



Structural Engineering Calculations

Vila Structural Consulting Engineers, Inc.

Structural Calculations for

***New Residence for
Eliges Investors Construction
at
9841 SW 73rd Court
Pinecrest, FL***

Pages

1 to 50

Project Number: 22CB006



Jose E. Vila Alvarez

FL PE# 89027

June 21, 2022



Structural Engineering Calculations

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Project Name: 9841 SW 73rd Court - Main Structure Date: 5/3/2022

Project No. 22CB006 Engineer: J. Vila

Risk Category II
Wind Speed 175 mph
 $K_d = 0.85$
 Exposure C
 $K_{zt} = 1.00$
 Enclosure Classification **Enclosed**
 $GC_{pi} = 0.18$
 $K_z = 0.95$
 $K_e = 1.00$
 $q_z = 37.95$ psf

z = 25.5 ft
 Length = 82 ft
 Width = 58 ft
 Height = 25.5 ft
 $h/B = 0.44$
a = 5.8 ft
 Pitch = 0.25 in 12"
 $\theta = 1.19$

Service	Wind Perpendicular to Ridge (Load Case A)									
	Windward Wall				Leeward Wall				Side Wall	
	Zone	GCp	p (+Gcpi)	p (-GCpi)	Zone	GCp	p (+Gcpi)	p (-GCpi)		
	1E	0.61	16.32	29.98	4E	-0.43	-23.15	-9.49	N/A	
	1	0.40	8.35	22.01	4	-0.29	-17.84	-4.17		

Service	Wind Perpendicular to Ridge (Load Case A)									
	Windward Roof				Leeward Roof				Side Roof	
	Zone	GCp	p (+Gcpi)	p (-GCpi)	Zone	GCp	p (+Gcpi)	p (-GCpi)		
	2E	-1.07	-47.44	-33.78	3E	-0.53	-26.95	-13.28	N/A	
	2	-0.69	-33.02	-19.36	3	-0.37	-20.87	-7.21		

Service	Wind Parallel to Ridge (Load Case B)											
	Windward Wall				Leeward Wall				Side Wall			
	Zone	GCp	p (+Gcpi)	p (-GCpi)	Zone	GCp	p (+Gcpi)	p (-GCpi)	Zone	GCp	p (+Gcpi)	p (-GCpi)
	1E	-0.48	-25.05	-11.39	4E	-0.48	-25.05	-11.39	5E	0.61	16.32	29.98
	1	-0.45	-23.91	-10.25	4	-0.45	-23.91	-10.25	5	0.4	8.35	22.01

Service	Wind Parallel to Ridge (Load Case B)											
	Windward Roof				Leeward Roof				Side Wall			
	Zone	GCp	p (+Gcpi)	p (-GCpi)	Zone	GCp	p (+Gcpi)	p (-GCpi)	Zone	GCp	p (+Gcpi)	p (-GCpi)
	2E	-1.07	-47.44	-33.78	3E	-0.53	-26.95	-13.28	6E	-0.43	-23.15	-9.49
	2	-0.69	-33.02	-19.36	3	-0.37	-20.87	-7.21	6	-0.29	-17.84	-4.17



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 $\theta = 0.00$

Service	Wall Pressures (PSF)											
	Effective Wind Area ft ²											
	10		20		50		100		200		>500	
GC _p	1.00	-1.10	0.99	-1.09	0.97	-1.07	0.94	-1.04	0.88	-0.98	0.70	-0.80
	1.00	-1.40	0.99	-1.38	0.97	-1.34	0.94	-1.28	0.88	-1.16	0.70	-0.80
Zone 4	44.78	-48.58	44.32	-48.12	43.62	-47.42	42.46	-46.26	40.14	-43.93	33.40	-37.19
Zone 5	44.78	-59.97	44.32	-59.04	43.62	-57.64	42.46	-55.32	40.14	-50.67	33.40	-37.19

Service	Roof Pressure Hip Roofs (PSF) Figs. 30.3-2A - D												
	Effective Wind Area ft ²												
	GCp	10	GCp	20	GCp	50	GCp	100	GCp	200	GCp	500	
Zone 1'	-0.90	-40.99	-0.90	-40.99	-0.90	-40.99	-0.90	-40.99	-0.78	-36.25	-0.40	-22.01	A
Zone 1	-1.70	-71.35	-1.69	-70.81	-1.64	-69.18	-1.57	-66.47	-1.43	-61.05	-1.00	-44.78	A
Zone 2	-2.30	-94.12	-2.28	-93.43	-2.23	-91.34	-2.13	-87.85	-1.95	-80.88	-1.40	-59.97	A
Zone 2e	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	N/A
Zone 2n	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	N/A
Zone 2r	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	N/A
Zone 3	-3.20	-128.28	-3.16	-126.89	-3.05	-122.71	-2.87	-115.73	-2.50	-101.79	-1.40	-59.97	A
Zone 3e	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	N/A
Zone 3r	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	N/A
Pos (+) A Zones	0.3	18.22	0.29	17.80	0.26	16.53	0.20	14.42	0.2	14.42	0.2	14.42	A

Per Spread Sheet

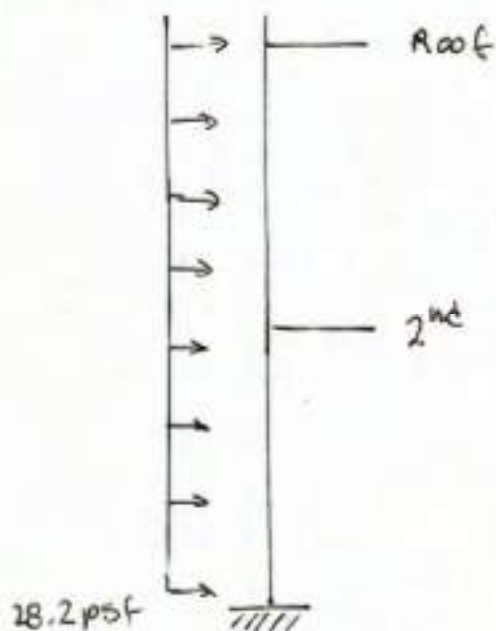
Edge	windward 29.98 psf →	Leeward -9.49 psf →	= 39.47 psf →
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Field	22.01 psf →	-4.17 psf →	= 26.18 psf. →
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Taking an average by wall length :

$$P_{ave} = \frac{39.47 \text{ psf} (2 \times 5.8 \text{ ft}) + 26.18 \text{ psf} (81.17 \text{ ft} - 2(5.8 \text{ ft}))}{81.17 \text{ ft}}$$

$$P_{ave} = 28.2 \text{ psf}$$



At Roof

$$28.2 \text{ psf} \times \frac{13'-0''}{2} = 183 \text{ plf}$$

At 2nd

$$28.2 \text{ psf} \times \frac{12'-6''}{2} + \frac{13'-0''}{2} = 360 \text{ plf.}$$

Analysis of Collector Load From Diaphragm Loading.

Diaphragm between GL A and B @ 2nd

$$360 \text{ plf} \times \left(\frac{33'-4''}{2} + \frac{38'-10''}{2} \right) = 13 \text{ K.}$$

Load happens @ collector along GL B

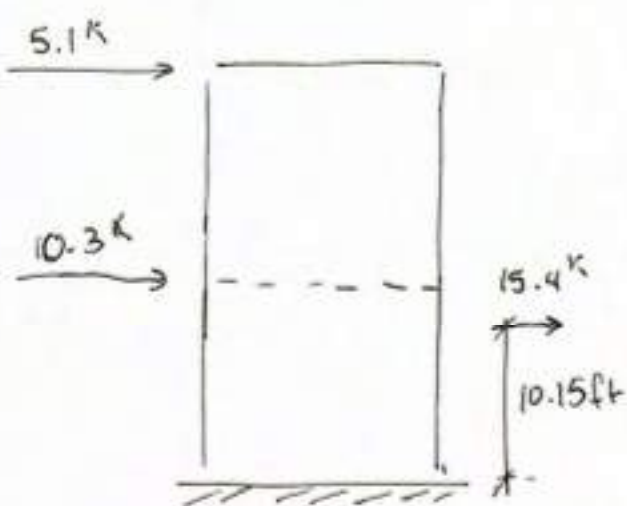
$$360 \text{ plf} \times \left(\frac{18'-2''}{2} + \frac{38'-10''}{2} \right) = 10.3 \text{ K}$$

Load happens @ collector along GL D.

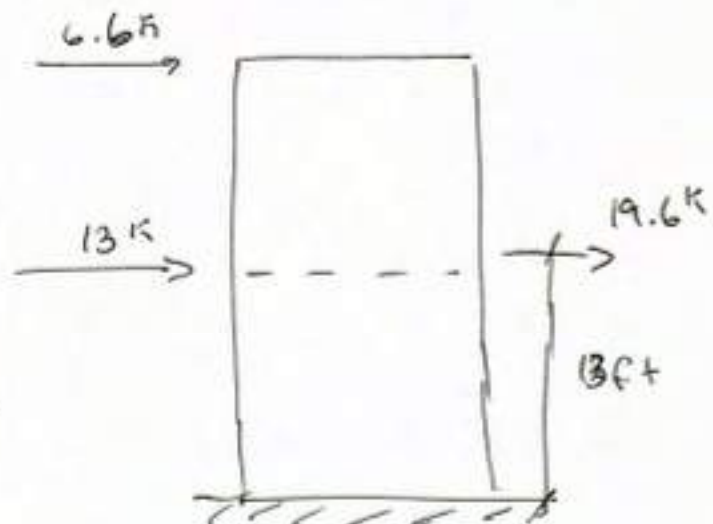
Diaphragm

$$183 \text{ plf} \times \left(\frac{33'-4''}{2} + \frac{38'-10''}{2} \right) = 6.6 \text{ K} \quad \text{Collector Along GL B}$$

$$183 \text{ plf} \times \left(\frac{18'-2''}{2} + \frac{38'-10''}{2} \right) = 5.1 \text{ K} \quad \text{Collector Along GL D}$$



Collector Along GL D



Collector Along GL B.

Critical Collector through the framing.

Collector B

CMU Properties

$f'_m =$ 2000 psi
 Grout Spacing 48"
 $A_{nv} =$ 529.10 in²

Wall Geometry

Length= 13 ft
 Height= 25.5 ft
 End Reinf.= 1 cells
 $d_v =$ 12.67 ft

Horizontal Steel

Bar #= #3
 Bar Qnt. = 1
 $A_v =$ 0.11 in²
 $s =$ 96 in
 $f_y =$ 60 ksi

Limits

$V_g =$ 0.75
 $M_u / (V_u d_v) =$ 0.941
 $V_n \leq$ 73.77 k

Loads

$P_u =$ 0 k
 $V_u =$ 35.73 k
 $M_u =$ 426 k-ft

Capacities

$V_{nm} =$ 55.67 k
 $\phi V_{nm} =$ 44.54 k
 $V_{ns} =$ 5.23 k

 $\phi V_n =$ 48.72 k

Checks

S.W. Works YES
 CMU Only YES

 CMU + Steel YES

Collector D

CMU Properties

$f'_m =$ 2000 psi
 Grout Spacing UngROUTed
 $A_{nv} =$ 360.00 in²

Wall Geometry

Length= 12 ft
 Height= 25.5 ft
 End Reinf.= 1 cells
 $d_v =$ 11.67 ft

Horizontal Steel

Bar #= #3
 Bar Qnt. = 1
 $A_v =$ 0.11 in²
 $s =$ 96 in
 $f_y =$ 60 ksi

Limits

$V_g =$ 0.75
 $M_u / (V_u d_v) =$ 0.870
 $V_n \leq$ 52.49 k

Loads

$P_u =$ 0 k
 $V_u =$ 25.72 k
 $M_u =$ 261 k-ft

Capacities

$V_{nm} =$ 39.89 k
 $\phi V_{nm} =$ 31.91 k
 $V_{ns} =$ 4.81 k

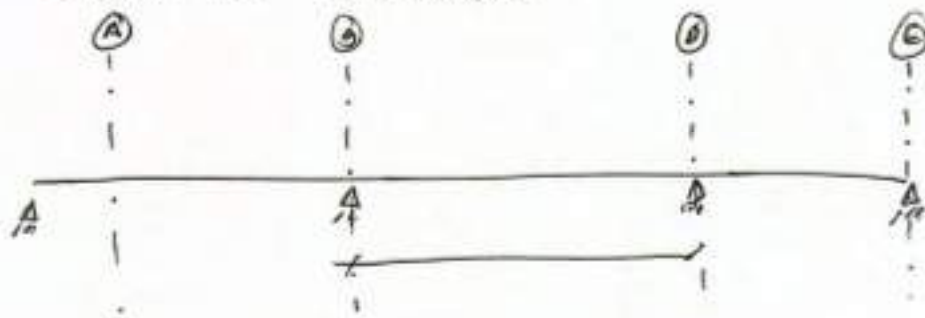
 $\phi V_n =$ 35.76 k

Checks

S.W. Works YES
 CMU Only YES

 CMU + Steel YES

Wood Roof Diaphragm



$$138 \text{ plf} \times \frac{38'-10''}{2} = 2680^{\#}$$

Collector Length (Critical @ GLO) = 22'-4"

$$\frac{2680^{\#}}{22'-4''} = 120^{\#/\text{ft}}$$

See Diaphragm Capacity

$$\frac{600^{\#/\text{ft}}}{2} = \frac{600^{\#/\text{ft}}}{2.0} = 300^{\#/\text{ft}}$$

$$300^{\#/\text{ft}} \geq 120^{\#/\text{ft}} \quad \text{OK.}$$

Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Unlocked Wood Structural Panel Diaphragms^{1,2,3,4,5}

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A				B			
					SEISMIC				WIND			
					6 in. Nail Spacing at diaphragm boundaries and supported panel edges				6 in. Nail Spacing at diaphragm boundaries and supported panel edges			
Structural I	6d	1-1/4	5/16	2	Case 1		Cases 2,3,4,5,6		Case 1		Cases 2,3,4,5,6	
					V_s (plf)	G_a (kips/in.)	V_s (plf)	G_a (kips/in.)	V_w (plf)		V_w (plf)	
Sheathing and Single-Floor	10d	1-1/2	15/32	3	OSB	PLY	OSB	PLY	460		350	
					330	9.0	250	6.0	460		350	
					370	7.0	280	4.5	520		390	
					480	8.5	360	6.0	670		505	
					530	7.5	400	5.0	740		560	
					570	14	430	9.5	800		600	
					640	12	480	8.0	895		670	
					300	9.0	220	6.0	420		310	
					340	7.0	250	5.0	475		350	
					330	7.5	250	5.0	460		350	
6d	1-1/4	3/8	3/8	3	370	6.0	280	4.0	520		390	
					430	9.0	320	6.0	600		450	
					480	7.5	360	5.0	670		505	
					460	8.5	340	5.5	645		475	
					510	7.0	380	4.5	715		530	
					480	7.5	360	5.0	670		505	
					530	6.5	400	4.0	740		560	
					510	15	380	10	715		530	
					580	12	430	8.0	810		600	
					570	13	430	8.5	800		600	
10d	1-1/2	15/32	19/32	3	640	10	480	7.0	895		670	
					640	10	480	7.0	895		670	

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

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

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

GL 6 Between B and D Lateral Diaphragm Analysis.

$$22.01 \text{ psf} \times \left(\frac{12'-6''}{2} + \frac{13'-0''}{2} \right) \times \frac{38'-10''}{2} = 5.5 \text{ K At each main concrete diaphragm.}$$

Note 22.01 psf is only windward since it does not connect to the back wall

See ENEERCALC for Beam/Bridge Analysis loaded on its strong axis.

GL 5 Between GL C and D

Beam Dim. b x d
8" x 30"

$$22.01 \text{ psf} \times \left(\frac{12'-6''}{2} + \frac{13'-0''}{2} \right) \times \frac{26'-8''}{2} = 2.7 \text{ K}$$

See ENEERCALC for Lateral Beam Analysis loaded on its weak axis.

Concrete Beam

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CBenginfo

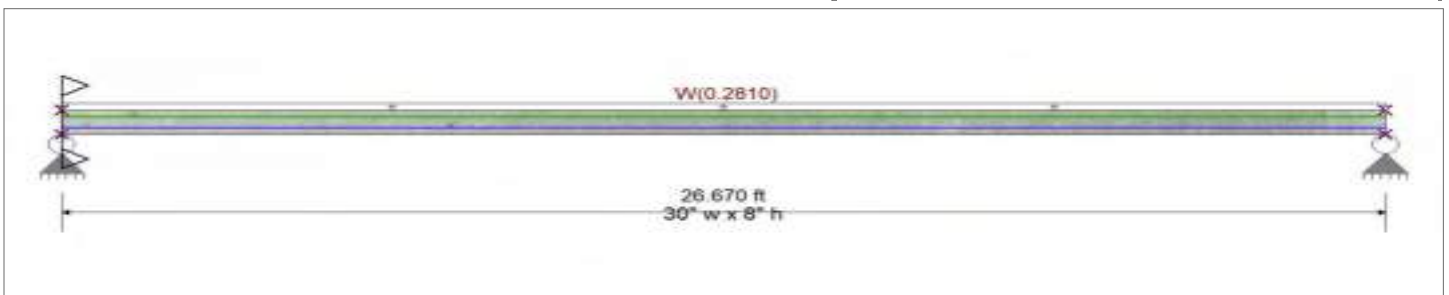
DESCRIPTION: 2B-1 LATERAL

CODE REFERENCES

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

Material Properties

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} * 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,644.15 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 = 30.0 in, Height = 8.0 in

Span #1 Reinforcing....

2-#7 at 2.30 in from Bottom, from 0.0 to 26.670 ft in this span

2-#7 at 2.30 in from Top, from 0.0 to 26.670 ft in this span

Loads on all spans...

W = 0.2810

Uniform Load on ALL spans : W = 0.2810 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.724	: 1	Maximum Deflection			
Section used for this span		Typical Section		Max Downward Transient Deflection	0.000 in	Ratio =	0 < 360.0
μ : Applied		24.984 k-ft		Max Upward Transient Deflection	0.000 in	Ratio =	0 < 360.0
$M_n * \Phi$: Allowable		34.494 k-ft		Max Downward Total Deflection	0.560 in	Ratio =	571 >= 360.0
Location of maximum on span		13.311 ft		Max Upward Total Deflection	0.000 in	Ratio =	0 < 360.0
Span # where maximum occurs		Span # 1					

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	$\Phi * M_n$ (k-ft)		Moment of Inertia (in ⁴)		
		Bottom	Top	I gross	I cr - Bottom	I cr - Top
Section 1	2- #7 @ d=5.7", 2- #7 @ d=2.3",	34.49	34.49	1,280.00	205.36	205.36

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	3.747	3.747
Overall MINimum	1.686	1.686
+D+0.60W+H	2.248	2.248
+D+0.750Lr+0.750L+0.450W+H	1.686	1.686
+D+0.750L+0.750S+0.450W+H	1.686	1.686
+0.60D+0.60W+0.60H	2.248	2.248
W Only	3.747	3.747
H Only		

Concrete Beam

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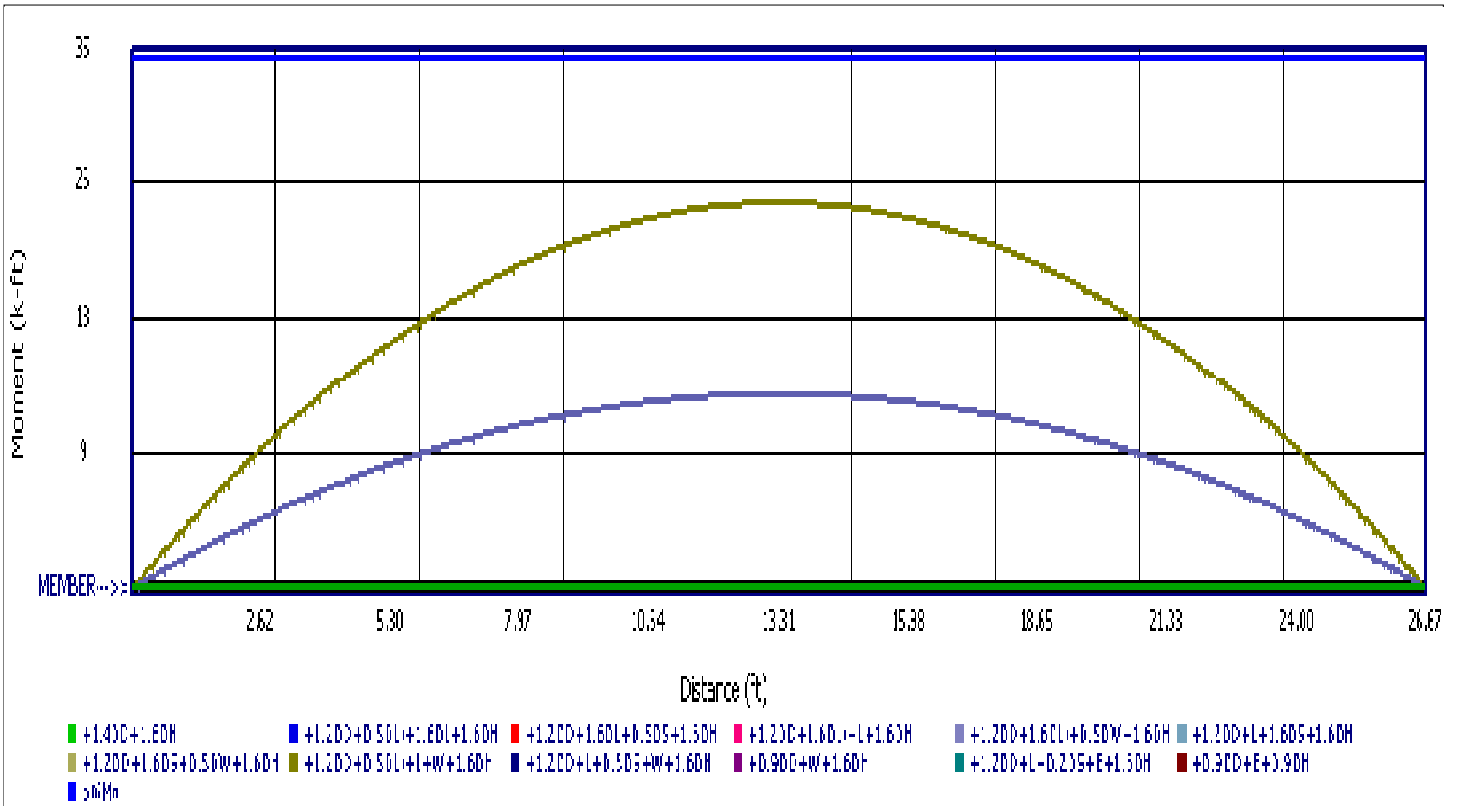
DESCRIPTION: 2B-1 LATERAL

Shear Stirrup Requirements

Entire Beam Span Length : $V_u < \Phi V_c/2$, Req'd Vs = Not Req'd 11.4.6.1, use #3 stirrups spaced at 0.000 in

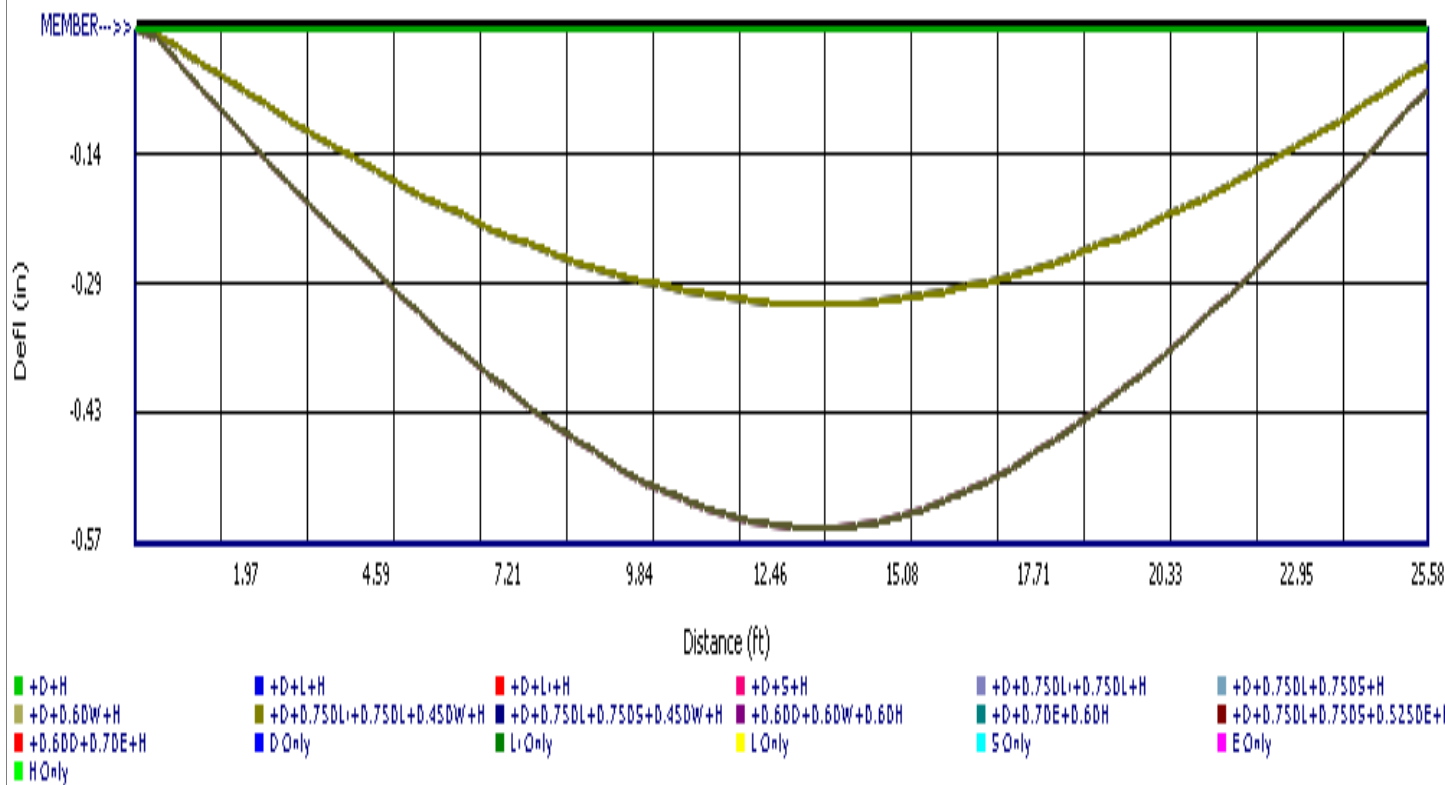
Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+0.60D+0.60W+0.60H	1	0.5602	13.335		0.0000	0.000

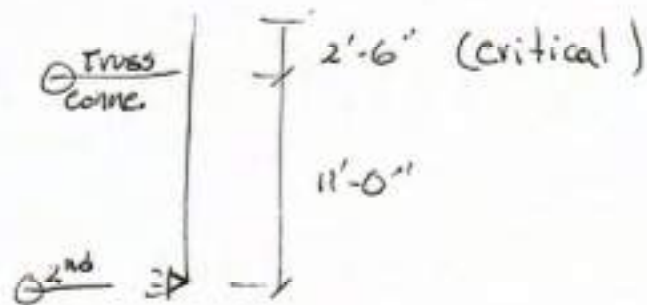


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Out of Plane CU wall Reaction @ Roof Diaphragm



Truss connection @ every truss

$$103.36 \text{ psf} \times 2'-6" \times 2'-0" = 517 \text{ \#}$$

$$57.64 \text{ psf} \times \frac{11'-0"}{2} \times 2'-0" = 634 \text{ \#}$$

$$\underline{1151 \text{ \#}}$$

Note that, windward parapet pressure is higher; but, it does not create tension in the connection.

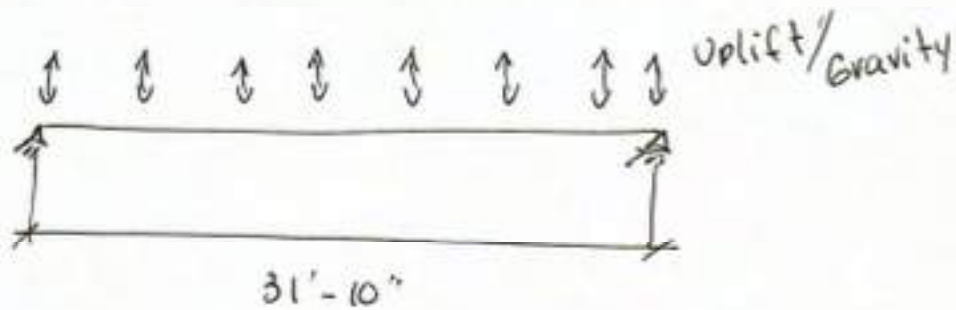
zone 5 + zone 5

$$44.32 \text{ psf} - 59.04 \text{ psf} \Rightarrow -44.32 - 59.04 = -103.36 \text{ psf}$$

where the negative sign means pulling away from the structure.

Roof Truss Loads

Area over GL A to B ; truss span from A to B



See MWFS Spread Sheet Uplift = 47.44 psf

$$47.44 \text{ psf} \times \frac{31'-10''}{2} \times 2 \text{ ft} = 1511^{\#} \text{ Uplift.}$$

$$10 \text{ psf} \times \frac{31'-10''}{2} \times 2 \text{ ft} = 318^{\#} \text{ Gravity}$$

$$-0.6(318^{\#}) + 1511^{\#} = 1320^{\#} \text{ Uplift. Net.}$$

Gravity

$$(20 \text{ psf} + 30 \text{ psf}) \times \frac{31'-10''}{2} \times 2 \text{ ft} = 1555^{\#} \text{ Gravity.}$$

Area over GL 5 to 6 ; truss span from 5 to 6

$$47.44 \text{ psf} \times \frac{26'-7''}{2} \times 2 \text{ ft} = 1262^{\#} \text{ Uplift}$$

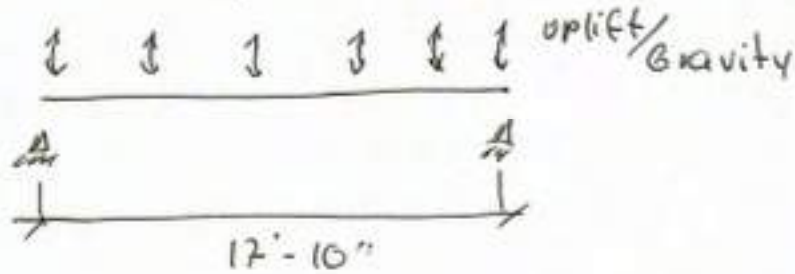
$$10 \text{ psf} \times \frac{26'-7''}{2} \times 2 \text{ ft} = 266^{\#} \text{ Gravity}$$

$$-0.6(266^{\#}) + 1262^{\#} = 1102^{\#} \text{ Net Uplift}$$

Gravity

$$50 \text{ psf} \times \frac{26'-7''}{2} \times 2 \text{ ft} = 1330^{\#} \text{ Gravity}$$

Area Over GL D to E; trusses span from D to E



$$\begin{aligned}
 47.44 \text{ psf} \times \frac{17'-10''}{2} \times 2 \text{ ft} &= 847^{\#} \text{ Uplift} \\
 10 \text{ psf} \times \frac{17'-10''}{2} \times 2 \text{ ft} &= 184^{\#} \text{ Gravity} \\
 -0.6(184^{\#}) + 847^{\#} &= 737^{\#} \text{ Net Uplift.}
 \end{aligned}$$

Gravity

$$50 \text{ psf} \times \frac{17'-10''}{2} \times 2 \text{ ft} = 892^{\#} \text{ Gravity}$$



Girder Truss

$$\begin{aligned}
 47.44 \text{ psf} \times \left(\frac{17'-10''}{2} + \frac{2'-0''}{2} \right) \times \frac{22'-8''}{2} &= 5329^{\#} \\
 10 \text{ psf} \times \left(\text{''} \quad \text{''} \right) \times \text{''} &= 1123^{\#} \\
 5329^{\#} - 0.6(1123^{\#}) &= 4655^{\#} \text{ Net Uplift}
 \end{aligned}$$

Gravity

$$50 \text{ psf} \times \left(\text{''} + \text{''} \right) \times \text{''} = 5617^{\#} \text{ Gravity.}$$

Masonry Slender Wall

File: New Residence.ec6
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Lic. #: KW-06012775

CBenginfo

DESCRIPTION: Ground Floor Critical

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

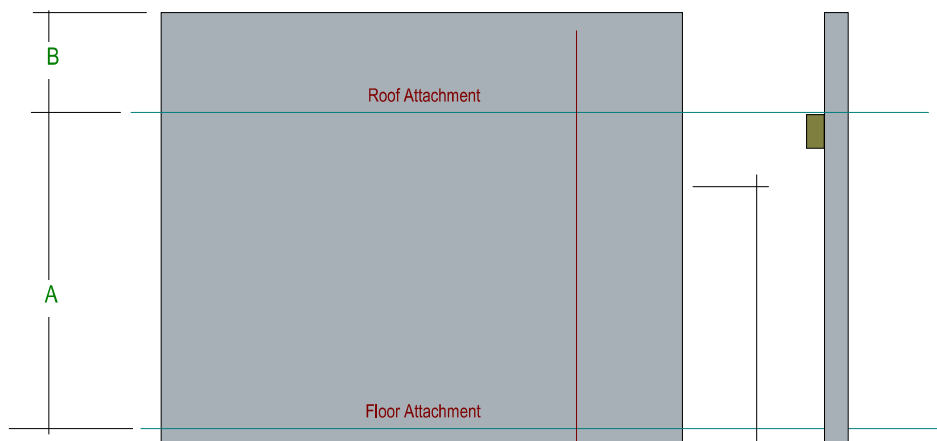
F'm	=	2.0	ksi	Nom. Wall Thickness	8	in	Temp Diff across thickness	=		deg F
Fy - Yield	=	60.0	ksi	Actual Thickness	7.625	in	Min Allow Out-of-plane Defl Ratio	=	0.0	
Fr - Rupture	=	96.30	psi	Rebar "d" distance	3.8125	in	Minimum Vertical Steel %	=	0.0020	
Em = f'm *	=	900.0		Lower Level Rebar . . .						
Max % of ρ bal.	=	0.01189		Bar Size	#	6				
Grout Density	=	140	pcf	Bar Spacing		24	in			
Block Weight		Normal Weight								
Wall Weight	=	61.0	psf							

Wall is grouted at rebar cells only

One-Story Wall Dimensions

A Clear Height	=	14.50	ft
B Parapet height	=		ft

Wall Support Condition Top & Bottom Pinned



Lateral Loads

Wind Loads :

Full area WIND load 96.3 psf

Seismic Loads :

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

Fp 1.0 = 0.0 psf

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CBenginfo

DESCRIPTION: Ground Floor Critical

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+1.60H	Maximum Bending Stress Ratio = 0.7936			
		Max Mu	2.566 k-ft	Phi * Mn	3.234 k-ft
PASS	Service Deflection Check +D+0.420W+H	Actual Defl. Ratio L/	684	Allowable Defl. Ratio	360.0
		Max. Deflection	0.2544 in		
PASS	Axial Load Check +1.20D+0.50Lr+L+W+1.60H	Max Pu / Ag	9.071 psi	Max. Allow. Defl.	0.4833 in
		Location	7.008 ft	0.2 * f'm	400.0 psi
PASS	Reinforcing Limit Check				
		Actual As/bd	0.004809	Max Allow As/bd	0.01189
Maximum Reactions . . . for Load Combination...					
	Top Horizontal	W Only			0.6982 k
	Base Horizontal	W Only			0.6982 k
	Vertical Reaction	+D+H			0.8845 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load		Mcr k-ft	Mu k-ft	Moment Values			As Ratio	0.6 * rho bal
	Pu k	0.2*f'm*b*t k			Phi	Phi Mn k-ft	As in^2		
	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
	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+1.60Lr+0.50W+1.60H at 6.77 to 7.25	0.566	24.960	0.74	1.28	0.90	3.27	0.220	0.0048	0.0117
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
+1.20D+1.60S+0.50W+1.60H at 6.77 to 7.25	0.566	24.960	0.74	1.28	0.90	3.27	0.220	0.0048	0.0117
+1.20D+0.50Lr+L+W+1.60H at 6.77 to 7.25	0.566	24.960	0.74	2.58	0.90	3.27	0.220	0.0048	0.0117
+1.20D+L+0.50S+W+1.60H at 6.77 to 7.25	0.566	24.960	0.74	2.58	0.90	3.27	0.220	0.0048	0.0117
+0.90D+W+1.60H at 6.77 to 7.25	0.425	24.960	0.74	2.57	0.90	3.24	0.220	0.0048	0.0117
	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

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values		I gross in^4	Stiffness		Deflections	
		Mcr k-ft	Mactual k-ft		I cracked in^4	I effective in^4	Deflection in	Defl. Ratio
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.420W+H at 7.25 to 7.73	0.442	0.74	1.07	353.60	31.87	35.772	0.254	683.9
+D+0.750Lr+0.750L+0.420W+H at 7.25 to 7.7	0.442	0.74	1.07	353.60	31.87	35.772	0.254	683.9
+D+0.750L+0.750S+0.420W+H at 7.25 to 7.73	0.442	0.74	1.07	353.60	31.87	35.772	0.254	683.9
+0.60D+0.420W+0.60H at 7.25 to 7.73	0.265	0.74	1.07	353.60	31.55	35.485	0.254	685.5
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0

Masonry Slender Wall

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CBenginfo

DESCRIPTION: Ground Floor Critical

0.000	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.0
0.000	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.0
0.000	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.0
0.000	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.0

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
+D+H	0.0 k	0.00 k	0.885 k
+D+L+H	0.0 k	0.00 k	0.885 k
+D+Lr+H	0.0 k	0.00 k	0.885 k
+D+S+H	0.0 k	0.00 k	0.885 k
+D+0.750Lr+0.750L+H	0.0 k	0.00 k	0.885 k
+D+0.750L+0.750S+H	0.0 k	0.00 k	0.885 k
+D+0.60W+H	0.4 k	0.42 k	0.884 k
+D+0.750Lr+0.750L+0.450W+H	0.3 k	0.32 k	0.884 k

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CBenginfo

DESCRIPTION: Ground Floor Critical

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
+D+0.750L+0.750S+0.450W+H	0.3 k	0.32 k	0.884 k
+0.60D+0.60W+0.60H	0.4 k	0.42 k	0.531 k
+D+0.70E+0.60H	0.0 k	0.00 k	0.885 k
+D+0.750L+0.750S+0.5250E+H	0.0 k	0.00 k	0.885 k
+0.60D+0.70E+H	0.0 k	0.00 k	0.531 k
D Only	0.0 k	0.00 k	0.885 k
Lr Only	0.0 k	0.00 k	0.000 k
L Only	0.0 k	0.00 k	0.000 k
S Only	0.0 k	0.00 k	0.000 k
W Only	0.7 k	0.70 k	0.000 k
E Only	0.0 k	0.00 k	0.000 k
H Only	0.0 k	0.00 k	0.000 k

Masonry Slender Wall

Lic. #: KW-06012775

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CBenginfo

DESCRIPTION: 2ND Floor Critical

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

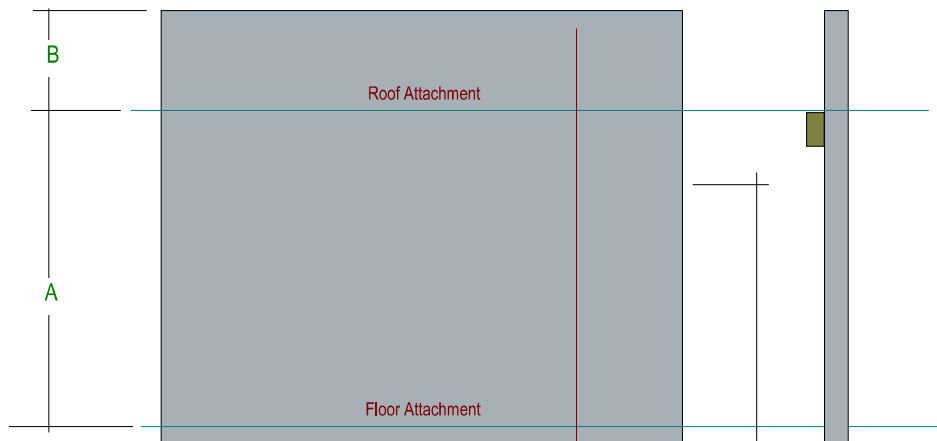
F'm	=	2.0	ksi	Nom. Wall Thickness	8	in	Temp Diff across thickness	=		deg F
Fy - Yield	=	60.0	ksi	Actual Thickness	7.625	in	Min Allow Out-of-plane Defl Ratio	=	0.0	
Fr - Rupture	=	96.30	psi	Rebar "d" distance	3.8125	in	Minimum Vertical Steel %	=	0.0020	
Em = f'm *	=	900.0		Lower Level Rebar . . .						
Max % of $\rho_{bal.}$	=	0.01189		Bar Size	#	5				
Grout Density	=	140	pcf	Bar Spacing		48	in			
Block Weight		Normal Weight								
Wall Weight	=	55.0	psf							

Wall is grouted at rebar cells only

One-Story Wall Dimensions

A	Clear Height	=	9	ft
B	Parapet height	=		ft

Wall Support Condition Top & Bottom Pinned



Lateral Loads

Wind Loads :

Full area WIND load 96.30 psf

Seismic Loads :

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

Fp 1.0 = 0.0 psf

Masonry Slender Wall

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DESCRIPTION: 2ND Floor Critical

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+1.60H	Maximum Bending Stress Ratio = 0.7967			
		Max Mu	0.9756 k-ft	Phi * Mn	1.225 k-ft
PASS	Service Deflection Check +D+0.420W+H	Actual Defl. Ratio L/	10799	Allowable Defl. Ratio	360.0
		Max. Deflection	0.010 in		
PASS	Axial Load Check +1.20D+0.50Lr+L+W+1.60H	Max Pu / Ag	5.739 psi	Max. Allow. Defl.	0.30 in
		Location	4.350 ft	0.2 * f'm	400.0 psi
PASS	Reinforcing Limit Check	Actual As/bd	0.001694	Max Allow As/bd	0.01189
Maximum Reactions . . . for Load Combination...					
	Top Horizontal	W Only			0.4334 k
	Base Horizontal	W Only			0.4334 k
	Vertical Reaction	+D+H			0.4950 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load		Mcr k-ft	Mu k-ft	Moment Values			As Ratio	0.6 * rho bal
	Pu k	0.2*f'm*b*t k			Phi	Phi Mn k-ft	As in^2		
	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
	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+1.60Lr+0.50W+1.60H at 4.20 to 4.50	0.317	22.080	0.70	0.49	0.90	1.25	0.078	0.0017	0.0118
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000
+1.20D+1.60S+0.50W+1.60H at 4.20 to 4.50	0.317	22.080	0.70	0.49	0.90	1.25	0.078	0.0017	0.0118
+1.20D+0.50Lr+L+W+1.60H at 4.20 to 4.50	0.317	22.080	0.70	0.98	0.90	1.25	0.078	0.0017	0.0118
+1.20D+L+0.50S+W+1.60H at 4.20 to 4.50	0.317	22.080	0.70	0.98	0.90	1.25	0.078	0.0017	0.0118
+0.90D+W+1.60H at 4.20 to 4.50	0.238	22.080	0.70	0.98	0.90	1.23	0.078	0.0017	0.0118
	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

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values		I gross in^4	Stiffness		Deflections	
		Mcr k-ft	Mactual k-ft		I cracked in^4	I effective in^4	Deflection in	Defl. Ratio
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.420W+H at 4.20 to 4.50	0.264	0.70	0.41	331.10	14.02	331.100	0.010	10,798.9
+D+0.750Lr+0.750L+0.420W+H at 4.20 to 4.5	0.264	0.70	0.41	331.10	14.02	331.100	0.010	10,798.9
+D+0.750L+0.750S+0.420W+H at 4.20 to 4.50	0.264	0.70	0.41	331.10	14.02	331.100	0.010	10,798.9
+0.60D+0.420W+0.60H at 4.20 to 4.50	0.158	0.70	0.41	331.10	13.77	331.100	0.010	10,801.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0

Masonry Slender Wall

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CBenginfo

DESCRIPTION: 2ND Floor Critical

0.000	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.0
0.000	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.0
0.000	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.0
0.000	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.0

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
+D+H	0.0 k	0.00 k	0.495 k
+D+L+H	0.0 k	0.00 k	0.495 k
+D+Lr+H	0.0 k	0.00 k	0.495 k
+D+S+H	0.0 k	0.00 k	0.495 k
+D+0.750Lr+0.750L+H	0.0 k	0.00 k	0.495 k
+D+0.750L+0.750S+H	0.0 k	0.00 k	0.495 k
+D+0.60W+H	0.3 k	0.26 k	0.495 k
+D+0.750Lr+0.750L+0.450W+H	0.2 k	0.20 k	0.495 k

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DESCRIPTION: 2ND Floor Critical

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
+D+0.750L+0.750S+0.450W+H	0.2 k	0.20 k	0.495 k
+0.60D+0.60W+0.60H	0.3 k	0.26 k	0.297 k
+D+0.70E+0.60H	0.0 k	0.00 k	0.495 k
+D+0.750L+0.750S+0.5250E+H	0.0 k	0.00 k	0.495 k
+0.60D+0.70E+H	0.0 k	0.00 k	0.297 k
D Only	0.0 k	0.00 k	0.495 k
Lr Only	0.0 k	0.00 k	0.000 k
L Only	0.0 k	0.00 k	0.000 k
S Only	0.0 k	0.00 k	0.000 k
W Only	0.4 k	0.43 k	0.000 k
E Only	0.0 k	0.00 k	0.000 k
H Only	0.0 k	0.00 k	0.000 k

Fill Cell vs. tie Column

Max. Opening size per jamb condition

$$1 \text{ Cell Capacity} = 4.187 \text{ K-ft}$$

$$4.187 \text{ K-ft} \geq (0.09839 + 0.0492 S_o) \frac{13.83^2}{8}$$

$$\frac{4.187 \times \frac{8}{13.83^2} - 0.09839}{0.0492} \geq S_o$$

$$1.5 \text{ ft} \geq S_o$$

1'-6"

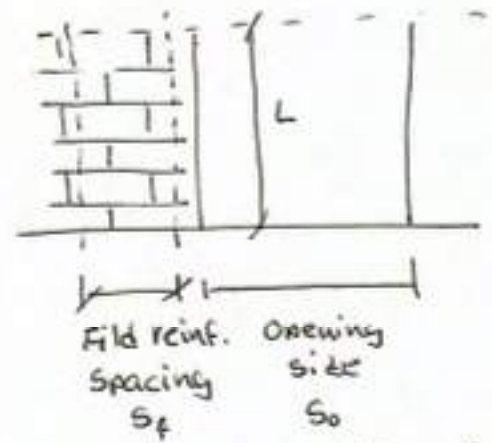
$$2 \text{ Cell Capacity} = 8.374 \text{ K-ft}$$

$$5.12 \text{ ft } 5'-1"$$

$$3 \text{ Cell Capacity} = 12.561 \text{ K-ft}$$

$$8.78 \text{ ft}$$

$$8'-8"$$



$$\text{wind Pressure} \times \left(\frac{S_o}{2} + \frac{S_o}{2} \right)$$

wind load on (plf)

$$\text{wind load} \times \frac{L^2}{8} = \text{Moment (Ultimate)}$$

Ground

$$L = 14'-6"$$

$$S_f = 24"$$

Zone 5. (Ultimate)

$$0.09839 \text{ Ksf}$$

$$0.09839 \text{ psf} \times \left(\frac{2 \text{ ft}}{2} + \frac{S_o}{2} \right)$$

$$(0.09839 + 0.0492 S_o) \text{ plf}$$

$$(98.39 + 49.2 S_o) \times \frac{13.83^2}{8}$$

Concrete Beam

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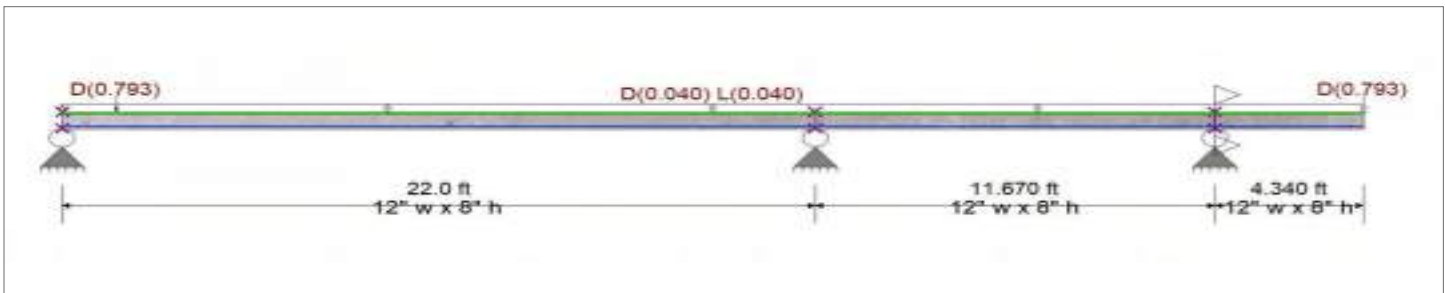
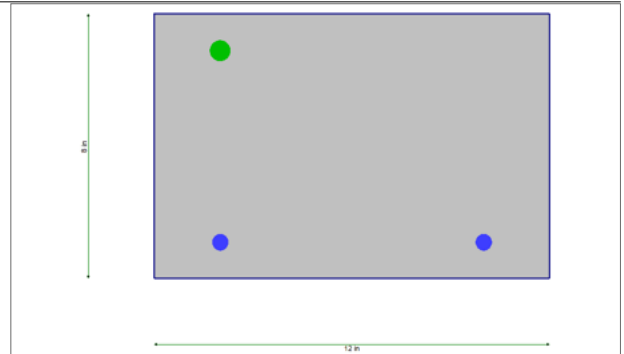
DESCRIPTION: Slab Strip Between GL A and B

CODE REFERENCES

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

Material Properties

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2}$	=	7.50		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,644.15 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 = 12.0 in, Height = 8.0 in

Span #1 Reinforcing....

2-#4 at 1.10 in from Bottom, from 0.0 to 22.0 ft in this span

1-#5 at 1.10 in from Top, from 0.0 to 22.0 ft in this span

Span #2 Reinforcing....

1-#5 at 1.10 in from Bottom, from 0.0 to 11.670 ft in this span

1-#6 at 1.10 in from Top, from 0.0 to 11.670 ft in this span

Span #3 Reinforcing....

2-#4 at 1.10 in from Bottom, from 0.0 to 4.340 ft in this span

1-#5 at 1.10 in from Top, from 0.0 to 4.340 ft in this span

Beam self weight calculated and added to loads

Loads on all spans...

D = 0.040, L = 0.040

Uniform Load on ALL spans : D = 0.040, L = 0.040 k/ft

Load for Span Number 1

Point Load : D = 0.7930 k @ 1.60 ft, (wALL)

Load for Span Number 3

Point Load : D = 0.7930 k @ 4.340 ft, (Wall)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.960 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.065 in	Ratio = 4091 >=360.
Mu : Applied		-9.325 k-ft	Max Upward Transient Deflection	-0.008 in	Ratio = 16908 >=360.
Mn * Phi : Allowable		9.717 k-ft	Max Downward Total Deflection	0.690 in	Ratio = 382 >=360.
Location of maximum on span		21.853 ft	Max Upward Total Deflection	-0.079 in	Ratio = 1767 >=360.
Span # where maximum occurs		Span # 1			

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	Phi*Mn (k-ft)		Moment of Inertia (in^4)		
		Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	2-#4 @ d=6.9", 1-#5 @ d=1.1",	12.10	9.72	512.00	196.34	85.85
Section 2	1-#5 @ d=6.9", 1-#6 @ d=1.1",	9.73	13.18	512.00	85.85	115.11

Concrete Beam

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DESCRIPTION: Slab Strip Between GL A and B

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	Phi*Mn (k-ft)		Moment of Inertia (in^4)		
		Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 3	2- #4 @ d=6.9", 1- #5 @ d=1.1",	12.10	9.72	512.00	106.34	85.85

Vertical Reactions

Support notation : Far left is #1

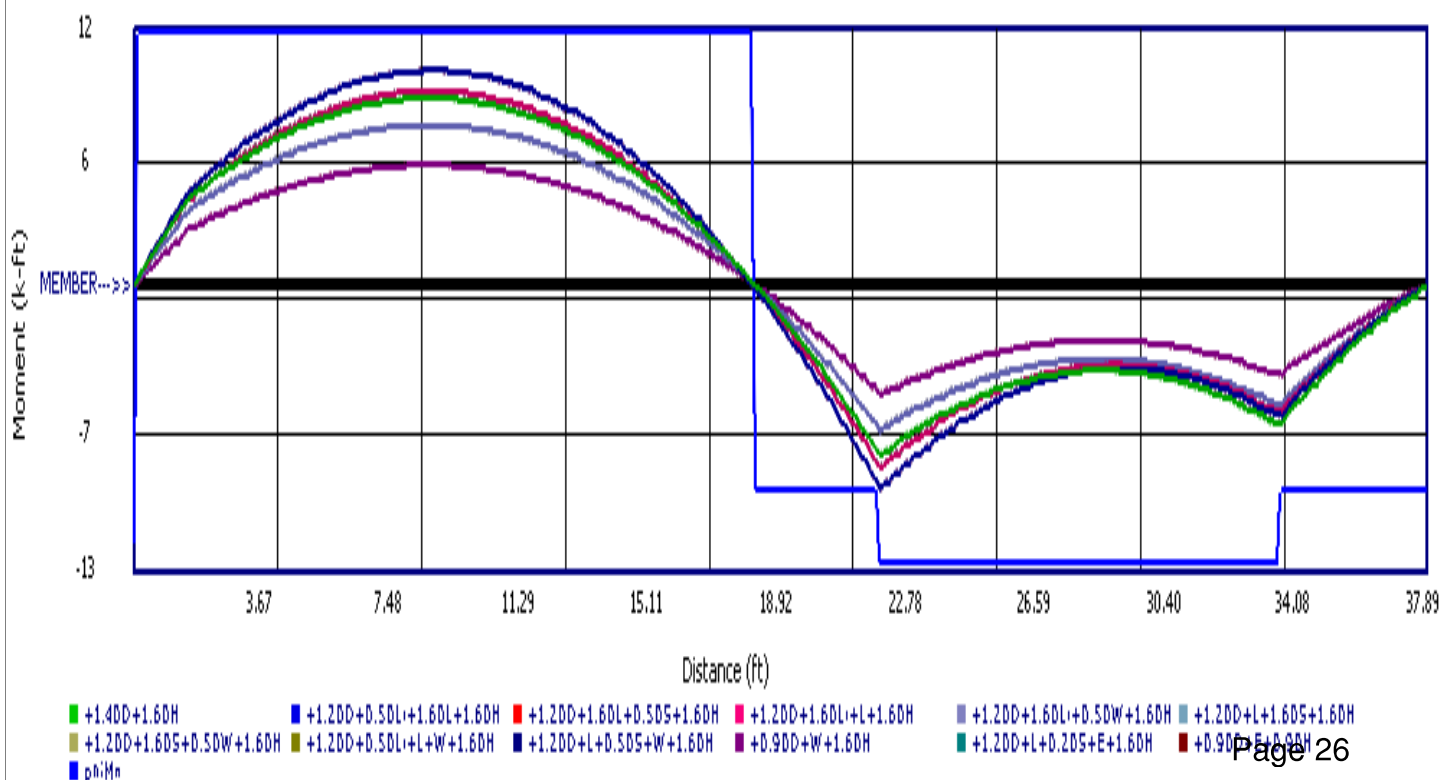
Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	2.261	3.800	2.240	
Overall MINimum	0.360	0.871	0.289	
+D+H	1.932	2.837	2.011	
+D+L+H	2.261	3.800	2.240	
+D+Lr+H	1.932	2.837	2.011	
+D+S+H	1.932	2.837	2.011	
+D+0.750Lr+0.750L+H	2.176	3.568	2.177	
+D+0.750L+0.750S+H	2.176	3.568	2.177	
+D+0.60W+H	1.932	2.837	2.011	
+D+0.750Lr+0.750L+0.450W+H	2.176	3.568	2.177	
+D+0.750L+0.750S+0.450W+H	2.176	3.568	2.177	
+0.60D+0.60W+0.60H	1.185	1.629	1.255	
+D+0.70E+0.60H	1.932	2.837	2.011	
+D+0.750L+0.750S+0.5250E+H	2.176	3.568	2.177	
+0.60D+0.70E+H	1.185	1.629	1.255	
D Only	1.932	2.837	2.011	
L Only	0.360	0.871	0.289	
H Only				

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Req'd 11.4.6.1, use #3 stirrups spaced at 0.000 in

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L+H	1	0.6900	9.240	+D+L+H	-0.0093	22.233
+D+L+H	2	0.0022	11.757	+D+L+H	-0.0792	5.368
+D+L+H	3	0.1359	4.340		0.0000	5.368

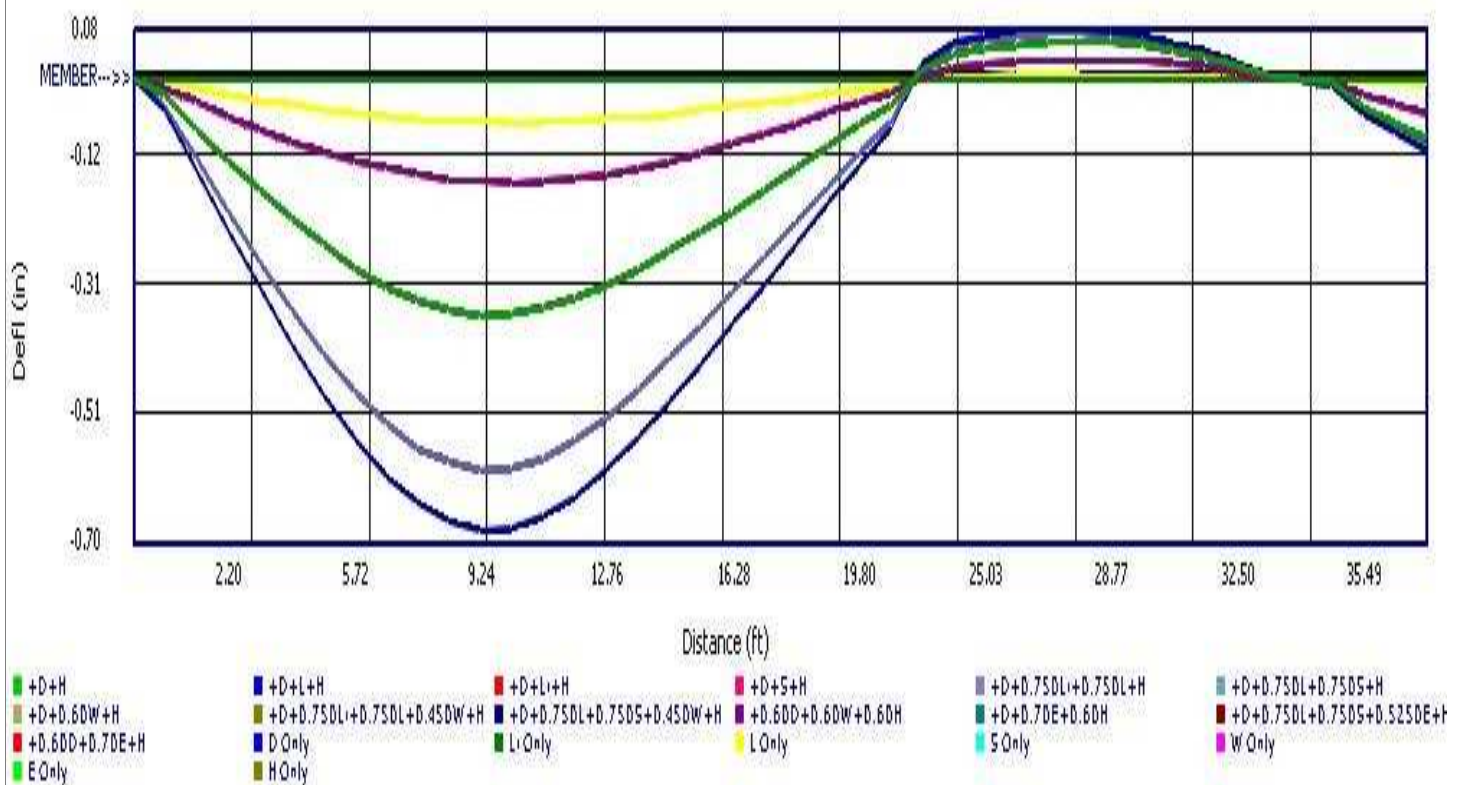
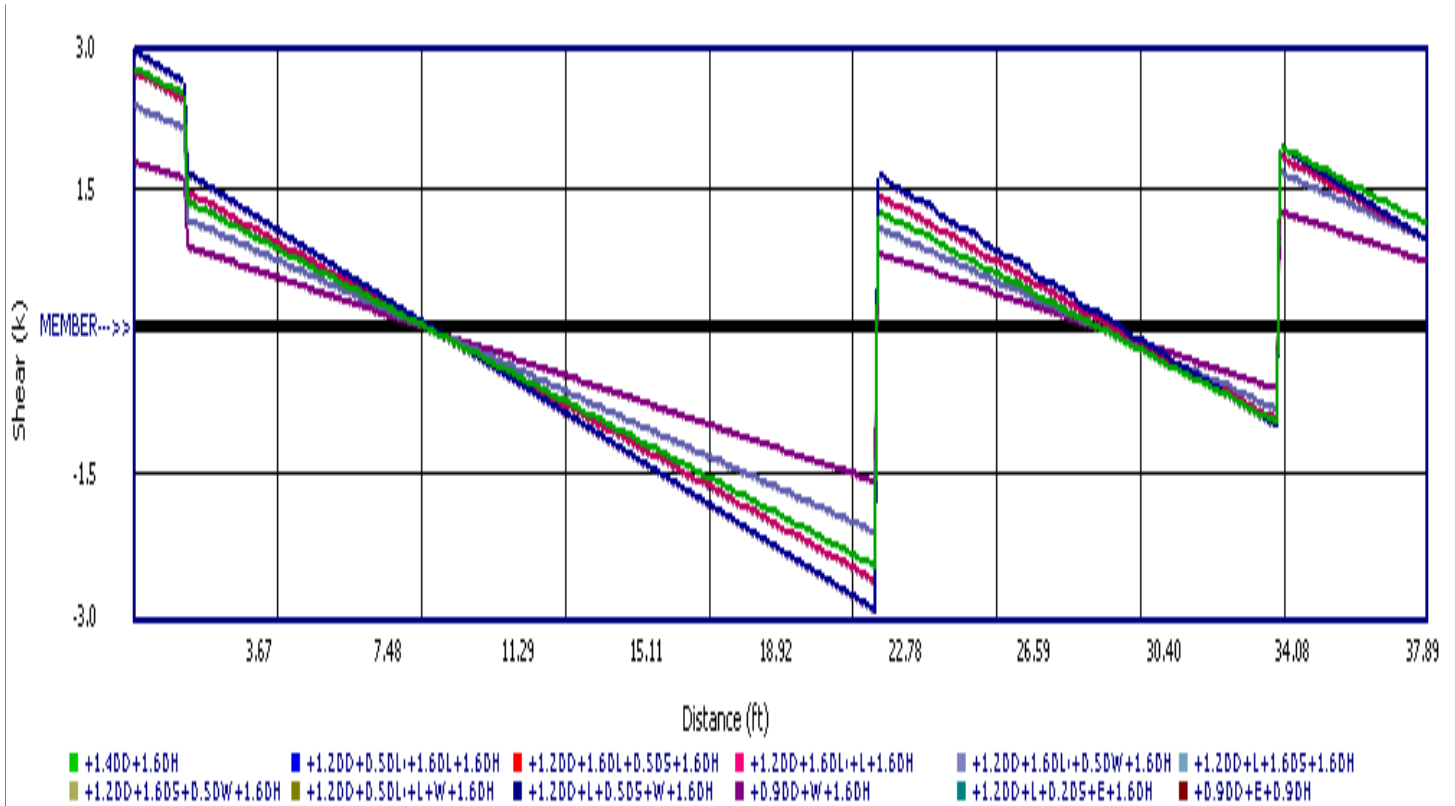


Concrete Beam

File: New Residence.ec6
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CBenginfo

Lic. #: KW-06012775

DESCRIPTION: Slab Strip Between GL A and B



Concrete Beam

Lic. #: KW-06012775

File: New Residence.ec6
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CBenginfo

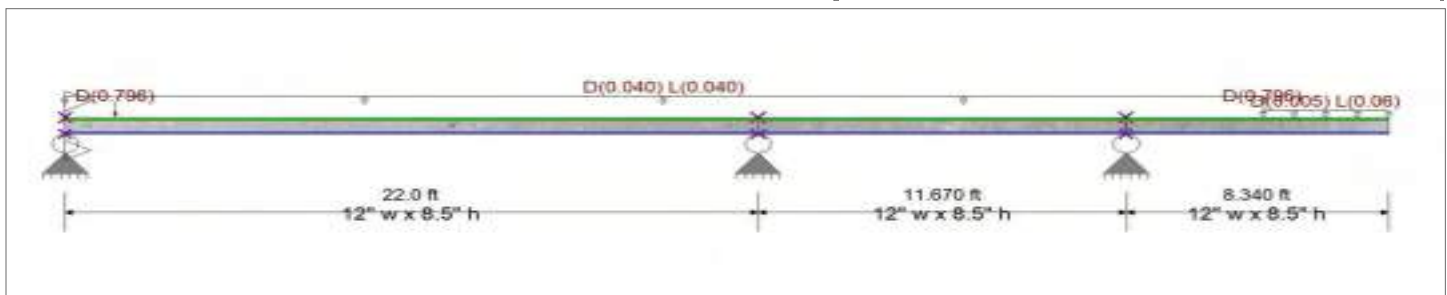
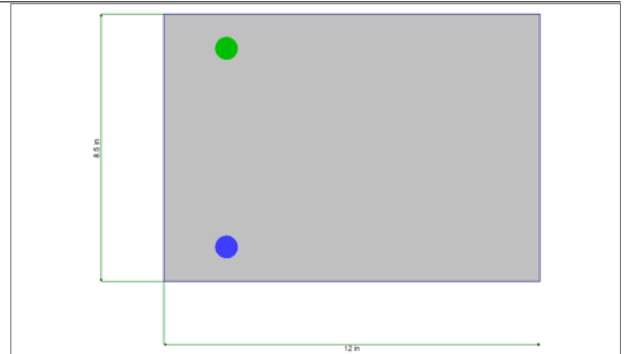
DESCRIPTION: Slab Strip Between GL A and B Throu Balcony

CODE REFERENCES

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

Material Properties

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} * 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,644.15 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 = 12.0 in, Height = 8.50 in

Span #1 Reinforcing....

1-#6 at 1.10 in from Bottom, from 0.0 to 22.0 ft in this span

1-#6 at 1.10 in from Top, from 0.0 to 22.0 ft in this span

Span #2 Reinforcing....

1-#6 at 1.10 in from Bottom, from 0.0 to 11.670 ft in this span

1-#6 at 1.10 in from Top, from 0.0 to 11.670 ft in this span

Span #3 Reinforcing....

1-#6 at 1.10 in from Bottom, from 0.0 to 8.340 ft in this span

1-#6 at 1.10 in from Top, from 0.0 to 8.340 ft in this span

Beam self weight calculated and added to loads

Loads on all spans...

Partial Length Uniform Load : D = 0.040, L = 0.040 k/ft, Extent = 0.0 -->> 38.010 ft

Load for Span Number 1

Point Load : D = 0.7960 k @ 1.60 ft, (WALL)

Load for Span Number 3

Point Load : D = 0.7960 k @ 4.340 ft, (Wall)

Uniform Load : D = 0.0050, L = 0.060 k/ft, Extent = 4.340 -->> 8.340 ft, Tributary Width = 1.0 ft, (Balcony Design)

DESIGN SUMMARY

				Design OK	
Maximum Bending Stress Ratio	=	0.851	: 1	Maximum Deflection	
Section used for this span		Typical Section		Max Downward Transient Deflection	0.090 in Ratio = 2230 >= 360.
μ_u : Applied		-12.073 k-ft		Max Upward Transient Deflection	-0.021 in Ratio = 6774 >= 360.
$M_n * \Phi$: Allowable		14.190 k-ft		Max Downward Total Deflection	0.733 in Ratio = 272 >= 240.
Location of maximum on span		0.000 ft		Max Upward Total Deflection	-0.124 in Ratio = 1133 >= 240.
Span # where maximum occurs		Span # 3			

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

		$\Phi * M_n$ (k-ft)		Moment of Inertia (in ⁴)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	I cr - Bottom	I cr - Top
Section 1	1- #6 @ d=7.4", 1- #6 @ d=1.1",	14.19	14.19	614.13	134.52	134.52

Concrete Beam

File: New Residence.ec6
Software copyright ENERCALC, INC. 1983-2020, Build:12.20.8.24

Lic. # : KW-06012775

CBenginfo

DESCRIPTION: Slab Strip Between GL A and B Throu Balcony

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	Phi*Mn (k-ft)		Moment of Inertia (in^4)		
		Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 2	1- #6 @ d=7.4", 1- #6 @ d=1.1",	14.19	14.19	614.13	134.52	134.52
Section 3	1- #6 @ d=7.4", 1- #6 @ d=1.1",	14.19	14.19	614.13	134.52	134.52

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	2.334	3.717	3.612	
Overall MINimum	0.004	0.059	0.078	
+D+H	2.022	2.641	2.784	
+D+L+H, LL Comb Run (**L)	2.041	2.424	3.396	
+D+L+H, LL Comb Run (*L*)	2.015	2.894	3.005	
+D+L+H, LL Comb Run (*LL)	2.031	2.684	3.612	
+D+L+H, LL Comb Run (L**)	2.323	3.482	2.522	
+D+L+H, LL Comb Run (L*L)	2.334	3.286	3.120	
+D+L+H, LL Comb Run (LL*)	2.322	3.717	2.755	
+D+L+H, LL Comb Run (LLL)	2.327	3.540	3.341	
+D+Lr+H, LL Comb Run (**L)	2.022	2.641	2.784	
+D+Lr+H, LL Comb Run (*L*)	2.022	2.641	2.784	
+D+Lr+H, LL Comb Run (*LL)	2.022	2.641	2.784	
+D+Lr+H, LL Comb Run (L**)	2.022	2.641	2.784	
+D+Lr+H, LL Comb Run (L*L)	2.022	2.641	2.784	
+D+Lr+H, LL Comb Run (LL*)	2.022	2.641	2.784	
+D+Lr+H, LL Comb Run (LLL)	2.022	2.641	2.784	
+D+S+H	2.022	2.641	2.784	
+D+0.750Lr+0.750L+H, LL Comb Run (	2.035	2.482	3.241	
+D+0.750Lr+0.750L+H, LL Comb Run (	2.017	2.831	2.950	
+D+0.750Lr+0.750L+H, LL Comb Run (	2.028	2.675	3.404	
+D+0.750Lr+0.750L+H, LL Comb Run (	2.240	3.293	2.574	
+D+0.750Lr+0.750L+H, LL Comb Run (	2.248	3.147	3.022	
+D+0.750Lr+0.750L+H, LL Comb Run (	2.240	3.467	2.750	
+D+0.750Lr+0.750L+H, LL Comb Run (	2.244	3.334	3.189	
+D+0.750L+0.750S+H, LL Comb Run (*)	2.035	2.482	3.241	
+D+0.750L+0.750S+H, LL Comb Run (*)	2.017	2.831	2.950	
+D+0.750L+0.750S+H, LL Comb Run (*)	2.028	2.675	3.404	
+D+0.750L+0.750S+H, LL Comb Run (L	2.240	3.293	2.574	
+D+0.750L+0.750S+H, LL Comb Run (L	2.248	3.147	3.022	
+D+0.750L+0.750S+H, LL Comb Run (L	2.240	3.467	2.750	
+D+0.750L+0.750S+H, LL Comb Run (L	2.244	3.334	3.189	
+D+0.60W+H	2.022	2.641	2.784	
+D+0.750Lr+0.750L+0.450W+H, LL Com	2.035	2.482	3.241	
+D+0.750Lr+0.750L+0.450W+H, LL Com	2.017	2.831	2.950	
+D+0.750Lr+0.750L+0.450W+H, LL Com	2.028	2.675	3.404	
+D+0.750Lr+0.750L+0.450W+H, LL Com	2.240	3.293	2.574	
+D+0.750Lr+0.750L+0.450W+H, LL Com	2.248	3.147	3.022	
+D+0.750Lr+0.750L+0.450W+H, LL Com	2.240	3.467	2.750	
+D+0.750Lr+0.750L+0.450W+H, LL Com	2.244	3.334	3.189	
+D+0.750L+0.750S+0.450W+H, LL Comb	2.035	2.482	3.241	
+D+0.750L+0.750S+0.450W+H, LL Comb	2.017	2.831	2.950	
+D+0.750L+0.750S+0.450W+H, LL Comb	2.028	2.675	3.404	
+D+0.750L+0.750S+0.450W+H, LL Comb	2.240	3.293	2.574	
+D+0.750L+0.750S+0.450W+H, LL Comb	2.248	3.147	3.022	
+D+0.750L+0.750S+0.450W+H, LL Comb	2.240	3.467	2.750	
+D+0.750L+0.750S+0.450W+H, LL Comb	2.244	3.334	3.189	
+0.60D+0.60W+0.60H	1.232	1.529	1.707	
+D+0.70E+0.60H	2.022	2.641	2.784	
+D+0.750L+0.750S+0.5250E+H, LL Com	2.035	2.482	3.241	
+D+0.750L+0.750S+0.5250E+H, LL Com	2.017	2.831	2.950	
+D+0.750L+0.750S+0.5250E+H, LL Com	2.028	2.675	3.404	
+D+0.750L+0.750S+0.5250E+H, LL Com	2.240	3.293	2.574	
+D+0.750L+0.750S+0.5250E+H, LL Com	2.248	3.147	3.022	

Concrete Beam

Lic. # : KW-06012775

DESCRIPTION: Slab Strip Between GL A and B Throu Balcony

Vertical Reactions

Support notation : Far left is #1

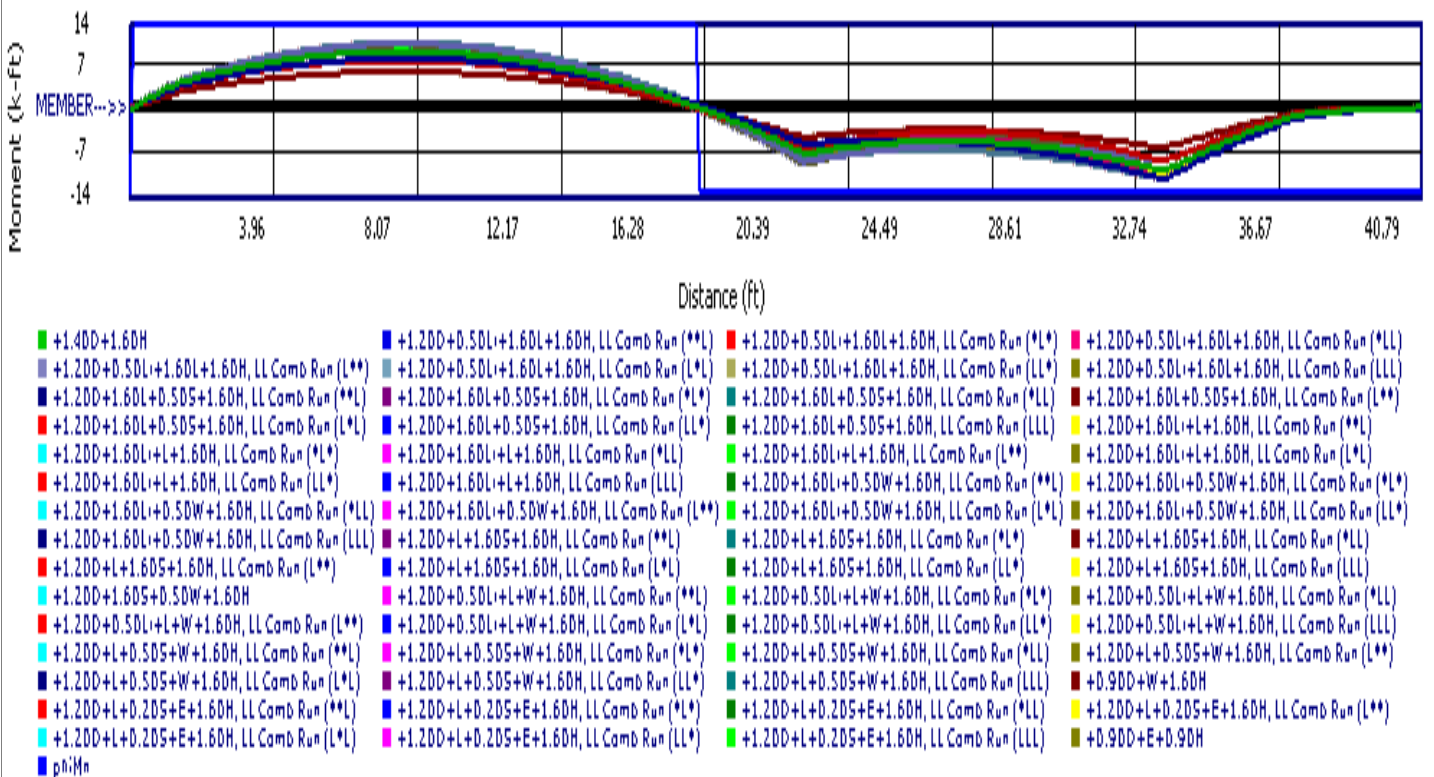
Load Combination	Support 1	Support 2	Support 3	Support 4
+D+0.750L+0.750S+0.5250E+H, LL Com	2.240	3.467	2.750	
+D+0.750L+0.750S+0.5250E+H, LL Com	2.244	3.334	3.189	
+0.60D+0.70E+H	1.232	1.529	1.707	
D Only	2.022	2.641	2.784	
L Only, LL Comb Run (**L)	0.015	-0.206	0.604	
L Only, LL Comb Run (*L*)	-0.011	0.264	0.213	
L Only, LL Comb Run (*LL)	0.004	0.059	0.818	
L Only, LL Comb Run (L**)	0.368	0.647	-0.135	
L Only, LL Comb Run (L*L)	0.383	0.442	0.469	
L Only, LL Comb Run (LL*)	0.357	0.912	0.078	
L Only, LL Comb Run (LLL)	0.372	0.706	0.682	
H Only				

Shear Stirrup Requirements

Entire Beam Span Length : $V_u < \Phi V_c/2$, Req'd Vs = Not Req'd 11.4.6.1, use #3 stirrups spaced at 0.000 in

Overall Maximum Deflections

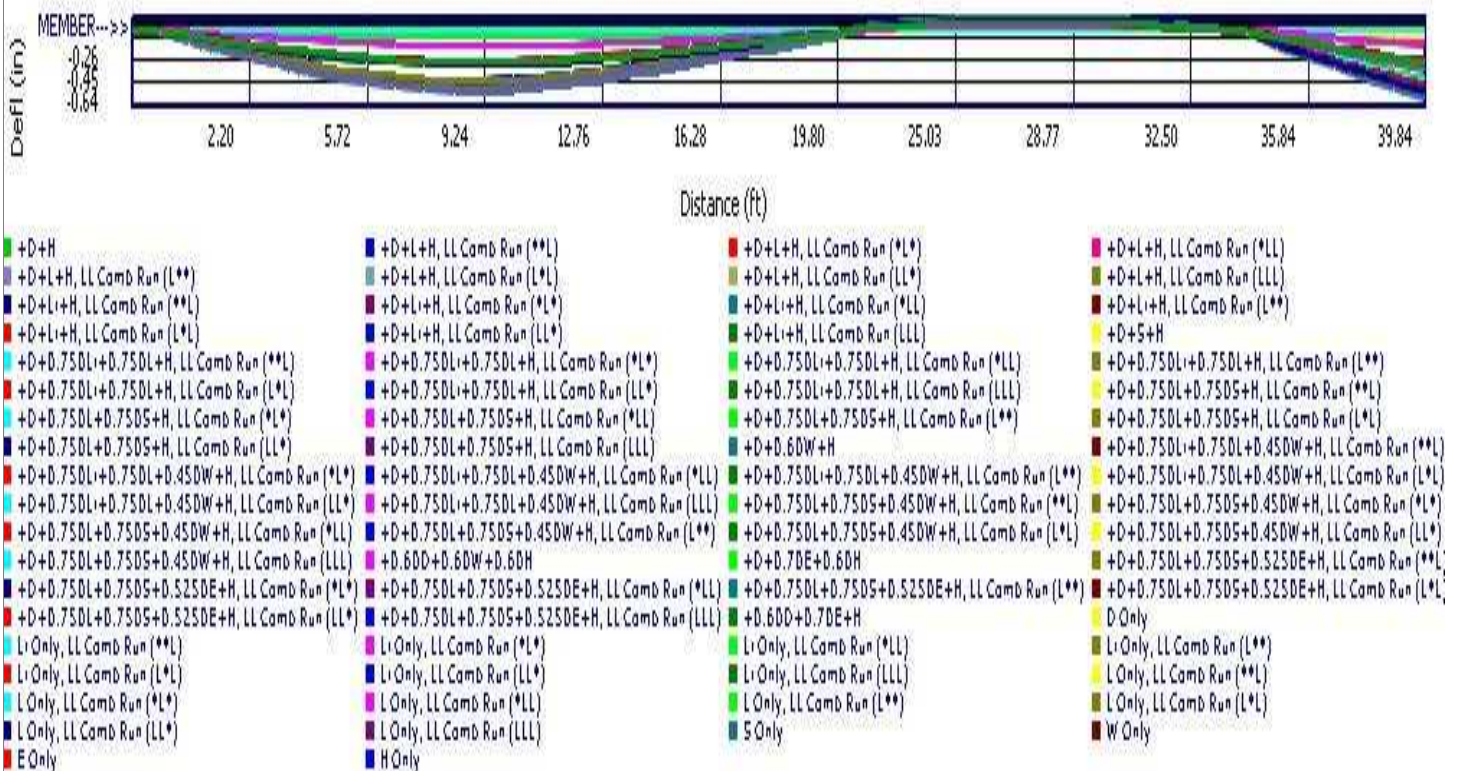
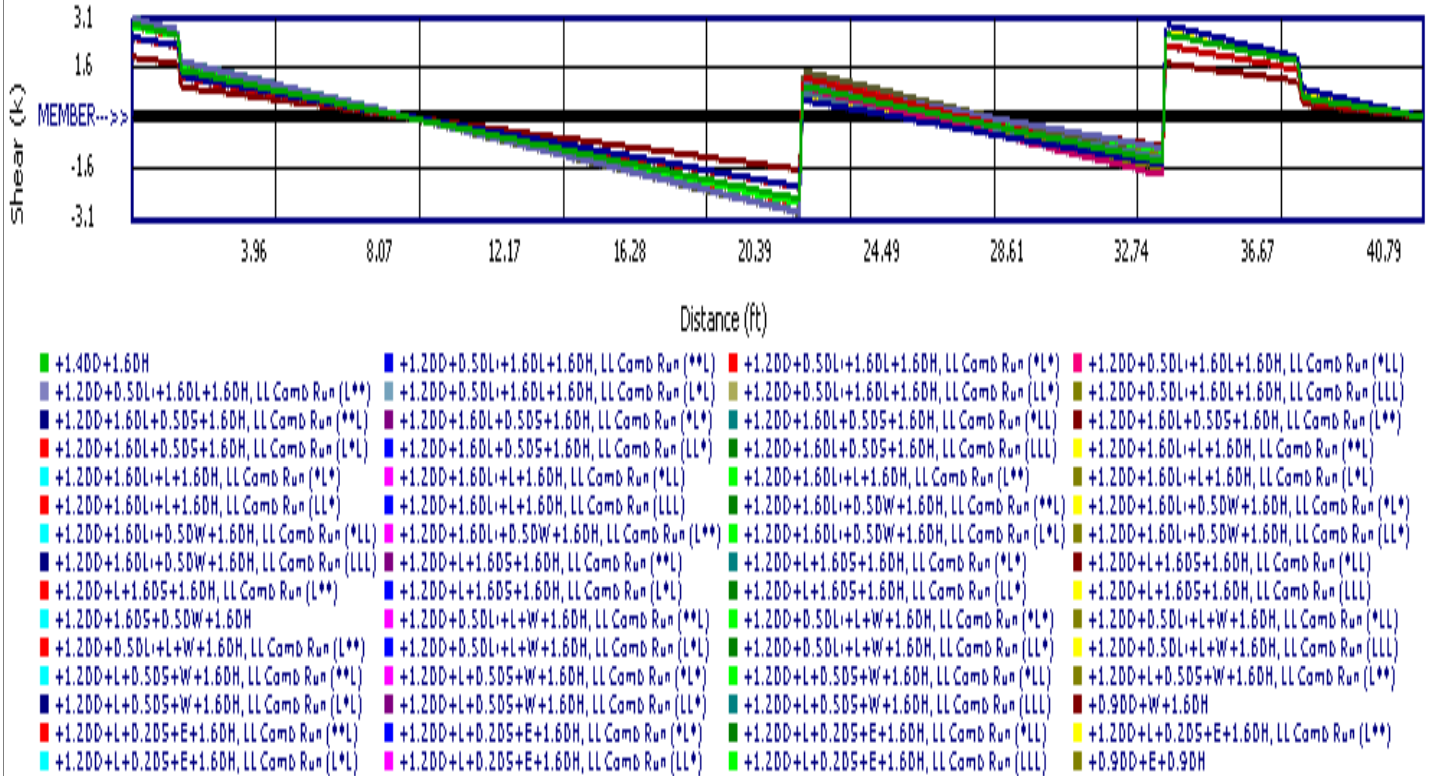
Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L+H, LL Comb Run (L*L)	1	0.5583	9.240	+D+L+H, LL Comb Run (L*L)	-0.0095	22.233
+D+L+H, LL Comb Run (L*L)	2	0.0110	11.837	+D+L+H, LL Comb Run (L*L)	-0.1236	6.769
+D+L+H, LL Comb Run (L*L)	3	0.7334	8.340		0.0000	6.769



Concrete Beam

Lic. # : KW-06012775

DESCRIPTION: Slab Strip Between GL A and B Throu Balcony



Concrete Beam

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CBenginfo

DESCRIPTION: Slab Strip Between GL 4 and 5 Left

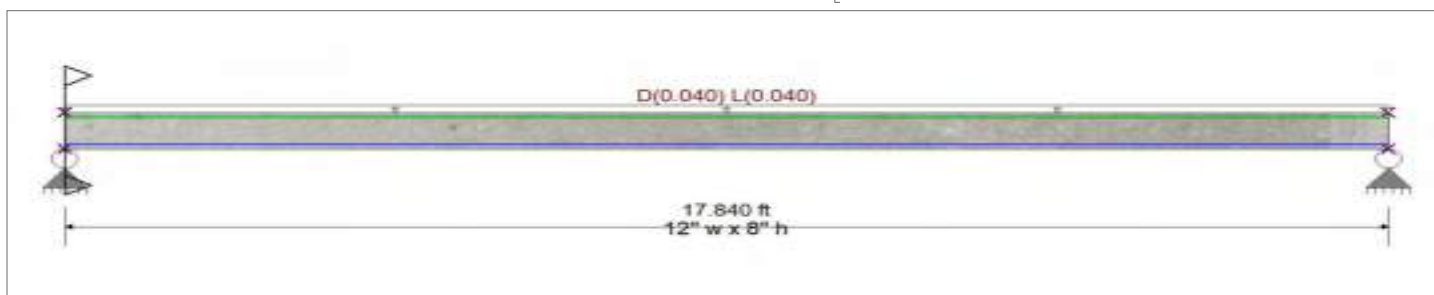
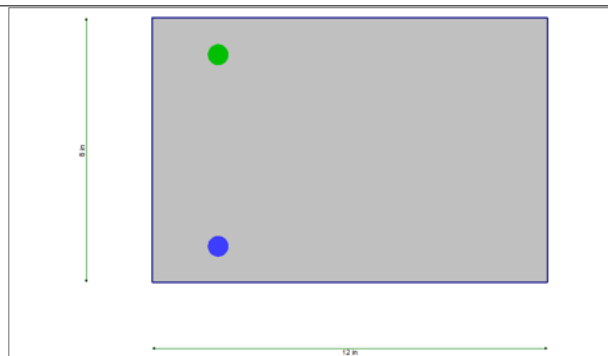
CODE REFERENCES

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

Load Combination Set : ASCE 7-16

Material Properties

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2}$	=	7.50		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,644.15 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....

1-#5 at 1.10 in from Bottom, from 0.0 to 17.840 ft in this span

1-#5 at 1.10 in from Top, from 0.0 to 17.840 ft in this span

Beam self weight calculated and added to loads

Loads on all spans...

D = 0.040, L = 0.040

Uniform Load on ALL spans : D = 0.040, L = 0.040 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.937 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.049 in	Ratio = 4382 >=360.
Mu : Applied		9.071 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 <360.0
Mn * Phi : Allowable		9.685 k-ft	Max Downward Total Deflection	0.429 in	Ratio = 498 >=360.
Location of maximum on span		8.936 ft	Max Upward Total Deflection	0.000 in	Ratio = 0 <360.0
Span # where maximum occurs		Span # 1			

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	Phi*Mn (k-ft)		Moment of Inertia (in^4)		
		Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	1- #5 @ d=6.9", 1- #5 @ d=1.1",	9.69	9.69	512.00	85.73	85.73

Support notation : Far left is #1

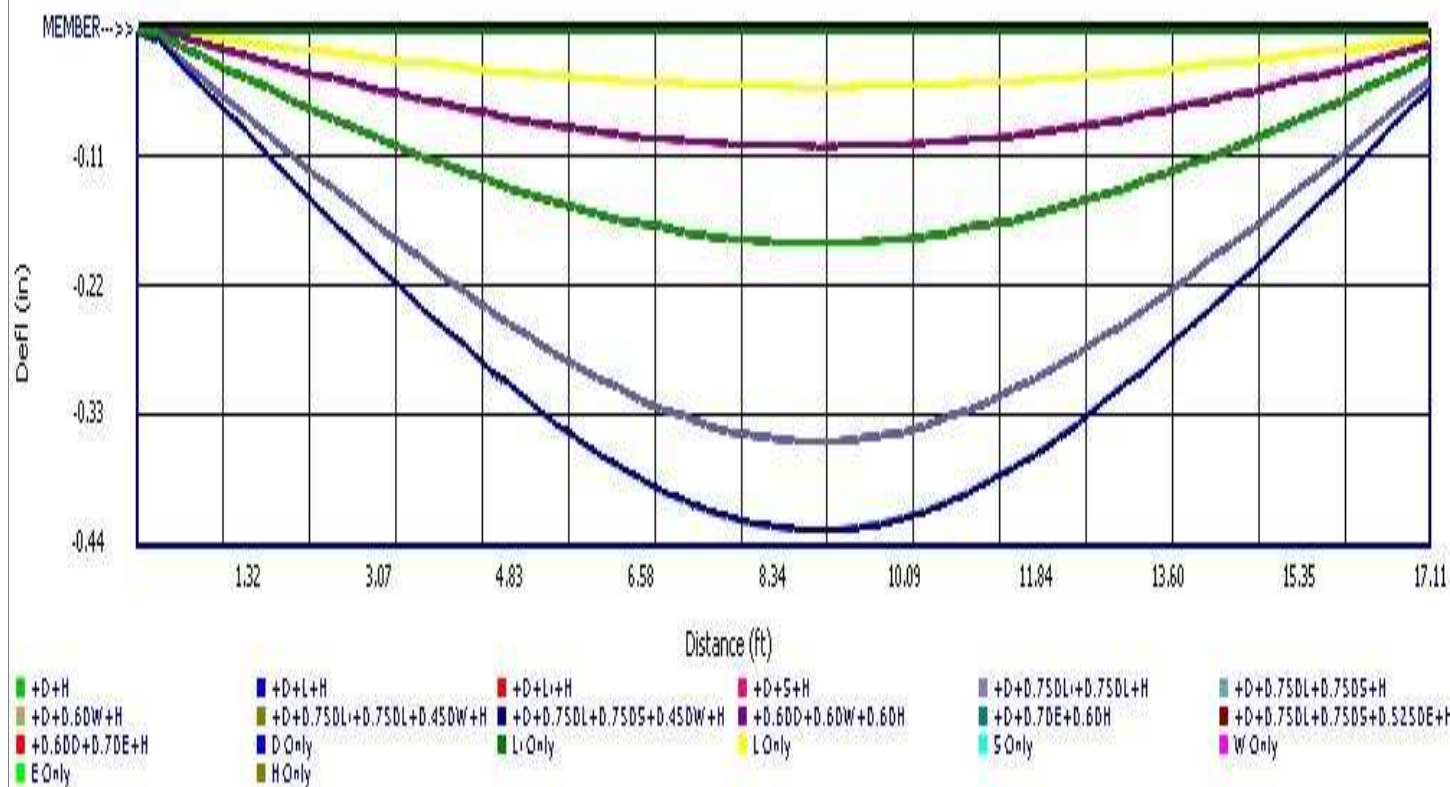
Vertical Reactions

Load Combination	Support 1	Support 2
Overall MAXimum	1.576	1.576
Overall MINimum	0.357	0.357
+D+H	1.219	1.219
+D+L+H	1.576	1.576
+D+Lr+H	1.219	1.219
+D+S+H	1.219	1.219
+D+0.750Lr+0.750L+H	1.487	1.487

Lic. # : KW-06012775

Support notation : Far left is #1

DESCRIPTION: Slab Strip Between GL 4 and 5 Left



Concrete Beam

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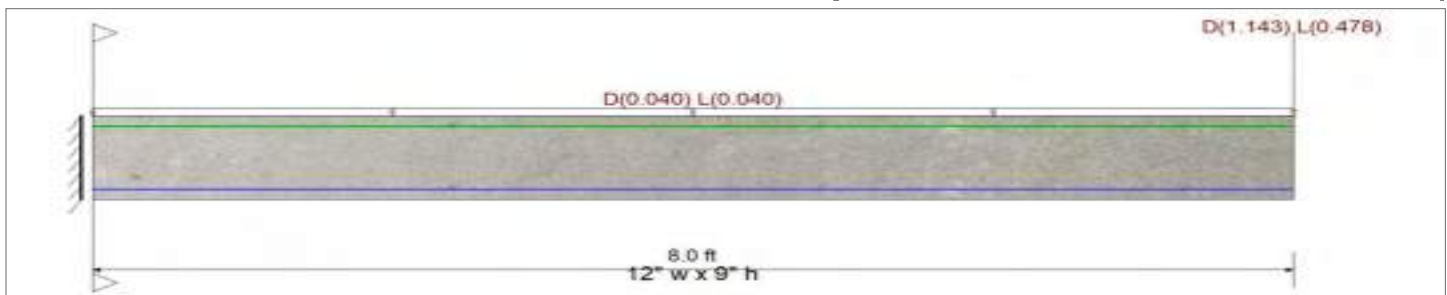
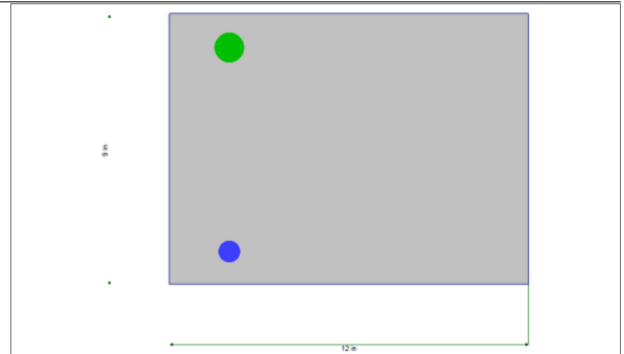
DESCRIPTION: Slab Strip Between GL 4 and 5

CODE REFERENCES

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

Material Properties

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2}$	=	7.50		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,644.15 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 = 9.0 in

Span #1 Reinforcing....

1-#6 at 1.10 in from Bottom, from 0.0 to 8.0 ft in this span

1-#8 at 1.10 in from Top, from 0.0 to 8.0 ft in this span

Beam self weight calculated and added to loads

Loads on all spans...

D = 0.040, L = 0.040

Uniform Load on ALL spans : D = 0.040, L = 0.040 k/ft

Point Load : D = 1.143, L = 0.478 k @ 8.0 ft, (Wall)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.955 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.066 in	Ratio = 2892 >=360.
Mu : Applied		-24.849 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 <360.0
Mn * Phi : Allowable		26.022 k-ft	Max Downward Total Deflection	0.598 in	Ratio = 320 >=240.
Location of maximum on span		0.000 ft	Max Upward Total Deflection	0.000 in	Ratio = 0 <240.0
Span # where maximum occurs		Span # 1			

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	Phi*Mn (k-ft)		Moment of Inertia (in^4)		
		Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	1- #6 @ d=7.9", 1- #8 @ d=1.1",	15.21	26.02	729.00	156.19	250.23

Vertical Reactions

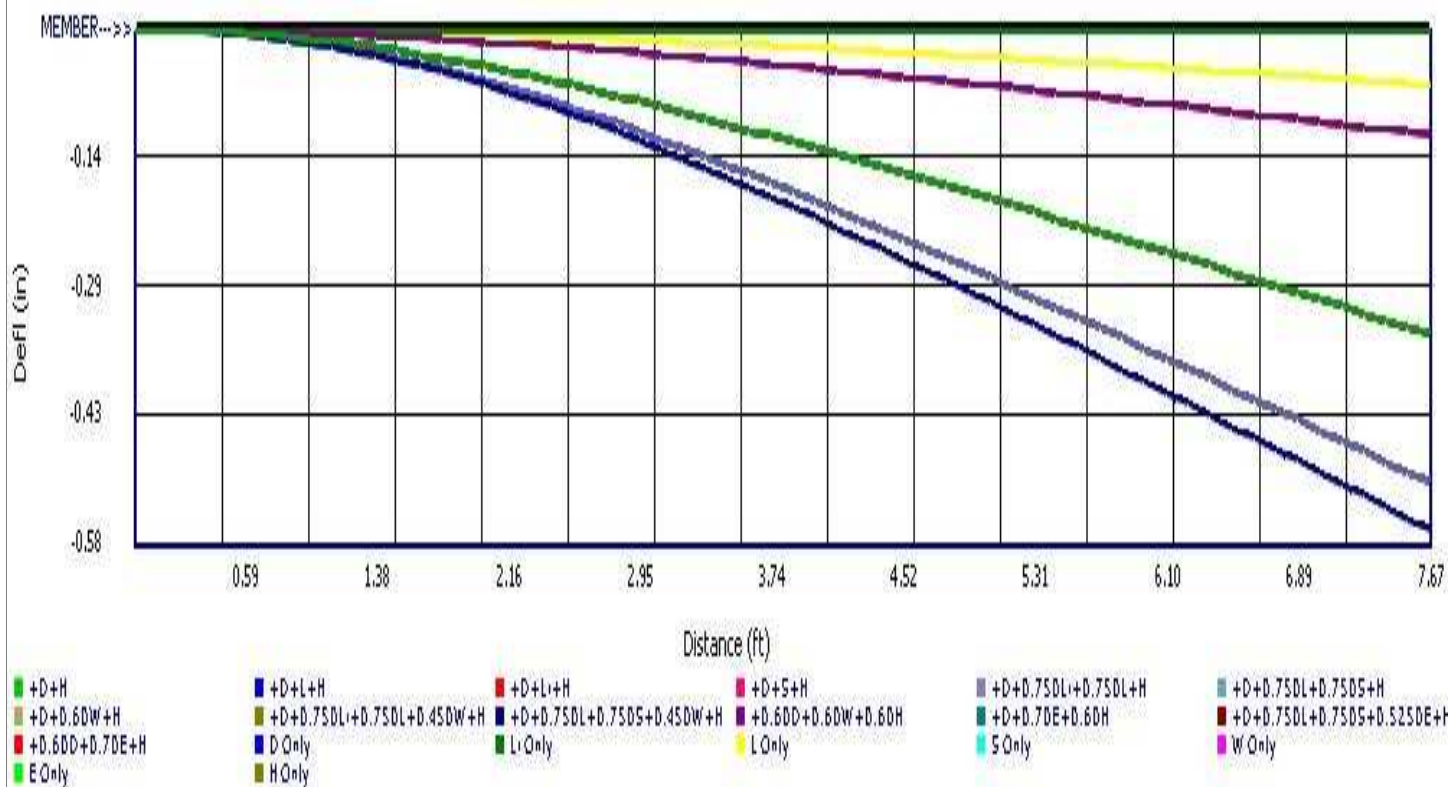
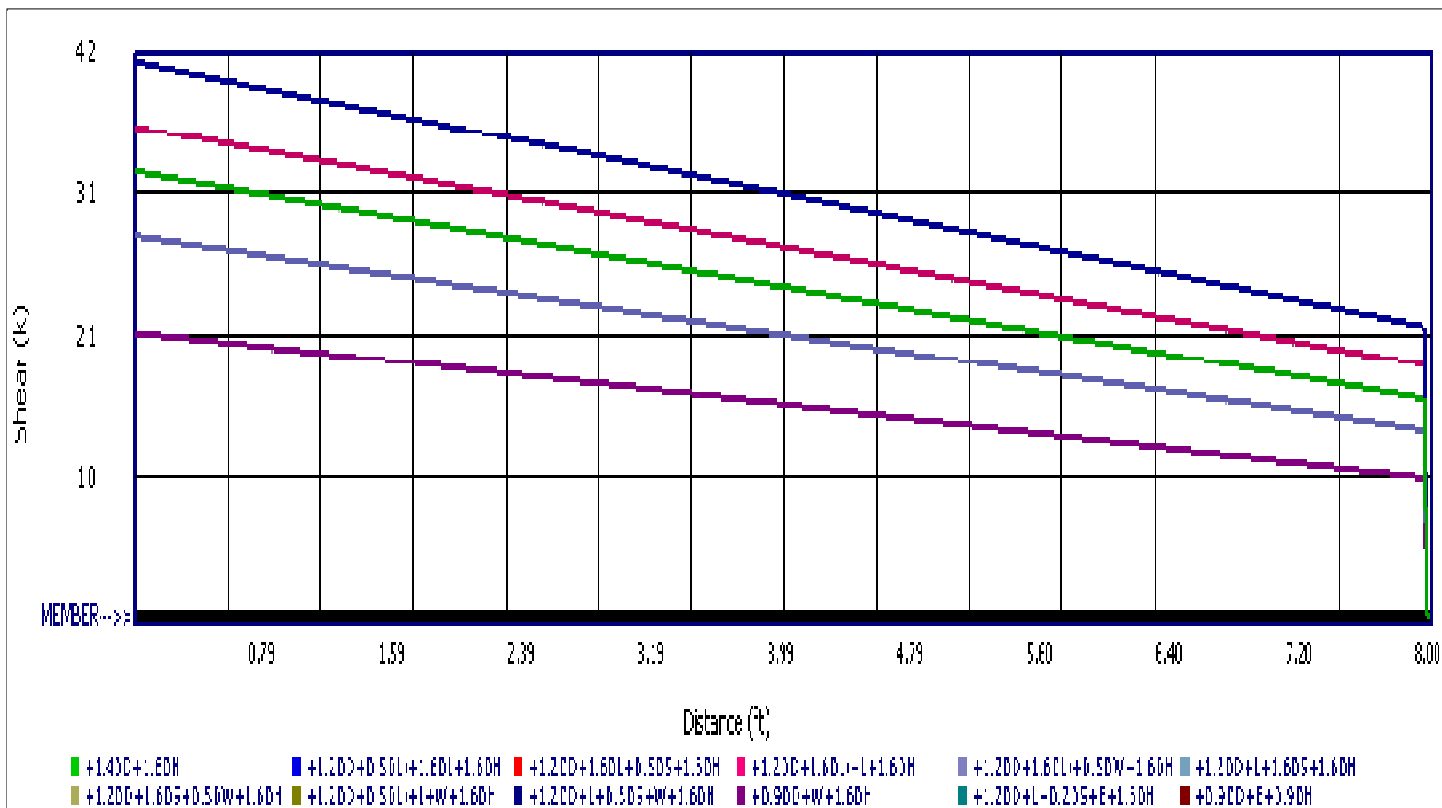
Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	3.131	
Overall MINimum	0.798	
+D+H	2.333	
+D+L+H	3.131	
+D+Lr+H	2.333	
+D+S+H	2.333	

Lic. # : KW-06012775

Support notation : Far left is #1

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Concrete Beam

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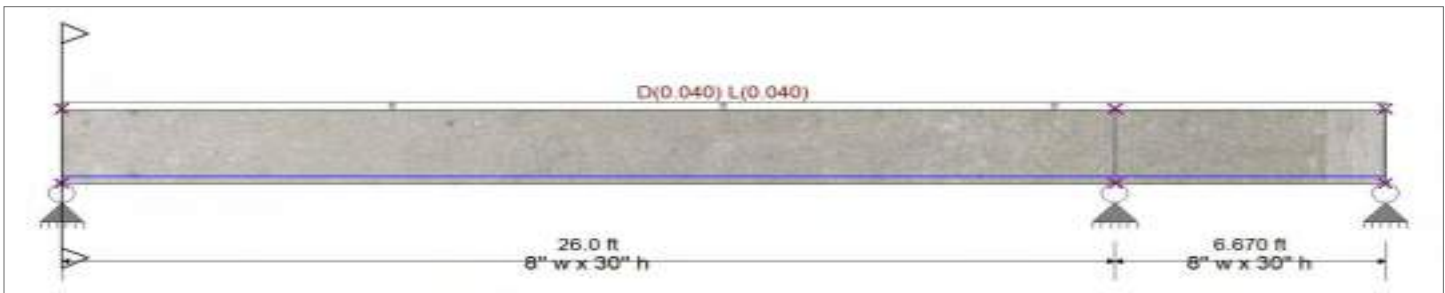
DESCRIPTION: Bridge

CODE REFERENCES

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

Material Properties

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2}$	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,644.15 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

L Section, Stem Width = 8.0 in, Total Height = 30.0 in, Bottom Flange Width = 68.0 in, Flange Thickness = 8.0 in

Span #1 Reinforcing....

2-#6 at 3.0 in from Bottom, from 0.0 to 26.0 ft in this span

Span #2 Reinforcing....

2-#6 at 3.0 in from Bottom, from 0.0 to 6.670 ft in this span

Beam self weight calculated and added to loads

Loads on all spans...

D = 0.040, L = 0.040

Uniform Load on ALL spans : D = 0.040, L = 0.040 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.559	: 1	Maximum Deflection		
Section used for this span		Typical Section		Max Downward Transient Deflection	0.001 in	Ratio = 208055 >= 360.
μ_u : Applied		57.584	k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 < 360.0
$M_n \cdot \Phi$: Allowable		103.076	k-ft	Max Downward Total Deflection	0.030 in	Ratio = 10481 >= 240.
Location of maximum on span		10.400	ft	Max Upward Total Deflection	-0.002 in	Ratio = 48907 >= 240.
Span # where maximum occurs		Span # 1				

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	ΦM_n (k-ft)		Moment of Inertia (in ⁴)		
		Bottom	Top	I gross	I cr - Bottom	I cr - Top
Section 1	2- #6 @ d=27",	103.08	0.00	39,920.00	3,664.16	44.82
Section 2	2- #6 @ d=27",	103.08	0.00	39,920.00	3,664.16	44.82

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	8.349	23.520	-5.695
Overall MINimum	-0.002	0.142	0.127
+D+H	7.933	22.351	-5.292
+D+L+H, LL Comb Run (*L)	7.931	22.493	-5.165

Concrete Beam

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CBenginfo

DESCRIPTION: Bridge

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3
+D+L+H, LL Comb Run (L*)	8.349	23.378	-5.695
+D+L+H, LL Comb Run (LL)	8.348	23.520	-5.569
+D+Lr+H, LL Comb Run (*L)	7.933	22.351	-5.292
+D+Lr+H, LL Comb Run (L*)	7.933	22.351	-5.292
+D+Lr+H, LL Comb Run (LL)	7.933	22.351	-5.292
+D+S+H	7.933	22.351	-5.292
+D+0.750Lr+0.750L+H, LL Comb Run (	7.932	22.458	-5.197
+D+0.750Lr+0.750L+H, LL Comb Run (	8.245	23.121	-5.594
+D+0.750Lr+0.750L+H, LL Comb Run (	8.244	23.228	-5.499
+D+0.750L+0.750S+H, LL Comb Run (*)	7.932	22.458	-5.197
+D+0.750L+0.750S+H, LL Comb Run (L	8.245	23.121	-5.594
+D+0.750L+0.750S+H, LL Comb Run (L	8.244	23.228	-5.499
+D+0.60W+H	7.933	22.351	-5.292
+D+0.750Lr+0.750L+0.450W+H, LL Com	7.932	22.458	-5.197
+D+0.750Lr+0.750L+0.450W+H, LL Com	8.245	23.121	-5.594
+D+0.750Lr+0.750L+0.450W+H, LL Com	8.244	23.228	-5.499
+D+0.750L+0.750S+0.450W+H, LL Comb	7.932	22.458	-5.197
+D+0.750L+0.750S+0.450W+H, LL Comb	8.245	23.121	-5.594
+D+0.750L+0.750S+0.450W+H, LL Comb	8.244	23.228	-5.499
+0.60D+0.60W+0.60H	4.760	13.411	-3.175
+D+0.70E+0.60H	7.933	22.351	-5.292
+D+0.750L+0.750S+0.5250E+H, LL Com	7.932	22.458	-5.197
+D+0.750L+0.750S+0.5250E+H, LL Com	8.245	23.121	-5.594
+D+0.750L+0.750S+0.5250E+H, LL Com	8.244	23.228	-5.499
+0.60D+0.70E+H	4.760	13.411	-3.175
D Only	7.933	22.351	-5.292
L Only, LL Comb Run (*L)	-0.002	0.142	0.127
L Only, LL Comb Run (L*)	0.417	1.027	-0.403
L Only, LL Comb Run (LL)	0.415	1.169	-0.277
H Only			

Shear Stirrup Requirements

Entire Beam Span Length : $\Phi V_c/2 < V_u \leq \Phi V_c$, Req'd Vs = Not Req'd 11.4.6.1, use #3 stirrups spaced at 0.000 in

Overall Maximum Deflections

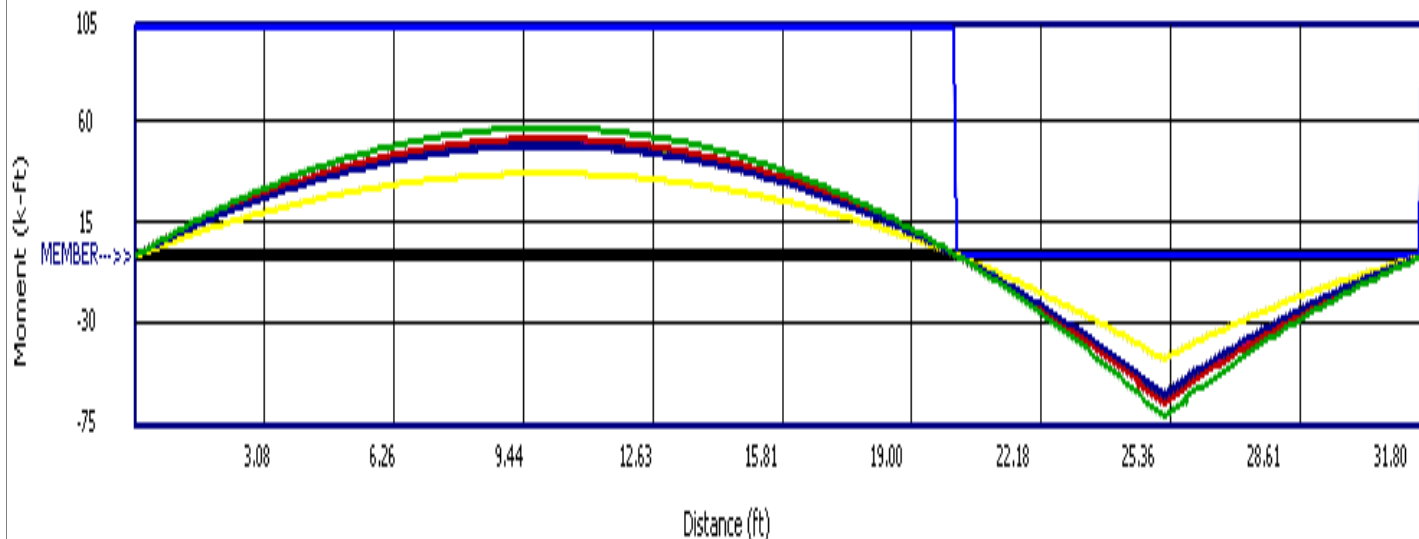
Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L+H, LL Comb Run (L*)	1	0.0298	11.514	+D+L+H, LL Comb Run (L*)	-0.0001	26.095
	2	0.0000	11.514	+D+L+H, LL Comb Run (L*)	-0.0016	2.763

Concrete Beam

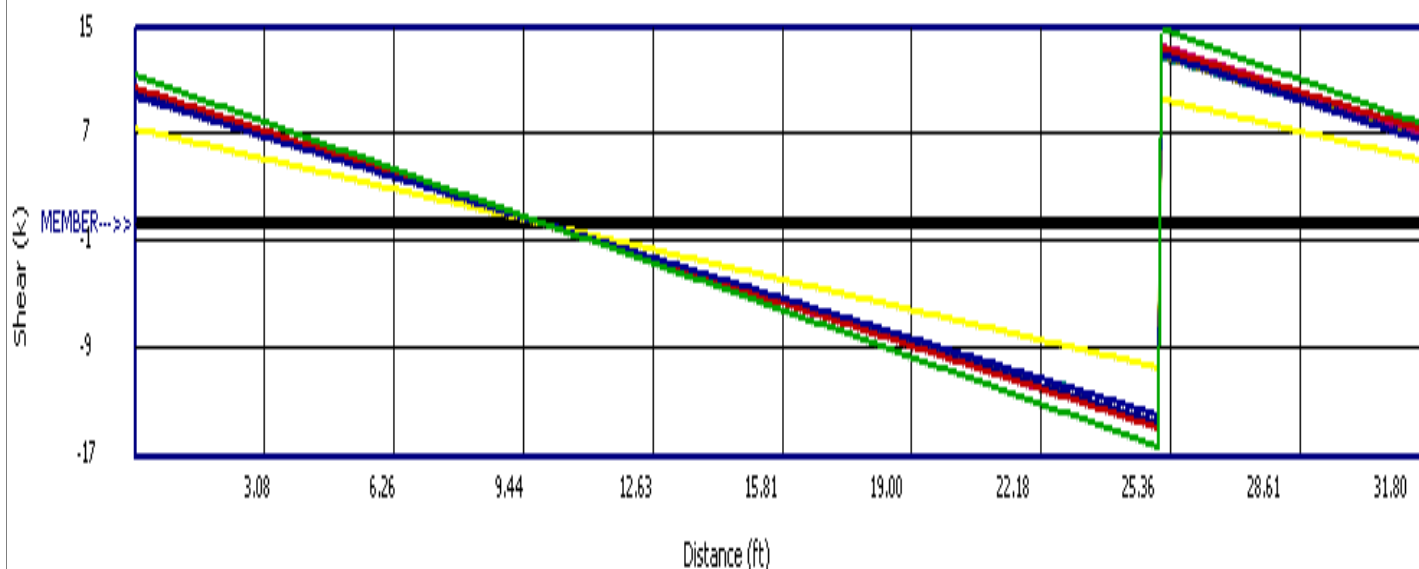
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DESCRIPTION: Bridge



- | | | | |
|--|--|--|--|
| +1.4DD+1.6DH | +1.2DD+0.5DL+1.6DL+1.6DH, LL Comb Run (*L) | +1.2DD+0.5DL+1.6DL+1.6DH, LL Comb Run (L*) | +1.2DD+0.5DL+1.6DL+1.6DH, LL Comb Run (LL) |
| +1.2DD+1.6DL+0.5DS+1.6DH, LL Comb Run (*L) | +1.2DD+1.6DL+0.5DS+1.6DH, LL Comb Run (L*) | +1.2DD+1.6DL+0.5DS+1.6DH, LL Comb Run (LL) | +1.2DD+1.6DL+1.6DH, LL Comb Run (*L) |
| +1.2DD+1.6DL+L+1.6DH, LL Comb Run (L*) | +1.2DD+1.6DL+L+1.6DH, LL Comb Run (LL) | +1.2DD+1.6DL+0.5DW+1.6DH, LL Comb Run (*L) | +1.2DD+1.6DL+0.5DW+1.6DH, LL Comb Run (L*) |
| +1.2DD+1.6DL+0.5DW+1.6DH, LL Comb Run (LL) | +1.2DD+L+1.6DS+1.6DH, LL Comb Run (*L) | +1.2DD+L+1.6DS+1.6DH, LL Comb Run (L*) | +1.2DD+L+1.6DS+1.6DH, LL Comb Run (LL) |
| +1.2DD+1.6DS+0.5DW+1.6DH | +1.2DD+0.5DL+L+W+1.6DH, LL Comb Run (*L) | +1.2DD+0.5DL+L+W+1.6DH, LL Comb Run (L*) | +1.2DD+0.5DL+L+W+1.6DH, LL Comb Run (LL) |
| +1.2DD+L+0.5DS+W+1.6DH, LL Comb Run (*L) | +1.2DD+L+0.5DS+W+1.6DH, LL Comb Run (L*) | +1.2DD+L+0.5DS+W+1.6DH, LL Comb Run (LL) | +0.9DD+W+1.6DH |
| +1.2DD+L+0.2DS+E+1.6DH, LL Comb Run (*L) | +1.2DD+L+0.2DS+E+1.6DH, LL Comb Run (L*) | +1.2DD+L+0.2DS+E+1.6DH, LL Comb Run (LL) | +0.9DD+E+0.9DH |
| phi*Mn | | | |



- | | | | |
|--|--|--|--|
| +1.4DD+1.6DH | +1.2DD+0.5DL+1.6DL+1.6DH, LL Comb Run (*L) | +1.2DD+0.5DL+1.6DL+1.6DH, LL Comb Run (L*) | +1.2DD+0.5DL+1.6DL+1.6DH, LL Comb Run (LL) |
| +1.2DD+1.6DL+0.5DS+1.6DH, LL Comb Run (*L) | +1.2DD+1.6DL+0.5DS+1.6DH, LL Comb Run (L*) | +1.2DD+1.6DL+0.5DS+1.6DH, LL Comb Run (LL) | +1.2DD+1.6DL+L+1.6DH, LL Comb Run (*L) |
| +1.2DD+1.6DL+L+1.6DH, LL Comb Run (L*) | +1.2DD+1.6DL+L+1.6DH, LL Comb Run (LL) | +1.2DD+1.6DL+0.5DW+1.6DH, LL Comb Run (*L) | +1.2DD+1.6DL+0.5DW+1.6DH, LL Comb Run (L*) |
| +1.2DD+1.6DL+0.5DW+1.6DH, LL Comb Run (LL) | +1.2DD+L+1.6DS+1.6DH, LL Comb Run (*L) | +1.2DD+L+1.6DS+1.6DH, LL Comb Run (L*) | +1.2DD+L+1.6DS+1.6DH, LL Comb Run (LL) |
| +1.2DD+1.6DS+0.5DW+1.6DH | +1.2DD+0.5DL+L+W+1.6DH, LL Comb Run (*L) | +1.2DD+0.5DL+L+W+1.6DH, LL Comb Run (L*) | +1.2DD+0.5DL+L+W+1.6DH, LL Comb Run (LL) |
| +1.2DD+L+0.5DS+W+1.6DH, LL Comb Run (*L) | +1.2DD+L+0.5DS+W+1.6DH, LL Comb Run (L*) | +1.2DD+L+0.5DS+W+1.6DH, LL Comb Run (LL) | +0.9DD+W+1.6DH |
| +1.2DD+L+0.2DS+E+1.6DH, LL Comb Run (*L) | +1.2DD+L+0.2DS+E+1.6DH, LL Comb Run (L*) | +1.2DD+L+0.2DS+E+1.6DH, LL Comb Run (LL) | +0.9DD+E+0.9DH |
| phi*Mn | | | |

Concrete Beam

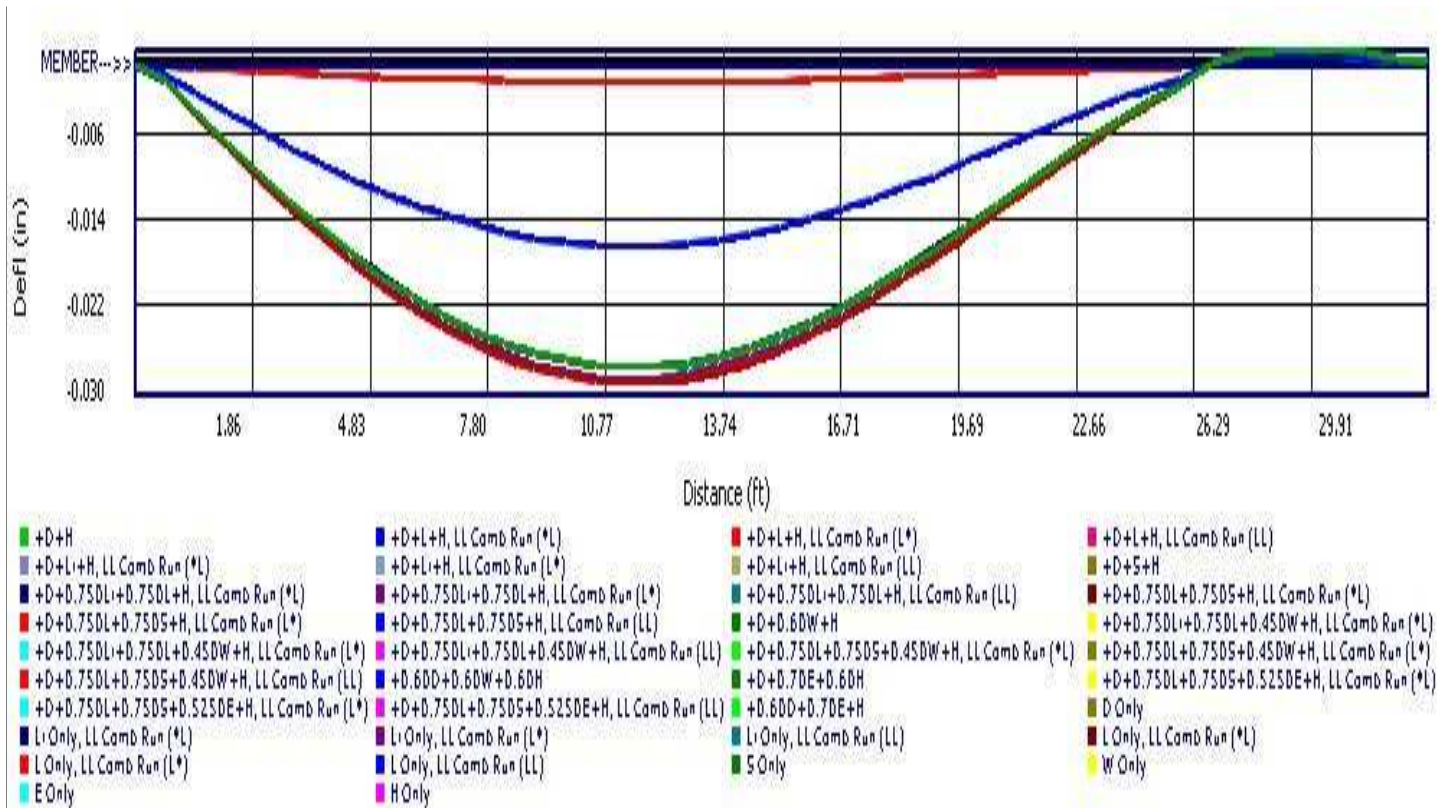
File: New Residence.ec6

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DESCRIPTION: Bridge



Concrete Beam

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DESCRIPTION: Flange Bridge

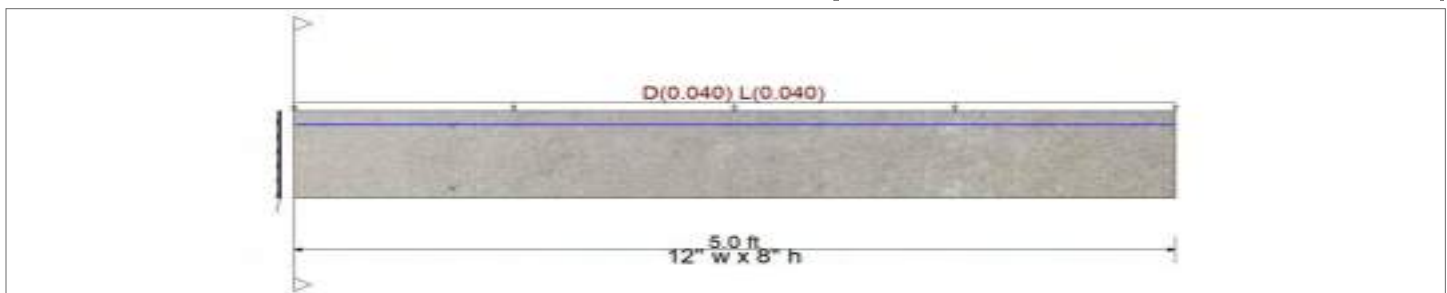
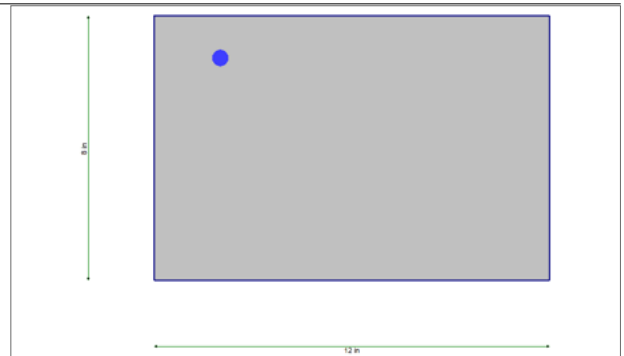
CODE REFERENCES

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

Load Combination Set : ASCE 7-16

Material Properties

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} * 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,644.15 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 = 12.0 in, Height = 8.0 in

Span #1 Reinforcing....

1-#4 at 1.25 in from Top, from 0.0 to 5.0 ft in this span

Beam self weight calculated and added to loads

Loads on all spans...

D = 0.040, L = 0.040

Uniform Load on ALL spans : D = 0.040, L = 0.040 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.480 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.003 in	Ratio = 41462 >=360.
μ : Applied		-2.850 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 <360.0
$M_n * \phi$: Allowable		5.943 k-ft	Max Downward Total Deflection	0.013 in	Ratio = 9386 >=240.
Location of maximum on span		0.000 ft	Max Upward Total Deflection	0.000 in	Ratio = 0 <240.0
Span # where maximum occurs		Span # 1			

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	$\phi * M_n$ (k-ft)		Moment of Inertia (in ⁴)		
		Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	1- #4 @ d=1.25",	0.00	5.94	512.00	1.41	55.94

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	0.883	
Overall MINimum	0.200	
+D+H	0.683	
+D+L+H	0.883	
+D+Lr+H	0.683	
+D+S+H	0.683	
+D+0.750Lr+0.750L+H	0.833	

Concrete Beam

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DESCRIPTION: Flange Bridge

Vertical Reactions

Support notation : Far left is #1

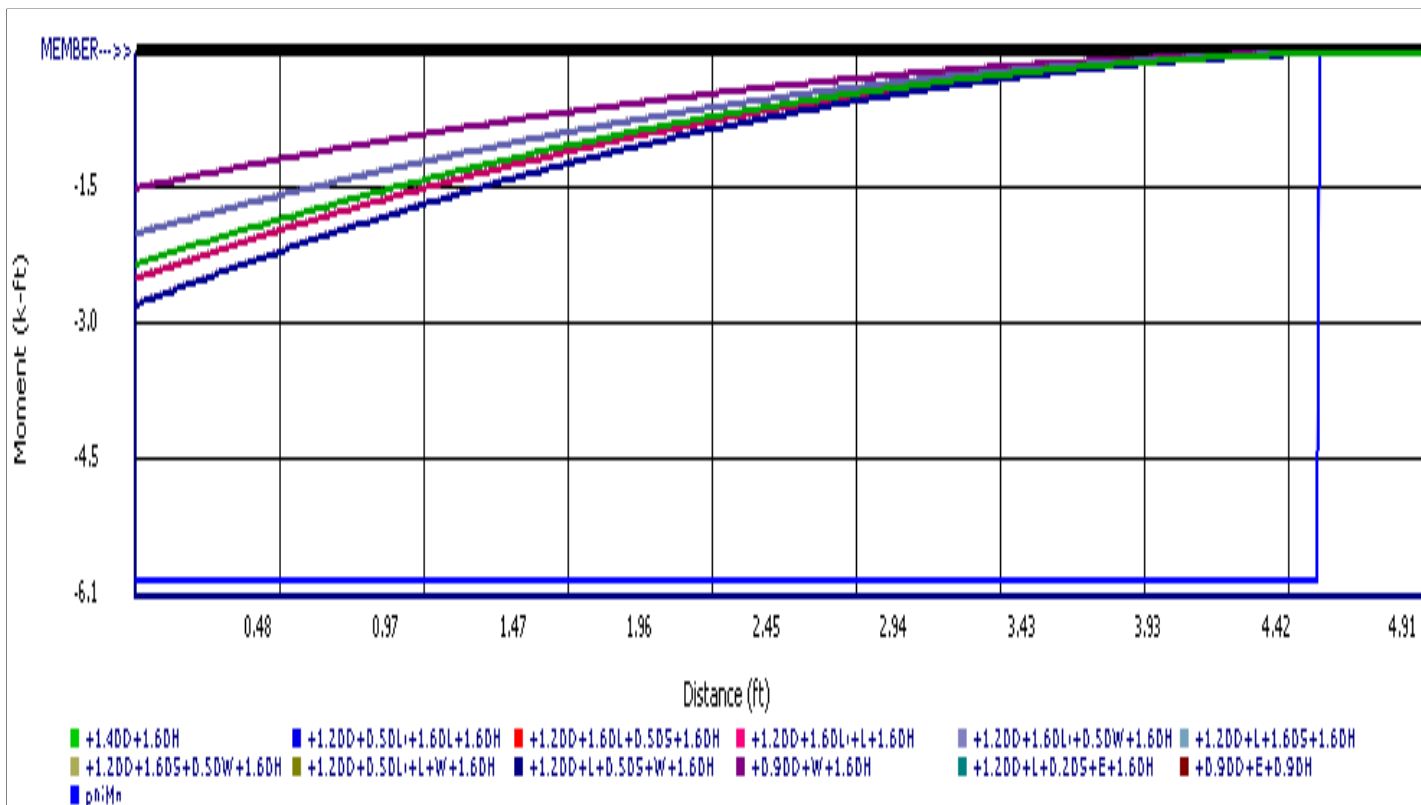
Load Combination	Support 1	Support 2
+D+0.750L+0.750S+H	0.833	
+D+0.60W+H	0.683	
+D+0.750Lr+0.750L+0.450W+H	0.833	
+D+0.750L+0.750S+0.450W+H	0.833	
+0.60D+0.60W+0.60H	0.410	
+D+0.70E+0.60H	0.683	
+D+0.750L+0.750S+0.5250E+H	0.833	
+0.60D+0.70E+H	0.410	
D Only	0.683	
L Only	0.200	
H Only		

Shear Stirrup Requirements

Entire Beam Span Length : $V_u < \Phi V_c/2$, Req'd Vs = Not Req'd 11.4.6.1, use #3 stirrups spaced at 0.000 in

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L+H	1	0.0128	5.000		0.0000	0.000

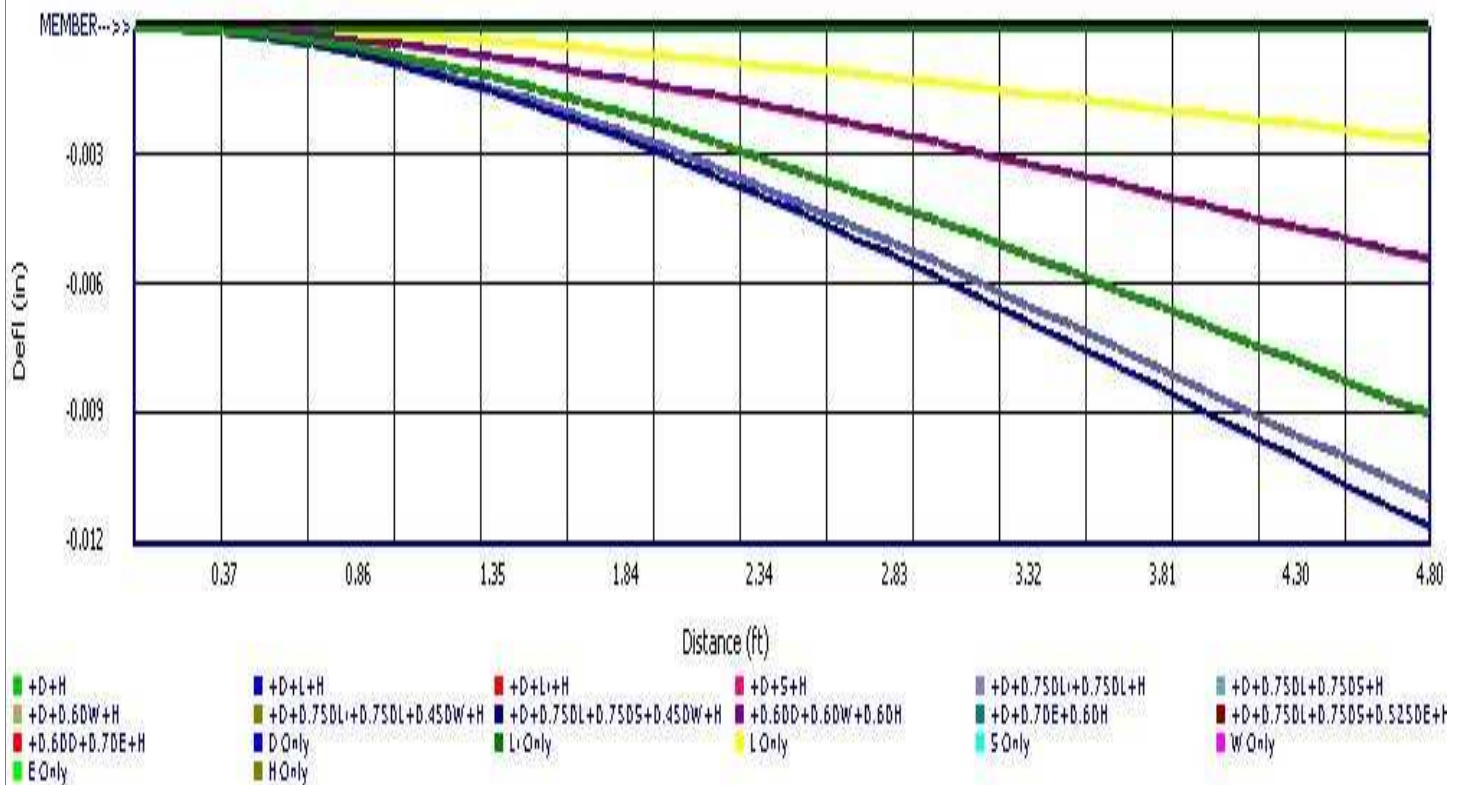
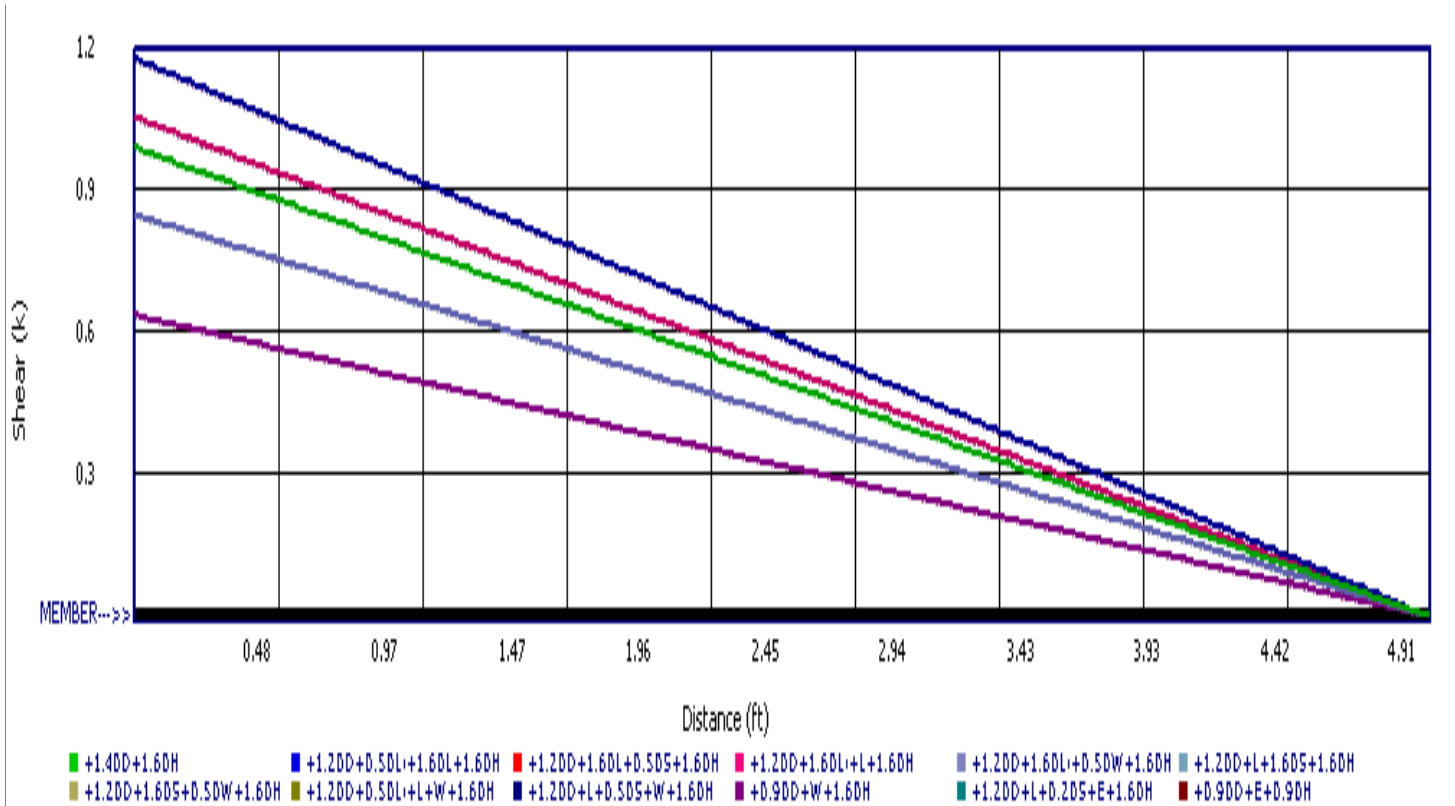


Concrete Beam

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DESCRIPTION: Flange Bridge



Concrete Beam

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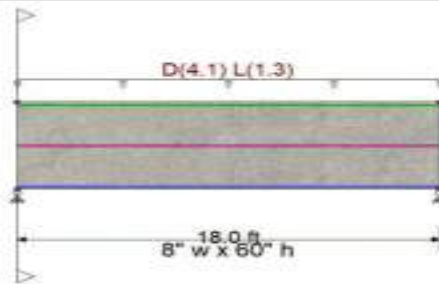
DESCRIPTION: 2B-2

CODE REFERENCES

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

Material Properties

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} * 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,644.15 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 = 60.0 in

Span #1 Reinforcing...

2-#7 at 2.30 in from Bottom, from 0.0 to 18.0 ft in this span
2-#7 at 30.0 in from Bottom, from 0.0 to 18.0 ft in this span

2-#7 at 2.50 in from Top, from 0.0 to 18.0 ft in this span

Beam self weight calculated and added to loads

Load for Span Number 1

Uniform Load : D = 4.10, L = 1.30 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.680 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.006 in	Ratio = 36926 >= 360.
Mu : Applied		306.989 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 < 360.0
Mn * Phi : Allowable		451.402 k-ft	Max Downward Total Deflection	0.040 in	Ratio = 5344 >= 240.
Location of maximum on span		9.016 ft	Max Upward Total Deflection	0.000 in	Ratio = 0 < 240.0
Span # where maximum occurs		Span # 1			

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	Phi*Mn (k-ft)		Moment of Inertia (in^4)		
		Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	2- #7 @ d=57.7", 2- #7 @ d=2.5", 2- #7 @ d=30",	451.40	451.26	144000.00	28,381.69	28,236.71

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	52.950	52.950
Overall MINimum	11.700	11.700
+D+H	41.250	41.250
+D+L+H	52.950	52.950
+D+Lr+H	41.250	41.250
+D+S+H	41.250	41.250
+D+0.750Lr+0.750L+H	50.025	50.025

Concrete Beam

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DESCRIPTION: 2B-2

Vertical Reactions

Support notation : Far left is #1

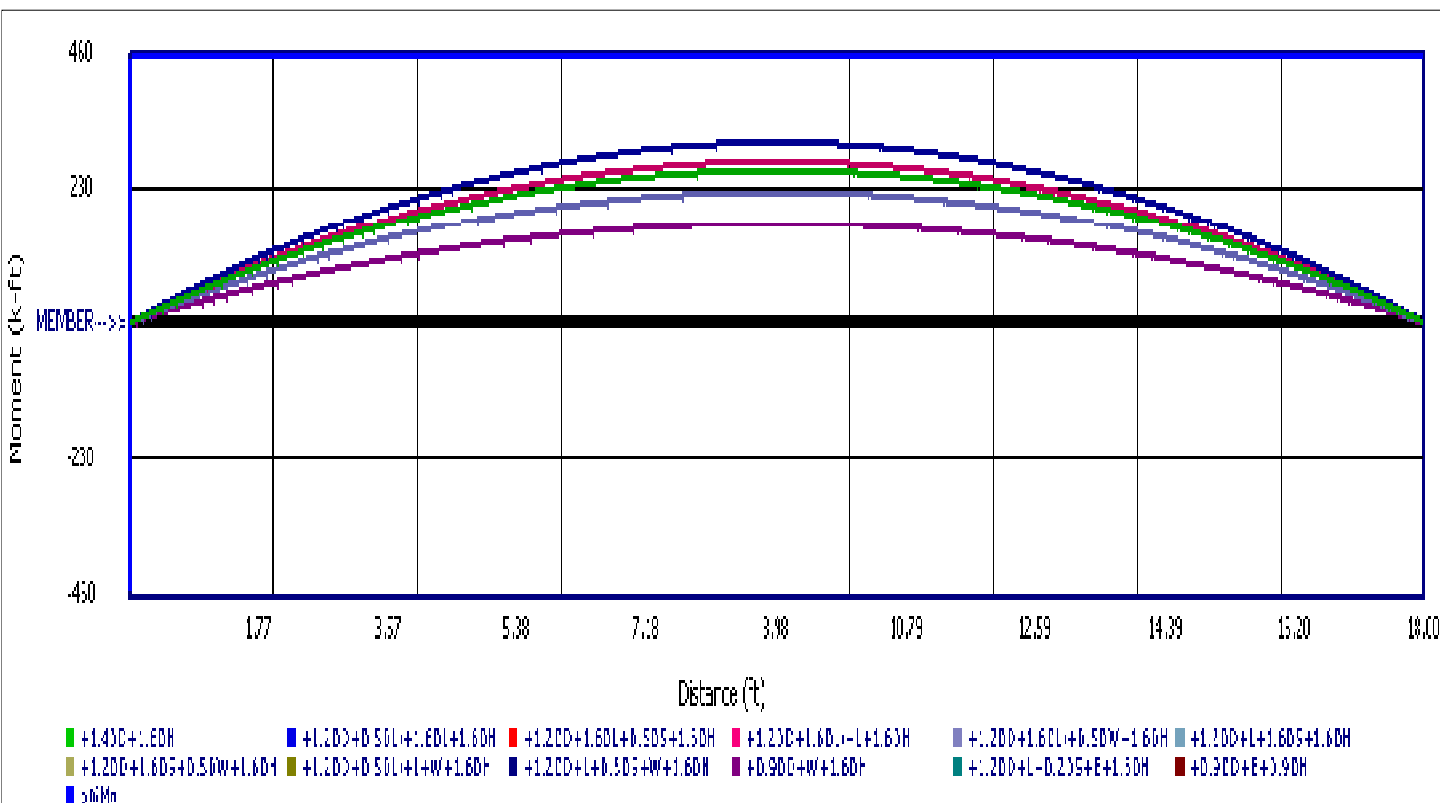
Load Combination	Support 1	Support 2
+D+0.750L+0.750S+H	50.025	50.025
+D+0.60W+H	41.250	41.250
+D+0.750Lr+0.750L+0.450W+H	50.025	50.025
+D+0.750L+0.750S+0.450W+H	50.025	50.025
+0.60D+0.60W+0.60H	24.750	24.750
+D+0.70E+0.60H	41.250	41.250
+D+0.750L+0.750S+0.5250E+H	50.025	50.025
+0.60D+0.70E+H	24.750	24.750
D Only	41.250	41.250
L Only	11.700	11.700
H Only		

Shear Stirrup Requirements

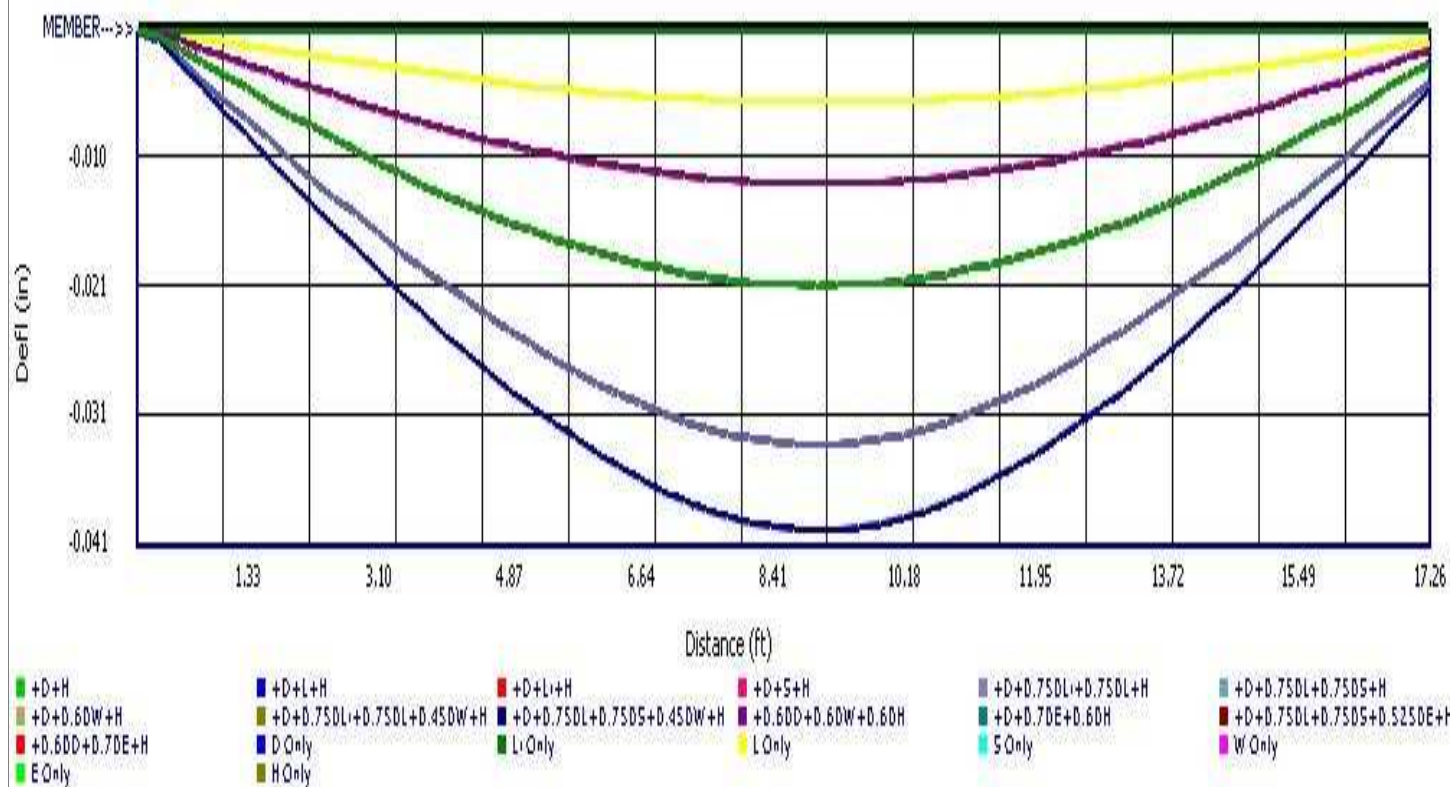
Between 0.00 to 6.13 ft, $\Phi V_c/2 < V_u \leq \Phi V_c$, Req'd Vs = Min 11.4.6.1, use #3 stirrups spaced at 24.000 in
Between 6.16 to 11.84 ft, $V_u < \Phi V_c/2$, Req'd Vs = Not Reqd 11.4.6.1, use #3 stirrups spaced at 0.000 in
Between 11.87 to 17.97 ft, $\Phi V_c < V_u$, Req'd Vs = 22.118, use #3 stirrups spaced at 24.000 in

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L+H	1	0.0404	9.000		0.0000	0.000



DESCRIPTION: 2B-2



Concrete Beam

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CBenginfo

DESCRIPTION: 2B-3

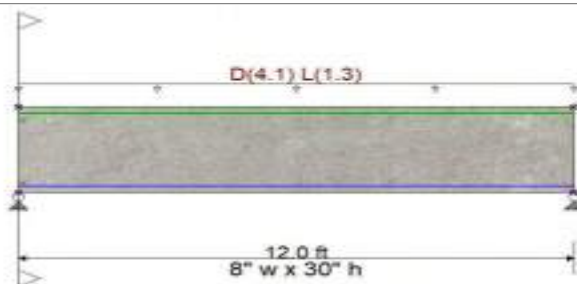
CODE REFERENCES

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

Load Combination Set : ASCE 7-16

Material Properties

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} * 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,644.15 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 = 30.0 in

Span #1 Reinforcing....

2-#7 at 2.30 in from Bottom, from 0.0 to 12.0 ft in this span

2-#7 at 2.30 in from Top, from 0.0 to 12.0 ft in this span

Beam self weight calculated and added to loads

Load for Span Number 1

Uniform Load : D = 4.10, L = 1.30 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.921 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.009 in	Ratio = 15578 >= 360.
M_u : Applied		131.220 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 < 360.0
$M_n * \Phi$: Allowable		142.547 k-ft	Max Downward Total Deflection	0.113 in	Ratio = 1278 >= 240.
Location of maximum on span		5.989 ft	Max Upward Total Deflection	0.000 in	Ratio = 0 < 240.0
Span # where maximum occurs		Span # 1			

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	ΦM_n (k-ft)		Moment of Inertia (in ⁴)		
		Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	2- #7 @ d=27.7", 2- #7 @ d=2.3",	142.55	142.55	18,000.00	5,171.54	5,171.54

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	33.850	33.850
Overall MINimum	7.800	7.800
+D+H	26.050	26.050
+D+L+H	33.850	33.850
+D+Lr+H	26.050	26.050
+D+S+H	26.050	26.050
+D+0.750Lr+0.750L+H	31.900	31.900
+D+0.750L+0.750S+H	31.900	31.900

Concrete Beam

File: New Residence.ec6
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DESCRIPTION: 2B-3

Vertical Reactions

Support notation : Far left is #1

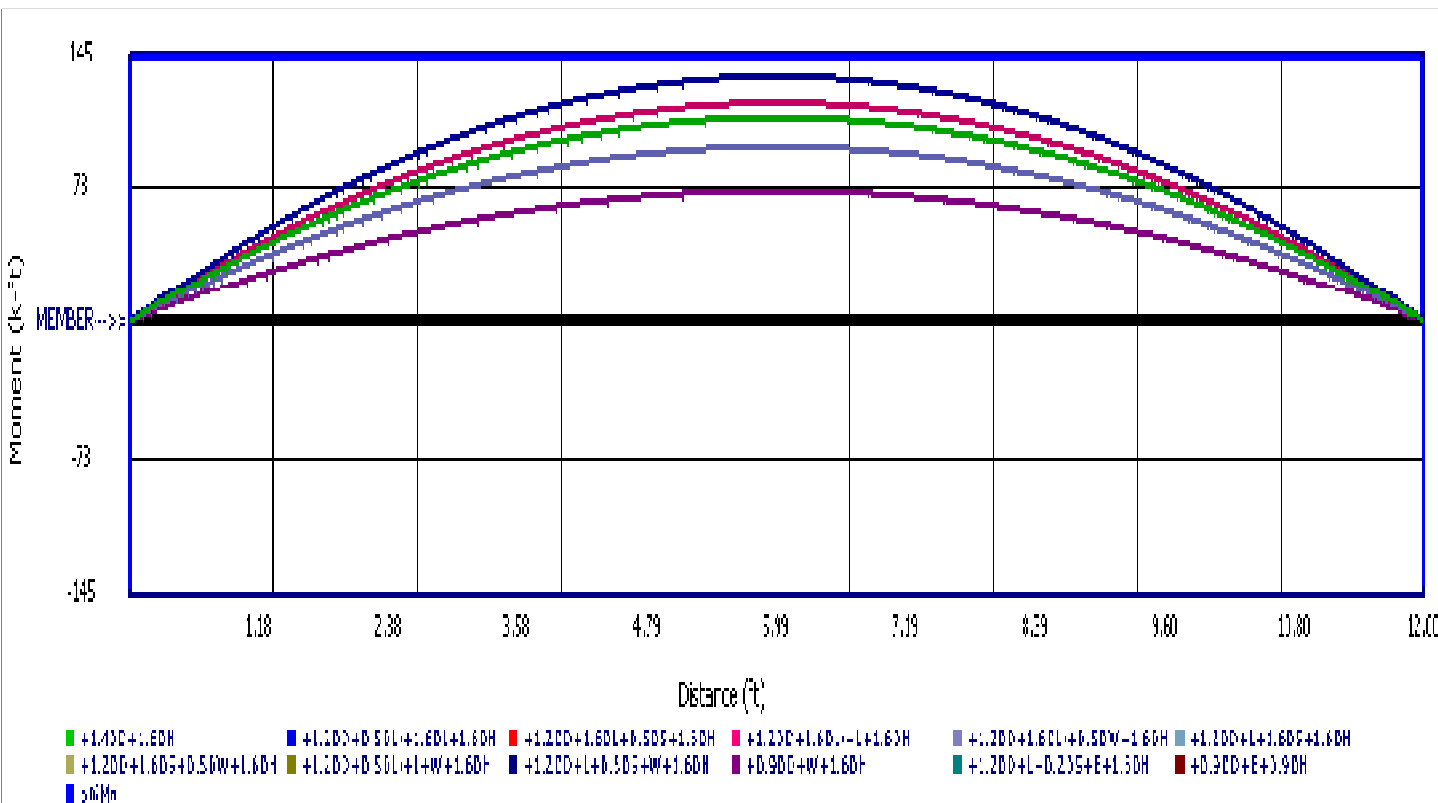
Load Combination	Support 1	Support 2
+D+0.60W+H	26.050	26.050
+D+0.750L+0.750L+0.450W+H	31.900	31.900
+D+0.750L+0.750S+0.450W+H	31.900	31.900
+0.60D+0.60W+0.60H	15.630	15.630
+D+0.70E+0.60H	26.050	26.050
+D+0.750L+0.750S+0.5250E+H	31.900	31.900
+0.60D+0.70E+H	15.630	15.630
D Only	26.050	26.050
L Only	7.800	7.800
H Only		

Shear Stirrup Requirements

Between 0.00 to 3.10 ft, $\Phi V_c < V_u$, Req'd Vs = 0.05191, use #3 stirrups spaced at 12.000 in
Between 3.13 to 4.59 ft, $\Phi V_c/2 < V_u \leq \Phi V_c$, Req'd Vs = Min 11.4.6.1, use #3 stirrups spaced at 13.000 in
Between 4.61 to 7.39 ft, $V_u < \Phi V_c/2$, Req'd Vs = Not Req'd 11.4.6.1, use #3 stirrups spaced at 0.000 in
Between 7.41 to 8.87 ft, $\Phi V_c/2 < V_u \leq \Phi V_c$, Req'd Vs = Min 11.4.6.1, use #3 stirrups spaced at 13.000 in
Between 8.90 to 11.98 ft, $\Phi V_c < V_u$, Req'd Vs = 21.518, use #3 stirrups spaced at 12.000 in

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+L+H	1	0.1127	6.000		0.0000	0.000



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DESCRIPTION: 2B-3

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