Jess Andersen
Structural Calculations

Logan, Utah

REGISTERED PROFESSIONAL ENGINEER
NO. 275617
BRIAN G.
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3/29/25
STATE OF UTAH

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Design Criteria

Governing Code: *2021 IBC*

Snow Criteria

Roof Snow Load (P_f)	40 psf	
Ground Load (P_g)	57 psf	
Exposure Factor (C_e)	1.0	Partially
Thermal Factor (C_t)	1.0	Typical
Slope Factor (Cs)	66.8%	
Importance (I_s)	1.0	

Wind Criteria

Wind Speed (V_3)	115 mph	
Wind Exposure	C	Open Terrain
Wind Importance (I_w)	1.0	
Building Category	II	

Seismic Criteria

Site Class	D	Stiff Soil		
Ss	1.07		Fa	1.20
S1	0.36		Fv	1.50
S_{DS}	0.85		S_{D1}	0.36
Risk Category	II	Other		
Seismic Importance (I_E)	1.0			
Seismic Design Category (SDC)	D			

Wall Material	Design Base Shear	Seismic Response Coefficient, R	
OSB	.13Wp	6.5	Typ @ Ext
GYP	.43Wp	2	Typ @ Int
Conc.	.57Wp	1.5	

Live Loads

Typ Residential	40 psf
Garage (P.V.)	40 psf
	-

Soil Bearing

Typical	1500 psf
Frost Depth	30 in

Roof Dead Loads:

Deck	*1.5*
Insulation	*2.0*
Roofing	*3.0*
Joist	*2.5*
Ceiling	*3.0*
Misc	*2.5*
TOTAL	**15 psf**

Floor Dead Loads:

Deck	2.5
Joist	2.0
Ceiling	2.0
Flooring	2.5
Misc	3.0
TOTAL	**12 psf**

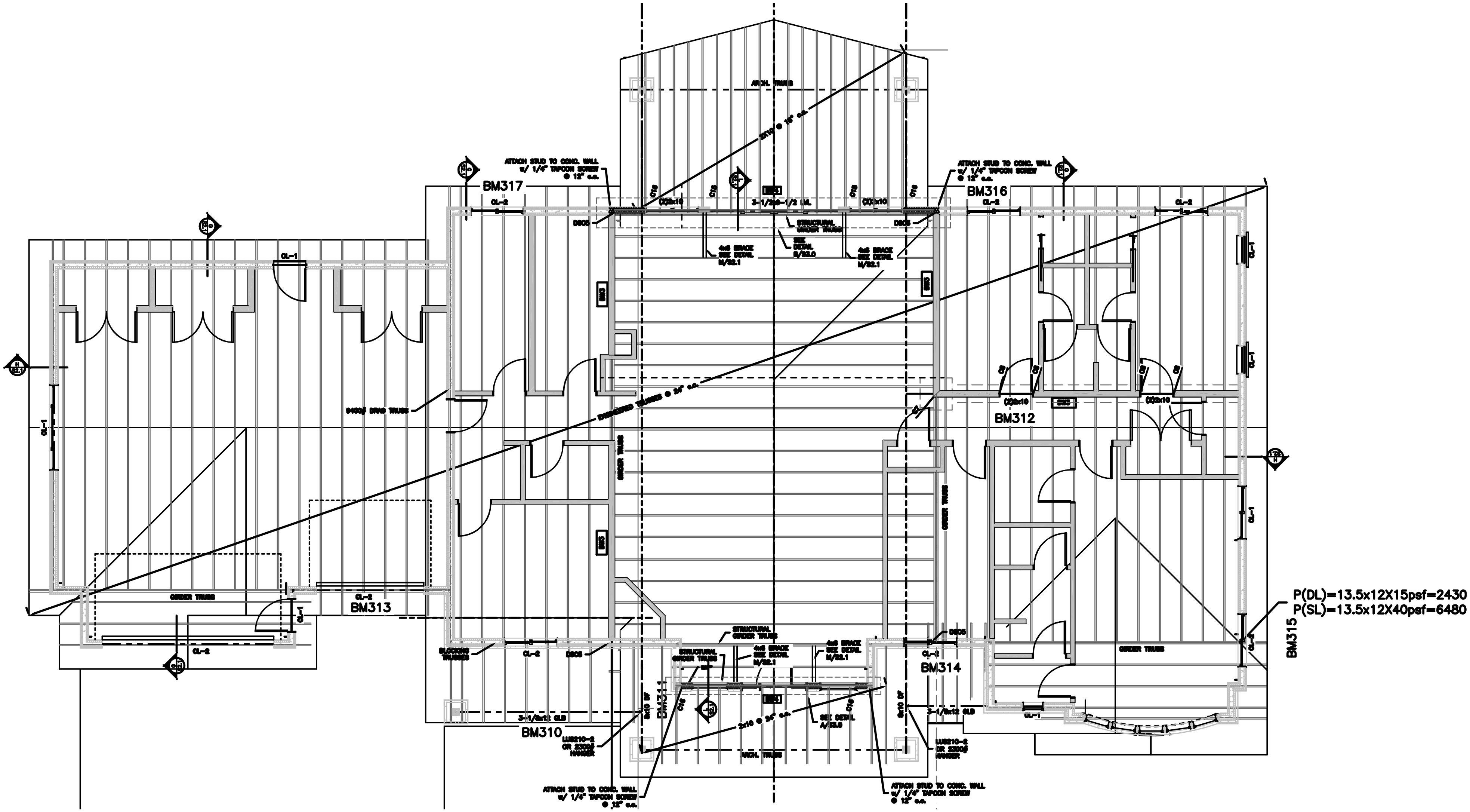
Exterior Wall Dead Loads:

Conc.	*75.0*
ICF	*4.0*
Misc	*6.0*
TOTAL	**85 psf**

Interior Wall Dead Loads:

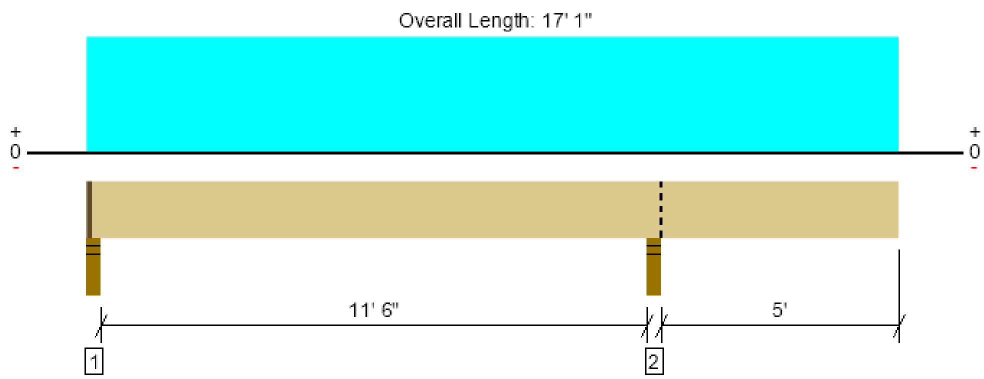
Studs	*2.0*
Gyp. Board	*2.5*
-	-
-	-
-	-
Misc	*3.0*
TOTAL	**8 psf**

BEAMS


BM317
BM316
BM312
BM313
BM310
BM311
BM314
BM315
P(DL)=13.5x12X15psf=2430
P(SL)=13.5x12X40psf=6480
ATTACH STUD TO CONC. WALL w/ 1/4" TAPCON SCREW @ 12" o.c.
GIRDER TRUSS
ARCH. TRUSS
STRUCTURAL GIRDER TRUSS
4x6 BRACE SEE DETAIL M/S2.1
BLOCKING TRUSSES
3-1/2x12 GLB
CL-1
CL-2

Roof, rear rafters

1 piece(s) 2 x 10 DF No.2 @ 16" OC



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	890 @ 11' 11 1/4"	3281 (3.50")	Passed (27%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	446 @ 11' 1/4"	1915	Passed (23%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	971 @ 5' 4 1/4"	2334	Passed (42%)	1.15	1.0 D + 1.0 S (Alt Spans)
Live Load Defl. (in)	0.105 @ 17' 1"	0.343	Passed (2L/999+)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.140 @ 5' 9 1/4"	0.586	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 16' 11 3/4"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (2L/360) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - DF	3.50"	2.25"	1.50"	99	294	393	1 1/4" Rim Board
2 - Stud wall - DF	3.50"	3.50"	1.50"	243	647	890	Blocking

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	13' 7" o/c	
Bottom Edge (Lu)	13' 7" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 17' 1"	16"	15.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Brian Lyon Alliance Consulting Engineers (435) 755-5121 alliancelogan@yahoo.com	

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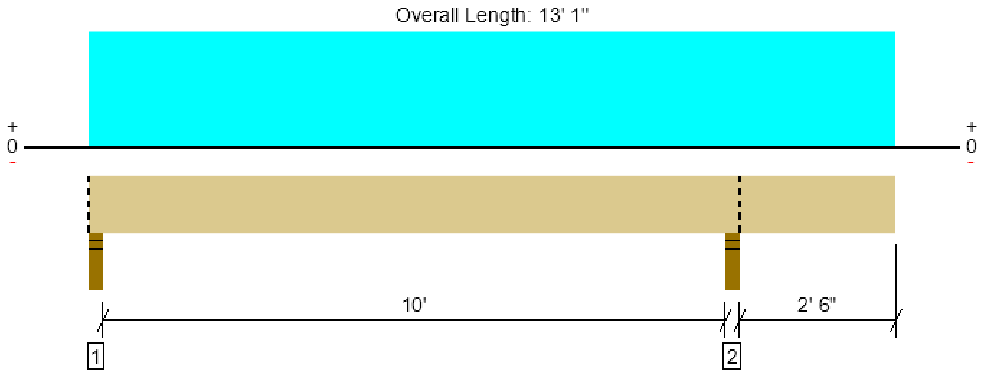
Roof, front rafters 2x10

1 piece(s) 2 x 10 DF No.2 @ 24" OC



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	891 @ 10' 5 1/4"	3281 (3.50")	Passed (27%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	499 @ 9' 6 1/4"	1915	Passed (26%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1319 @ 5' 1 1/4"	2334	Passed (57%)	1.15	1.0 D + 1.0 S (Alt Spans)
Live Load Defl. (in)	0.115 @ 5' 3 1/8"	0.511	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.154 @ 5' 2 7/8"	0.682	Passed (L/798)	--	1.0 D + 1.0 S (Alt Spans)

Member Length : 13' 1"
System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - DF	3.50"	3.50"	1.50"	149	412	562	Blocking
2 - Stud wall - DF	3.50"	3.50"	1.50"	243	648	891	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 7" o/c	
Bottom Edge (Lu)	13' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 13' 1"	24"	15.0	40.0	Default Load

Weyerhaeuser Notes

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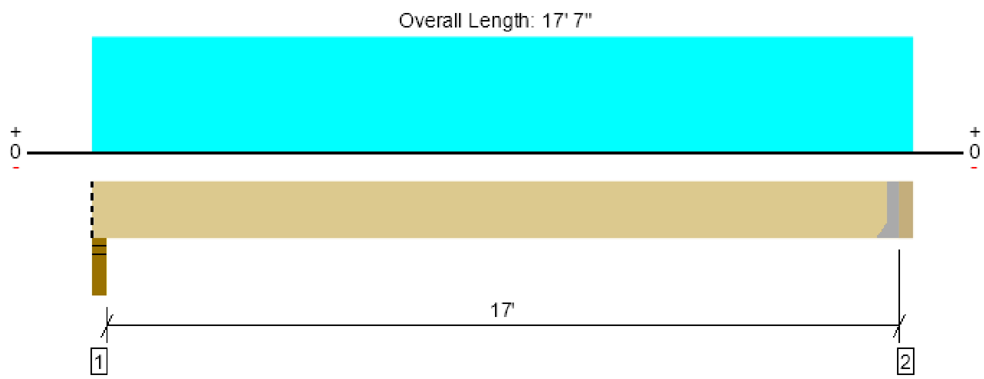
Brian Lyon Alliance Consulting Engineers (435) 755-5121 alliancelogan@yahoo.com	

Roof, BM310

1 piece(s) 3 1/8" x 12" 24F-V4 DF Glulam



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2197 @ 17' 3 1/2"	3047 (1.50")	Passed (72%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1941 @ 16' 3 1/2"	7619	Passed (25%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	9407 @ 8' 8 3/4"	17250	Passed (55%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.430 @ 8' 8 3/4"	0.571	Passed (L/478)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.613 @ 8' 8 3/4"	0.856	Passed (L/335)	--	1.0 D + 1.0 S (All Spans)

Member Length : 17' 3 1/2"
System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 17' 1 1/2".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - DF	3.50"	3.50"	1.50"	669	1571	2240	Blocking
2 - Hanger on 12" DF beam	3.50"	Hanger[1]	1.50"	676	1594	2269	See note [1]

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	17' 4" o/c	
Bottom Edge (Lu)	17' 4" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	LUS210-2	2.00"	N/A	8-SD9112	6-SD9212	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 17' 3 1/2"	N/A	9.1	--	
1 - Uniform (PSF)	0 to 17' 7" (Front)	4' 6"	15.0	40.0	Default Load

- Side loads are assumed to not induce cross-grain tension.

ForteWEB Software Operator	Job Notes
Brian Lyon Alliance Consulting Engineers (435) 755-5121 alliancelogan@yahoo.com	

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3/29/2025 9:10:35 PM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: Jess Anderson

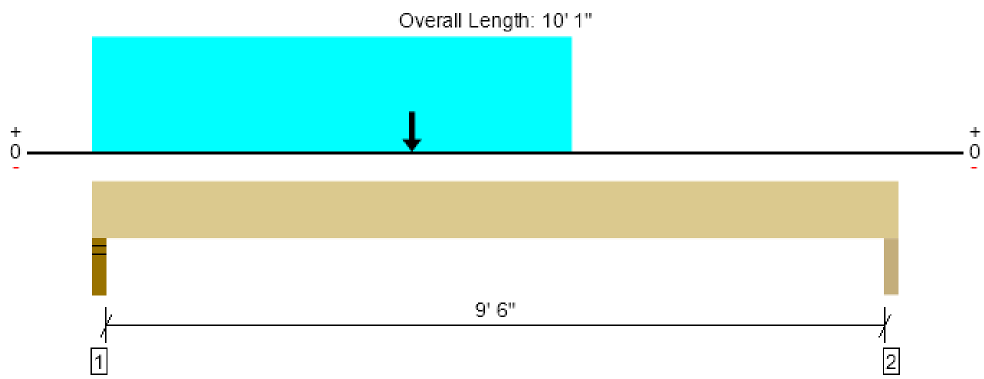
FORTEWEB

MEMBER REPORT

PASSED

Roof, BM311

1 piece(s) 8 x 10 DF No.1



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3341 @ 2"	16406 (3.50")	Passed (20%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2844 @ 1' 1"	9286	Passed (31%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	9148 @ 4'	12974	Passed (71%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.108 @ 4' 9 1/4"	0.325	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.156 @ 4' 9 5/16"	0.488	Passed (L/750)	--	1.0 D + 1.0 S (All Spans)

Member Length : 10' 1"
System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - DF	3.50"	3.50"	1.50"	1012	2329	3341	None
2 - Beam - DF	3.50"	3.50"	1.50"	566	1185	1751	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	10' 1" o/c	
Bottom Edge (Lu)	10' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 10' 1"	N/A	18.0	--	
1 - Uniform (PSF)	0 to 6' (Front)	8'	15.0	40.0	Default Load
2 - Point (lb)	4' (Front)	N/A	676	1594	Linked from: BM310, Support 2

• Side loads are assumed to not induce cross-grain tension.

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Brian Lyon Alliance Consulting Engineers (435) 755-5121 alliancelogan@yahoo.com	

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3/29/2025 9:10:35 PM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: Jess Anderson

COLUMNS

Wood Column Allowable Loads, kips

Project Name:
Job Number:
Location:

Governing Code: *2021 IBC*
Load Duration Factor: *1.0*
Eccentricity *0"*

	Member							
Height	***(2)2X4***	***(3)2X4***	***(4)2X4***	***(5)2X4***	***(2)2X6***	***(3)2X6***	***(4)2X6***	***(5)2X6***
7.625	3787	5680	7574	9467	10263	15394	20526	25657
8.625	3084	4626	6168	7710	9222	13834	18445	23056
9.625	2541	3812	5083	6354	8153	12229	16305	20381

Note: Braced in x axis

	Member			
Height	***4X4***	***4X6***	***6X6***	***8X8***
8	6810	19217	24795	69915
9	5568	17148	22955	67633
10	4612	15095	20918	64892

Note: Unbraced

Wood Column

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: KING STUD L=14' TRIB=9.5'

Maximum Reactions

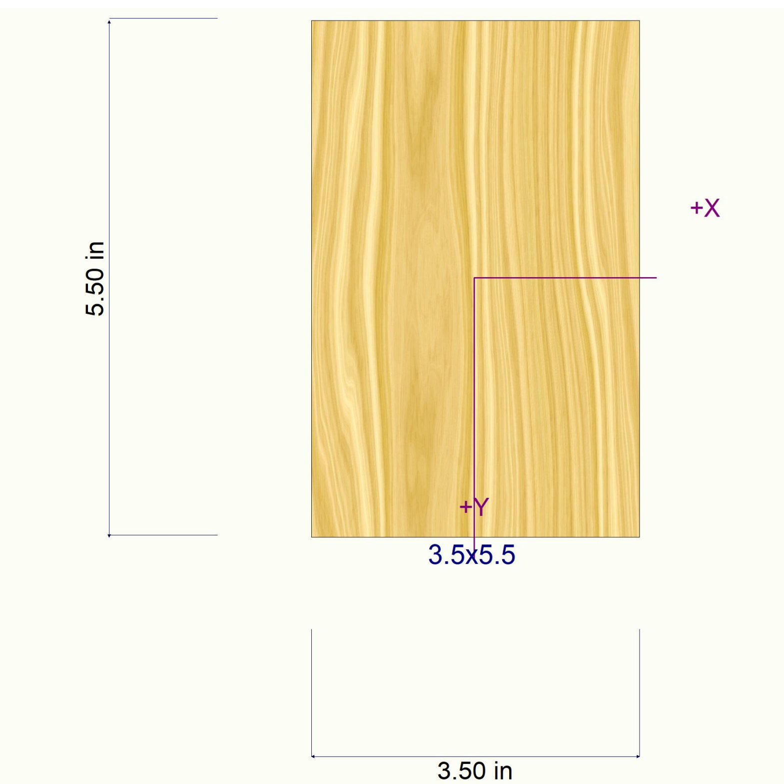
Note: Only non-zero reactions are listed.

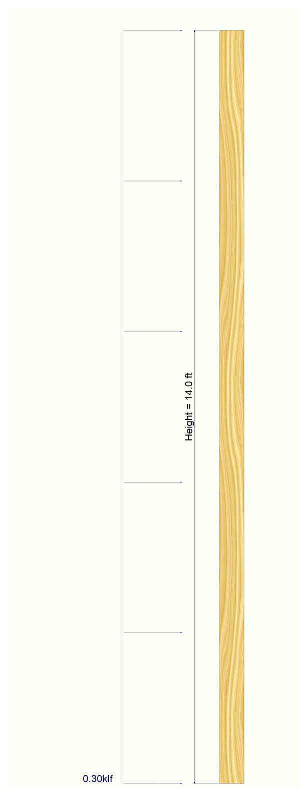
Load Combination	X-X Axis Reaction @ Base	k @ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments @ Base	k-ft @ Top	Mx - End Moments @ Base	@ Top
+D+0.450W			0.945	0.945	0.084				
+0.60D+0.60W			1.260	1.260	0.051				
+0.60D					0.051				
W Only			2.100	2.100					

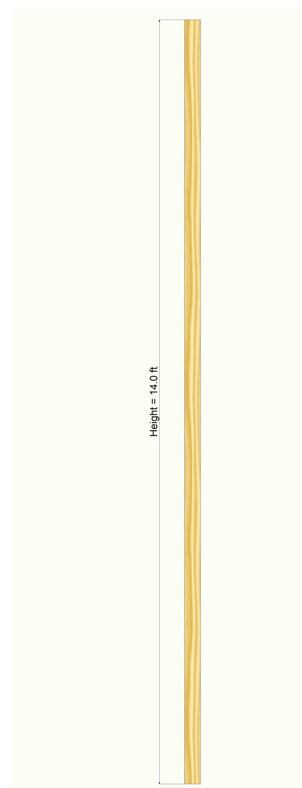
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	2.091 in	7.047 ft
+D+0.450W	0.0000 in	0.000 ft	1.568 in	7.047 ft
+0.60D+0.60W	0.0000 in	0.000 ft	2.091 in	7.047 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	3.485 in	7.047 ft

Sketches







Project Title:
Engineer:
Project ID:
Project Descr:

Wood Column

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING (c) ENERCALC INC 1983-2023

DESCRIPTION: KING STUD L=11' TRIB=4'

Code References

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

General Information

Analysis Method	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	14 ft		
(Used for non-slender calculations)			
Wood Species	Douglas Fir-Larch		
Wood Grade	No.2		
Fb +	900 psi	Fv	180 psi
Fb -	900 psi	Ft	575 psi
Fc - Prll	1350 psi	Density	31.21 pcf
Fc - Perp	625 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1600	1600	1600 ksi
Minimum	580	580	

Wood Section Name	**2-2x6**
Wood Grading/Manuf.	Graded Lumber
Wood Member Type	Sawn
Exact Width	**3.0** in
Exact Depth	**5.50** in
Area	16.50 in^2
Ix	41.594 in^4
Iy	12.375 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.30
Cf or Cv for Compressioı	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Fact	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0
Use Cr : Repetitive ?	No

Column Buckling Condition:
ABOUT X-X Axis: Lux = 14 ft, Kx = 1.0
Fully braced against buckling ABOUT Y-Y Axis

Applied Loads

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

Column self weight included : 50.066 lbs * Dead Load Factor
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, W = 0.120 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.7726 : 1**
Load Combination	+D+0.60W
Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base	6.953 ft
At maximum location values are .	
Applied Axial	0.05007 k
Applied Mx	1.764 k-ft
Applied My	0.0 k-ft
Fc : Allowable	485.990 psi
PASS Maximum Shear Stress Ratio =	**0.1591 : 1**
Load Combination	+D+0.60W
Location of max.above base	0.0 ft
Applied Design Shear	68.727 psi
Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.840 k	Bottom along Y-Y	0.840 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 1.575 in at 7.047 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.346	0.006566	PASS	0.0 ft	0.0	PASS	14.0 ft
+D+0.60W	1.600	0.205	0.7726	PASS	6.953 ft	0.1591	PASS	0.0 ft
+D+0.450W	1.600	0.205	0.5795	PASS	7.047 ft	0.1193	PASS	0.0 ft
+0.60D+0.60W	1.600	0.205	0.7708	PASS	6.953 ft	0.1591	PASS	0.0 ft
+0.60D	1.600	0.205	0.003746	PASS	0.0 ft	0.0	PASS	14.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only					0.050				
+D+0.60W			0.504	0.504	0.050				

Wood Column

LIC# : KW-06017593, Build:20.23.10.02   ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: KING STUD L=11' TRIB=4'

Maximum Reactions

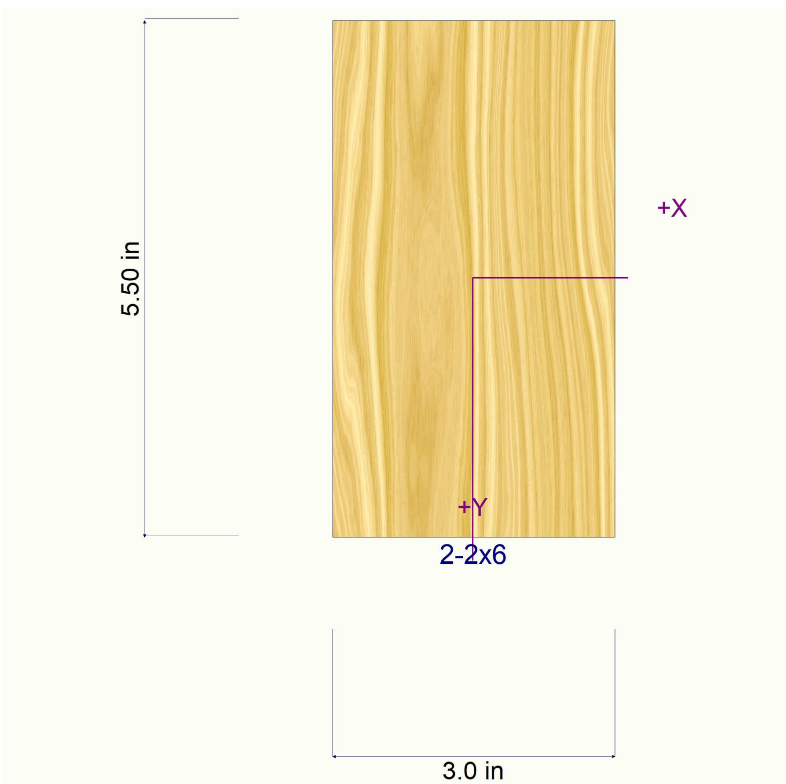
Note: Only non-zero reactions are listed.

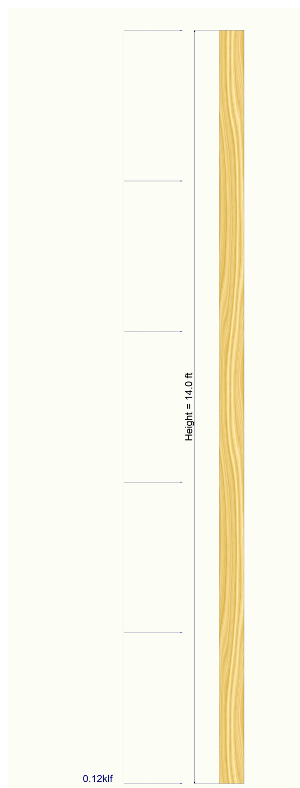
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top (k)	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top (k-ft)	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.450W			0.378	0.378	0.050				
+0.60D+0.60W			0.504	0.504	0.030				
+0.60D					0.030				
W Only			0.840	0.840					

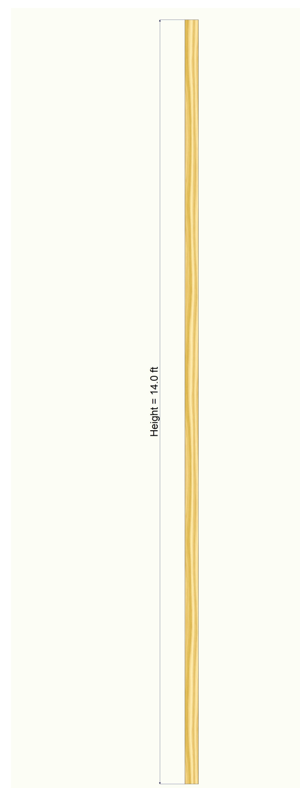
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.945 in	7.047 ft
+D+0.450W	0.0000 in	0.000 ft	0.709 in	7.047 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.945 in	7.047 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	1.575 in	7.047 ft

Sketches







Wood Column

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: KING STUD L=13' TRIB=10'

Code References

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

General Information

Analysis Method	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height		13 ft	

(Used for non-slender calculations)

Wood Species	iLevel Truss Joist		
Wood Grade	TimberStrand LSL 1.55E		
Fb +	2325 psi	Fv	310 psi
Fb -	2325 psi	Ft	1070 psi
Fc - Prll	2050 psi	Density	45.01 pcf
Fc - Perp	800 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1550	1550	1550 ksi
Minimum	787.815	787.815	

Wood Section Name	**3.5x5.5**
Wood Grading/Manuf.	Trus-Joist
Wood Member Type	TimberStrand LSL
Exact Width	**3.50** in
Exact Depth	**5.50** in
Area	19.250 in^2
Ix	48.526 in^4
Iy	19.651 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.074
Cf or Cv for Compressio	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Fact	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0
Use Cr : Repetitive ?	No

Column Buckling Condition:
ABOUT X-X Axis: Lux = 13 ft, Kx = 1.0
Fully braced against buckling ABOUT Y-Y Axis

Applied Loads

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

Column self weight included : 78.221 lbs * Dead Load Factor
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, W = 0.30 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.6724 : 1**
	Load Combination	+D+0.60W
	Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
	Location of max.above base	6.544 ft
	At maximum location values are .	
	Applied Axial	0.07822 k
	Applied Mx	3.802 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	780.58 psi
PASS	Maximum Shear Stress Ratio =	**0.1838 : 1**
	Load Combination	+D+0.60W
	Location of max.above base	13.0 ft
	Applied Design Shear	136.753 psi
	Allowable Shear	496.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.950 k	Bottom along Y-Y	1.950 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 2.591 in at 6.544 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.408	0.005396	PASS	0.0 ft	0.0	PASS	13.0 ft
+D+0.60W	1.600	0.238	0.6724	PASS	6.544 ft	0.1838	PASS	13.0 ft
+D+0.450W	1.600	0.238	0.5043	PASS	6.544 ft	0.1379	PASS	13.0 ft
+0.60D+0.60W	1.600	0.238	0.6710	PASS	6.544 ft	0.1838	PASS	13.0 ft
+0.60D	1.600	0.238	0.003123	PASS	0.0 ft	0.0	PASS	13.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only					0.078				
+D+0.60W			1.170	1.170	0.078				

Wood Column

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: KING STUD L=13' TRIB=10'

Maximum Reactions

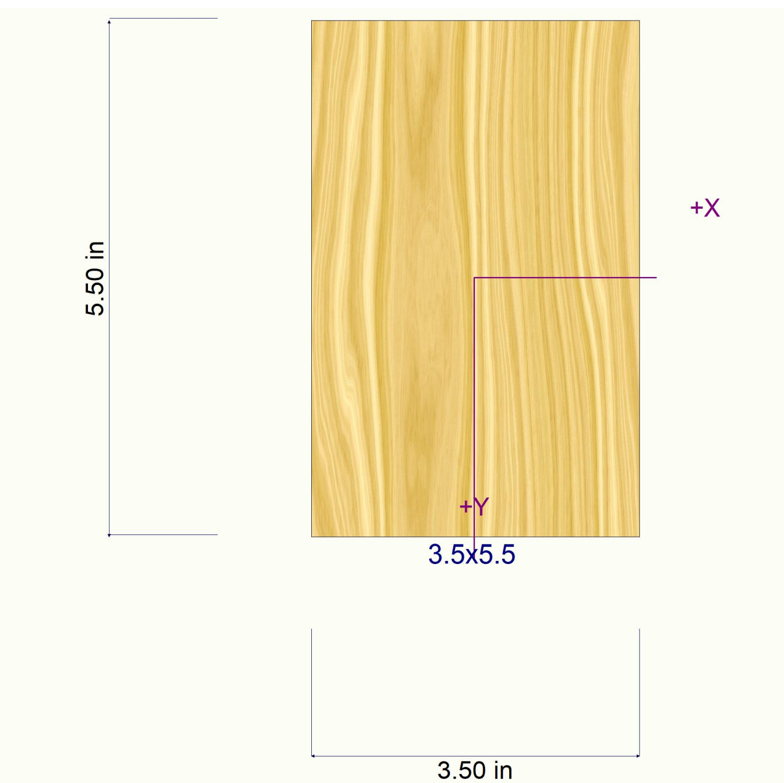
Note: Only non-zero reactions are listed.

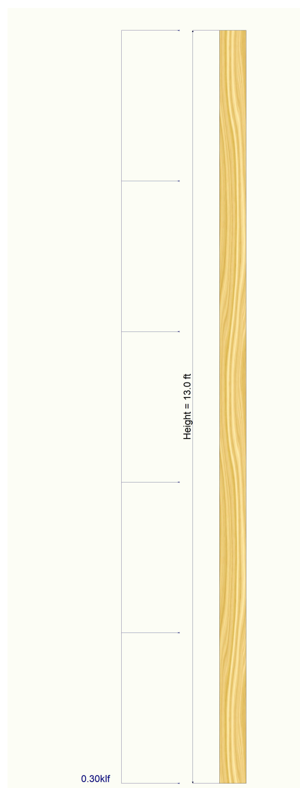
Load Combination	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	My - End Moments (k-ft) @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.450W			0.878	0.878	0.078				
+0.60D+0.60W			1.170	1.170	0.047				
+0.60D					0.047				
W Only			1.950	1.950					

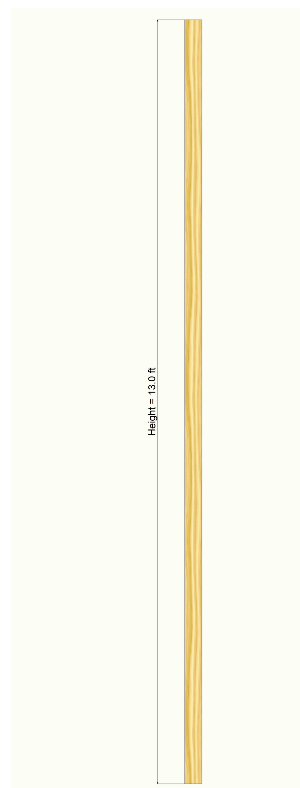
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000ft	1.554 in	6.544 ft
+D+0.450W	0.0000 in	0.000ft	1.166 in	6.544 ft
+0.60D+0.60W	0.0000 in	0.000ft	1.554 in	6.544 ft
+0.60D	0.0000 in	0.000ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000ft	2.591 in	6.544 ft

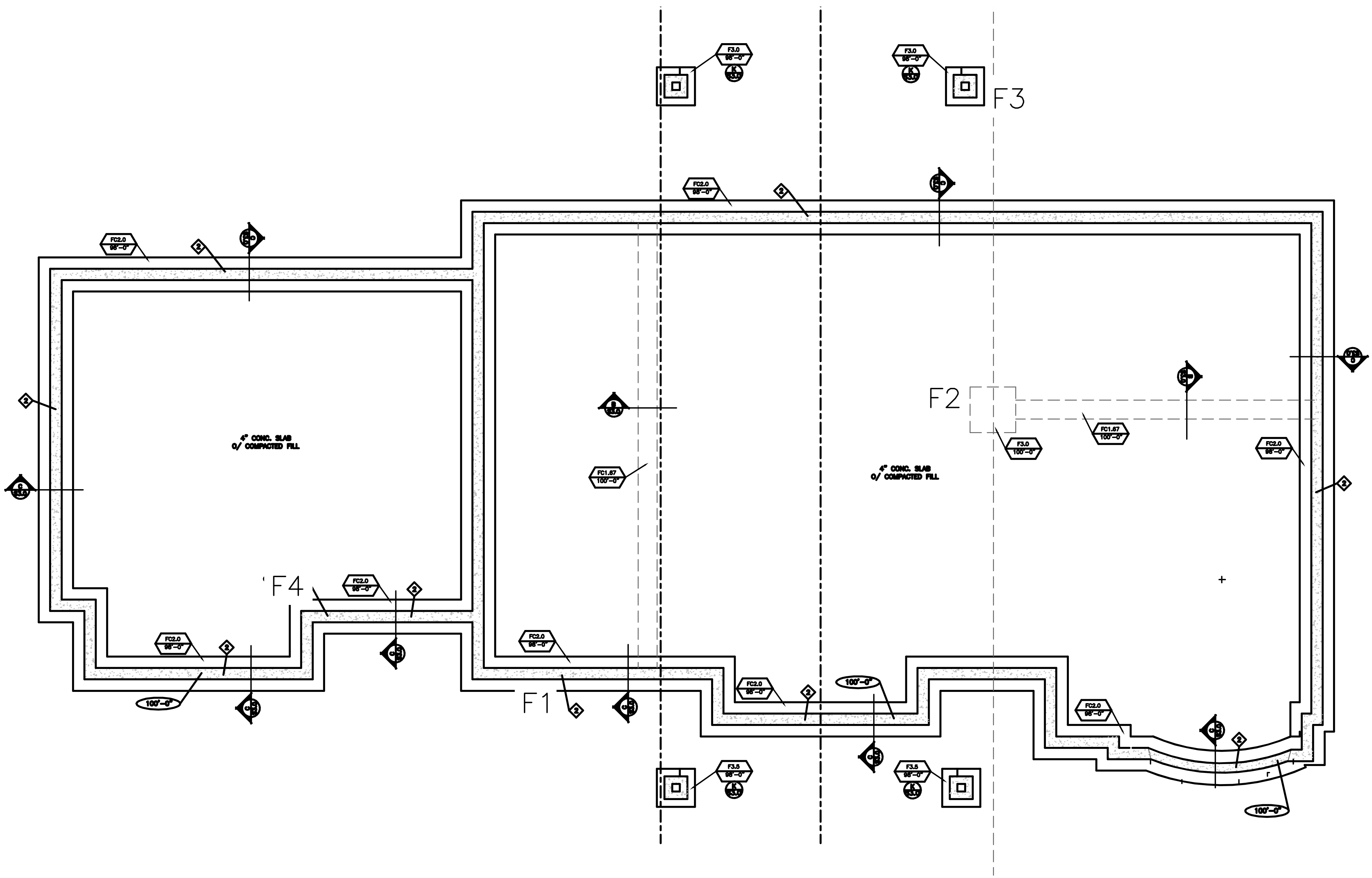
Sketches







FOOTINGS


F1
F2
F3
F4
4" CONC. SLAB
O/ COMPACTED FILL
4" CONC. SLAB
O/ COMPACTED FILL
FC2.0
98'-0"
FC1.67
100'-0"
F3.0
100'-0"
F3.0
98'-0"
F3.5
98'-0"
100'-0"

Concrete Slender Wall

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: ICF WALL (#4 @ 16" c.c. Vert./#4 @ 16" c.c. Hor.)

Code References

Calculations per ACI 318-19 Sec 11.8, IBC 2021, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

f'c : Concrete 28 day streng	=	3.0	ksi
Fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modu	=	3,122.0	ksi
λ : Lt Wt Conc Fact	=	1.0	
Fr : Rupture Modulus	=	273.861	psi
Max Allow As/bd	=	0.01355	
Max Pu/Ag = f'c *	=	0.060	
Concrete Density	=	144.0	pcf
Width of Design Strip	=	12.0	in

Wall Thickness		6.0	in
Rebar at wall center			
Rebar "d" distance		3.0	in
Lower Level Rebar . . .			
Bar Size	#	4	
Bar Spacing		16.0	in

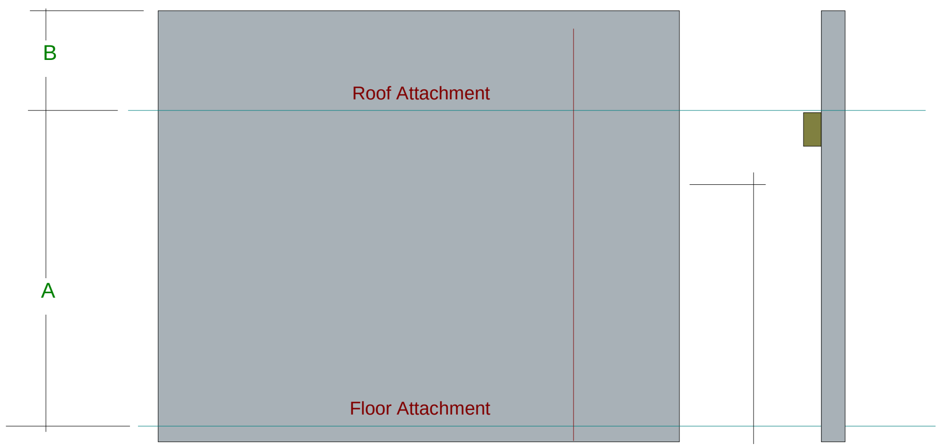
Temp Diff across thickness	=		deg F
Min Allow Out-of-Plane Defl Ratio	=	0.0	
Min allow As/bd	=	0.0020	

Using Stiff. Reduction Factor per ACI 318-14 Section 11.8.3

One-Story Wall Dimensions

A	Clear Height	=	10.0	ft
B	Parapet height	=		ft

Wall Support Condition Top & Bottom Pinned



Vertical Loads

Vertical Uniform Loads . . . *(Applied per foot of Strip Width*			DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind	
Ledger Load	Eccentricity	3.0 in	0.390			1.10		k/ft
Concentric Load								k/ft

Lateral Loads

Wind Loads :

Full area WIND load 28.0 psf

Seismic Loads :

Wall Weight Seismic Load Input Method : ASCE seismic factors entered

SDS Value per ASCE 12.11.1 S_{DS} * I = 0.7565

Fp = Wall Wt. * 0.3026 = 21.787 psf

Concrete Slender Wall

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING (c) ENERCALC INC 1983-2023

DESCRIPTION: ICF WALL (#4 @ 16" c.c. Vert./#4 @ 16" c.c. Hor.)

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +1.20D+1.60S	Maximum Bending Stress Rat	**0.2892**		
		Max Mu	0.5570 k-ft	Phi * Mn	1.926 k-ft
PASS	Service Deflection Check W Only	Actual Defl. Ratio L/	9,655	Allowable Defl. Ratio	150.0
		Max. Deflection	0.01243 in		
PASS	Axial Load Check +1.20D+1.60S+0.50W	Max Pu / Ag	32.544 psi	Max. Allow. Defl.	0.80 in
		Location	8.833 ft	0.06 * f'c	180.0 psi
PASS	Reinforcing Limit Check OK per ACI 318 Section 22.2	Actual As/bd	0.004167	Max Allow As/bd	0.01355

Maximum Reactions	*for Load Combination...*	
Top Horizontal	W Only	0.140 k
Base Horizontal	W Only	0.140 k
Vertical Reaction	+D+S	2.210 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Axial Load 0.06*f'c*b*t k	Mcr k-ft	Mu k-ft	Phi	Phi Mn k-ft	As in^2	As Ratio	0.6 * rho bal	Bar 'd'
+1.40D at 9.67 to 10.00	0.000	12.960	1.64	0.14	0.90	1.93	0.150	0.0042	0.0135	3.00
+1.20D at 9.67 to 10.00	0.000	12.960	1.64	0.12	0.90	1.93	0.150	0.0042	0.0135	3.00
+1.20D+0.50S at 9.67 to 10.00	0.000	12.960	1.64	0.25	0.90	1.93	0.150	0.0042	0.0135	3.00
+1.20D+0.50W at 5.67 to 6.00	0.842	12.960	1.64	0.24	0.90	2.10	0.150	0.0042	0.0135	3.00
+1.20D+1.60S at 9.67 to 10.00	0.000	12.960	1.64	0.56	0.90	1.93	0.150	0.0042	0.0135	3.00
+1.20D+1.60S+0.50W at 8.67 to 9.00	2.343	12.960	1.64	0.57	0.90	2.39	0.150	0.0042	0.0135	3.00
+1.20D+W at 5.33 to 5.67	0.871	12.960	1.64	0.41	0.90	2.10	0.150	0.0042	0.0135	3.00
+1.20D+0.50S+W at 5.67 to 6.00	1.392	12.960	1.64	0.49	0.90	2.21	0.150	0.0042	0.0135	3.00
+1.20D+0.70S+E at 6.33 to 6.67	1.555	12.960	1.64	0.45	0.90	2.24	0.150	0.0042	0.0135	3.00
+0.90D+W at 5.00 to 5.33	0.675	12.960	1.64	0.40	0.90	2.06	0.150	0.0042	0.0135	3.00
+0.90D+E at 5.33 to 5.67	0.653	12.960	1.64	0.32	0.90	2.06	0.150	0.0042	0.0135	3.00

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values Mcr k-ft	Moment Values Mactual k-ft	Stiffness I gross in^4	Stiffness I cracked in^4	Stiffness I effective in^4	Deflections Deflection in	Deflections Defl. Ratio
D Only at 5.67 to 6.00	0.702	1.64	0.06	216.00	10.01	162.000	0.002	56,123.3
+D+S at 5.67 to 6.00	1.802	1.64	0.22	216.00	10.96	162.000	0.008	14,644.2
+D+0.750S at 5.67 to 6.00	1.527	1.64	0.18	216.00	10.72	162.000	0.007	17,972.7
+D+0.60W at 5.00 to 5.33	0.750	1.64	0.26	216.00	10.06	162.000	0.010	12,528.5
+D+0.70E at 5.00 to 5.33	0.750	1.64	0.24	216.00	10.06	162.000	0.009	13,499.7
+D+0.450W at 5.00 to 5.33	0.750	1.64	0.21	216.00	10.06	162.000	0.008	15,564.6
+D+0.750S+0.450W at 5.33 to 5.67	1.551	1.64	0.32	216.00	10.74	162.000	0.012	9,823.8
+D+0.750S+0.5250E at 5.33 to 5.67	1.551	1.64	0.31	216.00	10.74	162.000	0.012	10,254.0
+0.60D+0.60W at 5.00 to 5.33	0.450	1.64	0.24	216.00	9.80	162.000	0.009	13,747.7
+0.60D+0.70E at 5.00 to 5.33	0.450	1.64	0.22	216.00	9.80	162.000	0.008	14,924.9
S Only at 5.67 to 6.00	1.100	1.64	0.16	216.00	10.36	162.000	0.006	19,875.6
W Only at 5.00 to 5.33	0.000	1.64	0.35	216.00	9.40	162.000	0.012	9,655.0
E Only at 5.00 to 5.33	0.000	1.64	0.27	216.00	9.40	162.000	0.010	12,408.3

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.01 k	1.110 k

Concrete Slender Wall

Project File: Jess Anderson.ec6

DESCRIPTION: ICF WALL (#4 @ 16" c.c. Vert./#4 @ 16" c.c. Hor.)

+D+S	0.0 k	0.04 k	2.210 k
+D+0.750S	0.0 k	0.03 k	1.935 k
+D+0.60W	0.1 k	0.07 k	1.110 k
+D+0.70E	0.1 k	0.07 k	1.110 k
+D+0.450W	0.1 k	0.05 k	1.110 k
+D+0.750S+0.450W	0.1 k	0.03 k	1.935 k
+D+0.750S+0.5250E	0.1 k	0.03 k	1.935 k
+0.60D+0.60W	0.1 k	0.08 k	0.666 k
+0.60D+0.70E	0.1 k	0.07 k	0.666 k
S Only	0.0 k	0.03 k	1.100 k
W Only	0.1 k	0.14 k	0.000 k
E Only	0.1 k	0.11 k	0.000 k

Concrete Slender Wall

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING (c) ENERCALC INC 1983-2023

DESCRIPTION: COLUMN GIRDER B

Code References

Calculations per ACI 318-19 Sec 11.8, IBC 2021, ASCE 7-16

Load Combinations Used :

General Information

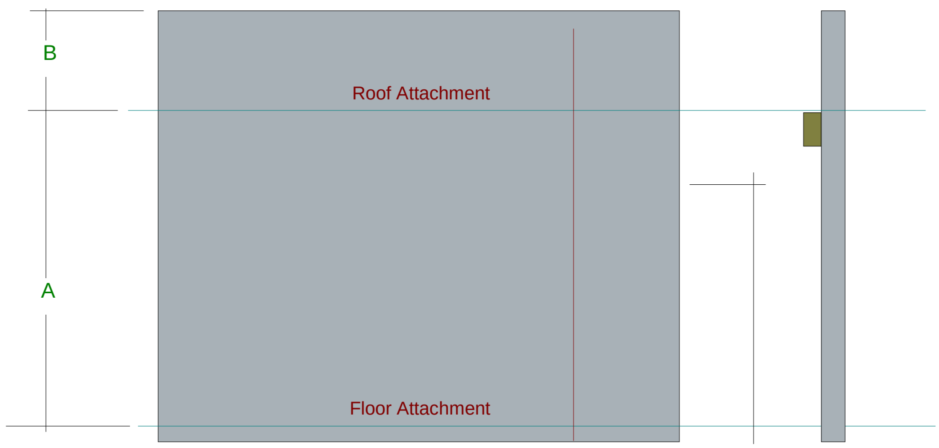
f'c : Concrete 28 day streng	=	3.0 ksi	Wall Thickness	6.0 in	Temp Diff across thickness	=	deg F
Fy : Rebar Yield	=	60.0 ksi	Rebar at wall center		Min Allow Out-of-Plane Defl Ratio =		0.0
Ec : Concrete Elastic Modu	=	3,122.0 ksi	Rebar "d" distance	3.0 in			
λ : Lt Wt Conc Fact	=	1.0	Lower Level Rebar . . .		Min allow As/bd	=	0.0020
Fr : Rupture Modulus	=	273.861 psi	Bar Size #	4	Using Stiff. Reduction Factor per ACI 318-14		
Max Allow As/bd	=	0.01355	Bar Spacing	16 in			Section 11.8.3
Max Pu/Ag = f'c *	=	0.060					
Concrete Density	=	144.0 pcf					
Width of Design Strip	=	20 in					

One-Story Wall Dimensions

A Clear Height = 10 ft

B Parapet height = ft

Wall Support Condition Top & Bottom Pinned



Vertical Loads

Vertical Concentrated Loads . . . *(Applied to full "Strip Width")*

			DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind
Beam Load #1	Eccentricity	2 in	4.407			10.848	k
	Dist. from Base	ft					

Lateral Loads

Wind Loads :

Full area WIND load 15.0 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

Concrete Slender Wall

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: COLUMN GIRDER B

DESIGN SUMMARY

Results reported for "Strip Width" of 20.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +D+S	Maximum Bending Stress Rat **0.0** Max Mu	0.0 k-ft	Phi * Mn	0.0 k-ft
PASS	Service Deflection Check +D+S	Actual Defl. Ratio L/ Max. Deflection	3,618 -0.03317 in	Allowable Defl. Ratio	150.0
PASS	Axial Load Check 0.0	Max Pu / Ag Location	0.0 psi 0.0 ft	Max. Allow. Defl. 0.06 * f'c	0.80 in 180.0 psi
PASS	Reinforcing Limit Check OK per ACI 318 Section 22.2	Actual As/bd	0.0	Max Allow As/bd	0.0

Maximum Reactions	*for Load Combination...*	
Top Horizontal	+D+S	253.802 k
Base Horizontal	+D+0.750S+0.5250E	318.181 k
Vertical Reaction	+D+S	16.455 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu (k)	Axial Load 0.06*f'c*b*t (k)	Mcr (k-ft)	Mu (k-ft)	Phi	Moment Values Phi Mn (k-ft)	As (in^2)	As Ratio	0.6 * rho bal	Bar 'd'

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu (k)	Moment Values Mcr (k-ft)	Moment Values Mactual (k-ft)	Stiffness I gross (in^4)	Stiffness I cracked (in^4)	Stiffness I effective (in^4)	Deflections Deflection (in)	Deflections Defl. Ratio
D Only at 4.00 to 4.33	0.432	1.64	0.26	216.00	9.78	162.000	0.010	12,499.5
+D+S at 4.00 to 4.33	0.432	1.64	0.89	216.00	9.78	162.000	0.033	3,617.7
+D+0.750S at 4.00 to 4.33	0.432	1.64	0.73	216.00	9.78	162.000	0.027	4,397.8
+D+0.60W at 3.67 to 4.00	0.456	1.64	0.17	216.00	9.80	162.000	0.006	20,738.4
+D+0.70E at 2.33 to 2.67	0.552	1.64	0.17	216.00	9.89	162.000	0.003	45,907.2
+D+0.450W at 3.67 to 4.00	0.456	1.64	0.19	216.00	9.80	162.000	0.007	17,851.3
+D+0.750S+0.450W at 4.00 to 4.33	0.432	1.64	0.65	216.00	9.78	162.000	0.024	4,920.7
+D+0.750S+0.5250E at 4.00 to 4.33	0.432	1.64	0.57	216.00	9.78	162.000	0.022	5,543.1
+0.60D+0.60W at 2.67 to 3.00	0.317	1.64	0.10	216.00	9.68	162.000	0.002	57,783.3
+0.60D+0.70E at 6.00 to 6.33	0.173	1.64	0.11	216.00	9.55	162.000	0.002	50,504.1
S Only at 4.00 to 4.33	0.000	1.64	0.63	216.00	9.40	162.000	0.024	5,089.4
W Only at 5.00 to 5.33	0.000	1.64	0.19	216.00	9.40	162.000	0.007	18,022.8
E Only at 5.00 to 5.33	0.000	1.64	0.31	216.00	9.40	162.000	0.011	10,813.7

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.1 k	0.07 k	5.607 k
+D+S	0.3 k	0.25 k	16.455 k
+D+0.750S	0.2 k	0.21 k	13.743 k
+D+0.60W	0.1 k	0.00 k	5.607 k
+D+0.70E	0.2 k	0.07 k	5.607 k
+D+0.450W	0.1 k	0.02 k	5.607 k
+D+0.750S+0.450W	0.3 k	0.15 k	13.743 k
+D+0.750S+0.5250E	0.3 k	0.10 k	13.743 k
+0.60D+0.60W	0.1 k	0.03 k	3.364 k
+0.60D+0.70E	0.2 k	0.10 k	3.364 k
S Only	0.2 k	0.18 k	10.848 k

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING (c) ENERCALC INC 1983-2023

DESCRIPTION: BM313

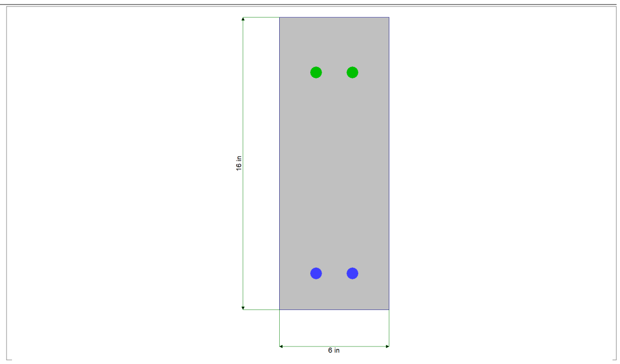
CODE REFERENCES

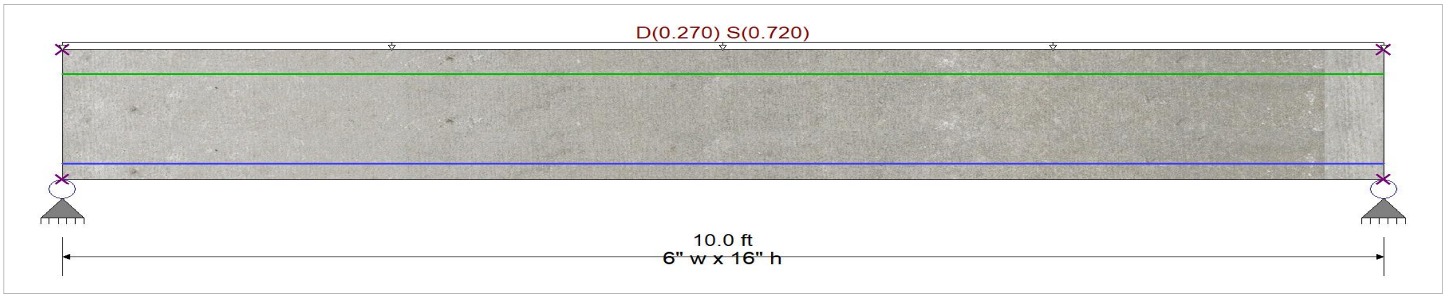
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

General Information

f'c =	3.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50 =	410.792 psi		Shear :	0.750
ψ Density =	145.0 pcf	β_1 =		0.850
λ LtWt Factor =	1.0			
Elastic Modulus =	3,122.0 ksi	Fy - Stirrups		40.0 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

Seismic Design Category = A





Cross Section & Reinforcing Details

Rectangular Section, Width = 6.0 in, Height = 16.0 in
Span #1 Reinforcing....
2-#5 at 2.0 in from Bottom, from 0.0 to 10.0 ft in this span 2-#5 at 3.0 in from Top, from 0.0 to 10.0 ft in this span

Loads on all spans...
D = 0.0150, S = 0.040
Uniform Load on ALL spans : D = 0.0150, S = 0.040 ksf, Tributary Width = 18.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.517** : 1
Section used for this span	**Typical Section**
Mu : Applied	18.450 k-ft
Mn * Phi : Allowable	35.711 k-ft
Location of maximum on span	4.991 ft
Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.026 in	Ratio =	4659	>=360.0	S Only
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0	S Only
Max Downward Total Deflection	0.057 in	Ratio =	2090	>=180.0	Span: 1 : +D+S
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0	Span: 1 : +D+S

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	4.950	4.950
Max Upward from Load Combinations	4.950	4.950
Max Upward from Load Cases	3.600	3.600
D Only	1.350	1.350
+D+S	4.950	4.950
+D+0.750S	4.050	4.050
+0.60D	0.810	0.810

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: BM313

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
S Only	3.600	3.600

Shear Stirrup Requirements

Between 0.00 to 2.66 ft, Phi*lambda*sqrt(f'c)*bw*d < Vu <= Phi*Vc, Req'd Vs = Min per 9.6.3.1, use #3 stirrups spaced at 7.000 in
Between 2.68 to 7.32 ft, Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.
Between 7.34 to 9.98 ft, Phi*Vc < Vu, Req'd Vs = 0.4787, use #3 stirrups spaced at 7.000 in

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu Actual	(k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+1.60S	1	0.00	14.00	7.38	7.38	0.00	1.00	6.90	Phi*Vc < Vu	0.4787	20.1	7.0
+1.20D+1.60S	1	0.11	14.00	7.22	7.22	0.80	1.00	6.90	Phi*Vc < Vu	0.3174	20.1	7.0
+1.20D+1.60S	1	0.22	14.00	7.06	7.06	1.58	1.00	6.90	Phi*Vc < Vu	0.1561	20.1	7.0
+1.20D+1.60S	1	0.33	14.00	6.90	6.90	2.34	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.44	14.00	6.73	6.73	3.09	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.55	14.00	6.57	6.57	3.81	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.66	14.00	6.41	6.41	4.52	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.77	14.00	6.25	6.25	5.21	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.87	14.00	6.09	6.09	5.89	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.98	14.00	5.93	5.93	6.55	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.09	14.00	5.77	5.77	7.18	0.94	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.20	14.00	5.61	5.61	7.81	0.84	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.31	14.00	5.44	5.44	8.41	0.76	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.42	14.00	5.28	5.28	9.00	0.69	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.53	14.00	5.12	5.12	9.56	0.62	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.64	14.00	4.96	4.96	10.12	0.57	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.75	14.00	4.80	4.80	10.65	0.53	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.86	14.00	4.64	4.64	11.16	0.48	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.97	14.00	4.48	4.48	11.66	0.45	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.08	14.00	4.32	4.32	12.14	0.41	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.19	14.00	4.15	4.15	12.61	0.38	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.30	14.00	3.99	3.99	13.05	0.36	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.40	14.00	3.83	3.83	13.48	0.33	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.51	14.00	3.67	3.67	13.89	0.31	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.62	14.00	3.51	3.51	14.28	0.29	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.73	14.00	3.35	3.35	14.65	0.27	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	2.84	14.00	3.19	3.19	15.01	0.25	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	2.95	14.00	3.02	3.02	15.35	0.23	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	3.06	14.00	2.86	2.86	15.67	0.21	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	3.17	14.00	2.70	2.70	15.98	0.20	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	3.28	14.00	2.54	2.54	16.26	0.18	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	3.39	14.00	2.38	2.38	16.53	0.17	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	3.50	14.00	2.22	2.22	16.78	0.15	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	3.61	14.00	2.06	2.06	17.02	0.14	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	3.72	14.00	1.90	1.90	17.23	0.13	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	3.83	14.00	1.73	1.73	17.43	0.12	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	3.93	14.00	1.57	1.57	17.61	0.10	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	4.04	14.00	1.41	1.41	17.78	0.09	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	4.15	14.00	1.25	1.25	17.92	0.08	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	4.26	14.00	1.09	1.09	18.05	0.07	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	4.37	14.00	0.93	0.93	18.16	0.06	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	4.48	14.00	0.77	0.77	18.25	0.05	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	4.59	14.00	0.60	0.60	18.33	0.04	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	4.70	14.00	0.44	0.44	18.38	0.03	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	4.81	14.00	0.28	0.28	18.42	0.02	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	4.92	14.00	0.12	0.12	18.45	0.01	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	5.03	14.00	-0.04	0.04	18.45	0.00	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	5.14	14.00	-0.20	0.20	18.44	0.01	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	5.25	14.00	-0.36	0.36	18.41	0.02	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0
+1.20D+1.60S	1	5.36	14.00	-0.52	0.52	18.36	0.03	4.91	Vu <= Phi*lambd	ot Reqd pe	4.9	0.0

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: BM313

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+1.60S	1	5.46	14.00	-0.69	0.69	18.29	0.04	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	5.57	14.00	-0.85	0.85	18.21	0.05	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	5.68	14.00	-1.01	1.01	18.11	0.06	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	5.79	14.00	-1.17	1.17	17.99	0.08	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	5.90	14.00	-1.33	1.33	17.85	0.09	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	6.01	14.00	-1.49	1.49	17.70	0.10	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	6.12	14.00	-1.65	1.65	17.52	0.11	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	6.23	14.00	-1.81	1.81	17.33	0.12	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	6.34	14.00	-1.98	1.98	17.13	0.13	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	6.45	14.00	-2.14	2.14	16.90	0.15	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	6.56	14.00	-2.30	2.30	16.66	0.16	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	6.67	14.00	-2.46	2.46	16.40	0.17	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	6.78	14.00	-2.62	2.62	16.12	0.19	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	6.89	14.00	-2.78	2.78	15.83	0.21	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	6.99	14.00	-2.94	2.94	15.51	0.22	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	7.10	14.00	-3.11	3.11	15.18	0.24	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	7.21	14.00	-3.27	3.27	14.84	0.26	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	7.32	14.00	-3.43	3.43	14.47	0.28	4.91	Vu <= Phi*lambd	t Reqd pe	4.9	0.0
+1.20D+1.60S	1	7.43	14.00	-3.59	3.59	14.09	0.30	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	7.54	14.00	-3.75	3.75	13.69	0.32	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	7.65	14.00	-3.91	3.91	13.27	0.34	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	7.76	14.00	-4.07	4.07	12.83	0.37	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	7.87	14.00	-4.23	4.23	12.38	0.40	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	7.98	14.00	-4.40	4.40	11.90	0.43	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	8.09	14.00	-4.56	4.56	11.42	0.47	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	8.20	14.00	-4.72	4.72	10.91	0.50	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	8.31	14.00	-4.88	4.88	10.38	0.55	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	8.42	14.00	-5.04	5.04	9.84	0.60	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	8.52	14.00	-5.20	5.20	9.28	0.65	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	8.63	14.00	-5.36	5.36	8.70	0.72	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	8.74	14.00	-5.52	5.52	8.11	0.79	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	8.85	14.00	-5.69	5.69	7.50	0.88	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	8.96	14.00	-5.85	5.85	6.87	0.99	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	9.07	14.00	-6.01	6.01	6.22	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	9.18	14.00	-6.17	6.17	5.55	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	9.29	14.00	-6.33	6.33	4.87	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	9.40	14.00	-6.49	6.49	4.17	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	9.51	14.00	-6.65	6.65	3.45	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	9.62	14.00	-6.82	6.82	2.71	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	9.73	14.00	-6.98	6.98	1.96	1.00	6.90	Phi*Vc < Vu	0.07542	20.1	7.0
+1.20D+1.60S	1	9.84	14.00	-7.14	7.14	1.19	1.00	6.90	Phi*Vc < Vu	0.2367	20.1	7.0
+1.20D+1.60S	1	9.95	14.00	-7.30	7.30	0.40	1.00	6.90	Phi*Vc < Vu	0.3980	20.1	7.0

Maximum Forces & Stresses for Load Combinations

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	10.000	18.45	35.71	0.52
+1.40D					
Span # 1	1	10.000	4.72	35.71	0.13
+1.20D					
Span # 1	1	10.000	4.05	35.71	0.11
+1.20D+0.50S					
Span # 1	1	10.000	8.55	35.71	0.24
+1.20D+1.60S					
Span # 1	1	10.000	18.45	35.71	0.52
+1.20D+0.70S					
Span # 1	1	10.000	10.35	35.71	0.29
+0.90D					
Span # 1	1	10.000	3.04	35.71	0.09

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING (c) ENERCALC INC 1983-2023

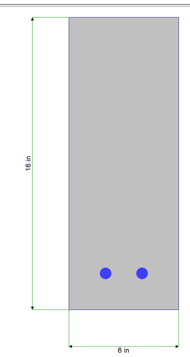
DESCRIPTION: BM314

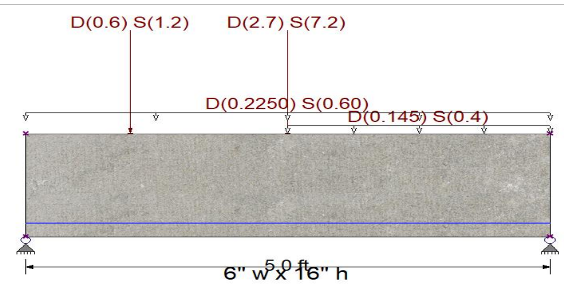
CODE REFERENCES

Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

General Information

f'c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	410.792 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 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
Seismic Design Category =		A			





Cross Section & Reinforcing Details

Rectangular Section, Width = 6.0 in, Height = 16.0 in
Span #1 Reinforcing....
2-#5 at 2.0 in from Bottom, from 0.0 to 5.0 ft in this span

Loads on all spans...

Partial Length Uniform Load : D = 0.0150, S = 0.040 ksf, Extent = 0.0 -->> 0.0 ft, Tributary Width = 15.0 ft

Point Load : D = 2.70, S = 7.20 k @ 2.50 ft
Point Load : D = 0.60, S = 1.20 k @ 1.0 ft
Uniform Load : D = 0.1450, S = 0.40 k/ft, Extent = 2.50 -->> 5.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.589** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	21.008	k-ft
Mn * Phi : Allowable	35.668	k-ft
Location of maximum on span	2.495	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.007 in	Ratio =	9196	>=360.0	S Only
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0	S Only
Max Downward Total Deflection	0.014 in	Ratio =	4375	>=180.0	Span: 1 : +D+S
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0	Span: 1 : +D+S

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	6.731	6.332
Max Upward from Load Combinations	6.731	6.332
Max Upward from Load Cases	4.810	4.590

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: BM314

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
D Only	1.921	1.742
+D+S	6.731	6.332
+D+0.750S	5.528	5.184
+0.60D	1.152	1.045
S Only	4.810	4.590

Shear Stirrup Requirements

Entire Beam Span Length : Phi*Vc < Vu, Req'd Vs = 3.099, use #3 stirrups spaced at 7.000 in

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+1.60S	1	0.00	14.00	10.00	10.00	0.00	0.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.05	14.00	10.00	10.00	0.55	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.11	14.00	10.00	10.00	1.09	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.16	14.00	10.00	10.00	1.64	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.22	14.00	10.00	10.00	2.19	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.27	14.00	10.00	10.00	2.73	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.33	14.00	10.00	10.00	3.28	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.38	14.00	10.00	10.00	3.83	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.44	14.00	10.00	10.00	4.37	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.49	14.00	10.00	10.00	4.92	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.55	14.00	10.00	10.00	5.46	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.60	14.00	10.00	10.00	6.01	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.66	14.00	10.00	10.00	6.56	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.71	14.00	10.00	10.00	7.10	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.77	14.00	10.00	10.00	7.65	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.82	14.00	10.00	10.00	8.20	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.87	14.00	10.00	10.00	8.74	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.93	14.00	10.00	10.00	9.29	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	0.98	14.00	10.00	10.00	9.84	1.00	6.90	Phi*Vc < Vu	3.099	20.1	7.0
+1.20D+1.60S	1	1.04	14.00	7.36	7.36	10.28	0.84	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.09	14.00	7.36	7.36	10.68	0.80	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.15	14.00	7.36	7.36	11.09	0.77	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.20	14.00	7.36	7.36	11.49	0.75	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.26	14.00	7.36	7.36	11.89	0.72	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.31	14.00	7.36	7.36	12.29	0.70	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.37	14.00	7.36	7.36	12.70	0.68	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.42	14.00	7.36	7.36	13.10	0.66	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.48	14.00	7.36	7.36	13.50	0.64	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.53	14.00	7.36	7.36	13.90	0.62	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.58	14.00	7.36	7.36	14.30	0.60	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.64	14.00	7.36	7.36	14.71	0.58	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.69	14.00	7.36	7.36	15.11	0.57	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.75	14.00	7.36	7.36	15.51	0.55	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.80	14.00	7.36	7.36	15.91	0.54	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.86	14.00	7.36	7.36	16.32	0.53	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.91	14.00	7.36	7.36	16.72	0.51	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	1.97	14.00	7.36	7.36	17.12	0.50	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	2.02	14.00	7.36	7.36	17.52	0.49	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	2.08	14.00	7.36	7.36	17.92	0.48	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	2.13	14.00	7.36	7.36	18.33	0.47	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	2.19	14.00	7.36	7.36	18.73	0.46	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	2.24	14.00	7.36	7.36	19.13	0.45	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	2.30	14.00	7.36	7.36	19.53	0.44	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	2.35	14.00	7.36	7.36	19.94	0.43	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	2.40	14.00	7.36	7.36	20.34	0.42	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	2.46	14.00	7.36	7.36	20.74	0.41	6.90	Phi*Vc < Vu	0.4594	20.1	7.0
+1.20D+1.60S	1	2.51	14.00	-7.41	7.41	20.94	0.41	6.90	Phi*Vc < Vu	0.5091	20.1	7.0
+1.20D+1.60S	1	2.57	14.00	-7.45	7.45	20.53	0.42	6.90	Phi*Vc < Vu	0.5535	20.1	7.0

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: BM314

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu Actual	(k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+1.60S	1	2.62	14.00	-7.50	7.50	20.13	0.43	6.90	Phi*Vc < Vu	0.5980	20.1	7.0
+1.20D+1.60S	1	2.68	14.00	-7.54	7.54	19.71	0.45	6.90	Phi*Vc < Vu	0.6425	20.1	7.0
+1.20D+1.60S	1	2.73	14.00	-7.59	7.59	19.30	0.46	6.90	Phi*Vc < Vu	0.6870	20.1	7.0
+1.20D+1.60S	1	2.79	14.00	-7.63	7.63	18.89	0.47	6.90	Phi*Vc < Vu	0.7315	20.1	7.0
+1.20D+1.60S	1	2.84	14.00	-7.68	7.68	18.47	0.49	6.90	Phi*Vc < Vu	0.7760	20.1	7.0
+1.20D+1.60S	1	2.90	14.00	-7.72	7.72	18.05	0.50	6.90	Phi*Vc < Vu	0.8204	20.1	7.0
+1.20D+1.60S	1	2.95	14.00	-7.77	7.77	17.62	0.51	6.90	Phi*Vc < Vu	0.8649	20.1	7.0
+1.20D+1.60S	1	3.01	14.00	-7.81	7.81	17.20	0.53	6.90	Phi*Vc < Vu	0.9094	20.1	7.0
+1.20D+1.60S	1	3.06	14.00	-7.86	7.86	16.77	0.55	6.90	Phi*Vc < Vu	0.9539	20.1	7.0
+1.20D+1.60S	1	3.11	14.00	-7.90	7.90	16.34	0.56	6.90	Phi*Vc < Vu	0.9984	20.1	7.0
+1.20D+1.60S	1	3.17	14.00	-7.94	7.94	15.91	0.58	6.90	Phi*Vc < Vu	1.043	20.1	7.0
+1.20D+1.60S	1	3.22	14.00	-7.99	7.99	15.47	0.60	6.90	Phi*Vc < Vu	1.087	20.1	7.0
+1.20D+1.60S	1	3.28	14.00	-8.03	8.03	15.03	0.62	6.90	Phi*Vc < Vu	1.132	20.1	7.0
+1.20D+1.60S	1	3.33	14.00	-8.08	8.08	14.59	0.65	6.90	Phi*Vc < Vu	1.176	20.1	7.0
+1.20D+1.60S	1	3.39	14.00	-8.12	8.12	14.15	0.67	6.90	Phi*Vc < Vu	1.221	20.1	7.0
+1.20D+1.60S	1	3.44	14.00	-8.17	8.17	13.71	0.70	6.90	Phi*Vc < Vu	1.265	20.1	7.0
+1.20D+1.60S	1	3.50	14.00	-8.21	8.21	13.26	0.72	6.90	Phi*Vc < Vu	1.310	20.1	7.0
+1.20D+1.60S	1	3.55	14.00	-8.26	8.26	12.81	0.75	6.90	Phi*Vc < Vu	1.354	20.1	7.0
+1.20D+1.60S	1	3.61	14.00	-8.30	8.30	12.36	0.78	6.90	Phi*Vc < Vu	1.399	20.1	7.0
+1.20D+1.60S	1	3.66	14.00	-8.34	8.34	11.90	0.82	6.90	Phi*Vc < Vu	1.443	20.1	7.0
+1.20D+1.60S	1	3.72	14.00	-8.39	8.39	11.44	0.86	6.90	Phi*Vc < Vu	1.488	20.1	7.0
+1.20D+1.60S	1	3.77	14.00	-8.43	8.43	10.98	0.90	6.90	Phi*Vc < Vu	1.532	20.1	7.0
+1.20D+1.60S	1	3.83	14.00	-8.48	8.48	10.52	0.94	6.90	Phi*Vc < Vu	1.577	20.1	7.0
+1.20D+1.60S	1	3.88	14.00	-8.52	8.52	10.06	0.99	6.90	Phi*Vc < Vu	1.621	20.1	7.0
+1.20D+1.60S	1	3.93	14.00	-8.57	8.57	9.59	1.00	6.90	Phi*Vc < Vu	1.666	20.1	7.0
+1.20D+1.60S	1	3.99	14.00	-8.61	8.61	9.12	1.00	6.90	Phi*Vc < Vu	1.710	20.1	7.0
+1.20D+1.60S	1	4.04	14.00	-8.66	8.66	8.65	1.00	6.90	Phi*Vc < Vu	1.755	20.1	7.0
+1.20D+1.60S	1	4.10	14.00	-8.70	8.70	8.18	1.00	6.90	Phi*Vc < Vu	1.799	20.1	7.0
+1.20D+1.60S	1	4.15	14.00	-8.74	8.74	7.70	1.00	6.90	Phi*Vc < Vu	1.843	20.1	7.0
+1.20D+1.60S	1	4.21	14.00	-8.79	8.79	7.22	1.00	6.90	Phi*Vc < Vu	1.888	20.1	7.0
+1.20D+1.60S	1	4.26	14.00	-8.83	8.83	6.74	1.00	6.90	Phi*Vc < Vu	1.932	20.1	7.0
+1.20D+1.60S	1	4.32	14.00	-8.88	8.88	6.25	1.00	6.90	Phi*Vc < Vu	1.977	20.1	7.0
+1.20D+1.60S	1	4.37	14.00	-8.92	8.92	5.77	1.00	6.90	Phi*Vc < Vu	2.021	20.1	7.0
+1.20D+1.60S	1	4.43	14.00	-8.97	8.97	5.28	1.00	6.90	Phi*Vc < Vu	2.066	20.1	7.0
+1.20D+1.60S	1	4.48	14.00	-9.01	9.01	4.79	1.00	6.90	Phi*Vc < Vu	2.110	20.1	7.0
+1.20D+1.60S	1	4.54	14.00	-9.06	9.06	4.29	1.00	6.90	Phi*Vc < Vu	2.155	20.1	7.0
+1.20D+1.60S	1	4.59	14.00	-9.10	9.10	3.80	1.00	6.90	Phi*Vc < Vu	2.199	20.1	7.0
+1.20D+1.60S	1	4.64	14.00	-9.15	9.15	3.30	1.00	6.90	Phi*Vc < Vu	2.244	20.1	7.0
+1.20D+1.60S	1	4.70	14.00	-9.19	9.19	2.80	1.00	6.90	Phi*Vc < Vu	2.288	20.1	7.0
+1.20D+1.60S	1	4.75	14.00	-9.23	9.23	2.30	1.00	6.90	Phi*Vc < Vu	2.333	20.1	7.0
+1.20D+1.60S	1	4.81	14.00	-9.28	9.28	1.79	1.00	6.90	Phi*Vc < Vu	2.377	20.1	7.0
+1.20D+1.60S	1	4.86	14.00	-9.32	9.32	1.28	1.00	6.90	Phi*Vc < Vu	2.422	20.1	7.0
+1.20D+1.60S	1	4.92	14.00	-9.37	9.37	0.77	1.00	6.90	Phi*Vc < Vu	2.466	20.1	7.0
+1.20D+1.60S	1	4.97	14.00	-9.41	9.41	0.26	1.00	6.90	Phi*Vc < Vu	2.511	20.1	7.0

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	5.000	21.01	35.67	0.59
+1.40D					
Span # 1	1	5.000	5.45	35.67	0.15
+1.20D					
Span # 1	1	5.000	4.67	35.67	0.13
+1.20D+0.50S					
Span # 1	1	5.000	9.78	35.67	0.27
+1.20D+1.60S					
Span # 1	1	5.000	21.01	35.67	0.59
+1.20D+0.70S					
Span # 1	1	5.000	11.82	35.67	0.33

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: BM315

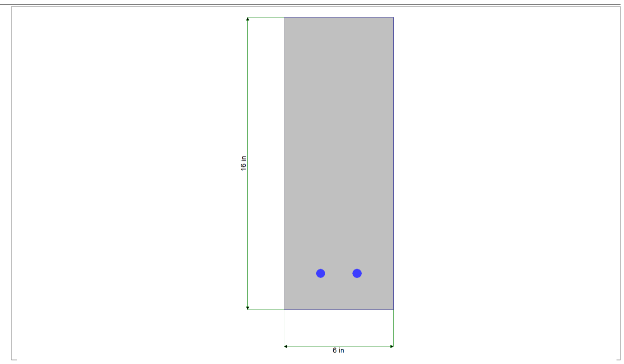
CODE REFERENCES

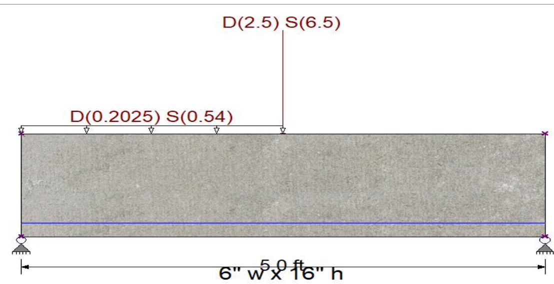
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

General Information

f'c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2} * 7.50$	=	410.792 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 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

Seismic Design Category = A





Cross Section & Reinforcing Details

Rectangular Section, Width = 6.0 in, Height = 16.0 in
Span #1 Reinforcing....
2-#4 at 2.0 in from Bottom, from 0.0 to 5.0 ft in this span

Point Load : D = 2.50, S = 6.50 k @ 2.50 ft
Uniform Load : D = 0.0150, S = 0.040 ksf, Extent = 0.0 -->> 2.50 ft, Tributary Width = 13.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.776** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	18.452	k-ft
Mn * Phi : Allowable	23.788	k-ft
Location of maximum on span	2.495	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.005 in	Ratio =	11599	>=360.0	S Only
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0	S Only
Max Downward Total Deflection	0.011 in	Ratio =	5576	>=180.0	Span: 1 : +D+S
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0	Span: 1 : +D+S

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	5.892	4.964
Max Upward from Load Combinations	5.892	4.964
Max Upward from Load Cases	4.262	3.587
D Only	1.630	1.377
+D+S	5.892	4.964
+D+0.750S	4.827	4.067
+0.60D	0.978	0.826
S Only	4.262	3.587

Concrete Beam

Project File: Jess Anderson.ec6

DESCRIPTION: BM315

Shear Stirrup Requirements

Entire Beam Span Length : Phi*Vc < Vu, Req'd Vs = 1.874, use #3 stirrups spaced at 7.000 in

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+1.60S	1	0.00	14.00	8.78	8.78	0.00	1.00	6.90	Phi*Vc < Vu	1.874	20.1	7.0
+1.20D+1.60S	1	0.05	14.00	8.72	8.72	0.48	1.00	6.90	Phi*Vc < Vu	1.814	20.1	7.0
+1.20D+1.60S	1	0.11	14.00	8.65	8.65	0.95	1.00	6.90	Phi*Vc < Vu	1.753	20.1	7.0
+1.20D+1.60S	1	0.16	14.00	8.59	8.59	1.42	1.00	6.90	Phi*Vc < Vu	1.693	20.1	7.0
+1.20D+1.60S	1	0.22	14.00	8.53	8.53	1.89	1.00	6.90	Phi*Vc < Vu	1.632	20.1	7.0
+1.20D+1.60S	1	0.27	14.00	8.47	8.47	2.36	1.00	6.90	Phi*Vc < Vu	1.572	20.1	7.0
+1.20D+1.60S	1	0.33	14.00	8.41	8.41	2.82	1.00	6.90	Phi*Vc < Vu	1.511	20.1	7.0
+1.20D+1.60S	1	0.38	14.00	8.35	8.35	3.28	1.00	6.90	Phi*Vc < Vu	1.451	20.1	7.0
+1.20D+1.60S	1	0.44	14.00	8.29	8.29	3.73	1.00	6.90	Phi*Vc < Vu	1.390	20.1	7.0
+1.20D+1.60S	1	0.49	14.00	8.23	8.23	4.18	1.00	6.90	Phi*Vc < Vu	1.330	20.1	7.0
+1.20D+1.60S	1	0.55	14.00	8.17	8.17	4.63	1.00	6.90	Phi*Vc < Vu	1.269	20.1	7.0
+1.20D+1.60S	1	0.60	14.00	8.11	8.11	5.07	1.00	6.90	Phi*Vc < Vu	1.209	20.1	7.0
+1.20D+1.60S	1	0.66	14.00	8.05	8.05	5.52	1.00	6.90	Phi*Vc < Vu	1.148	20.1	7.0
+1.20D+1.60S	1	0.71	14.00	7.99	7.99	5.95	1.00	6.90	Phi*Vc < Vu	1.088	20.1	7.0
+1.20D+1.60S	1	0.77	14.00	7.93	7.93	6.39	1.00	6.90	Phi*Vc < Vu	1.027	20.1	7.0
+1.20D+1.60S	1	0.82	14.00	7.87	7.87	6.82	1.00	6.90	Phi*Vc < Vu	0.9669	20.1	7.0
+1.20D+1.60S	1	0.87	14.00	7.81	7.81	7.25	1.00	6.90	Phi*Vc < Vu	0.9065	20.1	7.0
+1.20D+1.60S	1	0.93	14.00	7.75	7.75	7.67	1.00	6.90	Phi*Vc < Vu	0.8460	20.1	7.0
+1.20D+1.60S	1	0.98	14.00	7.69	7.69	8.10	1.00	6.90	Phi*Vc < Vu	0.7855	20.1	7.0
+1.20D+1.60S	1	1.04	14.00	7.63	7.63	8.51	1.00	6.90	Phi*Vc < Vu	0.7250	20.1	7.0
+1.20D+1.60S	1	1.09	14.00	7.57	7.57	8.93	0.99	6.90	Phi*Vc < Vu	0.6645	20.1	7.0
+1.20D+1.60S	1	1.15	14.00	7.51	7.51	9.34	0.94	6.90	Phi*Vc < Vu	0.6040	20.1	7.0
+1.20D+1.60S	1	1.20	14.00	7.44	7.44	9.75	0.89	6.90	Phi*Vc < Vu	0.5435	20.1	7.0
+1.20D+1.60S	1	1.26	14.00	7.38	7.38	10.16	0.85	6.90	Phi*Vc < Vu	0.4830	20.1	7.0
+1.20D+1.60S	1	1.31	14.00	7.32	7.32	10.56	0.81	6.90	Phi*Vc < Vu	0.4225	20.1	7.0
+1.20D+1.60S	1	1.37	14.00	7.26	7.26	10.96	0.77	6.90	Phi*Vc < Vu	0.3620	20.1	7.0
+1.20D+1.60S	1	1.42	14.00	7.20	7.20	11.35	0.74	6.90	Phi*Vc < Vu	0.3015	20.1	7.0
+1.20D+1.60S	1	1.48	14.00	7.14	7.14	11.74	0.71	6.90	Phi*Vc < Vu	0.2410	20.1	7.0
+1.20D+1.60S	1	1.53	14.00	7.08	7.08	12.13	0.68	6.90	Phi*Vc < Vu	0.1806	20.1	7.0
+1.20D+1.60S	1	1.58	14.00	7.02	7.02	12.52	0.65	6.90	Phi*Vc < Vu	0.1201	20.1	7.0
+1.20D+1.60S	1	1.64	14.00	6.96	6.96	12.90	0.63	6.90	Phi*Vc < Vu	0.05957	20.1	7.0
+1.20D+1.60S	1	1.69	14.00	6.90	6.90	13.28	0.61	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.75	14.00	6.84	6.84	13.65	0.58	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.80	14.00	6.78	6.78	14.03	0.56	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.86	14.00	6.72	6.72	14.39	0.54	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.91	14.00	6.66	6.66	14.76	0.53	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	1.97	14.00	6.60	6.60	15.12	0.51	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.02	14.00	6.54	6.54	15.48	0.49	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.08	14.00	6.48	6.48	15.84	0.48	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.13	14.00	6.42	6.42	16.19	0.46	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.19	14.00	6.36	6.36	16.54	0.45	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.24	14.00	6.30	6.30	16.88	0.44	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.30	14.00	6.23	6.23	17.23	0.42	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.35	14.00	6.17	6.17	17.56	0.41	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.40	14.00	6.11	6.11	17.90	0.40	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.46	14.00	6.05	6.05	18.23	0.39	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	2.51	14.00	-7.39	7.39	18.38	0.47	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	2.57	14.00	-7.39	7.39	17.97	0.48	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	2.62	14.00	-7.39	7.39	17.57	0.49	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	2.68	14.00	-7.39	7.39	17.17	0.50	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	2.73	14.00	-7.39	7.39	16.76	0.51	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	2.79	14.00	-7.39	7.39	16.36	0.53	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	2.84	14.00	-7.39	7.39	15.96	0.54	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	2.90	14.00	-7.39	7.39	15.55	0.55	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	2.95	14.00	-7.39	7.39	15.15	0.57	6.90	Phi*Vc < Vu	0.4906	20.1	7.0

Concrete Beam

Project File: Jess Anderson.ec6

DESCRIPTION: BM315

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu Actual	(k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+1.60S	1	3.01	14.00	-7.39	7.39	14.74	0.58	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.06	14.00	-7.39	7.39	14.34	0.60	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.11	14.00	-7.39	7.39	13.94	0.62	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.17	14.00	-7.39	7.39	13.53	0.64	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.22	14.00	-7.39	7.39	13.13	0.66	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.28	14.00	-7.39	7.39	12.72	0.68	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.33	14.00	-7.39	7.39	12.32	0.70	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.39	14.00	-7.39	7.39	11.92	0.72	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.44	14.00	-7.39	7.39	11.51	0.75	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.50	14.00	-7.39	7.39	11.11	0.78	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.55	14.00	-7.39	7.39	10.70	0.81	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.61	14.00	-7.39	7.39	10.30	0.84	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.66	14.00	-7.39	7.39	9.90	0.87	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.72	14.00	-7.39	7.39	9.49	0.91	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.77	14.00	-7.39	7.39	9.09	0.95	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.83	14.00	-7.39	7.39	8.68	0.99	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.88	14.00	-7.39	7.39	8.28	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.93	14.00	-7.39	7.39	7.88	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	3.99	14.00	-7.39	7.39	7.47	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.04	14.00	-7.39	7.39	7.07	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.10	14.00	-7.39	7.39	6.66	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.15	14.00	-7.39	7.39	6.26	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.21	14.00	-7.39	7.39	5.86	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.26	14.00	-7.39	7.39	5.45	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.32	14.00	-7.39	7.39	5.05	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.37	14.00	-7.39	7.39	4.65	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.43	14.00	-7.39	7.39	4.24	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.48	14.00	-7.39	7.39	3.84	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.54	14.00	-7.39	7.39	3.43	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.59	14.00	-7.39	7.39	3.03	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.64	14.00	-7.39	7.39	2.63	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.70	14.00	-7.39	7.39	2.22	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.75	14.00	-7.39	7.39	1.82	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.81	14.00	-7.39	7.39	1.41	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.86	14.00	-7.39	7.39	1.01	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.92	14.00	-7.39	7.39	0.61	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0
+1.20D+1.60S	1	4.97	14.00	-7.39	7.39	0.20	1.00	6.90	Phi*Vc < Vu	0.4906	20.1	7.0

Maximum Forces & Stresses for Load Combinations

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	5.000	18.45	23.79	0.78
+1.40D					
Span # 1	1	5.000	4.81	23.79	0.20
+1.20D					
Span # 1	1	5.000	4.12	23.79	0.17
+1.20D+0.50S					
Span # 1	1	5.000	8.60	23.79	0.36
+1.20D+1.60S					
Span # 1	1	5.000	18.45	23.79	0.78
+1.20D+0.70S					
Span # 1	1	5.000	10.39	23.79	0.44
+0.90D					
Span # 1	1	5.000	3.09	23.79	0.13

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	.ocation in Span (ft	Load Combination	Vlax. "+" Defl (in	.ocation in Span (ft
+D+S	1	0.0108	2.500		0.0000	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: BM316

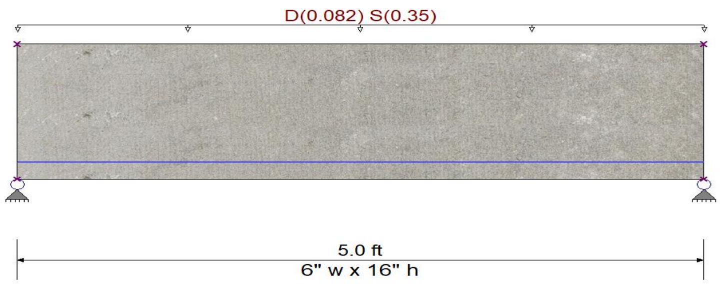
CODE REFERENCES

Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

General Information

f'c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	410.792 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 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

Seismic Design Category = A



Cross Section & Reinforcing Details

Rectangular Section, Width = 6.0 in, Height = 16.0 in
Span #1 Reinforcing....
2-#4 at 2.0 in from Bottom, from 0.0 to 5.0 ft in this span

Load for Span Number 1
Uniform Load : D = 0.0820, S = 0.350 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.086** : 1
Section used for this span **Typical Section**
Mu : Applied 2.057 k-ft
Mn * Phi : Allowable 23.788 k-ft
Location of maximum on span 2.495 ft
Span # where maximum occurs Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360.0	S Only
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0	S Only
Max Downward Total Deflection	0.000 in	Ratio =	0 <180.0	Span: 1 : +D+S
Max Upward Total Deflection	0.000 in	Ratio =	0 <180.0	Span: 1 : +D+S

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.080	1.080
Max Upward from Load Combinations	1.080	1.080
Max Upward from Load Cases	0.875	0.875
D Only	0.205	0.205
+D+S	1.080	1.080
+D+0.750S	0.861	0.861
+0.60D	0.123	0.123
S Only	0.875	0.875

DESCRIPTION: BM316

Shear Stirrup Requirements

Entire Beam Span Length : Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+1.60S	1	0.00	14.00	1.65	1.65	0.00	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.05	14.00	1.61	1.61	0.09	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.11	14.00	1.57	1.57	0.18	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.16	14.00	1.54	1.54	0.26	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.22	14.00	1.50	1.50	0.34	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.27	14.00	1.47	1.47	0.43	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.33	14.00	1.43	1.43	0.50	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.38	14.00	1.39	1.39	0.58	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.44	14.00	1.36	1.36	0.66	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.49	14.00	1.32	1.32	0.73	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.55	14.00	1.29	1.29	0.80	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.60	14.00	1.25	1.25	0.87	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.66	14.00	1.21	1.21	0.94	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.71	14.00	1.18	1.18	1.00	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.77	14.00	1.14	1.14	1.07	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.82	14.00	1.11	1.11	1.13	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.87	14.00	1.07	1.07	1.19	1.00	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.93	14.00	1.03	1.03	1.24	0.97	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	0.98	14.00	1.00	1.00	1.30	0.90	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.04	14.00	0.96	0.96	1.35	0.83	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.09	14.00	0.93	0.93	1.41	0.77	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.15	14.00	0.89	0.89	1.46	0.71	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.20	14.00	0.85	0.85	1.50	0.66	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.26	14.00	0.82	0.82	1.55	0.62	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.31	14.00	0.78	0.78	1.59	0.57	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.37	14.00	0.75	0.75	1.63	0.53	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.42	14.00	0.71	0.71	1.67	0.50	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.48	14.00	0.67	0.67	1.71	0.46	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.53	14.00	0.64	0.64	1.75	0.43	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.58	14.00	0.60	0.60	1.78	0.39	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.64	14.00	0.57	0.57	1.81	0.36	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.69	14.00	0.53	0.53	1.84	0.34	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.75	14.00	0.49	0.49	1.87	0.31	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.80	14.00	0.46	0.46	1.90	0.28	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.86	14.00	0.42	0.42	1.92	0.26	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.91	14.00	0.39	0.39	1.94	0.23	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	1.97	14.00	0.35	0.35	1.96	0.21	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.02	14.00	0.31	0.31	1.98	0.19	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.08	14.00	0.28	0.28	2.00	0.16	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.13	14.00	0.24	0.24	2.01	0.14	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.19	14.00	0.21	0.21	2.02	0.12	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.24	14.00	0.17	0.17	2.04	0.10	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.30	14.00	0.13	0.13	2.04	0.08	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.35	14.00	0.10	0.10	2.05	0.06	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.40	14.00	0.06	0.06	2.05	0.04	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.46	14.00	0.03	0.03	2.06	0.02	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.51	14.00	-0.01	0.01	2.06	0.01	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.57	14.00	-0.04	0.04	2.06	0.03	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.62	14.00	-0.08	0.08	2.05	0.05	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.68	14.00	-0.12	0.12	2.05	0.07	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.73	14.00	-0.15	0.15	2.04	0.09	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.79	14.00	-0.19	0.19	2.03	0.11	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.84	14.00	-0.22	0.22	2.02	0.13	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.90	14.00	-0.26	0.26	2.01	0.15	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0
+1.20D+1.60S	1	2.95	14.00	-0.30	0.30	1.99	0.17	4.24	Vu <= Phi*lambdɔt Reqd pe		4.2	0.0

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: BM316

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu Actual	(k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+1.60S	1	3.01	14.00	-0.33	0.33	1.97	0.20	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.06	14.00	-0.37	0.37	1.95	0.22	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.11	14.00	-0.40	0.40	1.93	0.24	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.17	14.00	-0.44	0.44	1.91	0.27	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.22	14.00	-0.48	0.48	1.88	0.30	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.28	14.00	-0.51	0.51	1.86	0.32	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.33	14.00	-0.55	0.55	1.83	0.35	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.39	14.00	-0.58	0.58	1.80	0.38	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.44	14.00	-0.62	0.62	1.76	0.41	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.50	14.00	-0.66	0.66	1.73	0.44	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.55	14.00	-0.69	0.69	1.69	0.48	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.61	14.00	-0.73	0.73	1.65	0.51	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.66	14.00	-0.76	0.76	1.61	0.55	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.72	14.00	-0.80	0.80	1.57	0.59	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.77	14.00	-0.84	0.84	1.53	0.64	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.83	14.00	-0.87	0.87	1.48	0.69	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.88	14.00	-0.91	0.91	1.43	0.74	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.93	14.00	-0.94	0.94	1.38	0.80	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.99	14.00	-0.98	0.98	1.33	0.86	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.04	14.00	-1.02	1.02	1.27	0.93	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.10	14.00	-1.05	1.05	1.22	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.15	14.00	-1.09	1.09	1.16	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.21	14.00	-1.12	1.12	1.10	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.26	14.00	-1.16	1.16	1.04	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.32	14.00	-1.20	1.20	0.97	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.37	14.00	-1.23	1.23	0.90	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.43	14.00	-1.27	1.27	0.84	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.48	14.00	-1.30	1.30	0.77	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.54	14.00	-1.34	1.34	0.69	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.59	14.00	-1.38	1.38	0.62	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.64	14.00	-1.41	1.41	0.54	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.70	14.00	-1.45	1.45	0.46	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.75	14.00	-1.48	1.48	0.38	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.81	14.00	-1.52	1.52	0.30	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.86	14.00	-1.56	1.56	0.22	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.92	14.00	-1.59	1.59	0.13	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.97	14.00	-1.63	1.63	0.04	1.00	4.24	Vu <= Phi*lambd ot Reqd pe		4.2	0.0

Maximum Forces & Stresses for Load Combinations

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	5.000	2.06	23.79	0.09
+1.40D					
Span # 1	1	5.000	0.36	23.79	0.02
+1.20D					
Span # 1	1	5.000	0.31	23.79	0.01
+1.20D+0.50S					
Span # 1	1	5.000	0.85	23.79	0.04
+1.20D+1.60S					
Span # 1	1	5.000	2.06	23.79	0.09
+1.20D+0.70S					
Span # 1	1	5.000	1.07	23.79	0.05
+0.90D					
Span # 1	1	5.000	0.23	23.79	0.01

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	.ocation in Span (ft	Load Combination	Vlax. "+" Defl (in)	.ocation in Span (ft
+D+S	1	0.0009	2.500		0.0000	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: BM317

CODE REFERENCES

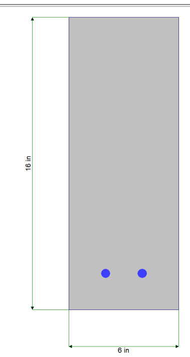
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

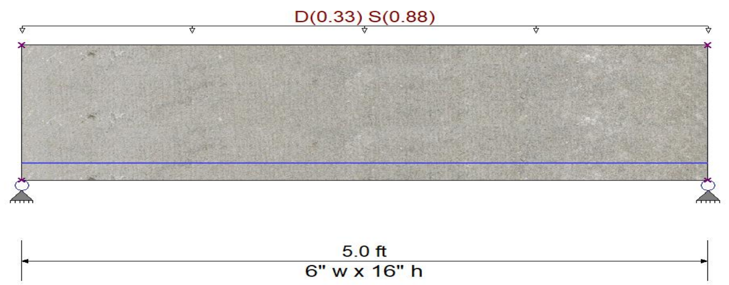
General Information

f'c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2} * 7.50$	=	410.792 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 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

Seismic Design Category = A





Cross Section & Reinforcing Details

Rectangular Section, Width = 6.0 in, Height = 16.0 in
Span #1 Reinforcing....
2-#4 at 2.0 in from Bottom, from 0.0 to 5.0 ft in this span

Load for Span Number 1
Uniform Load : D = 0.0150, S = 0.040 ksf, Tributary Width = 22.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.237**	: 1
Section used for this span	**Typical Section**	
Mu : Applied	5.637	k-ft
Mn * Phi : Allowable	23.788	k-ft
Location of maximum on span	2.495	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.002 in	Ratio =	31010	>=360.0	S Only
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0	S Only
Max Downward Total Deflection	0.003 in	Ratio =	22553	>=180.0	Span: 1 : +D+S
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0	Span: 1 : +D+S

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.025	3.025
Max Upward from Load Combinations	3.025	3.025
Max Upward from Load Cases	2.200	2.200
D Only	0.825	0.825
+D+S	3.025	3.025
+D+0.750S	2.475	2.475
+0.60D	0.495	0.495
S Only	2.200	2.200

Concrete Beam

Project File: Jess Anderson.ec6

DESCRIPTION: BM317

Shear Stirrup Requirements

Between 0.00 to 0.58 ft, Phi*lambda*sqrt(f'c)*bw*d < Vu <= Phi*Vc, Req'd Vs = Min per 9.6.3.1, use #3 stirrups spaced at 7.000 in
Between 0.59 to 4.41 ft, Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.
Between 4.42 to 4.99 ft, Phi*lambda*sqrt(f'c)*bw*d < Vu <= Phi*Vc, Req'd Vs = Min per 9.6.3.1, use #3 stirrups spaced at 7.000 in

Detailed Shear Information

	Span	Distance	'd'	Vu	(k)	Mu	d*Vu/Mu	Phi*Vc	Comment	Phi*Vs	Phi*Vn	Spacing (in)
Load Combination	Number	(ft)	(in)	Actual	Design	(k-ft)		(k)		(k)	(k)	Req'd
+1.20D+1.60S	1	0.00	14.00	4.51	4.51	0.00	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.05	14.00	4.41	4.41	0.24	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.11	14.00	4.31	4.31	0.48	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.16	14.00	4.21	4.21	0.72	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.22	14.00	4.12	4.12	0.94	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.27	14.00	4.02	4.02	1.16	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.33	14.00	3.92	3.92	1.38	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.38	14.00	3.82	3.82	1.59	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.44	14.00	3.72	3.72	1.80	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.49	14.00	3.62	3.62	2.00	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.55	14.00	3.52	3.52	2.20	1.00	6.90	Phi*lambda*sqrt	in per 9.6.	20.1	7.0
+1.20D+1.60S	1	0.60	14.00	3.43	3.43	2.39	1.00	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	0.66	14.00	3.33	3.33	2.57	1.00	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	0.71	14.00	3.23	3.23	2.75	1.00	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	0.77	14.00	3.13	3.13	2.92	1.00	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	0.82	14.00	3.03	3.03	3.09	1.00	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	0.87	14.00	2.93	2.93	3.25	1.00	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	0.93	14.00	2.83	2.83	3.41	0.97	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	0.98	14.00	2.74	2.74	3.56	0.90	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.04	14.00	2.64	2.64	3.71	0.83	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.09	14.00	2.54	2.54	3.85	0.77	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.15	14.00	2.44	2.44	3.99	0.71	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.20	14.00	2.34	2.34	4.12	0.66	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.26	14.00	2.24	2.24	4.24	0.62	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.31	14.00	2.14	2.14	4.36	0.57	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.37	14.00	2.05	2.05	4.48	0.53	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.42	14.00	1.95	1.95	4.59	0.50	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.48	14.00	1.85	1.85	4.69	0.46	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.53	14.00	1.75	1.75	4.79	0.43	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.58	14.00	1.65	1.65	4.88	0.39	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.64	14.00	1.55	1.55	4.97	0.36	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.69	14.00	1.45	1.45	5.05	0.34	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.75	14.00	1.36	1.36	5.13	0.31	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.80	14.00	1.26	1.26	5.20	0.28	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.86	14.00	1.16	1.16	5.27	0.26	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.91	14.00	1.06	1.06	5.33	0.23	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	1.97	14.00	0.96	0.96	5.38	0.21	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.02	14.00	0.86	0.86	5.43	0.19	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.08	14.00	0.76	0.76	5.48	0.16	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.13	14.00	0.67	0.67	5.51	0.14	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.19	14.00	0.57	0.57	5.55	0.12	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.24	14.00	0.47	0.47	5.58	0.10	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.30	14.00	0.37	0.37	5.60	0.08	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.35	14.00	0.27	0.27	5.62	0.06	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.40	14.00	0.17	0.17	5.63	0.04	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.46	14.00	0.07	0.07	5.64	0.02	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.51	14.00	-0.02	0.02	5.64	0.01	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.57	14.00	-0.12	0.12	5.63	0.03	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.62	14.00	-0.22	0.22	5.62	0.05	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.68	14.00	-0.32	0.32	5.61	0.07	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.73	14.00	-0.42	0.42	5.59	0.09	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.79	14.00	-0.52	0.52	5.56	0.11	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.84	14.00	-0.62	0.62	5.53	0.13	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0
+1.20D+1.60S	1	2.90	14.00	-0.71	0.71	5.50	0.15	4.24	Vu <= Phi*lambd	ot Reqd per	4.2	0.0

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: BM317

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu Actual	(k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+1.60S	1	2.95	14.00	-0.81	0.81	5.45	0.17	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.01	14.00	-0.91	0.91	5.41	0.20	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.06	14.00	-1.01	1.01	5.35	0.22	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.11	14.00	-1.11	1.11	5.30	0.24	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.17	14.00	-1.21	1.21	5.23	0.27	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.22	14.00	-1.31	1.31	5.16	0.30	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.28	14.00	-1.40	1.40	5.09	0.32	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.33	14.00	-1.50	1.50	5.01	0.35	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.39	14.00	-1.60	1.60	4.93	0.38	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.44	14.00	-1.70	1.70	4.84	0.41	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.50	14.00	-1.80	1.80	4.74	0.44	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.55	14.00	-1.90	1.90	4.64	0.48	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.61	14.00	-2.00	2.00	4.53	0.51	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.66	14.00	-2.09	2.09	4.42	0.55	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.72	14.00	-2.19	2.19	4.30	0.59	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.77	14.00	-2.29	2.29	4.18	0.64	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.83	14.00	-2.39	2.39	4.05	0.69	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.88	14.00	-2.49	2.49	3.92	0.74	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.93	14.00	-2.59	2.59	3.78	0.80	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	3.99	14.00	-2.69	2.69	3.64	0.86	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.04	14.00	-2.78	2.78	3.49	0.93	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.10	14.00	-2.88	2.88	3.33	1.00	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.15	14.00	-2.98	2.98	3.17	1.00	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.21	14.00	-3.08	3.08	3.01	1.00	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.26	14.00	-3.18	3.18	2.84	1.00	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.32	14.00	-3.28	3.28	2.66	1.00	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.37	14.00	-3.38	3.38	2.48	1.00	4.24	Vu <= Phi*lambd)t Reqd pe		4.2	0.0
+1.20D+1.60S	1	4.43	14.00	-3.47	3.47	2.29	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0
+1.20D+1.60S	1	4.48	14.00	-3.57	3.57	2.10	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0
+1.20D+1.60S	1	4.54	14.00	-3.67	3.67	1.90	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0
+1.20D+1.60S	1	4.59	14.00	-3.77	3.77	1.70	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0
+1.20D+1.60S	1	4.64	14.00	-3.87	3.87	1.49	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0
+1.20D+1.60S	1	4.70	14.00	-3.97	3.97	1.27	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0
+1.20D+1.60S	1	4.75	14.00	-4.07	4.07	1.05	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0
+1.20D+1.60S	1	4.81	14.00	-4.16	4.16	0.83	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0
+1.20D+1.60S	1	4.86	14.00	-4.26	4.26	0.60	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0
+1.20D+1.60S	1	4.92	14.00	-4.36	4.36	0.36	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0
+1.20D+1.60S	1	4.97	14.00	-4.46	4.46	0.12	1.00	6.90	Phi*lambda*sqrt in per 9.6.		20.1	7.0

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	5.000	5.64	23.79	0.24
+1.40D					
Span # 1	1	5.000	1.44	23.79	0.06
+1.20D					
Span # 1	1	5.000	1.24	23.79	0.05
+1.20D+0.50S					
Span # 1	1	5.000	2.61	23.79	0.11
+1.20D+1.60S					
Span # 1	1	5.000	5.64	23.79	0.24
+1.20D+0.70S					
Span # 1	1	5.000	3.16	23.79	0.13
+0.90D					
Span # 1	1	5.000	0.93	23.79	0.04

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+S	1	0.0027	2.500		0.0000	0.000

Concrete Shear Wall

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: FRONT WALL

Code References

Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

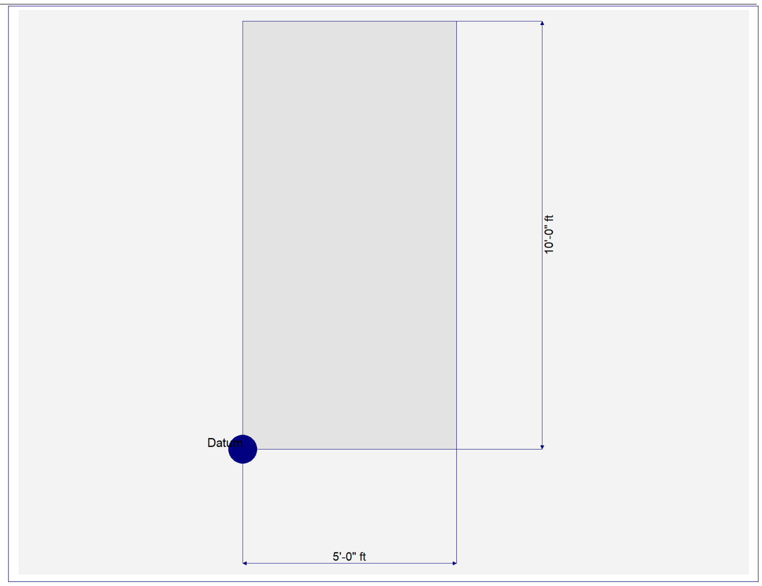
Wall Material CONCRETE
Sds 0.30

Material Properties

f'c	3.0	ksi	Ec	3,120.0	ksi
fy	60.0	ksi	Ev	1,248.0	ksi
Density	150.0	pcf	Phi - Shear	0.650	

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	5.0 ft
Wall Thickness	12.0 in
Structural Depth	12.0 ft



DESIGN SUMMARY

	Bottom Level
Vu : Story Shear	1.30 +1.20D+W
Mu : Story Moment	13.0 +1.20D+W
Nu : Axial	18.840 +1.20D+1.60S
Uplift @ Left End	0.0
Uplift @ Right End	0.0
Phi * 8 * sqrt(f'c)*h*Lw	205.067 k
Phi * Vc	51.267 k
Phi * Vs Req'd	0.0 k
Horizontal As Req'd	0.2880 in^2
Vertical As Req'd	0.360 in^2
Bending As Req'd	1.037 in^2

Force Summary

Load Combination / **Wall Level**	Values for Wall section Vu (k)	Mu (k)	Pu (k)	Resultant Ecc (ft)	Overturning Ratio	Uplift (k) Left	Right
+1.40D							
Wall Level : 1			13.020				
+1.20D							
Wall Level : 1			11.160				
+1.20D+0.50S							
Wall Level : 1			13.560				
+1.20D+0.50W							
Wall Level : 1	0.650	6.500	11.160	0.582	4.292		
+1.20D+1.60S							

Concrete Shear Wall

Project File: Jess Anderson.ec6

DESCRIPTION: FRONT WALL

Force Summary

Load Combination / Wall Level	Values for Wall section Vu (k)	Mu (k)	Pu (k)	Resultant Ecc (ft)	Overturning Ratio	Uplift (k) Left	Right
Wall Level : 1			18.840				
+1.20D+1.60S+0.50W							
Wall Level : 1	0.650	6.500	18.840	0.345	7.246		
+1.20D+W							
Wall Level : 1	1.300	13.000	11.160	1.165	2.146		
+1.20D+0.50S+W							
Wall Level : 1	1.300	13.000	13.560	0.959	2.608		
+1.20D+0.70S+E							
Wall Level : 1			14.520				
+0.90D+W							
Wall Level : 1	1.300	13.000	8.370	1.553	1.610		
+0.90D+E							
Wall Level : 1			8.370				

Wall Footing

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: F1

Code References

Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
AutoCalc Footing Weight as DL :		Yes

Soil Design Values

Allowable Soil Bearing	=	1.50 ksf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Reference Depth below Surface	=	ft
Allow. Pressure Increase per foot of depth	=	ksf
when base footing is below	=	ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=	ksf
when footing is wider than	=	ft

Adjusted Allowable Bearing Pressure = 1.50 ksf

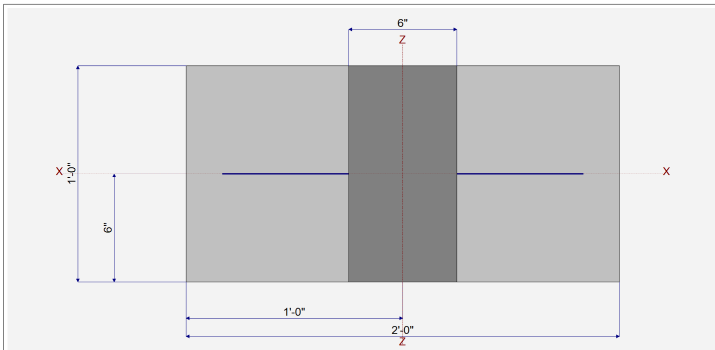
Dimensions

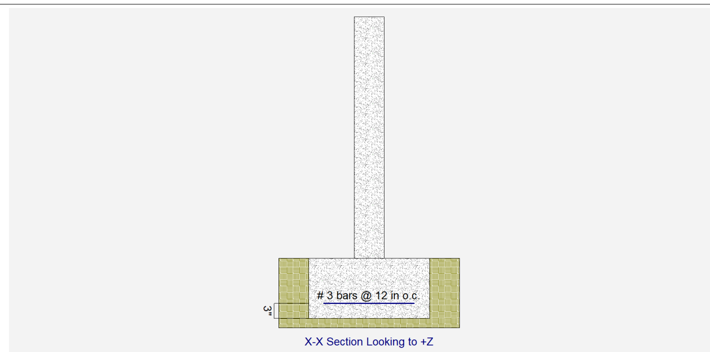
Footing Width	=	2.0 ft
Wall Thickness	=	6.0 in
Wall center offset from center of footing	=	0 in
Footing Thickness	=	12.0 in
Rebar Centerline to Edge of Concrete... at Bottom of footing	=	3.0 in

Reinforcing

Bars along X-X Axis

Bar spacing	=	12.00
Reinforcing Bar Size	=	# 3





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	1.50			0.960				k
OB : Overburden	=								ksf
V-x	=								k
M-zz	=								k-ft

Vx applied = in above top of footing

Wall Footing

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: F1

DESIGN SUMMARY

Design OK

	Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift

	Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.9167	Soil Bearing	1.375 ksf	1.50 ksf	+D+S
PASS	0.1177	Z Flexure (+X)	0.5181 k-ft	4.402 k-ft	+1.20D+1.60S
PASS	0.05147	Z Flexure (-X)	0.2265 k-ft	4.402 k-ft	+0.90D
PASS	n/a	1-way Shear (+X)	0.0 psi	82.158 psi	n/a
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Actual Soil Bearing Stress -X	Actual Soil Bearing Stress +X	Actual / Allowable Ratio
, D Only	1.50 ksf	0.0 in	0.8950 ksf	0.8950 ksf	0.597
, +D+S	1.50 ksf	0.0 in	1.375 ksf	1.375 ksf	0.917
, +D+0.750S	1.50 ksf	0.0 in	1.255 ksf	1.255 ksf	0.837
, +0.60D	1.50 ksf	0.0 in	0.5370 ksf	0.5370 ksf	0.358

Overturning Stability

Units : k-ft

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio	Status
Footing Has NO Sliding				

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Which Side ?	Tension @ Bot. or Top ?	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
, +1.40D	0.3524	-X	Bottom	0.0087	Min for Bending	0.11	4.402	OK
, +1.40D	0.3524	+X	Bottom	0.0087	Min for Bending	0.11	4.402	OK
, +1.20D	0.3021	-X	Bottom	0.0075	Min for Bending	0.11	4.402	OK
, +1.20D	0.3021	+X	Bottom	0.0075	Min for Bending	0.11	4.402	OK
, +1.20D+0.50S	0.3696	-X	Bottom	0.0091	Min for Bending	0.11	4.402	OK
, +1.20D+0.50S	0.3696	+X	Bottom	0.0091	Min for Bending	0.11	4.402	OK
, +1.20D+1.60S	0.5181	-X	Bottom	0.0128	Min for Bending	0.11	4.402	OK
, +1.20D+1.60S	0.5181	+X	Bottom	0.0128	Min for Bending	0.11	4.402	OK
, +1.20D+0.70S	0.3966	-X	Bottom	0.0098	Min for Bending	0.11	4.402	OK
, +1.20D+0.70S	0.3966	+X	Bottom	0.0098	Min for Bending	0.11	4.402	OK
, +0.90D	0.2265	-X	Bottom	0.0056	Min for Bending	0.11	4.402	OK
, +0.90D	0.2265	+X	Bottom	0.0056	Min for Bending	0.11	4.402	OK

One Way Shear

Units : k

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0 psi	0 psi	0 psi	82.158 psi	0	OK
+1.20D	0 psi	0 psi	0 psi	82.158 psi	0	OK
+1.20D+0.50S	0 psi	0 psi	0 psi	82.158 psi	0	OK
+1.20D+1.60S	0 psi	0 psi	0 psi	82.158 psi	0	OK
+1.20D+0.70S	0 psi	0 psi	0 psi	82.158 psi	0	OK
+0.90D	0 psi	0 psi	0 psi	82.158 psi	0	OK

General Footing

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: F2

Code References

Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=		
Min Allow % Temp Reinf.	=	0.00180	
Min. Overturning Safety Factor	=	1.0	: 1
Min. Sliding Safety Factor	=	1.0	: 1
Add Ftg Wt for Soil Pressure	:	Yes	
Use ftg wt for stability, moments & shears	:	Yes	
Add Pedestal Wt for Soil Pressure	:	Yes	
Use Pedestal wt for stability, mom & shear	:	No	

Soil Design Values

Allowable Soil Bearing	=	1.50	ksf
Soil Density	=	110.0	pcf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

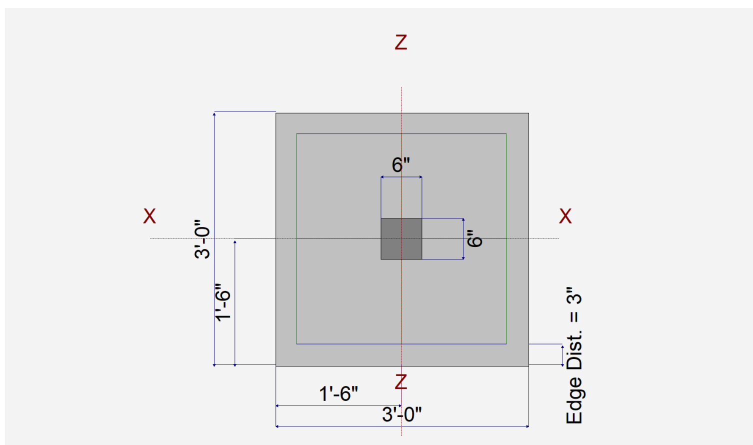
Footing base depth below soil surface	=		ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

Width parallel to X-X Axis	=	3.0	ft
Length parallel to Z-Z Axis	=	3.0	ft
Footing Thickness	=	10.0	in



Pedestal dimensions...

px : parallel to X-X Axis	=	6.0	in
pz : parallel to Z-Z Axis	=	6.0	in
Height	=	6.0	in

Rebar Centerline to Edge of Concrete...

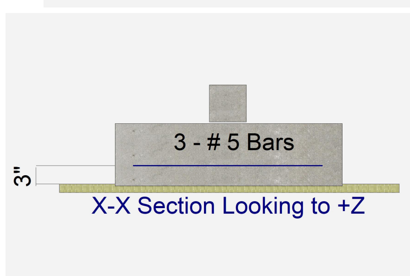
at Bottom of footing	=	3.0	in

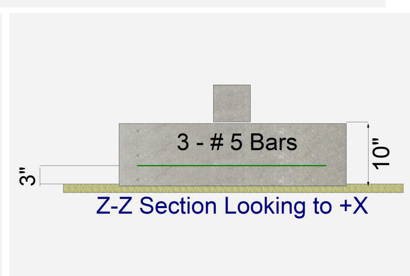
Reinforcing

Bars parallel to X-X Axis			
Number of Bars	=		3
Reinforcing Bar Size	=	#	5
Bars parallel to Z-Z Axis			
Number of Bars	=		3
Reinforcing Bar Size	=	#	5

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	2.70			7.20				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Project File: Jess Anderson.ec6

DESCRIPTION: F2

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8153	Soil Bearing	1.223 ksf	1.50 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.1374	Z Flexure (+X)	1.283 k-ft/ft	9.341 k-ft/ft	+1.20D+1.60S
PASS	0.1374	Z Flexure (-X)	1.283 k-ft/ft	9.341 k-ft/ft	+1.20D+1.60S
PASS	0.1374	X Flexure (+Z)	1.283 k-ft/ft	9.341 k-ft/ft	+1.20D+1.60S
PASS	0.1374	X Flexure (-Z)	1.283 k-ft/ft	9.341 k-ft/ft	+1.20D+1.60S
PASS	0.1571	1-way Shear (+X)	12.905 psi	82.158 psi	+1.20D+1.60S
PASS	0.1571	1-way Shear (-X)	12.905 psi	82.158 psi	+1.20D+1.60S
PASS	0.1571	1-way Shear (+Z)	12.905 psi	82.158 psi	+1.20D+1.60S
PASS	0.1571	1-way Shear (-Z)	12.905 psi	82.158 psi	+1.20D+1.60S
PASS	0.2151	2-way Punching	35.346 psi	164.317 psi	+1.20D+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	1.50	n/a	0.0	0.4228	0.4228	n/a	n/a	0.282
X-X, +D+S	1.50	n/a	0.0	1.223	1.223	n/a	n/a	0.815
X-X, +D+0.750S	1.50	n/a	0.0	1.023	1.023	n/a	n/a	0.682
X-X, +0.60D	1.50	n/a	0.0	0.2537	0.2537	n/a	n/a	0.169
Z-Z, D Only	1.50	0.0	n/a	n/a	n/a	0.4228	0.4228	0.282
Z-Z, +D+S	1.50	0.0	n/a	n/a	n/a	1.223	1.223	0.815
Z-Z, +D+0.750S	1.50	0.0	n/a	n/a	n/a	1.023	1.023	0.682
Z-Z, +0.60D	1.50	0.0	n/a	n/a	n/a	0.2537	0.2537	0.169

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.3303	+Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +1.40D	0.3303	-Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +1.20D	0.2831	+Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +1.20D	0.2831	-Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +1.20D+0.50S	0.5956	+Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +1.20D+0.50S	0.5956	-Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +1.20D+1.60S	1.283	+Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +1.20D+1.60S	1.283	-Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +1.20D+0.70S	0.7206	+Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +1.20D+0.70S	0.7206	-Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +0.90D	0.2123	+Z	Bottom	0.2160	AsMin	0.310	9.341	OK
X-X, +0.90D	0.2123	-Z	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +1.40D	0.3303	-X	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +1.40D	0.3303	+X	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +1.20D	0.2831	-X	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +1.20D	0.2831	+X	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +1.20D+0.50S	0.5956	-X	Bottom	0.2160	AsMin	0.310	9.341	OK

General Footing

Project File: Jess Anderson.ec6

DESCRIPTION: F2

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.20D+0.50S	0.5956	+X	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +1.20D+1.60S	1.283	-X	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +1.20D+1.60S	1.283	+X	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +1.20D+0.70S	0.7206	-X	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +1.20D+0.70S	0.7206	+X	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +0.90D	0.2123	-X	Bottom	0.2160	AsMin	0.310	9.341	OK
Z-Z, +0.90D	0.2123	+X	Bottom	0.2160	AsMin	0.310	9.341	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	3.32 psi	3.32 psi	3.32 psi	3.32 psi	3.32 psi	82.16 psi	0.04	OK
+1.20D	2.85 psi	2.85 psi	2.85 psi	2.85 psi	2.85 psi	82.16 psi	0.03	OK
+1.20D+0.50S	5.99 psi	5.99 psi	5.99 psi	5.99 psi	5.99 psi	82.16 psi	0.07	OK
+1.20D+1.60S	12.91 psi	12.91 psi	12.91 psi	12.91 psi	12.91 psi	82.16 psi	0.16	OK
+1.20D+0.70S	7.25 psi	7.25 psi	7.25 psi	7.25 psi	7.25 psi	82.16 psi	0.09	OK
+0.90D	2.14 psi	2.14 psi	2.14 psi	2.14 psi	2.14 psi	82.16 psi	0.03	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	9.10 psi	164.32 psi	0.05538	OK
+1.20D	7.80 psi	164.32 psi	0.04747	OK
+1.20D+0.50S	16.41 psi	164.32 psi	0.09986	OK
+1.20D+1.60S	35.35 psi	164.32 psi	0.2151	OK
+1.20D+0.70S	19.85 psi	164.32 psi	0.1208	OK
+0.90D	5.85 psi	164.32 psi	0.0356	OK

General Footing

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: F3

Code References

Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	Yes
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	1.50	ksf
Soil Density	=	110.0	pcf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

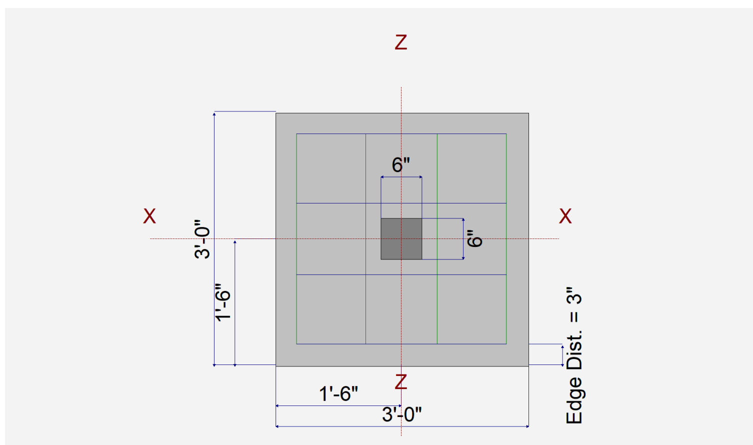
Footing base depth below soil surface	=		ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

Width parallel to X-X Axis	=	3.0	ft
Length parallel to Z-Z Axis	=	3.0	ft
Footing Thickness	=	10.0	in



Pedestal dimensions...

px : parallel to X-X Axis	=	6.0	in
pz : parallel to Z-Z Axis	=	6.0	in
Height	=	6.0	in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0	in

Reinforcing

Bars parallel to X-X Axis

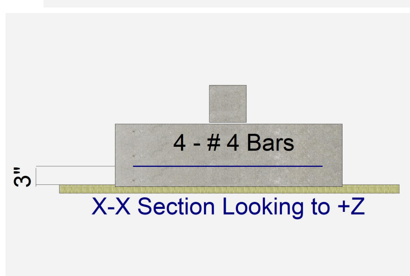
Number of Bars	=	4.0
Reinforcing Bar Size	= #	4

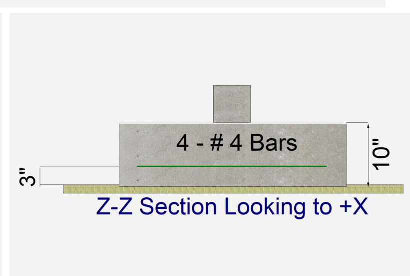
Bars parallel to Z-Z Axis

Number of Bars	=	4.0
Reinforcing Bar Size	= #	4

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	2.40			6.20				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: F3

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.7187	Soil Bearing	1.078 ksf	1.50 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.1376	Z Flexure (+X)	1.113 k-ft/ft	8.086 k-ft/ft	+1.20D+1.60S
PASS	0.1376	Z Flexure (-X)	1.113 k-ft/ft	8.086 k-ft/ft	+1.20D+1.60S
PASS	0.1376	X Flexure (+Z)	1.113 k-ft/ft	8.086 k-ft/ft	+1.20D+1.60S
PASS	0.1376	X Flexure (-Z)	1.113 k-ft/ft	8.086 k-ft/ft	+1.20D+1.60S
PASS	0.1362	1-way Shear (+X)	11.194 psi	82.158 psi	+1.20D+1.60S
PASS	0.1362	1-way Shear (-X)	11.194 psi	82.158 psi	+1.20D+1.60S
PASS	0.1362	1-way Shear (+Z)	11.194 psi	82.158 psi	+1.20D+1.60S
PASS	0.1362	1-way Shear (-Z)	11.194 psi	82.158 psi	+1.20D+1.60S
PASS	0.1866	2-way Punching	30.659 psi	164.317 psi	+1.20D+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	1.50	n/a	0.0	0.3895	0.3895	n/a	n/a	0.260
X-X, +D+S	1.50	n/a	0.0	1.078	1.078	n/a	n/a	0.719
X-X, +D+0.750S	1.50	n/a	0.0	0.9062	0.9062	n/a	n/a	0.604
X-X, +0.60D	1.50	n/a	0.0	0.2337	0.2337	n/a	n/a	0.156
Z-Z, D Only	1.50	0.0	n/a	n/a	n/a	0.3895	0.3895	0.260
Z-Z, +D+S	1.50	0.0	n/a	n/a	n/a	1.078	1.078	0.719
Z-Z, +D+0.750S	1.50	0.0	n/a	n/a	n/a	0.9062	0.9062	0.604
Z-Z, +0.60D	1.50	0.0	n/a	n/a	n/a	0.2337	0.2337	0.156

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.2939	+Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +1.40D	0.2939	-Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +1.20D	0.2519	+Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +1.20D	0.2519	-Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +1.20D+0.50S	0.5210	+Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +1.20D+0.50S	0.5210	-Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +1.20D+1.60S	1.113	+Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +1.20D+1.60S	1.113	-Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +1.20D+0.70S	0.6286	+Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +1.20D+0.70S	0.6286	-Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +0.90D	0.1889	+Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
X-X, +0.90D	0.1889	-Z	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +1.40D	0.2939	-X	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +1.40D	0.2939	+X	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +1.20D	0.2519	-X	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +1.20D	0.2519	+X	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +1.20D+0.50S	0.5210	-X	Bottom	0.2160	AsMin	0.2667	8.086	OK

General Footing

Project File: Jess Anderson.ec6

DESCRIPTION: F3

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.20D+0.50S	0.5210	+X	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +1.20D+1.60S	1.113	-X	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +1.20D+1.60S	1.113	+X	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +1.20D+0.70S	0.6286	-X	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +1.20D+0.70S	0.6286	+X	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +0.90D	0.1889	-X	Bottom	0.2160	AsMin	0.2667	8.086	OK
Z-Z, +0.90D	0.1889	+X	Bottom	0.2160	AsMin	0.2667	8.086	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	2.96 psi	2.96 psi	2.96 psi	2.96 psi	2.96 psi	82.16 psi	0.04	OK
+1.20D	2.53 psi	2.53 psi	2.53 psi	2.53 psi	2.53 psi	82.16 psi	0.03	OK
+1.20D+0.50S	5.24 psi	5.24 psi	5.24 psi	5.24 psi	5.24 psi	82.16 psi	0.06	OK
+1.20D+1.60S	11.19 psi	11.19 psi	11.19 psi	11.19 psi	11.19 psi	82.16 psi	0.14	OK
+1.20D+0.70S	6.32 psi	6.32 psi	6.32 psi	6.32 psi	6.32 psi	82.16 psi	0.08	OK
+0.90D	1.90 psi	1.90 psi	1.90 psi	1.90 psi	1.90 psi	82.16 psi	0.02	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	8.10 psi	164.32 psi	0.04927	OK
+1.20D	6.94 psi	164.32 psi	0.04223	OK
+1.20D+0.50S	14.35 psi	164.32 psi	0.08734	OK
+1.20D+1.60S	30.66 psi	164.32 psi	0.1866	OK
+1.20D+0.70S	17.32 psi	164.32 psi	0.1054	OK
+0.90D	5.20 psi	164.32 psi	0.03167	OK

LATERAL ANALYSIS

OSB Seismic Loading Analysis

S_s =	*1.067*	C_T =	*0.020*	
S_1 =	*0.356*	h_n =	*20.00*	ft
F_a =	*1.2*			
F_v =	*1.5*			
R =	*6.5*			
I_E =	*1.0*			

$S_{MS} = F_a S_s$ =	1.2804	
$S_{M1} = F_v S_1$ =	0.5340	
		Seismic Design Category
$S_{DS} = 2/3\ S_{MS}$ =	0.8536	*D*
$S_{D1} = 2/3\ S_{M1}$ =	0.3560	*D*
$C_s = S_{DS}/(R/I_E)$ =	0.1313	**Controls**
$T_a = C_T h_n^{3/4}$ =	0.1891	
$C_s < S_{D1}/[(R/I_E)T]$ =	0.2896	
$C_s > 0.044 S_{DS} I_E$ =	0.0376	
$C_s > 0.5 S_1/(R/I_E)$ =	0.0274	

$V = C_s W$ = **0.1313** W

0.7*V = **0.0919** W

OSB Seismic Component Loading

w_p =	*1*	psf	weight of element
V_{px} =	*0*	plf	Portion of seismic shear load at the level of the diaphragm, required to be transferred to the components of the vertical seismic-force-resisting system beacause of the offsets or changes in the stiffness of the vertical components above of below the diaphragm.
w_w =	*85*	psf	weight of wall
L_b =	*60*	ft	length of the building

NOTE: Use 1 for unit weight to achieve an answer per element unit weight

Connections

$F_p = 0.133\ S_{DS}\ w_p$ =	**0.11**	psf		
or				
$F_p = 0.05\ w_p$ =	**0.05**	psf		

Diaphragm

$F_p = 0.2\ I_E\ S_{DS}\ w_p + V_{px}$ =	**0.17**	psf		
$F_{p,max} = 0.4\ I_E\ S_{DS}\ w_p + V_{px}$ =	**0.34**	psf		

Bearing Walls & Shear Walls

Out of Plane Forces

$F_p = 0.40\ I_E\ S_{DS}\ w_w$ =	**29.02**	psf	**Controls**	*12.11.1*
$F_p = 0.10\ w_w$ =	**8.50**	psf		*12.11.1*

Anchorage

$F_p = 0.40\ I_E\ S_{DS}\ w_w\ k_a$ =	**46.4**	psf	**Controls**	*12.11-1*
$F_p = 0.2\ I_E\ k_a\ w_w$ =	**27.2000**	psf		
$k_a = 1.0 + L_b / 100$ =	**1.6000**			*12.11-2*

Note: *12.11.2.2.2* The strength design forces for steel elements of the structural wall anchorage system, with exception of anchor bolts and reinforcing steel, shall be increased by 1.4 times the forces otherwise noted above.

GYP Seismic Loading Analysis

S_s =	*1.067*	C_T =	*0.020*	
S_1 =	*0.356*	h_n =	*20.00*	ft
F_a =	*1.2*			
F_v =	*1.5*			
R =	*2.0*			
I_E =	*1.0*			

$S_{MS} = F_aS_s$ =	1.2804	
$S_{M1} = F_vS_1$ =	0.5340	
		Seismic Design Category
S_{DS} = 2/3 S_{MS} =	0.8536	*D*
S_{D1} = 2/3 S_{M1} =	0.3560	*D*
$C_s = S_{DS}/(R/I_E)$ =	0.4268	**Controls**
$T_a = C_Th_n^{3/4}$ =	0.1891	
$C_s < S_{D1}/[(R/I_E)T]$ =	0.9411	
$C_s > 0.044S_{DS}I_E$ =	0.0376	
$C_s > 0.5S_1/(R/I_E)$ =	0.0890	

$V = C_sW$ = **0.4268** W

0.7*V = **0.2988** W

GYP Seismic Component Loading

w_p =	*1*	psf	weight of element
V_{px} =	*0*	plf	Portion of seismic shear load at the level of the diaphragm, required to be transferred to the components of the vertical seismic-force-resisting system beacause of the offsets or changes in the stiffness of the vertical components above of below the diaphragm.
w_w =	*85*	psf	weight of wall
L_b =	*60*	ft	length of the building

NOTE: Use 1 for unit weight to achieve an answer per element unit weight

Connections

$F_p = 0.133\ S_{DS}\ w_p$ =	**0.11**	psf		
or				
$F_p = 0.05\ w_p$ =	**0.05**	psf		

Diaphragm

$F_p = 0.2\ I_E\ S_{DS}\ w_p + V_{px}$ =	**0.17**	psf		

Bearing Walls & Shear Walls
Out of Plane Forces

$F_p = 0.40\ I_E\ S_{DS}\ w_w$ =	**29.02**	psf	**Controls**	*12.11.1*
$F_p = 0.10\ w_w$ =	**8.50**	psf		*12.11.1*

Anchorage

$F_p = 0.40\ I_E\ S_{DS}\ w_w\ k_a$ =	**46.4**	psf	**Controls**	*12.11-1*
$F_p = 0.2\ I_E\ k_a\ w_w$ =	**27.2000**	psf		
$k_a = 1.0 + L_b / 100$ =	**1.6000**			*12.11-2*

Note: *12.11.2.2.2* The strength design forces for steel elements of the structural wall anchorage system, with exception of anchor bolts and reinforcing steel, shall be increased by 1.4 times the forces otherwise noted above.

Conc. Seismic Loading Analysis

S_s =	*1.067*	C_T =	*0.020*	
S_1 =	*0.356*	h_n =	*20.00*	ft
F_a =	*1.2*			
F_v =	*1.5*			
R =	*1.5*			
I_E =	*1.0*			

$S_{MS} = F_aS_s$ =	1.2804	
$S_{M1} = F_vS_1$ =	0.5340	
		Seismic Design Category
S_{DS} = 2/3 S_{MS} =	0.8536	*D*
S_{D1} = 2/3 S_{M1} =	0.3560	*D*
$C_s = S_{DS}/(R/I_E)$ =	0.5691	**Controls**
$T_a = C_Th_n^{3/4}$ =	0.1891	
$C_s < S_{D1}/[(R/I_E)T]$ =	1.2547	
$C_s > 0.044S_{DS}I_E$ =	0.0376	
$C_s > 0.5S_1/(R/I_E)$ =	0.1187	

$V = C_sW$ = **0.5691** W

0.7*V = **0.3983** W

Conc. Seismic Component Loading

w_p =	*1*	psf	weight of element
V_{px} =	*0*	plf	Portion of seismic shear load at the level of the diaphragm, required to be transferred to the components of the vertical seismic-force-resisting system beacause of the offsets or changes in the stiffness of the vertical components above of below the diaphragm.
w_w =	*85*	psf	weight of wall
L_b =	*60*	ft	length of the building

NOTE: Use 1 for unit weight to achieve an answer per element unit weight

Connections

$F_p = 0.133\ S_{DS}\ w_p$ =	**0.11**	psf		
or				
$F_p = 0.05\ w_p$ =	**0.05**	psf		

Diaphragm

$F_p = 0.2\ I_E\ S_{DS}\ w_p + V_{px}$ =	**0.17**	psf		

Bearing Walls & Shear Walls
Out of Plane Forces

$F_p = 0.40\ I_E\ S_{DS}\ w_w$ =	**29.02**	psf	**Controls**	*12.11.1*
$F_p = 0.10\ w_w$ =	**8.50**	psf		*12.11.1*

Anchorage

$F_p = 0.40\ I_E\ S_{DS}\ w_w\ k_a$ =	**46.4**	psf	**Controls**	*12.11-1*
$F_p = 0.2\ I_E\ k_a\ w_w$ =	**27.2000**	psf		
$k_a = 1.0 + L_b / 100$ =	**1.6000**			*12.11-2*

Note: *12.11.2.2.2* The strength design forces for steel elements of the structural wall anchorage system, with exception of anchor bolts and reinforcing steel, shall be increased by 1.4 times the forces otherwise noted above.

Wood Column

Project File: Jess Anderson.ec6

LIC# : KW-06017593, Build:20.23.10.02 ALLIANCE CONSUTLING ENGINEERING

DESCRIPTION: KING STUD L=14' TRIB=9.5'

Code References

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

General Information

Analysis Method	Allowable Stress Design	Wood Section Name	**3.5x5.5**
End Fixities	Top & Bottom Pinned	Wood Grading/Manuf.	Trus-Joist
Overall Column Height	14 ft	Wood Member Type	TimberStrand LSL

(Used for non-slender calculations)

Wood Species	iLevel Truss Joist	Exact Width	**3.50** in
Wood Grade	TimberStrand LSL 1.55E	Exact Depth	**5.50** in
		Area	19.250 in^2
		Ix	48.526 in^4
		Iy	19.651 in^4

Fb +	2325 psi	Fv	310 psi
Fb -	2325 psi	Ft	1070 psi
Fc - Prll	2050 psi	Density	45.01 pcf
Fc - Perp	800 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1550	1550	1550 ksi
Minimum	787.815	787.815	

Allow Stress Modification Factors	
Cf or Cv for Bending	1.074
Cf or Cv for Compressio	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Fact	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0
Use Cr : Repetitive ?	No

Column Buckling Condition:
ABOUT X-X Axis: Lux = 14 ft, Kx = 1.0
Fully braced against buckling ABOUT Y-Y Axis

Applied Loads

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

Column self weight included : 84.237 lbs * Dead Load Factor
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, W = 0.30 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.7808 : 1**
Load Combination	+D+0.60W
Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base	6.953 ft
At maximum location values are .	
Applied Axial	0.08424 k
Applied Mx	4.410 k-ft
Applied My	0.0 k-ft
Fc : Allowable	676.49 psi
PASS Maximum Shear Stress Ratio =	**0.1979 : 1**
Load Combination	+D+0.60W
Location of max.above base	14.0 ft
Applied Design Shear	147.273 psi
Allowable Shear	496.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	2.10 k	Bottom along Y-Y	2.10 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 3.485 in at 7.047 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.356	0.006654	PASS	0.0 ft	0.0	PASS	14.0 ft
+D+0.60W	1.600	0.206	0.7808	PASS	6.953 ft	0.1979	PASS	14.0 ft
+D+0.450W	1.600	0.206	0.5856	PASS	7.047 ft	0.1485	PASS	0.0 ft
+0.60D+0.60W	1.600	0.206	0.7788	PASS	6.953 ft	0.1979	PASS	14.0 ft
+0.60D	1.600	0.206	0.003881	PASS	0.0 ft	0.0	PASS	14.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only					0.084				
+D+0.60W			1.260	1.260	0.084				

Wind Shear Force Calculations

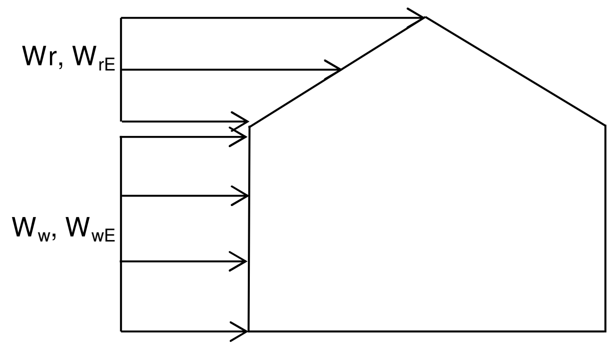
From 'ASCE 7-16 Wind Loading Analysis':

LOAD CASE 'A'	
a = 3.00 feet	2a = 6.00 feet
Z1 = 9.28 psf	Z1E = 12.46 psf
Z2 = 0.73 psf	Z2E = 2.20 psf
Z3 = -14.90 psf	Z3E = -17.34 psf
Z4 = -13.44 psf	Z4E = -16.12 psf

LOAD CASE 'B'	
a = 3.00 psf	2a = 6.00 feet
Z1 = -15.39 psf	Z1E = -16.12 psf
Z2 = -21.25 psf	Z2E = -30.54 psf
Z3 = -13.44 psf	Z3E = -17.34 psf
Z4 = -15.39 psf	Z4E = -16.12 psf

'A' FACTORED LOADS	
$0.6^*W_r = (Z_2 + Z_3) * 0.6 =$	**9.4 psf**
$0.6^*W_{rE} = (Z_{2E} + Z_{3E}) * 0.6 =$	**11.7 psf**
$0.6^*W_w = (Z_1 + Z_4) * 0.6 =$	**13.6 psf**
$0.6^*W_{wE} = (Z_{1E} + Z_{4E}) * 0.6 =$	**17.1 psf**

'B' FACTORED LOADS	
$0.6^*W_r = (Z_2 + Z_3) * 0.6 =$	**4.7 psf**
$0.6^*W_{rE} = (Z_{2E} + Z_{3E}) * 0.6 =$	**7.9 psf**
$0.6^*W_w = (Z_1 + Z_4) * 0.6 =$	**0.0 psf**
$0.6^*W_{wE} = (Z_{1E} + Z_{4E}) * 0.6 =$	**0.0 psf**



Wall Line	Wind Force (psf)	Wall ht (ft)	Upr. Flr Wall ht (ft)	wall line dist. (ft)	+	Wind Force (psf)	Wall ht (ft)	Wr, We truss trib (ft)	wall line dist (ft)	+	Shear, Upper (#)	=	**Wind Force (kips)**
X1-1	14.20	9	0	37.00	+	9.76	0	11.00	37	+	0.00	=	**3.17**
X2-1	13.82	9	0	111.67	+	9.60	0	11.00	111.667	+	0.00	=	**9.37**
X3-1	13.91	9	0	74.67	+	9.60	0	11.00	74.667	+	0.00	=	**6.28**
Y1-1	14.08	9	0	47.00	+	9.68	0	11.00	47	+	0.00	=	**3.99**
Y2-1	14.08	9	0	47.00	+	9.68	0	11.00	47	+	0.00	=	**3.99**

Seismic Shear Force Calculations

From 'ASCE7-16 Seismic Loading Analysis':

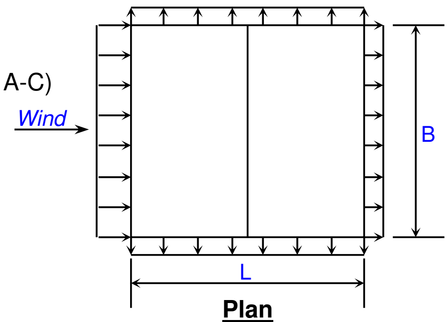
Wall Line	Roof (psf)	Area W (ft)	Area L (ft)	+	Floor (psf)	Area W (ft)	Area L (ft)	+	Wall Type	Wall (psf)	Wall Height (ft)	Perp Wall length (ft)	*C_s	+	Shear upper (kips)	=	**Shear Force (kips)**	**Seismic Shear Force at Wall (kips)**	**Lateral Control**
X1-1	17	37	47	+	18	0	0	+	OSB	75.0	10	37.00	.03Wp	+	0	=	**1.45**	**1.45**	**Wind**
X2-1	17	112	47	+	18	0	0	+	OSB	75.0	10	111.67	.03Wp	+	0	=	**4.37**	**4.37**	**Wind**
X3-1	17	74.7	47	+	18	0	0	+	OSB	75.0	10	74.67	.03Wp	+	0	=	**2.92**	**2.92**	**Wind**
Y1-1	17	47	112	+	18	0	0	+	OSB	12.0	10	47.00	.03Wp	+	0	=	**1.72**	**1.72**	**Wind**
Y2-1	17	47	112	+	18	0	0	+	OSB	12.0	10	47.00	.03Wp	+	0	=	**1.71**	**1.71**	**Wind**

WIND LOADING ANALYSIS - Main Wind-Force Resisting System

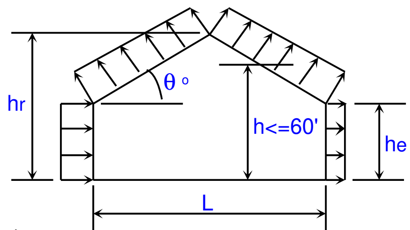
Per ASCE 7-16 Code for Enclosed or Partially Enclosed Buildings
Using Part 1 of ASCE Chapter 28 for Low-Rise Buildings (Envelope Procedure)

Input Data:

Wind Speed, V =	115	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	22.00	ft. (hr >= he)
Eave Height, he =	10.00	ft. (he <= hr)
Building Width =	47.00	ft. (Normal to Building Ridge)
Building Length =	112.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8.2 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6-1)
Enclosed? (Y/N)	Y	(Sect. 26.2)
Hurricane Region?	N	



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	27.05	deg.
Mean Roof Ht., h =	16.00	ft. (h = (hr+he)/2, for angle >10 deg.)

Check Criteria for a Low-Rise Building: (Section 26.2)

1. Is h <= 60' ? Yes, O.K.
2. Is h <= Lesser of L or B? Yes, O.K.

External Pressure Coeff's., GCpf (Fig. 28.3-1):
(For values, see following wind load tabulations.)
Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.13-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If h < 15 then: Kh = 2.01*(15/zg)^(2/α) (Table 26.10-1, Footnote 1)
If h >= 15 then: Kh = 2.01*(z/zg)^(2/α) (Table 26.10-1, Footnote 1)

α =	9.50	(Table 26.11-1)
zg =	900	(Table 26.11-1)
Kh =	0.86	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 26.10.2, Eq. 26.10-1)
qh = 24.76 psf qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)
Design Net External Wind Pressures (Sect. 28.3.1):
p = qh*[(GCpf) - (+/-GCpi)] (psf, Eq. 28.3-1)

Wall and Roof End Zone Widths 'a' and '2*a' (Fig. 28.3-1):

a =	4.70	ft.
2*a =	9.40	ft.

MWFRS Wind Load for Load Case A				MWFRS Wind Load for Load Case B			
Surface	GCpf	p = Net Pressures (psf)		Surface	*GCpf	p = Net Pressures (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1	0.55	9.19	18.11	Zone 1	-0.45	-15.60	-6.69
Zone 2	-0.06	-5.83	3.08	Zone 2	-0.69	-21.54	-12.63
Zone 3	-0.44	-15.47	-6.56	Zone 3	-0.37	-13.62	-4.70
Zone 4	-0.39	-14.06	-5.14	Zone 4	-0.45	-15.60	-6.69
Zone 5	---	---	---	Zone 5	0.40	5.45	14.36
Zone 6	---	---	---	Zone 6	-0.29	-11.64	-2.72
Zone 1E	0.72	13.43	22.35	Zone 1E	-0.48	-16.34	-7.43
Zone 2E	-0.13	-7.56	1.36	Zone 2E	-1.07	-30.95	-22.04
Zone 3E	-0.58	-18.75	-9.84	Zone 3E	-0.53	-17.58	-8.67
Zone 4E	-0.53	-17.51	-8.60	Zone 4E	-0.48	-16.34	-7.43
Zone 5E	---	---	---	Zone 5E	0.61	10.65	19.56
Zone 6E	---	---	---	Zone 6E	-0.43	-15.11	-6.19

*Note: Use roof angle θ = 0 degrees for Longitudinal Direction.

For Case A when GCpf is neg. in Zones 2/2E:
Zones 2/2E dist. = 23.50 ft. (Fig. 28.3-1)

For Case B when GCpf is neg. in Zones 2/2E:
Zones 2/2E dist. = 25.00 ft. (Fig. 28.3-1)

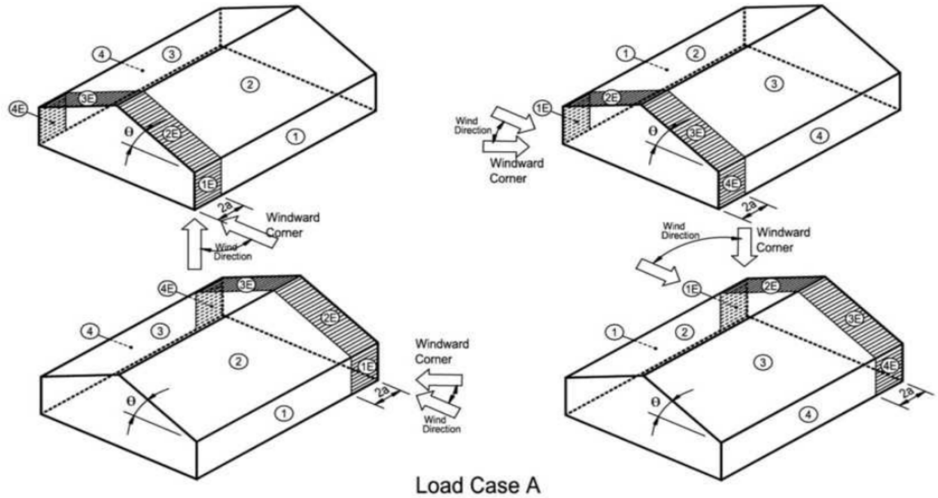
Remainder of roof Zones 2/2E extending to ridge line shall use roof Zones 3/3E pressure coefficients.

MWFRS Wind Load for Load Case A, Torsional Case				MWFRS Wind Load for Case B, Torsional Case			
Surface	GCpf	p = Net Pressure (psf)		Surface	GCpf	p = Net Pressure (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1T	---	2.30	4.53	Zone 1T	---	-3.90	-1.67
Zone 2T	---	-1.46	0.77	Zone 2T	---	-5.39	-3.16
Zone 3T	---	-3.87	-1.64	Zone 3T	---	-3.40	-1.18
Zone 4T	---	-3.51	-1.29	Zone 4T	---	-3.90	-1.67
Zone 5T	---	---	---	Zone 5T	---	1.36	3.59
Zone 6T	---	---	---	Zone 6T	---	-2.91	-0.68

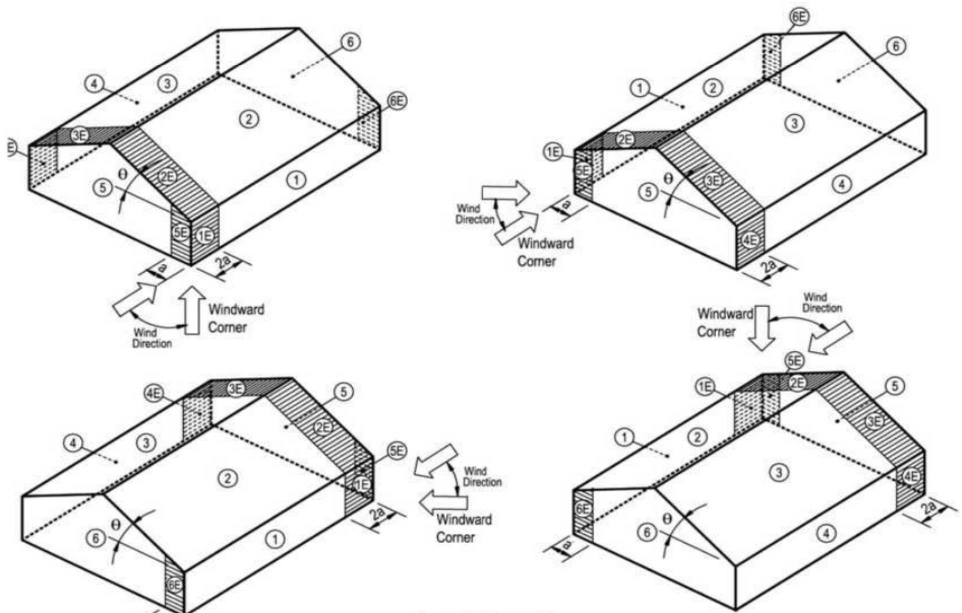
Notes: 1. For Load Case A (Transverse), Load Case B (Longitudinal), and Torsional Cases:
Zone 1 is windward wall for interior zone.
Zone 2 is windward roof for interior zone.
Zone 3 is leeward roof for interior zone.
Zone 4 is leeward wall for interior zone.
Zones 5 and 6 are sidewalls.
Zone 1T is windward wall for torsional case
Zone 3T is leeward roof for torsional case
Zones 5T and 6T are sidewalls for torsional case.
Zone 1E is windward wall for end zone.
Zone 2E is windward roof for end zone.
Zone 3E is leeward roof for end zone.
Zone 4E is leeward wall for end zone.
Zone 5E & 6E is sidewalls for end zone.
Zone 2T is windward roof for torsional case.
Zone 4T is leeward wall for torsional case.

2. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
3. Building must be designed for all wind directions using the 8 load cases shown below. The load cases are applied to each building corner in turn as the reference corner.
4. Wind loads for torsional cases are 25% of respective transverse or longitudinal zone load values. Torsional loading shall apply to all 8 basic load cases applied at each reference corner.
   Exception: One-story buildings with "h" <= 30', buildings <= 2 stories framed with light frame construction, and buildings <=2 stories designed with flexible diaphragms need not be designed for torsional load cases.
5. Per Code Section 28.3.4, the minimum wind load for MWFRS shall not be less than 16 psf. for wall pressure and 8psf for roof pressure

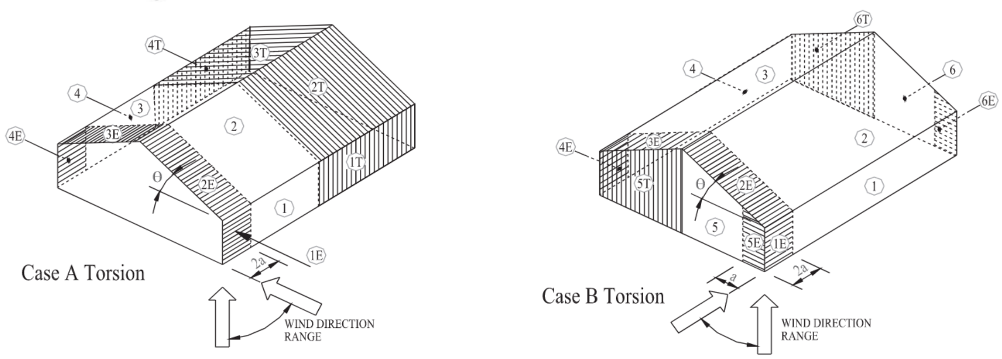
Low-Rise Buildings h <= 60'



Load Case A



Load Case B



Transverse Direction

Longitudinal Direction

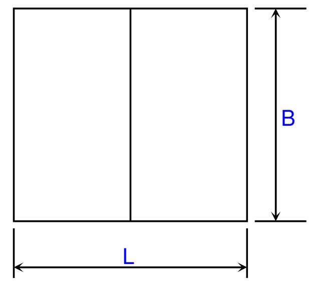
WIND LOADING ANALYSIS - Wall Components and Cladding

Per ASCE 7-16 Code for Buildings of Any Height
Using Part 1 & 3: Analytical Procedure (Section 30.3 & 30.5)

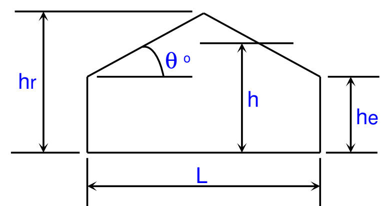
Input Data:

Wind Speed, V =	115	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	22	ft. (hr >= he)
Eave Height, he =	10	ft. (he <= hr)
Building Width =	47	ft. (Normal to Building Ridge)
Building Length =	112	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1	(Sect. 26.8.2 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6-1)
Enclosed? (Y/N)	Y	(Sect. 26.2)
Hurricane Region?	N	
Component Name =	Wall	(Girt, Siding, Wall, or Fastener)
Effective Area, Ae =	27	ft.^2 (Area Tributary to C&C)

Note: Worst Case Ae = Span Length * Length/3 (Sec 26.2)



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	27.05	deg.
Mean Roof Ht., h =	16.00	ft. (h = (hr+he)/2, for roof angle >10 deg.)

Wall External Pressure Coefficients, GCp:

GCp Zone 4 Pos. =	0.92	(Fig. 30.3-1)
GCp Zone 5 Pos. =	0.92	(Fig. 30.3-1)
GCp Zone 4 Neg. =	-1.02	(Fig. 30.3-1)
GCp Zone 5 Neg. =	-1.25	(Fig. 30.3-1)

Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.13-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 26.10-1, Footnote 1)

α =	9.50	(Table 26.11-1)
zg =	900	(Table 26.11-1)
Kh =	0.86	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 26.10.2, Eq. 26.10-1)

qh = 24.76 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

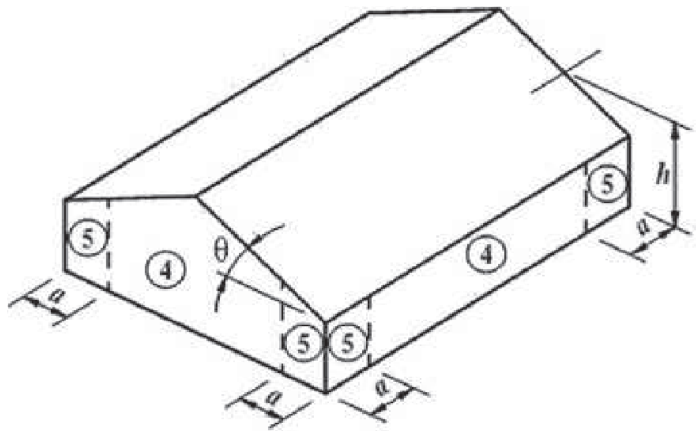
Design Net External Wind Pressures (Sect. 30.3.2 or 30.5.2):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qz for windward walls, q = qh for leeward walls and side walls
qi = qh for all walls (conservatively assumed per Sect. 30.5.2)

Wind Load Tabulation for Wall Components & Cladding

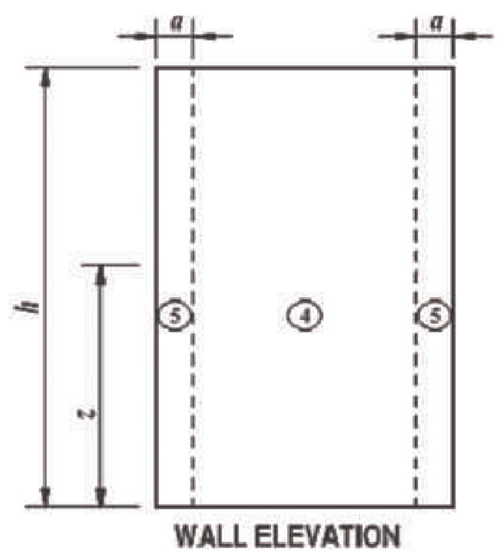
Component	z	Kh	qh	p = Net Design Pressures (psf)			
	(ft.)		(psf)	Zone 4 (+)	Zone 4 (-)	Zone 5 (+)	Zone 5 (-)
Wall	0	0.86	24.76	27.33	-29.81	27.33	-35.35
	15.00	0.86	24.76	27.33	-29.81	27.33	-35.35
	20.00	0.86	24.76	27.33	-29.81	27.33	-35.35
For z = hr:	22.00	0.86	24.76	27.33	-29.81	27.33	-35.35
For z = he:	10.00	0.86	24.76	27.33	-29.81	27.33	-35.35
For z = h:	16.00	0.86	24.76	27.33	-29.81	27.33	-35.35

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 5 (end zones), 'a' = 4.70 ft. (Fig. 30.3-1)
3. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
4. References : a. ASCE 7-16, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02"
by: Kishor C. Mehta and James M. Delahay (2004).

Wall Components and Cladding:



Wall Zones for Buildings with h <= 60 ft.



Wall Zones for Buildings with h > 60 ft.

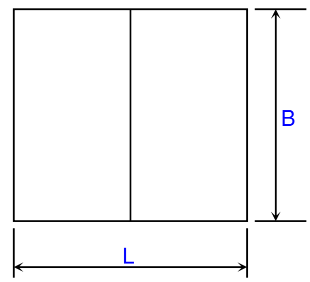
WIND LOADING ANALYSIS - Roof Components and Cladding

Per ASCE 7-16 Code for Bldgs. of Any Height with Gable Roof θ <= 45° or Monoslope Roof θ <= 3°
Using Part 1 & 3: Analytical Procedure (Section 30.3 & 30.5)

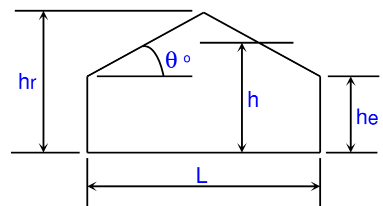
Input Data:

Wind Speed, V =	115	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1-1 Occupancy Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	22	ft. (hr >= he)
Eave Height, he =	10	ft. (he <= hr)
Building Width =	47	ft. (Normal to Building Ridge)
Building Length =	112	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1	(Sect. 26.8.2 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6-1)
Enclosed? (Y/N)	Y	(Sect. 26.2)
Hurricane Region?	N	
Component Name =	Joist	(Purlin, Joist, Decking, or Fastener)
Effective Area, Ae =	736.3333	ft.^2 (Area Tributary to C&C)
Overhangs? (Y/N)	Y	(if used, overhangs on all sides)

Note: Worst Case Ae = Span Length * Length/3 (Sec 26.2)



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	27.05	deg.
Mean Roof Ht., h =	16.00	ft. (h = (hr+he)/2, for roof angle >10 deg.)

Roof External Pressure Coefficients, GCp:

GCp Zone 1-3 Pos. =	0.80	(Fig. 30.3-2D)
GCp Zone 1 Neg. =	-0.80	(Fig. 30.3-2D)
GCp Zone 2 Neg. =	-1.80	(Fig. 30.3-2D)
GCp Zone 3 Neg. =	-1.80	(Fig. 30.3-2D)

Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.13-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 26.10-1, Footnote 1)

α =	9.50	(Table 26.11-1)
zg =	900	(Table 26.11-1)
Kh =	0.86	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 26.10.2, Eq. 26.10-1)

qh = 24.76 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

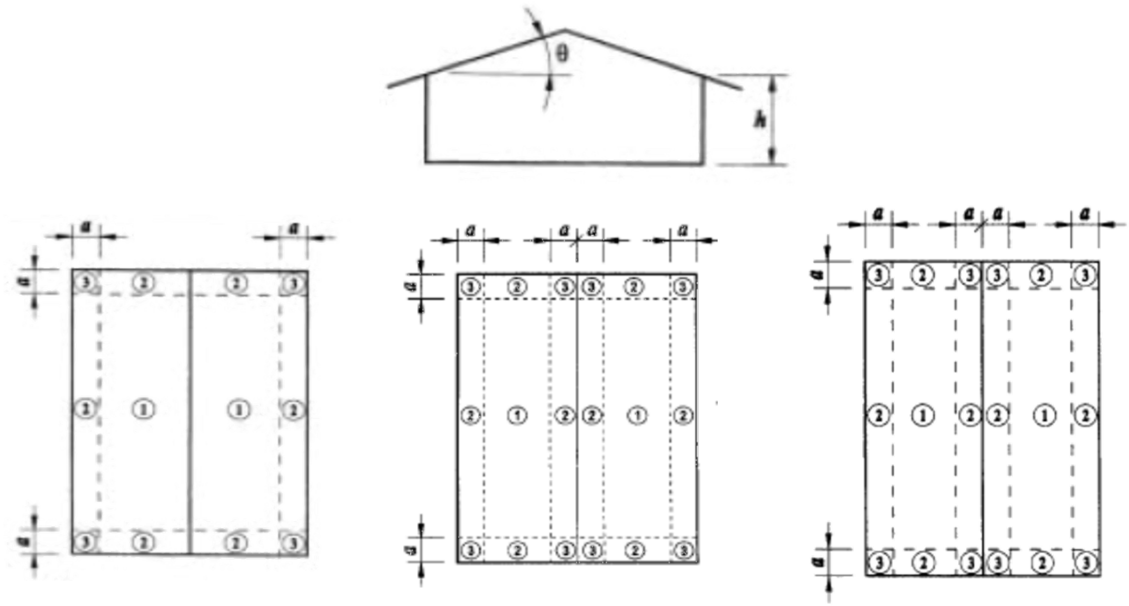
Design Net External Wind Pressures (Sect. 30.3.2 or 30.5.2):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qh for roof
qi = qh for all walls (conservatively assumed per Sect. 30.5.2)

Wind Load Tabulation for Roof Components & Cladding

Component	z	Kh	qh	p = Net Design Pressures (psf)			
	(ft.)		(psf)	Zone 1,2,3 (+)	Zone 1 (-)	Zone 2 (-)	Zone 3 (-)
Joist	0	0.86	24.76	24.27	-24.27	-49.03	-49.03
	15.00	0.86	24.76	24.27	-24.27	-49.03	-49.03
	20.00	0.86	24.76	24.27	-24.27	-49.03	-49.03
For z = hr:	22.00	0.86	24.76	24.27	-24.27	-49.03	-49.03
For z = he:	10.00	0.86	24.76	24.27	-24.27	-49.03	-49.03
For z = h:	16.00	0.86	24.76	24.27	-24.27	-49.03	-49.03

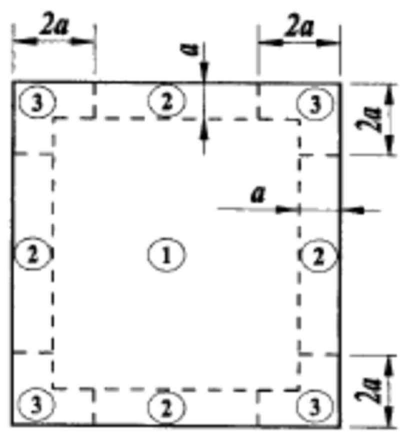
Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 2 (edge), 'a' = 4.70 ft.
3. Width of Zone 3 (corner), 'a' = 4.70 ft.
4. For monoslope roofs with θ <= 3 degrees, use Fig. 30.4-2A for 'GCp' values with 'qh'.
5. For buildings with h > 60' and θ > 10 degrees, use Fig. 30.6-1 for 'GCpi' values with 'qh'.
6. For all buildings with overhangs, use Fig. 30.4-2B for 'GCp' values per Sect. 30.10.
7. If a parapet >= 3' in height is provided around perimeter of roof with θ <= 10 degrees, Zone 3 shall be treated as Zone 2.
8. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
9. References : a. ASCE 7-16, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02" by: Kishor C. Mehta and James M. Delahay (2004).

Roof Components and Cladding:



θ <= 7 deg. **7 deg. < θ <= 27 deg.** **27 deg. < θ <= 45 deg.**

Roof Zones for Buildings with h <= 60 ft.
(for Gable Roofs <= 45^{o} and Monoslope Roofs <= 3^{o})



ROOF PLAN

Roof Zones for Buildings with h > 60 ft.
(for Gable Roofs <= 10^{o} and Monoslope Roofs <= 3^{o})

Check Roof Uplift Connection:

Average Roof Pressure:	-29.22	pdf	
Length of Roof Truss:	47	ft	
Truss Spacing:	2	ft	
Factored Force on Truss end:	-824.015	lbs	
DL truss reaction:	564	lbs	--> (Assume 7psf top chord dead load)
Net Uplift:	260.01	lbs	
Simpson H2.5A Cap.:	700	lbs	
	OK		