soil Site Class =	D (Default)		*Tab. 20.3-1, (Default = D)*
Response Spectral Acc. (0.2 sec) S_s =	126.90%g	= 1.269g	*Figs. 22-1, 22-3, 22-5, 22-6*
Response Spectral Acc.(1.0 sec) S_1 =	43.70%g	= 0.437g	*Figs. 22-2, 22-4, 22-5, 22-6*
Site Coefficient F_a		= 1.200	*Tab. 11.4-1*
Site Coefficient F_v		= 1.863	*Tab. 11.4-2*
Max Considered Earthquake Acc. S_{MS} =	$F_a.S_s$	= 1.523g	*Eq. (11.4-1)*
Max Considered Earthquake Acc. S_{M1} =	$F_v.S_1$	= 0.814g	*Eq. (11.4-2)*
@ 5% Damped Design S_{DS} =	$2/3(S_{MS})$	**= 1.015g**	*Eq. (11.4-3)*
S_{D1} =	$2/3(S_{M1})$	**= 0.543g**	*Eq. (11.4-4)*
Risk Category =	II, Standard		*Tab. 1.5-1*
	Flexible Diaphragm		*§12.3.1*
Seismic Design Category for 0.1 sec	D		*Tab. 11.6-1*
Seismic Design Category for 1.0 sec	D		*Tab. 11.6-2*
S1 < 0.75g	N/A		*§11.6*
Since Ta < .8Ts (see below), SDC =	**D**		Exception of §11.6 does not apply

§12.8 Equivalent Lateral Force Procedure

	A. BEARING WALL SYSTEMS			*Tab. 12.2-1*
Seismic Force Resisting System (E-W)	11. Ordinary plain masonry shear walls			
	A. BEARING WALL SYSTEMS			*Tab. 12.2-1*
Seismic Force Resisting System (N-S)	11. Ordinary plain masonry shear walls			
C_t =	0.02		x = 0.75	*Tab. 12.8-2*
Structural height h_n =	11.0 ft		Structural Height Limit = NP	*Tab. 12.2-1*
C_u =	1.400		for S_{D1} of 0.543g	*Tab. 12.8-1*
Approx Fundamental period, T_a =	$C_t(h_n)^x$	= 0.121		*Eq. (12.8-7)*
T_L =	6 sec			*Figs. 22-14 through 22-17*
Calculated T shall not exceed ≤	C_uT_a	= 0.169		*§12.8.2*
Use T =	**0.12 sec**			
T_S =	(S_{D1}/S_{DS})	= 0.535		Exception of §11.6 does not apply
Is structure Regular & ≤ 5 stories?	No			*§12.8.1.3*
Is structure light-framed 1-2 family dwelling?	No			

	E-W		**N-S**			
Response Modification Coefficient R =	1.5		1.5			*Tab. 12.2-1*
Over Strength Factor Ω_o =	2		2			*(foot note g)*
Deflection Amplification Factor C_d =	1.25		1.25			
Importance Factor I_e =	1.00		1.00			*Tab. 11.5-2*
Seismic Base Shear V =	C_sW		C_sW			
C_s =	$\frac{S_{DS}}{R/I_e}$	= 0.677	$\frac{S_{DS}}{R/I_e}$	= 0.677	For T ≤ 1.5Ts	*Eq. (12.8-2) & §11.4.8*
Max C_s =	$\frac{S_{D1}}{(R/I_e)T}$	N/A	$\frac{S_{D1}}{(R/I_e)T}$	N/A	For 1.5Ts < T ≤ TL	*Eq. (12.8-3) & §11.4.8*
or C_s =	$\frac{S_{D1}T_L}{T^2(R/I_e)}$	N/A	$\frac{S_{D1}T_L}{T^2(R/I_e)}$	N/A	For T > T_L	*Eq. (12.8-4) & §11.4.8*
Min C_s =	$0.5S_1I_e/R$	N/A	$0.5S_1I_e/R$	N/A	For S_1 ≥ 0.6g	*Eq. (12.8-6)*
or C_s =	$0.044S_{DS}I_e$	= 0.045	$0.044S_{DS}I_e$	= 0.045	For S_1 < 0.6g	*Eq. (12.8-5)*
Use C_s =	0.677		0.677			
Design base shear V =	**0.677 W**		**0.677 W**			

Seismic Weight Calculations

Job Number: 24-007

Roof Diaphragm:

Flat Roof Snow Load (P_f):	**25** psf		
Roof Dead Load:	**15** psf	(28.2 psf average)	
Roof Deck Dead Load:	**0** psf	(13.2 psf average)	
Roof Diaphragm Area:	**1586.0611** sq. ft.		
Roof Deck Diaphragm Area:	**0** sq. ft.	⟶	**23.8** kips
Average Height of Roof Diaphragm:	**11** ft		
Depth of Roof over Wall at Perimeter:	**1** ft	(Story Height = 11.0')	

<u>Wall Weights Below:</u>

Wall Height:	**7** ft		
Parapet Height:	**2** ft		
Parapet Wall Lengths:	**166** lf		
Exterior Storefront Wall Lengths:	**53.25** lf		
Exist Exterior Wall Lengths:	**50.49** lf		
New CMU Wall Lengths:	**23.75** lf		
Parapet Wall Weight:	**10** psf		
Exterior Storefront Wall Weight:	**10** psf		
Exist Exterior Wall Weight:	**51** psf		
New CMU Wall Weight:	**82** psf		
		⟶	**21.0** kips

w_R = **44.8** kips   Total Area = 1586 sq. ft.   **(28.2 psf average)**

1st Floor:

Base Elevation of Structure **0** ft w_T = **62.5** kips

Wind Design

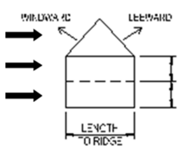
Job Number: 24-007

MWFRS Directional Procedure Part 1 - ASCE 7-16 §27.3.1

Length ⊥ to Main Ridge =	34 ft		Velocity Pressure q_z =	20.44 psf	*x k_z § 26.10.2*
Length Parallel to Main Ridge =	20 ft		Velocity Pressure q_h =	11.75 psf	*§ 26.10.2*
Main Ridge runs	(E-W)		Enclosure Classification =	Enclosed	*§ 26.12*
Wind Speed =	97 mph	*Fig. 26.5*	GC_{pi} =	±0.18	*Tab. 26.13-1*
Exposure Category =	B	*§ 26.7*	q_hGC_{pi} =	±2.11 psf	
Roof Pitch =	0:12	(Gable/Hip)			
Elevation =	44 ft above sea level		Number of Stories =	1	
Θ =	0.0 °		Height of 1st Story =	0.0 ft	
K_{zt} =	1.00	*§ 26.8*			
K_d =	0.85	*Tab. 26.6-1*			
K_h =	0.57	*Tab. 26.10-1*			
K_e =	1.00	*Tab. 26.9-1*			
Building Structure is	Rigid	*Assumed (§26.11)*	Height to Top Plate =	7.0 ft	
Approx. Natural Freq n_a =	is > 1	*Assumed (§26.11.1)*	Mean Roof Height =	11.0 ft	
Gust Factor G =	0.85	*Assumed (§26.1.1)*	Top of roof height =	7.0 ft	

Sum of Wall Pressures - Directional Procedure Part 1 - ASCE 7-16 §27.3.1

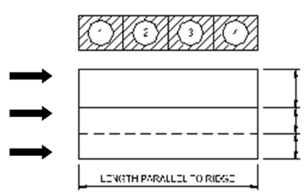
Wind Direction ⊥ to Ridge (N-S)



3.5' → 11.58 psf
3.5' → 11.58 psf

34 ft
((E-W) Elevation)

Wind Direction Parallel to Ridge (E-W)



0' → 12.98 psf
3.5' → 12.98 psf
3.5' → 12.98 psf

20 ft
((N-S) Elevation)

Sum of Roof Pressures - Directional Procedure Part 1 - ASCE 7-16 §27.3.1

Wind Direction ⊥ to Ridge (N-S)

Zone 1, from 0 ft to 5.5 ft = **-11.10 psf**
Zone 2, from 5.5 ft to 11 ft = **-11.10 psf**
Zone 3, from 11 ft to 22 ft = **-7.11 psf**
Zone 4, from 22 ft to 34 ft = **-5.11 psf**

Wind Direction Parallel to Ridge (E-W)

Zone 1, from 0 ft to 5.5 ft = **-11.50 psf**
Zone 2, from 5.5 ft to 11 ft = **-10.90 psf**
Zone 3, from 11 ft to 20 ft = **-7.31 psf**

Additonal Overhang Pressure - Directional Procedure Part 1 §27.3.3

Overhang Zone = **-7.99 psf**

-Plus and minus signs signify pressures acting toward and away from surfaces, respectively

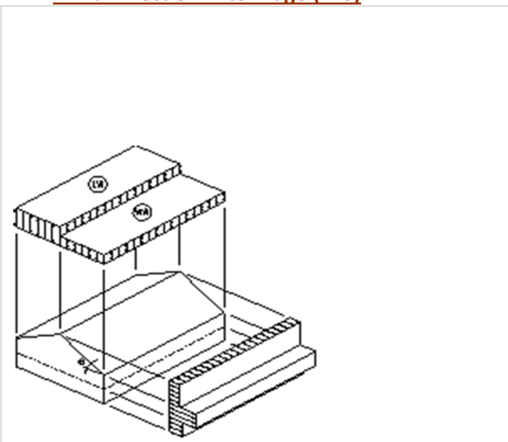
-MWFRS pressures for enclosed or partially enclosed buildings shall not be less than 16 psf (walls) and 8 psf (roof) §27.1.5

Wind Design

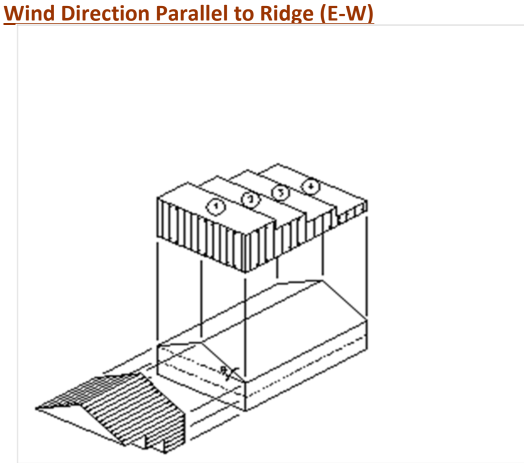
Job Number: 24-007

ASCE 7-16 Directional Procedure Loading - Case 1

Wind Direction ⊥ to Ridge (N-S)



Wind Direction Parallel to Ridge (E-W)



Wind Direction ⊥ to Ridge (N-S)

Vertical Wall Projected Area at Roof:	307 sq ft x 16.00 psf	= 4.92 kips	Roof Shear:	**4.92 kips**
Vertical Wall Projected Area at 1st Floor:	201 sq ft x 16.00 psf	= 3.22 kips	1st Floor Shear:	**3.22 kips**
Total Surface Area:	**508 sq ft**		Total Base Shear:	**8.13 kips**

Wind Direction Parallel to Ridge (E-W)

Vertical Wall Projected Area at Roof:	150 sq ft x 16.00 psf	= 2.39 kips	Roof Shear:	**2.39 kips**
Vertical Wall Projected Area at 1st Floor:	90 sq ft x 16.00 psf	= 1.44 kips	1st Floor Shear:	**1.44 kips**
Total Surface Area:	**240 sq ft**		Total Base Shear:	**3.83 kips**

Rho Factor Calculations

ASCE 7-16 Section 12.3.4

Job Number: **24-007**

Building Regular in plan? No
Total Seismic Base shear V = 21.2 k
Seismic Design Category: D

EAST-WEST DIRECTION

Story 1 h_{story} = 11.0'
V_{story} = 21.2 k (100% of base shear)
Story Strength = 23750.0 k Max Wall Strength = 0.0 k
Reduction in strength from removal of strongest wall with h/l>1:
1-(23750.0-0.0)/23750.0 = 0% < 33%, Condition (a) OK

Shear Line	l_{wall}	# bays	Perimeter?
A	23.8'	4.3	Yes

Condition (b) - Perimeter Bays OK

Rho = 1.0 (Enter below)
EAST-WEST DIRECTION: ρ = **1.0**

NORTH-SOUTH DIRECTION

Story 1 h_{story} = 11.0'
V_{story} = 21.2 k (100% of base shear)
Story Strength = 23.0 k Max Wall Strength = 0.0 k
Reduction in strength from removal of strongest wall with h/l>1:
1-(23.0-0.0)/23.0 = 0% < 33%, Condition (a) OK

Shear Line	l_{wall}	# bays	Perimeter?
2	36.0'	6.5	No

Condition (b) - Perimeter Bays OK

Rho = 1.0 (Enter below)
NORTH-SOUTH DIRECTION: ρ = **1.0**

Lateral Load Calculations

Job Number: 24-007

ASCE 7-16 §12.8 Equivalent Lateral Force Procedure

				E-W	N-S
			C_s =	*0.677*	*0.677*
Total Diaphragm Shear Force:			$V = C_s \times W$ =	30.32 kips	30.32 kips
Vertical Distribution:	C_{vx}			F_x	F_x
Roof Diaphragm Shear Force:	1.000	x 30.32 kips	=	**30.32** kips	**30.32** kips
Total Diaphragm Shear Force:	1.000			**30.32** kips	**30.32** kips

Diaphragm Design Forces:
ASCE 7-16 §12.10.1.1

Coef. Limits
$< 0.4_{SDS}I_e =$ **0.406**
$> 0.2_{SDS}I_e =$ **0.203**

	w_i, kips	Σw_i, kips	F_i, kips	ΣF_i, kips	F_{px} Coef.		Use F_{px}	F_{px}
Roof E-W	44.80	44.80	30.32	30.32	0.677	→	**0.406**	**18.2** kips
Roof N-S			30.32	30.32	0.677	→	**0.406**	**18.2** kips

ASCE 7-16 ASD Load Combinations: Basic - §2.4.1 & §2.4.5

Lateral Comparison (East-West): (Rho = 1.0)

Roof Design Shear **21.23 kips** > **1.44 kips**
EQ Controls for East-West Direction

Lateral Comparison (North-South): (Rho = 1.0)

Roof Design Shear **21.23 kips** > **2.95 kips**
EQ Controls for North-South Direction

Lateral Load Calculations

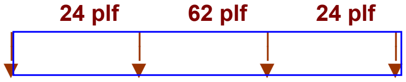
ASCE 7-16 §12.3.1.1

Job Number: 24-007
Roof Diaphragm E-W

Flexible Diaphragm Analysis:

V_e	=	21226 **lbs**	(Seismic Shear)	V_w	=	1437 **lbs**	(Wind Shear)
A_e	=	1586 **sq ft**	(Total Diaphragm Area)	A_w	=	150 **sq ft**	(Total Wind Pressure Area)
e	**=**	13.38 **psf**	(Seismic Area Load)	**p**	=	9.60 **psf**	(Wind Pressure)

EQ:	**d**	=	**53.0 '**	**53.0 '**	**53.0 '**
	ω_e	=	709 plf	709 plf	709 plf
Wind:	**h**	**=**	**2.5 '**	**6.5 '**	**2.5 '**
	ω_w	=	24 plf	62 plf	24 plf



	w	**=**	**4.8 '**	**20.0 '**	**4.8 '**

Resisting Lines:			
Gridlines:		A	B
	A_E =	786 sq ft	786 sq ft
	A_W =	77 sq ft	77 sq ft
Total SW Length:		23.8 '	
EQ:		10519 lbs	10519 lbs
Wind:		740 lbs	740 lbs
Total EQ Rxn:		**10519 lbs**	**10519 lbs**
Total Wind Rxn:		**740 lbs**	**740 lbs**

Check sum of forces: EQ: 21038 lbs **OK** Wind: 1480 lbs **OK**

Drag/Collector Load Calculations

ASCE 7-16 §12.10.2

Job Number: **24-007**

Roof Diaphragm E-W

EQ Controls - Design for Overstrength per 12.10.2.1

Line A

V_{diaph} = **21037.6 #**

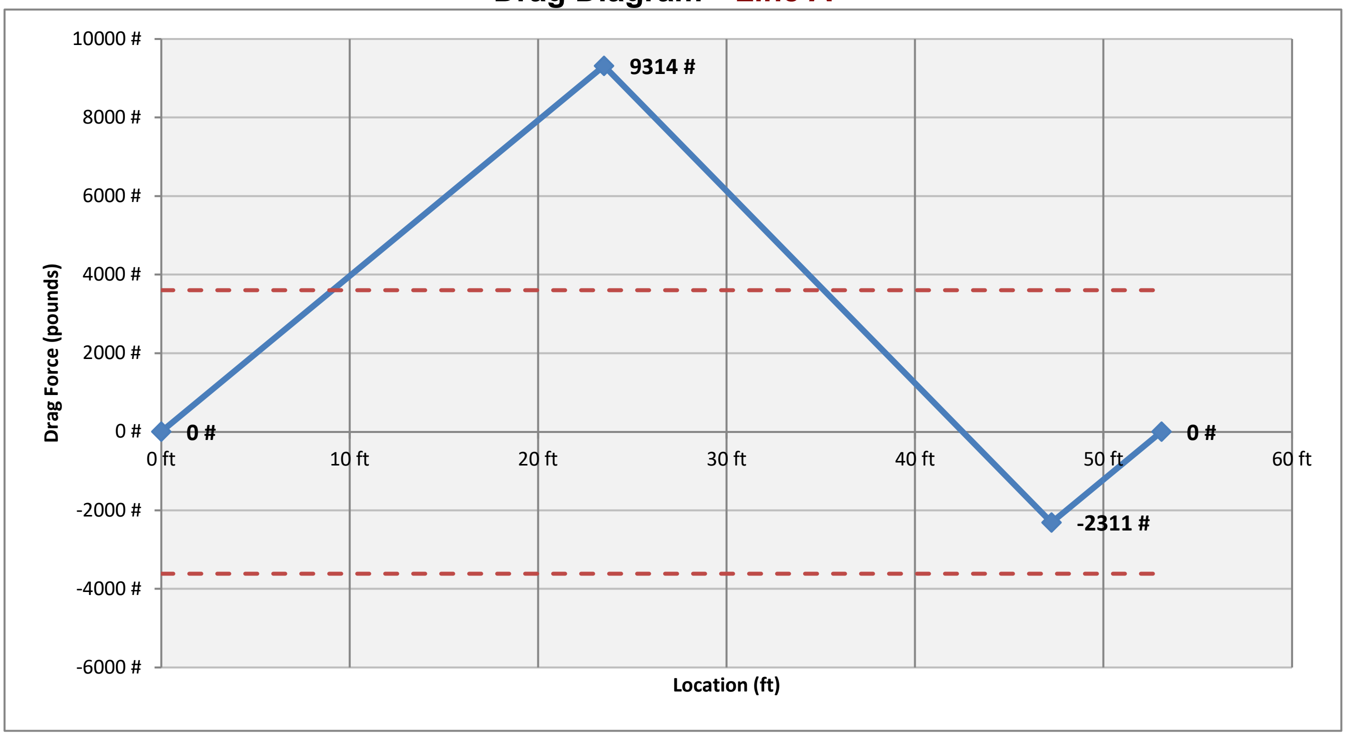
V_{total} = **21037.6 #**

e = 26.87 psf

53.1 ft	Total Diaphragm length
783 ft^2	Total Diaphragm area
23.8 ft	Total Wall Below length

Location		x	SW loc	w_{diaph}	Frmg dir.	Drag load	v_{diaph}	$v_{wall\ below}$	$v_{wall\ above}$	v_{total}	A_{diaph}	Diaph Callout
x_0	0.0'	0.0'										
x_1	23.5'	23.5'	none	14.8'	perp.	**9314 #**	396.3 plf	0.0 plf	0.0 plf	396.3 plf	347 ft^2	**F3**
x_2	23.8'	47.3'	below	14.8'	perp.	**-2311 #**	396.3 plf	-885.8 plf	0.0 plf	-489.5 plf	350 ft^2	**F3**
x_3	5.8'	53.1'	none	14.8'	perp.	**0 #**	396.3 plf	0.0 plf	0.0 plf	396.3 plf	86 ft^2	**F3**

Drag Diagram - Line A



Wood Column

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: Rim Drag Beam

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		
End Fixities	Top & Bottom Pinned		
Overall Column Height		16.75 ft	
(Used for non-slender calculations)			
Wood Species	Douglas Fir-Larch		
Wood Grade	No.1		
Fb +	1,000.0 psi	Fv	180.0 psi
Fb -	1,000.0 psi	Ft	675.0 psi
Fc - Prll	1,500.0 psi	Density	31.210 pcf
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
Minimum	620.0	620.0	

Wood Section Name	**2-2x10**
Wood Grading/Manuf.	Graded Lumber
Wood Member Type	Sawn
Exact Width	**3.0** in
Exact Depth	**9.250** in
Area	27.750 in^2
Ix	197.863 in^4
Iy	20.813 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.10
Cf or Cv for Compressio	1.0
Cf or Cv for Tension	1.10
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

Column Buckling Condition:
ABOUT X-X Axis: Lux = 16.75 ft, Kx = 1.0
ABOUT Y-Y Axis: Luy = 2.0 ft, Ky = 1.0

Applied Loads

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

Column self weight included : 100.742 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 16.750 ft, Xecc = 0.250 in, Yecc = 0.50 in, E = 13.306 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.3558 : 1**
Load Combination	+D+0.70E
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are .	
Applied Axial	9.415 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	953.59 psi
PASS Maximum Shear Stress Ratio =	**0.004349 : 1**
Load Combination	+D+0.70E
Location of max.above base	16.750 ft
Applied Design Shear	1.879 psi
Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.03310 k	Bottom along Y-Y	0.03310 k
Top along X-X	0.01655 k	Bottom along X-X	0.01655 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y -0.05171 in at 9.780 ft above base
for load combination : E Only
Along X-X -0.2458 in at 9.780 ft above base
for load combination : E Only

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.609	0.004413	PASS	0.0 ft	0.0	PASS	16.750 ft
+0.60D	1.600	0.397	0.002284	PASS	0.0 ft	0.0	PASS	16.750 ft
+D+0.70E	1.600	0.397	0.3558	PASS	0.0 ft	0.004349	PASS	16.750 ft
+D+0.5250E	1.600	0.397	0.2678	PASS	0.0 ft	0.003261	PASS	16.750 ft
+0.60D+0.70E	1.600	0.397	0.3543	PASS	0.0 ft	0.004349	PASS	16.750 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	My - End Moments (k-ft) @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only					0.101				
+0.60D					0.060				

Wood Column

Project File: Dons Drivein calcs.ec6

LIC# : KW-06015057, Build:20.23.08.30 SFA ENGINEERING LLC

DESCRIPTION: Rim Drag Beam

Maximum Reactions

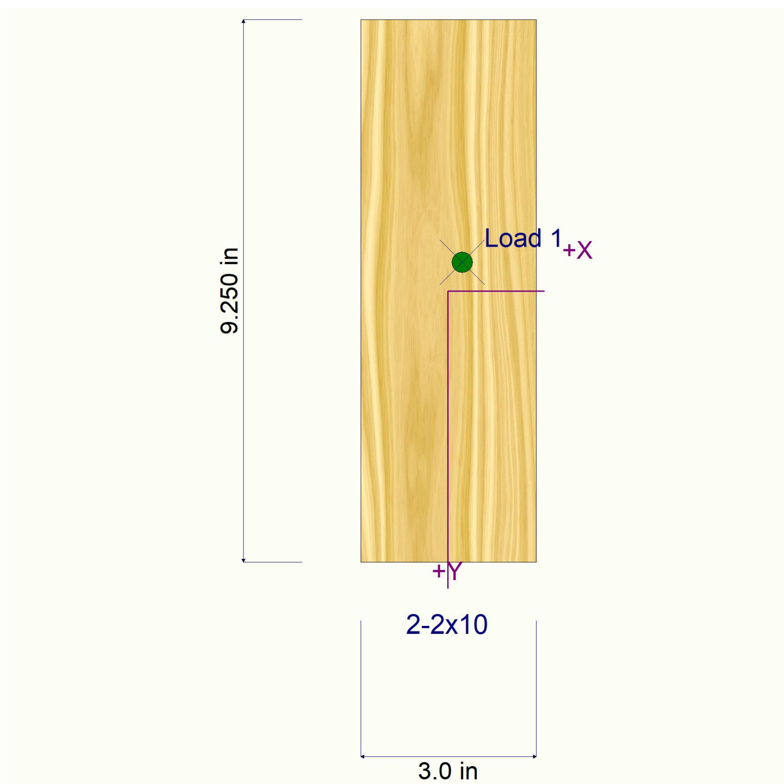
Note: Only non-zero reactions are listed.

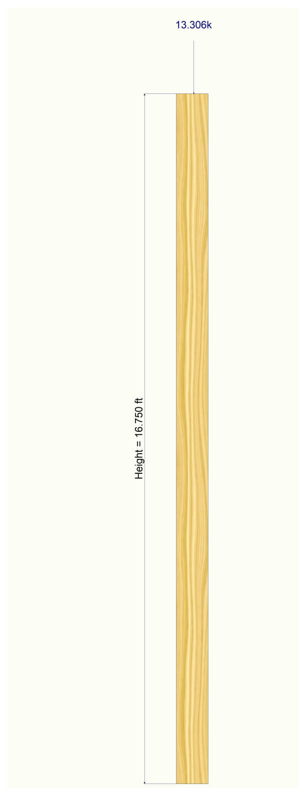
Load Combination	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	My - End Moments (k-ft) @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.70E	-0.012	0.012	-0.023	0.023	9.415				
+D+0.5250E	-0.009	0.009	-0.017	0.017	7.086				
+0.60D+0.70E	-0.012	0.012	-0.023	0.023	9.375				
E Only	-0.017	0.017	-0.033	0.033	13.306				

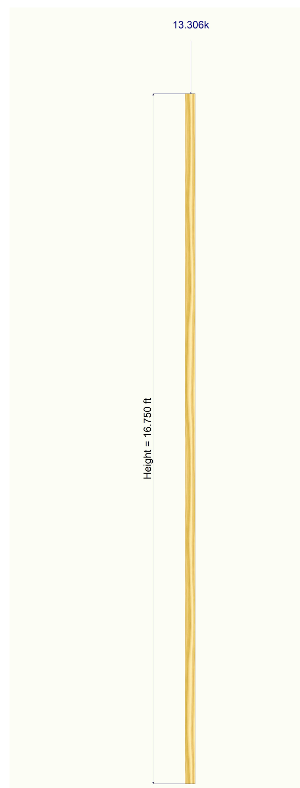
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	-0.1720 in	9.780 ft	-0.036 in	9.780 ft
+D+0.5250E	-0.1290 in	9.780 ft	-0.027 in	9.780 ft
+0.60D+0.70E	-0.1720 in	9.780 ft	-0.036 in	9.780 ft
E Only	-0.2458 in	9.780 ft	-0.052 in	9.780 ft

Sketches







SFA Design Group, LLC
STRUCTURAL | CIVIL | LAND USE PLANNING

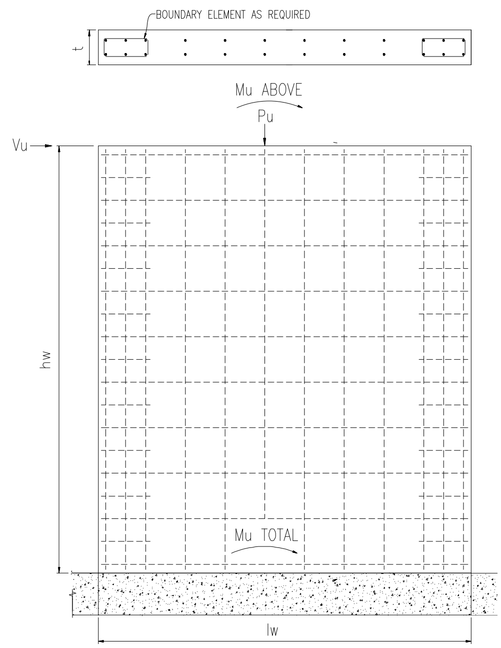
Project No.	Sheet No.
24-007	

Project	Date
Don's Drive-In	3/2024
Subject	By
Special Reinforce Masonry Shearwall Design	CVG

Stevens Court TI

Special Reinforced Masonry Shearwall Design - Strength Design (TMS402-16)

Wall Length (lw)	23.75	feet		R =	5	>= 1.5
Wall Thickness (t)	7.625	inches		rebar Fy =	60	ksi
Wall Story Height (hw)	9	feet		Clay/CMU?	CMU	
Wall Total Height (hw)	9	feet		t_{wall} equiv =	7.625	inches
CMU f'm	1500	psi		Em =	1350000	psi
Applied Axial (Pu)	3.3	kips		Running Bond?	Y	
Applied Shear (Vu)	11	kips		Fu =	90 ksi	
Applied Moment (Mu Above)	0	kip-ft (Upper story)				
Total Moment (Mu Total)	95	kip-ft				
As Vert Reinforcing	0.31 in^2	(0.13 in^2 min)	[7.3.2.6c]			
Spacing	24 in	(36" max)	[7.3.2.6a]			
As Horiz Reinforcing	0.31 in^2	(0.07 in^2 min)	[7.3.2.6c]			
Spacing	32 in	(36" max)	[7.3.2.6b]			



Mu/(Vu*dv) = 0.6416 < 1.0 Shear Dominated

Shear *TMS 402 9.3.4.1.2*

Φ =	0.8		[9.1.4.5]
γ =	0.75		[9.3.4.1.2]

Vm = [4 - 1.75 (Mu / Vu * dv)] x An x f'm^0.5 + 0.25Pu

Vm =	281	kips	[Eq 9-20]

Vs = 0.5 x (Av / s) fy x dv

Vs =	83	kips	[Eq 9-21]

Vn = n(Anv x sqrt(f'm)) [n=6 when Mu/(Vu*dv) <= 0.25 and 4 when Mu/(Vu*dv) >=1.0]

Vn =	417	kips	
$\Phi\gamma$Vn =	218	kips	**OK**

Flexure & Axial Loads *TMS 402 9.3.6.3, 9.3.4.1.1*

Φ =	0.6		[9.1.4.4]				
N.A.Depth c =	24.6	inches					
a = c β_1 =	19.66	inches					
Req. Axial Capacity Pn=	5.6	kips		h =	108	inches	
Req. Moment Capacity Mn=	158	kip-ft		r =	2.201148	inches	
Cc (Concrete Stress Block)	191	kips		h/r =	49	< 99	
Total Compressive Force (C) =	214.1	kips		ΦPn =	1106	kips	**OK**
Total Tensile Force (T) =	208.5	kips					
Axial Capacity Pn (C-T) =	5.6	kips	**OK**			α =	1.5
Moment Capacity ΦMn =	1696	kip-ft	**OK**			max tensile strain ($\alpha\varepsilon_y$) =	0.0031
						ε_{mu} =	0.0025

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Project No.	Sheet No.
24-007	

Project	Date
Don's Drive-In	3/2024
Subject	**By**
Special Reinforce Masonry Shearwall Design	CVG

Special Reinforced Masonry Shearwall Design - Strength Design (TMS402-16) - Continued

Boundary Zone Requirements	*TMS 402 9.3.6.6.1*		
Pu/(Agf'm) =	*0.001*		
Mu/(Vud) =	*0.64*		
Vu/(Anv x sqrt(f'm)) =	*0.00*	**Special boundary elements not required**	
Shear Friction Requirements	*TMS 402 9.3.6.5*		
ϕ =	0.8		
μ =	1		
Asp =	0.31 in^2		
Spacing =	24 in		
Shear Friction Cap ΦVnf =	146	kips	**OK**
Wall Deflection	*TMS 402 9.3.5.4*		
Modulus of rupture fr =	100	psi (per Table 9.1.9.2)	
n =	0.0215		
Icr =	34530839.82	inches	
Mcr =	5188479.66	lb-in	
Wall defl δu =	0.00519	inches	
Cd =	4		
Ie =	1.0		
Cdδu/Ie =	0.02	inches	
Δa =	1.08	inches	**OK**

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PROJECT NO.	SHEET NO.
24-001	

PROJECT	DATE
DON'S DRIVE-IN	3/2024

SUBJECT	BY
ROOF TO CMU SILL CONNECTIONS	CMG

DRAG TO CMU WALL

$P_{DRAG,\,ASD} = 9314\# < 6(1050)(1.6) = 10080\#$ (CD) ∴ $\boxed{(6)\ 7/8"\phi\ TB}$

ROOF DECKING TO BLOCKING

$V = \dfrac{9314\#}{23.75} = 392\,(1.5'\,oc) = 588 < 650$ ok ✓ ∴ $\boxed{\text{A35 AT 18" oc TO ROOF DECKING}}$

BLOCKING TO SILL PL

⊛ SAME AS ABOVE

SILL PL ANCHOR BOLTS

$V_{AB} = \dfrac{9314}{23.75}\left(\dfrac{32"\,oc}{12}\right) = 1046\#$

PER TMS 402, SECTION 8.1.3.3.2:

$B_{vb} = 1.25 A_v \sqrt{f'_m} = 1.25(44.1\ in^2)\sqrt{1500} = 2135\#$

$B_{vc} = 580 \sqrt[4]{f'_m A_b} = 580 \sqrt[4]{1500(.31)} = 2693\#$

$B_{vpry} = 2.5 A_{pt} \sqrt{f'_m} = 2.5(117.2\ in^2)\sqrt{1500} = 11348\#$

$B_{vs} = 0.36 A_b f_y = 0.36(.31)(36\ ksi) = 4018\#$

∴ $V_{AB} < B_{vb}$ ok ✓ ∴ $\boxed{5/8"\phi \times 8"\ AB\ AT\ 32"\ oc}$

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PROJECT NO. 24-007	SHEET NO.
PROJECT: DON'S DRIVE - 1H	DATE: 3/2024
SUBJECT: WALL ANCHORAGES	BY: CVG

$F_p = 0.4 S_{DS} k_a I_e W_p$ [ASCE 7-16 EQ 12.11-1]

$= 0.4(1.045)(1.45)(1.0)\,W_p$

$= 0.59\,W_p$ ← GOVERNS

$F_{p,MIN} = 0.2 k_a I_e W_p$

$= 0.2(1.45)(1.0)\,W_p$

$= 0.29\,W_p$

TRY ANCHORS AT 4' o.c.:

$F_p = 0.59\left[(82\text{ PSF})(7'/2)(4'\text{ TRIB}) + 15(4'\text{ TRIB})(4')\right]$

$= 819^{\#} \times 0.7$

$= 573^{\#} < 1845^{\#}$

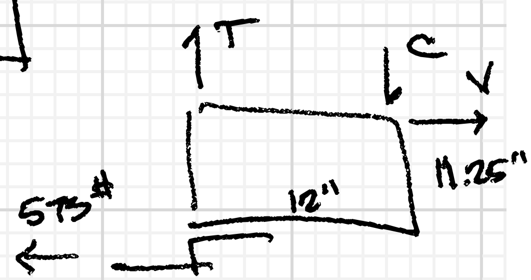
$T_{BOLT} = 573^{\#} < 1825(.83) = 1515^{\#}$ ok ✓

∴ $\boxed{\text{LTTP2 w/ } 1/2"\phi \times 4\tfrac{1}{2}" \text{ EMB THRU ROD w/ SIMP SET-XP}}$

BLOCKING:

$T = C = \dfrac{573(11.25)}{12"\text{ MIN}} = 537^{\#}.$

∴ $\boxed{(2)\ 1/4 \times 3"\text{ SDS}}$



$V = 573^{\#} < 280(1.6) = 896^{\#}$

∴ $\boxed{(2)\ 1/4"\phi \times 3"\text{ SDS}}$

CONN TO DECK: $F_p = 573 < 650^{\#}$ ∴ $\boxed{\text{A35 EA BAY}}$

STRAP: $T = 573$ ∴ $\boxed{\text{CS16}}$

Shearwall Footing Calculations

Overall Max Brg Pressure (Int): 0 psf
Overall Max Brg Pressure (Ext): 1121 psf
Overall Max Avg ω (Int): 0 plf
Overall Max Avg ω (Ext): 1054 plf

SW Footing Description										Additional Dead Loads		SW Footing Calculations								
Shear Wall		Ftg Type	Ftg extention	w_{ftg}	h_{ftg}	w_{stem}	h_{stem}	Ftg bury	$L_{SW\ ftg}$	Pt Ld	Unfrm Ld	M_{OT}	P_{DL}	Avg ω	e	kern	Min Brg Pressure	Max Brg Pressure	$FOS_{Sliding}$	FOS_{OT}
(1F)	A - 1	Exterior	*0.0 ft*	*1.33 ft*	10.0 in	*8.0 in*	14.0 in	18.0 in	23.75 ft			41023 ft-lbs	25040 lbs	1054 plf	2.34 ft	3.96 ft	465 psf	1121 psf	2.38	7.25

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		PROJECT NO. 24-007	SHEET NO.
PROJECT Don's Drive In Range Hood Support			DATE 4/17/2024
SUBJECT Hood Design Criteria			BY CVG

Seismic Loads

Project Site (Latitude,Longitude)	47.1833023, -122.2932043	
Importance Factor (I_P)	1.0	[ASCE 7-16 Section 13.1.3]
Spectral Acceleration (Short Period) (S_{DS})	101.5%	

Hood Information

Range Hood Weight (W_P)	506 lbs
Range Hood Width	30.0 in
Range Hood Length	12.0 ft
Range Hood Height	2.0 ft
Number of Connections (n)	8
Secondary Hood Weight (W_P)	88 lbs
Secondary Hood Width	20.0 in
Secondary Hood Length	12.0 ft
Secondary Hood Height	1.0 ft
Number of Connections (n_2)	8

Existing Building Information

Type of Wall Framing	Wood Studs
Type of Roof Framing	Solid-Sawn Lumber Joists
Spacing Between Roof Framing	1.3 ft
Roof Framing Direction Relative to Wall	Perpendicular

Variables

Amplification Factor (a_P) (Mechanical Componets constructed of sheet metal framing) =	2.5	[ASCE 7-16 Table 13.6-1]
Response Modifcation Factor (R_P) (Mechanical Componets constructed of sheet metal framing) =	6	[ASCE 7-16 Table 13.6-1]
Z/h (Conservative) =	1	[ASCE 7-16 Eq. (13.3-1)]
Braced Angle (θ) =	45 Degrees	
Redundancy Factor (ρ) =	1	[ASCE 7-16 13.3.1.1]
Steel Tensile Yielding ASD Factor of Safety ($\Omega_{t(y)}$) =	1.67	[AISC 15 Eq. (D2-1)]
Steel Tensile Rupture ASD Factor of Safety ($\Omega_{t(r)}$) =	2	[AISC 15 Eq. (D2-2)]
A36 Threaded Rod Yield Stress (F_y) =	36 ksi	[AISC 15 Table 2-6]
A36 Threaded Rod Tensile Strength (F_u) =	58 ksi	[AISC 15 Table 2-6]
Shear Lag Factor (U) =	1	[AISC 15 Table D3.1]
Overstrength Factor (Ω) =	2.5	[ASCE 7-16 Table 13.6-1]

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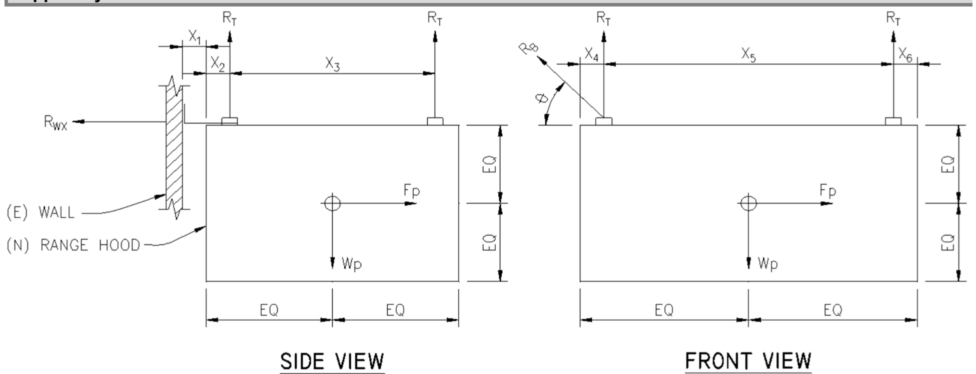
PROJECT NO. 24-007	SHEET NO.
PROJECT Don's Drive In Range Hood Support	DATE 4/17/2024
SUBJECT Range Hood Roof Lateral Support	BY CVG

Support System



X_1 = 3.0 in X_3 = 26.0 in X_5 = 140.0 in

X_2 = 2.0 in X_4 = 2.0 in X_6 = 2.0 in

Seismic Design

$F_p = \dfrac{0.4*a_P*S_{DS}*W_P*(1+2*Z/N)}{\dfrac{R_P}{I_P}}$ = 256.8 lbs [ASCE 7-16 Eq (13.3-1)]

Max F_p = $1.6S_{DS}I_pW_p$ = 821.7 lbs [ASCE 7-16 Eq (13.3-2)]

Min F_p = $0.3S_{DS}I_PW_P$ = 154.1 lbs [ASCE 7-16 Eq (13.3-3)]

Use F_p **256.8 lbs**

Cable Bracing Design

Diameter =	1/8 in	Prestretched Aircraft cable diameter
T =	254 Lbs $_{(ASD)}$	Tension per cable = $(0.7F_{ph})\cdot 2^{0.5}$
T' =	2000 Lbs $_{(ASD)}$	Per ASHRAE
DCR =	0.13	OK

Cable Connection to (N) Blocking

T= 254 Lbs $_{(ASD)}$

W actual acting on screw = 360 Lbs $_{(ASD)}$ Withdrawal force acting on Screw= (T)x2^0.5

Connection of P1000 Unistruct to (E) Roof Joist

#10x1 1/2" Wood Screws

Withdrawal:

$W' = W(C_DC_MC_tC_{eg}C_{tn})$ [NDS 18 Table 11.3.1]

G (Specific Gravity of D.F.) =	= 0.50	(Douglas Fir #2)
W =	= 135 lbs/in	[NDS 18 Table 12.2B]
C_D =	= 1.6	[NDS 18 Table 2.3.2]
C_t =	= 1.0	[NDS 18 Table 11.3.4]
$C_{eg} = C_{tn} = C_M$ =	= 1.0	[NDS 18 Table 11.3.3, 12.5.4, 12.5.2]
T-E (2/3 Screw Length) =	= 1.00	[NDS 18 Table L3]

W' = = 0.216 kips > 0.0899 kips OK

Use #10x1 1/2" Wood screws for connecting the "L" angle to (E) Roof Joist

Lateral:

$Z' = Z(C_DC_MC_tC_{eg}C_{tn})$ [NDS 18 Table 11.3.1]

G (Specific Gravity of D.F.) =	= 0.50	(Douglas Fir #2)
Z =	= 116 lbs	[NDS 18 Table 12K]
C_D =	= 1.6	[NDS 18 Table 2.3.2]
C_t =	= 1.0	[NDS 18 Table 11.3.4]
$C_{eg} = C_{tn} = C_M$ =	= 1.0	[NDS 18 Table 11.3.3, 12.5.4, 12.5.2]

Z' = = 0.186 kips > 0.0899 kips OK

Use #10x1 1/2" Wood screws for connecting the "L" angle to (E) Roof Joist

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PROJECT NO.	SHEET NO.
24-007	

PROJECT	DATE
Don's Drive In Range Hood Support	4/17/2024
SUBJECT	BY
Range Hood Roof Gravity Support	CVG

Tension in Threaded Rod

$R_T = W_P/n =$ = 63.3 lbs (8) connections

Threaded Rod

d = 0.375 in
Threads per inch, n^b = 16 [AISC 15 Table 7-17]

Gross Area $A_g = \pi(d)^2/4$ = 0.11 in²
Net Area $A_n = 0.7854(d-0.9743/n^b)^2$ = 0.08 in² [AISC 15 Table 7-17]
Effective Area $A_e = A_n*U =$ = 0.08 in² [AISC 15 Eq (D3-1)]

P_n = Lesser of $\frac{F_yA_g}{\Omega_{t(y)}}$ or $\frac{F_uA_e}{\Omega_{t(r)}}$

$\frac{F_yA_g}{\Omega_{t(y)}}$ = 2.38 ksi [AISC 15 (D2-1)]

$\frac{F_uA_e}{\Omega_{t(r)}}$ = 2.25 ksi Use 2.25 kips [AISC 15 (D2-2)]

$P_A = R_T$ = 63.3 lbs < 2247.2 lbs **OK**

Use 0.375" Ø Threaded Rod

Simpson DTT1Z (Simpson C-C-2019 p. 53)

Simpson DTT1Z Capacity = = 840.0 lbs < 63.25 lbs **OK**

P1000 Unistrut Capacity (Unistrut Engineering Catalog No. 17 p. 25)

Length (max) = 24.0 in
Pierced Capacity Factor (Fp) = 0.9 in
Actual Capacity (Ca) = 1130 lbs
Concentrated Load Factor = 0.5 Unistrut Catalog No.17 pg 18
Allowable Capacity = 509 lbs > 63.3 lbs **OK**

Connection of P1000 Unistruct to (E) Wood

#10x1 1/2" Wood Screws

Lateral Shear:

$Z' = Z(C_DC_MC_tC_{eg}C_{tn})$(T-E) [NDS 18 Table 11.3.1]

G (Specific Gravity of D.F.) = = 0.50 (Douglas Fir #2)
Z = = 116 lbs [NDS 18 Table 12L]
C_D = = 1.0 [NDS 18 Table 2.3.2]
C_t = = 1.0 [NDS 18 Table 11.3.4]
$C_{eg} = C_{tn} = C_M$ = = 1.0 [NDS 18 Table 11.3.3, 12.5.4, 12.5.2]

Z' = = 0.116 kips > 0.0316 kips **OK**

Use #10x1 1/2" Wood screws for connecting the "L" angle to (E) Roof Joist

RANGE HOOD CONNECTION TO (E)/(N) CMU WALL:

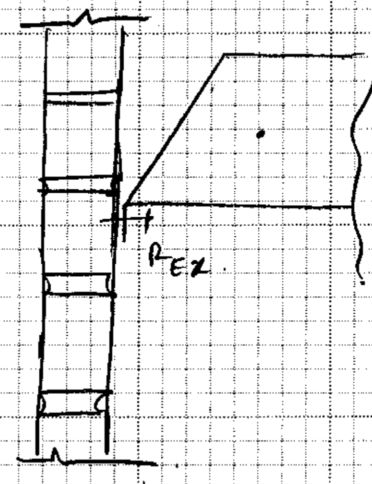
FROM PREVIOUS SHEETS:

$F_p = 257$ lbs.

$R_{Ex} = 0.7 \times 257 = 180$ lbs

LENGTH OF HOOD (L) = 12'

$\frac{R_{Ex}}{L} = \frac{180}{12'} = 15$ plf



∴ PROVIDE SIMPSON 3/8" φ x 2 3/4" EMBED TITEN HD ANCHORS AT 48" oc TO (E) & (N) CMU.

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PROJECT NO.	SHEET NO.
24-007	

PROJECT Don's Drive In Repair MAU	DATE 4/10/2024
SUBJECT Mechanical Unit Design Criteria	BY CG

General

Building Department	City of Puyallup
Building Code Conformance (Meets Or Exceeds Requirements)	2021 IBC

Unit Information

Unit 1 Weight	697 lbs		
Unit 1 Length (x)	160.0 in	Unit 1 CG (x)	35.5 in
Unit 1 Width (y)	28.0 in	Unit 1 CG (y)	22.0 in
Unit 1 Height (z)	30.0 in	Unit 1 CG (z)	30.0 in
Unit 1 Curb Weight	84 lbs		
Unit 1 Curb Length (x)	71.0 in	Unit 1 Curb CG (x)	35.5 in
Unit 1 Curb Width (y)	21.0 in	Unit 1 Curb CG (y)	10.5 in
Unit 1 Curb Height (z)	24.0 in	Unit 1 Curb CG (z)	12.0 in

Existing Building Information

Type of Roof Framing	Solid Sawn Joists
Roof Framing Spacing	24 in

Seismic Loads

Importance Factor (I_p)	1.0	[ASCE 7-10 Section 13.1.3]
Spectral Acceleration (Short Period) (S_{DS})	101.5%	

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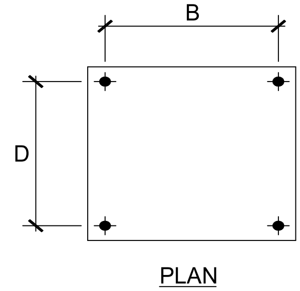
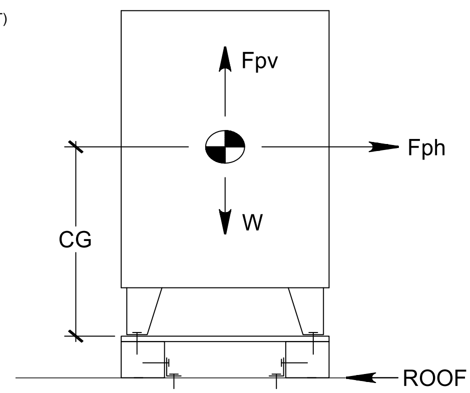
PROJECT NO	SHEET NO.
24-007	

PROJECT	DATE
Don's Drive In Repair MAU	4/10/2024
SUBJECT	BY
Make Up Air Unit Lateral Loads	CG

Seismic Loads (ASCE 7-10, Chapter 13)

a_p =	1.0		ASCE 7 Table 13.5-1 & Table 13.6-1
R_p =	2.5		ASCE 7 Table 13.5-1 & Table 13.6-1
Ω_o =	2.5		ASCE 7 Table 13.5-1 & Table 13.6-1
S_{DS} =	1.015	g	See ATC Hazards sheet
I_p =	1.00		ASCE Table 11.5-1
z/h =	1.00		$F_{ph} = \frac{0.4 S_{DS} a_p}{R_p/I_p}\left(1 + 2\frac{z}{h}\right) W_p$
F_{ph} =	0.49	W_p	
F_{pv} =	0.20	W_p	$F_{pv} = 0.2 \cdot S_{DS} \cdot W_p$
W_p =	**697**	Lbs	Equipment weight
F_{ph} =	340	Lbs (ULT)	$\Omega_o \cdot F_{ph}$ = 849 Lbs (ULT)
F_{pv} =	141	Lbs (ULT)	
$W_{p\ (w/\ curb)}$ =	781	Lbs	Equipment weight (w/ curb)
$F_{ph\ (w/\ curb)}$ =	381	Lbs (ULT)	$\Omega_o \cdot F_{ph}$ = 951 Lbs (ULT)
$F_{pv\ (w/\ curb)}$ =	159	Lbs (ULT)	





Wind Loads (ASCE 7-10, Section 29.5.1)

V =	97	mph	Basic wind velocity
Exp =	B		Exposure category
z =	10	ft	Roof height
L =	160	in	Equipment length
W =	28	in	Equipment width
H =	30	in	Equipment height
A_f =	33	ft^2	Vertical area exposed to wind
A_r =	31	ft^2	Horizontal area exposed to wind
$A_{f\ (w/\ curb)}$ =	45	ft^2	Vertical area exposed to wind
$A_{r\ (w/\ curb)}$ =	31	ft^2	Horizontal area exposed to wind
K_d =	0.85		Table 26.6-1
K_z =	0.57		Table 29.3-1
K_{zt} =	1.00		(Flat)
q_z =	11.8	psf	$= 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2$
G =	0.85		Section 26.9.1, rigid
C_f =	1.35		Figure 29.4-1
F_h =	**450**	Lbs (ULT)	$= q_z \cdot G \cdot C_f \cdot A_f$
F_v =	**366**	Lbs (ULT)	$= q_z \cdot A_r$
$F_{h\ (w/\ curb)}$ =	**610**	Lbs (ULT)	$= q_z \cdot G \cdot C_f \cdot A_f$
$F_{v\ (w/\ curb)}$ =	**366**	Lbs (ULT)	$= q_z \cdot A_r$

Unit Data

e =	1.5	in	Distance from attachments to edge of unit
L' =	157.0	in	Distance between attachments along length
W' =	25.0	in	Distance between attachments along width
H_C =	24.0	in	Height of curb
CG_x =	34.0		Centroid along length (from attachments)
CG_y =	4.5		Centroid along width (from attachments)
CG_z =	30.0	in	Height to centroid (from curb)

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PROJECT NO	SHEET NO.
24-007	

PROJECT Don's Drive In Repair MAU	DATE 4/10/2024
SUBJECT Make Up Air Unit Lateral Loads	BY CG

Connection of Unit to Curb (ASD)

Try:	#8		Metal = 20 ga struct (min)	
n =	4	No. of screws per side (min)		
$P_{x\ seismic}$ =	0	Lbs	Tension per screw (seismic loads)	$= [(0.7F_{ph} \cdot CG_z) - (0.6W_p - 0.7F_{pv}) \cdot CG_x] \div (L' \cdot n)$
$P_{y\ seismic}$ =	62	Lbs	Tension per screw (seismic loads)	$= [(0.7F_{ph} \cdot CG_z) - (0.6W_p - 0.7F_{pv}) \cdot CG_y] \div (W' \cdot n)$
$P_{x\ wind}$ =	11	Lbs	Tension per screw (wind loads)	$= (0.6F_h \cdot H/2 - 0.6W_p \cdot CG_x + 0.6F_v \cdot L'/2) \div (L' \cdot n)$
$P_{y\ wind}$ =	49	Lbs	Tension per screw (wind loads)	$= (0.6F_h \cdot H/2 - 0.6W_p \cdot CG_y + 0.6F_v \cdot W'/2) \div (W' \cdot n)$
V =	17	Lbs	Shear per screw	$= \max[0.6Fh, 0.7F_{ph}] \div (4 \cdot n)$
P' =	72	Lbs	Screw tension capacity per SSMA allowable loads table, p.70	
V' =	164	Lbs	Screw shear capacity per SSMA allowable loads table, p.70	
DCR =	**0.96**	**≤ 1.0 (OK)**		DCR = P/P' + V/V' ≤ 1.0

Connection of Curb to Wood Structure (ASD)

Try:	#8		Metal = 20 ga struct (min)	Wood G = 0.5 (min)
n =	4	No. of screws per side (min)		
L =	1.5	in	Length of threads into wood framing (min)	
C_D =	1.6		(Wind and Seismic)	
W' =	195	Lbs	(NDS Table 11.2A/Screw tension capacity per SSMA allowable loads table, p.70)	
Z' =	142	Lbs	(NDS Table 11K/Screw shear capacity per SSMA allowable loads table, p.70)	
$P_{x\ seismic}$ =	4	Lbs (ASD)	Tension per screw (seismic loads)	$= [(0.7F_{ph} \cdot CG_z) - (0.6W_p - 0.7F_{pv}) \cdot CG_x] \div (L' \cdot n)$
$P_{y\ seismic}$ =	107	Lbs (ASD)	Tension per screw (seismic loads)	$= [(0.7F_{ph} \cdot CG_z) - (0.6W_p - 0.7F_{pv}) \cdot CG_y] \div (W' \cdot n)$
$P_{x\ wind}$ =	18	Lbs	Tension per screw (wind loads)	$= (0.6F_h \cdot H/2 - 0.6W_p \cdot CG_x + 0.6F_v \cdot L'/2) \div (L' \cdot n)$
$P_{y\ wind}$ =	17	Lbs	Tension per screw (wind loads)	$= (0.6F_h \cdot H/2 - 0.6W_p \cdot CG_y + 0.6F_v \cdot W'/2) \div (W' \cdot n)$
$V_{seismic}$ =	17	Lbs (ASD)	$= 0.7F_{ph} / n \cdot 4$	
V_{wind} =	27	Lbs (ASD)	$= 0.7F_h / n \cdot 4$	
$\alpha_{seismic}$ =	57	°	$= \tan^{-1}(P/V)$	
α_{wind} =	33	°	$= \tan^{-1}(P/V)$	
$P_{\alpha\ seismic}$ =	108	Lbs	$= (P^2+V^2)^{1/2}$	
$P_{\alpha\ wind}$ =	32	Lbs	$= (P^2+V^2)^{1/2}$	
$Z'_{\alpha\ seismic}$ =	176	Lbs	$Z'_\alpha = \dfrac{W' \cdot Z'}{W' \cdot \cos^2\alpha + Z' \cdot \sin^2\alpha}$	
$Z'_{\alpha\ wind}$ =	155	Lbs		
DCR =	**0.61**	**≤ 1.0 (OK)**	$= P_\alpha / Z'_\alpha$	

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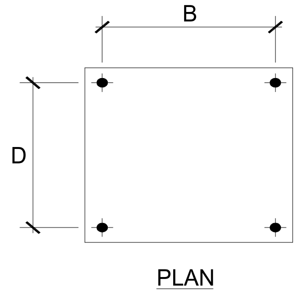
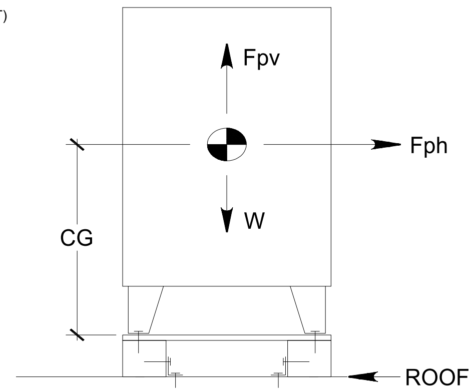
PROJECT NO 24-007		SHEET NO.
PROJECT Don's Drive In Repair RTU		DATE 5/20/2024
SUBJECT RTU Lateral Design		BY CG

Seismic Loads (ASCE 7-10, Chapter 13)

a_p =	2.5		ASCE 7 Table 13.5-1 & Table 13.6-1
R_p =	6.0		ASCE 7 Table 13.5-1 & Table 13.6-1
Ω_o =	2.5		ASCE 7 Table 13.5-1 & Table 13.6-1
S_{DS} =	1.015	g	See ATC Hazards sheet
I_p =	1.00		ASCE Table 11.5-1
z/h =	1.00		$F_{ph} = \frac{0.4 S_{DS} a_p}{R_p/I_p}\left(1+2\frac{z}{h}\right)W_p$
F_{ph} =	0.51	W_p	
F_{pv} =	0.20	W_p	$F_{pv} = 0.2 \cdot S_{DS} \cdot W_p$
W_p =	**580**	Lbs	Equipment weight
F_{ph} =	294	Lbs (ULT)	$\Omega_o \cdot F_{ph}$ = 736 Lbs (ULT)
F_{pv} =	118	Lbs (ULT)	
$W_{p\ (w/\ curb)}$ =	730	Lbs	Equipment weight (w/ curb)
$F_{ph\ (w/\ curb)}$ =	370	Lbs (ULT)	$\Omega_o \cdot F_{ph}$ = 926 Lbs (ULT)
$F_{pv\ (w/\ curb)}$ =	148	Lbs (ULT)	





Wind Loads (ASCE 7-10, Section 29.5.1)

V =	97	mph	Basic wind velocity
Exp =	B		Exposure category
z =	10	ft	Roof height
L =	80	in	Equipment length
W =	47	in	Equipment width
H =	42	in	Equipment height
A_f =	23	ft^2	Vertical area exposed to wind
A_r =	26	ft^2	Horizontal area exposed to wind
$A_{f\ (w/\ curb)}$ =	30	ft^2	Vertical area exposed to wind
$A_{r\ (w/\ curb)}$ =	26	ft^2	Horizontal area exposed to wind
K_d =	0.85		Table 26.6-1
K_z =	0.57		Table 29.3-1
K_{zt} =	1.00		(Flat)
q_z =	11.8	psf	$= 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2$
G =	0.85		Section 26.9.1, rigid
C_f =	1.45		Figure 29.4-1
F_h =	338	Lbs (ULT)	$= q_z \cdot G \cdot C_f \cdot A_f$
F_v =	**307**	Lbs (ULT)	$= q_z \cdot A_r$
$F_{h\ (w/\ curb)}$ =	**435**	Lbs (ULT)	$= q_z \cdot G \cdot C_f \cdot A_f$
$F_{v\ (w/\ curb)}$ =	**307**	Lbs (ULT)	$= q_z \cdot A_r$

Unit Data

e =	1.5	in	Distance from attachments to edge of unit
L' =	77.0	in	Distance between attachments along length
W' =	44.0	in	Distance between attachments along width
H_C =	12.0	in	Height of curb
CG_x =	38.5		Centroid along length (from attachments)
CG_y =	22.0		Centroid along width (from attachments)
CG_z =	21.0	in	Height to centroid (from curb)

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PROJECT	PROJECT NO	SHEET NO.
	24-007	
Don's Drive In Repair RTU	DATE 5/20/2024	
SUBJECT: RTU Lateral Design	BY CG	

Connection of Unit to Curb (ASD)

Try:	#8		Metal = 20 ga struct (min)	
n =	2	No. of screws per side (min)		
$P_{x\ seismic}$ =	0	Lbs	Tension per screw (seismic loads)	$= [(0.7F_{ph}\cdot CG_z) - (0.6W_p - 0.7F_{pv})\cdot CG_x] \div (L'\cdot n)$
$P_{y\ seismic}$ =	0	Lbs	Tension per screw (seismic loads)	$= [(0.7F_{ph}\cdot CG_z) - (0.6W_p - 0.7F_{pv})\cdot CG_y] \div (W'\cdot n)$
$P_{x\ wind}$ =	-13	Lbs	Tension per screw (wind loads)	$= (0.6F_h\cdot H/2 - 0.6W_p\cdot CG_x + 0.6F_v\cdot L'/2) \div (L'\cdot n)$
$P_{y\ wind}$ =	8	Lbs	Tension per screw (wind loads)	$= (0.6F_h\cdot H/2 - 0.6W_p\cdot CG_y + 0.6F_v\cdot W'/2) \div (W'\cdot n)$
V =	26	Lbs	Shear per screw	$= \max[0.6Fh, 0.7F_{ph}] \div (4\cdot n)$
P' =	72	Lbs	Screw tension capacity per SSMA allowable loads table, p.70	
V' =	164	Lbs	Screw shear capacity per SSMA allowable loads table, p.70	
DCR =	**0.26**	**≤ 1.0 (OK)**		$DCR = P/P' + V/V' \leq 1.0$

Connection of Curb to Wood Structure (ASD)

Try:	#8		Metal = 20 ga struct (min)	Wood G = 0.5 (min)
n =	2	No. of screws per side (min)		
L =	1.5	in	Length of threads into wood framing (min)	
C_D =	1.6		(Wind and Seismic)	
W' =	195	Lbs	(NDS Table 11.2A/Screw tension capacity per SSMA allowable loads table, p.70)	
Z' =	142	Lbs	(NDS Table 11K/Screw shear capacity per SSMA allowable loads table, p.70)	
$P_{x\ seismic}$ =	0	Lbs (ASD)	Tension per screw (seismic loads)	$= [(0.7F_{ph}\cdot CG_z) - (0.6W_p - 0.7F_{pv})\cdot CG_x] \div (L'\cdot n)$
$P_{y\ seismic}$ =	28	Lbs (ASD)	Tension per screw (seismic loads)	$= [(0.7F_{ph}\cdot CG_z) - (0.6W_p - 0.7F_{pv})\cdot CG_y] \div (W'\cdot n)$
$P_{x\ wind}$ =	-18	Lbs	Tension per screw (wind loads)	$= (0.6F_h\cdot H/2 - 0.6W_p\cdot CG_x + 0.6F_v\cdot L'/2) \div (L'\cdot n)$
$P_{y\ wind}$ =	10	Lbs	Tension per screw (wind loads)	$= (0.6F_h\cdot H/2 - 0.6W_p\cdot CG_y + 0.6F_v\cdot W'/2) \div (W'\cdot n)$
$V_{seismic}$ =	32	Lbs (ASD)	$= 0.7F_{ph} / n\cdot 4$	
V_{wind} =	38	Lbs (ASD)	$= 0.7F_h / n\cdot 4$	
$\alpha_{seismic}$ =	40	°	$= \tan^{-1}(P/V)$	
α_{wind} =	14	°	$= \tan^{-1}(P/V)$	
$P_{\alpha\ seismic}$ =	43	Lbs	$= (P^2+V^2)^{1/2}$	
$P_{\alpha\ wind}$ =	39	Lbs	$= (P^2+V^2)^{1/2}$	
$Z'_{\alpha\ seismic}$ =	160	Lbs	$Z'_\alpha = \dfrac{W'\cdot Z'}{W'\cdot\cos^2\alpha + Z'\cdot\sin^2\alpha}$	
$Z'_{\alpha\ wind}$ =	145	Lbs		
DCR =	**0.27**	**≤ 1.0 (OK)**	$= P_\alpha/Z'_\alpha$	

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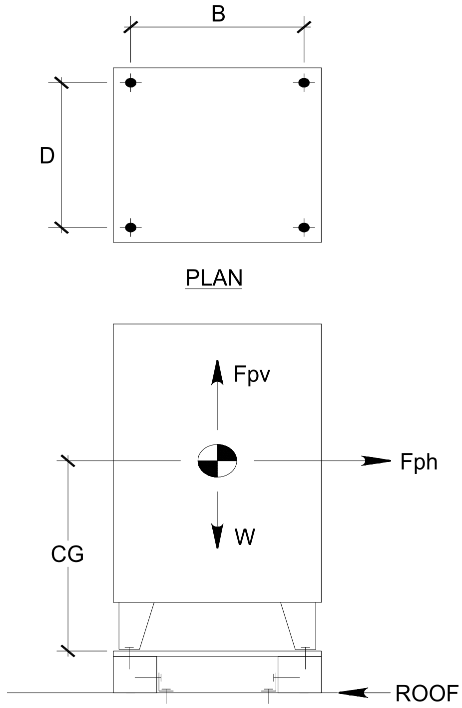
PROJECT NO	SHEET NO.
24-007	

PROJECT Don's Drive In	DATE 5/20/2024
SUBJECT Condenser Unit Lateral Design	BY CG

Seismic Loads (ASCE 7-10, Chapter 13)

a_p =	1.0		ASCE 7 Table 13.5-1 & Table 13.6-1
R_p =	2.5		ASCE 7 Table 13.5-1 & Table 13.6-1
Ω_o =	2.5		ASCE 7 Table 13.5-1 & Table 13.6-1
S_{DS} =	1.015	g	See ATC Hazards sheet
I_p =	1.00		ASCE Table 11.5-1
z/h =	1.00		$F_{ph} = \frac{0.4 S_{DS} a_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) W_p$
F_{ph} =	0.49	W_p	
F_{pv} =	0.20	W_p	$F_{pv} = 0.2 \cdot S_{DS} \cdot W_p$
W_p =	**170**	Lbs	Equipment weight
F_{ph} =	**83**	Lbs (ULT)	$\Omega_o \cdot F_{ph}$ = 207 Lbs (ULT)
F_{pv} =	**35**	Lbs (ULT)	
$W_{p\ (w/\ curb)}$ =	**170**	Lbs	Equipment weight (w/ curb)
$F_{ph\ (w/\ curb)}$ =	**83**	Lbs (ULT)	$\Omega_o \cdot F_{ph}$ = 207 Lbs (ULT)
$F_{pv\ (w/\ curb)}$ =	**35**	Lbs (ULT)	



Wind Loads (ASCE 7-10, Section 29.5.1)

V =	97	mph	Basic wind velocity
Exp =	B		Exposure category
z =	10	ft	Roof height
L =	29	in	Equipment length
W =	25	in	Equipment width
H =	20	in	Equipment height
A_f =	4	ft^2	Vertical area exposed to wind
A_r =	5	ft^2	Horizontal area exposed to wind
$A_{f\ (w/\ curb)}$ =	4	ft^2	Vertical area exposed to wind
$A_{r\ (w/\ curb)}$ =	5	ft^2	Horizontal area exposed to wind
K_d =	0.85		Table 26.6-1
K_z =	0.57		Table 29.3-1
K_{zt} =	1.00		(Flat)
q_z =	11.8	psf	$= 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2$
G =	0.85		Section 26.9.1, rigid
C_f =	1.45		Figure 29.4-1
F_h =	58	Lbs (ULT)	$= q_z \cdot G \cdot C_f \cdot A_f$
F_v =	59	Lbs (ULT)	$= q_z \cdot A_r$
$F_{h\ (w/\ curb)}$ =	58	Lbs (ULT)	$= q_z \cdot G \cdot C_f \cdot A_f$
$F_{v\ (w/\ curb)}$ =	59	Lbs (ULT)	$= q_z \cdot A_r$

Unit Data

e =	1.5	in	Distance from attachments to edge of unit
L' =	26.0	in	Distance between attachments along length
W' =	22.0	in	Distance between attachments along width
H_C =	0.0	in	Height of curb
CG_x =	13.0		Centroid along length (from attachments)
CG_y =	11.0		Centroid along width (from attachments)
CG_z =	10.0	in	Height to centroid (from curb)

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PROJECT		PROJECT NO	SHEET NO.
		24-007	
PROJECT Don's Drive In		DATE 5/20/2024	
SUBJECT Condenser Unit Lateral Design		BY CG	

Connection of Unit to Curb (ASD)

Try:	#8		Metal = 20 ga struct (min)	
n =	2	No. of screws per side (min)		
$P_{x\ seismic}$ =	0	Lbs	Tension per screw (seismic loads)	$= [(0.7F_{ph} \cdot CG_z) - (0.6W_p - 0.7F_{pv}) \cdot CG_x] \div (L' \cdot n)$
$P_{y\ seismic}$ =	0	Lbs	Tension per screw (seismic loads)	$= [(0.7F_{ph} \cdot CG_z) - (0.6W_p - 0.7F_{pv}) \cdot CG_y] \div (W' \cdot n)$
$P_{x\ wind}$ =	-10	Lbs	Tension per screw (wind loads)	$= (0.6F_h \cdot H/2 - 0.6W_p \cdot CG_x + 0.6F_v \cdot L'/2) \div (L' \cdot n)$
$P_{y\ wind}$ =	-9	Lbs	Tension per screw (wind loads)	$= (0.6F_h \cdot H/2 - 0.6W_p \cdot CG_y + 0.6F_v \cdot W'/2) \div (W' \cdot n)$
V =	7	Lbs	Shear per screw	$= \max[0.6Fh, 0.7F_{ph}] \div (4 \cdot n)$
P' =	72	Lbs	Screw tension capacity per SSMA allowable loads table, p.70	
V' =	164	Lbs	Screw shear capacity per SSMA allowable loads table, p.70	
DCR =	**0.04**	**≤ 1.0 (OK)**		DCR = P/P' + V/V' ≤ 1.0

Connection of Curb to Wood Structure (ASD)

Try:	#8		Metal = 20 ga struct (min)	Wood G = 0.5 (min)
n =	2	No. of screws per side (min)		
L =	1.5	in	Length of threads into wood framing (min)	
C_D =	1.6		(Wind and Seismic)	
W' =	195	Lbs	(NDS Table 11.2A/Screw tension capacity per SSMA allowable loads table, p.70)	
Z' =	142	Lbs	(NDS Table 11K/Screw shear capacity per SSMA allowable loads table, p.70)	
$P_{x\ seismic}$ =	0	Lbs (ASD)	Tension per screw (seismic loads)	$= [(0.7F_{ph} \cdot CG_z) - (0.6W_p - 0.7F_{pv}) \cdot CG_x] \div (L' \cdot n)$
$P_{y\ seismic}$ =	0	Lbs (ASD)	Tension per screw (seismic loads)	$= [(0.7F_{ph} \cdot CG_z) - (0.6W_p - 0.7F_{pv}) \cdot CG_y] \div (W' \cdot n)$
$P_{x\ wind}$ =	-10	Lbs	Tension per screw (wind loads)	$= (0.6F_h \cdot H/2 - 0.6W_p \cdot CG_x + 0.6F_v \cdot L'/2) \div (L' \cdot n)$
$P_{y\ wind}$ =	-7	Lbs	Tension per screw (wind loads)	$= (0.6F_h \cdot H/2 - 0.6W_p \cdot CG_y + 0.6F_v \cdot W'/2) \div (W' \cdot n)$
$V_{seismic}$ =	7	Lbs (ASD)	$= 0.7F_{ph} / n \cdot 4$	
V_{wind} =	5	Lbs (ASD)	$= 0.7F_h / n \cdot 4$	
$\alpha_{seismic}$ =	0	°	$= \tan^{-1}(P/V)$	
α_{wind} =	-51	°	$= \tan^{-1}(P/V)$	
$P_{\alpha\ seismic}$ =	7	Lbs	$= (P^2+V^2)^{1/2}$	
$P_{\alpha\ wind}$ =	9	Lbs	$= (P^2+V^2)^{1/2}$	
$Z'_{\alpha\ seismic}$ =	142	Lbs	$Z'_\alpha = \dfrac{W' \cdot Z'}{W' \cdot \cos^2\alpha + Z' \cdot \sin^2\alpha}$	
$Z'_{\alpha\ wind}$ =	170	Lbs		
DCR =	**0.05**	**≤ 1.0 (OK)**	$= P_\alpha / Z'_\alpha$	