

STRUCTURAL CALCULATIONS

NEW ADDITION RESIDENCE FOR:

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PREPARED BY:



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May 20, 2024

Job No. EA - Others

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FL P.E. # 91272

STRUCTURAL CALCULATIONS

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- Design Criteria.

- Wind Loads

• ASCE 7-22

$$V = 175 \text{ MPH (U.T.)}$$

Exp.: C

Cat.: II

Roof Loads :

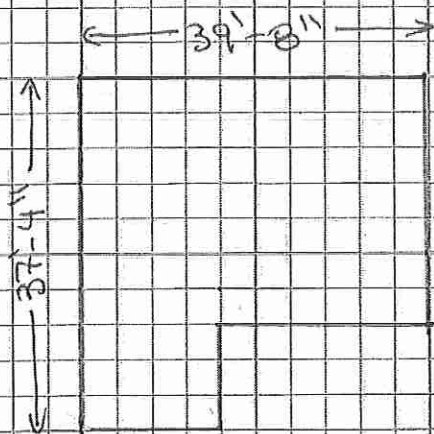
$$SBL = 20 \text{ PSF}$$

$$\text{System} = 5 \text{ PSF}$$

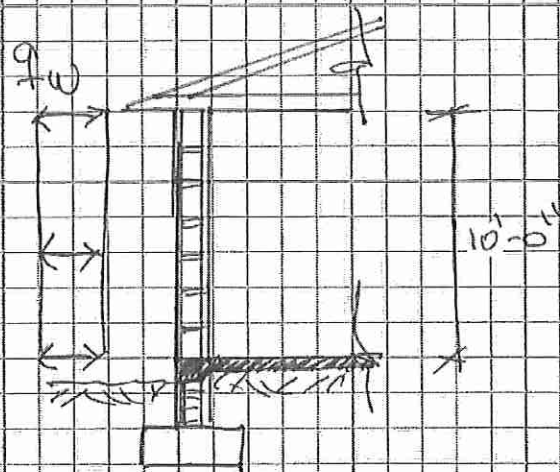
$$LL = 30 \text{ PSF}$$

$$DL = 25 \text{ PSF}$$

- Building Overall Dimensions for Wind Calculations

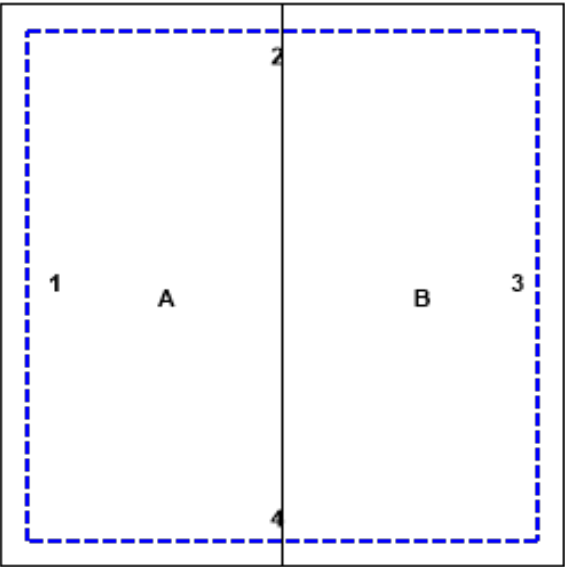


Plan View



Typical Section

Project Comments: All Wind Loads are service / ASD LOads



Disclaimer/Copyright

These calculations are derived from various editions of the ASCE 7 Standard (ASCE 7-98, ASCE 7-02, ASCE 7-05, ASCE 7-10, ASCE 7-16, ASCE7-22) to determine the minimum wind loads and performance goals for buildings, other structures and their nonstructural components that are subject to building code requirements. While the information presented by this program is believed to be correct, SDG, Inc. neither intends for this program to replace the sound judgment of a competent professional, having knowledge and experience in the relevant field(s) of practice, nor to substitute for the standard of care required of such a professional in interpreting and applying the results of the report provided by the WLS program.

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Signature / Stamp

Site Information

Wind Dir.	Exposure
1	C
2	C
3	C
4	C

Basic Wind Speed: 136 mph
Topography: Flat

Optional Factors

This project uses load combinations from ASCE 7.

Structure Information

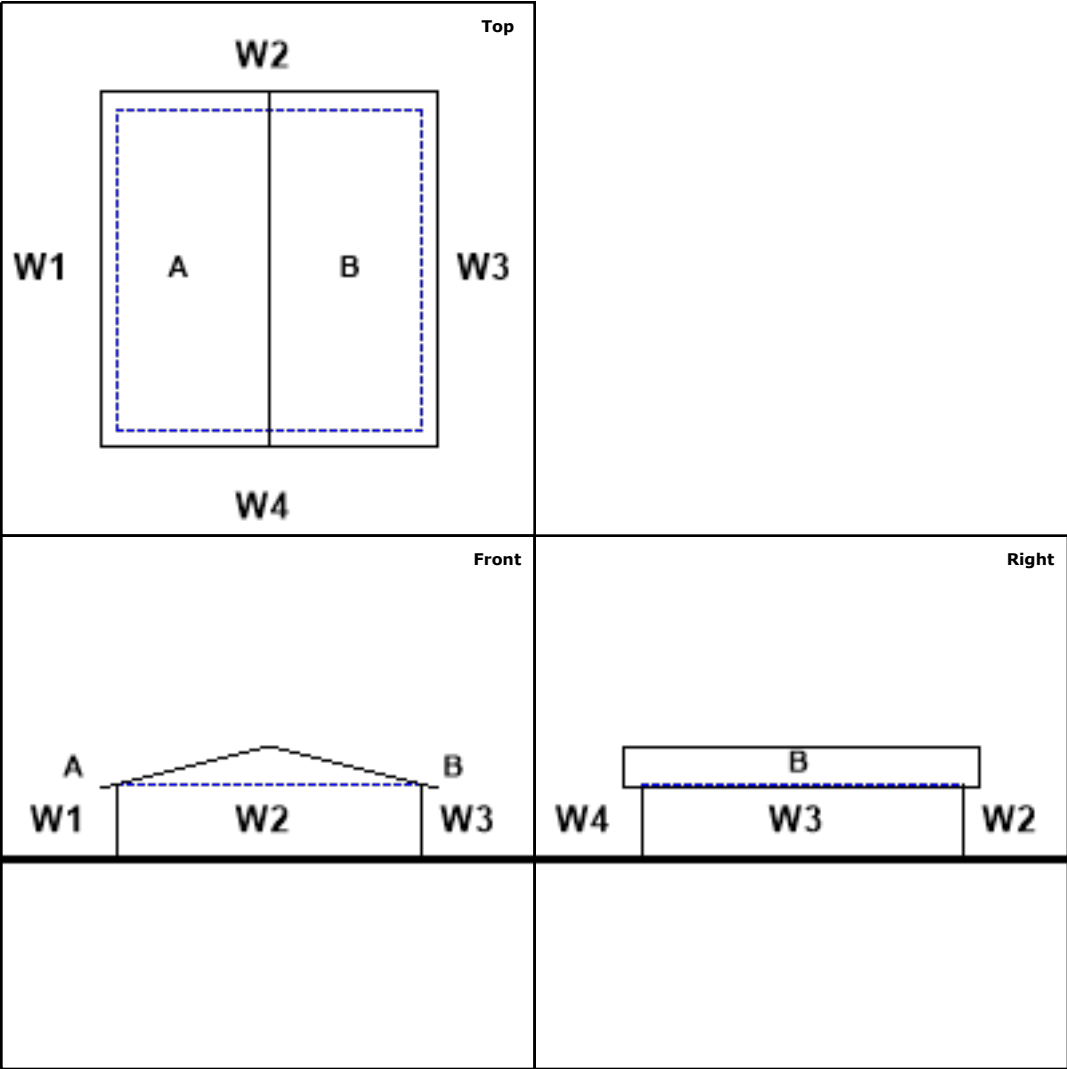
Structure Type: Building
Structure Category: II
Enclosure Classification: Enclosed

Main Section

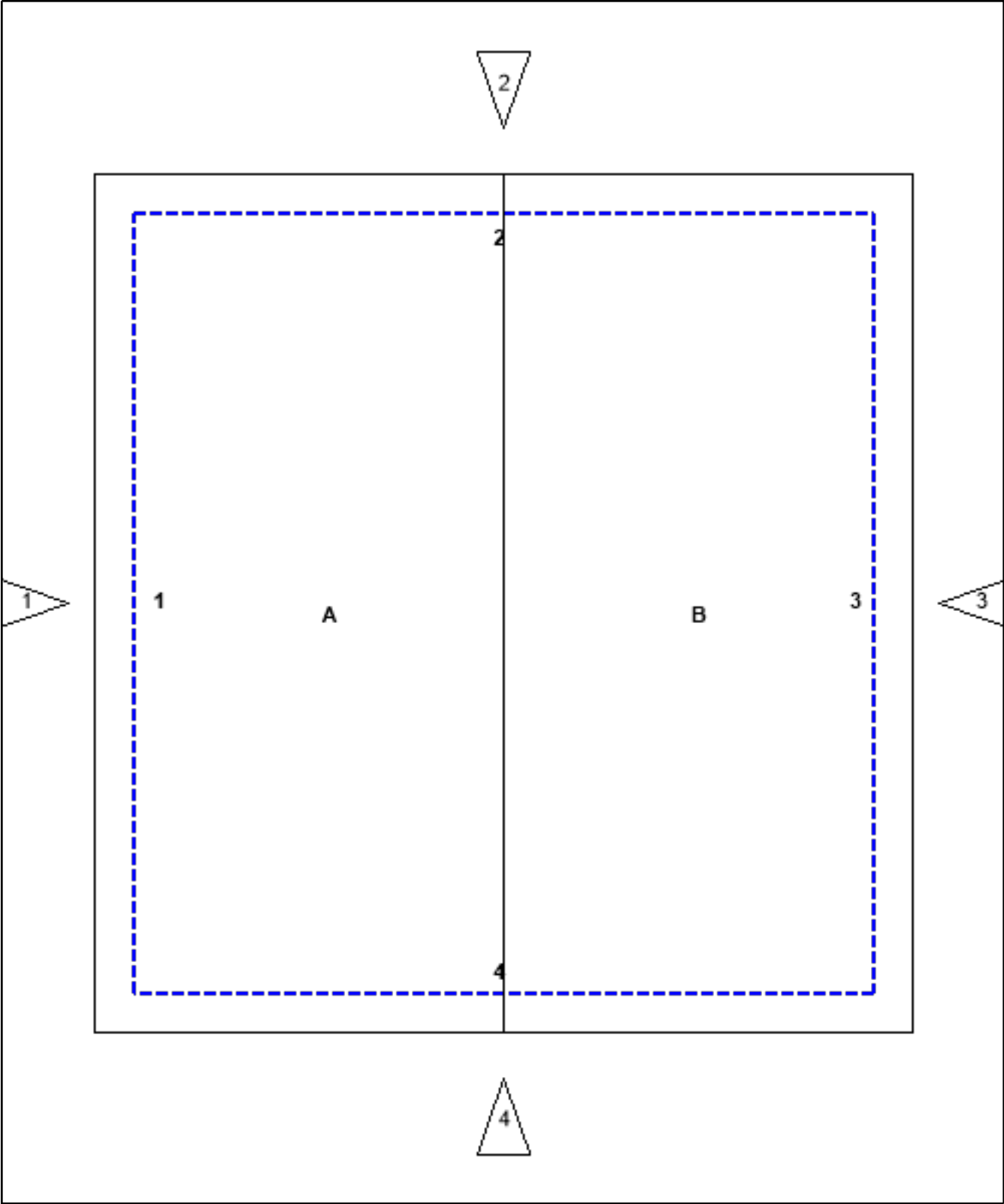
Wall	Length (ft)	Overhang (ft)
1	39.0	2.00
2	37.0	2.00
3	39.0	2.00
4	37.0	2.00

Eave Height: 9.20 ft
Parapet Height: 0.00 ft
Parapet Enclosure: Solid
Roof Shape: Gabled

Roof	Slope (X:12)
A & B	3.00



Composite Drawing



Components & Cladding

<u>Description</u>	<u>Wall/Roof</u>	<u>Surface</u>	<u>Zone</u>	<u>Span</u>	<u>Width</u>	<u>Area</u>
Roofing	Roof	A	(All)	1.000 ft	10.00 ft	10.00 ft ²
Wall	Wall	1	(All)	10.00 ft	3.00 ft	33.3 ft ²
Windows / Door	Wall	1	(All)	1.000 ft	10.00 ft	10.00 ft ²
Truss A	Roof	A	(All)	15.7 ft	2.00 ft	81.8 ft ²
Truss B	Roof	A	(All)	24.7 ft	2.00 ft	203 ft ²
Girder 1	Roof	A	(All)	15.7 ft	13.3 ft	209 ft ²

ASCE 7-22 Wind Load Report

Components & Cladding

May 20, 2024

This data was calculated using the components and cladding method.

Component Description	Surface	Zone	z (ft)	q (psf)	GCp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
Roofing	A	1	11.5	40.3	0.600	0.180	20.5	14.4	26.7
			11.5	40.3	-2.00		-68.5	-74.6	-62.3
			11.5	40.3	-3.10		-106		
		2	11.5	40.3	0.600	0.180	20.5	14.4	26.7
			11.5	40.3	-2.70		-92.4	-98.6	-86.3
			11.5	40.3	-4.10		-140		
		3	11.5	40.3	0.600	0.180	20.5	14.4	26.7
			11.5	40.3	-3.60		-123	-129	-117
			11.5	40.3	-4.70		-161		
Wall	1	4	11.5	40.3	0.908	0.180	31.1	24.9	37.2
			11.5	40.3	-1.01		-34.5	-40.7	-28.3
		5	11.5	40.3	0.908		31.1	24.9	37.2
			11.5	40.3	-1.22		-41.6	-47.8	-35.4
Windows / Door	1	4	11.5	40.3	1.00	0.180	34.2	28.1	40.4
			11.5	40.3	-1.10		-37.7	-43.8	-31.5
		5	11.5	40.3	1.00		34.2	28.1	40.4
			11.5	40.3	-1.40		-47.9	-54.1	-41.8
Truss A	A	1	11.5	40.3	0.389	0.180	13.3	7.17	19.5
			11.5	40.3	-1.07		-36.7	-42.9	-30.6
			11.5	40.3	-2.01		-68.9		
		2	11.5	40.3	0.389	0.180	13.3	7.17	19.5
			11.5	40.3	-1.51		-51.6	-57.8	-45.4
			11.5	40.3	-2.58		-88.5		
		3	11.5	40.3	0.389	0.180	13.3	7.17	19.5
			11.5	40.3	-1.96		-67.0	-73.1	-60.8
			11.5	40.3	-2.90		-99.1		
Truss B	A	1	11.5	40.3	0.300	0.180	10.3	4.11	16.4
			11.5	40.3	-0.673		-23.0	-29.2	-16.9
			11.5	40.3	-1.54		-52.8		
		2	11.5	40.3	0.300	0.180	10.3	4.11	16.4
			11.5	40.3	-1.00		-34.2	-40.4	-28.1
			11.5	40.3	-1.94		-66.4		
		3	11.5	40.3	0.300	0.180	10.3	4.11	16.4
			11.5	40.3	-1.80		-61.6	-67.8	-55.5
			11.5	40.3	-2.67		-91.4		
Girder 1	A	1	11.5	40.3	0.300	0.180	10.3	4.11	16.4
			11.5	40.3	-0.660		-22.6	-28.8	-16.4
			11.5	40.3	-1.53		-52.3		
		2	11.5	40.3	0.300	0.180	10.3	4.11	16.4
			11.5	40.3	-1.00		-34.2	-40.4	-28.1
			11.5	40.3	-1.93		-66.2		
		3	11.5	40.3	0.300	0.180	10.3	4.11	16.4
			11.5	40.3	-1.80		-61.6	-67.8	-55.5
			11.5	40.3	-2.67		-91.3		

ASCE 7-22 Wind Load Report

Main Wind Force Resisting System - Wind Direction 1

May 20, 2024

This data was calculated using the building of all heights method.

#	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
1	Windward Wall	9.20	40.3	0.887	0.800	0.180	24.3	18.1	30.4
	Overhang Top	11.5	40.3		-0.070		-2.13		
		11.5	40.3		-0.587		-17.8		
	Overhang Bot	9.20	40.3		0.800		24.3		
2	Side Wall	11.5	40.3	0.887	-0.700	0.180	-21.2	-27.4	-15.1
3	Leeward Wall	11.5	40.3	0.887	-0.500	0.180	-15.2	-21.3	-9.01
4	Side Wall	11.5	40.3	0.887	-0.700	0.180	-21.2	-27.4	-15.1
A	Windward Roof	11.5	40.3	0.887	-0.070	0.180	-2.13	-8.29	4.03
		11.5	40.3		-0.587		-17.8	-24.0	-11.7
B	Leeward Roof	11.5	40.3	0.887	-0.471	0.180	-14.3	-20.5	-8.13

This is load case 1 in ASCE 7-22 Figure 27.3-8. See Figure 27.3-8 for other cases.

ASCE 7-22 Wind Load Report

Main Wind Force Resisting System - Wind Direction 2

May 20, 2024

This data was calculated using the building of all heights method.

#	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
1	Side Wall	11.5	40.3	0.887	-0.700	0.180	-21.3	-27.4	-15.1
2	Windward Wall	11.5	40.3	0.887	0.800	0.180	24.3	18.1	30.5
		13.8	40.3				24.3	18.1	30.5
	Overhang Top	11.5	40.3		-0.900		-27.3		
	Overhang Bot	11.5	40.3		0.800		24.3		
3	Side Wall	11.5	40.3	0.887	-0.700	0.180	-21.3	-27.4	-15.1
4	Leeward Wall	11.5	40.3	0.887	-0.489	0.180	-14.9	-21.0	-8.70
A&B Roof		0.00 - 5.76*	40.3	0.887	-0.900	0.180	-27.3	-33.5	-21.2
		5.76 - 11.5*	40.3				-27.3	-33.5	-21.2
		11.5 - 23.0*	40.3		-0.500		-15.2	-21.4	-9.03
		23.0 - 39.0*	40.3		-0.300		-9.11	-15.3	-2.95
		0.00 - 39.0*	40.3		-0.180		-5.47	-11.6	0.694

This is load case 1 in ASCE 7-22 Figure 27.3-8. See Figure 27.3-8 for other cases.

* Distance from windward edge.

ASCE 7-22 Wind Load Report

Main Wind Force Resisting System - Wind Direction 3

May 20, 2024

This data was calculated using the building of all heights method.

#	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
1	Leeward Wall	11.5	40.3	0.887	-0.500	0.180	-15.2	-21.3	-9.01
2	Side Wall	11.5	40.3	0.887	-0.700	0.180	-21.2	-27.4	-15.1
3	Windward Wall	9.20	40.3	0.887	0.800	0.180	24.3	18.1	30.4
	Overhang Top	11.5	40.3		-0.070		-2.13		
		11.5	40.3		-0.587		-17.8		
	Overhang Bot	9.20	40.3		0.800		24.3		
4	Side Wall	11.5	40.3	0.887	-0.700	0.180	-21.2	-27.4	-15.1
B	Windward Roof	11.5	40.3	0.887	-0.070	0.180	-2.13	-8.29	4.03
		11.5	40.3		-0.587		-17.8	-24.0	-11.7
A	Leeward Roof	11.5	40.3	0.887	-0.471	0.180	-14.3	-20.5	-8.13

This is load case 1 in ASCE 7-22 Figure 27.3-8. See Figure 27.3-8 for other cases.

ASCE 7-22 Wind Load Report

Main Wind Force Resisting System - Wind Direction 4

May 20, 2024

This data was calculated using the building of all heights method.

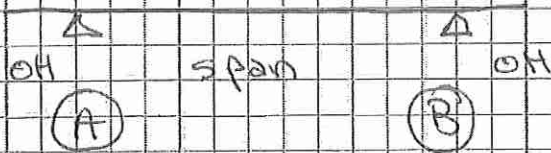
#	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
1	Side Wall	11.5	40.3	0.887	-0.700	0.180	-21.3	-27.4	-15.1
2	Leeward Wall	11.5	40.3	0.887	-0.489	0.180	-14.9	-21.0	-8.70
3	Side Wall	11.5	40.3	0.887	-0.700	0.180	-21.3	-27.4	-15.1
4	Windward Wall	11.5	40.3	0.887	0.800	0.180	24.3	18.1	30.5
		13.8	40.3				24.3	18.1	30.5
	Overhang Top	11.5	40.3		-0.900		-27.3		
	Overhang Bot	11.5	40.3		0.800		24.3		
A&B Roof		0.00 - 5.76*	40.3	0.887	-0.900	0.180	-27.3	-33.5	-21.2
		5.76 - 11.5*	40.3				-27.3	-33.5	-21.2
		11.5 - 23.0*	40.3		-0.500		-15.2	-21.4	-9.03
		23.0 - 39.0*	40.3		-0.300		-9.11	-15.3	-2.95
		0.00 - 39.0*	40.3		-0.180		-5.47	-11.6	0.694

This is load case 1 in ASCE 7-22 Figure 27.3-8. See Figure 27.3-8 for other cases.

* Distance from windward edge.

- Roof Design -

- Truss A.: span = 15'-8" OH = 2'-0"



$$q_{DL} = (25) \times 2' = 50 \text{ \#/ft}$$

$$q_{UL} = (30) \times 2' = 60 \text{ \#/ft}$$

$$q_{w \text{ Avg}} = \frac{z_1 + z_2}{2}$$

$$q_{w \text{ Avg}} = \frac{(42.9 + 57.8)}{2}$$

$$q_{w \text{ Avg}} = 50.35 \text{ psf}$$

$$q_{w \text{ (Avg)}} = 45.4 \text{ psf}$$

Reactions:

$$G_A = G_B = (110) \times (15'-8" + 2')$$

$$G_A = G_B = 1082 \text{ \#}$$

$$U_A = U_B = (45.4) \times 2' \times \left(\frac{15'-8" + 2'}{2} \right)$$

$$U_A = U_B = 893 \text{ \#}$$

→ Use Strap Type 

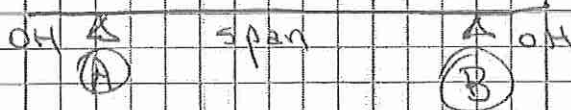
NU-VUE NYSTA-20

$$U_{Allow} = 1426 \text{ \#}$$

- Truss B:

$$\therefore \text{Span} = 24'-8''$$

$$\text{OH} = 2'-0''$$



$$q_{DL} = (25) \times 2' = 50 \text{ \#/ft}$$

$$q_{LL} = (30) \times 2' = 60 \text{ \#/ft}$$

$$q_{WE} = \frac{(29.2 + 40.4)}{2} = 34.8$$

Reactions:

$$G_A = G_B = (110) \times \left(\frac{24'-8''}{2} + 2' \right) = 1580 \text{ \#}$$

$$U_A = U_B = (34.8 \times 2') \times \left(\frac{24'-8''}{2} + 2' \right) = 998 \text{ \#}$$

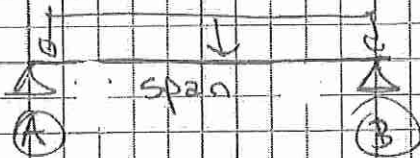
$\Rightarrow$ Use strap  NU-NUE NVHTA-20

$$U_{allow} = 2649 \text{ \#}$$

→ Girder 1

$$\text{Span} = 15'-8''$$

$$\text{B Trib.} = \frac{24'-8''}{2} + 1' = 13'-4''$$



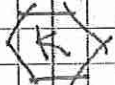
$$q_{DL} = 50 \text{ \#/ft}$$

$$q_u = 60 \text{ \#/ft}$$

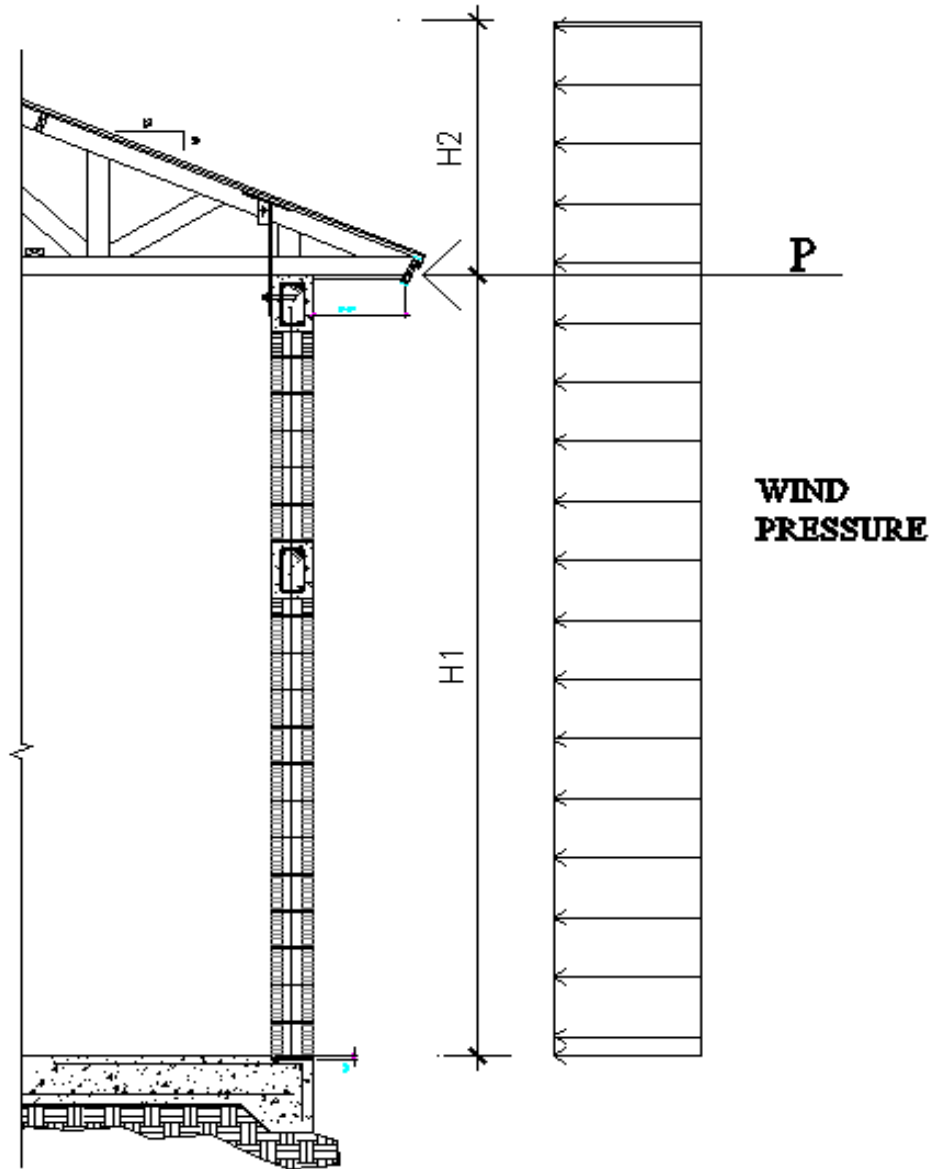
$$q_w = 34.8 \text{ PSF (as Truss B)} - 5 = 29.8$$

$$\Rightarrow G_A = G_B = (110) \times 13'-4'' \times \frac{15'-8''}{2} = 11490 \text{ \#}$$

$$U_A = U_B = (29.8) \times 2' \times 13'-4'' \times \frac{15'-8''}{2} = 6225 \text{ \#}$$

→ Use Strap  MU-VUE NYUCB-2

$$U_{Allow} = 9600 \text{ \#}$$



Calculation: ROOF DIAPHRAGM ANALYSIS

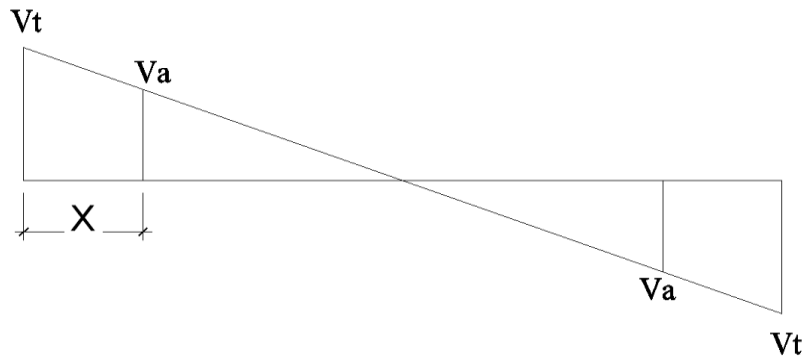
Distance between supports (Second Floor) :	H1 =	10.00 ft	(Winward + Leeward)
Distance from Eave to Ridge :	H2 =	3.83 ft	
Wind Pressure:	W.P	39.40 psf	
Enter Shear Capacity of Diaphragm Without Blocking	Va=	280.00 plf	
Width Perp. To Wind Dir (B)	B=	39.67 ft	
Width Paral. To Wind Dir (L)	L=	37.33 ft	
<u>-Compute Horizontal Load on Diaphragm (P)</u>			

$$P = W.P \times (H1/2 + H2) = 347.90 \text{ plf}$$

-Compute for Shear:

Total Reaction:	R =	$0.5 \times P \times B =$	6900.64 lbs	
Shear (Pounds per Foot):	V =	$R/L =$	184.85 plf	< Va OK

-Compute Required Length to be Blocked (X):



$$\text{Length (X):} \longrightarrow X = B/2 - (B \times Va)/2 \times Vt = \text{NA}$$

CONCLUSION: NO BLOCKING IS REQUIRED

Calculation: ROOF DIAPHRAGM ANALYSIS

Distance between supports (Second Floor) :	H1 =	10.00 ft	
Distance from Eave to Ridge :	H2 =	3.83 ft	
Wind Pressure:	W.P	39.40 psf	(Winward + Leeward)
Enter Shear Capacity of Diaphragm Without Blocking	Va=	280.00 plf	
Width Perp. To Wind Dir (B)	B=	37.33 ft	
Width Paral. To Wind Dir (L)	L=	39.67 ft	

-Compute Horizontal Load on Diaphragm (P)

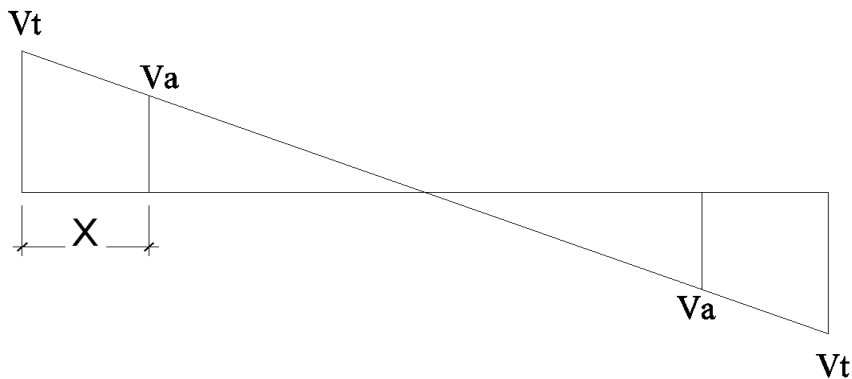
$$P = W.P \times (H1/2 + H2) = 347.90 \text{ plf}$$

-Compute for Shear:

$$\text{Total Reaction: } R = 0.5 \times P \times B = 6493.59 \text{ lbs}$$

$$\text{Shear (Pounds per Foot): } V = R/L = 163.69 \text{ plf} < Va \text{ OK}$$

-Compute Required Length to be Blocked (X):



$$\text{Length (X): } \longrightarrow X = B/2 - (B \times Va)/2 \times Vt = NA$$

CONCLUSION: NO BLOCKING IS REQUIRED

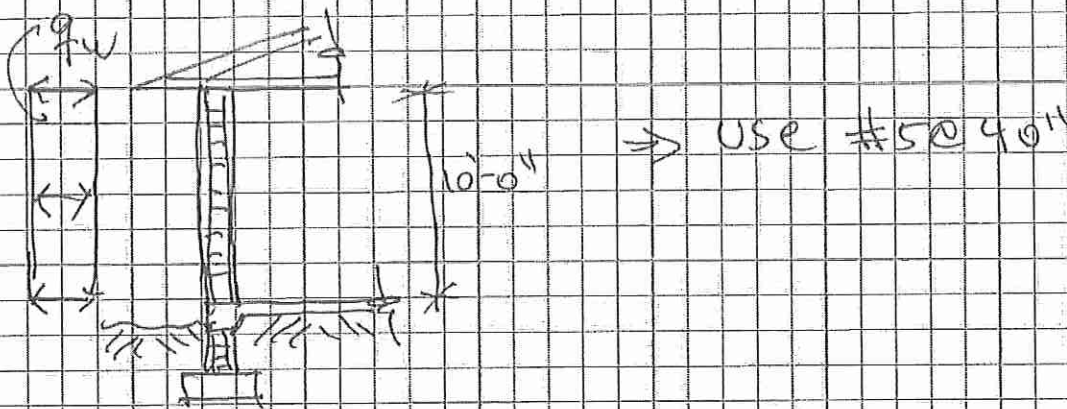
- CMU Wall Design.

Data:

$$h = 10'-0"$$

wind loads:

$$q_w = 47.6 \text{ PSF (Service)}$$



- Foundation Design.

Data:

$$\text{Soil Bearing Capacity} = 2000 \text{ PSF}$$

loads:

$$\text{Wall} : (60) \times 10'-0" = 600 \text{ \# / ft.}$$

$$\text{Roof} : (55) \times \left(\frac{24'-0"}{2} + 2' \right) = 789 \text{ \# / ft.}$$

$$q_t = 1389 \text{ \# / ft.}$$

$$\Rightarrow B \geq \frac{(q_t)}{(2000 - 150)} = \frac{1389}{1850} = 0.75' \Rightarrow \text{USE W/F-16}$$

16" x Cont. x 12"
w/ 2 #5 Cont.

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 20 OCT 2023, 7:57AM

Masonry Slender Wall

File = Z:\PMOWK0-T\Personal\ED4015-4\LDJYN-5\Calcs\Lau Resd.ec6
Software copyright ENERCALC, INC. 1983-2019, Build:10.19.1.30

Lic. #: KW-06012430

DESCRIPTION: Wall

Code References

Calculations per ACI 530-11, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-16

General Information

Calculations per ACI 530-11, IBC 2012, CBC 2013, ASCE 7-10

Construction Type : Grouted Hollow Concrete Masonry

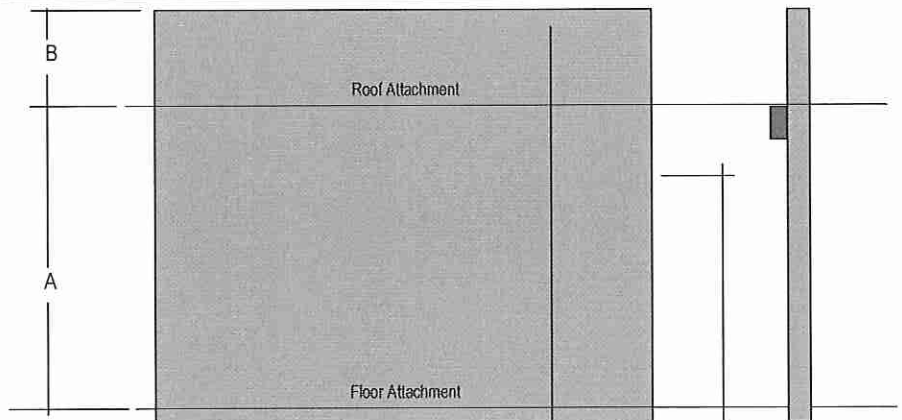
F'm	=	1.50 ksi	Nom. Wall Thickness	8 in	Temp Diff across thickness	=	deg F
Fy - Yield	=	60.0 ksi	Actual Thickness	7.625 in	Min Allow Out-of-plane Defl Ratio	=	0.0
Fr - Rupture	=	61.0 psi	Rebar "d" distance	3.8125 in	Minimum Vertical Steel %	=	0.0020
Em = f'm *	=	900.0	Lower Level Rebar . . .				
Max % of ρ_{bal}	=	0.1066	Bar Size	# 5			
Grout Density	=	140 pcf	Bar Spacing	40 in			
Block Weight		Normal Weight					
Wall Weight	=	56.0 psf					

Wall is grouted at rebar cells only

One-Story Wall Dimensions

A Clear Height	=	10.0 ft
B Parapet height	=	ft

Wall Support Condition Top & Bottom Pinned



Lateral Loads

Wind Loads :

Full area WIND load 79.330 psf

Seismic Loads :

Wall Weight Seismic Load Input Method : Direct entry of Lateral Wall Weight
Seismic Wall Lateral Load 25.0 psf

Fp 1.0 = 25.0 psf

Project Title:
Engineer:
Project ID:
Project Descr:

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Masonry Slender Wall

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DESCRIPTION: Wall

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

Governing Load Combination . . .		Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +0.90D+W	Maximum Bending Stress Ratio = 0.6923			
		Max Mu	0.9968 k-ft	Phi * Mn	1.440 k-ft
PASS	Service Deflection Check +0.60W	Actual Defl. Ratio L/	1,107	Allowable Defl. Ratio	150.0
		Max. Deflection	0.1085 in		
PASS	Axial Load Check +1.20D+W	Max Pu / Ag	6.354 psi	Max. Allow. Defl.	0.80 in
		Location	4.833 ft	0.2 * fm	300.0 psi
PASS	Reinforcing Limit Check	Actual As/bd	0.002033	Max Allow As/bd	0.1066
Maximum Reactions . . . for Load Combination....					
		Top Horizontal	W Only		0.3967 k
		Base Horizontal	W Only		0.3967 k
		Vertical Reaction	+D+0.5250E		0.560 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load		Mcr k-ft	Mu k-ft	Phi	Moment Values			0.6 * rho bal
	Pu k	0.2*fm*b*t k				Phi Mn k-ft	As in^2	As Ratio	
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
+1.20D+0.50W at 4.67 to 5.00	0.358	16.920	0.45	0.50	0.90	1.47	0.093	0.0020	0.1065
+1.20D+W at 4.67 to 5.00	0.358	16.920	0.45	1.00	0.90	1.47	0.093	0.0020	0.1065
+0.90D+W at 4.67 to 5.00	0.269	16.920	0.45	1.00	0.90	1.44	0.093	0.0020	0.1065
+1.20D+E at 4.67 to 5.00	0.358	16.920	0.45	0.31	0.90	1.47	0.093	0.0020	0.1065
+0.90D+E at 4.67 to 5.00	0.269	16.920	0.45	0.31	0.90	1.44	0.093	0.0020	0.1065

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values		I gross in^4	Stiffness I cracked in^4	I effective in^4	Deflections	
		Mcr k-ft	Mactual k-ft				Deflection in	Defl. Ratio
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.60W at 5.00 to 5.33	0.280	0.45	0.60	335.60	20.60	24.318	0.107	1,126.5
+D+0.450W at 4.67 to 5.00	0.299	0.45	0.45	335.60	20.65	335.600	0.018	6,777.6
+0.60D+0.60W at 5.00 to 5.33	0.168	0.45	0.60	335.60	20.28	23.974	0.107	1,118.6
+D+0.70E at 4.67 to 5.00	0.299	0.45	0.22	335.60	20.65	335.600	0.009	13,825.8
+D+0.5250E at 4.67 to 5.00	0.299	0.45	0.16	335.60	20.65	335.600	0.007	18,434.3
+0.60D+0.70E at 4.67 to 5.00	0.179	0.45	0.22	335.60	20.31	335.600	0.009	13,830.8
+0.60W at 5.00 to 5.33	0.000	0.45	0.59	335.60	19.79	23.454	0.108	1,106.5

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	0.560 k
+D+0.60W	0.2 k	0.24 k	0.560 k
+D+0.450W	0.2 k	0.18 k	0.560 k
+0.60D+0.60W	0.2 k	0.24 k	0.336 k
+D+0.70E	0.1 k	0.09 k	0.560 k
+D+0.5250E	0.1 k	0.07 k	0.560 k
+0.60D+0.70E	0.1 k	0.09 k	0.336 k
W Only	0.4 k	0.40 k	0.000 k

Conc. Beam Design

RB-1 (8" x 35")
El. 10'-0"



DL: Roof: $(25) \times (2' + 1') = 75 \text{ \#/ft.}$

Wall: $(60) \times 2' - 8" \times .6 = 100 \text{ \#/ft.}$

$\Rightarrow q_{DL} = 175 \text{ \#/ft.}$

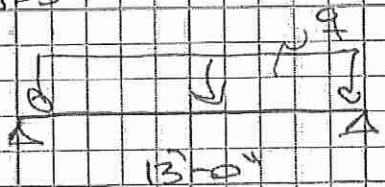
UL: Roof: $(30) \times (2' + 1') = 90 \text{ \#/ft.}$

$\Rightarrow \text{USE } 8" \times 35"$

W/ 2 #6 T & B

ST #3 @ 12"

RB-2 (8" x 24")
El. 10'-0"



$\Rightarrow \text{USE } 8" \times 24"$

W/ 2 #6 (T & B)

ST #3 @ 10"

$q_{DL} = \overset{\text{(Roof)}}{(25) \times 1'} + \overset{\text{(Eyebrow)}}{(50 + 20) \times 2'} + \overset{\text{(Wall)}}{(60) \times 3' - 10"} = 395 \text{ \#/ft.}$

$q_u = (30) \times 1' + (30) \times 2' = 90 \text{ \#/ft.}$

Project Title:
Engineer:
Project ID:
Project Descr:

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Concrete Beam

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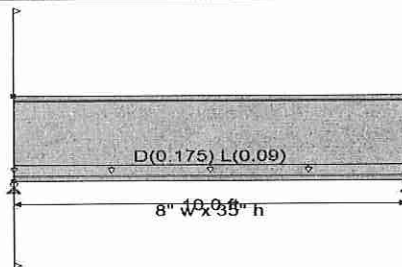
DESCRIPTION: RB-1

CODE REFERENCES

Calculations per ACI 318-11, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

f'_c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2}$	=	410.792 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.02 ksi	F_y - Stirrups	=	60.0 ksi
f_y - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	3
			Number of Resisting Legs Per Stirrup	=	2



Cross Section & Reinforcing Details

Rectangular Section, Width = 8.0 in, Height = 35.0 in

Span #1 Reinforcing....

2-#6 at 2.0 in from Top, from 0.0 to 10.0 ft in this span

2-#6 at 2.0 in from Bottom, from 0.0 to 10.0 ft in this span

Beam self weight calculated and added to loads

Load for Span Number 1

Uniform Load : $D = 0.1750$, $L = 0.090$ k/ft, Tributary Width = 1.0 ft, (Q)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.069 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.000 in	Ratio = 0 < 360.0
Mu : Applied		8.654 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 < 360.0
Mn * Phi : Allowable		125.820 k-ft	Max Downward Total Deflection	0.001 in	Ratio = 87045 >= 360.0
Location of maximum on span		4.991 ft	Max Upward Total Deflection	0.000 in	Ratio = 0 < 360.0
Span # where maximum occurs		Span # 1			

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	2.735	2.735
Overall MINimum	0.450	0.450
D Only	2.285	2.285
+D+L	2.735	2.735
+D+0.750L	2.622	2.622
+0.60D	1.371	1.371
L Only	0.450	0.450

Overall Maximum Deflections

Load Combination	Span	Max. "+" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L	1	0.0014	5.000		0.0000	0.000

Project Title:
 Engineer:
 Project ID:
 Project Descr:

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Concrete Beam

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DESCRIPTION: RB-2

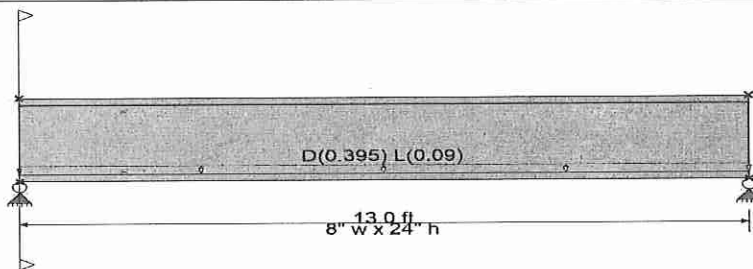
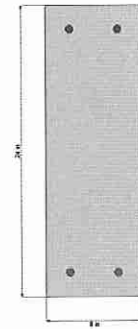
CODE REFERENCES

Calculations per ACI 318-11, IBC 2012, CBC 2013, ASCE 7-10

Load Combination Set : ASCE 7-16

Material Properties

f'_c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2}$	=	410.792 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.02 ksi	Fy - Stirrups	=	60.0 ksi
f_y - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	3
			Number of Resisting Legs Per Stirrup	=	2



Cross Section & Reinforcing Details

Rectangular Section, Width = 8.0 in, Height = 24.0 in

Span #1 Reinforcing....

2-#6 at 2.0 in from Top, from 0.0 to 13.0 ft in this span

2-#6 at 2.0 in from Bottom, from 0.0 to 13.0 ft in this span

Beam self weight calculated and added to loads

Load for Span Number 1

Uniform Load : D = 0.3950, L = 0.090 k/ft, Tributary Width = 1.0 ft, (Q)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.218 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.002 in	Ratio = 77632 >=360.
Mu : Applied		17.956 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 <360.0
Mn * Phi : Allowable		82.260 k-ft	Max Downward Total Deflection	0.015 in	Ratio = 10300 >=360.
Location of maximum on span		6.488 ft	Max Upward Total Deflection	0.000 in	Ratio = 0 <360.0
Span # where maximum occurs		Span # 1			

Vertical Reactions

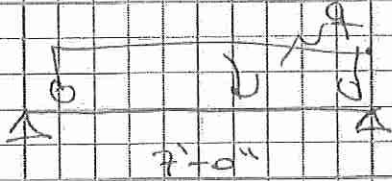
Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	4.409	4.409
Overall MINimum	0.585	0.585
D Only	3.824	3.824
+D+L	4.409	4.409
+D+0.750L	4.263	4.263
+0.60D	2.294	2.294
L Only	0.585	0.585

Overall Maximum Deflections

Load Combination	Span	Max. "+" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L	1	0.0151	6.500		0.0000	0.000

1B-1: (8" x 12") (worst cond.)
El. Nbs.



USE 8" x 12"

w/ 2#5 (top)

ST#3 @ 48"
(Not Rigid.)

$$q_{DL} = (25) \times \left(\frac{15'-8''}{2} + 2' \right) = 250 \text{ \#/ft.} \quad \left. \begin{array}{l} \text{Ratio} \\ .21 \end{array} \right\}$$

$$q_{LL} = (30) \times \left(\frac{15'-8''}{2} + 2' \right) = 295 \text{ \#/ft.}$$

$$q_{WL} = (47.6) \times \frac{10'-0''}{2} = 238 \text{ \#/ft.} \rightarrow .15$$

(LATERAL LOAD) (400 \text{ \#/ft. U.T.})

Project Title:
 Engineer:
 Project ID:
 Project Descr:

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DESCRIPTION: TB-1

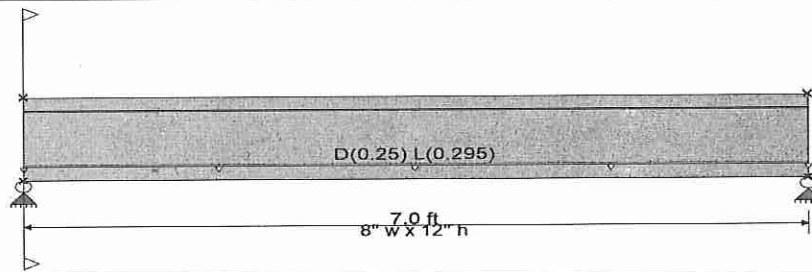
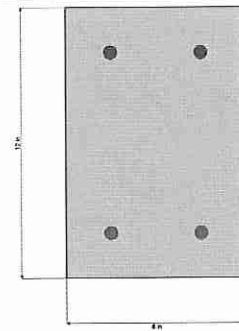
CODE REFERENCES

Calculations per ACI 318-11, IBC 2012, CBC 2013, ASCE 7-10

Load Combination Set : ASCE 7-16

Material Properties

f'_c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2}$	=	410.792 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.02 ksi	Fy - Stirrups	=	60.0 ksi
f_y - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	3
			Number of Resisting Legs Per Stirrup	=	2



Cross Section & Reinforcing Details

Rectangular Section, Width = 8.0 in, Height = 12.0 in

Span #1 Reinforcing....

2-#5 at 2.0 in from Top, from 0.0 to 7.0 ft in this span

2-#5 at 2.0 in from Bottom, from 0.0 to 7.0 ft in this span

Beam self weight calculated and added to loads

Load for Span Number 1

Uniform Load : D = 0.250, L = 0.2950 k/ft, Tributary Width = 1.0 ft, (Q)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.215 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.004 in	Ratio = 18963 >=360.0
Mu : Applied		5.439 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 <360.0
Mn * Phi : Allowable		25.332 k-ft	Max Downward Total Deflection	0.010 in	Ratio = 8718 >=360.0
Location of maximum on span		3.494 ft	Max Upward Total Deflection	0.000 in	Ratio = 0 <360.0
Span # where maximum occurs		Span # 1			

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	2.246	2.246
Overall MINimum	0.728	0.728
D Only	1.213	1.213
+D+L	2.246	2.246
+D+0.750L	1.988	1.988
+0.60D	0.728	0.728
L Only	1.032	1.032

Overall Maximum Deflections

Load Combination	Span	Max. "+" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L	1	0.0096	3.500		0.0000	0.000

Project Title:
 Engineer:
 Project ID:
 Project Descr:

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DESCRIPTION: TB-1 (Lateral Loads)

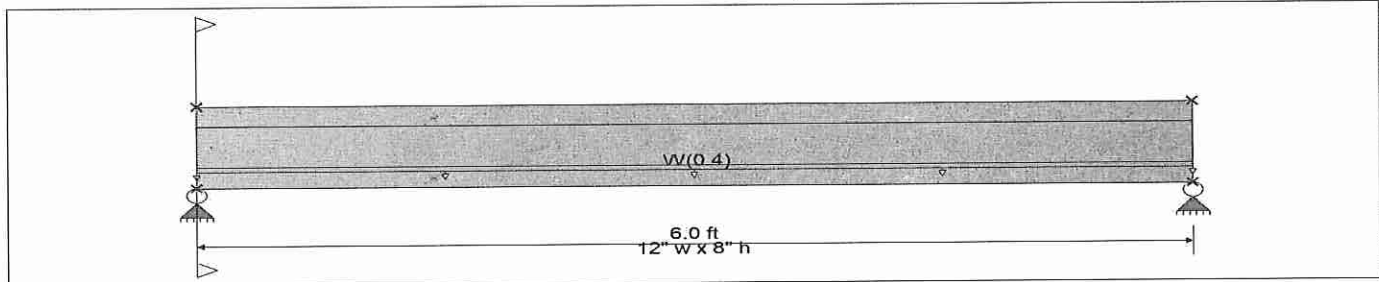
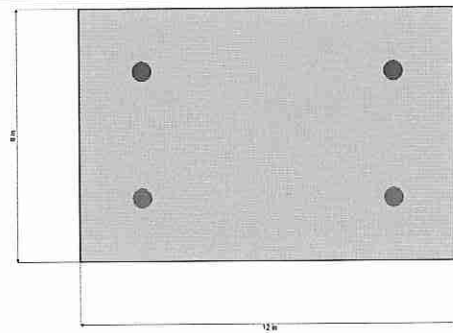
CODE REFERENCES

Calculations per ACI 318-11, IBC 2012, CBC 2013, ASCE 7-10

Load Combination Set : ASCE 7-16

Material Properties

f'_c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2}$	=	410.792 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LIWT Factor	=	1.0			
Elastic Modulus	=	3,122.02 ksi	Fy - Stirrups	=	60.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	3
			Number of Resisting Legs Per Stirrup	=	2



Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 8.0 in

Span #1 Reinforcing....

2-#5 at 2.0 in from Top, from 0.0 to 6.0 ft in this span

2-#5 at 2.0 in from Bottom, from 0.0 to 6.0 ft in this span

Beam self weight calculated and added to loads

Load for Span Number 1

Uniform Load : W = 0.40 k/ft, Tributary Width = 1.0 ft, (Q)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.150 : 1	Maximum Deflection	
Section used for this span	Typical Section	Max Downward Transient Deflection	0.007 in Ratio = 9870 >= 360.0
Mu : Applied	2.322 k-ft	Max Upward Transient Deflection	0.000 in Ratio = 0 < 360.0
Mn * Phi : Allowable	15.439 k-ft	Max Downward Total Deflection	0.007 in Ratio = 9870 >= 360.0
Location of maximum on span	2.995 ft	Max Upward Total Deflection	0.000 in Ratio = 0 < 360.0
Span # where maximum occurs	Span # 1		

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	1.200	1.200
Overall MINimum	0.174	0.174
D Only	0.290	0.290
+D+0.60W	1.010	1.010
+D+0.450W	0.830	0.830
+0.60D+0.60W	0.894	0.894
+0.60D	0.174	0.174
W Only	1.200	1.200

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
W Only	1	0.0073	3.000		0.0000	0.000