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STRUCTURAL CALCULATIONS

FOR

BUILDING IMPROVEMENT FOR CORONA SOUTH MALL BUILDING "B"



BY

IDS GROUP
1 PETERS CANYON, SUITE 130
IRVINE, CA 92606

DECEMBER, 2025

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1 Peters Canyon Road, Suite 140

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Job No.: 24x084.01	
Calc. By.: EW	Date: 11-25-2025
Chck'd By.:	Date:

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Narrative

This project involves converting an existing 99 Cents Only store into a food hall featuring a bar area, roller rink, and playground. Following the proposed improvements, the building is classified as Risk Category III. The structural scope includes a seismic retrofit as well as structural support for architectural and mechanical upgrades.

The specific structural scope of work includes:

- Seismic upgrade of major structural systems in accordance with ASCE 41-17. The performance criteria follow the Basic Performance Objective for Existing Buildings (BPOE), which requires Damage Control under BSE-2E and Limited Safety under BSE-1E.

- New building addition on the south side.

- Structural support for new rooftop mechanical units and new skylights.

- Creation of new openings in existing masonry walls.

- Framing for new nonbearing partitions and ceilings, including retail space build-out.

Table 17-2. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Building System—General			
(C) NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C (NC) N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity.	5.4.1.2	A.2.1.2
(C) NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC (N/A) U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2
C NC (N/A) U	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3
(C) NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC (N/A) U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines.	5.4.2.4	A.2.2.5
C NC (N/A) U	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6
(C) NC N/A U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.	5.4.2.6	A.2.2.7

continues

Table 17-2 *(Continued)*. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Geologic Site Hazards			
(C) NC N/A U	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1
(C) NC N/A U	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2
(C) NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)			
Foundation Configuration			
C (NC) N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. 0.6*Sa = 0.6*1.74 = 1.04, 18/23.33 = 0.77 < 1.04, 7.08/11.5 = 0.62 < 1.04	5.4.3.3	A.6.2.1
C NC (N/A) U	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C.	5.4.3.4	A.6.2.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-34. Collapse Prevention Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
(C) NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
(C) NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.^{2} (0.48 MPa).	5.5.3.1.1	A.3.2.4.1
C (NC) N/A U	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in. (1220 mm), and all vertical bars extend to the top of the walls.	5.5.3.1.3	A.3.2.4.2
Stiff Diaphragms	#4@16" o.c. ew, typ. steel ratio = 0.2/9/16 = 0.00138, less than 0.002 but grater than 0.0007		
C NC (N/A) U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
Connections			
(C) NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC (N/A) U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
(C) NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC (N/A) U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.	5.7.2	A.5.2.3
(C) NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation.	5.7.3.4	A.5.3.5
C NC (N/A) U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Stiff Diaphragms			
(C) NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
(C) NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long.	5.6.1.3	A.4.1.6

continues

Table 17-34 *(Continued)*. Collapse Prevention Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Flexible Diaphragms	Building classified as Sitff diaphragm		
C NC (N/A) U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC (N/A) U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC (N/A) U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long.	5.6.1.3	A.4.1.6
C NC (N/A) U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC (N/A) U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC (N/A) U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC (N/A) U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC (N/A) U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3 mm) before engagement of the anchors.	5.7.1.2	A.5.1.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

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Unit Weights

Unit weight of 8" thick brick masonry wall =	75	psf
Unit weight of 8" thick masonry wall =	84	psf
Unit weight of partition =	10	psf
Unit weight of glazing =	10	psf

Roof Dead Load - Building

Roofing =	2	psf
Insulation =	2	psf
Plywood sheathing =	1.5	psf
Framing	5	psf
Fire sprinkler =	2	psf
Ceiling =	1	psf
MEP =	2	psf
Misc. =	0.5	psf
	16	psf

Additional weight for new steel members a	12	kips
Quonset Hut =	11	kips
Rooftop unit =	3	psf

Roof Dead Load - Front Canopy

Roofing =	2	psf
Insulation =	2	psf
Plywood sheathing =	1.5	psf
Framing	4	psf
Fire sprinkler =	2	psf
Ceiling =	10	psf
Misc.	1.5	psf
	23	psf

Roof Live Load 20 psf

Fundamental Period

ASCE 41-17 Section 4.4.2.4: The fundamental period of a building,
in the direction under consideration, shall be calculated in accordance with Eq. 4-4

C_t =	0.02	
β =	0.75	
H =	19	ft
$T = C_t h_n^{\beta}$	0.18	sec

Seismic Hazard Level and Performance Level

$S_{xs\ BSE\text{-}2E}$ =	1.747	g	Limited Safety
$S_{xS\ BSE\text{-}1E}$ =	1.002	g	Damage Control

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Excerpt from Soil Report

ASCE41-17 SEISMIC DESIGN PARAMETERS						
Site Latitude:	N 33.8740°	**Site Longitude:**	W 117.5658°	**Risk Category** [a]	II	
Site Class [b]	D					
Hazard Level:	BSE-2N	BSE-1N	BSE-2E	BSE-1E		
% Exceedance Probability in 50 Years:	2	10	5	20		
Seismic Parameter	**Recommended Value**					
Factored Uniform-Hazard Spectral Acceleration, SSUH	2.422	N/A	N/A	N/A		
Coefficient of Risk (0.2s), CRS [a]	0.915	N/A	N/A	N/A		
Probabilistic Risk-Targeted Ground Motion (0.2s) (g), SSRT	2.215	N/A	N/A	N/A		
Factored Deterministic Acceleration Value (0.2s) (g), SSD	2.065	N/A	N/A	N/A		
Factored Uniform-Hazard Spectral Acceleration, S1UH	0.866	N/A	N/A	N/A		
Coefficient of Risk (1.0s) CR1 [b]	0.904	N/A	N/A	N/A		
Probabilistic Risk-Targeted Ground Motion (1.0s) (g), S1RT	0.783	N/A	N/A	N/A		
Factored Deterministic Acceleration Value (1.0s) (g), S1D	0.809	N/A	N/A	N/A		
Site Coefficient, Fa [c]	1.000	N/A	1.000	1.152		
Site Coefficient, Fv [d]	1.700	N/A	1.700	2.023		
0.2-Second Spectral Response Acceleration, S_S [e]	2.065	N/A	1.747	0.870		
1.0-Second Spectral Response Acceleration, S_1 [f]	0.783	N/A	0.605	0.289		
Adjusted 0.2-Second Spectral Response Acceleration, S_{XS} [g]	2.065	1.376	1.747	1.002		
Adjusted 1.0-Second Spectral Response Acceleration, S_{X1} [h]	1.331	0.887	1.028	0.584		
Long -Period Transition Period, T_L [l]	8 sec					

a Per ASCE 7-16 Figure 22-18.
b Per ASCE 7-16 Figure 22-19.
c Per ASCE 7-16 Table 11-4.1.
d Per ASCE 7-16 Table 11-4.2.
e Per ASCE 7-16 Section11.4.2.
f Per ASCE 41-17 Equation 2-1 =F_aS_S
g Per ASCE 41-17 Equation 2-2 = F_vS_1
h Per ASCE 7-16 Figure 22-14.
g Per ASCE 41-17 Equation 2-2 = F_vS_1

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Excerpt from Soil Report

2022 CBC SEISMIC DESIGN PARAMETERS					
Site Latitude:	N 33.8740°	**Site Longitude:**	W 117.5658°	**Risk Category** [a]	II

Seismic Parameter	Recommended Value
Site Class [b]	D
Soil Profile Name [b]	Stiff Soil Profile
Site Coefficient, Fa [c]	1.0
Site Coefficient, Fv [d]	1.7
0.2-Second Spectral Response Acceleration, S_S [e]	2.065g
1.0-Second Spectral Response Acceleration, S_1 [f]	0.783g
Adjusted 0.2-Second Spectral Response Acceleration, S_{MS} [g]	2.065g
Adjusted 1.0-Second Spectral Response Acceleration, S_{M1} [h]	1.331g
Design 0.2-Second Spectral Response Acceleration, S_{DS} [i]	1.376g
Design 1.0-Second Spectral Response Acceleration, S_{D1} [j]	0.887g
Long -Period Transition Period, T_L [k]	8 sec
Mapped MCE_G Geometric Mean Peak Ground Acceleration, PGA [l]	0.866g
Site Coefficient, F_{PGA} [m]	1.1
MCE_G Peak Ground Acceleration adjusted for Site Class Effect, PGA_M [n]	0.953g

Risk Category	I or II or III	IV
Seismic Design Category based on S_1 [o]	E	F
Seismic Design Category based on SD_S [p]	N/A	N/A
Seismic Design Category based on SD_1 [q]	N/A	N/A

a Per 2022 CBC Table 1604.5
b Per 2022 CBC Section 1613.2.2
c Per 2022 CBC Table 1613.2.3(1). *Note: If simplified design procedure of Section 12.14 of ASCE 7-16 is adopted, the Fa value should be determined per Section 12.14.8.1 of ASCE 7-16 with no need for F_v, S_{MS}, S_{M1} values.*
d Per 2022 CBC Table 1613.2.3(2), *provided C_s values are determined by Equations 12.8-2, 12.8-3 and 12.8-4 of ASCE 7-16.*
e Per 2022 CBC Figure 1613.2.1(1)
f Per 2022 CBC Figure 1613.2.1(2)
g Per 2022 CBC Equation 16-20
h Per 2022 CBC Equation 16-21
i Per 2022 CBC Equation 16-22
j Per 2022 CBC Equation 16-23
k Per ASCE 7-16 Figure 22-14
l Per ASCE 7-16 Figure 22-9
m Per ASCE 7-16 Table 11.8-1
n Per ASCE 7-16 Equation 11.8-1 = PGA x F_{PGA}
o Per 2022 CBC Section 1613.2.5
p Per 2022 CBC Table 1613.2.5(1)
q Per 2022 CBC Table 1613.2.5(2)

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Seismic Performance Level

For the building evaluation, the Damage Control Structural Performance Level (S-2) and Limited Safety Performance Level (S-4) were selected to correspond with the ASCE41-17, Table 2-1 for Risk Category III building.

Damage Control Structural Performance Level (S-2) is defined as the postearthquake damage state between the Life Safety Structural Performance Level (S-3) and the Immediate Occupancy Structural Performance Level (S-1) .

Limited Safety Structural Performance Level (S-4) is defined as the postearthquake damage state between the Life Safety Structural Performance Level (S-3) and the Collapse Prevention Structural Performance Level (S-5)

Table 2-1. Basic Performance Objective for Existing Buildings (BPOE)

Risk Category	BSE-1E	BSE-2E
I and II	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Hazards Reduced Nonstructural Performance[a] (5-D)
III	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Hazards Reduced Nonstructural Performance[a] (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Hazards Reduced Nonstructural Performance[a] (3-D)

[a] Compliance with ASCE 7 provisions for new construction is deemed to comply.

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PROJECT: Corona South Mall Building B	BY: EW	JOB NO: 24X084.01
SUBJECT: Wind Load C&C	CHKD BY:	DATE: 11/11/2025

ASCE 7-16 Ch. 30 - Wind Loads: Components and Cladding

		Units	Notes	Reference
Structure Information				
Building Length L =	180	ft		
Building Width W =	150	ft		
Building Height H =	20	ft		
Parapet Height =	1	ft		
Roof Slope θ =	1.2	degree		
Wall Tributary Area =	0	sq-ft		
Roof Tributary Area =	0	sq-ft		
Building Corner a =	8	ft		
0.2h =	4	ft		
0.6h =	12	ft		
Wind Load Pressure:				
Risk Category =	II			
Wind Speed V =	95	mph		
Directionality factor, K_d =	0.85		MWFRS	*ASCE 7-16 Table 26.6-1*
Exposure Category =	C			*ASCE 7-16 26.7*
Exposure Coefficients K_h =	0.90			
Topographic factor, K_{zt} =	1.00		Flat terrain	*ASCE 7-16 26.8.2*
Ground Elevation Factor, K_e =	1.00		< 1,000 ft above sea level	*ASCE 7-16 26.9*
Gust effect factor, G =	0.85			*ASCE 7-16 26.11.1*
Wind Pressure q_h =	17.7	psf	$=0.00256*K_h*K_{zt}*K_d*K_e*V^2$	*ASCE 7-16 Eq. 26.10-1*

Wall Wind Pressure:

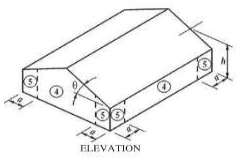
Roof Slope θ =	1.2	Degree	
Tributary Area =	0	sq-ft	

Unadjusted GCp

Zone	(+)	(-)	
4	1.00	-1.10	ASCE 7-16 Figure 30_3-1
5	1.00	-1.40	ASCE 7-16 Figure 30_3-1

Adjusted GCp Per Note 5: value of GCp to be reduced by 10% when θ <= 10 degree

Zone	(+)	(-)	
4	0.90	-0.99	ASCE 7-16 Figure 30_3-1
5	0.90	-1.26	ASCE 7-16 Figure 30_3-1
GCpi =	0.18	-0.18	ASCE 7-16 Table 26.13-1



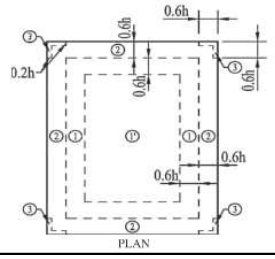
	LRFD	ASD			
$P_{Corner\ Zone\ 5}$ =	**19.09**	**11.45**	**psf**	$=q_h$ * (GCp + GCpi)	ASCE 7-16 EQ 30.3-1
$P_{Corner\ Zone\ 5}$ =	**-25.45**	**-15.27**	**psf**	$=q_h$ * (GCp - GCpi)	ASCE 7-16 EQ 30.3-1
$P_{Non\text{-}Corner\ Zone\ 4}$ =	**19.09**	**11.45**	**psf**	$=q_h$ * (GCp + GCpi)	ASCE 7-16 EQ 30.3-1
$P_{Non\text{-}Corner\ Zone\ 4}$ =	**-20.68**	**-12.41**	**psf**	$=q_h$ * (GCp - GCpi)	ASCE 7-16 EQ 30.3-1

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PROJECT: Corona South Mall Building B	BY: EW	JOB NO: 24X084.01
SUBJECT: Wind Load C&C	CHKD BY:	DATE: 11/11/2025

Roof Wind Pressure

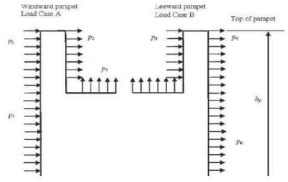
Roof Slope θ = q ≤ 7º
Tributary Area = 500 sq-ft
Parapet Height = 1 ft
GCpi = 0.18



	Gable Roof Ø ≤ 7º ASCE 7-16 Figure 30.3-2A							
	(-) Pressure (psf)							(+) Pressure
	3		2			1	1'	All Zone
GCp =	-1.40	0.00	-1.40	0.00	0.00	-1.00	-0.55	0.20
Wind Pressuer (LRFD) P =	**-27.93**	**0.00**	**-27.93**	**0.00**	**0.00**	**-20.86**	**-12.90**	**6.72**
Wind Pressure (ASD) P =	**-16.76**	**0.00**	**-16.76**	**0.00**	**0.00**	**-12.51**	**-7.74**	**4.03**

Wind Pressure $P = q_h * (GCp \pm GCpi)$ ASCE 7-16 EQ 30.3-1

Parapet Wind Pressure: ASECE 7-16 Figure 30.8-1



	LRFD	ASD		
Case A Corner =	**47.60**	**28.56**	**psf**	= [Wall Zone 5 (+) + Roof Zone 3 (-)]
Case A Non-Corner =	**47.60**	**28.56**	**psf**	= [Wall Zone 4 (+) + Roof Zone 2 (-)]
Case B Corner =	**45.09**	**27.06**	**psf**	= [Wall Zone 5 (+) - Wall Zone 5 (-)]
Case B Non-Corner =	**40.26**	**24.16**	**psf**	= [Wall Zone 4 (+) - Wall Zone 4 (-)]

NOTE: ALL DESIGN WIND PRESSURE SHALL BE 16PSF (LRFD) MINIMUM

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

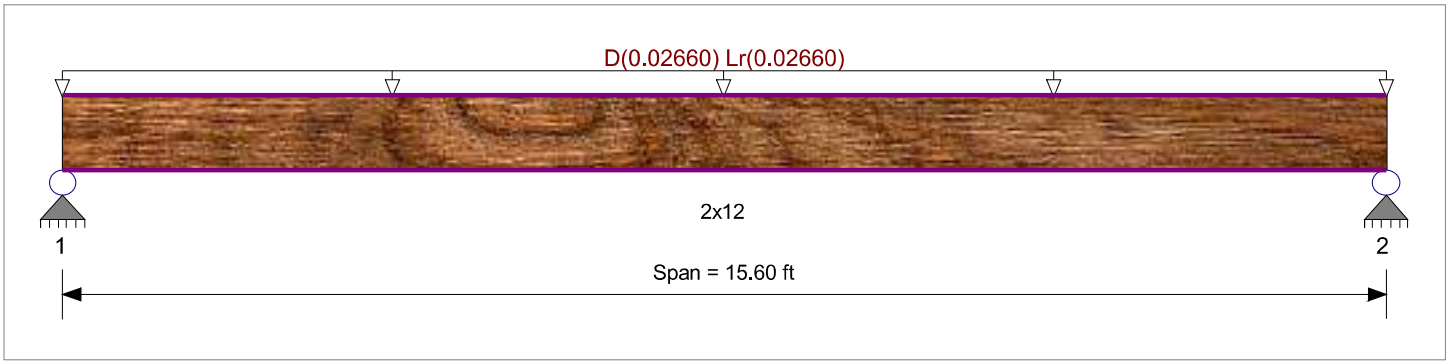
DESCRIPTION: --2x12 @16' o.c. at addition -

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.020, Lr = 0.020 ksf, Tributary Width = 1.330 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.491 : 1	Maximum Shear Stress Ratio	=	0.145 : 1
Section used for this span		2x12	Section used for this span		2x12
fb: Actual	=	613.77 psi	fv: Actual	=	32.58 psi
F'b	=	1,250.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	7.800 ft	Location of maximum on span	=	14.689 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.118 in	Ratio =	1588	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.236 in	Ratio =	794	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 15.60 ft	1	0.341	0.101	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.81	306.9	900.0	0.18	16.3	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 15.60 ft	1	0.491	0.145	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.62	613.8	1,250.0	0.37	32.6	225.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 15.60 ft	1	0.430	0.127	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.42	537.1	1,250.0	0.32	28.5	225.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 15.60 ft	1	0.115	0.034	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.49	184.1	1,600.0	0.11	9.8	288.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --2x12 @16' o.c.·at addition -

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.2357	7.857		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.415	0.415
Max Upward from Load Combinations	0.415	0.415
Max Upward from Load Cases	0.207	0.207
D Only	0.207	0.207
+D+Lr	0.415	0.415
+D+0.750Lr	0.363	0.363
+0.60D	0.124	0.124
Lr Only	0.207	0.207

Steel Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

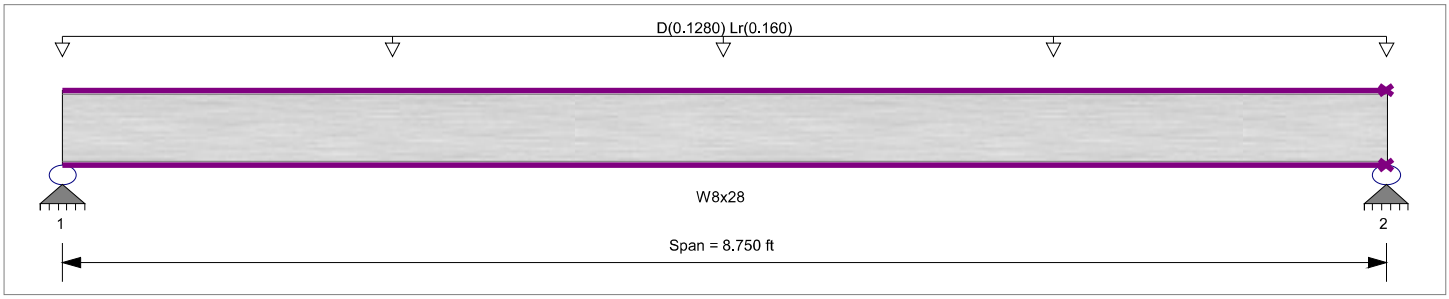
DESCRIPTION: --W8x28-at addition -

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Load Resistance Factor Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 8.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.042 : 1	Maximum Shear Stress Ratio =	0.028 : 1
Section used for this span	W8x28	Section used for this span	W8x28
Mu : Applied	4.242 k-ft	Vu : Applied	1.939 k
Mn * Phi : Allowable	102.000 k-ft	Vn * Phi : Allowable	68.913 k
Load Combination	+1.20D+1.60Lr	Load Combination	+1.20D+1.60Lr
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.007 in	Ratio =	14,076	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.015 in	Ratio =	7126	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	Summary of Shear Values VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 8.75 ft	1	0.020	0.014	2.09		2.09	113.33	102.00	1.00	1.00	0.96	68.91	68.91
+1.20D+0.50Lr													
Dsgn. L = 8.75 ft	1	0.025	0.017	2.56		2.56	113.33	102.00	1.00	1.00	1.17	68.91	68.91
+1.20D													
Dsgn. L = 8.75 ft	1	0.018	0.012	1.79		1.79	113.33	102.00	1.00	1.00	0.82	68.91	68.91
+1.20D+1.60Lr													
Dsgn. L = 8.75 ft	1	0.042	0.028	4.24		4.24	113.33	102.00	1.00	1.00	1.94	68.91	68.91
+0.90D													
Dsgn. L = 8.75 ft	1	0.013	0.009	1.34		1.34	113.33	102.00	1.00	1.00	0.61	68.91	68.91

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.0147	4.400		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.383	1.383
Max Upward from Load Combinations	1.383	1.383
Max Upward from Load Cases	0.700	0.700
D Only	0.683	0.683

Steel Beam

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --W8x28-at addition -

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
+D+Lr	1.383	1.383
+D+0.750Lr	1.208	1.208
+0.60D	0.410	0.410
Lr Only	0.700	0.700

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Skylight support 2-2x14--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

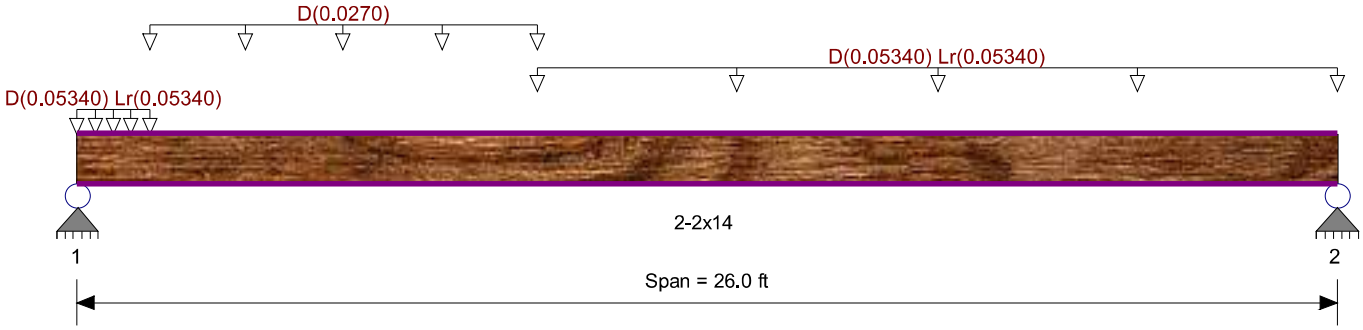
Material Properties

Analysis Method :	Allowable Stress Design
Load Combination :	ASCE 7-16
Wood Species :	Douglas Fir-Larch
Wood Grade :	No.1
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,500.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	675.0 psi

E : Modulus of Elasticity

Ebend- xx	1,700.0 ksi
Eminbend - xx	620.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Partial Length Uniform Load : D = 0.020, Lr = 0.020 ksf, Extent = 0.0 -->> 1.50 ft, Tributary Width = 2.670 ft
Partial Length Uniform Load : D = 0.020, Lr = 0.020 ksf, Extent = 9.50 -->> 26.0 ft, Tributary Width = 2.670 ft
Partial Length Uniform Load : D = 0.0270 k/ft, Extent = 1.50 -->> 9.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.894 : 1	Maximum Shear Stress Ratio	=	0.192 : 1
Section used for this span		2-2x14	Section used for this span		2-2x14
fb: Actual	=	1,005.36 psi	fv: Actual	=	43.09 psi
F'b	=	1,125.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	14.234 ft	Location of maximum on span	=	24.956 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.401 in	Ratio =	778	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.881 in	Ratio =	354	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.667	0.138	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.00	3.95	540.0	810.0	0.59	22.4	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.894	0.192	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	7.35	1,005.4	1,125.0	1.14	43.1	225.0
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.790	0.169	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	6.50	888.9	1,125.0	1.00	37.9	225.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Skylight support 2-2x14--

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.225	0.047	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.00	2.37	324.0	1,440.0	0.36	13.4	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.8808	13.380		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.885	1.253
Max Upward from Load Combinations	0.885	1.253
Max Upward from Load Cases	0.528	0.650
D Only	0.528	0.650
+D+Lr	0.885	1.253
+D+0.750Lr	0.796	1.102
+0.60D	0.317	0.390
Lr Only	0.357	0.604

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Skylight support 3-2x12--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

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

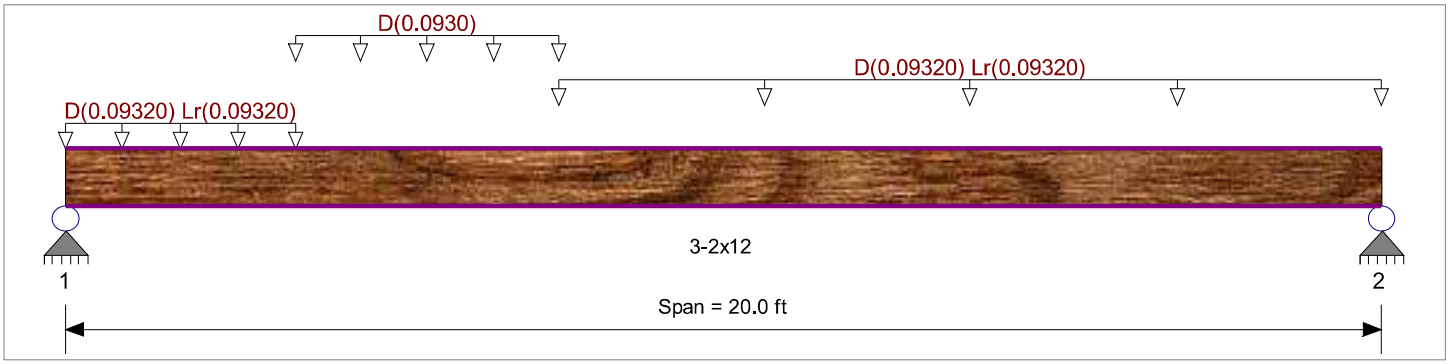
Wood Species : Douglas Fir-Larch
Wood Grade : No.1

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

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,500.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	675.0 psi

E : Modulus of Elasticity

Ebend- xx	1,700.0 ksi
Eminbend - xx	620.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Partial Length Uniform Load : D = 0.020, Lr = 0.020 ksf, Extent = 0.0 -->> 3.50 ft, Tributary Width = 4.660 ft
Partial Length Uniform Load : D = 0.020, Lr = 0.020 ksf, Extent = 7.50 -->> 20.0 ft, Tributary Width = 4.660 ft
Partial Length Uniform Load : D = 0.0930 k/ft, Extent = 3.50 -->> 7.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.842 : 1	Maximum Shear Stress Ratio	=	0.210 : 1
Section used for this span		3-2x12	Section used for this span		3-2x12
fb: Actual	=	1,051.92 psi	fv: Actual	=	47.35 psi
F'b	=	1,250.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	10.584 ft	Location of maximum on span	=	19.124 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.285 in	Ratio =	841 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.657 in	Ratio =	365 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 20.0 ft	1	0.654	0.155	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.66	588.8	900.0	0.85	25.2	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 20.0 ft	1	0.842	0.210	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.32	1,051.9	1,250.0	1.60	47.3	225.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 20.0 ft	1	0.749	0.186	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.40	935.8	1,250.0	1.41	41.8	225.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Skylight support 3-2x12--

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 20.0 ft	1	0.221	0.052	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.79	353.3	1,600.0	0.51	15.1	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.6565	10.146		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.593	1.761
Max Upward from Load Combinations	1.593	1.761
Max Upward from Load Cases	0.931	0.932
D Only	0.931	0.932
+D+Lr	1.593	1.761
+D+0.750Lr	1.428	1.554
+0.60D	0.559	0.559
Lr Only	0.662	0.829

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

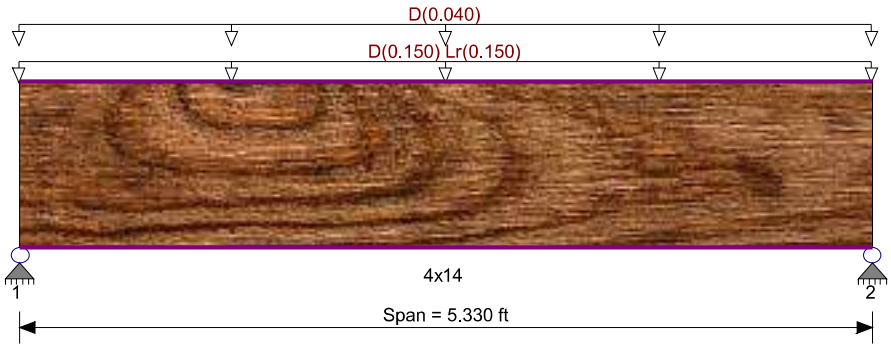
DESCRIPTION: --Skylight 4x support beam--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,000.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,000.0 psi	Ebend- xx	1,300.0 ksi
		Fc - Prll	1,000.0 psi	Eminbend - xx	1,300.0 ksi
Wood Species :	Douglas Fir-Larch (North)	Fc - Perp	1,000.0 psi		
Wood Grade :	No. 1/No. 2	Fv	65.0 psi		
		Ft	65.0 psi	Density	34.0 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.020, Lr = 0.020 ksf, Tributary Width = 7.50 ft
Uniform Load on ALL spans : D = 0.010 ksf, Tributary Width = 4.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.117 : 1	Maximum Shear Stress Ratio	=	0.220 : 1
Section used for this span		4x14	Section used for this span		4x14
fb: Actual	=	146.03 psi	fv: Actual	=	17.89 psi
F'b	=	1,250.00 psi	F'v	=	81.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	2.665 ft	Location of maximum on span	=	4.241 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.003 in	Ratio =	20591	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.007 in	Ratio =	8800	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 5.330 ft	1	0.093	0.175	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.71	83.6	900.0	0.32	10.2	58.5
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.330 ft	1	0.117	0.220	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.25	146.0	1,250.0	0.55	17.9	81.3
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.330 ft	1	0.104	0.197	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.11	130.4	1,250.0	0.49	16.0	81.3
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Skylight 4x support beam--

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 5.330 ft	1	0.031	0.059	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.43	50.2	1,600.0	0.19	6.1	104.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.0073	2.684		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.935	0.935
Max Upward from Load Combinations	0.935	0.935
Max Upward from Load Cases	0.536	0.536
D Only	0.536	0.536
+D+Lr	0.935	0.935
+D+0.750Lr	0.835	0.835
+0.60D	0.321	0.321
Lr Only	0.400	0.400

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Skylight 4x support beam (9.3 ft)--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

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

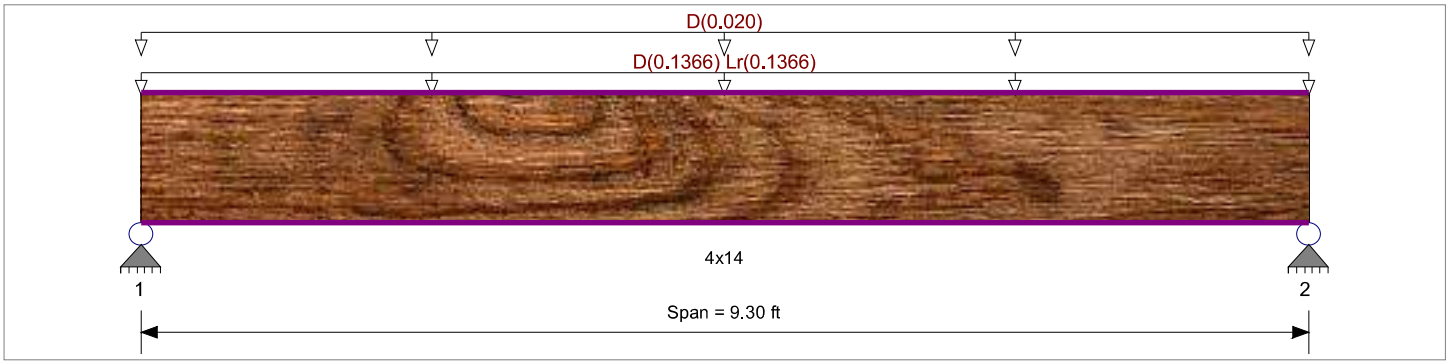
Wood Species : Douglas Fir-Larch (North)
Wood Grade : No. 1/No. 2

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

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,000.0 psi
Fc - Perp	1,000.0 psi
Fv	65.0 psi
Ft	65.0 psi

E : Modulus of Elasticity

Ebend- xx	1,300.0 ksi
Eminbend - xx	1,300.0 ksi
Density	34.0 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.020, Lr = 0.020 ksf, Tributary Width = 6.830 ft
Uniform Load on ALL spans : D = 0.010 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.297 : 1	Maximum Shear Stress Ratio	=	0.416 : 1
Section used for this span		4x14	Section used for this span		4x14
fb: Actual	=	371.43 psi	fv: Actual	=	33.80 psi
F'b	=	1,250.00 psi	F'v	=	81.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	4.650 ft	Location of maximum on span	=	8.214 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.026 in	Ratio =	4256 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.056 in	Ratio =	1983 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 9.30 ft	1	0.220	0.309	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.69	198.4	900.0	0.56	18.1	58.5
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.30 ft	1	0.297	0.416	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.17	371.4	1,250.0	1.04	33.8	81.3
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 9.30 ft	1	0.263	0.368	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.80	328.2	1,250.0	0.92	29.9	81.3
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Skylight 4x support beam (9.3 ft)--

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 9.30 ft	1	0.074	0.104	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.02	119.0	1,600.0	0.33	10.8	104.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.0563	4.684		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.363	1.363
Max Upward from Load Combinations	1.363	1.363
Max Upward from Load Cases	0.728	0.728
D Only	0.728	0.728
+D+Lr	1.363	1.363
+D+0.750Lr	1.205	1.205
+0.60D	0.437	0.437
Lr Only	0.635	0.635

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: -- AC-1, 2, 3, 4--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

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

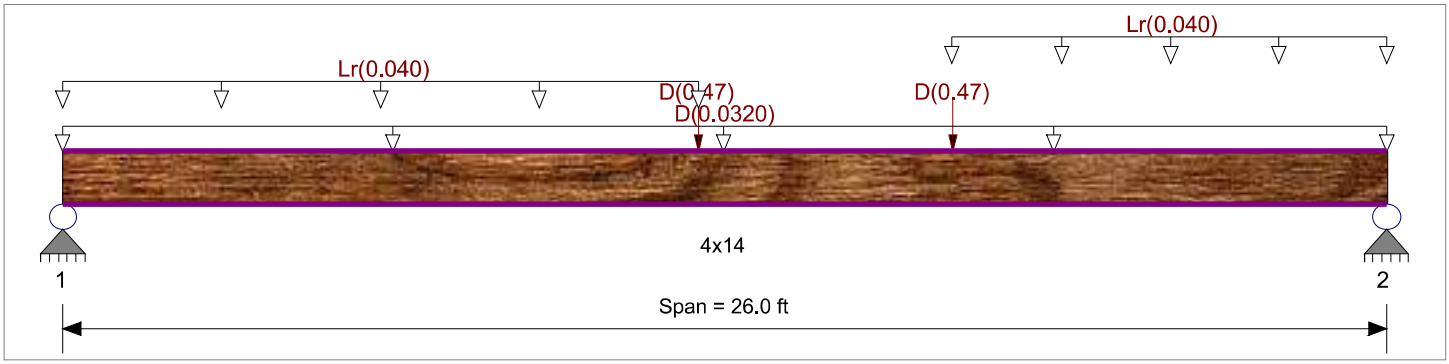
Wood Species : Douglas Fir-Larch
Wood Grade : No.1 & Better

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

Fb +	1200 psi
Fb -	1200 psi
Fc - Prll	1550 psi
Fc - Perp	625 psi
Fv	180 psi
Ft	800 psi

E : Modulus of Elasticity

Ebend- xx	1800 ksi
Eminbend - xx	660 ksi
Density	31.21 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160 ksf, Tributary Width = 2.0 ft
Partial Length Uniform Load : Lr = 0.020 ksf, Extent = 0.0 -->> 12.50 ft, Tributary Width = 2.0 ft
Partial Length Uniform Load : Lr = 0.040 k/ft, Extent = 17.50 -->> 26.0 ft
Point Load : D = 0.470 k @ 12.50 ft
Point Load : D = 0.470 k @ 17.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.832 : 1	Maximum Shear Stress Ratio	=	0.185 : 1
Section used for this span		4x14	Section used for this span		4x14
fb: Actual	=	898.69 psi	fv: Actual	=	41.65 psi
F'b	=	1,080.00 psi	F'v	=	225.00 psi
Load Combination		D Only	Load Combination		+D+Lr
Location of maximum on span	=	12.526 ft	Location of maximum on span	=	24.956 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.240 in	Ratio =	1301 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.961 in	Ratio =	324 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.832	0.185	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.67	898.7	1,080.0	0.92	29.9	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.780	0.185	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	9.99	1,170.0	1,500.0	1.29	41.7	225.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: -- AC-1, 2, 3, 4--

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.735	0.172	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	9.41	1,102.2	1,500.0	1.20	38.7	225.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.281	0.062	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.60	539.2	1,920.0	0.55	17.9	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.9610	13.190		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.249	1.363
Max Upward from Load Combinations	1.249	1.363
Max Upward from Load Cases	0.814	0.958
D Only	0.814	0.958
+D+Lr	1.249	1.363
+D+0.750Lr	1.140	1.262
+0.60D	0.488	0.575
Lr Only	0.435	0.405

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: -- AC-1, 2, 3, 4--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

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

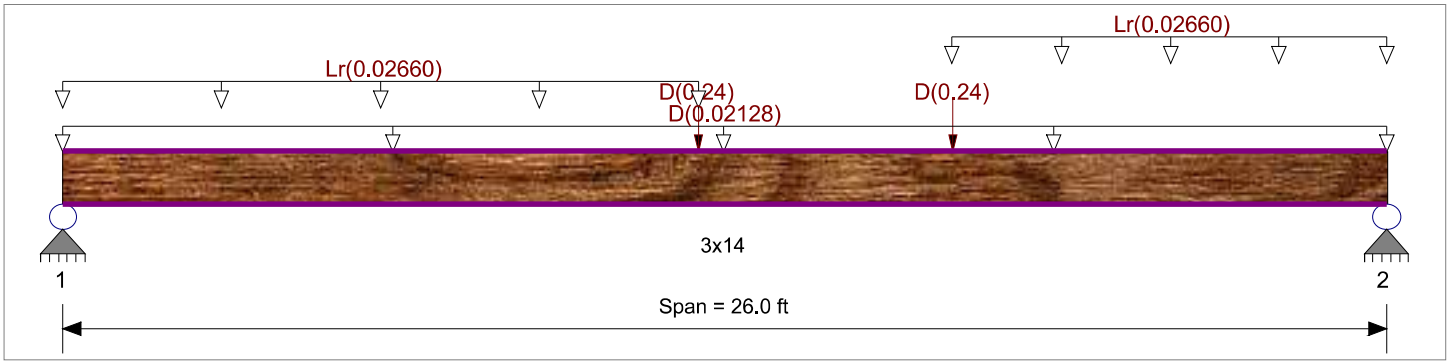
Wood Species : Douglas Fir-Larch
Wood Grade : No.1 & Better

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

Fb +	1,200.0 psi
Fb -	1,200.0 psi
Fc - Prll	1,550.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	800.0 psi

E : Modulus of Elasticity

Ebend- xx	1,800.0 ksi
Eminbend - xx	660.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160 ksf, Tributary Width = 1.330 ft
Partial Length Uniform Load : Lr = 0.020 ksf, Extent = 0.0 -->> 12.50 ft, Tributary Width = 1.330 ft
Partial Length Uniform Load : Lr = 0.02660 k/ft, Extent = 17.50 -->> 26.0 ft
Point Load : D = 0.240 k @ 12.50 ft
Point Load : D = 0.240 k @ 17.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.731 : 1	Maximum Shear Stress Ratio	=	0.156 : 1
Section used for this span		3x14	Section used for this span		3x14
fb: Actual	=	710.85 psi	fv: Actual	=	34.99 psi
F'b	=	972.00 psi	F'v	=	225.00 psi
Load Combination		D Only	Load Combination		+D+Lr
Location of maximum on span	=	12.526 ft	Location of maximum on span	=	24.956 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.223 in	Ratio =	1397 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.797 in	Ratio =	391 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.731	0.149	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.00	4.33	710.8	972.0	0.53	24.1	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.714	0.156	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	5.87	963.5	1,350.0	0.77	35.0	225.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: -- AC-1, 2, 3, 4--

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.667	0.143	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	5.49	900.3	1,350.0	0.71	32.3	225.0
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.247	0.050	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.00	2.60	426.5	1,728.0	0.32	14.4	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.7973	13.190		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.769	0.823
Max Upward from Load Combinations	0.769	0.823
Max Upward from Load Cases	0.480	0.554
D Only	0.480	0.554
+D+Lr	0.769	0.823
+D+0.750Lr	0.697	0.755
+0.60D	0.288	0.332
Lr Only	0.290	0.269

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --AC-5 -

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

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

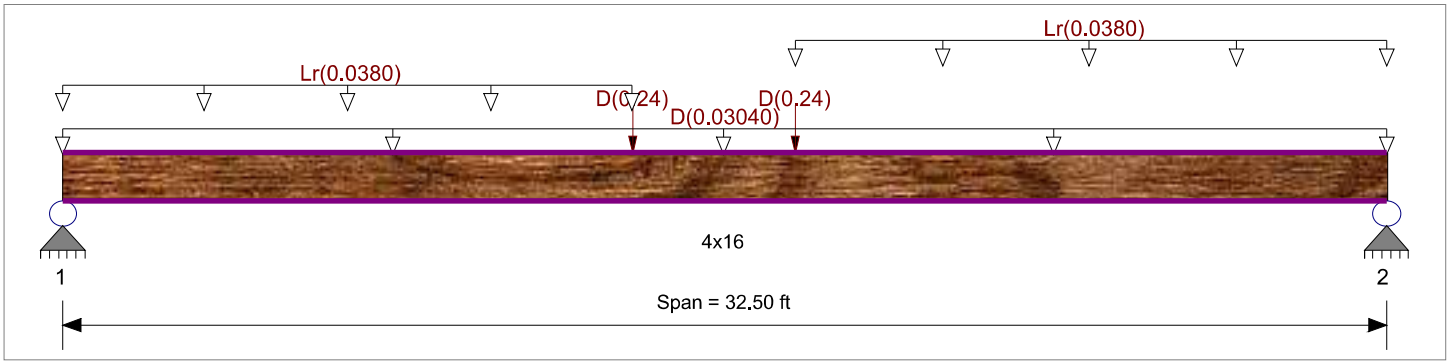
Wood Species : Douglas Fir-Larch
Wood Grade : No.1 & Better

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

Fb +	1,200.0 psi
Fb -	1,200.0 psi
Fc - Prll	1,550.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	800.0 psi

E : Modulus of Elasticity

Ebend- xx	1,800.0 ksi
Eminbend - xx	660.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160 ksf, Tributary Width = 1.90 ft
Partial Length Uniform Load : Lr = 0.020 ksf, Extent = 0.0 -->> 14.0 ft, Tributary Width = 1.90 ft
Partial Length Uniform Load : Lr = 0.0380 k/ft, Extent = 18.0 -->> 32.50 ft
Point Load : D = 0.240 k @ 14.0 ft
Point Load : D = 0.240 k @ 18.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.666 : 1	Maximum Shear Stress Ratio	=	0.149 : 1
Section used for this span		4x16	Section used for this span		4x16
fb: Actual	=	999.15 psi	fv: Actual	=	33.64 psi
F'b	=	1,500.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	16.606 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.415 in	Ratio =	940 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	1.140 in	Ratio =	342 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.609	0.122	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.43	657.6	1,080.0	0.70	19.7	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.666	0.149	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	11.30	999.1	1,500.0	1.20	33.6	225.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --AC-5 -

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.609	0.134	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	10.33	913.7	1,500.0	1.07	30.2	225.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.205	0.041	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.46	394.5	1,920.0	0.42	11.8	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.1401	16.250		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.278	1.273
Max Upward from Load Combinations	1.278	1.273
Max Upward from Load Cases	0.738	0.730
D Only	0.738	0.730
+D+Lr	1.278	1.273
+D+0.750Lr	1.143	1.137
+0.60D	0.443	0.438
Lr Only	0.540	0.543

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

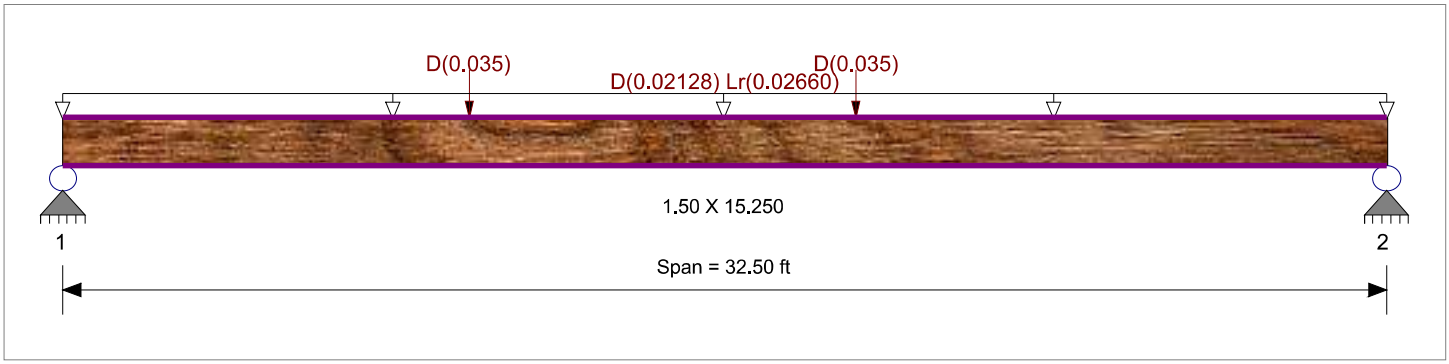
DESCRIPTION: --EF-1, 10--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,250.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,200.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,550.0 psi	Eminbend - xx	660.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1 & Better	Fv	180.0 psi		
		Ft	800.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 1.330 ft
Point Load : D = 0.0350 k @ 10.0 ft
Point Load : D = 0.0350 k @ 19.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.987 : 1	Maximum Shear Stress Ratio	=	0.221 : 1
Section used for this span		**1.50 X 15.250**	Section used for this span		**1.50 X 15.250**
fb: Actual	=	1,387.85 psi	fv: Actual	=	49.80 psi
F'b	=	1,406.25 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	16.369 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.842 in	Ratio =	463 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	1.610 in	Ratio =	242 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.655	0.145	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.00	3.21	663.0	1,012.5	0.36	23.5	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.987	0.221	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	6.72	1,387.8	1,406.3	0.76	49.8	225.0
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.858	0.192	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	5.85	1,206.6	1,406.3	0.66	43.2	225.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --EF-1, 10--

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.221	0.049	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.00	1.93	397.8	1,800.0	0.22	14.1	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.6104	16.250		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.816	0.810
Max Upward from Load Combinations	0.816	0.810
Max Upward from Load Cases	0.432	0.432
D Only	0.384	0.378
+D+Lr	0.816	0.810
+D+0.750Lr	0.708	0.702
+0.60D	0.230	0.227
Lr Only	0.432	0.432

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --EF-2, 7-

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

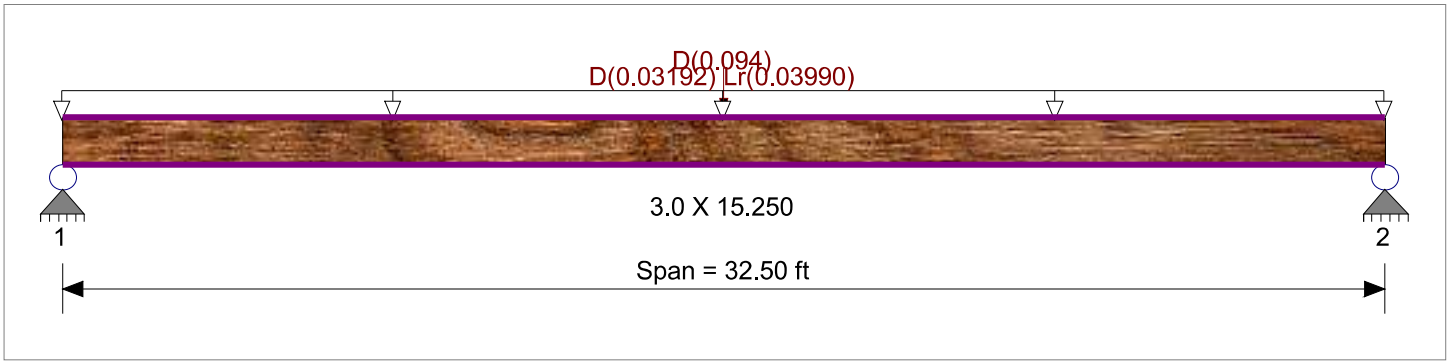
Material Properties

Analysis Method :	Allowable Stress Design
Load Combination :	ASCE 7-16
Wood Species :	Douglas Fir-Larch
Wood Grade :	No.1 & Better
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling

Fb +	1,200.0 psi
Fb -	1,200.0 psi
Fc - Prll	1,550.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	800.0 psi

E : Modulus of Elasticity

Ebend- xx	1,800.0 ksi
Eminbend - xx	660.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 1.995 ft
Point Load : D = 0.0940 k @ 16.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.783 : 1	Maximum Shear Stress Ratio	=	0.165 : 1
Section used for this span		3.0 X 15.250	Section used for this span		3.0 X 15.250
fb: Actual	=	1,057.39 psi	fv: Actual	=	37.01 psi
F'b	=	1,350.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	16.250 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.631 in	Ratio =	617 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	1.209 in	Ratio =	322 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.529	0.107	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.00	4.98	513.7	972.0	0.53	17.3	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.783	0.165	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	10.25	1,057.4	1,350.0	1.13	37.0	225.0
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.683	0.143	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	8.93	921.5	1,350.0	0.98	32.1	225.0
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --EF-2, 7-

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 32.50 ft	1	0.178	0.036	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.00	2.99	308.2	1,728.0	0.32	10.4	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.2094	16.369		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.214	1.214
Max Upward from Load Combinations	1.214	1.214
Max Upward from Load Cases	0.648	0.648
D Only	0.566	0.566
+D+Lr	1.214	1.214
+D+0.750Lr	1.052	1.052
+0.60D	0.339	0.339
Lr Only	0.648	0.648

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

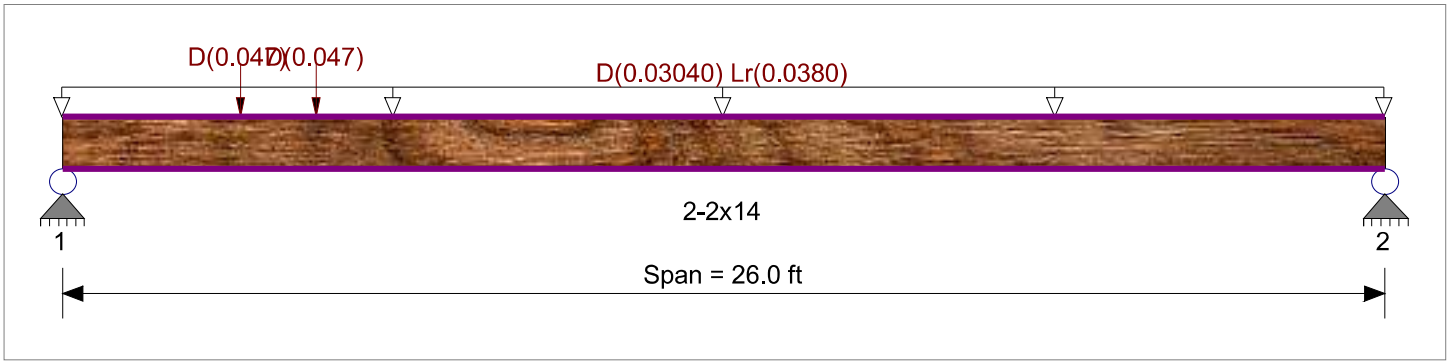
DESCRIPTION: --EF-3, 4, 5--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1000 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1000 psi	Ebend- xx	1700 ksi
		Fc - Prll	1500 psi	Eminbend - xx	620 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade :	No.1	Fv	180 psi		
		Ft	675 psi	Density	31.21 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 1.90 ft
Point Load : D = 0.0470 k @ 3.50 ft
Point Load : D = 0.0470 k @ 5.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.727 : 1	Maximum Shear Stress Ratio	=	0.150 : 1
Section used for this span		**2-2x14**	Section used for this span		**2-2x14**
fb: Actual	=	817.66 psi	fv: Actual	=	33.83 psi
F'b	=	1,125.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	12.810 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.398 in	Ratio =	784	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.744 in	Ratio =	419	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.468	0.103	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.00	2.77	379.0	810.0	0.44	16.7	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.727	0.150	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	5.98	817.7	1,125.0	0.90	33.8	225.0
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.629	0.131	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	5.18	708.0	1,125.0	0.78	29.5	225.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --EF-3, 4, 5--

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.158	0.035	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.00	1.66	227.4	1,440.0	0.27	10.0	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.7441	13.000		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.968	0.905
Max Upward from Load Combinations	0.968	0.905
Max Upward from Load Cases	0.494	0.494
D Only	0.474	0.411
+D+Lr	0.968	0.905
+D+0.750Lr	0.844	0.781
+0.60D	0.284	0.246
Lr Only	0.494	0.494

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --EF-6--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

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

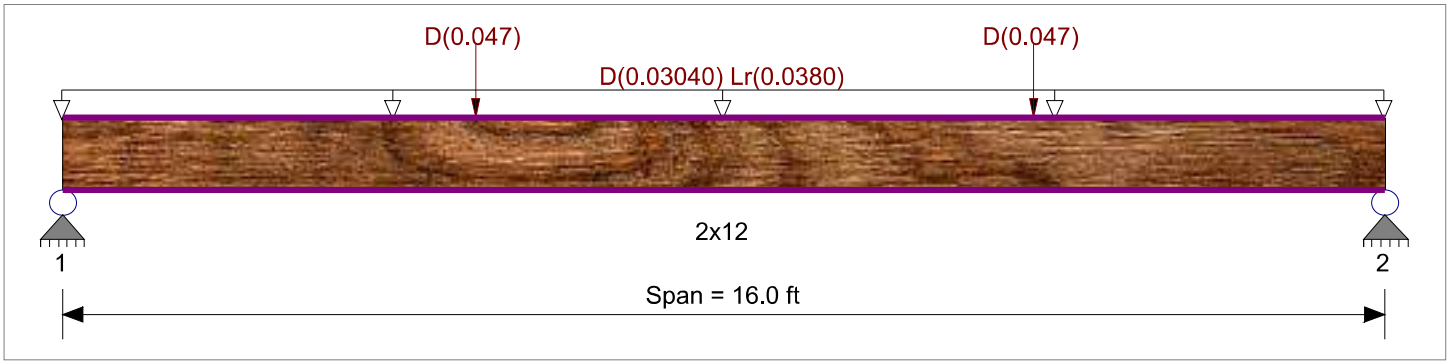
Wood Species : Douglas Fir-Larch
Wood Grade : No.1

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

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,500.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	675.0 psi

E : Modulus of Elasticity

Ebend- xx	1,700.0 ksi
Eminbend - xx	620.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 1.90 ft
Point Load : D = 0.0470 k @ 11.750 ft
Point Load : D = 0.0470 k @ 5.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.730 : 1	Maximum Shear Stress Ratio	=	0.210 : 1
Section used for this span		2x12	Section used for this span		2x12
fb: Actual	=	912.57 psi	fv: Actual	=	47.33 psi
F'b	=	1,250.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	7.942 ft	Location of maximum on span	=	15.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.186 in	Ratio =	1030 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.371 in	Ratio =	517 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.502	0.145	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.19	451.4	900.0	0.26	23.5	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.730	0.210	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.41	912.6	1,250.0	0.53	47.3	225.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.638	0.184	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.10	797.3	1,250.0	0.47	41.4	225.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --EF-6--

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.169	0.049	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.71	270.8	1,600.0	0.16	14.1	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.3707	8.058		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.592	0.596
Max Upward from Load Combinations	0.592	0.596
Max Upward from Load Cases	0.304	0.304
D Only	0.288	0.292
+D+Lr	0.592	0.596
+D+0.750Lr	0.516	0.520
+0.60D	0.173	0.175
Lr Only	0.304	0.304

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

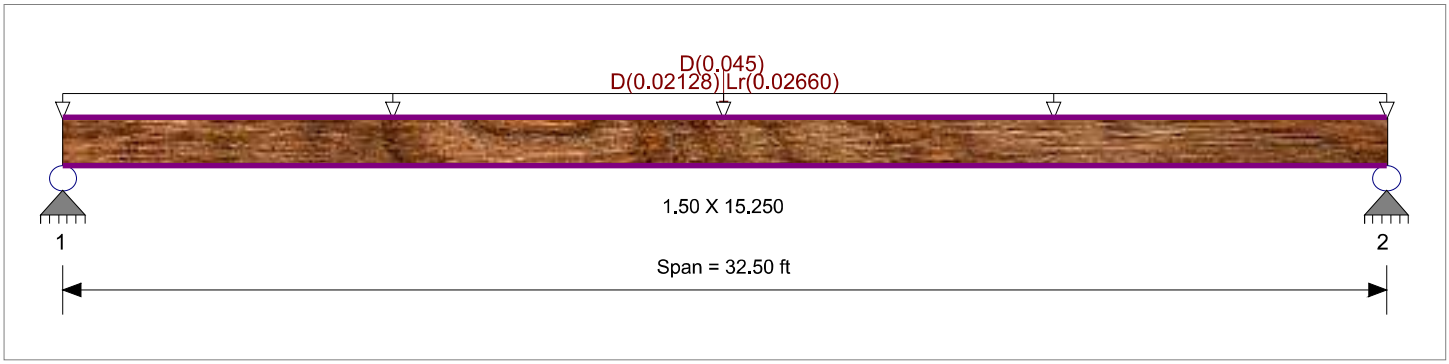
DESCRIPTION: --CU-1, 2, 3--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,250.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,200.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,550.0 psi	Eminbend - xx	660.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1 & Better	Fv	180.0 psi		
		Ft	800.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 1.330 ft
Point Load : D = 0.0450 k @ 16.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.981 : 1	Maximum Shear Stress Ratio	=	0.217 : 1
Section used for this span		**1.50 X 15.250**	Section used for this span		**1.50 X 15.250**
fb: Actual	=	1,380.23 psi	fv: Actual	=	48.77 psi
F'b	=	1,406.25 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	16.250 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.842 in	Ratio =	463 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	1.585 in	Ratio =	246 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.647	0.139	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.00	3.18	655.4	1,012.5	0.34	22.5	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.981	0.217	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	6.69	1,380.2	1,406.3	0.74	48.8	225.0
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 32.50 ft	1	0.853	0.188	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	5.81	1,199.0	1,406.3	0.64	42.2	225.0
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-1, 2, 3--

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 32.50 ft	1	0.218	0.047	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.00	1.91	393.2	1,800.0	0.21	13.5	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.5851	16.369		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.801	0.801
Max Upward from Load Combinations	0.801	0.801
Max Upward from Load Cases	0.432	0.432
D Only	0.368	0.368
+D+Lr	0.801	0.801
+D+0.750Lr	0.692	0.692
+0.60D	0.221	0.221
Lr Only	0.432	0.432

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-4, 7-

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

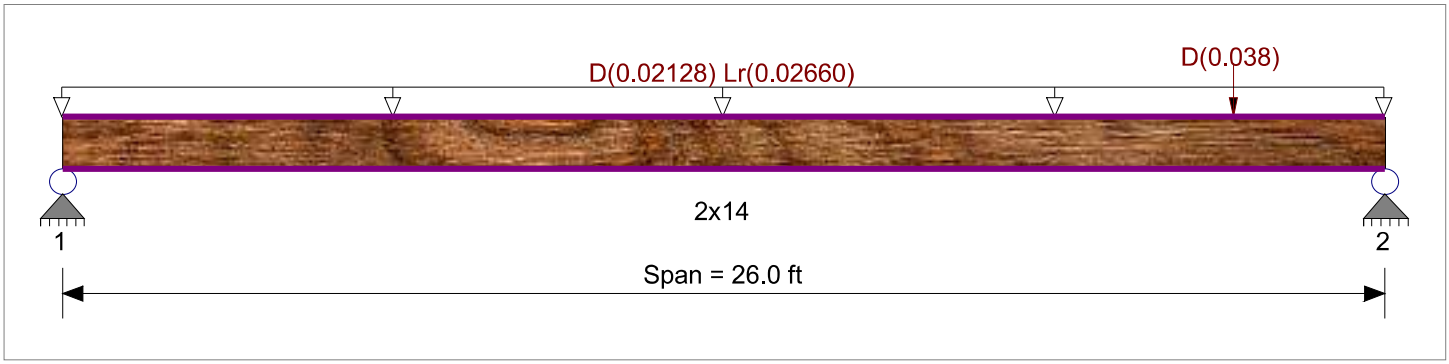
Material Properties

Analysis Method :	Allowable Stress Design
Load Combination :	ASCE 7-16
Wood Species :	Douglas Fir-Larch
Wood Grade :	No.1
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,500.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	675.0 psi

E : Modulus of Elasticity

Ebend- xx	1,700.0 ksi
Eminbend - xx	620.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 1.330 ft
Point Load : D = 0.0380 k @ 23.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.997 : 1	Maximum Shear Stress Ratio	=	0.203 : 1
Section used for this span		2x14	Section used for this span		2x14
fb: Actual	=	1,121.81 psi	fv: Actual	=	45.74 psi
F'b	=	1,125.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	13.095 ft	Location of maximum on span	=	24.956 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.557 in	Ratio =	560 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	1.018 in	Ratio =	306 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.626	0.134	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.00	1.86	507.3	810.0	0.29	21.7	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.997	0.203	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	4.10	1,121.8	1,125.0	0.61	45.7	225.0
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.861	0.177	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	3.54	968.2	1,125.0	0.53	39.7	225.0
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-4, 7-

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 26.0 ft	1	0.211	0.045	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.00	1.11	304.4	1,440.0	0.17	13.0	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.0184	13.095		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.627	0.656
Max Upward from Load Combinations	0.627	0.656
Max Upward from Load Cases	0.346	0.346
D Only	0.281	0.310
+D+Lr	0.627	0.656
+D+0.750Lr	0.540	0.570
+0.60D	0.169	0.186
Lr Only	0.346	0.346

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-5, 6-

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

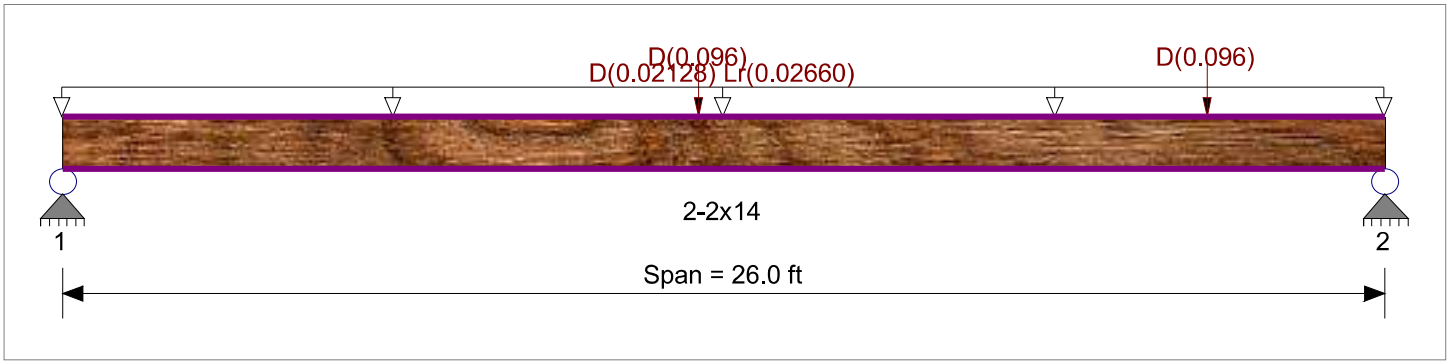
Material Properties

Analysis Method :	Allowable Stress Design
Load Combination :	ASCE 7-16
Wood Species :	Douglas Fir-Larch
Wood Grade :	No.1
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,500.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	675.0 psi

E : Modulus of Elasticity

Ebend- xx	1,700.0 ksi
Eminbend - xx	620.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 1.330 ft
Point Load : D = 0.0960 k @ 12.50 ft
Point Load : D = 0.0960 k @ 22.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.586 : 1	Maximum Shear Stress Ratio	=	0.118 : 1
Section used for this span		2-2x14	Section used for this span		2-2x14
fb: Actual	=	659.49 psi	fv: Actual	=	26.48 psi
F'b	=	1,125.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	12.526 ft	Location of maximum on span	=	24.956 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.278 in	Ratio =	1121 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.587 in	Ratio =	531 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.435	0.089	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.00	2.58	352.6	810.0	0.38	14.5	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.586	0.118	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	4.82	659.5	1,125.0	0.70	26.5	225.0
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.518	0.104	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	4.26	582.8	1,125.0	0.62	23.5	225.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-5, 6-

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 26.0 ft	1	0.147	0.030	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.00	1.55	211.6	1,440.0	0.23	8.7	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.5869	13.095		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.685	0.752
Max Upward from Load Combinations	0.685	0.752
Max Upward from Load Cases	0.346	0.406
D Only	0.339	0.406
+D+Lr	0.685	0.752
+D+0.750Lr	0.599	0.665
+0.60D	0.204	0.244
Lr Only	0.346	0.346

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-8-

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

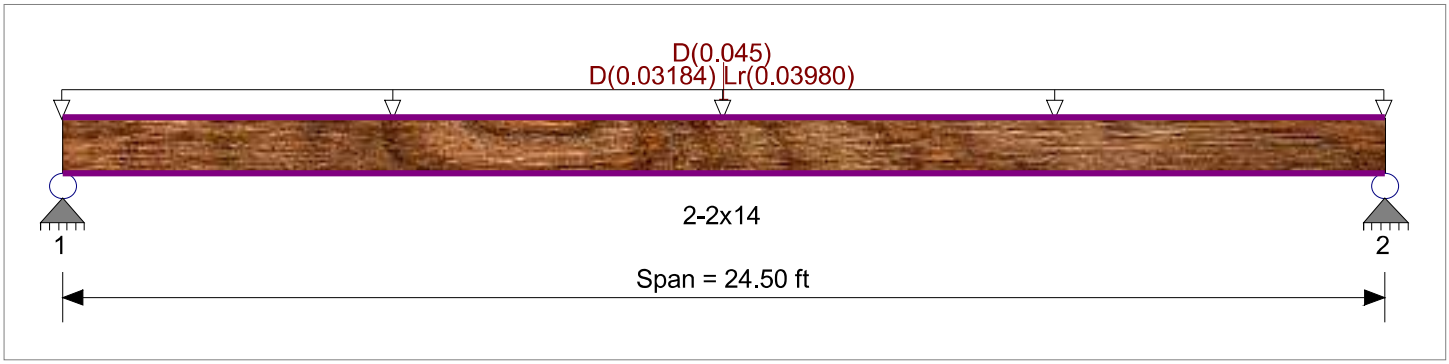
Material Properties

Analysis Method :	Allowable Stress Design
Load Combination :	ASCE 7-16
Wood Species :	Douglas Fir-Larch
Wood Grade :	No.1
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,500.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	675.0 psi

E : Modulus of Elasticity

Ebend- xx	1,700.0 ksi
Eminbend - xx	620.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 1.990 ft
Point Load : D = 0.0450 k @ 12.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.687 : 1	Maximum Shear Stress Ratio	=	0.138 : 1
Section used for this span		2-2x14	Section used for this span		2-2x14
fb: Actual	=	772.49 psi	fv: Actual	=	31.06 psi
F'b	=	1,125.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	12.250 ft	Location of maximum on span	=	23.427 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.328 in	Ratio =	895 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.615 in	Ratio =	477 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 24.50 ft	1	0.450	0.088	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.00	2.66	364.3	810.0	0.38	14.3	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 24.50 ft	1	0.687	0.138	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	5.65	772.5	1,125.0	0.82	31.1	225.0
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 24.50 ft	1	0.596	0.119	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.00	4.90	670.4	1,125.0	0.71	26.9	225.0
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-8-

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 24.50 ft	1	0.152	0.030	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.00	1.60	218.6	1,440.0	0.23	8.6	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.6151	12.339		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.900	0.900
Max Upward from Load Combinations	0.900	0.900
Max Upward from Load Cases	0.488	0.488
D Only	0.413	0.413
+D+Lr	0.900	0.900
+D+0.750Lr	0.778	0.778
+0.60D	0.248	0.248
Lr Only	0.488	0.488

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-9--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 1.330 ft
Point Load : D = 0.050 k @ 12.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.397 : 1	Maximum Shear Stress Ratio	=	0.137 : 1
Section used for this span		2x12	Section used for this span		2x12
fb: Actual	=	496.39 psi	fv: Actual	=	30.86 psi
F'b	=	1,250.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	7.409 ft	Location of maximum on span	=	13.600 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.088 in	Ratio =	1978 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.166 in	Ratio =	1050 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 14.50 ft	1	0.257	0.098	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.61	231.5	900.0	0.18	15.8	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 14.50 ft	1	0.397	0.137	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.31	496.4	1,250.0	0.35	30.9	225.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 14.50 ft	1	0.344	0.120	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.13	430.1	1,250.0	0.30	27.1	225.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-9--

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 14.50 ft	1	0.087	0.033	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.37	138.9	1,600.0	0.11	9.5	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.1657	7.303		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.354	0.390
Max Upward from Load Combinations	0.354	0.390
Max Upward from Load Cases	0.193	0.197
D Only	0.161	0.197
+D+Lr	0.354	0.390
+D+0.750Lr	0.306	0.342
+0.60D	0.097	0.118
Lr Only	0.193	0.193

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-11, 12--

CODE REFERENCES

Calculations per NDS 2018, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

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



Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0160, Lr = 0.020 ksf, Tributary Width = 1.330 ft
Point Load : D = 0.0250 k @ 10.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.841 : 1	Maximum Shear Stress Ratio	=	0.186 : 1
Section used for this span		2x12	Section used for this span		2x12
fb: Actual	=	1,050.79 psi	fv: Actual	=	41.88 psi
F'b	=	1,250.00 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	10.500 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.387 in	Ratio =	651 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.724 in	Ratio =	347 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 21.0 ft	1	0.550	0.119	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.30	494.7	900.0	0.22	19.2	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 21.0 ft	1	0.841	0.186	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.77	1,050.8	1,250.0	0.47	41.9	225.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 21.0 ft	1	0.729	0.161	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.40	911.8	1,250.0	0.41	36.2	225.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --CU-11, 12--

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 21.0 ft	1	0.186	0.040	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.78	296.8	1,600.0	0.13	11.5	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.7242	10.577		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.515	0.515
Max Upward from Load Combinations	0.515	0.515
Max Upward from Load Cases	0.279	0.279
D Only	0.236	0.236
+D+Lr	0.515	0.515
+D+0.750Lr	0.445	0.445
+0.60D	0.142	0.142
Lr Only	0.279	0.279

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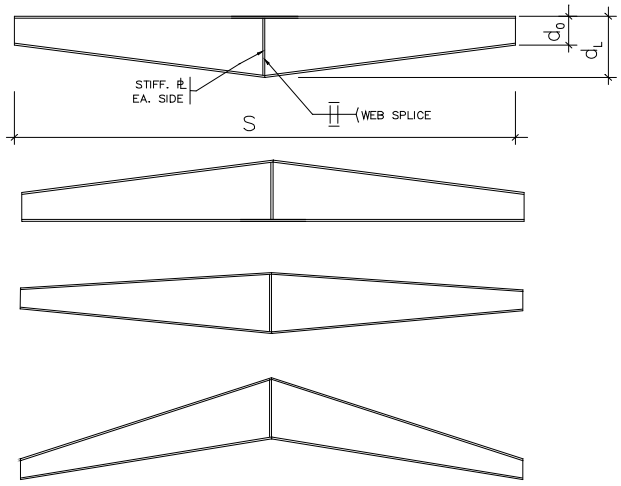
PAGE :
DESIGN BY :
REVIEW BY :

Web Tapered Girder Design Based on AISC-ASD 9th, Appendix F

INPUT DATA & DESIGN SUMMARY

STEEL YIELD STRESS	F_y =	36	ksi
SIMPLY SUPPORTED SPAN	S =	75	ft
SUPERIMPOSED DEAD LOAD	DL =	0.54	kips / ft
LIVE LOAD	LL =	0.54	kips / ft
FLANGE WIDTH	b_f =	16	in
FLANGE THICKNESS	t_f =	0.75	in
WEB THICKNESS	t_w =	0.5	in
HEEL DEPTH	d_0 =	27	in
MID-SPAN DEPTH	d_L =	48	in
DISTANCE BETWEEN STIFFENERS	a =	30	ft
UNBRACED LENGTH / PURLIN SPACING	L =	1.33	ft

(Diaphragm is not bracing member. L is different with " / " in F1.3, pg 5-47)

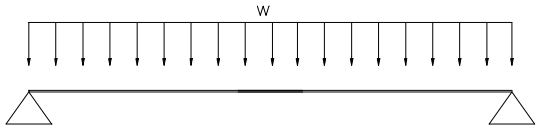


THE GIRDER DESIGN IS ADEQUATE.

ANALYSIS

TOTAL SUPERIMPOSED GRAVITY LOAD

w = DL + LL = 1.080 kips / ft



ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma} = \begin{cases} \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60 F_y \ , \ for \ F_{b\gamma} > F_y/3 \\ B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \ , \ for \ F_{b\gamma} \le F_y/3 \end{cases}$$ = 21.60 ksi

where A_f = 12.00 in^2

γ = MIN[(d_L - d_0) / d_0 , 0.268 L/d_0, 6.0] = 0.16

$A_{To} = t_f b_f + d_0 t_w / 6$ = 14.25 in^2

$I_{To} = (t_f b_f^3 + d_0 t_w^3 / 6) / 12$ = 256 in^4

$$r_{To} = \sqrt{\frac{I_{To}}{A_{To}}} = 4.24 \text{ in}$$

$$h_S = 1.0 + 0.0230\gamma\sqrt{\frac{Ld_0}{A_f}} = 1.02 \text{ in}$$

$$h_w = 1.0 + 0.00385\gamma\sqrt{\frac{L}{r_{To}}} = 1.00 \text{ in}$$

$$F_{s\gamma} = \frac{12000}{h_S L d_O / A_f} = 327.03 \text{ ksi}$$

$$F_{w\gamma} = \frac{170000}{(h_w L / r_{To})^2} = \#\#\#\#\#\# \text{ ksi}$$

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}} = 1.74$$

ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v = \begin{cases} 0.4F_y \ , \ for \ h/t_w \le 380\sqrt{F_y} \\ \frac{C_v F_y}{2.89} \le 0.4F_y \ , \ for \ h/t_w > 380\sqrt{F_y} \end{cases}$$ = 9.86 ksi

where $h = d_L - 2t_f$ = 47 in

h/t_w = 93 > 380 / $F_y^{0.5}$ = 63

$$k_v = \begin{cases} 4.0 + \frac{5.34}{(a/h)^2} \ , \ for \ a/h \le 1.0 \\ 5.34 + \frac{4.0}{(a/h)^2} \ , \ for \ a/h > 1.0 \end{cases}$$ = 5.41

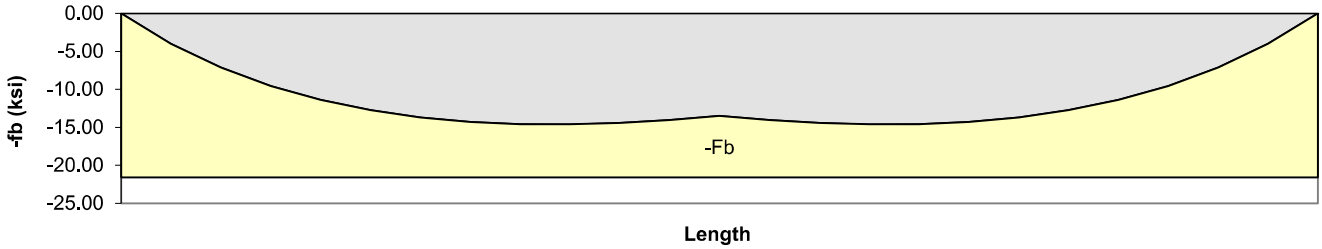
$$C_v = \begin{cases} \frac{45000 k_v}{F_y (h/t_w)^2} \ , \ for \ C_v \le 0.8 \\ \frac{190}{h/t_w}\sqrt{\frac{k_v}{F_y}} \ , \ for \ C_v > 0.8 \end{cases}$$ = 0.79

(cont'd)

CHECK EACH SECTION CAPACITIES

Section	HEEL	1/24 **S**	1/12 **S**	1/8 **S**	1/6 **S**	5/24 **S**	1/4 **S**	7/24 **S**	1/3 **S**	3/8 **S**	5/12 **S**	11/24 **S**	MID
Distance	0	3.13	6.25	9.38	12.50	15.63	18.75	21.88	25.00	28.13	31.25	34.38	37.50
d (in)	27	29	31	32	34	36	38	39	41	43	45	46	48
I (in^4)	5194	5950	6764	7638	8574	9572	10635	11763	12958	14221	15553	16957	18432
Wt (plf)	125.1	128.0	131.0	134.0	137.0	139.9	142.9	145.9	148.9	151.8	154.8	157.8	160.8
V (kips)	45.86	42.09	38.31	34.52	30.72	26.91	23.10	19.27	15.44	11.59	7.74	3.87	0.00
M (ft-k)	0	137	263	377	479	569	647	713	767	810	840	858	864
f_v (ksi)	3.40	2.93	2.51	2.14	1.81	1.51	1.23	0.98	0.75	0.54	0.35	0.17	0.00
f_b (ksi)	0.00	3.98	7.12	9.55	11.39	12.75	13.69	14.28	14.57	14.60	14.42	14.04	13.50

BENDING STRESS

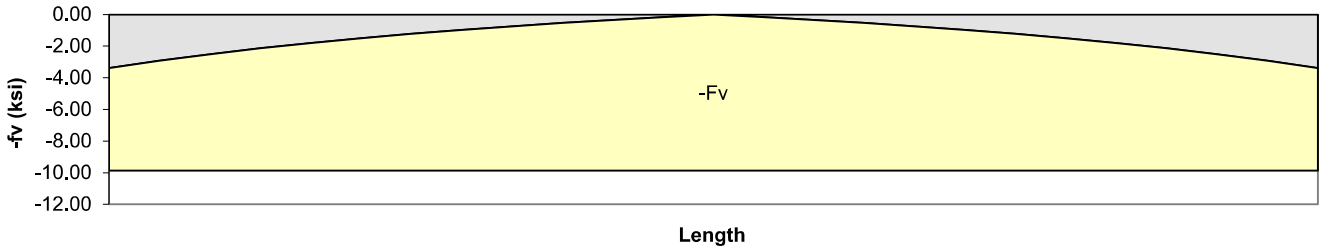


$f_{b,max}$ = 14.60 ksi @ 28.13 ft, from heel.

< F_b = 21.60 ksi

[Satisfactory]

SHEAR STRESS



$f_{v,max}$ = 3.40 ksi @ 0.00 ft, from heel.

< F_v = 9.86 ksi [Satisfactory]

DETERMINE DEFLECTION AT MID SPAN

$$\Delta_{DL} = \int_0^S \frac{m_{DL}\, m_{unit}}{E\, I_S} ds = \quad 1.28 \text{ in (Span / 704)}$$

(for camber, self Wt included.)

$$\Delta_{LL} = \int_0^S \frac{m_{LL}\, m_{unit}}{E\, I_S} ds = \quad 1.00 \text{ in (Span / 897)}$$

where E = 29000 ksi

Section	HEEL	1/24 **S**	1/12 **S**	1/8 **S**	1/6 **S**	5/24 **S**	1/4 **S**	7/24 **S**	1/3 **S**	3/8 **S**	5/12 **S**	11/24 **S**	MID
Distance	0.00	3.13	6.25	9.38	12.50	15.63	18.75	21.88	25.00	28.13	31.25	34.38	37.50
I_S (in^4)	5194	5950	6764	7638	8574	9572	10635	11763	12958	14221	15553	16957	18432
m_{DL} (ft-k)	0	77	147	211	268	318	362	399	430	454	471	481	484
m_{LL} (ft-k)	0	61	116	166	211	250	285	314	338	356	369	377	380
m_{unit}	0.00	1.56	3.13	4.69	6.25	7.81	9.38	10.94	12.50	14.06	15.63	17.19	18.75
$\Delta_{DL,S}$	0.00	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.09	0.09
$\Delta_{LL,S}$	0.00	0.00	0.01	0.01	0.02	0.03	0.04	0.05	0.06	0.06	0.07	0.07	0.07

Techincal Reference:

1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.

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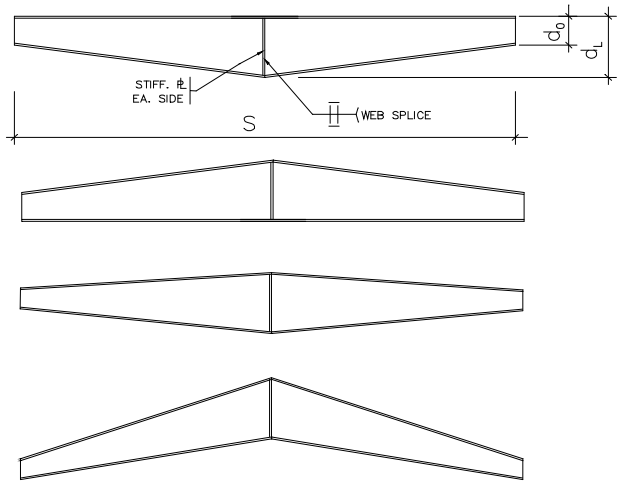
PAGE :
DESIGN BY :
REVIEW BY :

Web Tapered Girder Design Based on AISC-ASD 9th, Appendix F

INPUT DATA & DESIGN SUMMARY

STEEL YIELD STRESS	F_y =	36	ksi
SIMPLY SUPPORTED SPAN	S =	75	ft
SUPERIMPOSED DEAD LOAD	DL =	0.54	kips / ft
LIVE LOAD	LL =	0.54	kips / ft
FLANGE WIDTH	b_f =	16	in
FLANGE THICKNESS	t_f =	0.75	in
WEB THICKNESS	t_w =	0.5	in
HEEL DEPTH	d_0 =	18	in
MID-SPAN DEPTH	d_L =	60	in
DISTANCE BETWEEN STIFFENERS	a =	8	ft
UNBRACED LENGTH / PURLIN SPACING	L =	1.33	ft

(Diaphragm is not bracing member. L is different with " / " in F1.3, pg 5-47)

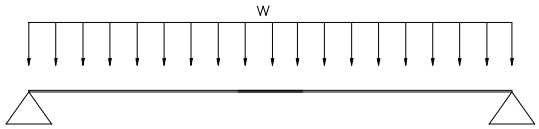


THE GIRDER DESIGN IS ADEQUATE.

ANALYSIS

TOTAL SUPERIMPOSED GRAVITY LOAD

w = DL + LL = 1.080 kips / ft



ALLOWABLE FLEXURAL STRESS (APP. F7.4, pg 5-103)

$$F_{b\gamma} = \begin{cases} \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60 F_y \ , \ for \ F_{b\gamma} > F_y/3 \\ B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \ , \ for \ F_{b\gamma} \le F_y/3 \end{cases}$$ = 21.60 ksi

where A_f = 12.00 in^2

γ = MIN[(d_L - d_0) / d_0 , 0.268 L/d_0, 6.0] = 0.24

$A_{To} = t_f b_f + d_0 t_w / 6$ = 13.50 in^2

$I_{To} = (t_f b_f^3 + d_0 t_w^3 / 6) / 12$ = 256 in^4

$$r_{To} = \sqrt{\frac{I_{To}}{A_{To}}} = 4.35 \text{ in}$$

$$h_S = 1.0 + 0.0230\gamma\sqrt{\frac{Ld_0}{A_f}} = 1.03 \text{ in}$$

$$h_w = 1.0 + 0.00385\gamma\sqrt{\frac{L}{r_{To}}} = 1.00 \text{ in}$$

$$F_{s\gamma} = \frac{12000}{h_S L d_O / A_f} = 488.20 \text{ ksi}$$

$$F_{w\gamma} = \frac{170000}{(h_w L / r_{To})^2} = \#\#\#\#\#\# \text{ ksi}$$

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}} = 1.73$$

ALLOWABLE SHEAR STRESS (F4, pg 5-49)

$$F_v = \begin{cases} 0.4F_y \ , \ for \ h/t_w \le 380\sqrt{F_y} \\ \frac{C_v F_y}{2.89} \le 0.4F_y \ , \ for \ h/t_w > 380\sqrt{F_y} \end{cases}$$ = 8.81 ksi

where $h = d_L - 2t_f$ = 59 in

h/t_w = 117 > 380 / $F_y^{0.5}$ = 63

$$k_v = \begin{cases} 4.0 + \frac{5.34}{(a/h)^2} \ , \ for \ a/h \le 1.0 \\ 5.34 + \frac{4.0}{(a/h)^2} \ , \ for \ a/h > 1.0 \end{cases}$$ = 6.83

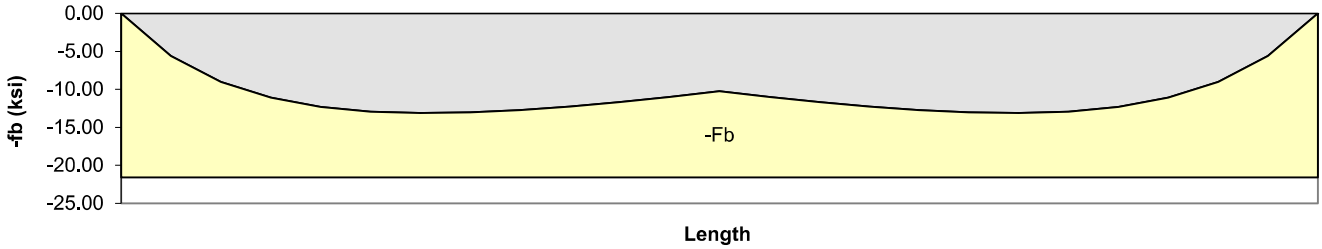
$$C_v = \begin{cases} \frac{45000 k_v}{F_y (h/t_w)^2} \ , \ for \ C_v \le 0.8 \\ \frac{190}{h/t_w}\sqrt{\frac{k_v}{F_y}} \ , \ for \ C_v > 0.8 \end{cases}$$ = 0.71

(cont'd)

CHECK EACH SECTION CAPACITIES

Section	HEEL	1/24 **S**	1/12 **S**	1/8 **S**	1/6 **S**	5/24 **S**	1/4 **S**	7/24 **S**	1/3 **S**	3/8 **S**	5/12 **S**	11/24 **S**	MID
Distance	0	3.13	6.25	9.38	12.50	15.63	18.75	21.88	25.00	28.13	31.25	34.38	37.50
d (in)	18	22	25	29	32	36	39	43	46	50	53	57	60
I (in^4)	2187	3188	4401	5838	7509	9426	11598	14036	16752	19755	23057	26669	30600
Wt (plf)	109.7	115.7	121.6	127.6	133.6	139.5	145.5	151.4	157.4	163.3	169.3	175.2	181.2
V (kips)	45.96	42.23	38.48	34.72	30.93	27.13	23.31	19.47	15.62	11.74	7.85	3.93	0.00
M (ft-k)	0	138	264	378	481	572	650	717	772	815	845	864	870
f_v (ksi)	5.11	3.93	3.08	2.44	1.93	1.53	1.20	0.92	0.68	0.47	0.30	0.14	0.00
f_b (ksi)	0.00	5.58	8.99	11.08	12.29	12.92	13.12	13.03	12.72	12.25	11.66	10.98	10.24

BENDING STRESS

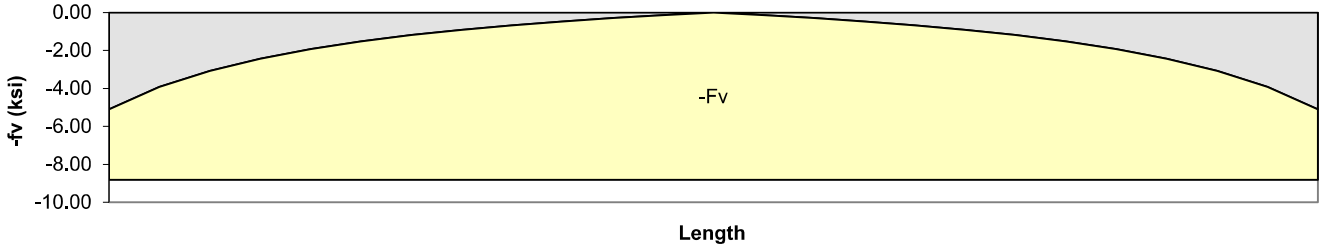


$f_{b,max}$ = 13.12 ksi @ 18.75 ft, from heel.

< F_b = 21.60 ksi

[Satisfactory]

SHEAR STRESS



$f_{v,max}$ = 5.11 ksi @ 0.00 ft, from heel.

< F_v = 8.81 ksi [Satisfactory]

DETERMINE DEFLECTION AT MID SPAN

$$\Delta_{DL} = \int_0^S \frac{m_{DL}\, m_{unit}}{E\, I_S} ds =$$ 1.06 in (Span / 848)
(for camber, self Wt included.)

$$\Delta_{LL} = \int_0^S \frac{m_{LL}\, m_{unit}}{E\, I_S} ds =$$ 0.83 in (Span / 1090)

where E = 29000 ksi

Section	HEEL	1/24 **S**	1/12 **S**	1/8 **S**	1/6 **S**	5/24 **S**	1/4 **S**	7/24 **S**	1/3 **S**	3/8 **S**	5/12 **S**	11/24 **S**	MID
Distance	0.00	3.13	6.25	9.38	12.50	15.63	18.75	21.88	25.00	28.13	31.25	34.38	37.50
I_S (in^4)	2187	3188	4401	5838	7509	9426	11598	14036	16752	19755	23057	26669	30600
m_{DL} (ft-k)	0	77	148	212	270	321	366	404	435	459	476	487	490
m_{LL} (ft-k)	0	61	116	166	211	250	285	314	338	356	369	377	380
m_{unit}	0.00	1.56	3.13	4.69	6.25	7.81	9.38	10.94	12.50	14.06	15.63	17.19	18.75
$\Delta_{DL,S}$	0.00	0.00	0.01	0.03	0.04	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06
$\Delta_{LL,S}$	0.00	0.00	0.01	0.02	0.03	0.04	0.04	0.04	0.05	0.05	0.05	0.05	0.04

Techincal Reference:

1. AISC: "Manual of Steel construction 9th", American Institute of Steel Construction, 1990.

Steel Column

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --New steel column--

Code References

Calculations per AISC 360-16, IBC 2021
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS12x12x1/2**

Analysis Method :	Load Resistance Factor	Overall Column Height	20.50 ft
Steel Stress Grade		Top & Bottom Fixity	Top & Bottom Pinned
Fy : Steel Yield	46.0 ksi	Brace condition :	
E : Elastic Bending Modulus	29,000.0 ksi		

Unbraced Length for buckling ABOUT X-X Axis = 20.50 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 20.50 ft, K = 1.0

Applied Loads

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

Column self weight included : 1,556.83 lbs * Dead Load Factor
AXIAL LOADS . . .
Loadings at GL3/GLF: Axial Load at 20.50 ft, D = 39.20, LR = 18.70, W = 37.60 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = 0.1359 : 1

Load Combination	+1.20D+1.60Lr+0.50W	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pu	97.628	k
0.9 * Pn	718.50	k
Mu-x	0.0	k-ft
0.9 * Mn-x :	309.120	k-ft
Mu-y	0.0	k-ft
0.9 * Mn-y :	309.120	k-ft

PASS Maximum Shear Stress Rati 0.0 : 1

Load Combination	0.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	0.0	k
Vn * Phi : Allowable	0.0	k

Maximum Load Reactions . .

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

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination :

Along X-X 0.0 in at 0.0 ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.079	PASS	0.00 ft	1.00	1.00	52.56	52.56	0.000	PASS	0.00 ft
+1.20D+0.50Lr	0.081	PASS	0.00 ft	1.00	1.00	52.56	52.56	0.000	PASS	0.00 ft
+1.20D	0.068	PASS	0.00 ft	1.00	1.00	52.56	52.56	0.000	PASS	0.00 ft
+1.20D+1.60Lr	0.110	PASS	0.00 ft	1.00	1.00	52.56	52.56	0.000	PASS	0.00 ft
+1.20D+1.60Lr+0.50W	0.136	PASS	0.00 ft	1.00	1.00	52.56	52.56	0.000	PASS	0.00 ft
+1.20D+0.50W	0.094	PASS	0.00 ft	1.00	1.00	52.56	52.56	0.000	PASS	0.00 ft
+1.20D+0.50Lr+W	0.133	PASS	0.00 ft	1.00	1.00	52.56	52.56	0.000	PASS	0.00 ft
+1.20D+W	0.120	PASS	0.00 ft	1.00	1.00	52.56	52.56	0.000	PASS	0.00 ft
+0.90D+W	0.103	PASS	0.00 ft	1.00	1.00	52.56	52.56	0.000	PASS	0.00 ft
+0.90D	0.051	PASS	0.00 ft	1.00	1.00	52.56	52.56	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction k @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments k-ft @ Base	@ Top
D Only	40.757								
+D+Lr	59.457								
+D+0.750Lr	54.782								
+D+0.60W	63.317								
+D+0.750Lr+0.450W	71.702								
+D+0.450W	57.677								

Steel Column

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --New steel column--

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
+0.60D+0.60W	47.014									
+0.60D	24.454									
Lr Only	18.700									
W Only	37.600									

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
Axial @ Base	Maximum	71.702									
"	Minimum	18.700									
Reaction, X-X Axis Base	Maximum	40.757									
"	Minimum	40.757									
Reaction, Y-Y Axis Base	Maximum	40.757									
"	Minimum	40.757									
Reaction, X-X Axis Top	Maximum	40.757									
"	Minimum	40.757									
Reaction, Y-Y Axis Top	Maximum	40.757									
"	Minimum	40.757									
Moment, X-X Axis Base	Maximum	40.757									
"	Minimum	40.757									
Moment, Y-Y Axis Base	Maximum	40.757									
"	Minimum	40.757									
Moment, X-X Axis Top	Maximum	40.757									
"	Minimum	40.757									
Moment, Y-Y Axis Top	Maximum	40.757									
"	Minimum	40.757									

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir		Distance		Max. Deflection in Y dir		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.60W	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.450W	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.450W	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D+0.60W	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
W Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : HSS12x12x1/2

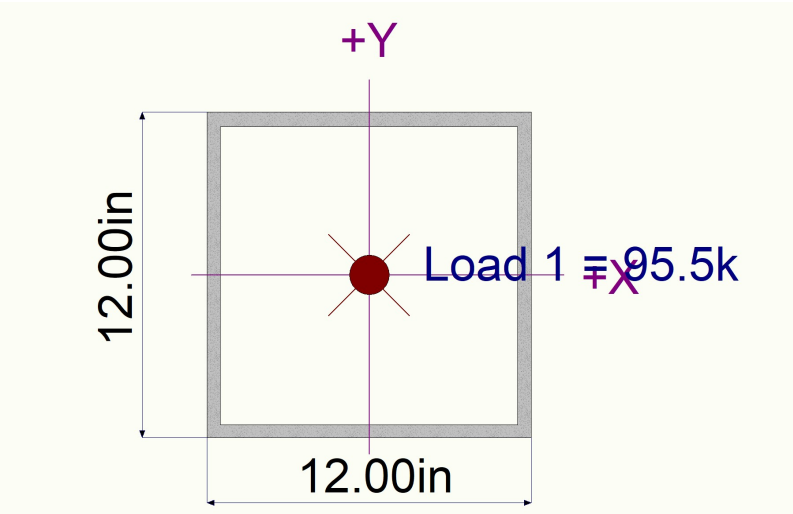
Depth	=	12.000 in	I xx	=	457.00 in^4	J	=	728.000 in^4	
Design Thick	=	0.465 in	S xx	=	76.20 in^3				
Width	=	12.000 in	R xx	=	4.680 in				
Wall Thick	=	0.500 in	Zx	=	89.600 in^3				
Area	=	20.900 in^2	I yy	=	457.000 in^4	C	=	123.000 in^3	
Weight	=	75.943 plf	S yy	=	76.200 in^3				
			R yy	=	4.680 in				
Ycg	=	0.000 in							

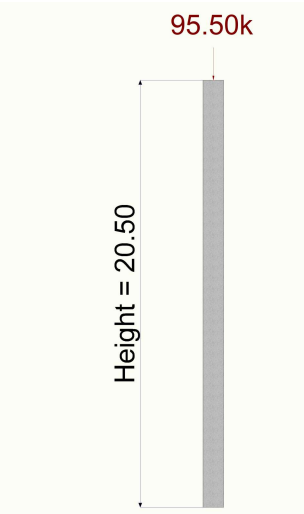
Steel Column

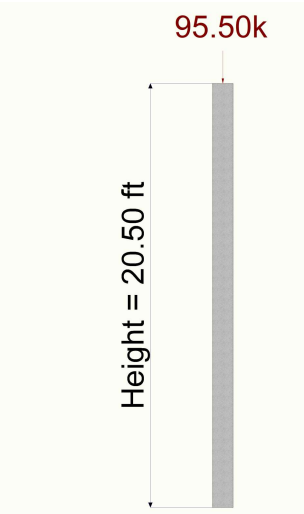
LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --New steel column--

Sketches







General Footing

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --New pad footing (worst case with loadings for pole footing)--

Code References

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

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,605.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00090
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	2.50	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

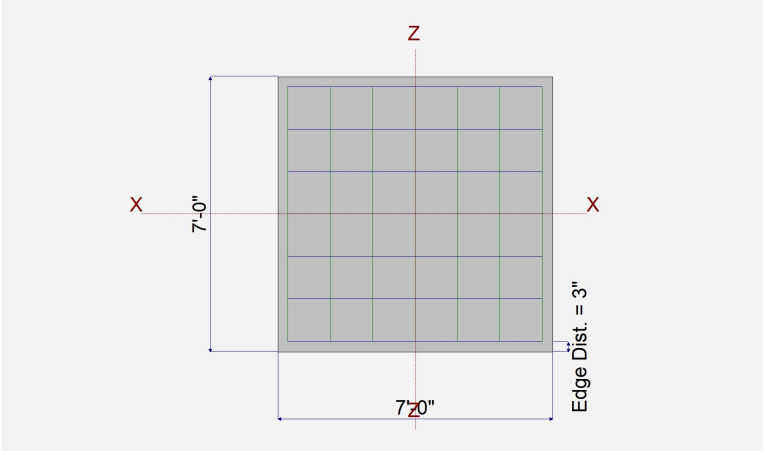
Width parallel to X-X Axis	=	7.0 ft
Length parallel to Z-Z Axis	=	7.0 ft
Footing Thickness	=	18.0 in

Pedestal dimensions...

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

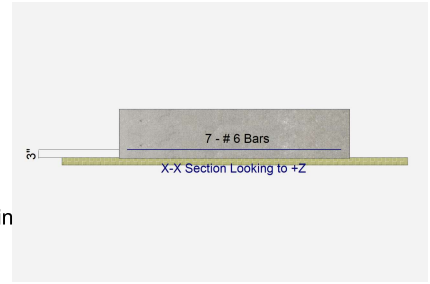
Bandwidth Distribution Check (ACI 15.4.4.2)

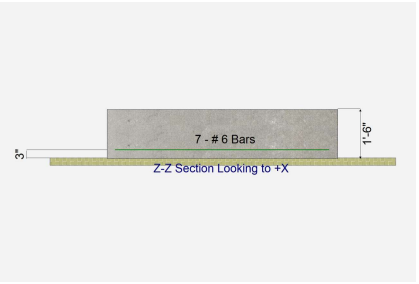
Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a



Bottom Reinforcing

Bars parallel to X-X Axis			
Number of Bars	=		7.0
Reinforcing Bar Size	=	#	6
Bars parallel to Z-Z Axis			
Number of Bars	=		7.0
Reinforcing Bar Size	=	#	6
Rebar Centerline to Edge of Concrete...			
at Bottom of footing	=		3.0 in





General Footing

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --New pad footing (worst case with loadings for pole footing)--

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	41.840	18.955			37.60	0.1260		k
OB : Overburden	=								ksf
M-xx	=	1.560	0.380				28.180		k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.6956	Soil Bearing	1.739 ksf	2.50 ksf	+D+0.750Lr+0.450W about X-X axis
PASS	5.351	Overturning - X-X	20.662 k-ft	110.553 k-ft	+0.60D+0.70E
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	0.4273	Z Flexure (+X) Bot Tens	12.417 k-ft/ft	29.059 k-ft/ft	+1.20D+1.60Lr+0.50W
PASS	0.4273	Z Flexure (-X) Bot Tens	12.417 k-ft/ft	29.059 k-ft/ft	+1.20D+1.60Lr+0.50W
PASS	0.4334	X Flexure (+Z) Bot Tens	12.594 k-ft/ft	29.059 k-ft/ft	+1.20D+1.60Lr+0.50W
PASS	0.4212	X Flexure (-Z) Bot Tens	12.240 k-ft/ft	29.059 k-ft/ft	+1.20D+1.60Lr+0.50W
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	Z Flexure (-X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (-Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.4935	1-way Shear (+X)	25.228 psi	51.118 psi	+1.20D+1.60Lr+0.50W
PASS	0.4935	1-way Shear (-X)	25.228 psi	51.118 psi	+1.20D+1.60Lr+0.50W
PASS	0.5007	1-way Shear (+Z)	25.595 psi	51.118 psi	+1.20D+1.60Lr+0.50W
PASS	0.4863	1-way Shear (-Z)	24.861 psi	51.118 psi	+1.20D+1.60Lr+0.50W
PASS	0.5629	2-way Punching	106.797 psi	189.737 psi	+1.20D+1.60Lr+0.50W

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	2.50	n/a	0.3566	1.044	1.098	n/a	n/a	0.439
X-X, +D+Lr	2.50	n/a	0.3258	1.425	1.492	n/a	n/a	0.597
X-X, +D+0.750Lr	2.50	n/a	0.3319	1.330	1.393	n/a	n/a	0.557
X-X, +D+0.60W	2.50	n/a	0.2494	1.505	1.559	n/a	n/a	0.624
X-X, +D+0.750Lr+0.450W	2.50	n/a	0.2647	1.675	1.739	n/a	n/a	0.696
X-X, +D+0.450W	2.50	n/a	0.2697	1.390	1.444	n/a	n/a	0.578
X-X, +0.60D+0.60W	2.50	n/a	0.2078	1.087	1.119	n/a	n/a	0.448
X-X, +D+0.70E	2.50	n/a	4.857	0.7046	1.442	n/a	n/a	0.577
X-X, +D+0.5250E	2.50	n/a	3.734	0.7895	1.356	n/a	n/a	0.542
X-X, +0.60D+0.70E	2.50	n/a	7.850	0.2868	1.002	n/a	n/a	0.401
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	1.071	1.071	0.428
Z-Z, +D+Lr	2.50	0.0	n/a	n/a	n/a	1.458	1.458	0.583
Z-Z, +D+0.750Lr	2.50	0.0	n/a	n/a	n/a	1.362	1.362	0.545
Z-Z, +D+0.60W	2.50	0.0	n/a	n/a	n/a	1.532	1.532	0.613
Z-Z, +D+0.750Lr+0.450W	2.50	0.0	n/a	n/a	n/a	1.707	1.707	0.683
Z-Z, +D+0.450W	2.50	0.0	n/a	n/a	n/a	1.417	1.417	0.567
Z-Z, +0.60D+0.60W	2.50	0.0	n/a	n/a	n/a	1.103	1.103	0.441
Z-Z, +D+0.70E	2.50	0.0	n/a	n/a	n/a	1.073	1.073	0.429
Z-Z, +D+0.5250E	2.50	0.0	n/a	n/a	n/a	1.073	1.073	0.429
Z-Z, +0.60D+0.70E	2.50	0.0	n/a	n/a	n/a	0.6446	0.6446	0.258

General Footing

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --New pad footing (worst case with loadings for pole footing)--

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D Only	1.560 k-ft	183.741 k-ft	117.783	OK
X-X, +D+Lr	1.940 k-ft	250.084 k-ft	128.909	OK
X-X, +D+0.750Lr	1.845 k-ft	233.498 k-ft	126.557	OK
X-X, +D+0.60W	1.560 k-ft	262.701 k-ft	168.398	OK
X-X, +D+0.750Lr+0.450W	1.845 k-ft	292.718 k-ft	158.655	OK
X-X, +D+0.450W	1.560 k-ft	242.961 k-ft	155.744	OK
X-X, +0.60D+0.60W	0.9360 k-ft	189.205 k-ft	202.142	OK
X-X, +D+0.70E	21.286 k-ft	184.050 k-ft	8.647	OK
X-X, +D+0.5250E	16.355 k-ft	183.973 k-ft	11.249	OK
X-X, +0.60D+0.70E	20.662 k-ft	110.553 k-ft	5.351	OK
Z-Z, D Only	None	0.0 k-ft	Infinity	OK
Z-Z, +D+Lr	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.750Lr	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.60W	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.750Lr+0.450W	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.450W	None	0.0 k-ft	Infinity	OK
Z-Z, +0.60D+0.60W	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.70E	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.5250E	None	0.0 k-ft	Infinity	OK
Z-Z, +0.60D+0.70E	None	0.0 k-ft	Infinity	OK

All units k

Sliding Stability

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

Footing Tension on Bottom

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	7.478	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.40D	7.166	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+0.50Lr	7.608	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+0.50Lr	7.313	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D	6.410	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D	6.142	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+1.60Lr	10.244	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+1.60Lr	9.890	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+1.60Lr+0.50W	12.594	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+1.60Lr+0.50W	12.240	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+0.50W	8.760	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+0.50W	8.492	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+0.50Lr+W	12.308	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+0.50Lr+W	12.013	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+W	11.110	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+W	10.842	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +0.90D+W	9.507	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +0.90D+W	9.307	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+E	8.438	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +1.20D+E	4.145	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +0.90D+E	6.836	+Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
X-X, +0.90D+E	2.610	-Z	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.40D	7.322	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.40D	7.322	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+0.50Lr	7.461	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+0.50Lr	7.461	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D	6.276	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D	6.276	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+1.60Lr	10.067	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+1.60Lr	10.067	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+1.60Lr+0.50W	12.417	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+1.60Lr+0.50W	12.417	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+0.50W	8.626	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK

General Footing

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --New pad footing (worst case with loadings for pole footing)--

Footing Tension on Bottom

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+0.50W	8.626	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+0.50Lr+W	12.161	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+0.50Lr+W	12.161	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+W	10.976	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+W	10.976	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +0.90D+W	9.407	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +0.90D+W	9.407	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+E	6.292	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +1.20D+E	6.292	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +0.90D+E	4.723	-X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK
Z-Z, +0.90D+E	4.723	+X	Bottom	0.1944	ACI 7.6.1.1	0.440	29.059	OK

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	14.88 psi	14.88 psi	14.88 psi	51.12 psi	0.29	OK
+1.20D+0.50Lr	15.16 psi	15.16 psi	15.16 psi	51.12 psi	0.30	OK
+1.20D	12.75 psi	12.75 psi	12.75 psi	51.12 psi	0.25	OK
+1.20D+1.60Lr	20.45 psi	20.45 psi	20.45 psi	51.12 psi	0.40	OK
+1.20D+1.60Lr+0.50W	25.23 psi	25.23 psi	25.23 psi	51.12 psi	0.49	OK
+1.20D+0.50W	17.53 psi	17.53 psi	17.53 psi	51.12 psi	0.34	OK
+1.20D+0.50Lr+W	24.71 psi	24.71 psi	24.71 psi	51.12 psi	0.48	OK
+1.20D+W	22.30 psi	22.30 psi	22.30 psi	51.12 psi	0.44	OK
+0.90D+W	19.11 psi	19.11 psi	19.11 psi	51.12 psi	0.37	OK
+1.20D+E	12.78 psi	12.78 psi	12.78 psi	51.12 psi	0.25	OK
+0.90D+E	9.60 psi	9.60 psi	9.60 psi	51.12 psi	0.19	OK

One Way Shear Z

Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	14.55 psi	15.20 psi	15.20 psi	51.12 psi	0.30	OK
+1.20D+0.50Lr	14.85 psi	15.46 psi	15.46 psi	51.12 psi	0.30	OK
+1.20D	12.47 psi	13.03 psi	13.03 psi	51.12 psi	0.25	OK
+1.20D+1.60Lr	20.09 psi	20.82 psi	20.82 psi	51.12 psi	0.41	OK
+1.20D+1.60Lr+0.50W	24.86 psi	25.60 psi	25.60 psi	51.12 psi	0.50	OK
+1.20D+0.50W	17.25 psi	17.80 psi	17.80 psi	51.12 psi	0.35	OK
+1.20D+0.50Lr+W	24.40 psi	25.01 psi	25.01 psi	51.12 psi	0.49	OK
+1.20D+W	22.02 psi	22.58 psi	22.58 psi	51.12 psi	0.44	OK
+0.90D+W	18.91 psi	19.32 psi	19.32 psi	51.12 psi	0.38	OK
+1.20D+E	8.34 psi	17.23 psi	17.23 psi	51.12 psi	0.34	OK
+0.90D+E	5.22 psi	13.98 psi	13.98 psi	51.12 psi	0.27	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	62.98 psi	189.74 psi	0.33	OK
+1.20D+0.50Lr	64.17 psi	189.74 psi	0.34	OK
+1.20D	53.98 psi	189.74 psi	0.28	OK
+1.20D+1.60Lr	86.59 psi	189.74 psi	0.46	OK
+1.20D+1.60Lr+0.50W	106.80 psi	189.74 psi	0.56	OK
+1.20D+0.50W	74.19 psi	189.74 psi	0.39	OK
+1.20D+0.50Lr+W	104.59 psi	189.74 psi	0.55	OK
+1.20D+W	94.40 psi	189.74 psi	0.50	OK
+0.90D+W	80.91 psi	189.74 psi	0.43	OK
+1.20D+E	54.12 psi	189.74 psi	0.29	OK
+0.90D+E	40.62 psi	189.74 psi	0.21	OK

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Seismic Weight

Roof Weight	Roof DL (psf)	Width (ft)	Length (ft)	Area (sf)	Weight (kips)
GL 1-7/GL A-F	20	180	150	26949	539
GL 1.2-4.8/F-F.2	23	129	9	1127	26
					565

Partition Weight	Partition (psf)	Area (sf)	Weight (kips)
GL 1-7/GL A-G	10	6905	69

Glazing Weight	Glazing (psf)	Length (ft)	Weight (kips)
GL F	10	98	1

Steel Column Weight	5" ϕ 1/4" thick (plf)	Height (ft)	Number of column	Weight (kips)
	15	9	13	2

Pilaster Weight	Pilaster (psf)	length (ft)	Height (ft)	Number of column	Weight (kips)
GL F	22.5	1	9	5	1

North-South Direction Wall						
Wall Weight	Wall (psf)	Length (ft)	H_{roof} (ft)	$H_{parapet}$ (ft)	Half wall weight (kips)	Weight (kips)
GL A	75	180	19	19	128	256
GL A.8	75	16	19	19	11	22
GL C.1/D.9	84	20	13	13	11	22
GL E.7	84	10	13	13	5	11
GL F	75	82	19	19	58	116
					214	427

East -West Direction Wall						
Wall Weight	Wall (psf)	Length (ft)	H_{roof} (ft)	$H_{parapet}$ (ft)	Half wall weight (kips)	Weight (kips)
GL 1	75	150	18	19	113	214
GL 6	75	126	18	19	94	179
GL 6.5	84	38	13	13	21	42
GL 7	84	150	13	15	109	189
					337	623

N-S Seismic Weight of Entire Building (kips)	
Roof DL: GL 1-7/GL A-G	562
Roof DL: GL 1.2-4.8/G-G.2	26
Partition: GL 1-7/GL A-G	69
Glazing: GL G	1
Steel Column	2
Pilaster: GL G	1
Wall : GL A -GL G	427
0.5Wall: GL 1- GL 7	337
	1425

E-W Seismic Weight of Entire Building (kips)	
Roof DL: GL 1-7/GL A-G	562
Roof DL: GL 1.2-4.8/G-G.2	26
Partition: GL 1-7/GL A-G	69
0.5Glazing: GL G	0.5
Steel Column	2
Pilaster: GL G	1
0.5Wall : GL A -GL G	214
Wall: GL 1- GL 7	623
	1497

Use larger value for both direction

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Pseudo Seismic Force (LSP)

ASCE 41 - 17 - Section 7.4.1.3

C_1C_2 =	1.4	ASCE 41-17, Table 7-3
C_m =	1.0	ASCE 41-17, Table 7-4
V=	$C_1C_2C_mS_aW$	ASCE 41-17, Eq. 7-21

	Damage Control	Limited Safety
V (kips)=	2100	3661

Diaphragm analysis

Diaphragm :	1/2" plywood sheathing	
Diaphragm Type:	Wood structural pannel, blocked, chord	ASCE 41-17, Table 12-3
κ =	1 (Damage Control)	ASCE 41-17 - Section 6.2.4
κ =	1 (Limited Safety)	ASCE 41-17 - Section 6.2.4

	Diaphragm Dimension			m-factor	
Seismic direction	Height h (ft)	Width b (ft)	h/b	DC	Ld S
Wood Diaphragms - GL 1-6/A-F , N-S	164	150	1.09	2.25	3.5
Wood Diaphragms - GL 1-6/A-F, E-W	150	164	0.91	2.25	3.5
Wood Diaphragms - GL 6-7/A-A.8 , N-S	16	25	0.63	2.25	3.5
Wood Diaphragms - GL 6-7/A-A.8, E-W	25	16	1.60	2.25	3.5
Wood Diaphragms - GL 6-7/A.8-C.1, N-S	16	37	0.43	2.25	3.5
Wood Diaphragms - GL 6-7/A.8-C.1, E-W	37	16	2.34	2.25	3.5
Wood Diaphragms - GL 1.2-4.8/F-F.2, N-S	129	9	14.71	2	2.75
Wood Diaphragms - GL 1.2-4.8/F-F.2, E-W	9	129	0.07	2.25	3.5
Metal Deck Diaphragms - GL 6-7/D.9-E.7, N-S	10	28	0.36	2.25	3.5
Metal Deck Diaphragms - GL 6-7/D.9-E.7, E-W	28	10	2.75	2.25	3.5

	Q_{UD} (plf)		$m\kappa Q_{CE}$ (plf) [(a)]		
Seismic direction	DC	Ld S	DC	Ld S	
Wood Diaphragms - GL 1-6/A-F , N-S	3854	6699	4928	7665	10d@2", 3", 12"
Wood Diaphragms - GL 1-6/A-F, E-W	4216	7350	4928	7665	10d@2", 3", 12"
Wood Diaphragms - GL 6-7/A-A.8 , N-S	971	1693	2160	3360	10d@6", 6", 12"
Wood Diaphragms - GL 6-7/A-A.8, E-W	608	1060	2160	3360	10d@6", 6", 12"
Wood Diaphragms - GL 6-7/A.8-C.1, N-S	1422	2479	2869	4463	10d@4", 6", 12"
Wood Diaphragms - GL 6-7/A.8-C.1, E-W	608	1060	2160	3360	10d@6", 6", 12"
Wood Diaphragms - GL 1.2-4.8/F-F.2, N-S	141	246	1920	2640	10d@6", 6", 12"
Wood Diaphragms - GL 1.2-4.8/F-F.2, E-W	2077	3621	2869	4463	10d@4", 6", 12"
Metal Deck Diaphragms - GL 6-7/D.9-E.7, N-S	1071	1868	4016	6248	18ga PLB 36 VSC2
Metal Deck Diaphragms - GL 6-7/D.9-E.7, E-W	390	679	4016	6248	18ga PLB 36 VSC2

2015 SDPWS Table 4.2A			
Blocked Diaphragm 1/2" Plywood w/ 10d@6", 6", 12"	**Blocked Diaphragm 1/2" Plywood w/ 10d@4", 6", 12"**	**Blocked Diaphragm 1/2" Plywood w/ 10d@2.5", 4", 12"**	**Blocked Diaphragm 1/2" Plywood w/ 10d@2", 3", 12"**
Shear capacity 640 plf	Shear capacity 850 plf	Shear capacity 1280 plf	Shear capacity 1460 plf

(a) ASCE 41-17, Section 12.5.3.6.2, expected strengths shall be permitted to be based on 1.5 times yielding strength of wood structural panel diaphragm. Yield strengths shall be determined using LRFD procedures contained in AWC SDPWS, except that the resistance factor, ϕ, shall be taken as 1.0

Verco Bare Metal Deck Diaphragm Design Tool	
18 ga. PLB-36 Roof Deck w/Hilti X-ENP19 PAF 36/7/4, Punchlok (VSC2) Sidelap Connections at 24" o.c.	**20 ga. PLB-36 Roof Deck w/Hilti X-ENP19 PAF 36/7/4, Punchlok (VSC2) Sidelap Connections**
8 ft span shear capacity 1785 plf	8 ft span shear capacity 2711 plf

18 ga PLB™-36 Grade 50 Roof Deck

Seismic Diaphragm Shear

For Both Ends Lapped Deck

Hilti X-ENP19 PAF Connections to Supports
36 / 7 / 4 Perpendicular Connection Pattern to Supports
PunchLok II Connection (VSC2) Sidelap Connections

A529 GR50 Support Member or Equivalent
0.465 ≤ Support Thickness (in.)
3 in. Minimum Deck End Bearing Length

LRFD Design Seismic Diaphragm Shear Strength ΦSn (plf) — Generic 3 Span Condition

Sidelap Connection Spacing (in.)	Span 7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	13'-0"	14'-0"	15'-0"
4	3569	3569	3018	2445	2021	1698	1447	1247	-
6	3553	3545	3018	2445	2021	1698	1447	1247	-
8	3308	3242	3018	2445	2021	1698	1447	1247	-
12	2734	2711	2692	2445	2021	1698	1447	1247	-
18	2274	2308	2121	2167	2021	1698	1447	1247	-
24	1985	1785	1883	1728	1815	1690	1447	1247	-
36	1654	1475	1328	1478	1358	1256	1379	1247	-

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connection Spacing (in.)	Span 7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	13'-0"	14'-0"	15'-0"
4	6	6	6	6	6	6	6	6	-
6	7	6	7	9	9	9	9	9	-
8	7	7	7	9	11	12	12	12	-
12	8	9	8	9	11	13	16	19	-
18	11	10	11	11	11	13	16	19	-
24	12	14	12	13	12	13	16	19	-
36	14	16	18	15	17	18	17	19	-

Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.) — Generic 3 Span Condition

Sidelap Connection Spacing (in.)	Span 7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	13'-0"	14'-0"	15'-0"
4	322	321	321	320	320	320	320	320	-
6	298	297	297	296	296	295	295	294	-
8	283	278	280	276	278	275	276	274	-
12	251	249	247	245	244	243	242	242	-
18	227	227	216	218	219	210	212	214	-
24	211	198	202	192	196	188	192	185	-
36	192	179	168	176	167	159	167	160	-

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Typ. Wall Out of-Plane

W_p =	75	psf		
f'_m =	1500	psi		
f_y =	41400	psi		
b =	12	in		
d =	8	in		
A_s =	0.17		#5@22" o.c. vertical	
κ =	1.00		ASCE 41-23, Table 6-1	
$\kappa_{damage\ control}$ =	1.00	(Damage Control)	ASCE 41-17, Table 6-1	
$\kappa_{limited\ safety}$ =	0.75	(Limited Safety)	ASCE 41-17, Table 6-1	
$S_{xs,BSE\text{-}1E}$ =	1.00	g		
$S_{xs,BSE\text{-}2E}$ =	1.75	g		
$\chi_{BSE\text{-}1E}$ =	1.40		ASCE 41-17, Table 7-2	
$\chi_{BSE\text{-}2E}$ =	0.95		ASCE 41-17, Table 7-2	
$F_{p,\ BSE\text{-}1E} = 0.4S_{xs,\ BSE\text{-}1E}\ \chi_{BSE\text{-}1E}\ W_p$ =	42	psf	ASCE 41-17, Eq. 7-13	
$F_{p,\ BSE\text{-}2E} = 0.4S_{xs,\ BSE\text{-}2E}\ \chi_{BSE\text{-}2E}\ W_p$ =	50	psf	ASCE 41-17, Eq. 7-13	governs
Trib. Width =	8.47	ft		
$0.9Q_{D,\ Roof}$ =	122	plf		
$0.9Q_{D,\ Wall}$ =	641	plf		
h =	19	ft		
$Q_{UF} = Q_E = M_u = w_u h^2/8$ =	2247	lb ft/ft		
$\kappa Q_{CL} = M_{n,\ lb} = (A_s f_y)(d)$ =	2333	lb ft/ft		
$Q_{UF}/\kappa Q_{CL}$	**0.96**			

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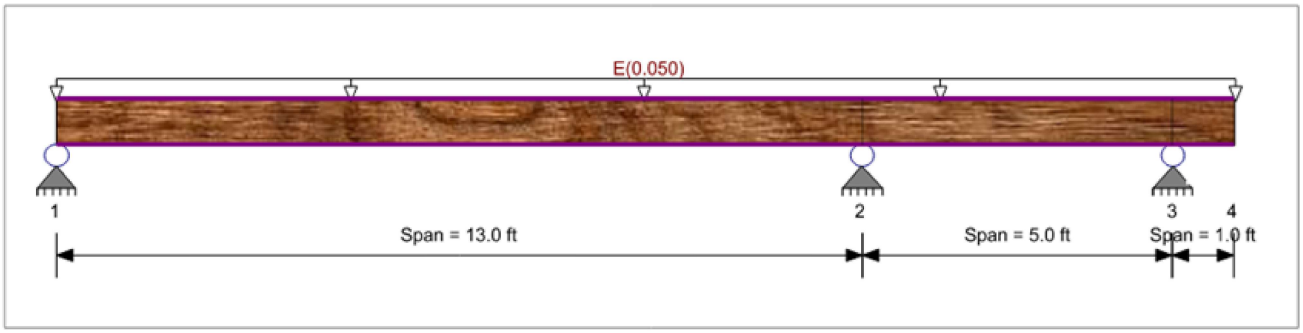
Wall Out of-Plane (GL 6)

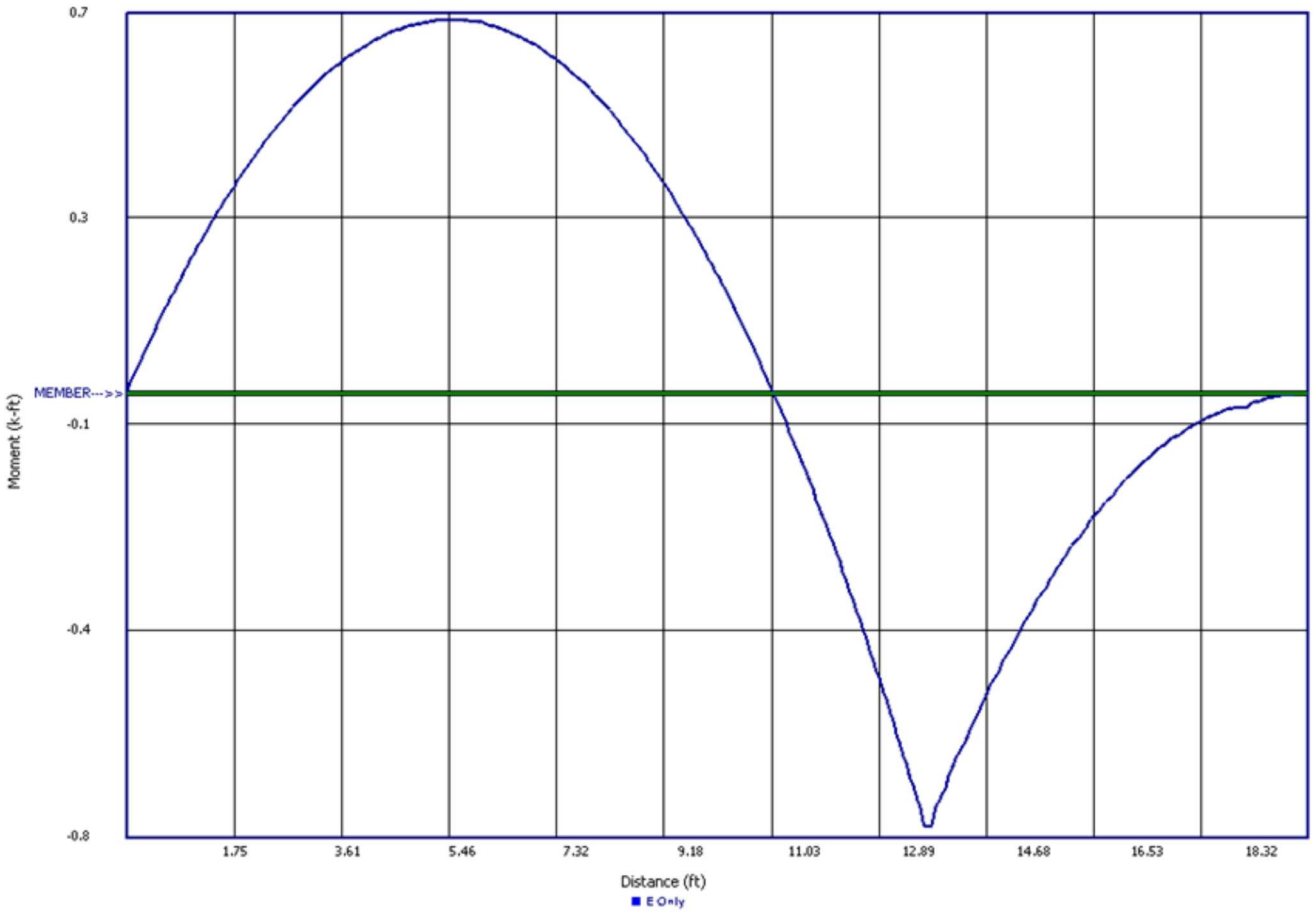
W_p =	75	psf		
f'_m =	1562	psi	material testing WP-1.4	
f_y =	43720	psi	material testing WPS-1.5	
f_r =	163	psi	TMS402, Table 9.1.9.2	
b =	12	in		
d =	8	in		
A_s =	0.17		#5@22" o.c. vertical	
$\kappa_{damage\ control}$ =	1.00	(Damage Control)	ASCE 41-17, Table 6-1	
$\kappa_{limited\ safety}$ =	0.75	(Limited Safety)	ASCE 41-17, Table 6-1	
$S_{xs,BSE\text{-}1E}$ =	1.00	g		
$S_{xs,BSE\text{-}2E}$ =	1.75	g		
$\chi_{BSE\text{-}1E}$ =	1.40		ASCE 41-17, Table 7-2	
$\chi_{BSE\text{-}2E}$ =	0.95		ASCE 41-17, Table 7-2	
$F_{p,\ BSE\text{-}1E} = 0.4S_{xs,\ BSE\text{-}1E}\ \chi_{BSE\text{-}1E}\ W_p$ =	42	psf	ASCE 41-17, Eq. 7-13	
$F_{p,\ BSE\text{-}2E} = 0.4S_{xs,\ BSE\text{-}2E}\ \chi_{BSE\text{-}2E}\ W_p$ =	50	psf	ASCE 41-17, Eq. 7-13	governs
Trib. Width =	8.47	ft		
$0.9Q_{D,\ Roof}$ =	152	plf		
$0.9Q_{D,\ Wall}$ =	641	plf		
$P_u/A_g = (0.9Q_{D,\ Roof}+0.9Q_{D,\ Wall})/Ag$ =	8	psi	TMS 402/602-16, Eq. 9-22	
$0.2f'_m$ =	312	psi	> P_u/A_g, O.K.	
$Q_{UF} = Q_E$ = M	800	lb ft/ft	Enercalc	
$M_{cr} = S_n (f_r+P_u/A_n)$ =	1827	lb ft/ft	> M_u	
$\delta_u = 5M_uh^2/48E_m I_n$ =	0.06	in	TMS 402/602-16, Eq. 9-25	
$Q_{UF} = Q_E = M_u = M+P_ue_u/2+P_u\delta$ =	804	lb ft/ft	TMS 402/602-16, Eq. 9-23	
$a = (A_sf_y+P_u/\phi)/0.8f'_mb$ =	0.55	in		
$\kappa Q_{CL} = M_{n,\ lb} = (A_sf_y+P_u/\phi)(d-a/2)$ =	1907	lb ft/ft	TMS 402/602-16, Sec. 9.3.5.2	
DC Ratio = $Q_{UF}/\kappa Q_{CL}$ =	**0.42**			

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Wall Out of-Plane (GL 6)





IDS GROUP

Job No.: 24X084.01
Date: 11/11/25
Engineer:

GL A wall anchor

Anchorage Demand per ASCE 41 §7.2.11.1

Wall weight, W_{wall} =	75	psf	
Roof height, h_n =	19	ft	
Height of anchor above base, z_a =	19	ft	$z_a \le h_n$
Parapet height, $H_{parapet}$ =	1	ft	
Perpendicular diaphragm span, L_f =	150	ft	
Flexibility factor, k_a =	2		$1.0+L_f/100$, ≤ 2.0 for flexible diaphragms
Height factor, k_h =	1		$(1+2z_a/h_n)/3$, or = 1.0 for flexible diaphragms
Tributary weight of wall, w_{wall} =	750	lb/ft	$w_{wall} = (H_{wall}/2 + H_{parapet})*(W_{wall})$
Anchor spacing =	4	ft	

BSE-1E Damage Control				BSE-2E Limited Safety		
χ =	1.65		Table 7-1	χ =	1.1	
$F_{p,\,min}$ =	0.66	* W	$0.2k_a\chi W_p$	$F_{p,\,min}$ =	0.44	* W
F_p =	1.33	* W	$0.4S_{XS}k_ak_h\chi W_p$	F_p =	1.54	* W
F_p =	4000	lbs.		F_p =	4700	lbs.

*Note that J-factors do not apply to F_p

Anchor Design

Simpson	HDB3		
Capacity, Q_{CL} =	5521	lb	per FEMA P-2006
# of anchors =	2		
Knowledge factor, κ =	1		Table 6-1
Anchorage Ultimate Capacity, κQ_{CL} =	11041	lbs.	(lower-bound)
Demand/Capacity Ratio (DCR) =	**43%**	**OK**	

USE (2) SIMPSON HDB3 ANCHORS

Beam Properties

Member size =	3x16		A =	38.125	in^2
Grade =	DF-Larch #1		S_x =	96.9	in^3
Lower-bound modification	0.85	per §12.2.2.5			

F_b =	1000	psi
Bending C_F =	0.9	
C_r =	1.00	
Bending K_F =	2.54	
F_b' =	**1940**	**psi (lower-bound)**

F_t =	675	psi
Tension C_F =	0.9	
Tension K_F =	2.7	
F_t' =	**1390**	**psi (lower-bound)**

Beam Check

Use 1.1(QD+QL) per equation 7-1

Beam spacing for gravity loads =	1.33	ft
Roof Dead Load, DL =	20.0	psf
Roof Live Load, LL =	5.0	psf (25% of unreduced per §7.2.2)

IDS GROUP

Job No.: 24X084.01
Date: 11/11/25
Engineer:

GL A wall anchor

Beam length, L =	32.5	ft	
1.1(D+L) =	37	plf	
Gravity moment, M =	4885	ft-lbs	$(wL^2)/8$
Bending stress, f_b =	605	psi	M/S_x
Bending DCR =	**0.31**		

BSE-1E Damage Control

Tension stress, f_t =	105	psi	F_p/A
Combined stress =	**0.39**	**< 1, OK**	

BSE-2E Limited Safety

Tension stress, f_t =	124	psi
Combined stress =	**0.40**	**< 1, OK**

3x16 OK FOR ANCHORAGE

IDS GROUP

Job No.: 24X084.01
Date: 11/11/25
Engineer:

GL 6/E-F wall anchor

Anchorage Demand per ASCE 41 §7.2.11.1

Wall weight, W_{wall} =	75	psf	
Roof height, h_n =	19	ft	
Height of anchor above base, z_a =	19	ft	$z_a \le h_n$
Parapet height, $H_{parapet}$ =	1	ft	
Perpendicular diaphragm span, L_f =	162	ft	
Flexibility factor, k_a =	2		$1.0+L_f/100$, ≤ 2.0 for flexible diaphragms
Height factor, k_h =	1		$(1+2z_a/h_n)/3$, or = 1.0 for flexible diaphragms
Tributary weight of wall, w_{wall} =	750	lb/ft	$w_{wall} = (H_{wall}/2 + H_{parapet})*(W_{wall})$
Anchor spacing =	4	ft	

BSE-1E Damage Control				BSE-2E Limited Safety		
χ =	1.65		Table 7-1	χ =	1.1	
$F_{p,\,min}$ =	0.66	* W	$0.2k_a\chi W_p$	$F_{p,\,min}$ =	0.44	* W
F_p =	1.33	* W	$0.4S_{XS}k_ak_h\chi W_p$	F_p =	1.54	* W
F_p =	4000	lbs.		F_p =	4700	lbs.

*Note that J-factors do not apply to F_p

Anchor Design

Simpson	HDB3		
Capacity, Q_{CL} =	5521	lb	per FEMA P-2006
# of anchors =	2		
Knowledge factor, κ =	1		Table 6-1
Anchorage Ultimate Capacity, κQ_{CL} =	11041	lbs.	(lower-bound)
Demand/Capacity Ratio (DCR) =	**43%**	**OK**	

USE (2) SIMPSON HDB3 ANCHORS

Beam Properties

Member size =	3x12		A =	28.125	in^2
Grade =	DF-Larch #1		S_x =	52.734	in^3
Lower-bound modification	0.85	per §12.2.2.5			

F_b =	1000	psi
Bending C_F =	1.0	
C_r =	1.00	
Bending K_F =	2.54	
F_b' =	**2150**	**psi (lower-bound)**

F_t =	675	psi
Tension C_F =	1.0	
Tension K_F =	2.7	
F_t' =	**1540**	**psi (lower-bound)**

Beam Check

Use 1.1(QD+QL) per equation 7-1

Beam spacing for gravity loads =	1.33	ft
Roof Dead Load, DL =	20.0	psf
Roof Live Load, LL =	5.0	psf (25% of unreduced per §7.2.2)

IDS GROUP

Job No.: 24X084.01
Date: 11/11/25
Engineer:

GL 6/E-F wall anchor

Beam length, L =	24.75	ft	
1.1(D+L) =	37	plf	
Gravity moment, M =	2833	ft-lbs	$(wL^2)/8$
Bending stress, f_b =	645	psi	M/S_x
Bending DCR =	**0.30**		

BSE-1E Damage Control

Tension stress, f_t =	143	psi	F_p/A
Combined stress =	**0.39**	**< 1, OK**	

BSE-2E Limited Safety

Tension stress, f_t =	168	psi
Combined stress =	**0.41**	**< 1, OK**

3x12 OK FOR ANCHORAGE

IDS GROUP

Job No.: 24X084.01
Date: 11/11/25
Engineer:

GLB at GL C&D

Anchorage Demand per ASCE 41 §7.2.11.1

Wall weight, W_{wall} =	75	psf	
Roof height, h_n =	19	ft	
Height of anchor above base, z_a =	19	ft	$z_a \le h_n$
Parapet height, $H_{parapet}$ =	1	ft	
Perpendicular diaphragm span, L_f =	162	ft	
Flexibility factor, k_a =	2		$1.0+L_f/100$, ≤ 2.0 for flexible diaphragms
Height factor, k_h =	1		$(1+2z_a/h_n)/3$, or = 1.0 for flexible diaphragms
Tributary weight of wall, w_{wall} =	750	lb/ft	$w_{wall} = (H_{wall}/2 + H_{parapet})*(W_{wall})$
Anchor spacing =	4	ft	

BSE-1E Damage Control				BSE-2E Limited Safety		
χ =	1.65			χ =	1.1	
$F_{p,\,min}$ =	0.66	* W	$0.2k_a\chi W_p$	$F_{p,\,min}$ =	0.44	* W
F_p =	1.33	* W	$0.4S_{XS}k_ak_h\chi W_p$	F_p =	1.54	* W
F_p =	4000	lbs.		F_p =	4700	lbs.

Table 7-1

*Note that J-factors do not apply to F_p

Anchor Design

Simpson	HDB3		
Capacity, Q_{CL} =	5521	lb	per FEMA P-2006
# of anchors =	2		
Knowledge factor, κ =	1		Table 6-1
Anchorage Ultimate Capacity, κQ_{CL} =	11041	lbs.	(lower-bound)
Demand/Capacity Ratio (DCR) =	**43%**	**OK**	

USE (2) SIMPSON HDB3 ANCHORS

Beam Properties

Member size =	5.5x27		A =	148.5	in^2
Grade =	DF-Larch #1		S_x =	668.25	in^3
Lower-bound modification	0.85	per §12.2.2.5			

F_b =	2400	psi
Bending C_F =	1.0	
C_r =	1.00	
Bending K_F =	2.54	
F_b' =	**5180**	**psi (lower-bound)**
F_t =	1100	psi
Tension C_F =	1.0	
Tension K_F =	2.7	
F_t' =	**2520**	**psi (lower-bound)**

Beam Check

Use 1.1(QD+QL) per equation 7-1

Beam spacing for gravity loads =	27	ft
Roof Dead Load, DL =	20.0	psf
Roof Live Load, LL =	3.0	psf (25% of unreduced per §7.2.2)

IDS GROUP

Job No.: 24X084.01
Date: 11/11/25
Engineer:

GLB at GL C&D

Beam length, L =	34.58	ft	
1.1(D+L) =	684	plf	
Gravity moment, M =	102239	ft-lbs	$(wL^2)/8$
Bending stress, f_b =	1836	psi	M/S_x
Bending DCR =	**0.35**		

BSE-1E Damage Control				BSE-2E Limited Safety		
Tension stress, f_t =	27	psi	F_p/A	Tension stress, f_t =	32	psi
Combined stress =	**0.37**	**< 1, OK**		**Combined stress =**	**0.37**	**< 1, OK**

5.5x27 OK FOR ANCHORAGE

Continuity Tie & Subdiaphragm Design

Spacing of continuity ties =	8	ft. max	
Assume 1/2" plywood w/ 10d @ 6" blocked, chorded			
Diaphragm capacity, v_s =	640	lb/ft	NDS SDPWS Table 4.2C

BSE-1E Damage Control				BSE-2E Limited Safety		
F_p =	8	kips		F_p =	9.3	kips
m =	2.25		Table 12-3	m =	3.5	
D_L =	**6**	**ft.**	**Min. subdiaphragm length**	**D_L =**	**5**	**ft.**

Continuity Tie Straps =	CMST12		requires 4x or 2-2x min. member
Capacity, Q_{CL} =	13080	lb	per Simpson Technical Bulletin TEB-ASCE4123
# of HD =	1		
Knowledge factor, κ =	1	Table 6-1	
HD Ultimate Capacity =	13	kips	
Demand/Capacity Ratio (DCR) =	**72%**	**OK**	

USE (1) SIMPSON CMST12 @ 8 FT O.C.

IDS GROUP

Job No.: 24X084.01
Date: 11/11/25
Engineer:

GLB at GL B

Anchorage Demand per ASCE 41 §7.2.11.1

Wall weight, W_{wall} =	75	psf	
Roof height, h_n =	19	ft	
Height of anchor above base, z_a =	19	ft	$z_a \le h_n$
Parapet height, $H_{parapet}$ =	1	ft	
Perpendicular diaphragm span, L_f =	162	ft	
Flexibility factor, k_a =	2		$1.0+L_f/100$, ≤ 2.0 for flexible diaphragms
Height factor, k_h =	1		$(1+2z_a/h_n)/3$, or = 1.0 for flexible diaphragms
Tributary weight of wall, w_{wall} =	750	lb/ft	$w_{wall} = (H_{wall}/2 + H_{parapet})*(W_{wall})$
Anchor spacing =	4	ft	

BSE-1E Damage Control				BSE-2E Limited Safety		
χ =	1.65		Table 7-1	χ =	1.1	
$F_{p,\,min}$ =	0.66	* W	$0.2k_a\chi W_p$	$F_{p,\,min}$ =	0.44	* W
F_p =	1.33	* W	$0.4S_{XS}k_ak_h\chi W_p$	F_p =	1.54	* W
F_p =	4000	lbs.		F_p =	4700	lbs.

*Note that J-factors do not apply to F_p

Anchor Design

Simpson	HDB3		
Capacity, Q_{CL} =	5521	lb	per FEMA P-2006
# of anchors =	2		
Knowledge factor, κ =	1		Table 6-1
Anchorage Ultimate Capacity, κQ_{CL} =	11041	lbs.	(lower-bound)
Demand/Capacity Ratio (DCR) =	**43%**	**OK**	

USE (2) SIMPSON HDB3 ANCHORS

Beam Properties

Member size =	5x30		A =	150	in^2
Grade =	DF-Larch #1		S_x =	825	in^3
Lower-bound modification	0.85	per §12.2.2.5			

F_b =	2400	psi
Bending C_F =	1.0	
C_r =	1.00	
Bending K_F =	2.54	
F_b' =	**5180**	**psi (lower-bound)**
F_t =	1100	psi
Tension C_F =	1.0	
Tension K_F =	2.7	
F_t' =	**2520**	**psi (lower-bound)**

Beam Check

Use 1.1(QD+QL) per equation 7-1

Beam spacing for gravity loads =	30.5	ft
Roof Dead Load, DL =	20.0	psf
Roof Live Load, LL =	3.0	psf (25% of unreduced per §7.2.2)

IDS GROUP

Job No.: 24X084.01
Date: 11/11/25
Engineer:

GLB at GL B

Beam length, L =	34.58	ft	
1.1(D+L) =	772	plf	
Gravity moment, M =	115392	ft-lbs	$(wL^2)/8$
Bending stress, f_b =	1678	psi	M/S_x
Bending DCR =	**0.32**		

BSE-1E Damage Control				BSE-2E Limited Safety		
Tension stress, f_t =	27	psi	F_p/A	Tension stress, f_t =	32	psi
Combined stress =	**0.33**	**< 1, OK**		**Combined stress =**	**0.34**	**< 1, OK**

5x30 OK FOR ANCHORAGE

Continuity Tie & Subdiaphragm Design

Spacing of continuity ties = 8 ft. max

Assume 1/2" plywood w/ 10d @ 6" blocked, chorded

Diaphragm capacity, v_s = 640 lb/ft NDS SDPWS Table 4.2C

BSE-1E Damage Control				BSE-2E Limited Safety		
F_p =	8	kips		F_p =	9.3	kips
m =	2.25		Table 12-3	m =	3.5	
D_L =	**6**	**ft.**	**Min. subdiaphragm length**	**D_L =**	**5**	**ft.**

Continuity Tie Straps =	CMST12		requires 4x or 2-2x min. member
Capacity, Q_{CL} =	13080	lb	per Simpson Technical Bulletin TEB-ASCE4123
# of HD =	1		
Knowledge factor, κ =	1	Table 6-1	
HD Ultimate Capacity =	13	kips	
Demand/Capacity Ratio (DCR) =	**72%**	**OK**	

USE (1) SIMPSON CMST12 @ 8 FT O.C.

IDS GROUP

Job No.: 24X084.01
Date: 11/11/25
Engineer:

GLB at GL E

Anchorage Demand per ASCE 41 §7.2.11.1

Wall weight, W_{wall} =	75	psf	
Roof height, h_n =	19	ft	
Height of anchor above base, z_a =	19	ft	$z_a \le h_n$
Parapet height, $H_{parapet}$ =	1	ft	
Perpendicular diaphragm span, L_f =	162	ft	
Flexibility factor, k_a =	2		$1.0+L_f/100$, ≤ 2.0 for flexible diaphragms
Height factor, k_h =	1		$(1+2z_a/h_n)/3$, or = 1.0 for flexible diaphragms
Tributary weight of wall, w_{wall} =	750	lb/ft	$w_{wall} = (H_{wall}/2 + H_{parapet})*(W_{wall})$
Anchor spacing =	4	ft	

BSE-1E Damage Control				BSE-2E Limited Safety		
χ =	1.65		Table 7-1	χ =	1.1	
$F_{p,min}$ =	0.66	* W	$0.2k_a\chi W_p$	$F_{p,min}$ =	0.44	* W
F_p =	1.33	* W	$0.4S_{XS}k_ak_h\chi W_p$	F_p =	1.54	* W
F_p =	4000	lbs.		F_p =	4700	lbs.

*Note that J-factors do not apply to F_p

Anchor Design

Simpson	HDB3		
Capacity, Q_{CL} =	5521	lb	per FEMA P-2006
# of anchors =	2		
Knowledge factor, κ =	1		Table 6-1
Anchorage Ultimate Capacity, κQ_{CL} =	11041	lbs.	(lower-bound)
Demand/Capacity Ratio (DCR) =	**43%**	**OK**	

USE (2) SIMPSON HDB3 ANCHORS

Beam Properties

Member size =	7x27		A =	182.25	in^2
Grade =	DF-Larch #1		S_x =	820.125	in^3
Lower-bound modification	0.85	per §12.2.2.5			

F_b =	2400	psi
Bending C_F =	1.0	
C_r =	1.00	
Bending K_F =	2.54	
F_b' =	**5180**	**psi (lower-bound)**
F_t =	1100	psi
Tension C_F =	1.0	
Tension K_F =	2.7	
F_t' =	**2520**	**psi (lower-bound)**

Beam Check

Use 1.1(QD+QL) per equation 7-1

Beam spacing for gravity loads =	31.25	ft
Roof Dead Load, DL =	20.0	psf
Roof Live Load, LL =	3.0	psf (25% of unreduced per §7.2.2)

IDS GROUP

Job No.: 24X084.01
Date: 11/11/25
Engineer:

GLB at GL E

Beam length, L =	34.58	ft	
1.1(D+L) =	791	plf	
Gravity moment, M =	118232	ft-lbs	$(wL^2)/8$
Bending stress, f_b =	1730	psi	M/S_x
Bending DCR =	**0.33**		

BSE-1E Damage Control				BSE-2E Limited Safety		
Tension stress, f_t =	22	psi	F_p/A	Tension stress, f_t =	26	psi
Combined stress =	**0.34**	**< 1, OK**		**Combined stress =**	**0.34**	**< 1, OK**

7x27 OK FOR ANCHORAGE

Continuity Tie & Subdiaphragm Design

Spacing of continuity ties = 8 ft. max

Assume 1/2" plywood w/ 10d @ 6" blocked, chorded

Diaphragm capacity, v_s = 640 lb/ft NDS SDPWS Table 4.2C

BSE-1E Damage Control				BSE-2E Limited Safety		
F_p =	8	kips		F_p =	9.3	kips
m =	2.25		Table 12-3	m =	3.5	
D_L =	**6**	**ft.**	**Min. subdiaphragm length**	**D_L =**	**5**	**ft.**

Continuity Tie Straps =	CMST12		requires 4x or 2-2x min. member
Capacity, Q_{CL} =	13080	lb	per Simpson Technical Bulletin TEB-ASCE4123
# of HD =	1		
Knowledge factor, κ =	1	Table 6-1	
HD Ultimate Capacity =	13	kips	
Demand/Capacity Ratio (DCR) =	**72%**	**OK**	

USE (1) SIMPSON CMST12 @ 8 FT O.C.

IDS GROUP

Job No.: 24X015.00
Date: 6/5/25
Engineer:

BLKG @ GL 1&6&F

Anchorage Demand per ASCE 41 §7.2.11.1

Wall weight, W_{wall} =	75	psf	
Roof height, h_n =	19	ft	
Height of anchor above base, z_a =	19	ft	$z_a \le h_n$
Parapet height, $H_{parapet}$ =	1	ft	
Perpendicular diaphragm span, L_f =	179.66	ft	
Flexibility factor, k_a =	2		1.0+L_f/100, ≤ 2.0 for flexible diaphragms
Height factor, k_h =	1		($1+2z_a/h_n$)/3, or = 1.0 for flexible diaphragms
Tributary weight of wall, w_{wall} =	750	lb/ft	$w_{wall} = (H_{wall}/2 + H_{parapet})*(W_{wall})$
Anchor spacing =	4	ft	

BSE-1E Damage Control				BSE-2E Limited Safety		
χ =	1.65		Table 7-1	χ =	1.1	
$F_{p,min}$ =	0.66	* W	$0.2k_a\chi W_p$	$F_{p,min}$ =	0.44	* W
F_p =	1.33	* W	$0.4S_{XS}k_ak_h\chi W_p$	F_p =	1.54	* W
F_p =	4000	lbs.		F_p =	4700	lbs.

*Note that J-factors do not apply to F_p

Anchor Design

Simpson	HDB3		
Capacity, Q_{CL} =	5521	lbs.	per FEMA P-2006
# of anchors =	2		
Knowledge factor, κ =	1	Table 6-1	
Anchorage Ultimate Capacity =	11041	lbs.	
Demand/Capacity Ratio (DCR) =	**43%**	**OK**	

USE (2) SIMPSON HDB3 ANCHORS @ 4 FT. O.C.

Strap Design for Blocking

Simpson	CMST12		
Capacity, Q_{CL} =	13080	lbs.	per Simpson Technical Bulletin TEB-ASCE4123
# of anchors =	1		
Knowledge factor, κ =	1	Table 6-1	
Anchorage Ultimate Capacity =	13080	lbs.	
Demand/Capacity Ratio (DCR) =	**36%**	**OK**	

USE (1) SIMPSON CMST12 ANCHOR @ 4 FT. O.C.

IDS GROUP

Job No.: 24X015.00
Date: 6/5/25
Engineer:

BLKG @ GL 1&6&F

Continuity Tie & Subdiaphragm Design

Spacing of continuity ties = 16 ft. max

Assume 1/2" plywood w/ 10d @ 6" blocked, chorded

Diaphragm capacity, v_s = 640 lb/ft NDS SDPWS Table 4.2C

BSE-1E Life Safety				BSE-2E Collapse Prevention		
F_p =	16	kips		F_p =	18.5	kips
m =	2.25		Table 12-3	m =	3.5	
D_L =	**12**	**ft.**	**Min. subdiaphragm length**	**D_L =**	**9**	**ft.**

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Corona south mall GLB OOP anchorage	Date:	11/20/2025
Fastening point:			

Specifier's comments:

1 Input data



Anchor type and diameter:	**HY 200-A V3 + threaded rod 5.8 5/8**
Item number:	385428 HAS 5.8 5/8"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Specification text:	Hilti ∅ 5/8 HY 200-A V3 + threaded rod 5.8 with 6 in nominal embedment depth per ICC-ES ESR-4878 , Hammer drilled installation per MPII
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	5.8
Evaluation Service Report:	ESR-4878
Issued I Valid:	11/1/2024 | 11/1/2025
Proof:	Design Method LRFD (AC58) Masonry + ACI 318-19
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 26.000 in. x 12.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked Grout-filled CMU, f=1500, f = 1,500 psi, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Solid Head Joint: yes; open ended unit: yes
Temp. short/long:	32 °F / 32 °F
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
Installation:	Face installation, Drill hole: Hammer drilled, Installation condition: Dry
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d))
	Shear load: yes (17.10.6.3 (c))

[R] - The anchor calculation is based on a rigid anchor plate assumption.

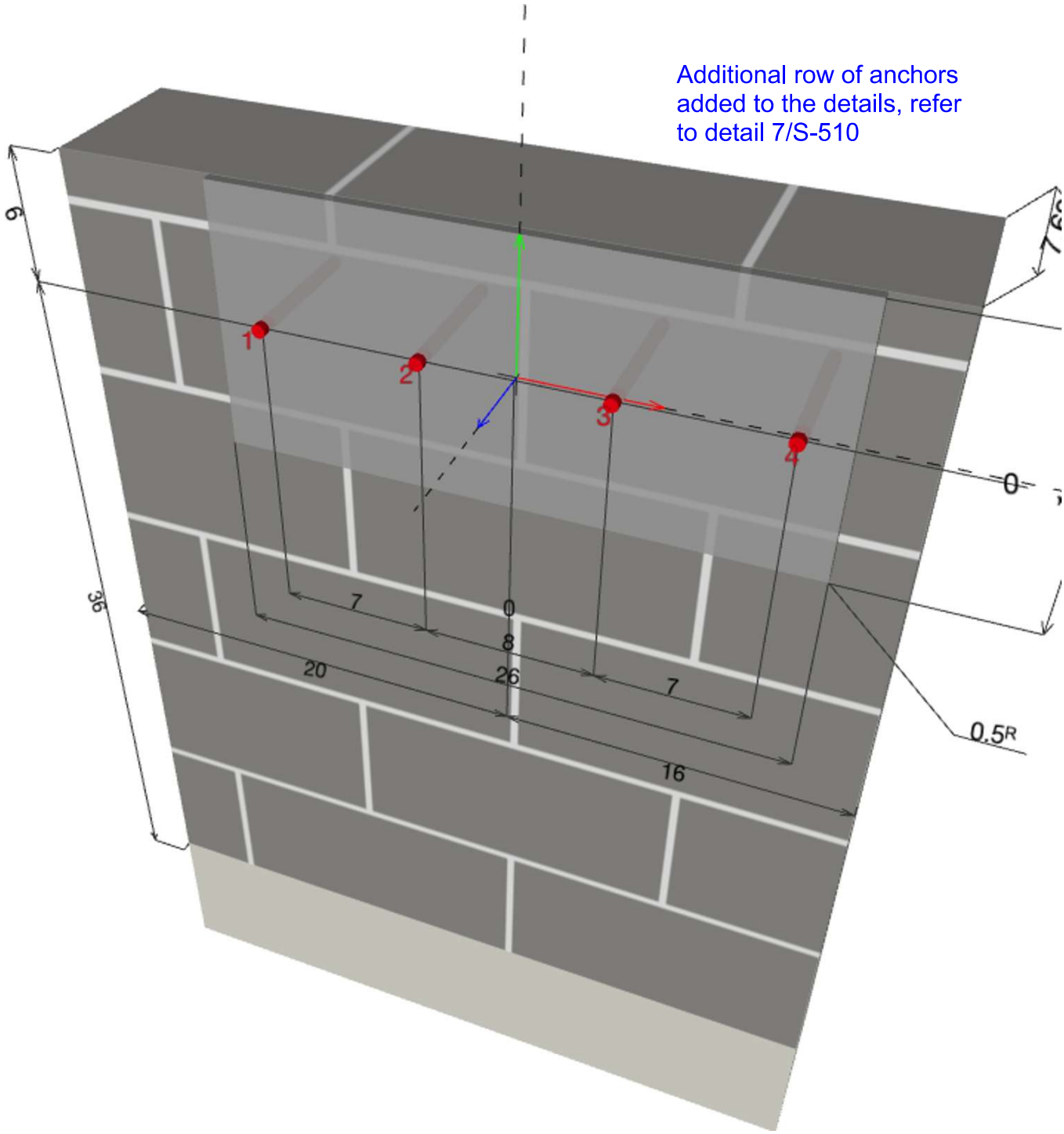
Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Corona south mall GLB OOP anchorage	Date:	11/20/2025
Fastening point:			

Geometry [in.]



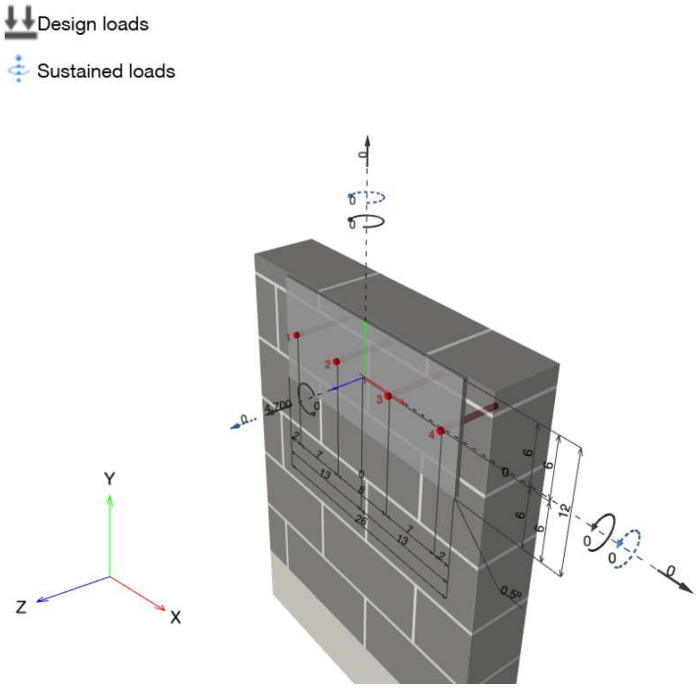
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Design:	Corona south mall GLB OOP anchorage	Date:	11/20/2025
Fastening point:			

Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

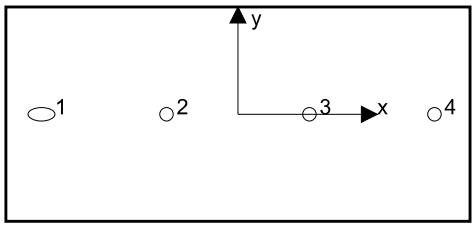
Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 4,700; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0; N_{sus} = 0; $M_{x,sus}$ = 0; $M_{y,sus}$ = 0;	yes	98

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,175	0	0	0
2	1,175	0	0	0
3	1,175	0	0	0
4	1,175	0	0	0



Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 0 [psi]
Resulting tension force in (x/y)=(-/-): 0 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Corona south mall GLB OOP anchorage	Date:	11/20/2025
Fastening point:			

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load [lb]	Capacity [lb]	Utilization β_N [%]	Status
Steel	1,175	10,650	12	OK
Bond	4,700	9,253	51	OK
Masonry breakout	4,700	4,809	98	OK

3.1 Steel strength

N_{sa} = ESR value refer to ICC-ES ESR-4878

$\phi\ N_{sa} \geq N_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

$N_{sa,seismic}$ [lb]	ϕ	$\phi_{seismic}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
16,384	0.650	1.000	10,650	1,175

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Phone I Fax:	|	E-Mail:	
Design:	Corona south mall GLB OOP anchorage	Date:	11/20/2025
Fastening point:			

3.2 Bond strength

$N_{mag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \cdot \psi_{ec1,Na} \cdot \psi_{ec2,Na} \cdot \psi_{ed,Na} \cdot N_{ba,m}$ — ACI 318-19 Eq. (17.6.5.1b)

$\phi\ N_{mag} \geq N_{ua}$ — AC58 Table 3.2 + ACI 318-19 Table D.4.1.1

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ — ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr,m}}{1100}}$ — ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e'_N}{c_{Na}}}\right) \leq 1.00$ — ACI 318-19 Eq. (17.6.5.3.1)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.00$ — ACI 318-19 Eq. (17.6.5.4.1b)

$N_{ba,m} = \lambda_a \cdot \tau_{cr,m} \cdot \pi \cdot d_a \cdot h_{ef}$ — ACI 318-19 Eq. (17.6.5.2.1)

Variables

d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{uncr,m}$ [psi]	$\tau_{cr,m}$ [psi]	α_{sat}	α_{top}
0.625	6.000	5.000	897	574	1.000	1.000

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	λ_a	$\alpha_{N,seis}$
0.000	0.000	1.000	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
5.618	366.53	126.27	0.967

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$N_{ba,m}$ [lb]
1.000	1.000	6,762

Results

N_{mag} [lb]	ϕ	$\phi_{seismic}$	$\phi\ N_{mag}$ [lb]	$\phi\ N_{ua}$ [lb]
18,981	0.650	0.750	9,253	4,700

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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3.3 Masonry breakout strength

$N_{mbg} = \frac{A_{Nm}}{A_{Nm0}} \cdot \psi_{ec,N,m} \ \psi_{ed,N,m} \cdot \psi_{c,N,m} \cdot N_{b,m}$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi \ N_{mbg} \geq N_{ua}$ — AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Nm} see ACI 318-19, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nm0} = 9 \cdot h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.2.1)

$\psi_{ec,N,m} = \left(\frac{1}{1 + \frac{2 \, e'_N}{3 \cdot h_{ef}}} \right) \leq 1.00$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5 \cdot h_{ef}} \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$N_{b,m} = k_m \cdot \lambda_a \cdot \sqrt{f'_m} \cdot h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N,m}$ [in.]	$e_{c2,N,m}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N,m}$	k_m
6.000	0.000	0.000	5.000	1.000	12

λ_a	f'_m [psi]
1.000	1,500

Calculations

A_{Nm} [in.^{2}]	A_{Nm0} [in.^{2}]	$\psi_{ec1,N,m}$	$\psi_{ec2,N,m}$	$\psi_{ed,N,m}$	$N_{b,m}$ [lb]
540.00	324.00	1.000	1.000	0.867	6,830

Results

N_{mbg} [lb]	ϕ	$\phi_{seismic}$	$\phi \ N_{mbg}$ [lb]	N_{ua} [lb]
9,865	0.650	0.750	4,809	4,700

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4 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry should be built according to industry standards.

Fastening meets the design criteria!

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5 Installation data

Anchor plate, steel: ASTM A36; E = 29,000,001 psi; f_{yk} = 36,000 psi

Profile: no profile

Hole diameter in the fixture: d_f = 0.750 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Drilling method: Drilled in hammer mode

Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: HY 200-A V3 + threaded rod 5.8 5/8

Item number: 385428 HAS 5.8 5/8"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

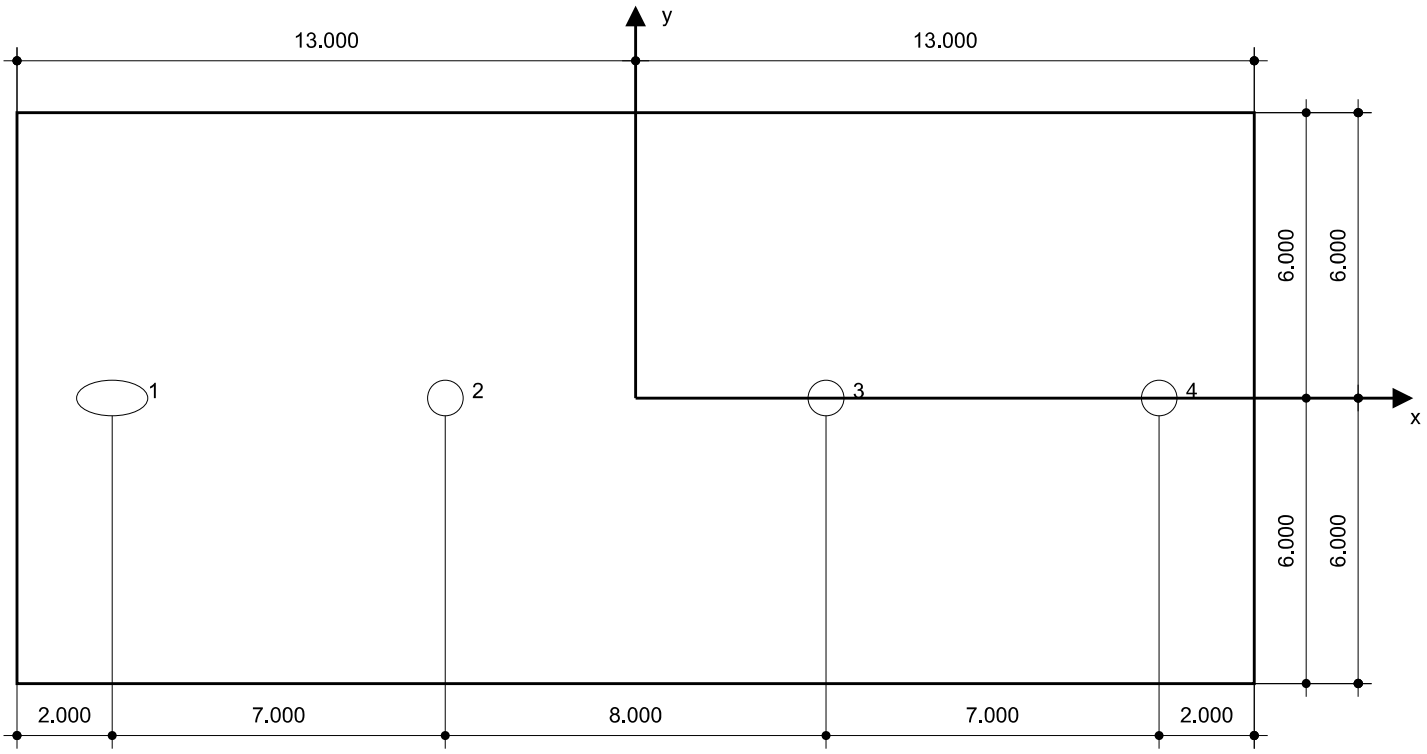
Maximum installation torque: 720 in.lb

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 6.000 in.

Minimum thickness of the base material: 7.625 in.

Hilti ∅ 5/8 HY 200-A V3 + threaded rod 5.8 with 6 in nominal embedment depth per ICC-ES ESR-4878 , Hammer drilled installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-11.000	0.000	9.000	27.000	36.000	6.000
2	-4.000	0.000	16.000	20.000	36.000	6.000
3	4.000	0.000	24.000	12.000	36.000	6.000
4	11.000	0.000	31.000	5.000	36.000	6.000

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6 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Specifier's comments:

1 Input data



Anchor type and diameter:	**HY 200-A V3 + threaded rod 5.8 5/8**
Item number:	385428 HAS 5.8 5/8"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Specification text:	Hilti ∅ 5/8 HY 200-A V3 + threaded rod 5.8 with 6 in nominal embedment depth per ICC-ES ESR-4878 , Hammer drilled installation per MPII
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	5.8
Evaluation Service Report:	ESR-4878
Issued I Valid:	11/1/2024 | 11/1/2025
Proof:	Design Method LRFD (AC58) Masonry + ACI 318-19
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.400 in.
Anchor plate[R] :	l_x x l_y x t = 12.000 in. x 12.000 in. x 0.400 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	Grout-filled CMU, f=1500, f = 1,500 psi, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Solid Head Joint: no; open ended unit: no
Temp. short/long:	32 °F / 32 °F
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
Installation:	Face installation, Drill hole: Hammer drilled, Installation condition: Dry
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d))
	Shear load: yes (17.10.6.3 (c))

[R] - The anchor calculation is based on a rigid anchor plate assumption.

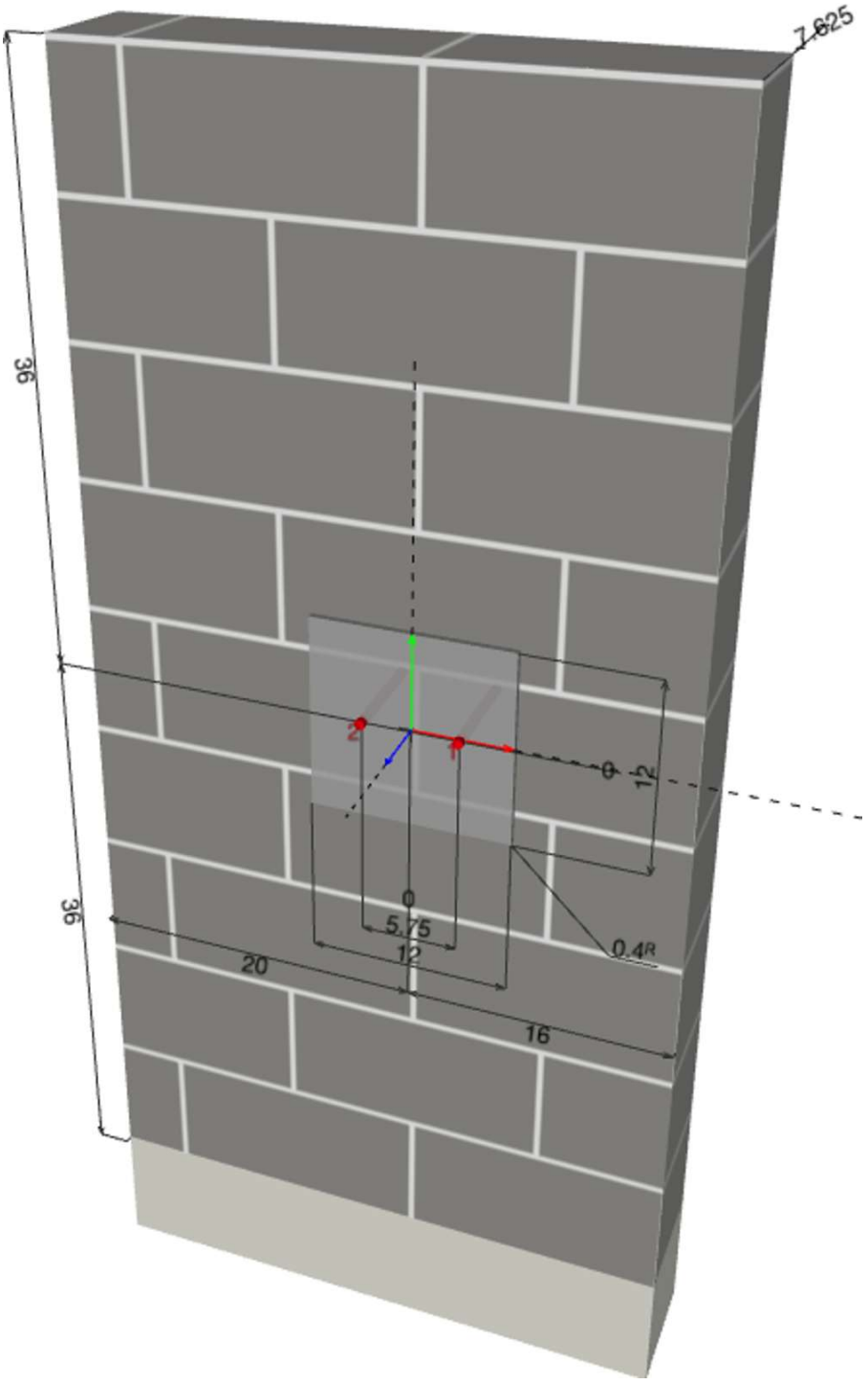
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Geometry [in.]



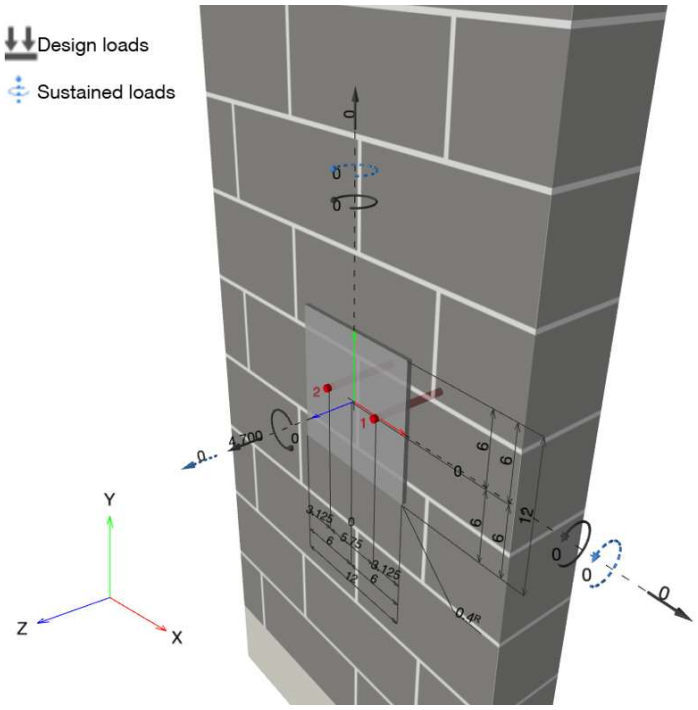
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Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 4,700; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0; N_{sus} = 0; $M_{x,sus}$ = 0; $M_{y,sus}$ = 0;	yes	98

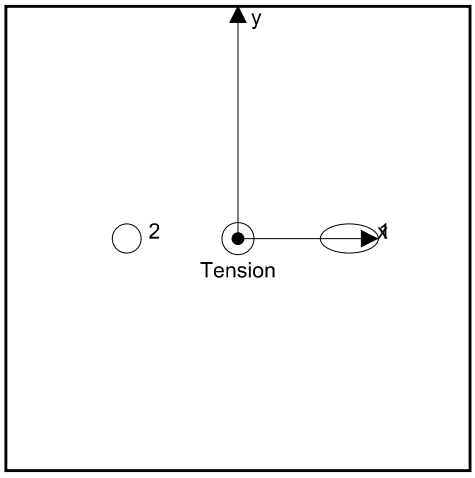
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2 Load case/Resulting anchor forces



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2,350	0	0	0
2	2,350	0	0	0

Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 0 [psi]
Resulting tension force in (x/y)=(0.000/0.000): 4,700 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load [lb]	Capacity [lb]	Utilization β_N [%]	Status
Steel	2,350	10,650	23	OK
Bond	2,350	3,212	74	OK
Masonry breakout	2,350	2,418	98	OK

3.1 Steel strength

N_{sa} = ESR value — refer to ICC-ES ESR-4878
$\phi\ N_{sa} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

$N_{sa,seismic}$ [lb]	ϕ	$\phi_{seismic}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
16,384	0.650	1.000	10,650	2,350

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3.2 Bond strength

$N_{mag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \cdot \psi_{ec1,Na} \cdot \psi_{ec2,Na} \cdot \psi_{ed,Na} \cdot N_{ba,m}$ — ACI 318-19 Eq. (17.6.5.1b)

$\phi\ N_{mag} \geq N_{ua}$ — AC58 Table 3.2 + ACI 318-19 Table D.4.1.1

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ — ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr,m}}{1100}}$ — ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e'_N}{c_{Na}}}\right) \leq 1.00$ — ACI 318-19 Eq. (17.6.5.3.1)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.00$ — ACI 318-19 Eq. (17.6.5.4.1b)

$N_{ba,m} = \lambda_a \cdot \tau_{uncr,m} \cdot \pi \cdot d_a \cdot h_{ef}$ — ACI 318-19 Eq. (17.6.5.2.1)

Variables

d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{uncr,m}$ [psi]	α_{sat}	α_{top}
0.625	6.000	2.688	897	1.000	1.000

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	λ_a	$\alpha_{N,seis}$
0.000	0.000	1.000	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
5.618	93.33	126.27	0.844

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$N_{ba,m}$ [lb]
1.000	1.000	10,568

Results

N_{mag} [lb]	ϕ	$\phi_{seismic}$	$\phi\ N_{mag}$ [lb]	$\phi\ N_{ua}$ [lb]
6,589	0.650	0.750	3,212	2,350

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3.3 Masonry breakout strength

$N_{mbg} = \frac{A_{Nm}}{A_{Nm0}} \cdot \psi_{ec,N,m} \ \psi_{ed,N,m} \cdot \psi_{c,N,m} \cdot N_{b,m}$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi \ N_{mbg} \geq N_{ua}$ — AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Nm} see ACI 318-19, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nm0} = 9 \cdot h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.2.1)

$\psi_{ec,N,m} = \left(\frac{1}{1 + \frac{2 \, e'_N}{3 \cdot h_{ef}}} \right) \leq 1.00$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5 \cdot h_{ef}} \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$N_{b,m} = k_m \cdot \lambda_a \cdot \sqrt{f'_m} \cdot h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N,m}$ [in.]	$e_{c2,N,m}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N,m}$	k_m
6.000	0.000	0.000	2.688	1.000	17

λ_a	f'_m [psi]
1.000	1,500

Calculations

A_{Nm} [in.^{2}]	A_{Nm0} [in.^{2}]	$\psi_{ec1,N,m}$	$\psi_{ec2,N,m}$	$\psi_{ed,N,m}$	$N_{b,m}$ [lb]
210.37	324.00	1.000	1.000	0.790	9,676

Results

N_{mbg} [lb]	ϕ	$\phi_{seismic}$	$\phi \ N_{mbg}$ [lb]	N_{ua} [lb]
4,960	0.650	0.750	2,418	2,350

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4 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
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- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry should be built according to industry standards.

Fastening meets the design criteria!

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5 Installation data

Anchor plate, steel: ASTM A36; E = 29,000,001 psi; f_{yk} = 36,000 psi

Profile: no profile

Hole diameter in the fixture: d_f = 0.750 in.

Plate thickness (input): 0.400 in.

Recommended plate thickness: not calculated

Drilling method: Drilled in hammer mode

Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: HY 200-A V3 + threaded rod 5.8 5/8

Item number: 385428 HAS 5.8 5/8"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

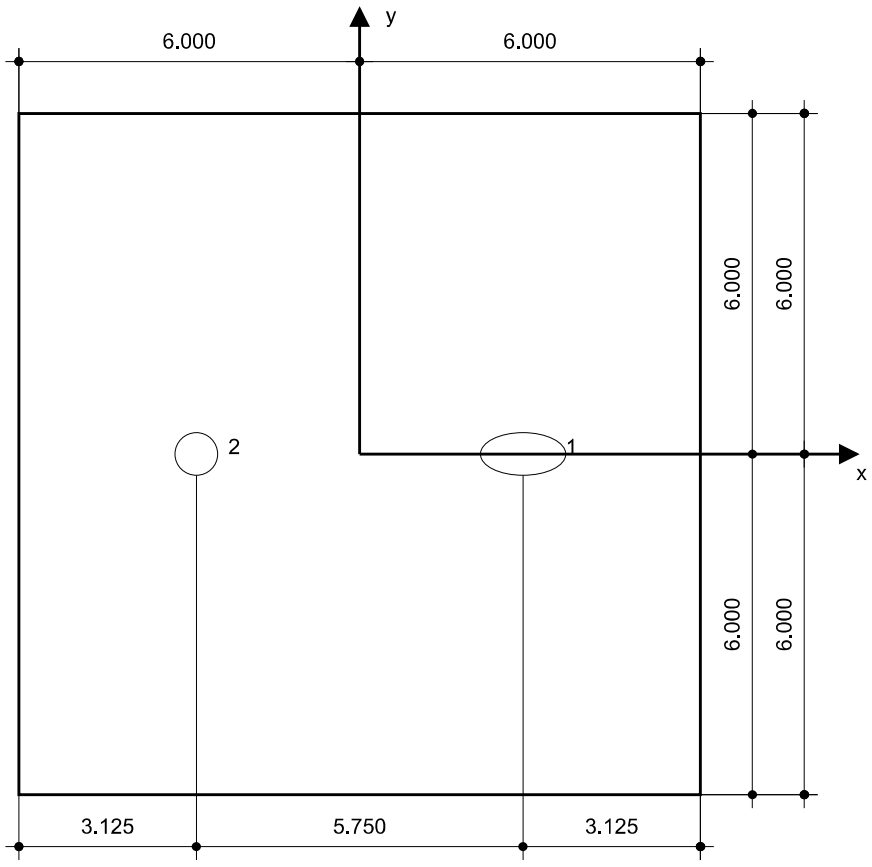
Maximum installation torque: 720 in.lb

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 6.000 in.

Minimum thickness of the base material: 7.625 in.

Hilti ∅ 5/8 HY 200-A V3 + threaded rod 5.8 with 6 in nominal embedment depth per ICC-ES ESR-4878 , Hammer drilled installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	2.875	0.000	22.875	13.125	36.000	36.000
2	-2.875	0.000	17.125	18.875	36.000	36.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Corona south mall OOP anchorage	Date:	11/20/2025
Fastening point:			

6 Remarks; Your Cooperation Duties

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Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Steel Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

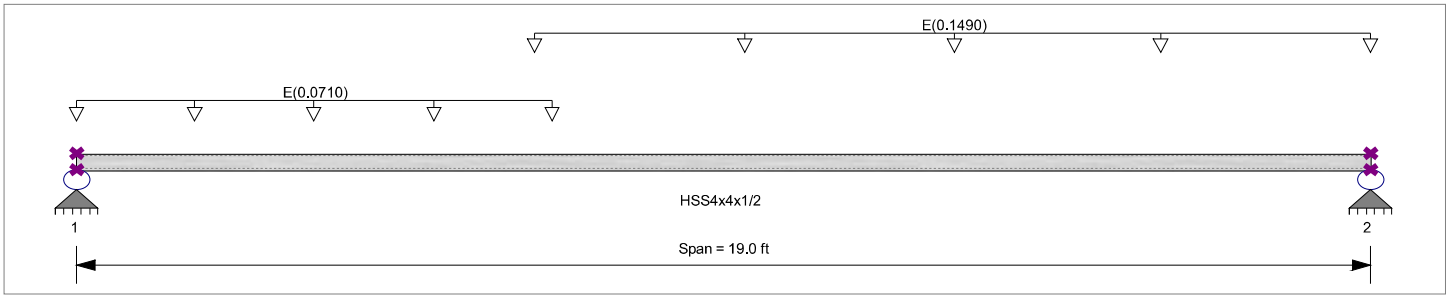
DESCRIPTION: --Strongback (3'-4" door)--

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Load Resistance Factor Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : E = 0.0710 k/ft, Extent = 0.0 -->> 7.0 ft, Tributary Width = 1.0 ft
Uniform Load : E = 0.1490 k/ft, Extent = 6.750 -->> 19.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.224 : 1	Maximum Shear Stress Ratio =	0.022 : 1
Section used for this span	HSS4x4x1/2	Section used for this span	HSS4x4x1/2
Mu : Applied	5.922 k-ft	Vu : Applied	1.328 k
Mn * Phi : Allowable	26.409 k-ft	Vn * Phi : Allowable	60.179 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	19.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	1.096 in	Ratio =	207 >=200.	Span: 1 : E Only
Max Upward Transient Deflection	0 in	Ratio =	0 <200.0	n/a
Max Downward Total Deflection	0.769 in	Ratio =	296 >=200.	Span: 1 : E Only * 0.70
Max Upward Total Deflection	0 in	Ratio =	0 <200.0	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	Summary of Shear Values VuMax	Vnx	Phi*Vnx
Dsgn. L = 19.00 ft	1		0.000				29.34	26.41	1.00	1.00	-0.00	66.87	60.18
E Only													
Dsgn. L = 19.00 ft	1	0.224	0.022	5.92		5.92	29.34	26.41	1.16	1.00	1.33	66.87	60.18

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	E Only	1.0988	9.717		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.994	1.328
Max Upward from Load Cases	0.994	1.328
E Only * 0.70	0.696	0.930
E Only * 0.5250	0.522	0.697
E Only	0.994	1.328

Steel Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Strongback (3'-4" door & 12'-) entrance door --

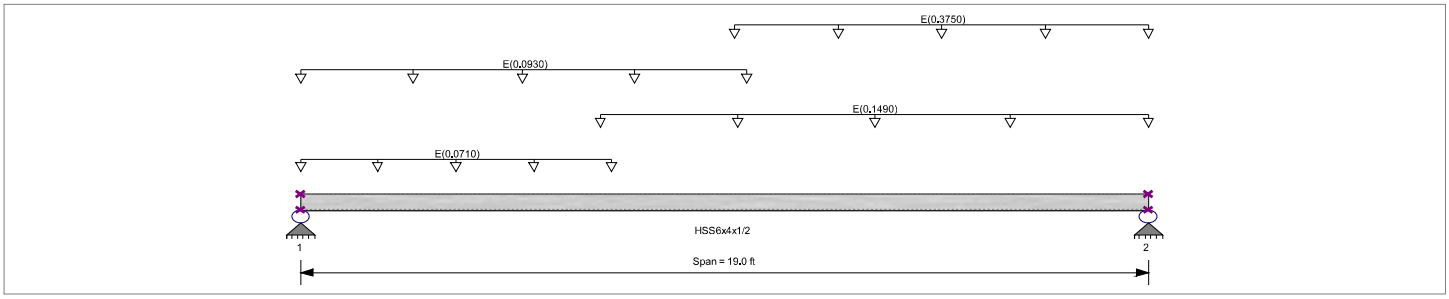
CODE REFERENCES

Calculations per AISC 360-16, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Load Resistance Factor Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending

Fy : Steel Yield : 46.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : E = 0.0710 k/ft, Extent = 0.0 -->> 7.0 ft, Tributary Width = 1.0 ft
Uniform Load : E = 0.1490 k/ft, Extent = 6.750 -->> 19.0 ft, Tributary Width = 1.0 ft
Uniform Load : E = 0.0930 k/ft, Extent = 0.0 -->> 10.0 ft, Tributary Width = 1.0 ft
Uniform Load : E = 0.3750 k/ft, Extent = 9.750 -->> 19.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.337 : 1	Maximum Shear Stress Ratio =	0.039 : 1
Section used for this span	HSS6x4x1/2	Section used for this span	HSS6x4x1/2
Mu : Applied	16.812 k-ft	Vu : Applied	4.198 k
Mn * Phi : Allowable	49.932 k-ft	Vn * Phi : Allowable	106.381 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	19.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	1.071 in	Ratio =	212 >=200.	Span: 1 : E Only
Max Upward Transient Deflection	0 in	Ratio =	0 <200.0	n/a
Max Downward Total Deflection	0.751 in	Ratio =	303 >=200.	Span: 1 : E Only * 0.70
Max Upward Total Deflection	0 in	Ratio =	0 <200.0	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	Summary of Shear Values VuMax	Vnx	Phi*Vnx
Dsgn. L = 19.00 ft	1		0.000				55.48	49.93	1.00	1.00	-0.00	118.20	106.38
E Only													
Dsgn. L = 19.00 ft	1	0.337	0.039	16.81		16.81	55.48	49.93	1.17	1.00	4.20	118.20	106.38

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	E Only	1.0733	9.880		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.523	4.198
Max Upward from Load Cases	2.523	4.198
E Only * 0.70	1.766	2.938
E Only * 0.5250	1.325	2.204
E Only	2.523	4.198

Steel Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Strongback (12'-0" door)--

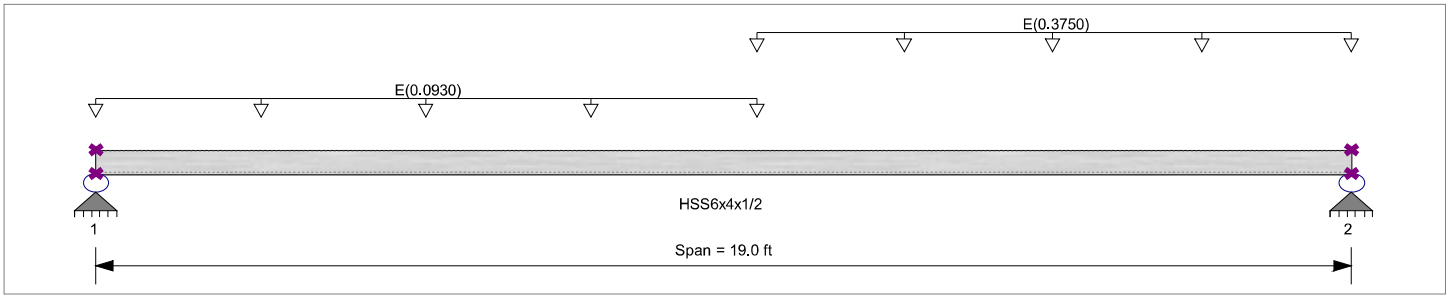
CODE REFERENCES

Calculations per AISC 360-16, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Load Resistance Factor Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending

Fy : Steel Yield : 46.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : E = 0.0930 k/ft, Extent = 0.0 -->> 10.0 ft, Tributary Width = 1.0 ft
Uniform Load : E = 0.3750 k/ft, Extent = 10.0 -->> 19.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.212 : 1	Maximum Shear Stress Ratio =	0.027 : 1
Section used for this span	HSS6x4x1/2	Section used for this span	HSS6x4x1/2
Mu : Applied	10.606 k-ft	Vu : Applied	2.820 k
Mn * Phi : Allowable	49.932 k-ft	Vn * Phi : Allowable	106.381 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	19.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.665 in	Ratio =	342 >=200.	Span: 1 : E Only
Max Upward Transient Deflection	0 in	Ratio =	0 <200.0	n/a
Max Downward Total Deflection	0.466 in	Ratio =	489 >=200.	Span: 1 : E Only * 0.70
Max Upward Total Deflection	0 in	Ratio =	0 <200.0	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	Summary of Shear Values VuMax	Vnx	Phi*Vnx
Dsgn. L = 19.00 ft	1		0.000				55.48	49.93	1.00	1.00	-0.00	118.20	106.38
E Only													
Dsgn. L = 19.00 ft	1	0.212	0.027	10.61		10.61	55.48	49.93	1.19	1.00	2.82	118.20	106.38

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	E Only	0.6656	10.043		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.485	2.820
Max Upward from Load Cases	1.485	2.820
E Only * 0.70	1.039	1.974
E Only * 0.5250	0.779	1.481
E Only	1.485	2.820

Steel Beam

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Strongback (6'-0" door)--

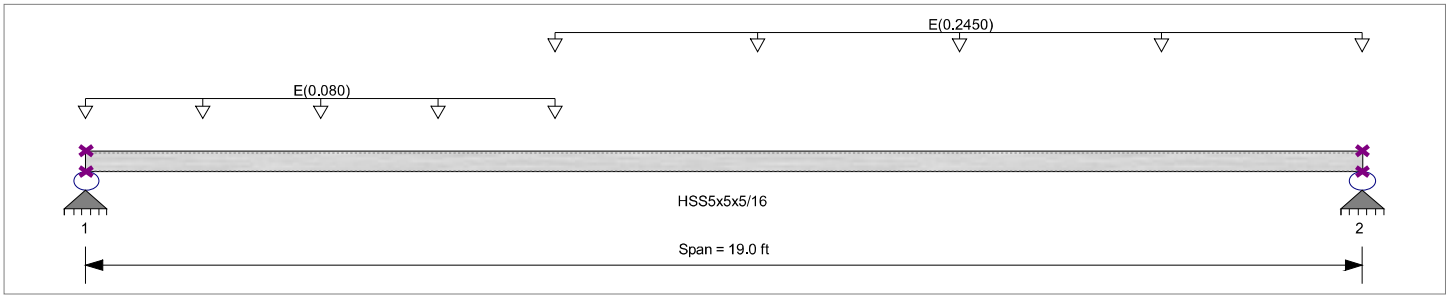
CODE REFERENCES

Calculations per AISC 360-16, IBC 2021
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Load Resistance Factor Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending

Fy : Steel Yield : 46.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : E = 0.080 k/ft, Extent = 0.0 -->> 7.0 ft, Tributary Width = 1.0 ft
Uniform Load : E = 0.2450 k/ft, Extent = 7.0 -->> 19.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.289 : 1	Maximum Shear Stress Ratio =	0.035 : 1
Section used for this span	HSS5x5x5/16	Section used for this span	HSS5x5x5/16
Mu : Applied	9.127 k-ft	Vu : Applied	2.115 k
Mn * Phi : Allowable	31.578 k-ft	Vn * Phi : Allowable	59.664 k
Load Combination	E Only	Load Combination	E Only
		Location of maximum on span	19.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	1.046 in	Ratio =	217 >=200.	Span: 1 : E Only
Max Upward Transient Deflection	0 in	Ratio =	0 <200.0	n/a
Max Downward Total Deflection	0.734 in	Ratio =	310 >=200.	Span: 1 : E Only * 0.70
Max Upward Total Deflection	0 in	Ratio =	0 <200.0	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	Summary of Shear Values VuMax	Vnx	Phi*Vnx
Dsgn. L = 19.00 ft	1		0.000				35.09	31.58	1.00	1.00	-0.00	66.29	59.66
E Only													
Dsgn. L = 19.00 ft	1	0.289	0.035	9.13		9.13	35.09	31.58	1.17	1.00	2.11	66.29	59.66

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	E Only	1.0491	9.771		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.385	2.115
Max Upward from Load Cases	1.385	2.115
E Only * 0.70	0.970	1.480
E Only * 0.5250	0.727	1.110
E Only	1.385	2.115

IDS GROUP

2022 CBC
BASE SHEAR COEFFICIENT

Job No.:
Date:
Engineer:

Equivalent Lateral Force Procedure of ASCE 7-16 §12.8

LFRS: A.9 Ordinary reinforced masonry shear walls (bearing) (conservative for new addition)

R =	2		ASCE 7-16 Table 12.2-1
Ω_0 =	2.5		ASCE 7-16 Table 12.2-1 (reduced by .5 per footnote b, if applicable)
C_d =	1.75		ASCE 7-16 Table 20.3-1

Site Specific Information

Building Height, h_n =	19	ft	
Risk Category =	III		Table 1604.5
I_e =	1.25		ASCE 7-16 Table 1.5-2
Site Class =	D		ASCE 7-16 §20.3
Seismic Design Category =	E		§1613.2.5

Mapped Acceleration Parameters (ASCE 7-16 §11.4.2)

S_s =	2.065	g	Soil report
S_1 =	0.783	g	Soil report

Site Coefficents and Spectral Acceleration Parameters (ASCE 7-16 §11.4.4)

F_a =	1		
F_v =	1.7		
S_{MS} =	2.065	g	ASCE 7-16 Eq. 11.4-1
S_{M1} =	1.332	g	ASCE 7-16 Eq. 11.4-2

Design Spectral Acceleration Parameters (ASCE 7-16 §11.4.5)

S_{DS} =	1.377	g	ASCE 7-16 Eq. 11.4-3
S_{D1} =	0.888	g	ASCE 7-16 Eq. 11.4-4

Period Determination (ASCE 7-16 §12.8.2)

Structure Type =	All other structural systems		
C_u =	1.4		ASCE 7-16 Table 12.8-1
C_t =	0.02		ASCE 7-16 Table 12.8-2
x =	0.75		ASCE 7-16 Table 12.8-2
T_a =	0.182	s	ASCE 7-16 Eq. 12.8.7

Seismic Response Coefficient Calculations (ASCE 7-16 §12.8.1.1)

C_s =	0.861		ASCE 7-16 Eq. 12.8-2
$(C_s)_{max}$ =	3.050		ASCE 7-16 Eq. 12.8-3
$(C_s)_{min}$ =	0.076		ASCE 7-16 Eq. 12.8-5
$(C_s)_{min}$ =	0.245		ASCE 7-16 Eq. 12.8-6
C_s =	0.861		***ASCE 7-16 Eq. 12.8-2 GOVERNS***

V = 0.861 * W (STR)

V = 0.615 * W (ASD)

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Corona South Mall - CBC diaphragm shear check	Date:	11/21/2025
Fastening point:	Addtion		

Specifier's comments:

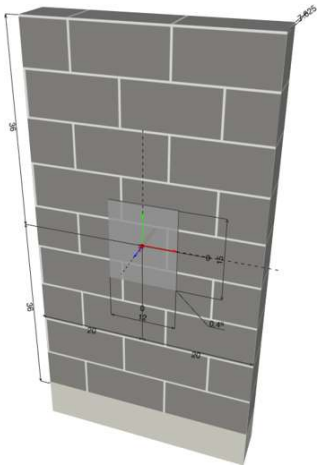
1 Input data



Anchor type and diameter:	**HY 200-A V3 + threaded rod 5.8 5/8**
Item number:	385428 HAS 5.8 5/8"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Specification text:	Hilti ∅ 5/8 HY 200-A V3 + threaded rod 5.8 with 5.625 in nominal embedment depth per Technical data , Hammer drilled installation per MPII
Effective embedment depth:	h_{ef} = 5.625 in.
Material:	5.8
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ASD Masonry
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.400 in.
Anchor plate[R]:	l_x x l_y x t = 12.000 in. x 15.000 in. x 0.400 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Joints: vertical: 0.375 in.; horizontal: 0.375 in. Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	yes

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.]



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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Corona South Mall - CBC diaphragm shear check	Date:	11/21/2025
Fastening point:	Addtion		

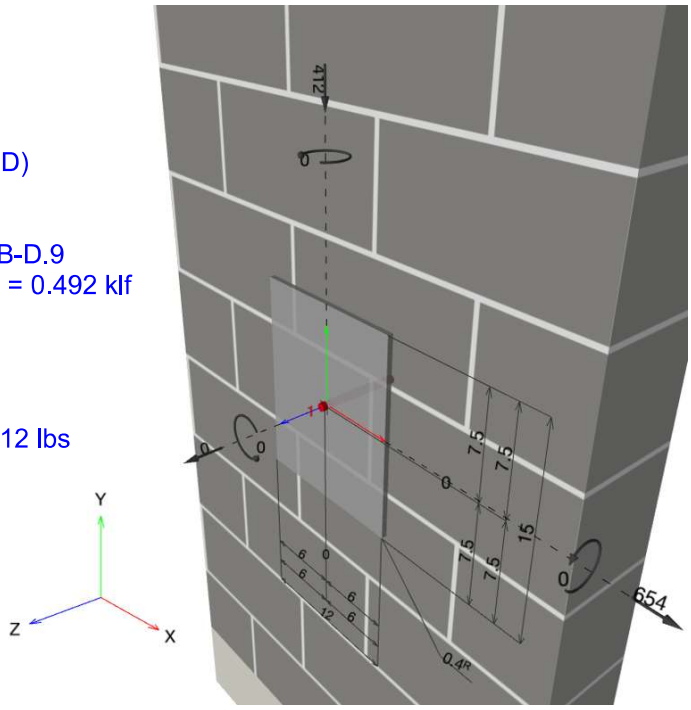
Geometry [in.] & Loading [lb, in.lb]

V = 0.7*0.861*W
= 0.7*0.861*1497 = 902 kips (ASD)

Diaphragm shear GL7 between GLB-D.9
=(1310 ft²/28076ft²)*902 kips/85.5 ft = 0.492 klf

Anchor bolt @16" o.c.
0.492 klf*1.33= 0.654 klf = 654 lbs

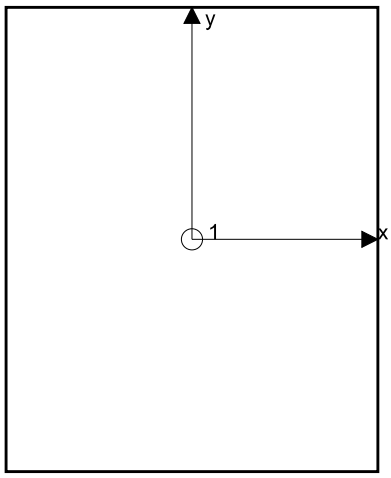
DL+LL = (20+20)*1.33 ft*15.5/2 = 412 lbs



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 654; V_y = -412; M_x = 0; M_y = 0; M_z = 0;	yes	51

2 Load case/Resulting anchor forces



Load case: Service loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	773	654	-412

max. compressive strain: - [‰]
max. compressive stress: - [psi]
Resulting tension force in (x/y)=(-/-): 0 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Corona South Mall - CBC diaphragm shear check at	Date:	11/21/2025
Fastening point:			

3 Tension load (Most utilized anchor 1)

	Load P_s [lb]	Capacity P_t [lb]	Utilization $\beta_P = P_s/P_t$ [%]	Status
Overall strength	N/A	N/A	N/A	N/A

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Corona South Mall - CBC diaphragm shear check at	Date:	11/21/2025
Fastening point:			

4 Shear load (Most utilized anchor 1)

	Load V_s [lb]	Capacity V_t [lb]	Utilization $\beta_V = V_s/V_t$ [%]	Status
Steel strength	773	3,780	21	OK
Bond strength para and perp, (Dir. x+)[1]	-	-	51	OK

[1]Shear utilization may result from parallel and perpendicular shear (see details)

4.1 Steel strength

$V_{t,s}$ = Value refer to Hilti Technical Data
$V_{t,s} \geq V_s$

Results

$V_{t,s}$ [lb]	V_s [lb]
3,780	773

4.2 Bond strength parallel

$V_{t,b,Base,||}$ = Value refer to Hilti Technical Data
$V_{t,b,||} = V_{t,b,Base,||} \cdot f_{red,E,||} \cdot f_{red,s,||} \cdot f_{red,Temp}$
$V_{t,b,||} \geq V_{s,||}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	20.000	4.000	22.500	68

Results

$V_{t,b,||}$ [lb]	$V_{t,b,Base,||}$ [lb]	$V_{s,||}$ [lb]	$f_{red,E,||}$	$f_{red,S,||}$	$f_{red,Temp}$	Utilization $\beta_{V,||}$ [%]
2,120	2,120	-412	1.000	1.000	1.000	19

4.3 Bond strength perpendicular

$V_{t,b,Base,\perp}$ = Value refer to Hilti Technical Data
$V_{t,b,\perp} = V_{t,b,Base,\perp} \cdot f_{red,E,\perp} \cdot f_{red,s,\perp} \cdot f_{red,Temp}$
$V_{t,b,\perp} \geq V_{s,\perp}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	20.000	4.000	22.500	68

Results

$V_{t,b,\perp}$ [lb]	$V_{t,b,Base,\perp}$ [lb]	$V_{s,\perp}$ [lb]	$f_{red,E,\perp}$	$f_{red,S,\perp}$	$f_{red,Temp}$	Utilization $\beta_{V,\perp}$ [%]
2,120	2,120	654	1.000	1.000	1.000	31

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Design:	Corona South Mall - CBC diaphragm shear check at	Date:	11/21/2025
Fastening point:			

4.4 Shear interaction

$\beta_{V,\parallel} = \frac{V_{s,\parallel}}{V_{t,\parallel}}$	$\beta_{V,\perp} = \frac{V_{s,\perp}}{V_{t,\perp}}$	δ	Utilization β_V [%]	Status
0.194	0.308	1.000	51	OK

$\beta_V = \beta_{V,\parallel}^{\delta} + \beta_{V,\perp}^{\delta} <= 1.0$

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
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- Refer to the manufacturer's product literature for cleaning and installation instructions.
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
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- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry should be built according to industry standards.

Fastening meets the design criteria!

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Design:	Corona South Mall - CBC diaphragm shear check at	Date:	11/21/2025
Fastening point:			

6 Installation data

Anchor type and diameter: HY 200-A V3 + threaded rod 5.8 5/8

Profile: no profile

Item number: 385428 HAS 5.8 5/8"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Hole diameter in the fixture: d_f = 0.687 in.

Maximum installation torque: 720 in.lb

Plate thickness (input): 0.400 in.

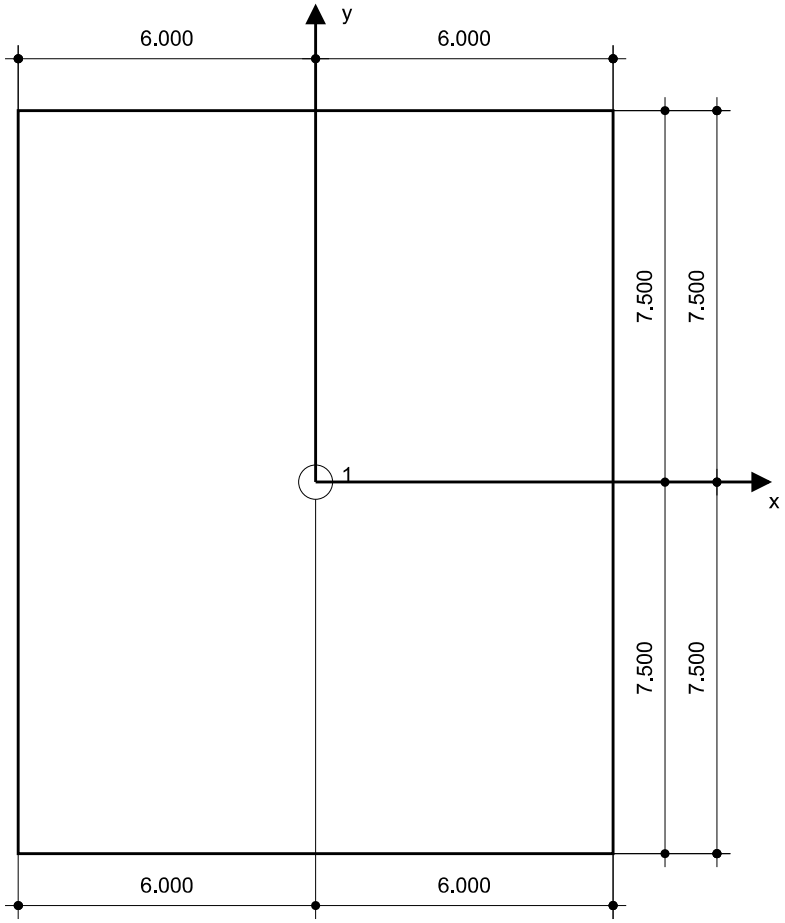
Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 5.625 in.

Drilling method: Drilled in hammer mode

Minimum thickness of the base material: 7.625 in.

Hilti ∅ 5/8 HY 200-A V3 + threaded rod 5.8 with 5.625 in nominal embedment depth per Technical data , Hammer drilled installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	20.000	20.000	36.000	36.000

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Design:	Corona South Mall - CBC diaphragm shear check at	Date:	11/21/2025
Fastening point:			

7 Remarks; Your Cooperation Duties

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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Specifier's comments:

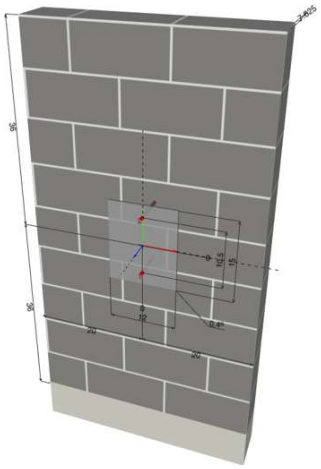
1 Input data



Anchor type and diameter:	**HY 200-A V3 + threaded rod 5.8 5/8**
Item number:	385428 HAS 5.8 5/8"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Specification text:	Hilti ∅ 5/8 HY 200-A V3 + threaded rod 5.8 with 5.625 in nominal embedment depth per Technical data , Hammer drilled installation per MPII
Effective embedment depth:	h_{ef} = 5.625 in.
Material:	5.8
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ASD Masonry
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.400 in.
Anchor plate[R]:	l_x x l_y x t = 12.000 in. x 15.000 in. x 0.400 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Joints: vertical: 0.375 in.; horizontal: 0.375 in. Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	yes

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.]



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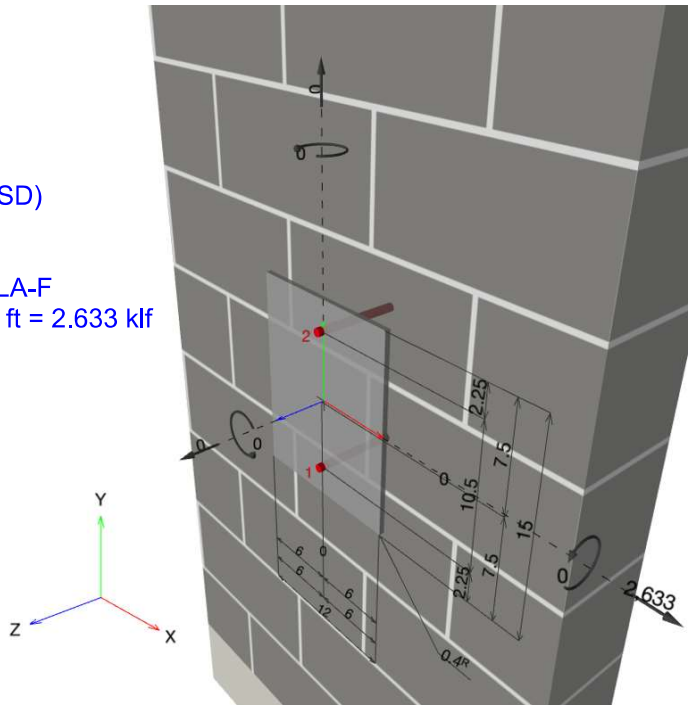
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Geometry [in.] & Loading [lb, in.lb]

V = 0.7*0.861*W
= 0.7*0.861*1497 = 902 kips (ASD)

Diaphragm shear GL1 between GLA-F
=(12300 ft²/28076ft²)*902 kips/150 ft = 2.633 klf



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 2,633; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	yes	92

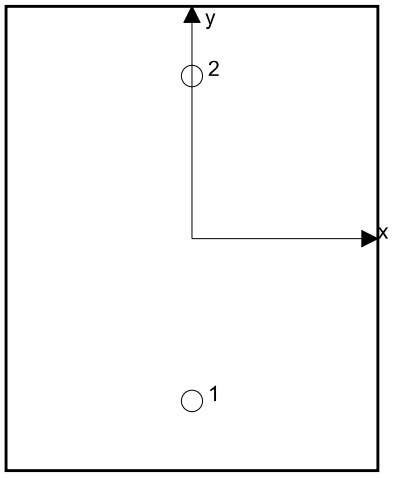
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Fastening point:			

2 Load case/Resulting anchor forces



Load case: Service loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	1,317	1,317	0
2	0	1,317	1,317	0

max. compressive strain: - [‰]
max. compressive stress: - [psi]
Resulting tension force in (x/y)=(-/-): 0 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Fastening point:			

3 Tension load (Most utilized anchor 1)

	Load P_s [lb]	Capacity P_t [lb]	Utilization $\beta_P = P_s/P_t$ [%]	Status
Overall strength	N/A	N/A	N/A	N/A

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Fastening point:			

4 Shear load (Most utilized anchor 1)

	Load V_s [lb]	Capacity V_t [lb]	Utilization $\beta_V = V_s/V_t$ [%]	Status
Steel strength	1,317	3,780	35	OK
Bond strength para and perp, (Dir. x+)[1]	-	-	92	OK

[1]Shear utilization may result from parallel and perpendicular shear (see details)

4.1 Steel strength

$V_{t,s}$ = Value refer to Hilti Technical Data
$V_{t,s} \geq V_s$

Results

$V_{t,s}$ [lb]	V_s [lb]
3,780	1,317

4.2 Bond strength parallel

$V_{t,b,Base,||}$ = Value refer to Hilti Technical Data
$V_{t,b,||} = V_{t,b,Base,||} \cdot f_{red,E,||} \cdot f_{red,s,||} \cdot f_{red,Temp}$
$V_{t,b,||} \geq V_{s,||}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	20.000	4.000	22.500	68

Results

$V_{t,b,||}$ [lb]	$V_{t,b,Base,||}$ [lb]	$V_{s,||}$ [lb]	$f_{red,E,||}$	$f_{red,S,||}$	$f_{red,Temp}$	Utilization $\beta_{V,||}$ [%]
0	2,120	0	0.000	0.000	1.000	0

4.3 Bond strength perpendicular

$V_{t,b,Base,\perp}$ = Value refer to Hilti Technical Data
$V_{t,b,\perp} = V_{t,b,Base,\perp} \cdot f_{red,E,\perp} \cdot f_{red,s,\perp} \cdot f_{red,Temp}$
$V_{t,b,\perp} \geq V_{s,\perp}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	20.000	4.000	22.500	68

Results

$V_{t,b,\perp}$ [lb]	$V_{t,b,Base,\perp}$ [lb]	$V_{s,\perp}$ [lb]	$f_{red,E,\perp}$	$f_{red,S,\perp}$	$f_{red,Temp}$	Utilization $\beta_{V,\perp}$ [%]
1,432	2,120	1,317	1.000	0.676	1.000	92

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4.4 Shear interaction

$\beta_{V,\parallel} = \frac{V_{s,\parallel}}{V_{t,\parallel}}$	$\beta_{V,\perp} = \frac{V_{s,\perp}}{V_{t,\perp}}$	δ	Utilization β_V [%]	Status
0.000	0.919	1.000	92	OK

$\beta_V = \beta_{V,\parallel}^{\delta} + \beta_{V,\perp}^{\delta} <= 1.0$

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
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Fastening meets the design criteria!

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Design:	Corona South Mall - CBC diaphragm shear check	Date:	11/21/2025
Fastening point:			

6 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = 0.687 in.

Plate thickness (input): 0.400 in.

Drilling method: Drilled in hammer mode

Anchor type and diameter: HY 200-A V3 + threaded rod 5.8 5/8

Item number: 385428 HAS 5.8 5/8"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

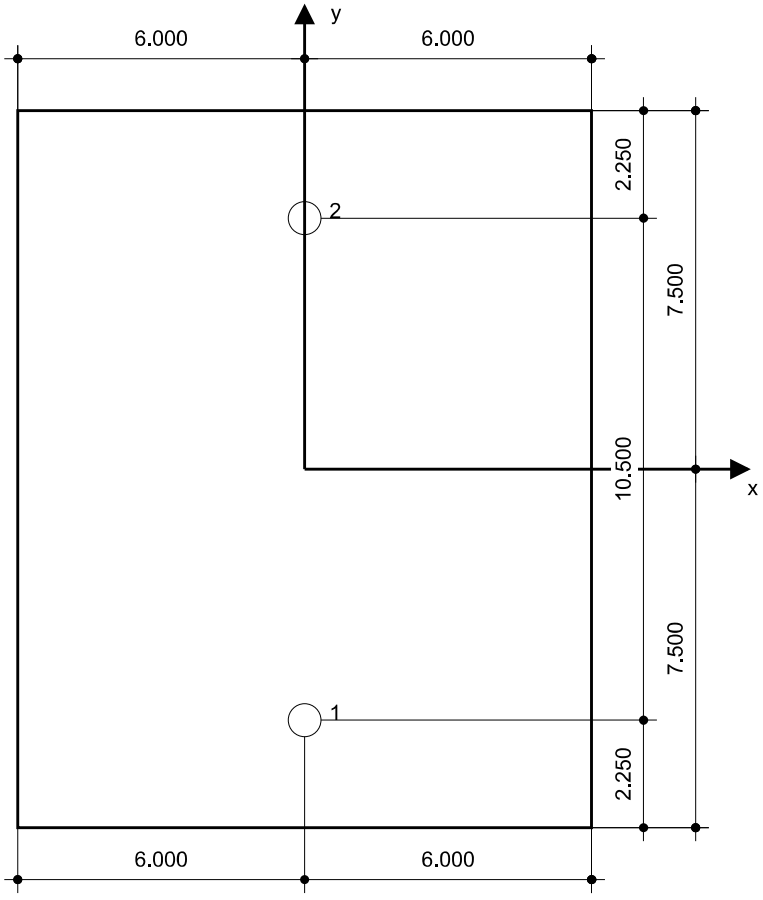
Maximum installation torque: 720 in.lb

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 5.625 in.

Minimum thickness of the base material: 7.625 in.

Hilti ∅ 5/8 HY 200-A V3 + threaded rod 5.8 with 5.625 in nominal embedment depth per Technical data , Hammer drilled installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	-5.250	20.000	20.000	30.750	41.250
2	0.000	5.250	20.000	20.000	41.250	30.750

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Design:	Corona South Mall - CBC diaphragm shear check	Date:	11/21/2025
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Masonry Slender Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Addtion - masonry wall oop --

Code References

Calculations per TMS 402-16, IBC 2021
Load Combinations Used : ASCE 7-22 / IBC 2024

General Information

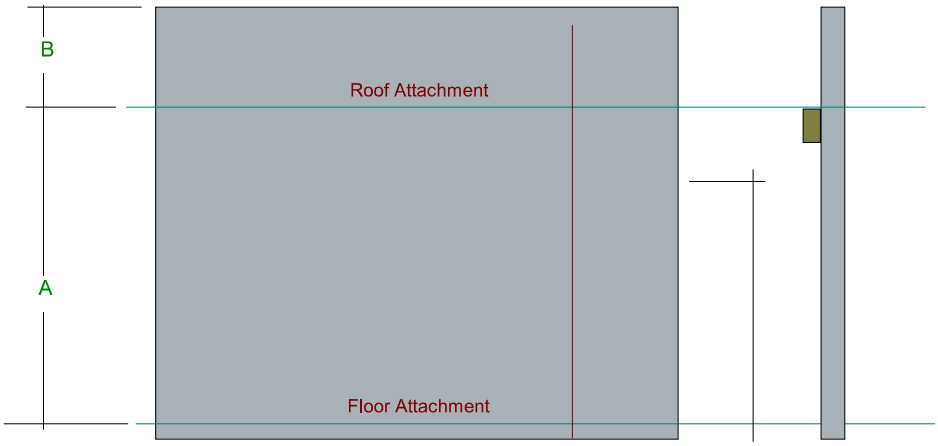
Calculations per TMS 402-16, IBC 2021

Construction Type :	Grouted Hollow Concrete Masonry							
F'm	=	1.50 ksi	Nom. Wall Thickness	8	in	Temp Diff across thickness	=	deg F
Fy - Yield	=	60.0 ksi	Actual Thickness	7.625	in	Min Allow Out-of-plane Defl Rat	=	0.0
Fr - Rupture	=	61.0 psi	Rebar "d" distance	3.8125	in			
Em = f'm *	=	900.0	Lower Level Rebar . . .			Minimum Vertical Steel %	=	0.0020
Max % of ρ bal.	=	0.007030	Bar Size	#	5			
Grout Density	=	140 pcf	Bar Spacing		16 in			
Block Weight		Normal Weight						
Wall Weight	=	86.0 psf						
		Wall is Solid Grouted						

One-Story Wall Dimensions

A Clear Height	=	13.0 ft
B Parapet height	=	2 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	2.0 in	0.1250	0.1560			k/ft
Concentric Load							k/ft

Lateral Loads

Wind Loads :

Full area WIND load 20.0 psf

Seismic Loads :

Wall Weight Seismic Load Input Method : ASCE seismic factors entered

SDS Value per ASCE 12.11.1 S_{DS} * I = 1.720

Fp = Wall Wt. * 0.6880 = 59.168 psf

Masonry Slender Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Addtion - masonry wall oop --

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS Moment Capacity Check +0.90D+E	Maximum Bending Stress Ra	0.3619		
	Max Mu	1.224 k-ft	Phi * Mn	3.382 k-ft
PASS Service Deflection Check E Only	Actual Defl. Ratio L/	400	Allowable Defl. Ratio	150.0
	Max. Deflection	0.3896 in		
PASS Axial Load Check +1.20D+E	Max Pu / Ag	11.707 psi	Max. Allow. Defl.	1.040 in
	Location	6.283 ft	0.2 * f'm	300.0 psi
Reinforcing Limit Check	Actual As/bd	0.005082	Max Allow As/bd	0.007030

Maximum Reactions *for Load Combination....*

Top Horizontal	E Only	0.5120 k
Base Horizontal	E Only	0.3755 k
Vertical Reaction	+D+Lr	1.571 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Axial Load 0.2*f'm*b*t k	Moment Values Mcr k-ft	Moment Values Mu k-ft	Moment Values Phi	Moment Values Phi Mn k-ft	As in^2	As Ratio	0.6 * rho bal	Bar 'd'
+1.40D at 12.57 to 13.00	0.468	27.468	0.59	0.03	0.90	3.31	0.233	0.0051	0.0070	0.00
+1.20D+0.50Lr at 12.57 to 13.00	0.479	27.468	0.59	0.04	0.90	3.31	0.233	0.0051	0.0070	0.00
+1.20D at 12.57 to 13.00	0.401	27.468	0.59	0.02	0.90	3.29	0.233	0.0051	0.0070	0.00
+1.20D+1.60Lr at 12.57 to 13.00	0.651	27.468	0.59	0.07	0.90	3.35	0.233	0.0051	0.0069	0.00
+1.20D+1.60Lr+0.50W at 6.50 to 6.93	1.277	27.468	0.59	0.24	0.90	3.49	0.233	0.0051	0.0067	0.00
+1.20D+0.50W at 6.50 to 6.93	1.027	27.468	0.59	0.21	0.90	3.43	0.233	0.0051	0.0068	0.00
+1.20D+0.50Lr+W at 6.07 to 6.50	1.150	27.468	0.59	0.42	0.90	3.46	0.233	0.0051	0.0067	0.00
+1.20D+W at 6.07 to 6.50	1.072	27.468	0.59	0.42	0.90	3.44	0.233	0.0051	0.0067	0.00
+0.90D+W at 6.07 to 6.50	0.804	27.468	0.59	0.41	0.90	3.38	0.233	0.0051	0.0068	0.00
+1.20D+E at 6.07 to 6.50	1.072	27.468	0.59	1.24	0.90	3.44	0.233	0.0051	0.0067	0.00
+0.90D+E at 6.07 to 6.50	0.804	27.468	0.59	1.22	0.90	3.38	0.233	0.0051	0.0068	0.00

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 7.37 to 7.80	0.781	0.59	0.01	443.30	41.63	443.300	0.001	238794.5
+D+Lr at 7.37 to 7.80	0.937	0.59	0.03	443.30	41.94	443.300	0.001	106158.1
+D+0.750Lr at 7.37 to 7.80	0.898	0.59	0.02	443.30	41.87	443.300	0.001	123285.8
+D+0.60W at 6.07 to 6.50	0.893	0.59	0.25	443.30	41.86	443.300	0.013	12,180.8
+D+0.750Lr+0.450W at 6.50 to 6.93	0.973	0.59	0.20	443.30	42.01	443.300	0.010	15,041.6
+D+0.450W at 6.50 to 6.93	0.856	0.59	0.19	443.30	41.78	443.300	0.010	15,976.7
+0.60D+0.60W at 6.07 to 6.50	0.536	0.59	0.25	443.30	41.14	443.300	0.013	12,442.6
+D+0.70E at 6.07 to 6.50	0.893	0.59	0.86	443.30	41.86	46.812	0.167	932.6
+D+0.5250E at 6.07 to 6.50	0.893	0.59	0.64	443.30	41.86	70.615	0.045	3,449.4
+0.60D+0.70E at 6.07 to 6.50	0.536	0.59	0.85	443.30	41.14	46.251	0.164	952.0
Lr Only at 7.37 to 7.80	0.156	0.59	0.02	443.30	40.36	443.300	0.001	191823.0
W Only at 6.07 to 6.50	0.000	0.59	0.40	443.30	40.04	443.300	0.020	7,713.4
E Only at 6.07 to 6.50	0.000	0.59	1.19	443.30	40.04	41.357	0.390	400.5

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	1.415 k

Masonry Slender Wall

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Addtion - masonry wall oop --

+D+Lr	0.0 k	0.00 k	1.571 k
+D+0.750Lr	0.0 k	0.00 k	1.532 k
+D+0.60W	0.1 k	0.10 k	1.415 k

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
+D+0.750Lr+0.450W	0.1 k	0.07 k	1.532 k
+D+0.450W	0.1 k	0.08 k	1.415 k
+0.60D+0.60W	0.1 k	0.10 k	0.849 k
+D+0.70E	0.3 k	0.36 k	1.415 k
+D+0.5250E	0.2 k	0.27 k	1.415 k
+0.60D+0.70E	0.3 k	0.36 k	0.849 k
Lr Only	0.0 k	0.00 k	0.156 k
W Only	0.1 k	0.17 k	0.000 k
E Only	0.4 k	0.51 k	0.000 k

IDS GROUP

Out-of-Plane Wall Anchorage

Job No.: 24X015.00
Date: 6/5/25
Engineer:

Grid line 7 wall anchor

Anchorage Demand per ASCE7 §12.11.2.1

Wall weight =	84	psf	
To roof height =	13	ft	
Parapet =	2	ft	
S_{DS} =	1.38	g	
L_f =	16	ft	
K_a =	1.2		
I_e =	1.25		
W_p =	714	plf	
$F_p = 0.4\ S_{DS}K_aI_eW_p$ =	570	plf	Eq. 12.11-1
Anchor spacing =	8.00	ft	
F_p =	4.56	kips	

f'_m =	2000	psi	
D =	0.625	in	5/8"ϕ bolt
A_b =	0.2260	in^2	
f_y =	36	ksi	
s=	8.00	in	
l_b =	6	in	
A'_{pt} =	583.04	in^2	projected area

Strength of bolt loaded in tension

Masonry tensile breakout

$\emptyset B_{anb} = \emptyset 4 A'_{pt}\sqrt{f'_m}$ = 52.1 kips

Steel tensile yield

$\emptyset B_{ans} = \emptyset A_b f_y$ = 7.3 kips governs

Strength of bolt loaded in shear

DL = 0.2 kips DL 20 psf, 1.33 ft trib.

Masonry shear crushing

$\emptyset B_{vc} = \emptyset 1050 \sqrt[4]{f'_m A_b}$ = 2.4 kips governs

Anchor shear pryout

$\emptyset B_{vpry} = \emptyset 8 A'_{pt}\sqrt{f'_m}$ = 104.3 kips

Steel shear yield

$\emptyset B_{vs} = \emptyset 0.6 A_b f_y$ = 4.4 kips

Load combination 1.2D+E

$\frac{F_p}{\emptyset B_{nab}} + \frac{1.2D}{\emptyset B_{vc}}$ = 0.02 < 1.0, OK

IDS GROUP

Out-of-Plane Wall Anchorage

Job No.: 24X015.00
Date: 6/5/25
Engineer:

Grid line 6.4 & 7 wall anchor (area with metal deck roof)

Anchorage Demand per ASCE7 §12.11.2.1

Wall weight =	84	psf	
To roof height =	13	ft	
Parapet =	2	ft	
S_{DS} =	1.38	g	
L_f =	10	ft	
K_a =	1.1		
I_e =	1.25		
W_p =	714	plf	
$F_p = 0.4\ S_{DS}K_aI_eW_p$ =	538	plf	Eq. 12.11-1
Anchor spacing =	6.75	ft	
F_p =	3.63	kips	(W beam @6.75 ft o.c.)
f'$_m$ =	2000	psi	
D =	0.750	in	3/4"ϕ bolt
A_b =	0.3345	in^2	
f_y =	36	ksi	
s=	8.00	in	
l_b =	6	in	
A'_{pt} =	583.04	in^2	projected area

Strength of bolt loaded in tension

Masonry tensile breakout

$\emptyset B_{anb} = \emptyset 4 A'_{pt}\sqrt{f'_m} =$ 52.1 kips

Steel tensile yield

$\emptyset B_{ans} = \emptyset A_b f_y =$ 10.8 kips governs

Strength of bolt loaded in shear

DL = 0.2 kips DL 20 psf, 1.33 ft trib.

Masonry shear crushing

$\emptyset B_{vc} = \emptyset 1050 \sqrt[4]{f'_m A_b} =$ 2.7 kips governs

Anchor shear pryout

$\emptyset B_{vpry} = \emptyset 8 A'_{pt}\sqrt{f'_m} =$ 104.3 kips

Steel shear yield

$\emptyset B_{vs} = \emptyset 0.6 A_b f_y =$ 6.5 kips

Load combination 1.2D+E

$\frac{F_p}{\emptyset B_{nab}} + \frac{1.2D}{\emptyset B_{vc}} =$ 0.04 < 1.0, OK

Steel Column

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --W8x28 subjected to out-of-plane force--

Code References

Calculations per AISC 360-16, IBC 2021
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **W8x28**

Analysis Method :	Load Resistance Factor	Overall Column Height	8.75 ft
Steel Stress Grade		Top & Bottom Fixity	Top & Bottom Pinned
Fy : Steel Yield	50 ksi	Brace condition :	
E : Elastic Bending Modulus	29,000.0 ksi		

Unbraced Length for buckling ABOUT X-X Axis = 8.75 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 8.75 ft, K = 1.0

Applied Loads

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

AXIAL LOADS . . .
Axial Load at 8.750 ft, E = 3.630 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.01331** : 1

Load Combination	E Only
Location of max.above base	0.0 ft
At maximum location values are . . .	
Pu	3.630 k
0.9 * Pn	272.735 k
Mu-x	0.0 k-ft
0.9 * Mn-x :	82.175 k-ft
Mu-y	0.0 k-ft
0.9 * Mn-y :	37.875 k-ft

PASS Maximum Shear Stress Rati **0.0** : 1

Load Combination	0.0
Location of max.above base	0.0 ft
At maximum location values are . . .	
Vu : Applied	0.0 k
Vn * Phi : Allowable	0.0 k

Maximum Load Reactions . .

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

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :

Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
	0.000	PASS	0.00 ft	0.00	0.00	0.00	0.00	0.000	PASS	0.00 ft
E Only	0.013	PASS	0.00 ft	1.00	1.00	30.43	64.81	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
	k					k-ft			
E Only * 0.70	2.541								
E Only * 0.5250	1.906								
E Only	3.630								

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
		k					k-ft			
Axial @ Base	Maximum	3.630								
"	Minimum									
Reaction, X-X Axis Base	Maximum									
"	Minimum									
Reaction, Y-Y Axis Base	Maximum									
"	Minimum									
Reaction, X-X Axis Top	Maximum									
"	Minimum									

Steel Column

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --W8x28 subjected to out-of-plane force--

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
Reaction, Y-Y Axis Top	Maximum										
"	Minimum										
Moment, X-X Axis Base	Maximum										
"	Minimum										
Moment, Y-Y Axis Base	Maximum										
"	Minimum										
Moment, X-X Axis Top	Maximum										
"	Minimum										
Moment, Y-Y Axis Top	Maximum										
"	Minimum										

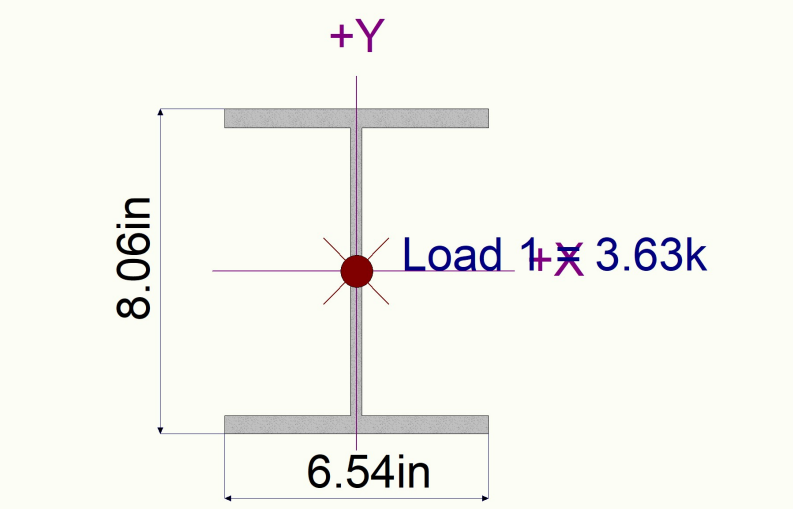
Maximum Deflections for Load Combinations

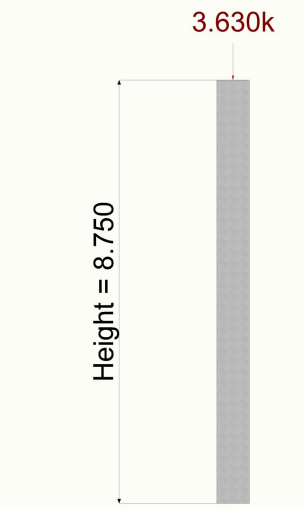
Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only * 0.70	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only * 0.5250	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

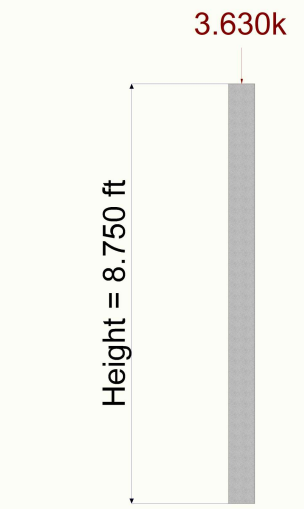
Steel Section Properties : W8x28

Depth	=	8.060 in	I xx	=	98.00 in^4	J	=	0.537 in^4
Web Thick	=	0.285 in	S xx	=	24.30 in^3	Cw	=	312.00 in^6
Flange Width	=	6.540 in	R xx	=	3.450 in			
Flange Thick	=	0.465 in	Zx	=	27.200 in^3			
Area	=	8.240 in^2	I yy	=	21.700 in^4			
Weight	=	28.049 plf	S yy	=	6.630 in^3	Wno	=	12.400 in^2
Kdesign	=	0.859 in	R yy	=	1.620 in	Sw	=	9.440 in^4
K1	=	0.625 in	Zy	=	10.100 in^3	Qf	=	5.520 in^3
rts	=	1.840 in				Qw	=	13.400 in^3
Ycg	=	0.000 in						

Sketches







Masonry Shear Wall

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Grid line 7 --

Code References

Calculations per TMS 402-16, IBC 2021
Load Combinations Used : ASCE 7-22 / IBC 2024

General Information

Wall Material	MASONRY	f'm	1.50 ksi	Block Class	
Total Wall Height	13.0 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	78.0 ft	Fy - HJR	70.0 ksi	Min. Bending As %	0.00180
R: Resp. Mod Factor		Em	3,120.0 ksi		
Ie: Seismic Import. Factor	1.0	Phi - Shear	0.80	Phi : Axial & Flexure	0.90

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	78.0 ft
Effective Length 'd'	928.0 in
Nominal Block Thickness	8 in
Solid Grout?	Solid Grouted

Reinforcing in Field of Wall

Vertical Bar Size #	5
Vertical Bar Spacing	16 in
Horiz. joint reinf. area (HJR	0.55 in
HJR Spacing	16 in
Bond beam reinf. area	0.4 in
Spacing of bond beams	48 in

In each chord cell:

Vertical rebar size #	6
# Chord Cells @ Each End	2

$V_{grid\ 7}$ = 0.861*[(15.6 ft/2*(20 psf+10 psf)+84 psf*78 ft* 15 ft+84 psf*7.5 ft*25.6 ft]/1000 = 98 kips
$DL_{grid\ 7}$ = 16 psf *(15.6 ft/2) = 125 plf
$LL_{grid\ 7}$ = 20 psf *(15.6 ft/2) = 156 plf

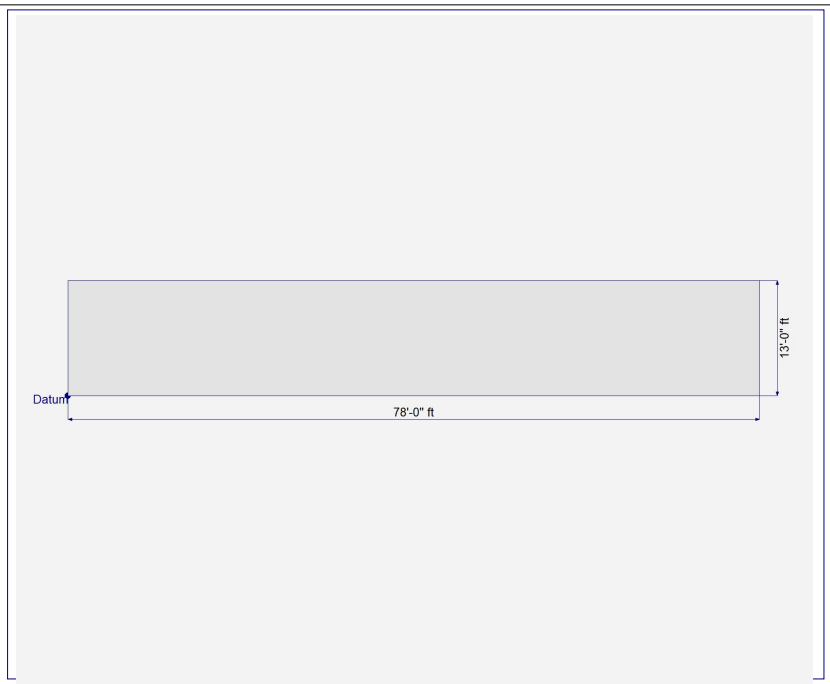
Masonry Shear Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Grid line 7 --

Wall Sketch



Applied Distributed Vertical Loads

Load Location (ft)			Load Magnitude (kips)				
Start Location	End Location	Height of Application	Dead Load	Roof Live Load	Live Load	Snow Load	Earth Load
	78.0	13.0	0.1250	0.1560			

Applied Concentrated Lateral Loads

	Load Magnitude (kips)					
Load "Y" Location (ft)	Dead Load	Roof Live Load	Floor Live Load	Wind Load	Seismic Load	Earth Load
13.0					98.0	

SHEAR ANALYSIS

	Bottom Level
Special Boundary Elements Req'd?	Not Req'd
Vu : Story Shear	98.0 k
for Load Combination	+1.20D+E
Anv	7,113.60 in^2
Controlling Mu/(Vud)	0.17
Vn Masonry	1,055.63 k
Vn Steel	181.350 k
Vn Masonry + Vn Steel	1,236.98 k
Vn Max	1,653.05 k
Phi Vn	989.58 k
Ratio: Vu/PhiVn (controlling)	**0.09903**
Vertical As >= Av/3	OK
Vertical Bar Spacing <= 96"	OK

Masonry Shear Wall

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Grid line 7 --

AXIAL ANALYSIS

	Bottom Level
H / d Ratio	0.17
Pu	135.814 k
for Load Combination	+1.20D+1.60Lr
Phi Pn	1.20D+1.60Lr k
Ratio: Pu/PhiPn (controlling)	**0.01717**

BENDING ANALYSIS

	Bottom Level
"a" : Flexural compression	5.77 in
Length of defined chord zone is >= the "a" dimension of the masonry (the compression zone)	OK
"d" : Eff depth to tension reinf	928.0
As-flex < As-max ?	J<=1, R<=1.5
Mu	1,274.0 k
for Load Combination	+1.20D+E
Phi Mn	3,663.45 k
Ratio: Mu/PhiMn (controlling)	**0.3478**

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			135.736				
+1.20D+0.50Lr							
Wall Level : 1			122.429				
+1.20D							
Wall Level : 1			116.345				
+1.20D+1.60Lr							
Wall Level : 1			135.814				
+0.90D							
Wall Level : 1			87.259				
+1.20D+E							
Wall Level : 1	98.000	1,274.000	116.345	10.950	3.562		
+0.90D+E							
Wall Level : 1	98.000	1,274.000	87.259	14.600	2.671		

Masonry Shear Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Grid line 7 --

Footing Information

Footing Dimensions

Dist. Left	1.0 ft	f'c	4.0 ksi	Rebar Cover	3.0 in
Wall Length	78.0 ft	Fy	60.0 ksi	Footing Thickness	18.0 in
Dist. Right	1.0 ft			Width	2.0 ft
Total Ftg Length	80.0 ft				

Max Factored Soil Pressures

@ Left Side of Footing	1,164.11	psf
.... governing load comb	+1.40D	
@ Right Side of Footing	1,663.90	psf
.... governing load comb	+1.20D+E	

Max UNfactored Soil Pressures

@ Left Side of Footing	907.56	psf
.... governing load comb	+D+Lr	
@ Right Side of Footing	1,297.77	psf
.... governing load comb	+D+0.70E	

Footing One-Way Shear Check...

vu @ Left End of Footing	0.0 psi
Phi * vn : Allowable	49.611 psi
vu / Phi * vn	0.00
vu @ Right End of Footing	0.0 psi
Phi * vn : Allowable	49.61 psi
vu / Phi * vn	0.00

Overturning Stability...

	@ Left End of Ftg	@ Right End of Ftg
Overturning Moment	1,043.53 k-ft	1,043.53 k-ft
Resisting Moment	3,018.10 k-ft	3,018.10 k-ft
Stability Ratio	2.892 : 1	2.892 : 1
.... governing load comb	+0.60D+0.70E	+0.60D+0.70E

Footing Bending Design...

	@ Left End	@ Right End
Mu	1.164 k-ft	1.658 k-ft
Ru	2.874 psi	4.095 psi
As % Req'd	0.00180 in^2	0.00180 in^2
As Req'd in Footing Width	0.7776 in^2	0.7776 in^2

Masonry Shear Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Grid line D.3 --

Code References

Calculations per TMS 402-16, IBC 2021
Load Combinations Used : ASCE 7-22 / IBC 2024

General Information

Wall Material	MASONRY	f'm	1.50 ksi	Block Class	
Total Wall Height	13.0 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	10.0 ft	Fy - HJR	70.0 ksi	Min. Bending As %	0.00180
R: Resp. Mod Factor	1	Em	3,120.0 ksi		
Ie: Seismic Import. Factor	1.0	Phi - Shear	0.80	Phi : Axial & Flexure	0.90

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	10.0 ft
Effective Length 'd'	104.0 in
Nominal Block Thickness	8 in
Solid Grout?	Solid Grouted

Reinforcing in Field of Wall

Vertical Bar Size #	4
Vertical Bar Spacing	16 in
Horiz. joint reinf. area (HJR)	0.55 in
HJR Spacing	16 in
Bond beam reinf. area	0.4 in
Spacing of bond beams	48 in

In each chord cell:

Vertical rebar size #	5
# Chord Cells @ Each End	4.0

Masonry Shear Wall

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Grid line C.1 --

Code References

Calculations per TMS 402-16, IBC 2021
Load Combinations Used : ASCE 7-22 / IBC 2024

General Information

Wall Material MASONRY		f'm	1.50 ksi	Block Class	
Total Wall Height	13.0 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	10.0 ft	Fy - HJR	70.0 ksi	Min. Bending As %	0.00180
R: Resp. Mod Factor		Em	3,120.0 ksi		
Ie: Seismic Import. Factor	1.0	Phi - Shear	0.80	Phi : Axial & Flexure	0.90

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	10.0 ft
Effective Length 'd'	108.0 in
Nominal Block Thickness	8 in
Solid Grout?	Solid Grouted

Reinforcing in Field of Wall

Vertical Bar Size #	5
Vertical Bar Spacing	16 in
Horiz. joint reinf. area (HJR	0.55 in
HJR Spacing	16 in
Bond beam reinf. area	0.4 in
Spacing of bond beams	48 in

In each chord cell:

Vertical rebar size #	5
# Chord Cells @ Each End	3.0

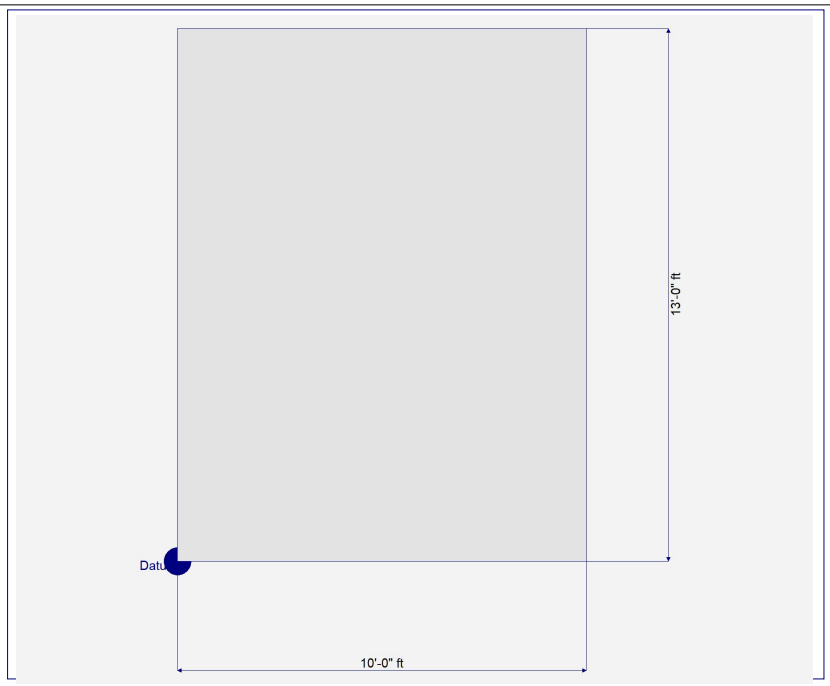
$V_{grid\ C.1}$ = 0.861*[(44 ft*(20 psf+10 psf)+84 psf*10 ft* 13 ft+84 psf*44 ft*(13/2) ft]/1000 = 31 kips

Masonry Shear Wall

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Grid line C.1 --

Wall Sketch



Applied Concentrated Lateral Loads

	Load Magnitude (kips)					
Load "Y" Location (ft)	Dead Load	Roof Live Load	Floor Live Load	Wind Load	Seismic Load	Earth Load
13.0	0.0	0.0	0.0	0.0	31.0	0.0

SHEAR ANALYSIS

	Bottom Level
Special Boundary Elements Req'd?	Not Req'd
Vu : Story Shear	31.0 k
for Load Combination	+1.20D+E
Anv	912.0 in^2
Controlling Mu/(Vud)	1.00
Vn Masonry	83.387 k
Vn Steel	139.50 k
Vn Masonry + Vn Steel	222.887 k
Vn Max	141.286 k
Phi Vn	113.029 k
Ratio: Vu/PhiVn (controlling)	**0.2743**
Vertical As >= Av/3	OK
Vertical Bar Spacing <= 96"	OK

Masonry Shear Wall

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Grid line C.1 --

AXIAL ANALYSIS

	Bottom Level
H / d Ratio	1.44
Pu	15.652 k
for Load Combination	+1.40D
Phi Pn	+1.40D k
Ratio: Pu/PhiPn (controlling)	**0.01528**

BENDING ANALYSIS

	Bottom Level
"a" : Flexural compression	6.10 in
Length of defined chord zone is >= the "a" dimension of the masonry (the compression zone)	OK
"d" : Eff depth to tension reinf	108.0
As-flex < As-max ?	930 <= 7.297
Mu	403.0 k
for Load Combination	+1.20D+E
Phi Mn	439.219 k
Ratio: Mu/PhiMn (controlling)	**0.9175**

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			15.652				
+1.20D							
Wall Level : 1			13.416				
+0.90D							
Wall Level : 1			10.062				
+1.20D+E							
Wall Level : 1	31.000	403.000	13.416	30.039	0.167	37.324	37.324
+0.90D+E							
Wall Level : 1	31.000	403.000	10.062	40.052	0.125	39.188	39.188

Masonry Shear Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: --Grid line C.1 --

Footing Information

Footing Dimensions

Dist. Left	1.0 ft	f'c	4.0 ksi	Rebar Cover	3.0 in
Wall Length	10.0 ft	Fy	60.0 ksi	Footing Thickness	18.0 in
Dist. Right	1.0 ft			Width	2.0 ft
Total Ftg Length	12.0 ft				

Max Factored Soil Pressures

@ Left Side of Footing	967.75	psf
.... governing load comb	+1.40D	
@ Right Side of Footing	967.75	psf
.... governing load comb	+1.40D	

Max UNfactored Soil Pressures

@ Left Side of Footing	691.25	psf
.... governing load comb	D Only	
@ Right Side of Footing	691.25	psf
.... governing load comb	D Only	

Footing One-Way Shear Check...

vu @ Left End of Footing	0.0 psi
Phi * vn : Allowable	49.611 psi
vu / Phi * vn	0.00
vu @ Right End of Footing	0.0 psi
Phi * vn : Allowable	49.61 psi
vu / Phi * vn	0.00

Overturning Stability...

	@ Left End of Ftg	@ Right End of Ftg
Overturning Moment	320.911 k-ft	320.911 k-ft
Resisting Moment	55.80 k-ft	55.80 k-ft
Stability Ratio	0.1739 : 1	0.1739 : 1
.... governing load comb	+0.60D+0.70E	+0.60D+0.70E

Footing Bending Design...

	@ Left End	@ Right End
Mu	0.9678 k-ft	0.9678 k-ft
Ru	2.390 psi	2.390 psi
As % Req'd	0.00180 in^2	0.00180 in^2
As Req'd in Footing Width	0.7776 in^2	0.7776 in^2

Cantilevered Retaining Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Retaining wall @ GL 7

Code Reference

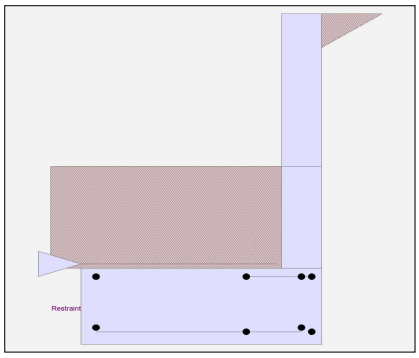
Calculations per IBC 2021, ACI 318-19, TMS 402-16

Criteria

Retained Height	=	5.00 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	24.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	41.0 psf/ft
	=	
Passive Pressure	=	241.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	300.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	1,242.0 lbs
Axial Live Load	=	150.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	300.0 #/ft
...Height to Top	=	5.00 ft
...Height to Bottom	=	4.00 ft
Load Type	=	Seismic (E) (Strength Level)
Wind on Exposed Stem (Strength Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Spread Footing
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.400

Stem Weight Seismic Load

F_p / W_p Weight Multiplier = 0.790 g

Added seismic base force 267.3 lbs

Cantilevered Retaining Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Retaining wall @ GL 7

Design Summary

Wall Stability Ratios

Overturning	=	1.63	OK
Slab Resists All Sliding !			
Global Stability	=	###.##	
Total Bearing Load	=	3,706 lbs	
...resultant ecc.	=	9.33 in	
Eccentricity outside middle third			
Soil Pressure @ Toe	=	2,103 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	2,500 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	2,681 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	15.6 psi	OK
Footing Shear @ Heel	=	3.0 psi	OK
Allowable	=	94.9 psi	

Sliding Calcs

Lateral Sliding Force	=	2,070.2 lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		3rd	2nd	Bottom		
		Stem OK	Stem OK	Stem OK		
Design Height Above Ftg	ft =	5.00	2.00	0.00		
Wall Material Above "Ht"	=	Concrete	Concrete	Concrete		
Design Method	=	SD	SD	SD	SD	SD
Thickness	=	8.00	8.00	8.00		
Rebar Size	=	# 5	# 5	# 5		
Rebar Spacing	=	16.00	16.00	8.00		
Rebar Placed at	=	Center	Center	Center		
Design Data						
fb/FB + fa/Fa	=	**0.000**	**0.547**	**0.771**		
Total Force @ Section						
Service Level	lbs =					
Strength Level	lbs =		1,361.0	2,396.4		
Moment....Actual						
Service Level	ft-# =					
Strength Level	ft-# =		2,193.9	5,907.6		
Moment.....Allowable	ft-# =	4,005.6	4,005.6	7,652.4		
Shear.....Actual						
Service Level	psi =					
Strength Level	psi =		28.4	49.9		
Shear.....Allowable	psi =	64.2	64.2	80.9		
Anet (Masonry)	in2 =					
Wall Weight	psf =	96.7	96.7	96.7		
Rebar Depth 'd'	in =	4.00	4.00	4.00		

Masonry Data

f'm	psi =	
Fs	psi =	
Solid Grouting	=	
Modular Ratio 'n'	=	
Equiv. Solid Thick.	=	
Masonry Block Type	=	
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	4,000.0	4,000.0	4,000.0
Fy	psi =	60,000.0	60,000.0	60,000.0

Cantilevered Retaining Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Retaining wall @ GL 7

Concrete Stem Rebar Area Details

3rd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0 in2/ft		
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 0.00 in	#4@ 0.00 in
Provided Area :	0.2325 in2/ft	#5@ 0.00 in	#5@ 0.00 in
Maximum Area :	0.867 in2/ft	#6@ 0.00 in	#6@ 0.00 in

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.1324 in2/ft		
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 13.89 in	#4@ 27.78 in
Provided Area :	0.2325 in2/ft	#5@ 21.53 in	#5@ 43.06 in
Maximum Area :	0.867 in2/ft	#6@ 30.56 in	#6@ 61.11 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.3566 in2/ft		
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.3566 in2/ft	#4@ 13.89 in	#4@ 27.78 in
Provided Area :	0.465 in2/ft	#5@ 21.53 in	#5@ 43.06 in
Maximum Area :	0.867 in2/ft	#6@ 30.56 in	#6@ 61.11 in

Footing Data

Toe Width	=	3.33 ft
Heel Width	=	0.67
Total Footing Width	=	4.00
Footing Thickness	=	18.00 in

f'c = 4,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	2,681	0	psf
Mu' : Upward	=	10,364	0	ft-#
Mu' : Downward	=	2,961	2	ft-#
Mu: Design	=	7,403	2	ft-#
φ Mn	=	28,317	OK - Flush	
Actual 1-Way Shear	=	15.65	2.99	psi
Allow 1-Way Shear	=	51.55	55.99	psi

Toe Reinforcing = # 6 @ 12.00 in
Heel Reinforcing = Flush heel condition. No reinforcing required.
Key Reinforcing = None Spec'd
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, φ Tn = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: #4@ 6.17 in, #5@ 9.56 in, #6@ 13.58 in, #7@ 18 in, #8@ 18 in, #9@ 18 in, #10@ 18 in

Heel: Flush heel condition. No reinforcing required.

Key: No key defined

Min footing T&S reinf Area 1.56 in2
Min footing T&S reinf Area per foot 0.39 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 6.17 in	#4@ 12.35 in
#5@ 9.57 in	#5@ 19.14 in
#6@ 13.58 in	#6@ 27.16 in

Cantilevered Retaining Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Retaining wall @ GL 7

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		866.1	2.17	1,876.6	Soil Over HL (ab. water tbl)	1.8	4.00	7.3
HL Act Pres (be water tbl)					Soil Over HL (bel. water tbl)		4.00	7.3
Hydrostatic Force					Water Table			
Buoyant Force	=				Sloped Soil Over Heel	=		
Surcharge over Heel	=	726.8	3.25	2,362.2	Surcharge Over Heel	= 1.0	4.00	4.0
Surcharge Over Toe	=				Adjacent Footing Load	=		
Adjacent Footing Load	=				Axial Dead Load on Stem	= 1,242.0	3.66	4,549.9
Added Lateral Load	=	210.0	6.00	1,260.0	* Axial Live Load on Stem	= 150.0	3.66	549.5
Load @ Stem Above Soil	=				Soil Over Toe	= 732.6	1.67	1,219.8
	=				Surcharge Over Toe	=		
Seismic Stem Self Wt		267.3	4.00	1,069.1	Stem Weight(s)	= 483.3	3.66	1,770.6
					Earth @ Stem Transitions	=		
					Footing Weight	= 900.0	2.00	1,800.0
					Key Weight	=	3.58	
					Vert. Component	= 344.9	4.00	1,379.5
Total	=	2,070.2	**O.T.M.** =	6,567.9	**Total =**	3,705.6 lbs	**R.M.=**	10,731.1

Resisting/Overturning Ratio = **1.63**

Vertical Loads used for Soil Pressure = 3,705.6 lbs

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

If seismic is included, the OTM and sliding ratios may be 1.1 per section 1807.2.3 of IBC.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	120.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.152	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

Cantilevered Retaining Wall

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Retaining wall @ GL 7

Rebar Lap & Embedment Lengths Information

Stem Design Segment: 3rd
Stem Design Height: 5.00 ft above top of footing

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) =	18.50 in
Development length for #5 bar specified in this stem design segment =	14.23 in

---

Stem Design Segment: 2nd
Stem Design Height: 2.00 ft above top of footing

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) =	18.50 in
Development length for #5 bar specified in this stem design segment =	14.23 in

---

Stem Design Segment: Bottom
Stem Design Height: 0.00 ft above top of footing

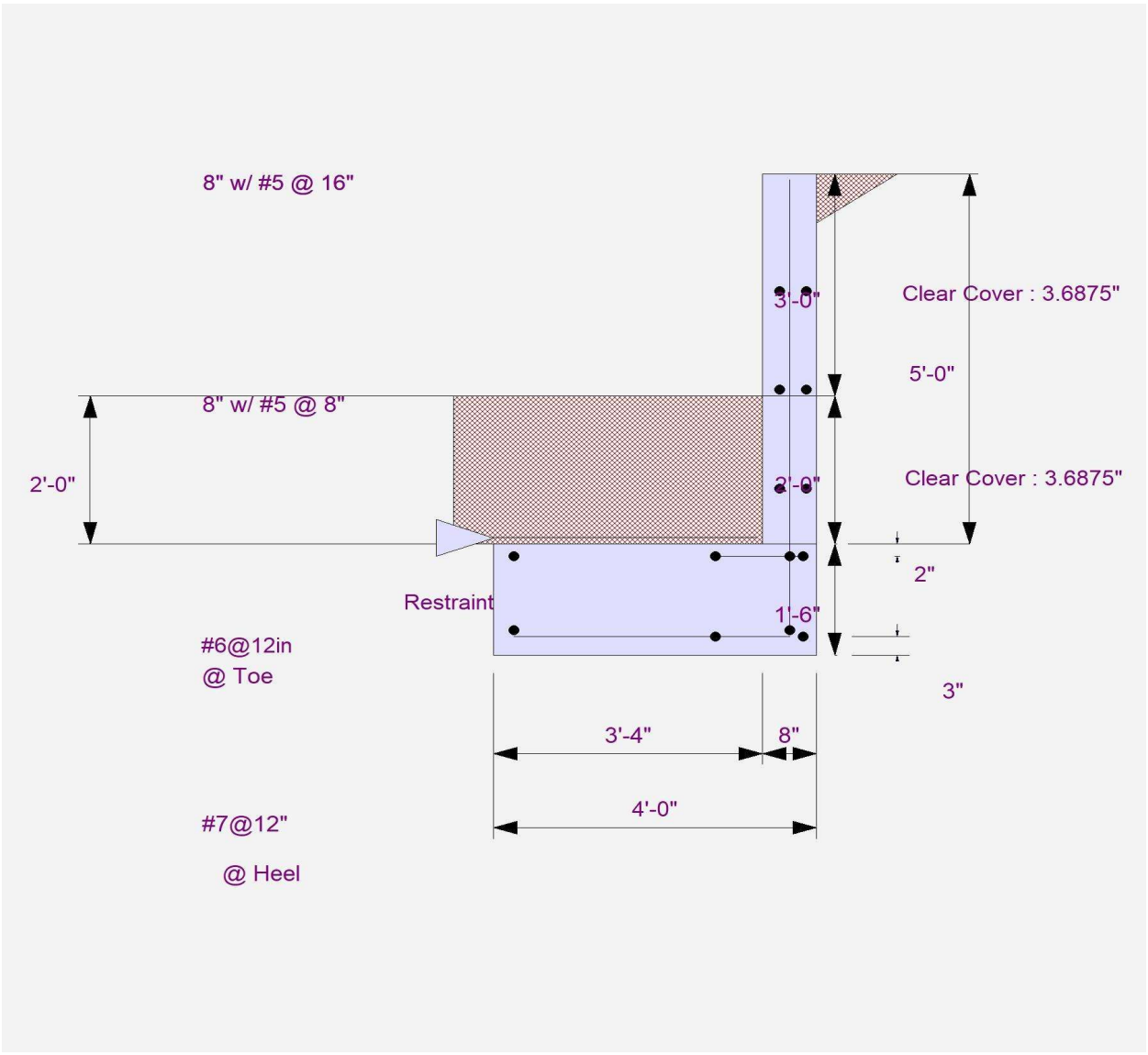
Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) =	18.50 in
Development length for #5 bar specified in this stem design segment =	14.23 in
Hooked embedment length into footing for #5 bar specified in this stem design segment =	7.39 in
As Provided =	0.4650 in2/ft
As Required =	0.3566 in2/ft

Cantilevered Retaining Wall

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

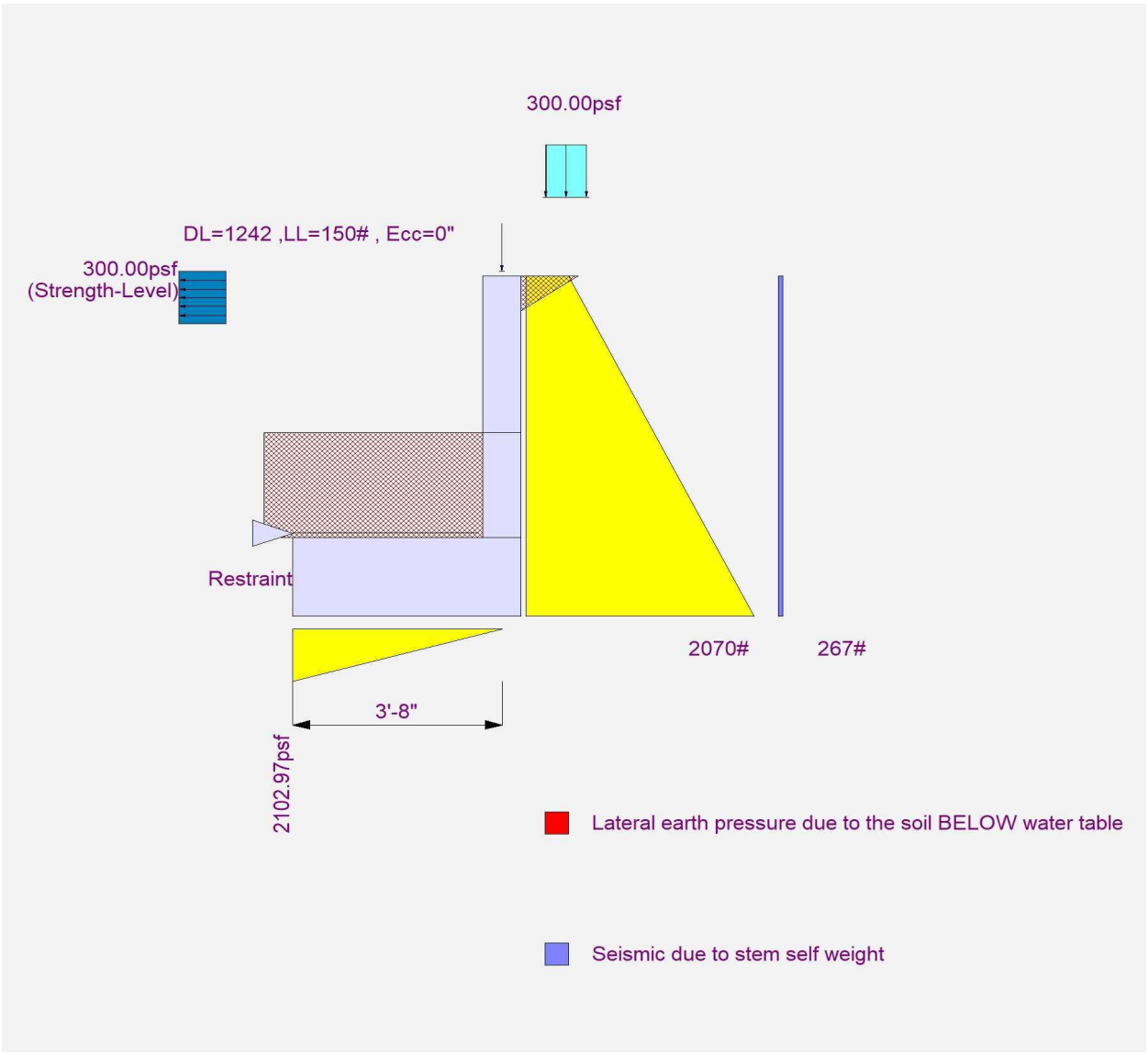
DESCRIPTION: Retaining wall @ GL 7



Cantilevered Retaining Wall

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Retaining wall @ GL 7



IDS GROUP
Structural Engineers
1 Peters Canyon Road, Suite 140

Sheet:	
Job No.:	
Calc. By.:	Date:
Chck'd By.:	Date:

Switchboard Equipment Anchorage

Indoor-Switchboard Section 1		Long Direction		Seismic Only
ap =	2.5	Sds =	1.377	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	825	Weight 2 =	0	
e_1 =	0.0	L =	25.5	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.34 g
fp,min =	0.3*Sds*Ip =		0.62	g
fp,max =	1.6*Sds*Ip =		3.30	g
fph =	511	lbs		
fv =	227	lbs		
Tu1 =	959	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**959**	lbs		
Vu1 =	256	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**256**	lbs		

Indoor-Switchboard Section 1		Short Direction		Seismic Only
ap =	2.5	Sds =	1.377	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	825	Weight 2 =	0	
e_2 =	0.0	W =	22	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.34 g
fp,min =	0.3*Sds*Ip =		0.62	g
fp,max =	1.6*Sds*Ip =		3.30	g
fph =	511	lbs		
fv =	227	lbs		
Tu1 =	1102	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**1102**	lbs		
Vu1 =	256	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**256**	lbs		

Indoor-Switchboard Section 1		Long Direction		Gravity Only
ap =	2.5	Sds =	0	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	825	Weight 2 =	0	
e_1 =	0.0	d =	25.5	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.00 g
fp,min =	0.3*Sds*Ip =		0.00	g
fp,max =	1.6*Sds*Ip =		0.00	g
fph =	0	lbs		
fv =	0	lbs		
Tu1 =	-186	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**-186**	lbs		
Vu1 =	0	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**0**	lbs		

Indoor-Switchboard Section 1		Short Direction		Gravity Only
ap =	2.5	Sds =	0	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	825	Weight 2 =	0	
e_2 =	0.0	d =	22	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.00 g
fp,min =	0.3*Sds*Ip =		0.00	g
fp,max =	1.6*Sds*Ip =		0.00	g
fph =	0	lbs		
fv =	0	lbs		
Tu1 =	-186	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**-186**	lbs		
Vu1 =	0	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**0**	lbs		

Orthogonal combination **Tmax = 1102+ 0.3 x 959 - 186 = 1204 lbs**
Vmax =256 + 256 x 0.3 = 333 lbs

IDS GROUP
Structural Engineers
1 Peters Canyon Road, Suite 140

Sheet:	
Job No.:	
Calc. By.:	Date:
Chck'd By.:	Date:

Switchboard Equipment Anchorage

Indoor-Switchboard Section 2		Long Direction		Seismic Only
ap =	2.5	Sds =	1.377	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	987	Weight 2 =	0	
e_1 =	0.0	L =	35	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =		0.34	g
fp,min =	0.3*Sds*Ip =	0.62	g	
fp,max =	1.6*Sds*Ip =	3.30	g	
fph =	612	lbs		
fv =	272	lbs		
Tu1 =	854	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**854**	lbs		
Vu1 =	306	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**306**	lbs		

Indoor-Switchboard Section 2		Short Direction		Seismic Only
ap =	2.5	Sds =	1.377	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	987	Weight 2 =	0	
e_2 =	0.0	W =	25.5	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =		0.34	g
fp,min =	0.3*Sds*Ip =	0.62	g	
fp,max =	1.6*Sds*Ip =	3.30	g	
fph =	612	lbs		
fv =	272	lbs		
Tu1 =	1147	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**1147**	lbs		
Vu1 =	306	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**306**	lbs		

Indoor-Switchboard Section 2		Long Direction		Gravity Only
ap =	2.5	Sds =	0	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	987	Weight 2 =	0	
e_1 =	0.0	d =	35	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =		0.00	g
fp,min =	0.3*Sds*Ip =	0.00	g	
fp,max =	1.6*Sds*Ip =	0.00	g	
fph =	0	lbs		
fv =	0	lbs		
Tu1 =	-222	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**-222**	lbs		
Vu1 =	0	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**0**	lbs		

Indoor-Switchboard Section 2		Short Direction		Gravity Only
ap =	2.5	Sds =	0	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	987	Weight 2 =	0	
e_2 =	0.0	d =	25.5	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =		0.00	g
fp,min =	0.3*Sds*Ip =	0.00	g	
fp,max =	1.6*Sds*Ip =	0.00	g	
fph =	0	lbs		
fv =	0	lbs		
Tu1 =	-222	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**-222**	lbs		
Vu1 =	0	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**0**	lbs		

Orthogonal combination **Tmax = 1147+ 0.3 x 854 - 222 = 1181 lbs**
Vmax =306 + 306 x 0.3 = 398 lbs

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Switchboard Equipment Anchorage

Outdoor-Switchboard Section 1		Long Direction		Seismic Only
ap =	2.5	Sds =	1.377	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	960	Weight 2 =	0	
e_1 =	0.0	L =	45	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.34 g
fp,min =	0.3*Sds*Ip =		0.62	g
fp,max =	1.6*Sds*Ip =		3.30	g
fph =	595 lbs			
fv =	264 lbs			
Tu1 =	661 lbs	(No. of supports =	2	)
Tu2 =	0 lbs	(No. of supports =	2	)
Tmax =	**661** lbs			
Vu1 =	297 lbs	(No. of supports =	4	)
Vu2 =	0 lbs	(No. of supports =	4	)
Vmax =	**297** lbs			

Outdoor-Switchboard Section 1		Short Direction		Seismic Only
ap =	2.5	Sds =	1.377	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	960	Weight 2 =	0	
e_2 =	0.0	W =	35.5	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.34 g
fp,min =	0.3*Sds*Ip =		0.62	g
fp,max =	1.6*Sds*Ip =		3.30	g
fph =	595 lbs			
fv =	264 lbs			
Tu1 =	820 lbs	(No. of supports =	2	)
Tu2 =	0 lbs	(No. of supports =	2	)
Tmax =	**820** lbs			
Vu1 =	297 lbs	(No. of supports =	4	)
Vu2 =	0 lbs	(No. of supports =	4	)
Vmax =	**297** lbs			

Outdoor-Switchboard Section 1		Long Direction		Gravity Only
ap =	2.5	Sds =	0	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	960	Weight 2 =	0	
e_1 =	0.0	d =	45	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.00 g
fp,min =	0.3*Sds*Ip =		0.00	g
fp,max =	1.6*Sds*Ip =		0.00	g
fph =	0 lbs			
fv =	0 lbs			
Tu1 =	-216 lbs	(No. of supports =	2	)
Tu2 =	0 lbs	(No. of supports =	2	)
Tmax =	**-216** lbs			
Vu1 =	0 lbs	(No. of supports =	4	)
Vu2 =	0 lbs	(No. of supports =	4	)
Vmax =	**0** lbs			

Outdoor-Switchboard Section 1		Short Direction		Gravity Only
ap =	2.5	Sds =	0	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	960	Weight 2 =	0	
e_2 =	0.0	d =	35.5	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.00 g
fp,min =	0.3*Sds*Ip =		0.00	g
fp,max =	1.6*Sds*Ip =		0.00	g
fph =	0 lbs			
fv =	0 lbs			
Tu1 =	-216 lbs	(No. of supports =	2	)
Tu2 =	0 lbs	(No. of supports =	2	)
Tmax =	**-216** lbs			
Vu1 =	0 lbs	(No. of supports =	4	)
Vu2 =	0 lbs	(No. of supports =	4	)
Vmax =	**0** lbs			

Orthogonal combination **Tmax = 820+ 0.3 x 661 - 216 = 802 lbs**

Vmax =297 + 297 x 0.3 = 386 lbs

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Switchboard Equipment Anchorage

Outdoor-Switchboard Section 2		Long Direction		Seismic Only
ap =	2.5	Sds =	1.377	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	1115	Weight 2 =	0	
e_1 =	0.0	L =	35.5	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.34 g
fp,min =	0.3*Sds*Ip =		0.62	g
fp,max =	1.6*Sds*Ip =		3.30	g
fph =	691	lbs		
fv =	307	lbs		
Tu1 =	953	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**953**	lbs		
Vu1 =	345	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**345**	lbs		

Outdoor-Switchboard Section 2		Short Direction		Seismic Only
ap =	2.5	Sds =	1.377	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	1115	Weight 2 =	0	
e_2 =	0.0	W =	35	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.34 g
fp,min =	0.3*Sds*Ip =		0.62	g
fp,max =	1.6*Sds*Ip =		3.30	g
fph =	691	lbs		
fv =	307	lbs		
Tu1 =	965	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**965**	lbs		
Vu1 =	345	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**345**	lbs		

Outdoor-Switchboard Section 2		Long Direction		Gravity Only
ap =	2.5	Sds =	0	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	1115	Weight 2 =	0	
e_1 =	0.0	d =	35.5	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.00 g
fp,min =	0.3*Sds*Ip =		0.00	g
fp,max =	1.6*Sds*Ip =		0.00	g
fph =	0	lbs		
fv =	0	lbs		
Tu1 =	-251	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**-251**	lbs		
Vu1 =	0	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**0**	lbs		

Outdoor-Switchboard Section 2		Short Direction		Gravity Only
ap =	2.5	Sds =	0	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	1115	Weight 2 =	0	
e_2 =	0.0	d =	35	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.00 g
fp,min =	0.3*Sds*Ip =		0.00	g
fp,max =	1.6*Sds*Ip =		0.00	g
fph =	0	lbs		
fv =	0	lbs		
Tu1 =	-251	lbs	(No. of supports =	2)
Tu2 =	0	lbs	(No. of supports =	2)
Tmax =	**-251**	lbs		
Vu1 =	0	lbs	(No. of supports =	4)
Vu2 =	0	lbs	(No. of supports =	4)
Vmax =	**0**	lbs		

Orthogonal combination **Tmax = 965+ 0.3 x 953 - 251 = 1000 lbs**

Vmax =345 + 345 x 0.3 = 449 lbs

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Switchboard Equipment Anchorage

Outdoor-Switchboard Section 3		Long Direction		Seismic Only
ap =	2.5	Sds =	1.377	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	875	Weight 2 =	0	
e_1 =	0.0	L =	35.5	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.34 g
fp,min =	0.3*Sds*Ip =	0.62	g	
fp,max =	1.6*Sds*Ip =	3.30	g	
fph =	542 lbs			
fv =	241 lbs			
Tu1 =	748 lbs	(No. of supports =	2)	
Tu2 =	0 lbs	(No. of supports =	2)	
Tmax =	**748** lbs			
Vu1 =	271 lbs	(No. of supports =	4)	
Vu2 =	0 lbs	(No. of supports =	4)	
Vmax =	**271** lbs			

Outdoor-Switchboard Section 3		Short Direction		Seismic Only
ap =	2.5	Sds =	1.377	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	875	Weight 2 =	0	
e_2 =	0.0	W =	29	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.34 g
fp,min =	0.3*Sds*Ip =	0.62	g	
fp,max =	1.6*Sds*Ip =	3.30	g	
fph =	542 lbs			
fv =	241 lbs			
Tu1 =	902 lbs	(No. of supports =	2)	
Tu2 =	0 lbs	(No. of supports =	2)	
Tmax =	**902** lbs			
Vu1 =	271 lbs	(No. of supports =	4)	
Vu2 =	0 lbs	(No. of supports =	4)	
Vmax =	**271** lbs			

Outdoor-Switchboard Section 3		Long Direction		Gravity Only
ap =	2.5	Sds =	0	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	875	Weight 2 =	0	
e_1 =	0.0	d =	35.5	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.00 g
fp,min =	0.3*Sds*Ip =	0.00	g	
fp,max =	1.6*Sds*Ip =	0.00	g	
fph =	0 lbs			
fv =	0 lbs			
Tu1 =	-197 lbs	(No. of supports =	2)	
Tu2 =	0 lbs	(No. of supports =	2)	
Tmax =	**-197** lbs			
Vu1 =	0 lbs	(No. of supports =	4)	
Vu2 =	0 lbs	(No. of supports =	4)	
Vmax =	**0** lbs			

Outdoor-Switchboard Section 3		Short Direction		Gravity Only
ap =	2.5	Sds =	0	g
Rp =	6	Ωo =	2	(concrete anchor)
Ip =	1.5			
z/h =	0			
Weight =	875	Weight 2 =	0	
e_2 =	0.0	d =	29	
Hcg =	45.0			
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =			0.00 g
fp,min =	0.3*Sds*Ip =	0.00	g	
fp,max =	1.6*Sds*Ip =	0.00	g	
fph =	0 lbs			
fv =	0 lbs			
Tu1 =	-197 lbs	(No. of supports =	2)	
Tu2 =	0 lbs	(No. of supports =	2)	
Tmax =	**-197** lbs			
Vu1 =	0 lbs	(No. of supports =	4)	
Vu2 =	0 lbs	(No. of supports =	4)	
Vmax =	**0** lbs			

Orthogonal combination

Tmax = 902+ 0.3 x 748 - 197 = 929 lbs

Vmax =271 + 271 x 0.3 = 352 lbs

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Corona South Mall - BLDG B- Switchboard	Date:	5/27/2026
Fastening point:			

Specifier's comments:

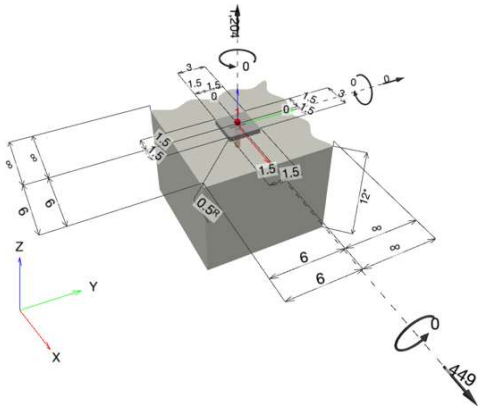
1 Input data



Anchor type and diameter:	**Kwik Bolt TZ2 - SS 316 1/2 (2) hnom1**
Item number:	2210264 KB-TZ2 1/2x3 3/4 SS316
Specification text:	HILTI ∅ 1/2 IN KWIK BOLT TZ2 - SS 316 WITH 2.5 IN NOMINAL EMBEDMENT DEPTH PER ICC-ES ESR-4266 , HAMMER DRILL BIT INSTALLATION PER MPII
Effective embedment depth:	$h_{ef,act}$ = 2.000 in., h_{nom} = 2.500 in.
Material:	AISI 316
Evaluation Service Report:	ESR-4266
Issued I Valid:	12/1/2025 | 12/1/2027
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R]:	l_x x l_y x t = 3.000 in. x 3.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 12.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d)) Shear load: yes (17.10.6.3 (c))

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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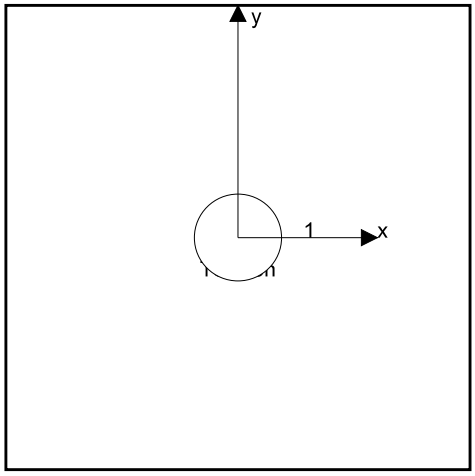
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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 1,204; V_x = 449; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	yes	82

2 Load case/Resulting anchor forces



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,204	449	449	0

Max. concrete compressive strain: - [‰]
Max. concrete compressive stress: - [psi]
Resulting tension force in (x/y)=(-0.000/0.000): 1,204 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	1,204	8,906	14	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	1,204	1,483	82	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} [lb]	ϕ	ϕ N_{sa} [lb]	N_{ua} [lb]
11,875	0.750	8,906	1,204

3.2 Concrete Breakout Failure

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$c_{a,min}$ [in.]	c_{ac} [in.]	$\Psi_{c,N}$
36.00	36.00	6.000	5.500	1.000

$e_{c1,N}$ [in.]	$\Psi_{ec1,N}$	$e_{c2,N}$ [in.]	$\Psi_{ec2,N}$	$\Psi_{ed,N}$	$\Psi_{cp,N}$	k_{cr}
0.000	1.000	0.000	1.000	1.000	1.000	17

λ_a	N_b [lb]	ϕ	$\phi_{seismic}$	ϕ N_{cbg} [lb]	N_{ua} [lb]
1.000	3,041	0.650	0.750	1,483	1,204

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	449	5,426	9	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	449	2,129	22	OK
Concrete edge failure in direction x+**	449	3,187	15	OK

* highest loaded anchor **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

4.1 Steel Strength

V_{sa} [lb]	$\alpha_{V,seis}$	ϕ	ϕ V_{sa} [lb]	V_{ua} [lb]
8,348	1.000	0.650	5,426	449

4.2 Pryout Strength

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$c_{a,min}$ [in.]	k_{cp}	c_{ac} [in.]	$\Psi_{c,N}$
36.00	36.00	6.000	1	5.500	1.000

$e_{c1,V}$ [in.]	$\Psi_{ec1,V}$	$e_{c2,V}$ [in.]	$\Psi_{ec2,V}$	$\Psi_{ed,N}$	k_{cr}
0.000	1.000	0.000	1.000	1.000	17

λ_a	N_b [lb]	ϕ	$\phi_{seismic}$	ϕV_{cpg} [lb]	V_{ua} [lb]
1.000	3,041	0.700	1.000	2,129	449

4.3 Concrete edge failure in direction x+

l_e [in.]	d_a [in.]	c_{a1} [in.]	A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]
2.000	0.500	6.000	135.00	162.00

$\Psi_{ed,V}$	$\Psi_{parallel,V}$	$e_{c,V}$ [in.]	$\Psi_{ec,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$
0.900	1.000	0.000	1.000	1.000	1.000

λ_a	V_b [lb]	ϕ	$\phi_{seismic}$	ϕV_{cbg} [lb]	V_{ua} [lb]
1.000	6,071	0.700	1.000	3,187	449

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.812	0.211	5/3	79	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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Hilti PROFIS Engineering 3.1.31

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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Corona South Mall - BLDG B- Switchboard	Date:	5/27/2026
Fastening point:			

7 Installation data

Anchor type and diameter: Kwik Bolt TZ2 - SS 316 1/2 (2)
hnom1

Profile: no profile

Item number: 2210264 KB-TZ2 1/2x3 3/4 SS316

Hole diameter in the fixture: d_f = 0.562 in.

Maximum installation torque: 481 in.lb

Plate thickness (input): 0.500 in.

Hole diameter in the base material: 0.500 in.

Recommended plate thickness: not calculated

Hole depth in the base material: 2.750 in.

Drilling method: Hammer drilled

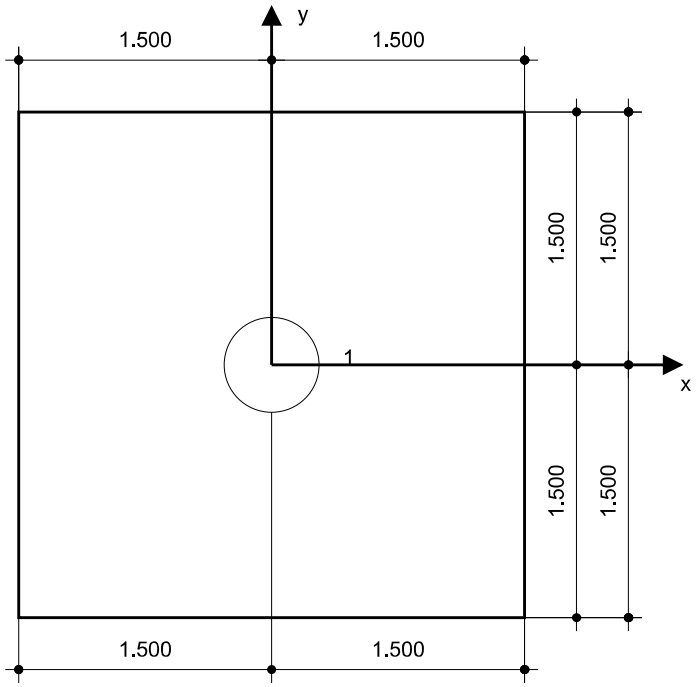
Minimum thickness of the base material: 4.000 in.

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti ∅ 1/2 in Kwik Bolt TZ2 - SS 316 with 2.5 in nominal embedment depth per ICC-ES ESR-4266 , Hammer drill bit installation per MPII

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hammer



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	-	6.000	6.000	-

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Corona South Mall - BLDG B- Switchboard	Date:	5/27/2026
Fastening point:			

8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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General Footing

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: --Indoor switchgear footing--

Code References

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

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,155.92	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
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

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

Increases based on footing Depth

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

Increases based on footing plan dimension

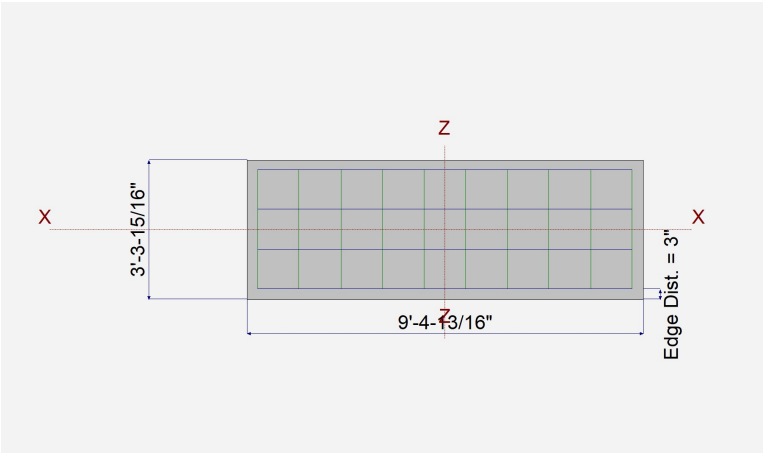
Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

Width parallel to X-X Axis	=	9.40 ft
Length parallel to Z-Z Axis	=	3.330 ft
Footing Thickness	=	12.0 in

Pedestal dimensions...

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



Bandwidth Distribution Check (ACI 15.4.4.2)

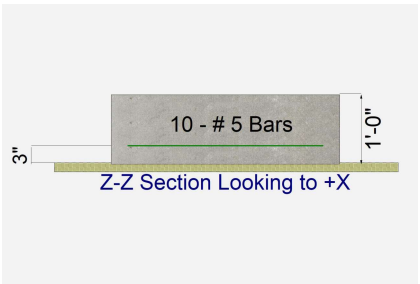
Direction Requiring Closer Separation Bars along Z-Z Axis

# Bars required within zone	52.3 %
# Bars required on each side of zone	47.7 %

Bottom Reinforcing

Bars parallel to X-X Axis			
Number of Bars	=		4.0
Reinforcing Bar Size	=	#	5
Bars parallel to Z-Z Axis			
Number of Bars	=		10.0
Reinforcing Bar Size	=	#	5
Rebar Centerline to Edge of Concrete...			
at Bottom of footing	=		3.0 in





Top Reinforcing

Bars parallel to X-X Axis		
Number of Bars	=	4.0
Reinforcing Bar Size	=	3
Bars parallel to Z-Z Axis		
Number of Bars	=	3.0
Reinforcing Bar Size	=	4
Rebar Centerline to Edge of Concrete...		
at Top of footing	=	3.0 in

General Footing

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: --Indoor switchgear footing--

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	2.799							k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=						6.50		k-ft
V-x	=						1.735		k
V-z	=								k

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.2339	Soil Bearing	0.3508 ksf	1.50 ksf	+D+0.70E about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	3.590	Overturning - Z-Z	5.765 k-ft	20.693 k-ft	+0.60D+0.70E
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.1656	Z Flexure (+X) Bot Tens	2.422 k-ft/ft	14.622 k-ft/ft	+1.20D+E
PASS	0.09456	Z Flexure (-X) Bot Tens	1.383 k-ft/ft	14.622 k-ft/ft	+1.40D
PASS	0.01335	X Flexure (+Z) Bot Tens	0.1735 k-ft/ft	12.997 k-ft/ft	+1.40D
PASS	0.01335	X Flexure (-Z) Bot Tens	0.1735 k-ft/ft	12.997 k-ft/ft	+1.40D
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.06697	Z Flexure (-X) Top Tens	0.3475 k-ft/ft	5.189 k-ft/ft	+0.90D+E
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (-Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.1305	1-way Shear (+X)	7.483 psi	57.328 psi	+1.20D+E
PASS	0.07983	1-way Shear (-X)	4.576 psi	57.328 psi	+1.40D
PASS	0.01893	1-way Shear (+Z)	1.042 psi	55.053 psi	+1.40D
PASS	0.01893	1-way Shear (-Z)	1.042 psi	55.053 psi	+1.40D
PASS	0.06262	2-way Punching	11.882 psi	189.737 psi	+1.40D

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	0.2344	0.2344	n/a	n/a	0.156
X-X, +0.60D	1.50	n/a	0.0	0.1407	0.1407	n/a	n/a	0.094
X-X, +D+0.70E	1.50	n/a	0.0	0.2344	0.2344	n/a	n/a	0.156
X-X, +D+0.5250E	1.50	n/a	0.0	0.2344	0.2344	n/a	n/a	0.156
X-X, +0.60D+0.70E	1.50	n/a	0.0	0.1407	0.1407	n/a	n/a	0.094
Z-Z, D Only	1.50	0.0	n/a	n/a	n/a	0.2344	0.2344	0.156
Z-Z, +0.60D	1.50	0.0	n/a	n/a	n/a	0.1407	0.1407	0.094
Z-Z, +D+0.70E	1.50	9.427	n/a	n/a	n/a	0.1180	0.3508	0.234
Z-Z, +D+0.5250E	1.50	7.070	n/a	n/a	n/a	0.1471	0.3217	0.215
Z-Z, +0.60D+0.70E	1.50	15.712	n/a	n/a	n/a	0.02428	0.2570	0.171

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D Only	None	0.0 k-ft	Infinity	OK
X-X, +0.60D	None	0.0 k-ft	Infinity	OK
X-X, +D+0.70E	None	0.0 k-ft	Infinity	OK
X-X, +D+0.5250E	None	0.0 k-ft	Infinity	OK
X-X, +0.60D+0.70E	None	0.0 k-ft	Infinity	OK
Z-Z, D Only	None	0.0 k-ft	Infinity	OK
Z-Z, +0.60D	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.70E	5.765 k-ft	34.488 k-ft	5.983	OK
Z-Z, +D+0.5250E	4.323 k-ft	34.488 k-ft	7.977	OK
Z-Z, +0.60D+0.70E	5.765 k-ft	20.693 k-ft	3.590	OK

General Footing

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: --Indoor switchgear footing--

Footing Tension on Bottom

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	0.1735	+Z	Bottom	0.2592	ACI 7.6.1.1	0.3298	12.997	OK
X-X, +1.40D	0.1735	-Z	Bottom	0.2592	ACI 7.6.1.1	0.3298	12.997	OK
X-X, +1.20D	0.1487	+Z	Bottom	0.2592	ACI 7.6.1.1	0.3298	12.997	OK
X-X, +1.20D	0.1487	-Z	Bottom	0.2592	ACI 7.6.1.1	0.3298	12.997	OK
X-X, +0.90D	0.1116	+Z	Bottom	0.2592	ACI 7.6.1.1	0.3298	12.997	OK
X-X, +0.90D	0.1116	-Z	Bottom	0.2592	ACI 7.6.1.1	0.3298	12.997	OK
X-X, +1.20D+E	0.1487	+Z	Bottom	0.2592	ACI 7.6.1.1	0.3298	12.997	OK
X-X, +1.20D+E	0.1487	-Z	Bottom	0.2592	ACI 7.6.1.1	0.3298	12.997	OK
X-X, +0.90D+E	0.1116	+Z	Bottom	0.2592	ACI 7.6.1.1	0.3298	12.997	OK
X-X, +0.90D+E	0.1116	-Z	Bottom	0.2592	ACI 7.6.1.1	0.3298	12.997	OK
Z-Z, +1.40D	1.383	-X	Bottom	0.2592	ACI 7.6.1.1	0.3724	14.622	OK
Z-Z, +1.40D	1.383	+X	Bottom	0.2592	ACI 7.6.1.1	0.3724	14.622	OK
Z-Z, +1.20D	1.185	-X	Bottom	0.2592	ACI 7.6.1.1	0.3724	14.622	OK
Z-Z, +1.20D	1.185	+X	Bottom	0.2592	ACI 7.6.1.1	0.3724	14.622	OK
Z-Z, +0.90D	0.8889	-X	Bottom	0.2592	ACI 7.6.1.1	0.3724	14.622	OK
Z-Z, +0.90D	0.8889	+X	Bottom	0.2592	ACI 7.6.1.1	0.3724	14.622	OK
Z-Z, +1.20D+E	2.422	+X	Bottom	0.2592	ACI 7.6.1.1	0.3724	14.622	OK
Z-Z, +0.90D+E	2.125	+X	Bottom	0.2592	ACI 7.6.1.1	0.3724	14.622	OK

Footing Tension on Top

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+E	0.05120	-X	Top	0.2592	ACI 7.6.1.1	0.1321	5.189	OK
Z-Z, +0.90D+E	0.3475	-X	Top	0.2592	ACI 7.6.1.1	0.1321	5.189	OK

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	4.58 psi	4.58 psi	4.58 psi	57.33 psi	0.08	OK
+1.20D	3.92 psi	3.92 psi	3.92 psi	57.33 psi	0.07	OK
+0.90D	2.94 psi	2.94 psi	2.94 psi	57.33 psi	0.05	OK
+1.20D+E	0.36 psi	7.48 psi	7.48 psi	57.33 psi	0.13	OK
+0.90D+E	0.62 psi	6.50 psi	6.50 psi	57.33 psi	0.11	OK

One Way Shear Z

Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.04 psi	1.04 psi	1.04 psi	55.05 psi	0.02	OK
+1.20D	0.89 psi	0.89 psi	0.89 psi	55.05 psi	0.02	OK
+0.90D	0.67 psi	0.67 psi	0.67 psi	55.05 psi	0.01	OK
+1.20D+E	0.89 psi	0.89 psi	0.89 psi	55.05 psi	0.02	OK
+0.90D+E	0.67 psi	0.67 psi	0.67 psi	55.05 psi	0.01	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	11.88 psi	189.74 psi	0.06	OK
+1.20D	10.18 psi	189.74 psi	0.05	OK
+0.90D	7.64 psi	189.74 psi	0.04	OK
+1.20D+E	10.18 psi	189.74 psi	0.05	OK
+0.90D+E	7.64 psi	189.74 psi	0.04	OK

General Footing

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: --Outdoor switchgear footing--

Code References

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

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,155.92	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
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

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

Increases based on footing Depth

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

Increases based on footing plan dimension

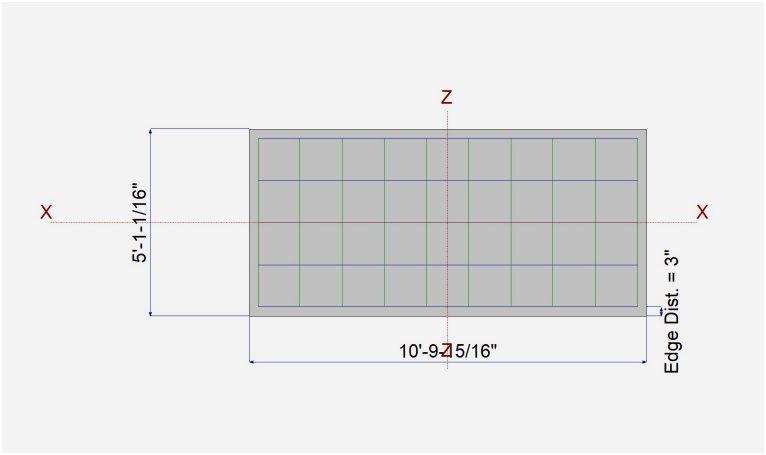
Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

Width parallel to X-X Axis	=	10.830	ft
Length parallel to Z-Z Axis	=	5.090	ft
Footing Thickness	=	12.0	in

Pedestal dimensions...

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



Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation Bars along Z-Z Axis
Bars required within zone 63.9 %
Bars required on each side of zone 36.1 %

Bottom Reinforcing

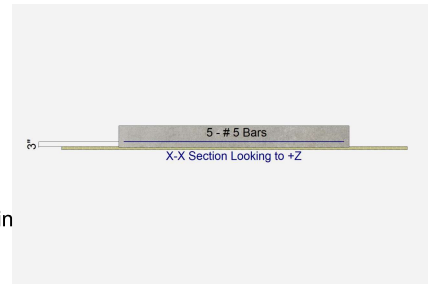
Bars parallel to X-X Axis

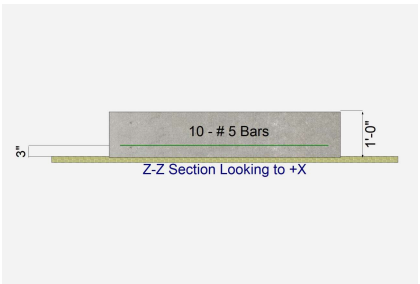
Number of Bars	=		5.0
Reinforcing Bar Size	=	#	5

Bars parallel to Z-Z Axis

Number of Bars	=		10.0
Reinforcing Bar Size	=	#	5

Rebar Centerline to Edge of Concrete...
at Bottom of footing = 3.0 in





Top Reinforcing

Bars parallel to X-X Axis

Number of Bars	=	4.0
Reinforcing Bar Size	=	3

Bars parallel to Z-Z Axis

Number of Bars	=	3.0
Reinforcing Bar Size	=	4

Rebar Centerline to Edge of Concrete...
at Top of footing = 3.0 in

General Footing

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: --Outdoor switchgear footing--

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	2.950							k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=						5.579		k-ft
V-x	=						1.829		k
V-z	=								k

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.1667	Soil Bearing	0.2501 ksf	1.50 ksf	+D+0.70E about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	6.856	Overturning - Z-Z	5.186 k-ft	35.554 k-ft	+0.60D+0.70E
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.1388	Z Flexure (+X) Bot Tens	1.669 k-ft/ft	12.026 k-ft/ft	+1.20D+E
PASS	0.09134	Z Flexure (-X) Bot Tens	1.098 k-ft/ft	12.026 k-ft/ft	+1.40D
PASS	0.02143	X Flexure (+Z) Bot Tens	0.2426 k-ft/ft	11.322 k-ft/ft	+1.40D
PASS	0.02143	X Flexure (-Z) Bot Tens	0.2426 k-ft/ft	11.322 k-ft/ft	+1.40D
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.006297	Z Flexure (-X) Top Tens	0.02150 k-ft/ft	3.414 k-ft/ft	+0.90D+E
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (-Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.08579	1-way Shear (+X)	4.599 psi	53.610 psi	+1.20D+E
PASS	0.06026	1-way Shear (-X)	3.231 psi	53.610 psi	+1.40D
PASS	0.02353	1-way Shear (+Z)	1.236 psi	52.515 psi	+1.40D
PASS	0.02353	1-way Shear (-Z)	1.236 psi	52.515 psi	+1.40D
PASS	0.06662	2-way Punching	12.640 psi	189.737 psi	+1.40D

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	0.1985	0.1985	n/a	n/a	0.132
X-X, +0.60D	1.50	n/a	0.0	0.1191	0.1191	n/a	n/a	0.079
X-X, +D+0.70E	1.50	n/a	0.0	0.1985	0.1985	n/a	n/a	0.132
X-X, +D+0.5250E	1.50	n/a	0.0	0.1985	0.1985	n/a	n/a	0.132
X-X, +0.60D+0.70E	1.50	n/a	0.0	0.1191	0.1191	n/a	n/a	0.079
Z-Z, D Only	1.50	0.0	n/a	n/a	n/a	0.1985	0.1985	0.132
Z-Z, +0.60D	1.50	0.0	n/a	n/a	n/a	0.1191	0.1191	0.079
Z-Z, +D+0.70E	1.50	5.686	n/a	n/a	n/a	0.1469	0.2501	0.167
Z-Z, +D+0.5250E	1.50	4.265	n/a	n/a	n/a	0.1598	0.2372	0.158
Z-Z, +0.60D+0.70E	1.50	9.477	n/a	n/a	n/a	0.06751	0.1707	0.114

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D Only	None	0.0 k-ft	Infinity	OK
X-X, +0.60D	None	0.0 k-ft	Infinity	OK
X-X, +D+0.70E	None	0.0 k-ft	Infinity	OK
X-X, +D+0.5250E	None	0.0 k-ft	Infinity	OK
X-X, +0.60D+0.70E	None	0.0 k-ft	Infinity	OK
Z-Z, D Only	None	0.0 k-ft	Infinity	OK
Z-Z, +0.60D	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.70E	5.186 k-ft	59.257 k-ft	11.427	OK
Z-Z, +D+0.5250E	3.889 k-ft	59.257 k-ft	15.236	OK
Z-Z, +0.60D+0.70E	5.186 k-ft	35.554 k-ft	6.856	OK

General Footing

Project File: Building B.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: --Outdoor switchgear footing--

Footing Tension on Bottom

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	0.2426	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2862	11.322	OK
X-X, +1.40D	0.2426	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2862	11.322	OK
X-X, +1.20D	0.2080	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2862	11.322	OK
X-X, +1.20D	0.2080	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2862	11.322	OK
X-X, +0.90D	0.1560	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2862	11.322	OK
X-X, +0.90D	0.1560	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2862	11.322	OK
X-X, +1.20D+E	0.2080	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2862	11.322	OK
X-X, +1.20D+E	0.2080	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2862	11.322	OK
X-X, +0.90D+E	0.1560	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2862	11.322	OK
X-X, +0.90D+E	0.1560	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2862	11.322	OK
Z-Z, +1.40D	1.098	-X	Bottom	0.2592	ACI 7.6.1.1	0.3045	12.026	OK
Z-Z, +1.40D	1.098	+X	Bottom	0.2592	ACI 7.6.1.1	0.3045	12.026	OK
Z-Z, +1.20D	0.9415	-X	Bottom	0.2592	ACI 7.6.1.1	0.3045	12.026	OK
Z-Z, +1.20D	0.9415	+X	Bottom	0.2592	ACI 7.6.1.1	0.3045	12.026	OK
Z-Z, +0.90D	0.7061	-X	Bottom	0.2592	ACI 7.6.1.1	0.3045	12.026	OK
Z-Z, +0.90D	0.7061	+X	Bottom	0.2592	ACI 7.6.1.1	0.3045	12.026	OK
Z-Z, +1.20D+E	0.2139	-X	Bottom	0.2592	ACI 7.6.1.1	0.3045	12.026	OK
Z-Z, +1.20D+E	1.669	+X	Bottom	0.2592	ACI 7.6.1.1	0.3045	12.026	OK
Z-Z, +0.90D+E	1.434	+X	Bottom	0.2592	ACI 7.6.1.1	0.3045	12.026	OK

Footing Tension on Top

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, +0.90D+E	0.02150	-X	Top	0.2592	ACI 7.6.1.1	0.08644	3.414	OK

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	3.23 psi	3.23 psi	3.23 psi	53.61 psi	0.06	OK
+1.20D	2.77 psi	2.77 psi	2.77 psi	53.61 psi	0.05	OK
+0.90D	2.08 psi	2.08 psi	2.08 psi	53.61 psi	0.04	OK
+1.20D+E	0.94 psi	4.60 psi	4.60 psi	53.61 psi	0.09	OK
+0.90D+E	0.25 psi	3.91 psi	3.91 psi	53.61 psi	0.07	OK

One Way Shear Z

Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.24 psi	1.24 psi	1.24 psi	52.52 psi	0.02	OK
+1.20D	1.06 psi	1.06 psi	1.06 psi	52.52 psi	0.02	OK
+0.90D	0.79 psi	0.79 psi	0.79 psi	52.52 psi	0.02	OK
+1.20D+E	1.06 psi	1.06 psi	1.06 psi	52.52 psi	0.02	OK
+0.90D+E	0.79 psi	0.79 psi	0.79 psi	52.52 psi	0.02	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	12.64 psi	189.74 psi	0.07	OK
+1.20D	10.83 psi	189.74 psi	0.06	OK
+0.90D	8.13 psi	189.74 psi	0.04	OK
+1.20D+E	10.83 psi	189.74 psi	0.06	OK
+0.90D+E	8.13 psi	189.74 psi	0.04	OK

IDS GROUP
Structural Engineers
1 Peters Canyon Road, Suite 140

Sheet:	
Job No.:	
Calc. By.:	Date:
Chck'd By.:	Date:

Inverter Equipment Anchorage

Inverter		Long Direction	Seismic Only
ap =	2.5	Sds =	1.377 g
Rp =	6	Ωo =	2 (concrete anchor)
Ip =	1.5		
z/h =	0		
Weight =	900	Weight 2 =	0
e_1 =	0.0	L =	24
Hcg =	23.5		
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =		0.34 g
fp,min =	0.3*Sds*Ip =	0.62	g
fp,max =	1.6*Sds*Ip =	3.30	g
fph =	558 lbs		
fv =	248 lbs		
Tu1 =	608 lbs	(No. of supports =	2)
Tu2 =	0 lbs	(No. of supports =	2)
Tmax =	**608** lbs		
Vu1 =	279 lbs	(No. of supports =	4)
Vu2 =	0 lbs	(No. of supports =	4)
Vmax =	**279** lbs		

Inverter		Short Direction	Seismic Only
ap =	2.5	Sds =	1.377 g
Rp =	6	Ωo =	2 (concrete anchor)
Ip =	1.5		
z/h =	0		
Weight =	900	Weight 2 =	0
e_2 =	0.0	W =	19
Hcg =	23.5		
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =		0.34 g
fp,min =	0.3*Sds*Ip =	0.62	g
fp,max =	1.6*Sds*Ip =	3.30	g
fph =	558 lbs		
fv =	248 lbs		
Tu1 =	752 lbs	(No. of supports =	2)
Tu2 =	0 lbs	(No. of supports =	2)
Tmax =	**752** lbs		
Vu1 =	279 lbs	(No. of supports =	4)
Vu2 =	0 lbs	(No. of supports =	4)
Vmax =	**279** lbs		

Inverter		Long Direction	Gravity Only
ap =	2.5	Sds =	0 g
Rp =	6	Ωo =	2 (concrete anchor)
Ip =	1.5		
z/h =	0		
Weight =	900	Weight 2 =	0
e_1 =	0.0	d =	24
Hcg =	23.5		
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =		0.00 g
fp,min =	0.3*Sds*Ip =	0.00	g
fp,max =	1.6*Sds*Ip =	0.00	g
fph =	0 lbs		
fv =	0 lbs		
Tu1 =	-203 lbs	(No. of supports =	2)
Tu2 =	0 lbs	(No. of supports =	2)
Tmax =	**-203** lbs		
Vu1 =	0 lbs	(No. of supports =	4)
Vu2 =	0 lbs	(No. of supports =	4)
Vmax =	**0** lbs		

Inverter		Short Direction	Gravity Only
ap =	2.5	Sds =	0 g
Rp =	6	Ωo =	2 (concrete anchor)
Ip =	1.5		
z/h =	0		
Weight =	900	Weight 2 =	0
e_2 =	0.0	d =	19
Hcg =	23.5		
fp =	0.4*ap*Sds*(1+2*z/h)/(Rp/Ip) =		0.00 g
fp,min =	0.3*Sds*Ip =	0.00	g
fp,max =	1.6*Sds*Ip =	0.00	g
fph =	0 lbs		
fv =	0 lbs		
Tu1 =	-203 lbs	(No. of supports =	2)
Tu2 =	0 lbs	(No. of supports =	2)
Tmax =	**-203** lbs		
Vu1 =	0 lbs	(No. of supports =	4)
Vu2 =	0 lbs	(No. of supports =	4)
Vmax =	**0** lbs		

Orthogonal combination **Tmax =752+ 0.3 x 608 - 203 = 731 lbs**

Vmax =279 + 279 x 0.3 = 363 lbs

Hilti PROFIS Engineering 3.1.24

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Corona South Mall - BLDG B- Inverter	Date:	12/9/2025
Fastening point:			

Specifier's comments:

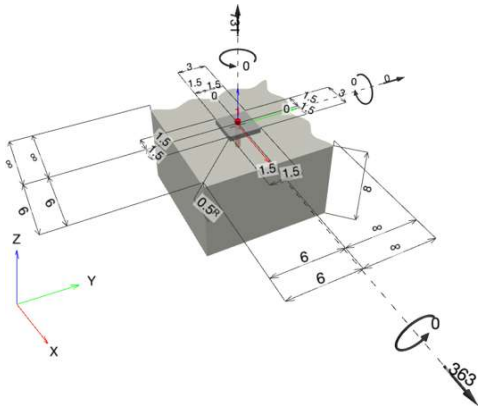
1 Input data



Anchor type and diameter:	**Kwik Bolt TZ2 - CS 1/2 (2) hnom2**
Item number:	2210254 KB-TZ2 1/2x3 3/4
Specification text:	Hilti ∅ 1/2 in Kwik Bolt TZ2 - CS with 2.5 in nominal embedment depth per ICC-ES ESR-4266 , Hammer drill bit installation per MPII
Effective embedment depth:	$h_{ef,act}$ = 2.000 in., h_{nom} = 2.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-4266
Issued I Valid:	10/1/2024 | 12/1/2025
Proof:	Design Method ACI 318-19 / Mech
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 3.000 in. x 3.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 8.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d)) Shear load: yes (17.10.6.3 (c))

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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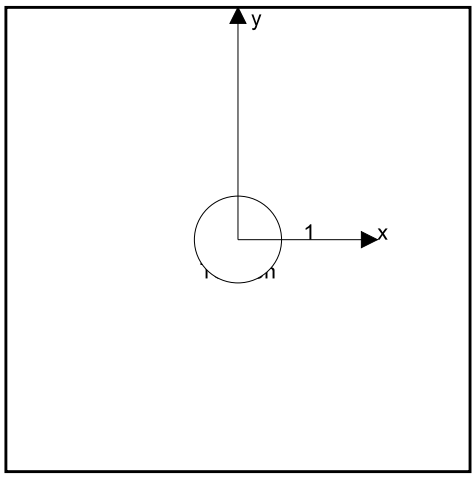
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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 731; V_x = 363; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	yes	40

2 Load case/Resulting anchor forces



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	731	363	363	0

Max. concrete compressive strain: - [‰]
Max. concrete compressive stress: - [psi]
Resulting tension force in (x/y)=(-0.000/0.000): 731 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	731	8,433	9	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	731	1,831	40	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} [lb]	ϕ	ϕ N_{sa} [lb]	N_{ua} [lb]
11,244	0.750	8,433	731

3.2 Concrete Breakout Failure

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$c_{a,min}$ [in.]	c_{ac} [in.]	$\Psi_{c,N}$
36.00	36.00	6.000	5.500	1.000

$e_{c1,N}$ [in.]	$\Psi_{ec1,N}$	$e_{c2,N}$ [in.]	$\Psi_{ec2,N}$	$\Psi_{ed,N}$	$\Psi_{cp,N}$	k_{cr}
0.000	1.000	0.000	1.000	1.000	1.000	21

λ_a	N_b [lb]	ϕ	$\phi_{seismic}$	ϕ N_{cbg} [lb]	N_{ua} [lb]
1.000	3,757	0.650	0.750	1,831	731

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	363	3,599	11	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	363	2,630	14	OK
Concrete edge failure in direction x+**	363	3,005	13	OK

* highest loaded anchor **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

4.1 Steel Strength

V_{sa} [lb]	$\alpha_{V,seis}$	ϕ	ϕ V_{sa} [lb]	V_{ua} [lb]
5,537	1.000	0.650	3,599	363

4.2 Pryout Strength

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$c_{a,min}$ [in.]	k_{cp}	c_{ac} [in.]	$\Psi_{c,N}$
36.00	36.00	6.000	1	5.500	1.000

$e_{c1,V}$ [in.]	$\Psi_{ec1,V}$	$e_{c2,V}$ [in.]	$\Psi_{ec2,V}$	$\Psi_{ed,N}$	k_{cr}
0.000	1.000	0.000	1.000	1.000	21

λ_a	N_b [lb]	ϕ	$\phi_{seismic}$	ϕV_{cpg} [lb]	V_{ua} [lb]
1.000	3,757	0.700	1.000	2,630	363

4.3 Concrete edge failure in direction x+

l_e [in.]	d_a [in.]	c_{a1} [in.]	A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]
2.000	0.500	6.000	120.00	162.00

$\Psi_{ed,V}$	$\Psi_{parallel,V}$	$e_{c,V}$ [in.]	$\Psi_{ec,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$
0.900	1.000	0.000	1.000	1.000	1.061

λ_a	V_b [lb]	ϕ	$\phi_{seismic}$	ϕV_{cbg} [lb]	V_{ua} [lb]
1.000	6,071	0.700	1.000	3,005	363

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.399	0.138	5/3	26	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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Fastening point:			

7 Installation data

Anchor type and diameter: Kwik Bolt TZ2 - CS 1/2 (2)
hnom2

Profile: no profile

Item number: 2210254 KB-TZ2 1/2x3 3/4

Hole diameter in the fixture: d_f = 0.562 in.

Maximum installation torque: 602 in.lb

Plate thickness (input): 0.500 in.

Hole diameter in the base material: 0.500 in.

Recommended plate thickness: not calculated

Hole depth in the base material: 2.750 in.

Drilling method: Hammer drilled

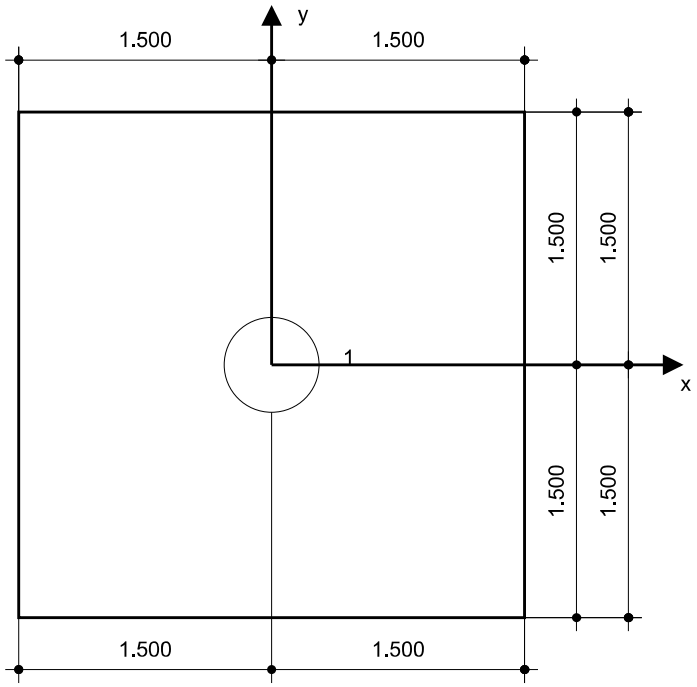
Minimum thickness of the base material: 4.000 in.

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti ∅ 1/2 in Kwik Bolt TZ2 - CS with 2.5 in nominal embedment depth per ICC-ES ESR-4266 , Hammer drill bit installation per MPII

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hammer



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	-	6.000	6.000	-

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Phone I Fax:		E-Mail:	
Design:	Corona South Mall - BLDG B- Inverter	Date:	12/9/2025
Fastening point:			

8 Remarks; Your Cooperation Duties

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13.3 SEISMIC DEMANDS ON NONSTRUCTURAL COMPONENTS (AC UNITS/DOAS)

$$F_P = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)}\left(1+2\frac{z}{h}\right) \qquad (13.3\text{-}1)$$

F_p is not required to be taken as greater than

$$F_p = 1.6 S_{DS} I_p W_p \qquad (13.3\text{-}2)$$

and F_p shall not be taken as less than

$$F_p = 0.3 S_{DS} I_p W_p \qquad (13.3\text{-}3)$$

F_p = seismic design force
S_{DS} = spectral acceleration, short period, as determined from Section 11.4.4
a_p = component amplification factor that varies from 1.00 to 2.50 (select appropriate value from Table 13.5-1 or 13.6-1)
I_p = component importance factor that varies from 1.00 to 1.50 (see Section 13.1.3)
W_p = component operating weight
R_p = component response modification factor that varies from 1.00 to 12 (select appropriate value from Table 13.5-1 or 13.6-1)
z = height in structure of point of attachment of component with respect to the base. For items at or below the base, z shall be taken as 0. The value of z/h need not exceed 1.0
h = average roof height of structure with respect to the base

ap=	2.5
Rp=	6.0
SDS=	1.38
IP=	1.5
z=	20.0 ft
h=	20.0 ft

Fp = 0.4 ap SDS Wp Ip (1+2 z/h) / Rp = 1.03 Wp

(Fp)max= 1.6 SDS Ip Wp = 3.30 Wp

(Fp)min= 0.3 SDS Ip Wp = 0.62 Wp

Seismic Force Fp= 1.03 Wp

Wind Load Calculations

F_h= Lateral Force

$F_h = q_z (G C_r) A_f$ Equation 29.5-2

F_v= Vertical Uplift Force

$F_v = q_z (G C_r) A_r$ Equation 29.5-3

$q_h = 0.00256 K_z K_{zt} K_d V^2$

Exposure: C
Building Height z= 20
Risk Category: III

Kz= velocity exposure factor — Kz= 0.9
Kzt=topographic factor — Kzt= 1.0
Kd= wind dirctionality factor — Kd= 0.85
K_e = ground elevation factor — K_e = 1
V= 95

qh= 17.7 psf

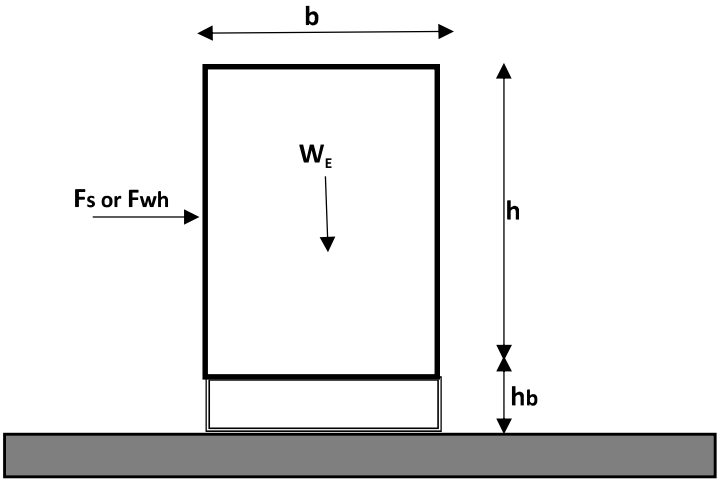
GCr= 1.9
GCr= 1.5

Lateral Pressure P_H=	33.58 psf
Vertical Pressure P_V=	26.51 psf

IDS GROUP
Structural Engineers
1 Peters Canyon Road, Suite 140
Irvine, CA 92606(949) 387-8500, Fax: (949) 387-0800

Sheet:			
Job No.:	24X84.00		
Calc. By:		Date:	12/19/2025
Chck'd By:		Date:	12/19/2025

Mechanical Equipment Anchorage



Equipment ID **AC Units/DOAS**

Seismic Factor=	SDS=	1.38
Equipment Data:		
Weight	W_E=	2.8 kips
Width	b=	41.0 in
Length	L=	91.0 in
Height	h=	59.0 in
Height of the base	hb=	14.0 in

Design Forces:

Seismic	**F_s=** 1.03 Wp	→			**F_s=**	**2.91**	**kips**
Wind	**P_H=** 33.58 psf	→	Af=	37.28 ft^2 →	**F_{wh}=**	**1.25**	**kips**
	P_V= 26.51 psf	→	Ar=	25.91 ft^2 →	**F_{wv}=**	**0.69**	**kips**

Overturning Moment

Seismic M_s= Fs (0.5h+hb)= 126.64 k-in Wind M_w= Fwh (0.5 h+hb) +Fwv (b/2)= 68.55 k-in

Resis. Moment M_r= (W_E)*b/2 = 57.79 k-in

Design Load Combinations

Seismic 0.6D + 0.7 E = (0.6 - 0.14 S_{DS})D + 0.7 E Wind 0.6D + 0.6 W

Check Overturning

Seismic Mover-s= 0.7Ms - (0.6-0.14 SDS) Mr= 65.1 K-in

Wind Mover-w= 0.6Mw - 0.6 Mr= 6.5 K-in

Anchors design Forces ASD Level

Min. numbers of connectors in the long side n : 6

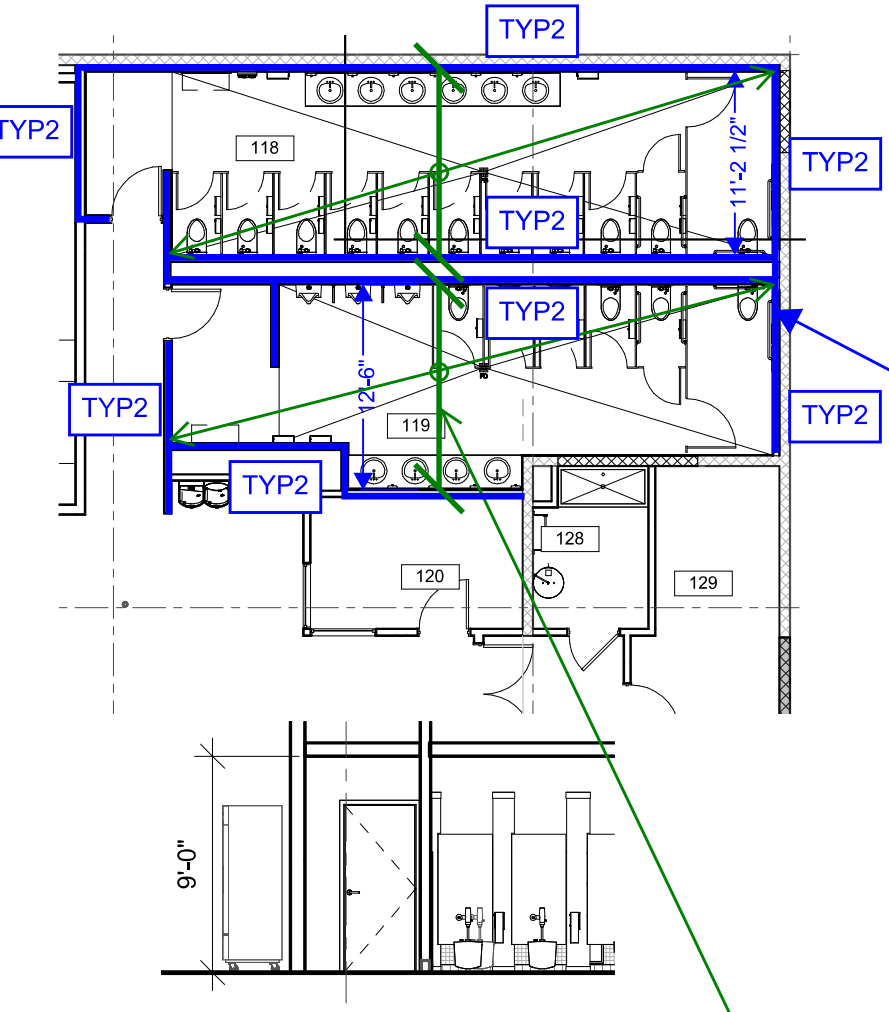
Shear force at each anchor V= Largest (0.7Fs or 0.6Fwh)/(2n)
Uplift force at each anchor P= Largest (Mover-s or Mover-w)/(bn)

V=	**170 lbs**
P=	**265 lbs**

Use 1/2" ø lag screw x 2.5" V_{allow} = 441 lbs DC = 0.69
P_{allow} = 870 lbs

IDS GROUP

PROJECT: City of Corona, South Mall Development	BY:	JOB NO: 24X084.01
SUBJECT: Metal Stud wall design	CHKD:	Date:



Stud wall:

TYPE 1 (General area w/ T-bar ceiling)
Height: 21'-0" MAX
Load: 5 psf horizontal load per CBC 2022 section 1607.16 interior walls and partitions
Use 400S162-54 (punched)@ 16" O.C.
w/ full height bridge @ 4' O.C.
see page 2

TYPE 2 (restroom area w/ hard lid ceiling)
Height: 21'-0" MAX.
Load: 5 psf horizontal load,
65 lbs point load @ 9' height (hard lid ceiling level),
100 lbs point load @ 3' height (Miscellaneous: counter-top, Urinal etc.)
Use 400S200-54 (punched) @ 16" O.C.
w/ full height bridge @ 4' O.C.
see page 3

TYPE 3 (Roller entrance gate w/ Half-height and glass on top)
Height: 10'-0" MAX.
Load: 5 psf horizontal load,
50 lbs point load @ top per stud (glass)
Use 400S162-54 (punched)@ 16" O.C.
w/ full height bridge @ 4' O.C.
see page 4

Hard lid ceiling framing in restroom area:

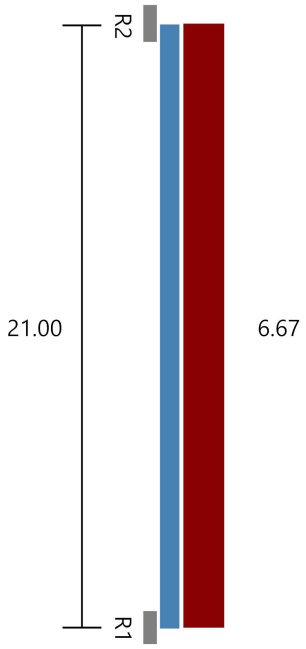
Span: 12'-6" MAX
Load: 10 psf

Use 600S200-54 (punched)@ 16" O.C.

w/ strap/CRC @ midspan

Project Name: Type 1 (General area)
Model: Type 1 (General area)
Code: 2007 NASPEC [AISI S100-2007]

Date: 10/24/2025
Simpson Strong-Tie® CFS Designer™ 1.4.2.0



Reactions and Connections

Number of Connectors Required at each Reaction: 1

Support	Reaction (lb)	Simpson Strong-Tie Connector	No. of Req'd screws to stud	No. of Req'd # 12-14 Anchors
R1	70.04	No Solutions	-	-
R2	70.04	No Solutions	-	-

Total Vertical Shear Carried by Connectors (lb) : 0

Bridging Connectors

Design Method = AISI S100

Span/CantiLever	Simpson Strong-Tie Bridging Connector	Stress Ratio
Span	N/A	-

Bending and Shear (Unstiffened): 2.1% Stressed @R2

Bending and Shear (Stiffened): N/A

Web Stiffeners Required?: No

Section : 400S162-54 Single C Stud
Fy = 50.0 ksi
Maxo = 1241.3 Ft-Lb **Moment of Intertia, I =** 1.10 in^4 **Va =** 3371.6 lb

Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations
Reactions have been multiplied by 1.0 for opposite load direction for connection design

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Span	367.7	0.296	367.7	48.0	1143.9	0.321	0.631	L/399

Distortional Buckling Check

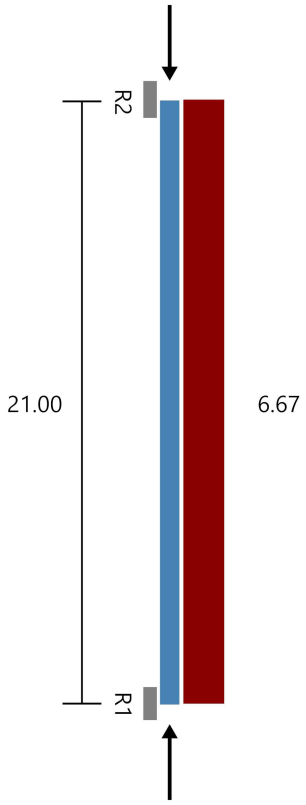
Span	K-phi lb-in/in	Lm Brac in	Ma-d Ft-Lb	Mmax/ Ma-d
Span	0.00	252.0	1270.7	0.289

L/399 < L/360 per CBC table 1604A.3 deflection limits.....OK

TYPE 1 (General area w/ T-bar ceiling)
Height: 21'-0" MAX
Load: 5 psf horizontal load per CBC 2022 section 1607.16 interior walls and partitions
Use 400S162-54 (punched) @ 16" O.C.
w/ solid BLKG. @ 8' O.C.

Project Name: Type 2 (restroom area)
Model: Type 2 (Restroom area)
Code: 2007 NASPEC [AISI S100-2007]

Date: 10/24/2025
Simpson Strong-Tie® CFS Designer™ 1.4.2.0



Reactions and Connections

Number of Connectors Required at each Reaction: 1

Support	Reaction (lb)	Simpson Strong-Tie Connector	No. of Req'd screws to stud	No. of Req'd # 12-14 Anchors
R1	70.04	No Solutions	-	-
R2	70.04	No Solutions	-	-

Total Vertical Shear Carried by Connectors (lb) : 0

Bridging Connectors

Design Method = AISI S100

Span/CantiLever	Simpson Strong-Tie Bridging Connector	Stress Ratio
Span	N/A	-

Bending and Shear (Unstiffened): 2.1% Stressed @R2

Bending and Shear (Stiffened): N/A

Web Stiffeners Required?: No

Section : 400S200-54 Single C Stud
Maxo = 1368.8 Ft-Lb **Moment of Intertia, I =** 1.29 in^4
Fy = 50.0 ksi
Va = 3371.6 lb

Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations
Reactions have been multiplied by 1.0 for opposite load direction for connection design

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Span	367.7	0.269	367.7	48.0	1349.5	0.272	0.536	L/470

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac in	Ma-d Ft-Lb	Mmax/ Ma-d
Span	0.00	252.0	1442.7	0.255

Combined Bending and Axial Load Details

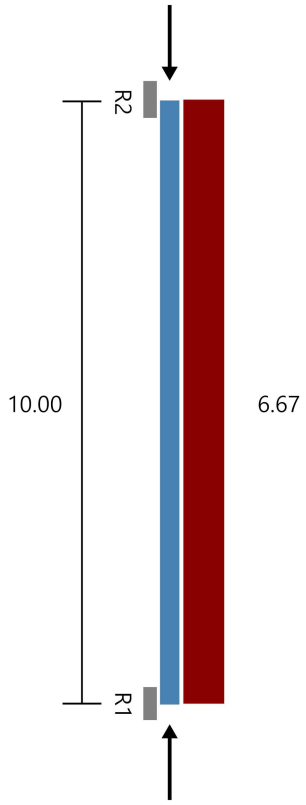
Span	Axial Ld (lb)	Bracing(in) KyLy	Bracing(in) KtLt	Max KL/r	K-phi (lb-in/in)	Lm Bracing (in)
Span	165.0(c)	96.0	96.0	157	0.0	252.0

L/470 < L/360 per CBC table 1604A.3 deflection limits.....OK

TYPE 2 (restroom area w/ hard lid ceiling)
Height: 21'-0" MAX.
Load: 5 psf horizontal load,
65 lbs point load @ 9' height (hard lid ceiling level), 100 lbs point load @ 3' height (Miscellaneous: counter-top, Urinal etc.)
Use 400S200-54 (punched) @ 16" O.C.
w/ solid BLKG. @ 8' O.C.

Project Name: TYPE 3 (Roller entrance
Model: TYPE 3 (Roller entrance
Code: 2007 NASPEC [AISI S100-2007]

Date: 10/24/2025
Simpson Strong-Tie® CFS Designer™ 1.4.2.0



Reactions and Connections

Number of Connectors Required at each Reaction: 1

Support	Reaction (lb)	Simpson Strong-Tie Connector	No. of Req'd screws to stud	No. of Req'd # 12-14 Anchors
R1	33.35	No Solutions	-	-
R2	33.35	No Solutions	-	-

Total Vertical Shear Carried by Connectors (lb) : 0

Bridging Connectors

Design Method = AISI S100

Span/CantiLever	Simpson Strong-Tie Bridging Connector	Stress Ratio
Span	N/A	-

Bending and Shear (Unstiffened): 1.0% Stressed @R2
Bending and Shear (Stiffened): N/A
Web Stiffeners Required?: No

Section : 400S162-54 Single C Stud
Maxo = 1241.3 Ft-Lb **Moment of Intertia, I =** 1.10 in^4
Fy = 50.0 ksi
Va = 3371.6 lb

Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations
Reactions have been multiplied by 1.0 for opposite load direction for connection design

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Ratio
Span	83.4	0.067	83.4	48.0	1146.9	0.073	0.032	L/3699

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac in	Ma-d Ft-Lb	Mmax/ Ma-d
Span	0.00	120.0	1270.7	0.066

Combined Bending and Axial Load Details

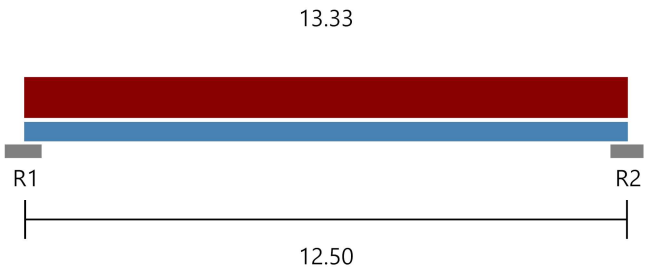
Span	Axial Ld (lb)	Bracing(in) KyLy	Bracing(in) KtLt	Max KL/r	K-phi (lb-in/in)	Lm Bracing (in)	Allow load(lb)	P/Pa	Intr. Value
Span	50.0(c)	96.0	96.0	160	0.0	120.0	1892.4(c)	0.03	0.10

TYPE 3 (Roller entrance gate w/ Half-height and glass on top)
Height: 10'-0" MAX.
Load: 5 psf horizontal load,
50 lbs point load @ top per stud (glass)
Use 400S162-54 (punched) @ 16" O.C.
w/ solid BLKG. @ 8' O.C.

L/3699 < L/360 per CBC table 1604A.3 deflection limits.....OK

Project Name: hard lid ceiling framing
Model: Hard lid ceiling framing
Code: 2007 NASPEC [AISI S100-2007]

Date: 10/15/2025
Simpson Strong-Tie® CFS Designer™ 1.4.2.0



Reactions

Support	Reactions (lb)
R1	83.31
R2	83.31

Bending and Shear (Unstiffened): 3.0% Stressed @R2

Bending and Shear (Stiffened): N/A

Web Stiffeners Required?: No

Section : 600S200-54 Single C Stud
Maxo = 2532.9 Ft-Lb **Moment of Intertia, I =** 3.32 in^4
Fy = 50.0 ksi
Va = 2822.9 lb

Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Ratio
Span	260.4	0.103	260.4	None	736.1	0.354	0.052	L/2866

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac in	Ma-d Ft-Lb	Mmax/ Ma-d
Span	0.00	150.0	2281.9	0.114

Hard lid ceiling framing in restroom area:

Span: 12'-6" MAX
Load: 10 psf

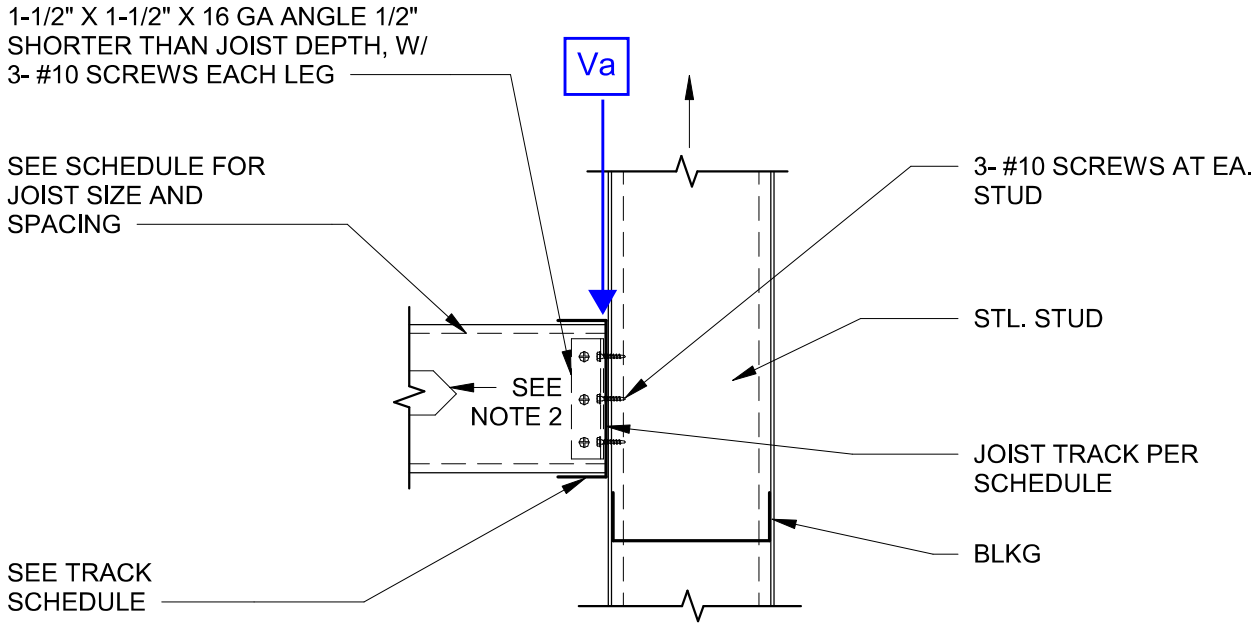
Use 600S200-54 (punched)@ 16" O.C.
w/ strap/CRC @ midspan

L/2866 < L/360 per CBC table 1604A.3 deflection limits.....OK

IDS GROUP

PROJECT: City of Corona, South Mall Development	BY:	JOB NO: 24X084.01
SUBJECT: Connection check	CHKD:	Date:

VERTICAL SLIP
FOR TOP CONNECTIONS



CEILING JOIST SCHEDULE

MAX. SPAN	STEEL STUD SIZE	STEEL TRACK SIZE
16'-0"	600S200-54 AT 16" O.C.	600T200-54

V = 83.3 lb (see previous page) use 85lb
provide (3) #10 SMS
V_{ASD}= 289 * (3) = 867 lb (see next page)

V < V_{ASD}.....O.K.

TABLE 3—SHEAR (BEARING) CAPACITY OF SCREW CONNECTIONS, pounds-force[1,2,3,4,5]

SCREW TYPE	SCREW DESIGNATION	NOMINAL DIAMETER (in.)	DESIGN THICKNESS (in.)[6]						
			0.048-0.048	0.048-0.075	0.060-0.060	0.075-0.075	1/8"-3/16"	3/16"-1/4"	1/4"-0.105"
ALLOWABLE STRENGTH (ASD)									
1	#10-16	0.190	289	289	404	-	-	-	-
2	#10-16	0.190	369	395	453	-	-	-	-
3, 4	#12-14	0.216	356	573	513	497	-	-	-
6	1/4-14	0.250	377	626	520	661	638	-	-
7, 8	1/4-20	0.250	386[7,8]	526[7,8]	533[8]	670[8]	595[9]	624[9]	554[9]
11	5/16-18	0.313	408	622	561	891	-	-	-
12	5/16-24	0.313	-	-	-	-	1347	984	887
DESIGN STRENGTH (LRFD)									
1	#10-16	0.190	433	433	605	-	-	-	-
2	#10-16	0.190	590	631	724	-	-	-	-
3, 4	#12-14	0.216	569	917	820	795	-	-	-
6	1/4-14	0.250	603	1001	833	1058	1021		
7, 8	1/4-20	0.250	617[7,8]	842[7,8]	852[8]	1072[8]	952[9]	999[9]	886[9]
11	5/16-18	0.313	653	996	897	1425	-	-	-
12	5/16-24	0.313	-	-	-	-	2155	1575	1419

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 Mpa.

[1]Available strengths are based on laboratory tests, with safety factors/resistance factors calculated in accordance with AISI S100.
[2]For shear connections, the lower of the available fastener shear strength and the available shear (bearing) capacity must be used for design.
[3]Values are based on steel members with a minimum yield strength of F_y = 33 ksi and a minimum tensile strength of F_u = 45 ksi.
[4]Available capacity for other member thickness may be determined by interpolating within the table.
[5]Unless otherwise noted, when both steel sheets have a minimum specified tensile strength $F_u \geq 58$ ksi, multiply tabulated values by 1.29 and when both steel sheets have a minimum tensile strength $F_u \geq 65$ ksi steel, multiply tabulated values by 1.44.
[6]The first number is the thickness of the steel sheet in in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet).
[7]When both steel sheets have a minimum specified tensile strength of $F_u \geq 55$ ksi (e.g. ASTM A653 SS Grade 40), multiply tabulated values by 1.22.
[8]When both steel sheets have a minimum specified tensile strength of $F_u \geq 52$ ksi (e.g. ASTM A653 SS Grade 37), multiply tabulated values by 1.15.
[9]When both steel sheets have a minimum specified tensile strength of $F_u \geq 58$ ksi (e.g. ASTM A36), multiply tabulated values by 1.29.

TABLE 4—TENSILE PULL-OVER CAPACITY OF SCREW CONNECTIONS, pounds-force[1,3,4,5,6]

SCREW TYPE	SCREW DESIGNATION	NOMINAL DIAMETER (in.)	MINIMUM EFFECTIVE PULL-OVER DIAMETER (in.)	DESIGN THICKNESS OF MEMBER IN CONTACT WITH SCREW HEAD (in.)							
				0.048	0.060	0.075	0.105	1/8"	3/16"	1/4"	5/16"
ALLOWABLE STRENGTH (ASD)											
1	#10-16	0.190	0.357	386	481[2]	481[2]	481[2]	481[2]	-	-	-
2	#10-16	0.190	0.384	415	481[2]	481[2]	481[2]	481[2]	-	-	-
3, 4	#12-14	0.216	0.398	430	537	672	734[2]	734[2]	734[2]	-	-
5	#12-24	0.216	0.398	430	537	672	734[2]	734[2]	734[2]	734[2]	734[2]
6	1/4-14	0.250	0.480	518	648	810	1126[2]	1126[2]	1126[2]	-	-
7, 8	1/4-20	0.250	0.480	-	648	810	1126[2]	1126[2]	1126[2]	1126[2]	1126[2]
11	5/16-18	0.313	n/a[2]	-	-	-	1169[2]	1169[2]	-	-	-
12	5/16-24	0.313	n/a[2]	-	-	-	1326[2]	1326[2]	1326[2]	1326[2]	1326[2]
DESIGN STRENGTH (LRFD)											
1	#10-16	0.190	0.357	578	723	781[2]	781[2]	781[2]	-	-	-
2	#10-16	0.190	0.384	622	778	781[2]	781[2]	781[2]	-	-	-
3, 4	#12-14	0.216	0.398	645	806	1007	1192[2]	1192[2]	1192[2]	-	-
5	#12-24	0.216	0.398	645	806	1007	1192[2]	1192[2]	1192[2]	1192[2]	1192[2]
6	1/4-14	0.250	0.480	778	972	1215	1701	1830[2]	1830[2]	-	-
7, 8	1/4-20	0.250	0.480	-	972	1215	1701	1830[2]	1830[2]	1830[2]	1830[2]
11	5/16-18	0.313	n/a[2]	-	-	-	1871[2]	1871[2]	-	-	-
12	5/16-24	0.313	n/a[2]	-	-	-	2121[2]	2121[2]	2121[2]	2121[2]	2121[2]

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 Mpa.

[1]Available strengths are based on calculations in accordance with AISI S100, unless otherwise noted.
[2]Available strengths are based on laboratory tests, with safety factors/resistance factors calculated in accordance with AISI S100, or on the shear strength of the integral washer. Increasing values for higher steel tensile strength per Note 6 is not allowed.
[3]For tension connections, the lowest of the available pull-out, pull-over, and fastener tension strength must be used for design.
[4]Values are based on steel members with a minimum yield strength of F_y = 33 ksi and a minimum tensile strength of F_u = 45 ksi.
[5]Available capacity for other member thickness may be determined by interpolating within the table.
[6]For steel with a minimum tensile strength $F_u \geq 58$ ksi, multiply tabulated values by 1.29 and for steel with a minimum tensile strength $F_u \geq 65$ ksi steel, multiply tabulated values by 1.44.

101379343

percentages to account for the effects of vertical impact or vibration:

Monorail cranes (powered)	25 percent
Cab-operated or remotely operated bridge cranes (powered)	25 percent
Pendant-operated bridge cranes (powered)	10 percent
Bridge cranes or monorail cranes with hand-geared bridge, trolley and hoist	0 percent

1607.15.3 Lateral force. The lateral force on crane runway beams with electrically powered trolleys shall be calculated as 20 percent of the sum of the rated capacity of the crane and the weight of the hoist and trolley. The lateral force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction perpendicular to the beam, and shall be distributed with due regard to the lateral stiffness of the runway beam and supporting structure.

1607.15.4 Longitudinal force. The longitudinal force on crane runway beams, except for bridge cranes with hand-geared bridges, shall be calculated as 10 percent of the maximum wheel loads of the crane. The longitudinal force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction parallel to the beam.

1607.16 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength and stiffness to resist the loads to which they are subjected but not less than a horizontal load of 5 psf (0.240 kN/m^2).

1607.16.1 Fabric partitions. Fabric partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength and stiffness to resist the following load conditions:

1. The horizontal distributed load need only be applied to the partition framing. The total area used to determine the distributed load shall be the area of the fabric face between the framing members to which the fabric is attached. The total distributed load shall be uniformly applied to such framing members in proportion to the length of each member.
2. A concentrated load of 40 pounds (0.176 kN) applied to an 8-inch-diameter (203 mm) area [50.3 square inches (32 452 mm^2)] of the fabric face at a height of 54 inches (1372 mm) above the floor.

1607.16.2 Fire walls. In order to meet the structural stability requirements of Section 706.2 where the structure on either side of the wall has collapsed, fire walls and their supports shall be designed to withstand a minimum horizontal allowable stress load of 5 psf (0.240 kN/m^2).

1607.17 Fixed ladders. Fixed ladders with rungs shall be designed to resist a single concentrated load of 300 pounds (1.33 kN) in accordance with Section 4.5.4 of ASCE 7. Where rails of fixed ladders extend above a floor or platform at the top of the ladder, each side rail extension shall be designed to resist a single concentrated load of 100 pounds (0.445 kN) in accordance with Section 4.5.4 of ASCE 7. Ship's ladders shall be designed to resist the stair loads given in Table 1607.1.

1607.18 Library stack rooms. The live loading indicated in Table 1607.1 for library stack rooms applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:

1. The nominal book stack unit height shall not exceed 90 inches (2290 mm).
2. The nominal shelf depth shall not exceed 12 inches (305 mm) for each face.
3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches (914 mm) in width.

1607.19 Seating for assembly uses. Bleachers, folding and telescopic seating and grandstands shall be designed for the loads specified in ICC 300. Stadiums and arenas with fixed seats shall be designed for the horizontal sway loads in Section 1607.19.1.

1607.19.1 Horizontal sway loads. The design of stadiums and arenas with fixed seats shall include horizontal swaying forces applied to each row of seats as follows:

1. 24 pounds per linear foot (0.35 kN/m) of seat applied in a direction parallel to each row of seats.
2. 10 pounds per linear foot (0.15 kN/m) of seat applied in a direction perpendicular to each row of seats.

The parallel and perpendicular horizontal swaying forces are not required to be applied simultaneously.

1607.20 Sidewalks, vehicular driveways, and yards subject to trucking. The live loading indicated in Table 1607.1 for sidewalks, vehicular driveways, and yards subject to trucking shall comply with the requirements of this section.

1607.20.1 Uniform loads. In addition to the loads indicated in Table 1607.1, other uniform loads in accordance with an approved method that contains provisions for truck loading shall be considered where appropriate.

1607.20.2 Concentrated loads. The concentrated wheel load indicated in Table 1607.1 shall be applied on an area of $4^1/_2$ inches by $4^1/_2$ inches (114 mm by 114 mm).

1607.21 Stair treads. The concentrated load indicated in Table 1607.1 for stair treads shall be applied on an area of 2 inches by 2 inches (51 mm by 51 mm). This load need not be assumed to act concurrently with the uniform load.

1607.22 Residential attics. The live loads indicated in Table 1607.1 for attics in residential occupancies shall comply with the requirements of this section.

1607.22.1 Uninhabitable attics without storage. In residential occupancies, uninhabitable attic areas without storage are those where the maximum clear height between the joists and rafters is less than 42 inches (1067 mm), or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches (1067 mm) in height by 24 inches (610 mm) in width, or greater, within the plane of the trusses. The live load in Table 1607.1 need not be assumed to act concurrently with any other live load requirement.

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Sheet:			
Job No.:	24X084.01		
Calc. By.:	EC		
Chck'd By.:		Date:	11/7/2025

Design Criteria Summary

Building Code

2022 CBC

Dead Loads

Ceiling Roof

Framing	=	0.00 psf
Misc.	=	8.00 psf
		8.00 psf

Live Loads

Ceiling Roof

Maintenance	=	5.00 psf
		5.00 psf

Lateral Loads

Seismic Criteria:

Risk Category	=	III	S_S	=	2.065
Site Class	=	D - Stiff Soil	S_1	=	0.783

Wind Criteria:

Inside structure no wind load

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Chck'd By.:		Date:	11/7/2025

Single Story Building Lateral - ASCE 7-16

Buidling Specifications:

L	=	69.00 ft		Risk Cat.	=	III
B	=	27.00 ft		Site Class	=	D - Stiff Soil
h_{high}	=	15.75 ft	*avg.*			
h_{low}	=	12.00 ft	*max.*			
$h_{avg.}$	=	13.88 ft	*max.*			

Seismic Design Parameters:

F_a	=	1.000	*ASCE 7-16 Tbl. 11.4-1*
F_v	=	1.700	*ASCE 7-16 Tbl. 11.4-2*
S_S	=	2.065	*ASCE 7-16 Ch.22*
S_1	=	0.783	*ASCE 7-16 Ch.22*
S_{MS}	=	2.065	*ASCE 7-16 Eq. 11.4-1*
S_{M1}	=	1.331	*ASCE 7-16 Eq. 11.4-2*
S_{DS}	=	1.3767	*ASCE 7-16 Eq. 11.4-3*
S_{D1}	=	0.887	*ASCE 7-16 Eq. 11.4-4*
T_L	=	8.000	*ASCE 7-16 Ch. 22*
SDC	=	D	*ASCE 7-16 Tbl. 11.6-1 / Tbl. 11.6-2*

Lateral Force Resisting System:

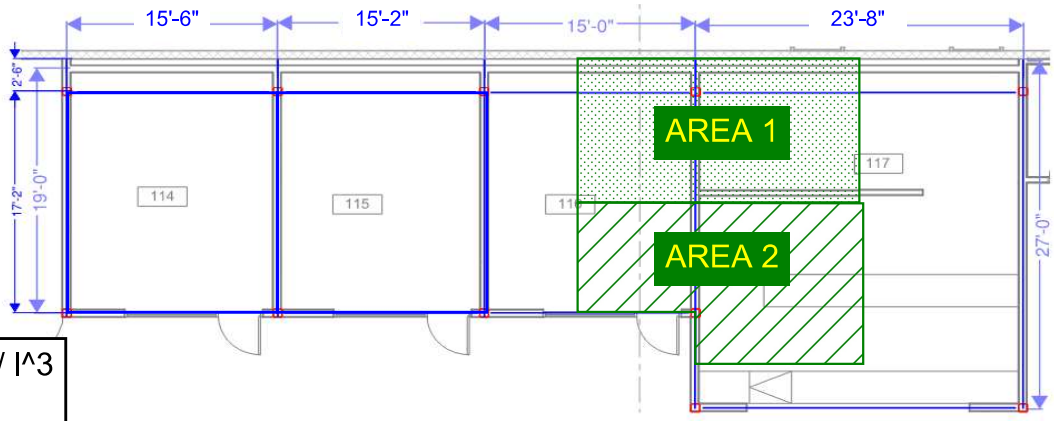
G. Cantilevered Column Systems detailed to conform to the requirements for:

2. Steel ordinary cantilever column system

R	=	1.250	*ASCE 7-16 Tbl. 12.2-1*
Ω_0	=	1.250	*ASCE 7-16 Tbl. 12.2-1*
C_d	=	1.250	*ASCE 7-16 Tbl. 12.2-1*
I_e	=	1.250	*ASCE 7-16 Tbl. 1.5-2*
C_t	=	0.020	*ASCE 7-16 Tbl. 12.8-2*
x	=	0.750	*ASCE 7-16 Tbl. 12.8-2*
T	=	0.158	*ASCE 7-16 Eq. 12.8-7*
C_s	=	1.377	*ASCE 7-16 Eq. 12.8-2*
C_{s_max}	=	8.418	*ASCE 7-16 Eq. 12.8-3*
C_{s_min}	=	0.010	*ASCE 7-16 Eq. 12.8-5*
C_s	=	1.377	
V	**=**	**1.377 W**	*ASCE 7-16 Eq. 12.8-1*
E_v	**=**	**0.275 W**	*ASCE 7-16 Eq. 12.8-2*

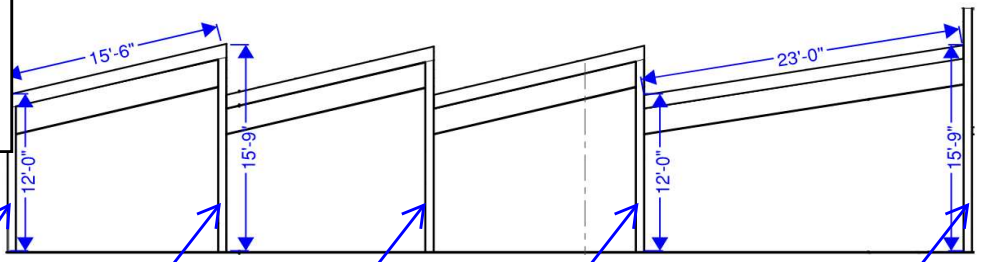
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k= P/δ MAX = 3EI / l^3
Same material & section column with different height

k= 1 / l^3



1/12^3 = 1/1728

Relative stiffness 36%

1/15.75^3 = 64/250047

Relative stiffness 16%

1/15.75^3 = 64/250047

Relative stiffness 16%

1/15.75^3 = 64/250047

Relative stiffness 16%

1/15.75^3 = 64/250047

Relative stiffness 16%

Total roof area= 19'-8" x 46'-2" + 23'-8" x 27'-0" = 908 sqft + 639 sqft = 1547 sqft

DL1 (using column relative stiffness)
= 1547 sqft * 1/2 * 36% * 8 psf (design roof DL) = 2228 lb per column

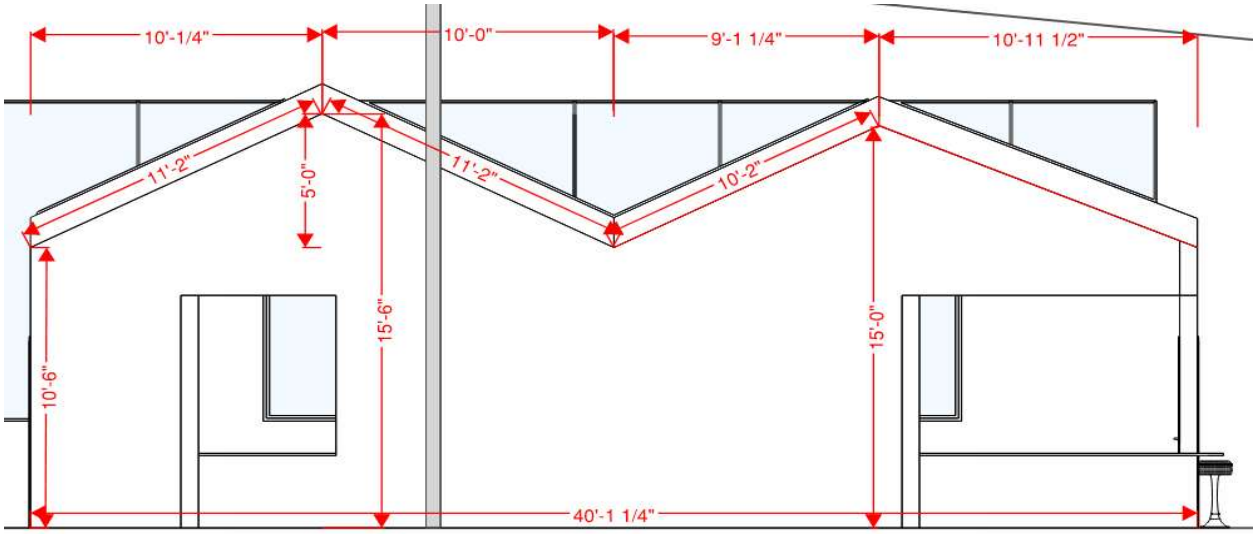
DL2 (area 1)
= (19.6' * 11.08') * 8psf = 217.2 sqft * 8 psf = 1738 lb per column

DL3 (area 2)
= (19.6' * 16') * 8psf = 313.6 sqft * 8 psf= 2509 lb per column

Using DL1 = 2228 lb for checking 12 ft height column and pole footing
Using DL3 = 2509 lb for design 15'-9" height column and pole footing

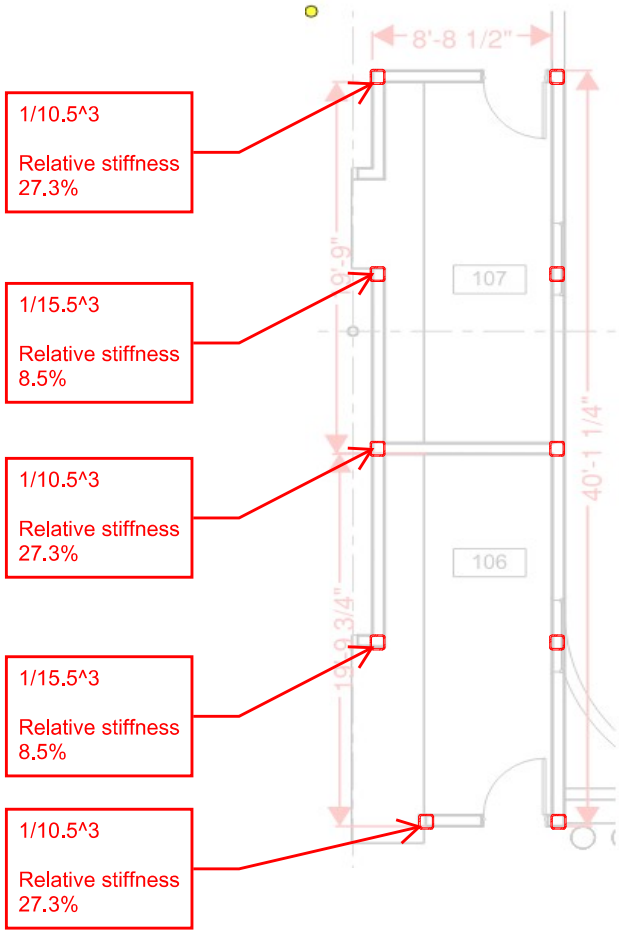
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Total roof area= 8'-8" x 42'-8"= 370 sqft

370 sqft * 1/2 * 27.3% * 8 psf = 404 lb per column



Steel Column

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: 12 ft cantilever column

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, SDPWS 2015
Load Combinations Used : ASCE 7-22 / IBC 2024 (L<=100psf)

General Information

Steel Section Name : **HSS7x7x5/16**

Analysis Method :	Allowable Strength
Steel Stress Grade	
Fy : Steel Yield	46.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	12.0 ft
Top & Bottom Fixity	Top Free, Bottom Fixed
Brace condition :	

Unbraced Length for buckling ABOUT X-X Axis = 12 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 12 ft, K = 2.1

Applied Loads

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

Column self weight included : 331.080 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 12.0 ft, D = 2.228, L = 1.390, E = 0.6130 k
BENDING LOADS . . .
ceiling roof: Lat. Point Load at 12.0 ft creating Mx-x, E = 3.068 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.6104** : 1

Load Combination	+D+0.70E	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	2.988	k
Pn / Omega : Allowabl	91.031	k
Ma-x : Applied	-25.771	k-ft
Mn-x / Omega : Allowable	43.383	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	43.383	k-ft

PASS Maximum Shear Stress Rati **0.03644** : 1

Load Combination	+D+0.70E	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	2.148	k
Vn / Omega : Allowable	58.934	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	3.068 k

Maximum Load Deflections . . .

Along Y-Y 1.868 in at 12.0 ft above base
for load combination : E Only

Along X-X 0.0 in at 0.0 ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.028	PASS	0.00 ft	1.67	1.00	111.18	111.18	0.000	PASS	0.00 ft
+D+L	0.043	PASS	0.00 ft	1.67	1.00	111.18	111.18	0.000	PASS	0.00 ft
+D+0.750L	0.040	PASS	0.00 ft	1.67	1.00	111.18	111.18	0.000	PASS	0.00 ft
+0.60D	0.017	PASS	0.00 ft	1.67	1.00	111.18	111.18	0.000	PASS	0.00 ft
+D+0.70E	0.610	PASS	0.00 ft	1.67	1.00	111.18	111.18	0.036	PASS	0.00 ft
+D+0.750L+0.5250E	0.467	PASS	0.00 ft	1.67	1.00	111.18	111.18	0.027	PASS	0.00 ft
+0.60D+0.70E	0.605	PASS	0.00 ft	1.67	1.00	111.18	111.18	0.036	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	2.559									
+D+L	3.949									
+D+0.750L	3.602									
+0.60D	1.535									
+D+0.70E	2.988				2.148		-25.771			
+D+0.750L+0.5250E	3.923				1.611		-19.328			
+0.60D+0.70E	1.965				2.148		-25.771			

Steel Column

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: 12 ft cantilever column

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
L Only	1.390									
E Only	0.613				3.068		-36.816			

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	3.949									
"	Minimum	0.613				3.068		-36.816			
Reaction, X-X Axis Base	Maximum	2.559									
"	Minimum	2.559									
Reaction, Y-Y Axis Base	Maximum	0.613				3.068		-36.816			
"	Minimum	2.559									
Reaction, X-X Axis Top	Maximum	2.559									
"	Minimum	2.559									
Reaction, Y-Y Axis Top	Maximum	2.559									
"	Minimum	2.559									
Moment, X-X Axis Base	Maximum	2.559									
"	Minimum	0.613	-36.816			3.068		-36.816			
Moment, Y-Y Axis Base	Maximum	2.559									
"	Minimum	2.559									
Moment, X-X Axis Top	Maximum	2.559									
"	Minimum	2.559									
Moment, Y-Y Axis Top	Maximum	2.559									
"	Minimum	2.559									

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	1.307 in	12.000 ft
+D+0.750L+0.5250E	0.0000 in	0.000 ft	0.980 in	12.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	1.307 in	12.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	1.849 in	11.919 ft

Steel Section Properties : HSS7x7x5/16

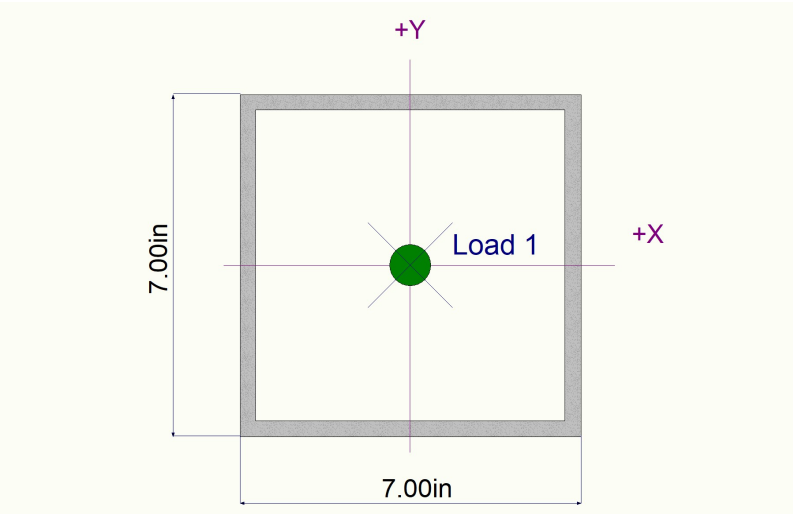
Depth	=	7.000 in	I xx	=	56.10 in^4		J	=	89.700 in^4	
Design Thick	=	0.291 in	S xx	=	16.00 in^3					
Width	=	7.000 in	R xx	=	2.720 in					
Wall Thick	=	0.313 in	Zx	=	18.900 in^3					
Area	=	7.590 in^2	I yy	=	56.100 in^4		C	=	26.100 in^3	
Weight	=	27.590 plf	S yy	=	16.000 in^3					
			R yy	=	2.720 in					
Ycg	=	0.000 in								

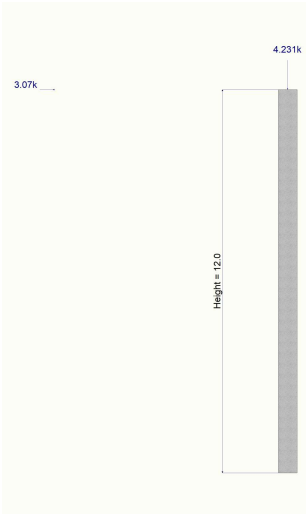
Steel Column

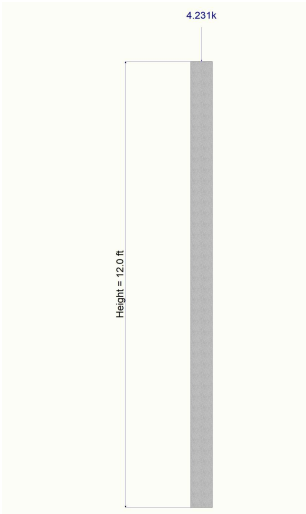
LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: 12 ft cantilever column

Sketches







Steel Column

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: check 15% axial_12 ft cantilever column

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, SDPWS 2015
Load Combinations Used : ASCE 7-22 / IBC 2024 (L<=100psf)

General Information

Steel Section Name : **HSS7x7x5/16**

Analysis Method :	Allowable Strength
Steel Stress Grade	
Fy : Steel Yield	46.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	12.0 ft
Top & Bottom Fixity	Top Free, Bottom Fixed
Brace condition :	

Unbraced Length for buckling ABOUT X-X Axis = 12 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 12 ft, K = 2.1

Applied Loads

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

Column self weight included : 331.080 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 12.0 ft, D = 2.228, L = 1.390, E = 0.6130 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = 0.04338 : 1

Load Combination	+D+L
Location of max.above base	0.0 ft
At maximum location values are . . .	
Pa : Axial	3.949 k
Pn / Omega : Allowabl	91.031 k
Ma-x : Applied	0.0 k-ft
Mn-x / Omega : Allowable	43.383 k-ft
Ma-y : Applied	0.0 k-ft
Mn-y / Omega : Allowable	43.383 k-ft

PASS Maximum Shear Stress Rati 0.0 : 1

Load Combination	0.0
Location of max.above base	0.0 ft
At maximum location values are . . .	
Va : Applied	0.0 k
Vn / Omega : Allowable	0.0 k

Maximum Load Reactions . .

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

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination :

Along X-X 0.0 in at 0.0 ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.028	PASS	0.00 ft	1.00	1.00	111.18	111.18	0.000	PASS	0.00 ft
+D+L	0.043	PASS	0.00 ft	1.00	1.00	111.18	111.18	0.000	PASS	0.00 ft
+D+0.750L	0.040	PASS	0.00 ft	1.00	1.00	111.18	111.18	0.000	PASS	0.00 ft
+0.60D	0.017	PASS	0.00 ft	1.00	1.00	111.18	111.18	0.000	PASS	0.00 ft
+D+0.70E	0.033	PASS	0.00 ft	1.00	1.00	111.18	111.18	0.000	PASS	0.00 ft
+D+0.750L+0.5250E	0.043	PASS	0.00 ft	1.00	1.00	111.18	111.18	0.000	PASS	0.00 ft
+0.60D+0.70E	0.022	PASS	0.00 ft	1.00	1.00	111.18	111.18	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	2.559								
+D+L	3.949								
+D+0.750L	3.602								
+0.60D	1.535								
+D+0.70E	2.988								
+D+0.750L+0.5250E	3.923								
+0.60D+0.70E	1.965								
L Only	1.390								
E Only	0.613								

Steel Column

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: check 15% axial_12 ft cantilever column

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	3.949								
"	Minimum	0.613								
Reaction, X-X Axis Base	Maximum	2.559								
"	Minimum	2.559								
Reaction, Y-Y Axis Base	Maximum	2.559								
"	Minimum	2.559								
Reaction, X-X Axis Top	Maximum	2.559								
"	Minimum	2.559								
Reaction, Y-Y Axis Top	Maximum	2.559								
"	Minimum	2.559								
Moment, X-X Axis Base	Maximum	2.559								
"	Minimum	2.559								
Moment, Y-Y Axis Base	Maximum	2.559								
"	Minimum	2.559								
Moment, X-X Axis Top	Maximum	2.559								
"	Minimum	2.559								
Moment, Y-Y Axis Top	Maximum	2.559								
"	Minimum	2.559								

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.5250E	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : HSS7x7x5/16

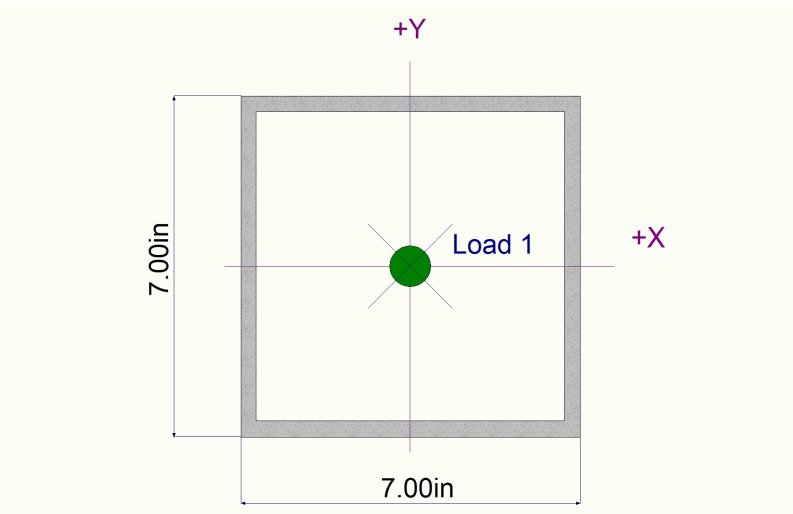
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Design Thick	=	0.291 in	S xx	=	16.00 in^3			
Width	=	7.000 in	R xx	=	2.720 in			
Wall Thick	=	0.313 in	Zx	=	18.900 in^3			
Area	=	7.590 in^2	I yy	=	56.100 in^4	C	=	26.100 in^3
Weight	=	27.590 plf	S yy	=	16.000 in^3			
			R yy	=	2.720 in			
Ycg	=	0.000 in						

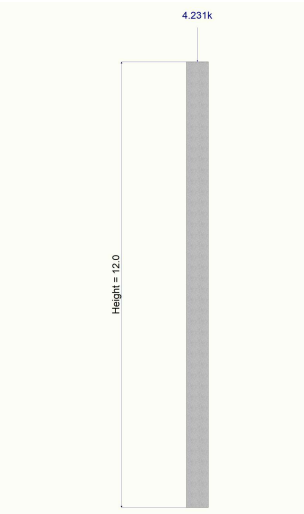
Steel Column

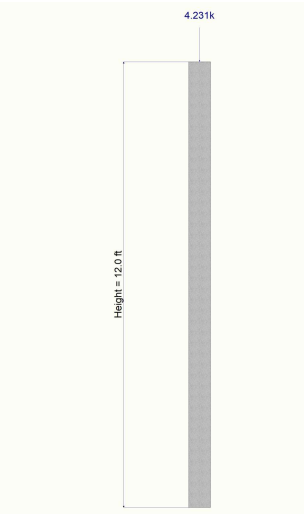
LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: check 15% axial_12 ft cantilever column

Sketches







Pole Footing Embedded in Soil

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: 12ft CANT. COL. pole foundation

Code References

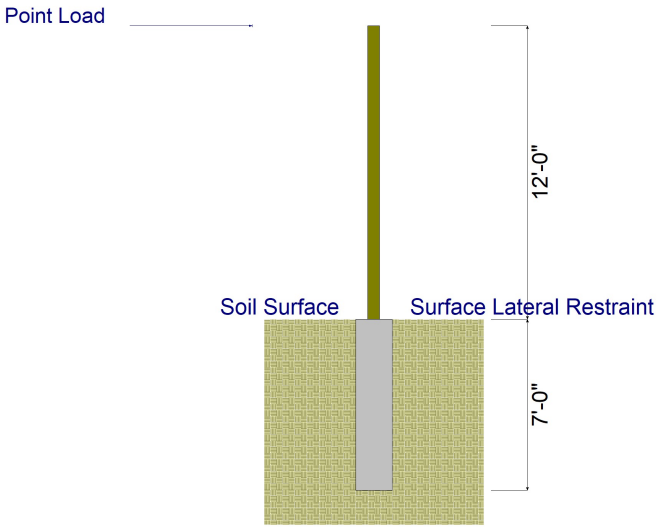
Calculations per IBC 2018 1807.3, CBC 2019, SDPWS 2015
Load Combinations Used : ASCE 7-22 / IBC 2024 (L<=100psf)

General Information

Pole Footing Shape	Circular
Pole Footing Diameter	18.0 in
Calculate Min. Depth for Allowable Pressures	
Lateral Restraint at Ground Surface	
Allow Passive	250.0 pcf
Max Passive	1,500.0 psf

Controlling Values

Governing Load Combinatio	+D+0.70E
Lateral Load	2.148 k
Moment	25.771 k-ft
Restraint @ Ground Surface	
Pressure at Depth	
Actual	**1,490.17** psf
Allowable	**1,500.0** psf
Surface Retraint Force	9,971.0 lbs
Minimum Required Depth	**7.0 ft**
Footing Base Area	1.767 ft^2
Maximum Soil Pressure	2.047 ksf



Applied Loads

Lateral Concentrated Load (k)		Lateral Distributed Loads (k		Applied Moment (kft)	Vertical Load (k)
D : Dead Load	k		k/ft	k-ft	2.228 k
Lr : Roof Live	k		k/ft	k-ft	k
L : Live	k		k/ft	k-ft	1.390 k
S : Snow	k		k/ft	k-ft	k
W : Wind	k		k/ft	k-ft	k
E : Earthquake	3.068 k		k/ft	k-ft	0.6130 k
H : Lateral Earth	k		k/ft	k-ft	k
Load distance above ground surface	12.0 ft	TOP of Load above ground surface	ft		
		BOTTOM of Load above ground surface	ft		

Load Combination Results

	Forces @ Ground Surface		Required	Pressure at Depth		Soil Increase
Load Combination	Loads - (k)	Moments - (ft-k)	Depth - (ft)	Actual - (psf)	Allow - (psf)	Factor
D Only	0.000	0.000	0.13	0.0	31.3	1.000
+D+L	0.000	0.000	0.13	0.0	31.3	1.000
+D+0.750L	0.000	0.000	0.13	0.0	31.3	1.000
+0.60D	0.000	0.000	0.13	0.0	31.3	1.000
+D+0.70E	2.148	25.771	7.00	1,490.2	1,500.0	1.000
+D+0.750L+0.5250E	1.611	19.328	6.13	1,459.8	1,500.0	1.000
+0.60D+0.70E	2.148	25.771	7.00	1,490.2	1,500.0	1.000

Pole Footing Embedded in Soil

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: w/ overstrength factor_12ft CANT. COL. pole foundation

Code References

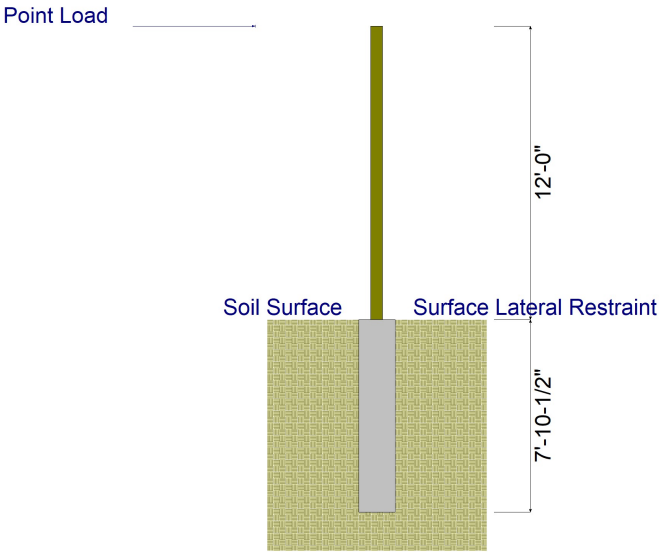
Calculations per IBC 2018 1807.3, CBC 2019, SDPWS 2015
Load Combinations Used : ASCE 7-22 / IBC 2024 (L<=100psf)

General Information

Pole Footing Shape	Circular
Pole Footing Diameter	18.0 in
Calculate Min. Depth for Allowable Pressures	
Lateral Restraint at Ground Surface	
Allow Passive	250.0 pcf
Max Passive	1,500.0 psf

Controlling Values

Governing Load Combinatio	+D+0.70E
Lateral Load	2.685 k
Moment	32.214 k-ft
Restraint @ Ground Surface	
Pressure at Depth	
Actual	**1,471.77** psf
Allowable	**1,500.0** psf
Surface Retraint Force	11,377.2 lbs
Minimum Required Depth	**7.875 ft**
Footing Base Area	1.767 ft^2
Maximum Soil Pressure	2.047 ksf



Applied Loads

Lateral Concentrated Load (k)		Lateral Distributed Loads (k		Applied Moment (kft)	Vertical Load (k)
D : Dead Load	k		k/ft	k-ft	2.228 k
Lr : Roof Live	k		k/ft	k-ft	k
L : Live	k		k/ft	k-ft	1.390 k
S : Snow	k		k/ft	k-ft	k
W : Wind	k		k/ft	k-ft	k
E : Earthquake	3.835 k		k/ft	k-ft	0.6130 k
H : Lateral Earth	k		k/ft	k-ft	k
Load distance above ground surface	12.0 ft	TOP of Load above ground surface	ft		
		BOTTOM of Load above ground surface	ft		

Load Combination Results

	Forces @ Ground Surface		Required	Pressure at Depth		Soil Increase
Load Combination	Loads - (k)	Moments - (ft-k)	Depth - (ft)	Actual - (psf)	Allow - (psf)	Factor
D Only	0.000	0.000	0.13	0.0	31.3	1.000
+D+L	0.000	0.000	0.13	0.0	31.3	1.000
+D+0.750L	0.000	0.000	0.13	0.0	31.3	1.000
+0.60D	0.000	0.000	0.13	0.0	31.3	1.000
+D+0.70E	2.685	32.214	7.88	1,471.8	1,500.0	1.000
+D+0.750L+0.5250E	2.013	24.161	6.88	1,448.3	1,500.0	1.000
+0.60D+0.70E	2.685	32.214	7.88	1,471.8	1,500.0	1.000

Steel Column

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: 15.75 ft cantilever column

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, SDPWS 2015
Load Combinations Used : ASCE 7-22 / IBC 2024 (L<=100psf)

General Information

Steel Section Name : **HSS7x7x1/2**

Analysis Method :	Allowable Strength
Steel Stress Grade	
Fy : Steel Yield	46.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	15.750 ft
Top & Bottom Fixity	Top Free, Bottom Fixed
Brace condition :	

Unbraced Length for buckling ABOUT X-X Axis = 12 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 12 ft, K = 2.1

Applied Loads

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

Column self weight included : 662.29 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 15.750 ft, D = 2.509, L = 1.568, E = 0.690 k
BENDING LOADS . . .
ceiling roof: Lat. Point Load at 13.875 ft creating Mx-x, E = 3.455 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.5379** : 1

Load Combination	+D+0.70E
Location of max.above base	0.0 ft
At maximum location values are . . .	
Pa : Axial	3.654 k
Pn / Omega : Allowabl	131.302 k
Ma-x : Applied	-33.557 k-ft
Mn-x / Omega : Allowable	64.042 k-ft
Ma-y : Applied	0.0 k-ft
Mn-y / Omega : Allowable	64.042 k-ft

PASS Maximum Shear Stress Rati **0.02807** : 1

Load Combination	+D+0.70E
Location of max.above base	0.0 ft
At maximum location values are . . .	
Va : Applied	2.419 k
Vn / Omega : Allowable	86.149 k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	3.455 k

Maximum Load Deflections . . .

Along Y-Y 2.726 in at 15.750 ft above base
for load combination : E Only

Along X-X 0.0 in at 0.0 ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.024	PASS	0.00 ft	1.84	1.00	114.98	114.98	0.000	PASS	0.00 ft
+D+L	0.036	PASS	0.00 ft	1.84	1.00	114.98	114.98	0.000	PASS	0.00 ft
+D+0.750L	0.033	PASS	0.00 ft	1.84	1.00	114.98	114.98	0.000	PASS	0.00 ft
+0.60D	0.014	PASS	0.00 ft	1.84	1.00	114.98	114.98	0.000	PASS	0.00 ft
+D+0.70E	0.538	PASS	0.00 ft	1.84	1.00	114.98	114.98	0.028	PASS	0.00 ft
+D+0.750L+0.5250E	0.411	PASS	0.00 ft	1.84	1.00	114.98	114.98	0.021	PASS	0.00 ft
+0.60D+0.70E	0.533	PASS	0.00 ft	1.84	1.00	114.98	114.98	0.028	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	3.171									
+D+L	4.739									
+D+0.750L	4.347									
+0.60D	1.903									
+D+0.70E	3.654				2.419		-33.557			
+D+0.750L+0.5250E	4.710				1.814		-25.168			
+0.60D+0.70E	2.386				2.419		-33.557			

Steel Column

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: 15.75 ft cantilever column

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
L Only	1.568									
E Only	0.690				3.455		-47.938			

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	4.739									
"	Minimum	0.690				3.455		-47.938			
Reaction, X-X Axis Base	Maximum	3.171									
"	Minimum	3.171									
Reaction, Y-Y Axis Base	Maximum	0.690				3.455		-47.938			
"	Minimum	3.171									
Reaction, X-X Axis Top	Maximum	3.171									
"	Minimum	3.171									
Reaction, Y-Y Axis Top	Maximum	3.171									
"	Minimum	3.171									
Moment, X-X Axis Base	Maximum	3.171									
"	Minimum	0.690	-47.938			3.455		-47.938			
Moment, Y-Y Axis Base	Maximum	3.171									
"	Minimum	3.171									
Moment, X-X Axis Top	Maximum	3.171									
"	Minimum	3.171									
Moment, Y-Y Axis Top	Maximum	3.171									
"	Minimum	3.171									

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	1.908 in	15.750 ft
+D+0.750L+0.5250E	0.0000 in	0.000 ft	1.431 in	15.750 ft
+0.60D+0.70E	0.0000 in	0.000 ft	1.908 in	15.750 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	2.700 in	15.644 ft

Steel Section Properties : HSS7x7x1/2

Depth	=	7.000	in	I xx	=	80.50	in^4	J	=	133.000 in^4
Design Thick	=	0.465	in	S xx	=	23.00	in^3			
Width	=	7.000	in	R xx	=	2.630	in			
Wall Thick	=	0.500	in	Zx	=	27.900	in^3			
Area	=	11.600	in^2	I yy	=	80.500	in^4	C	=	39.300 in^3
Weight	=	42.050	plf	S yy	=	23.000	in^3			
				R yy	=	2.630	in			

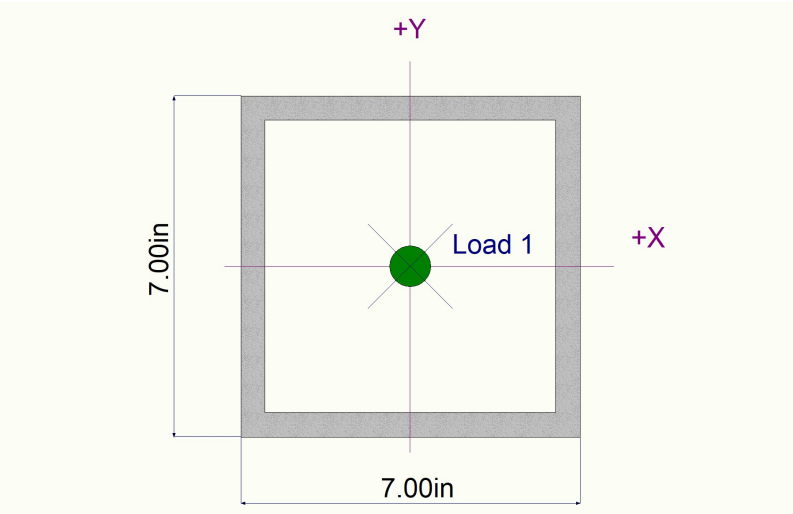
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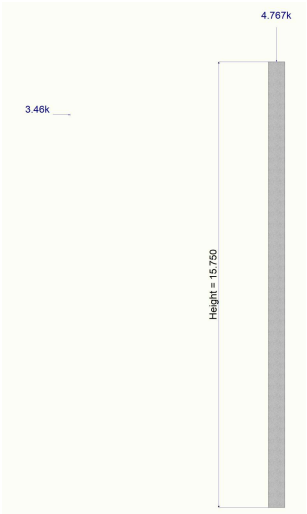
Steel Column

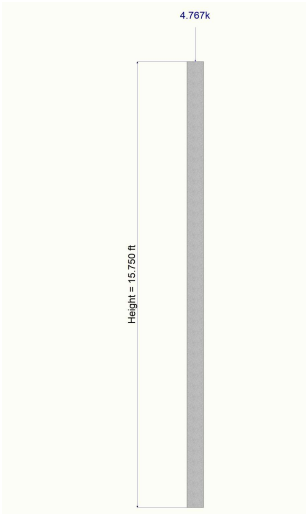
LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: 15.75 ft cantilever column

Sketches







Steel Column

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: check 15% axial_15.75 ft cantilever column

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, SDPWS 2015
Load Combinations Used : ASCE 7-22 / IBC 2024 (L<=100psf)

General Information

Steel Section Name : **HSS7x7x1/2**

Analysis Method :	Allowable Strength
Steel Stress Grade	
Fy : Steel Yield	46.0 ksi
E : Elastic Bending Modulus	29,000.0 ksi

Overall Column Height	15.750 ft
Top & Bottom Fixity	Top Free, Bottom Fixed
Brace condition :	

Unbraced Length for buckling ABOUT X-X Axis = 12 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 12 ft, K = 2.1

Applied Loads

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

Column self weight included : 662.29 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 15.750 ft, D = 2.509, L = 1.568, E = 0.690 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = 0.03610 : 1

Load Combination	+D+L
Location of max.above base	0.0 ft
At maximum location values are . . .	
Pa : Axial	4.739 k
Pn / Omega : Allowabl	131.302 k
Ma-x : Applied	0.0 k-ft
Mn-x / Omega : Allowable	64.042 k-ft
Ma-y : Applied	0.0 k-ft
Mn-y / Omega : Allowable	64.042 k-ft

PASS Maximum Shear Stress Rati 0.0 : 1

Load Combination	0.0
Location of max.above base	0.0 ft
At maximum location values are . . .	
Va : Applied	0.0 k
Vn / Omega : Allowable	0.0 k

Maximum Load Reactions . .

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

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination :

Along X-X 0.0 in at 0.0 ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.024	PASS	0.00 ft	1.00	1.00	114.98	114.98	0.000	PASS	0.00 ft
+D+L	0.036	PASS	0.00 ft	1.00	1.00	114.98	114.98	0.000	PASS	0.00 ft
+D+0.750L	0.033	PASS	0.00 ft	1.00	1.00	114.98	114.98	0.000	PASS	0.00 ft
+0.60D	0.014	PASS	0.00 ft	1.00	1.00	114.98	114.98	0.000	PASS	0.00 ft
+D+0.70E	0.028	PASS	0.00 ft	1.00	1.00	114.98	114.98	0.000	PASS	0.00 ft
+D+0.750L+0.5250E	0.036	PASS	0.00 ft	1.00	1.00	114.98	114.98	0.000	PASS	0.00 ft
+0.60D+0.70E	0.018	PASS	0.00 ft	1.00	1.00	114.98	114.98	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	3.171									
+D+L	4.739									
+D+0.750L	4.347									
+0.60D	1.903									
+D+0.70E	3.654									
+D+0.750L+0.5250E	4.710									
+0.60D+0.70E	2.386									
L Only	1.568									
E Only	0.690									

Steel Column

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: check 15% axial_15.75 ft cantilever column

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	4.739								
"	Minimum	0.690								
Reaction, X-X Axis Base	Maximum	3.171								
"	Minimum	3.171								
Reaction, Y-Y Axis Base	Maximum	3.171								
"	Minimum	3.171								
Reaction, X-X Axis Top	Maximum	3.171								
"	Minimum	3.171								
Reaction, Y-Y Axis Top	Maximum	3.171								
"	Minimum	3.171								
Moment, X-X Axis Base	Maximum	3.171								
"	Minimum	3.171								
Moment, Y-Y Axis Base	Maximum	3.171								
"	Minimum	3.171								
Moment, X-X Axis Top	Maximum	3.171								
"	Minimum	3.171								
Moment, Y-Y Axis Top	Maximum	3.171								
"	Minimum	3.171								

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.5250E	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : HSS7x7x1/2

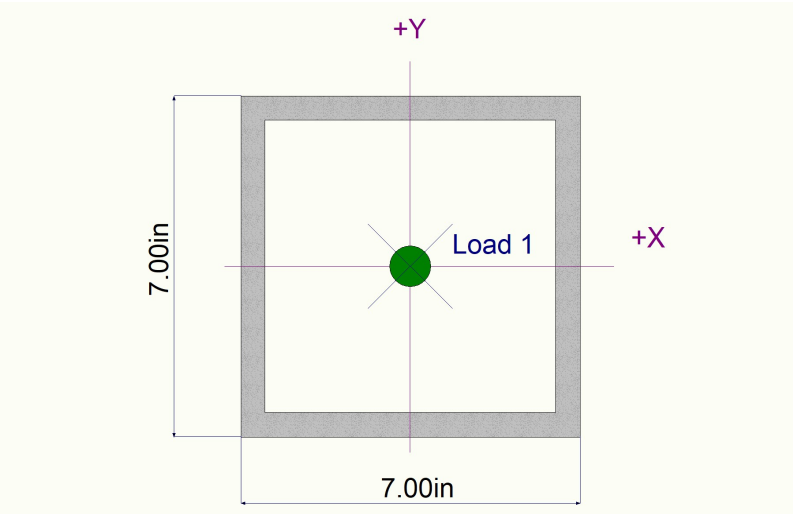
Depth	=	7.000 in	I xx	=	80.50 in^4	J	=	133.000 in^4
Design Thick	=	0.465 in	S xx	=	23.00 in^3			
Width	=	7.000 in	R xx	=	2.630 in			
Wall Thick	=	0.500 in	Zx	=	27.900 in^3			
Area	=	11.600 in^2	I yy	=	80.500 in^4	C	=	39.300 in^3
Weight	=	42.050 plf	S yy	=	23.000 in^3			
			R yy	=	2.630 in			
Ycg	=	0.000 in						

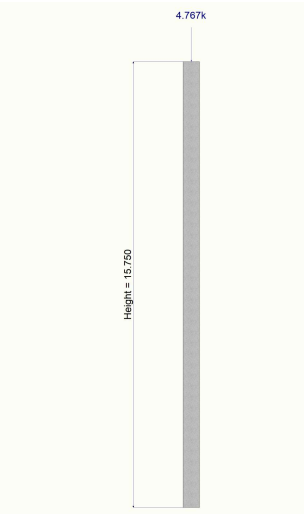
Steel Column

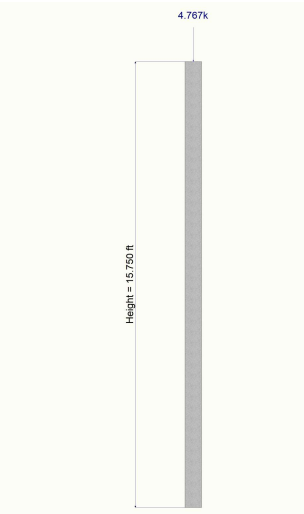
LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: check 15% axial_15.75 ft cantilever column

Sketches







Pole Footing Embedded in Soil

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: 15.75ft CANT. COL. pole foundation

Code References

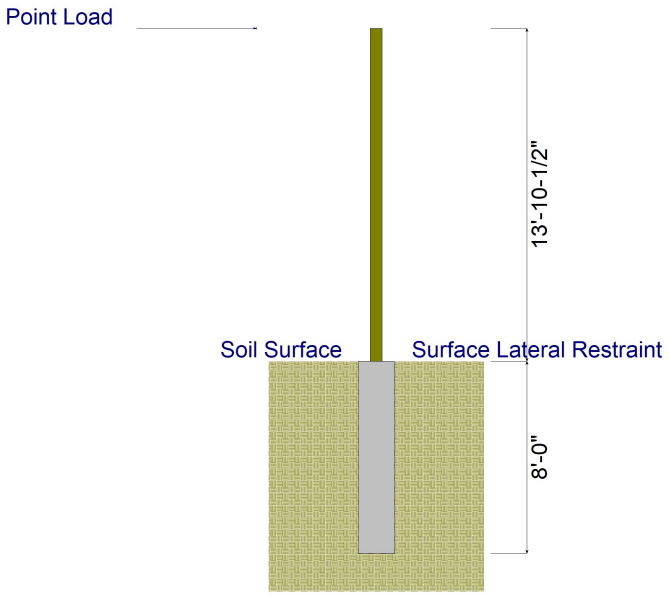
Calculations per IBC 2018 1807.3, CBC 2019, SDPWS 2015
Load Combinations Used : ASCE 7-22 / IBC 2024 (L<=100psf)

General Information

Pole Footing Shape	Circular
Pole Footing Diameter	18.0 in
Calculate Min. Depth for Allowable Pressures	
Lateral Restraint at Ground Surface	
Allow Passive	250.0 pcf
Max Passive	1,500.0 psf

Controlling Values

Governing Load Combinatio	+D+0.70E
Lateral Load	2.419 k
Moment	33.557 k-ft
Restraint @ Ground Surface	
Pressure at Depth	
Actual	**1,485.58** psf
Allowable	**1,500.0** psf
Surface Retraint Force	11,332.0 lbs
Minimum Required Depth	**8.0 ft**
Footing Base Area	1.767 ft^2
Maximum Soil Pressure	2.307 ksf



Applied Loads

Lateral Concentrated Load (k)		Lateral Distributed Loads (k		Applied Moment (kft)	Vertical Load (k)
D : Dead Load	k		k/ft	k-ft	2.509 k
Lr : Roof Live	k		k/ft	k-ft	k
L : Live	k		k/ft	k-ft	1.568 k
S : Snow	k		k/ft	k-ft	k
W : Wind	k		k/ft	k-ft	k
E : Earthquake	3.455 k		k/ft	k-ft	0.690 k
H : Lateral Earth	k		k/ft	k-ft	k
Load distance above ground surface	13.875 ft	TOP of Load above ground surface	ft		
		BOTTOM of Load above ground surface	ft		

Load Combination Results

	Forces @ Ground Surface		Required	Pressure at Depth		Soil Increase
Load Combination	Loads - (k)	Moments - (ft-k)	Depth - (ft)	Actual - (psf)	Allow - (psf)	Factor
D Only	0.000	0.000	0.13	0.0	31.3	1.000
+D+L	0.000	0.000	0.13	0.0	31.3	1.000
+D+0.750L	0.000	0.000	0.13	0.0	31.3	1.000
+0.60D	0.000	0.000	0.13	0.0	31.3	1.000
+D+0.70E	2.419	33.557	8.00	1,485.6	1,500.0	1.000
+D+0.750L+0.5250E	1.814	25.168	7.00	1,455.3	1,500.0	1.000
+0.60D+0.70E	2.419	33.557	8.00	1,485.6	1,500.0	1.000

Pole Footing Embedded in Soil

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: w/ overstrength factor_15.75ft CANT. COL. pole foundation

Code References

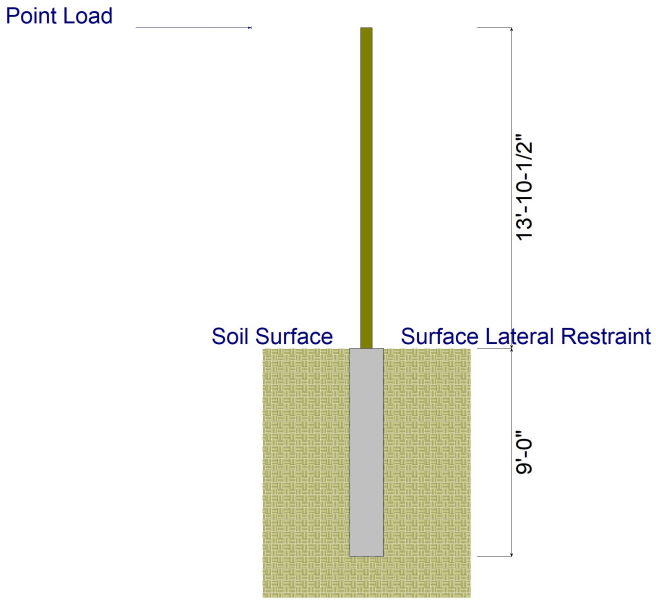
Calculations per IBC 2018 1807.3, CBC 2019, SDPWS 2015
Load Combinations Used : ASCE 7-22 / IBC 2024 (L<=100psf)

General Information

Pole Footing Shape	Circular
Pole Footing Diameter	18.0 in
Calculate Min. Depth for Allowable Pressures	
Lateral Restraint at Ground Surface	
Allow Passive	250.0 pcf
Max Passive	1,500.0 psf

Controlling Values

Governing Load Combinatio	+D+0.70E
Lateral Load	3.023 k
Moment	41.948 k-ft
Restraint @ Ground Surface	
Pressure at Depth	
Actual	**1,467.33** psf
Allowable	**1,500.0** psf
Surface Retraint Force	12,927.8 lbs
Minimum Required Depth	**9.0 ft**
Footing Base Area	1.767 ft^2
Maximum Soil Pressure	2.307 ksf



Applied Loads

Lateral Concentrated Load (k)		Lateral Distributed Loads (k		Applied Moment (kft)	Vertical Load (k)
D : Dead Load	k		k/ft	k-ft	2.509 k
Lr : Roof Live	k		k/ft	k-ft	k
L : Live	k		k/ft	k-ft	1.568 k
S : Snow	k		k/ft	k-ft	k
W : Wind	k		k/ft	k-ft	k
E : Earthquake	4.319 k		k/ft	k-ft	0.690 k
H : Lateral Earth	k		k/ft	k-ft	k
Load distance above ground surface	13.875 ft	TOP of Load above ground surface	ft		
		BOTTOM of Load above ground surface	ft		

Load Combination Results

	Forces @ Ground Surface		Required	Pressure at Depth		Soil Increase
Load Combination	Loads - (k)	Moments - (ft-k)	Depth - (ft)	Actual - (psf)	Allow - (psf)	Factor
D Only	0.000	0.000	0.13	0.0	31.3	1.000
+D+L	0.000	0.000	0.13	0.0	31.3	1.000
+D+0.750L	0.000	0.000	0.13	0.0	31.3	1.000
+0.60D	0.000	0.000	0.13	0.0	31.3	1.000
+D+0.70E	3.023	41.948	9.00	1,467.3	1,500.0	1.000
+D+0.750L+0.5250E	2.267	31.461	7.75	1,484.1	1,500.0	1.000
+0.60D+0.70E	3.023	41.948	9.00	1,467.3	1,500.0	1.000

Steel Beam

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

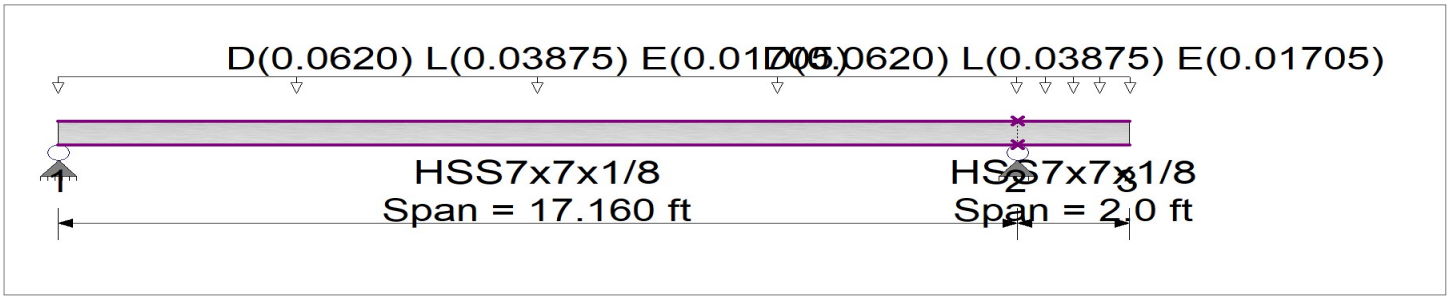
DESCRIPTION: 17'-2" BEAM w/ 2'-0" CANT.

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-22 / IBC 2024 (L<=100psf)

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : D = 0.0080, L = 0.0050, E = 0.00220 ksf, Tributary Width = 7.750 ft, (Ceiling framing)
Load for Span Number 2
Uniform Load : D = 0.0080, L = 0.0050, E = 0.00220 ksf, Tributary Width = 7.750 ft, (Ceiling framing)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	: 1	Maximum Shear Stress Ratio =	: 1
Section used for this span		Section used for this span	
Ma : Applied	k-ft	Va : Applied	k
Mn / Omega : Allowable	k-ft	Vn/Omega : Allowable	k
Load Combination		Load Combination	
		Location of maximum on span	ft
Span # where maximum occurs		Span # where maximum occurs	

Maximum Deflection

Max Downward Transient Deflection	in	Ratio =	<360
Max Upward Transient Deflection	in	Ratio =	<360
Max Downward Total Deflection	in	Ratio =	<180
Max Upward Total Deflection	in	Ratio =	<180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values					Summary of Shear Values	
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx Mnx/Omega Cb	Rm	Va Max	VnxVnx/Omega

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3

Steel Beam

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

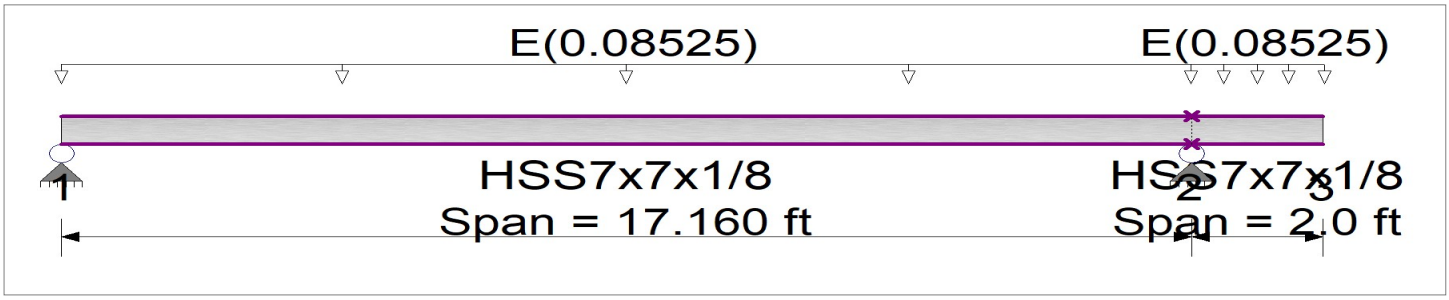
DESCRIPTION: 17'-2" BEAM w/ 2'-0" CANT._seismic only

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-22 / IBC 2024 (L<=100psf)

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : E = 0.0110 ksf, Tributary Width = 7.750 ft, (Ceiling framing)
Load for Span Number 2
Uniform Load : E = 0.0110 ksf, Tributary Width = 7.750 ft, (Ceiling framing)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.167 : 1	Maximum Shear Stress Ratio =	0.020 : 1
Section used for this span	HSS7x7x1/8	Section used for this span	HSS7x7x1/8
Ma : Applied	2.137 k-ft	Va : Applied	0.5190 k
Mn / Omega : Allowable	12.806 k-ft	Vn/Omega : Allowable	25.505 k
Load Combination	E Only * 0.70	Load Combination	E Only * 0.70
		Location of maximum on span	17.160 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.225 in	Ratio =	914	>=360	Span: 2 : E Only
Max Upward Transient Deflection	-0.081 in	Ratio =	591	>=360	Span: 2 : E Only
Max Downward Total Deflection	0.158 in	Ratio =	1306	>=180	Span: 2 : E Only * 0.70
Max Upward Total Deflection	-0.057 in	Ratio =	845	>=180	Span: 2 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx Mnx/Omega Cb	Rm	Summary of Shear Values Va Max	VnxVnx/Omega
Dsgn. L = 17.16 ft	1		0.000				21.39 12.81	1.00 1.00	-0.00	42.59 25.51
Dsgn. L = 2.00 ft	2		0.000				21.39 12.81	1.00 1.00	-0.00	42.59 25.51
E Only * 0.70										
Dsgn. L = 17.16 ft	1	0.167	0.020	2.14	-0.12	2.14	21.39 12.81	1.00 1.00	0.52	42.59 25.51
Dsgn. L = 2.00 ft	2	0.009	0.005		-0.12	0.12	21.39 12.81	1.00 1.00	0.12	42.59 25.51
E Only * 0.5250										
Dsgn. L = 17.16 ft	1	0.125	0.015	1.60	-0.09	1.60	21.39 12.81	1.00 1.00	0.39	42.59 25.51
Dsgn. L = 2.00 ft	2	0.007	0.004		-0.09	0.09	21.39 12.81	1.00 1.00	0.09	42.59 25.51

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	0.722	0.912	
Max Upward from Load Cases	0.722	0.912	
E Only * 0.70	0.505	0.638	
E Only * 0.5250	0.379	0.479	
E Only	0.722	0.912	

Steel Beam

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

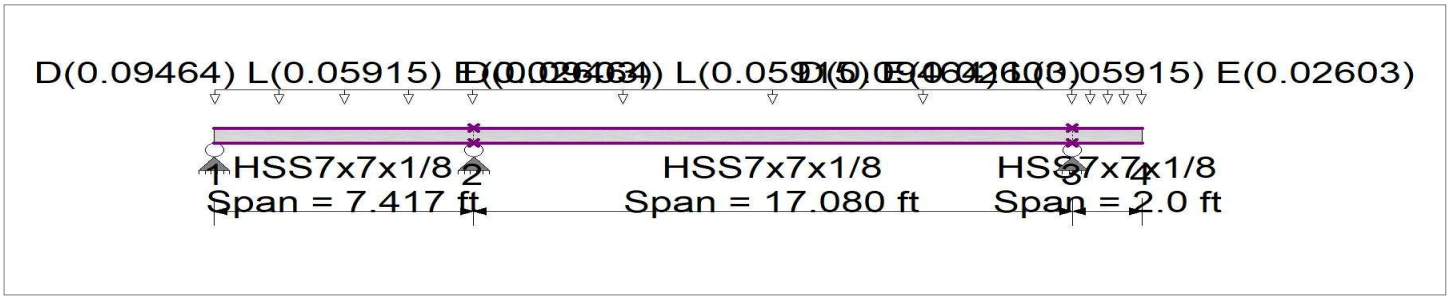
DESCRIPTION: 7'-5"& 17'-2" BEAM w 2'-0 CANT

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-22 / IBC 2024 (L<=100psf)

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : D = 0.0080, L = 0.0050, E = 0.00220 ksf, Tributary Width = 11.830 ft, (Ceiling framing)
Load for Span Number 2
Uniform Load : D = 0.0080, L = 0.0050, E = 0.00220 ksf, Tributary Width = 11.830 ft, (Ceiling framing)
Load for Span Number 3
Uniform Load : D = 0.0080, L = 0.0050, E = 0.00220 ksf, Tributary Width = 11.830 ft, (Ceiling framing)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.322 : 1	Maximum Shear Stress Ratio =	0.060 : 1
Section used for this span	HSS7x7x1/8	Section used for this span	HSS7x7x1/8
Ma : Applied	4.123 k-ft	Va : Applied	1.537 k
Mn / Omega : Allowable	12.806 k-ft	Vn/Omega : Allowable	25.505 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	7.417 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.084 in	Ratio =	2,433	>=360	Span: 3 : L Only
Max Upward Transient Deflection	-0.034 in	Ratio =	1,420	>=360	Span: 3 : L Only
Max Downward Total Deflection	0.219 in	Ratio =	936	>=180	Span: 3 : +D+L
Max Upward Total Deflection	-0.088 in	Ratio =	546	>=180	Span: 3 : +D+L

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 7.42 ft	1	0.198	0.037	0.00	-2.54	2.54	21.39	12.81	1.00	1.00	0.95	42.59	25.51
Dsgn. L = 17.08 ft	2	0.198	0.037	2.19	-2.54	2.54	21.39	12.81	1.00	1.00	0.95	42.59	25.51
Dsgn. L = 2.00 ft	3	0.015	0.007		-0.19	0.19	21.39	12.81	1.00	1.00	0.19	42.59	25.51
+D+L													
Dsgn. L = 7.42 ft	1	0.322	0.060	0.00	-4.12	4.12	21.39	12.81	1.00	1.00	1.54	42.59	25.51
Dsgn. L = 17.08 ft	2	0.322	0.060	3.55	-4.12	4.12	21.39	12.81	1.00	1.00	1.54	42.59	25.51
Dsgn. L = 2.00 ft	3	0.024	0.012		-0.31	0.31	21.39	12.81	1.00	1.00	0.31	42.59	25.51
+D+0.750L													
Dsgn. L = 7.42 ft	1	0.291	0.054	0.00	-3.73	3.73	21.39	12.81	1.00	1.00	1.39	42.59	25.51
Dsgn. L = 17.08 ft	2	0.291	0.054	3.21	-3.73	3.73	21.39	12.81	1.00	1.00	1.39	42.59	25.51
Dsgn. L = 2.00 ft	3	0.022	0.011		-0.28	0.28	21.39	12.81	1.00	1.00	0.28	42.59	25.51
+0.60D													
Dsgn. L = 7.42 ft	1	0.119	0.022	0.00	-1.52	1.52	21.39	12.81	1.00	1.00	0.57	42.59	25.51
Dsgn. L = 17.08 ft	2	0.119	0.022	1.31	-1.52	1.52	21.39	12.81	1.00	1.00	0.57	42.59	25.51
Dsgn. L = 2.00 ft	3	0.009	0.004		-0.11	0.11	21.39	12.81	1.00	1.00	0.11	42.59	25.51
+D+0.70E													
Dsgn. L = 7.42 ft	1	0.236	0.044	0.00	-3.03	3.03	21.39	12.81	1.00	1.00	1.13	42.59	25.51
Dsgn. L = 17.08 ft	2	0.236	0.044	2.61	-3.03	3.03	21.39	12.81	1.00	1.00	1.13	42.59	25.51

Steel Beam

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: 7'-5"& 17'-2" BEAM w 2'-0 CANT

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	VnxVnx/Omega
Dsgn. L = 2.00 ft	3	0.018	0.009		-0.23	0.23	21.39	12.81	1.00	1.00	0.23	42.59	25.51
+D+0.750L+0.5250E													
Dsgn. L = 7.42 ft	1	0.320	0.060	0.00	-4.09	4.09	21.39	12.81	1.00	1.00	1.53	42.59	25.51
Dsgn. L = 17.08 ft	2	0.320	0.060	3.53	-4.09	4.09	21.39	12.81	1.00	1.00	1.53	42.59	25.51
Dsgn. L = 2.00 ft	3	0.024	0.012		-0.31	0.31	21.39	12.81	1.00	1.00	0.31	42.59	25.51
+0.60D+0.70E													
Dsgn. L = 7.42 ft	1	0.157	0.029	0.00	-2.01	2.01	21.39	12.81	1.00	1.00	0.75	42.59	25.51
Dsgn. L = 17.08 ft	2	0.157	0.029	1.73	-2.01	2.01	21.39	12.81	1.00	1.00	0.75	42.59	25.51
Dsgn. L = 2.00 ft	3	0.012	0.006		-0.15	0.15	21.39	12.81	1.00	1.00	0.15	42.59	25.51

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions	0.014	2.663	1.398	
Max Upward from Load Combinations	0.014	2.663	1.398	
Max Upward from Load Cases	0.009	1.639	0.860	
D Only	0.009	1.639	0.860	
+D+L	0.014	2.663	1.398	
+D+0.750L	0.013	2.407	1.263	
+0.60D	0.005	0.983	0.516	
+D+0.70E	0.011	1.954	1.026	
+D+0.750L+0.5250E	0.014	2.644	1.387	
+0.60D+0.70E	0.007	1.299	0.682	
L Only	0.006	1.024	0.538	
E Only	0.002	0.451	0.237	

Steel Beam

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

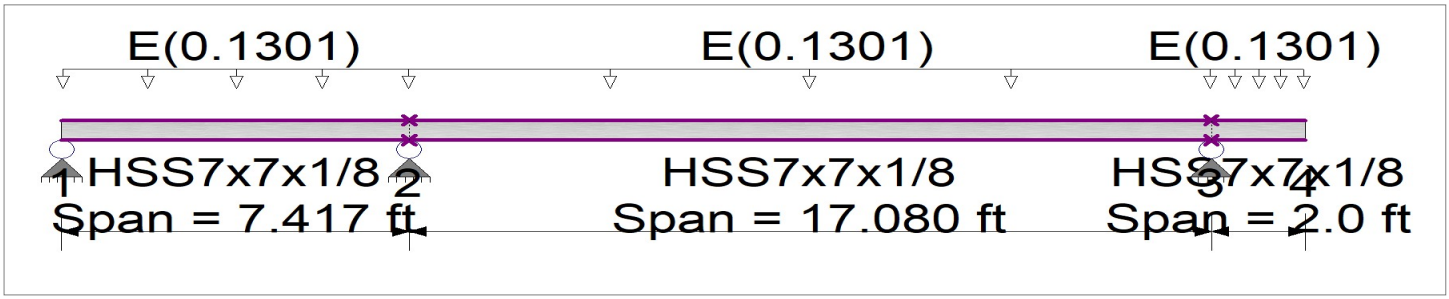
DESCRIPTION: 7'-5"& 17'-2" BEAM w 2'-0 CANT_seismic only

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-22 / IBC 2024 (L<=100psf)

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : E = 0.0110 ksf, Tributary Width = 11.830 ft, (Ceiling framing)
Load for Span Number 2
Uniform Load : E = 0.0110 ksf, Tributary Width = 11.830 ft, (Ceiling framing)
Load for Span Number 3
Uniform Load : E = 0.0110 ksf, Tributary Width = 11.830 ft, (Ceiling framing)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.191 : 1	Maximum Shear Stress Ratio =	0.036 : 1
Section used for this span	HSS7x7x1/8	Section used for this span	HSS7x7x1/8
Ma : Applied	2.442 k-ft	Va : Applied	0.9102 k
Mn / Omega : Allowable	12.806 k-ft	Vn/Omega : Allowable	25.505 k
Load Combination	E Only * 0.70	Load Combination	E Only * 0.70
		Location of maximum on span	7.417 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.185 in	Ratio =	1,106	>=360	Span: 3 : E Only
Max Upward Transient Deflection	-0.074 in	Ratio =	645	>=360	Span: 3 : E Only
Max Downward Total Deflection	0.130 in	Ratio =	1580	>=180	Span: 3 : E Only * 0.70
Max Upward Total Deflection	-0.052 in	Ratio =	922	>=180	Span: 3 : E Only * 0.70

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	VnxVnx/Omega	
Dsgn. L = 7.42 ft	1		0.000				21.39	12.81	1.00	1.00	-0.00	42.59	25.51
Dsgn. L = 17.08 ft	2		0.000				21.39	12.81	1.00	1.00	-0.00	42.59	25.51
Dsgn. L = 2.00 ft	3		0.000				21.39	12.81	1.00	1.00	-0.00	42.59	25.51
E Only * 0.70													
Dsgn. L = 7.42 ft	1	0.191	0.036	0.00	-2.44	2.44	21.39	12.81	1.00	1.00	0.91	42.59	25.51
Dsgn. L = 17.08 ft	2	0.191	0.036	2.11	-2.44	2.44	21.39	12.81	1.00	1.00	0.91	42.59	25.51
Dsgn. L = 2.00 ft	3	0.014	0.007		-0.18	0.18	21.39	12.81	1.00	1.00	0.18	42.59	25.51
E Only * 0.5250													
Dsgn. L = 7.42 ft	1	0.143	0.027	0.00	-1.83	1.83	21.39	12.81	1.00	1.00	0.68	42.59	25.51
Dsgn. L = 17.08 ft	2	0.143	0.027	1.58	-1.83	1.83	21.39	12.81	1.00	1.00	0.68	42.59	25.51
Dsgn. L = 2.00 ft	3	0.011	0.005		-0.14	0.14	21.39	12.81	1.00	1.00	0.14	42.59	25.51

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions	0.012	2.253	1.183	
Max Upward from Load Cases	0.012	2.253	1.183	
E Only * 0.70	0.009	1.577	0.828	
E Only * 0.5250	0.006	1.183	0.621	
E Only	0.012	2.253	1.183	

Steel Beam

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

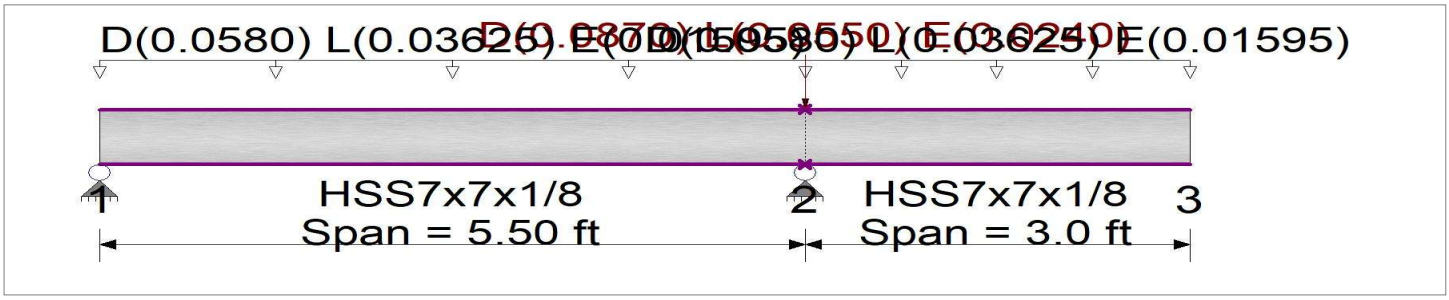
DESCRIPTION: [double roof] 5'- 1/2" BEAM w/ 3'-0" CANT.

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-22 / IBC 2024 (L<=100psf)

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : D = 0.0080, L = 0.0050, E = 0.00220 ksf, Tributary Width = 7.250 ft, (Ceiling framing)
Load for Span Number 2
Uniform Load : D = 0.0080, L = 0.0050, E = 0.00220 ksf, Tributary Width = 7.250 ft, (Ceiling framing)
Point Load : D = 0.0870, L = 0.0550, E = 0.0240 k @ 0.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.033 : 1	Maximum Shear Stress Ratio =	0.013 : 1
Section used for this span	HSS7x7x1/8	Section used for this span	HSS7x7x1/8
Ma : Applied	0.424 k-ft	Va : Applied	0.3363 k
Mn / Omega : Allowable	12.806 k-ft	Vn/Omega : Allowable	25.505 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	5.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.001 in	Ratio =	58,832	>=360	Span: 2 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.003 in	Ratio =	22628	>=180	Span: 2 : +D+L
Max Upward Total Deflection	-0.000 in	Ratio =		<180	Span: 2 : +D+L

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	VnxVnx/Omega
D Only												
Dsgn. L = 5.50 ft	1	0.020	0.008	0.11	-0.26	0.26	21.39	12.81	1.00	1.00	0.21	42.59 25.51
Dsgn. L = 3.00 ft	2	0.020	0.007		-0.26	0.26	21.39	12.81	1.00	1.00	0.17	42.59 25.51
+D+L												
Dsgn. L = 5.50 ft	1	0.033	0.013	0.18	-0.42	0.42	21.39	12.81	1.00	1.00	0.34	42.59 25.51
Dsgn. L = 3.00 ft	2	0.033	0.011		-0.42	0.42	21.39	12.81	1.00	1.00	0.28	42.59 25.51
+D+0.750L												
Dsgn. L = 5.50 ft	1	0.030	0.012	0.16	-0.38	0.38	21.39	12.81	1.00	1.00	0.30	42.59 25.51
Dsgn. L = 3.00 ft	2	0.030	0.010		-0.38	0.38	21.39	12.81	1.00	1.00	0.26	42.59 25.51
+0.60D												
Dsgn. L = 5.50 ft	1	0.012	0.005	0.06	-0.16	0.16	21.39	12.81	1.00	1.00	0.12	42.59 25.51
Dsgn. L = 3.00 ft	2	0.012	0.004		-0.16	0.16	21.39	12.81	1.00	1.00	0.10	42.59 25.51
+D+0.70E												
Dsgn. L = 5.50 ft	1	0.024	0.010	0.13	-0.31	0.31	21.39	12.81	1.00	1.00	0.25	42.59 25.51
Dsgn. L = 3.00 ft	2	0.024	0.008		-0.31	0.31	21.39	12.81	1.00	1.00	0.21	42.59 25.51
+D+0.750L+0.5250E												
Dsgn. L = 5.50 ft	1	0.033	0.013	0.17	-0.42	0.42	21.39	12.81	1.00	1.00	0.33	42.59 25.51
Dsgn. L = 3.00 ft	2	0.033	0.011		-0.42	0.42	21.39	12.81	1.00	1.00	0.28	42.59 25.51
+0.60D+0.70E												
Dsgn. L = 5.50 ft	1	0.016	0.006	0.09	-0.21	0.21	21.39	12.81	1.00	1.00	0.16	42.59 25.51

Steel Beam

Project File: task2.ec6

LIC# : KW-06018039, Build:20.24.09.03 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: [double roof] 5'- 1/2" BEAM w/ 3'-0" CANT.

Maximum Forces & Stresses for Load Combinations

Load Combination			Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length		Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	VnxVnx/Omega	
Dsgn. L =	3.00 ft	2	0.016	0.005		-0.21	0.21	21.39	12.81	1.00	1.00	0.14	42.59	25.51

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	0.182	0.761	
Max Upward from Load Combinations	0.182	0.761	
Max Upward from Load Cases	0.112	0.468	
D Only	0.112	0.468	
+D+L	0.182	0.761	
+D+0.750L	0.165	0.688	
+0.60D	0.067	0.281	
+D+0.70E	0.134	0.558	
+D+0.750L+0.5250E	0.181	0.755	
+0.60D+0.70E	0.089	0.371	
L Only	0.070	0.293	
E Only	0.031	0.129	

STRUCTURAL CALCULATIONS

FOR

BUILDING IMPROVEMENT FOR CORONA SOUTH AMLL BUILDING "B" TI



DATE STAMPED 10-14-2026

BY

IDS GROUP
1 PETERS CANYON, SUITE 130
IRVINE, CA 92606

OCTOBER 14, 2026

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IDS GROUP
1 Peters Canyon Road, Suite 130
Irvine, CA 92606
(949) 387-8500, Fax: (949) 387-0800

Sheet:	1		
Job No.:	24X084.01		
Calc. By.:		Date:	8/31/2026
Chck'd By.:		Date:	

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DESIGN CRITERIA	3
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(E) BEAM DESIGN	16

DESIGN CRITERIA

the said table). Please note that, as S_1 value is greater than 0.2, should exception per Item 1 in Section 11.4.8 of ASCE 7-16, Supplement 3 be applied, a 50% increase on the S_{M1} value derived from Equation 11.4-2 of ASCE 7-16 (or Equation 16-21 of 2022 CBC) is required. The increased S_{M1} value should then serve as the basis for the derivation of S_{D1} value for structural design. Please note that the S_{M1} and S_{D1} values listed in the table below do not reflect such 50% increase. If the structural design of the Remodel & Addition cannot be supported by the invoked exceptions, the Geotechnical Consultant should be contacted for performing additional, site-specific seismic hazard analysis such that values of site-specific design parameters could be established.

2022 CBC SEISMIC DESIGN PARAMETERS					
Site Latitude:	N 33.8740°	**Site Longitude:**	W 117.5658°	**Risk Category** [a]	II
Seismic Parameter				**Recommended Value**	
Site Class [b]				D	
Soil Profile Name [b]				Stiff Soil Profile	
Site Coefficient, Fa [c]				1.0	
Site Coefficient, Fv [d]				1.7	
0.2-Second Spectral Response Acceleration, S_S [e]				2.065g	
1.0-Second Spectral Response Acceleration, S_1 [f]				0.783g	
Adjusted 0.2-Second Spectral Response Acceleration, S_{MS} [g]				2.065g	
Adjusted 1.0-Second Spectral Response Acceleration, S_{M1} [h]				1.331g	
Design 0.2-Second Spectral Response Acceleration, S_{DS} [i]				1.376g	
Design 1.0-Second Spectral Response Acceleration, S_{D1} [j]				0.887g	
Long -Period Transition Period, T_L [k]				8 sec	
Mapped MCE_G Geometric Mean Peak Ground Acceleration, PGA [l]				0.866g	
Site Coefficient, F_{PGA} [m]				1.1	
MCE_G Peak Ground Acceleration adjusted for Site Class Effect, PGA_M [n]				0.953g	
Risk Category			**I or II or III**		**IV**
Seismic Design Category based on S_1 [o]			E		F
Seismic Design Category based on SD_S [p]			N/A		N/A
Seismic Design Category based on SD_1 [q]			N/A		N/A

a Per 2022 CBC Table 1604.5
b Per 2022 CBC Section 1613.2.2
c Per 2022 CBC Table 1613.2.3(1). *Note: If simplified design procedure of Section 12.14 of ASCE 7-16 is adopted, the Fa value should be determined per Section 12.14.8.1 of ASCE 7-16 with no need for F_v, S_{MS}, S_{M1} values.*
d Per 2022 CBC Table 1613.2.3(2), *provided C_s values are determined by Equations 12.8-2, 12.8-3 and 12.8-4 of ASCE 7-16.*
e Per 2022 CBC Figure 1613.2.1(1)
f Per 2022 CBC Figure 1613.2.1(2)
g Per 2022 CBC Equation 16-20
h Per 2022 CBC Equation 16-21
i Per 2022 CBC Equation 16-22
j Per 2022 CBC Equation 16-23
k Per ASCE 7-16 Figure 22-14
l Per ASCE 7-16 Figure 22-9
m Per ASCE 7-16 Table 11.8-1
n Per ASCE 7-16 Equation 11.8-1 = PGA x F_{PGA}
o Per 2022 CBC Section 1613.2.5
p Per 2022 CBC Table 1613.2.5(1)
q Per 2022 CBC Table 1613.2.5(2)

Additionally, the seismic design criteria for the Site for BSE-2N, BSE-1N, BSE-2E, and BSE-1E Hazard Levels, have been analyzed utilizing the "ASCE Hazard Tool" web-application (https://ascehazardtool.org/) per the criteria of Basic Performance Objective Equivalent to New Building Standards (BPON) and Basic

IDS GROUP
Structural Engineers
1 Peters Canyon Road, Suite 140

Sheet:	
Job No.: 24x084.01	
Calc. By.: EW	Date: 11-25-2025
Chck'd By.:	Date:

Unit Weights

Unit weight of 8" thick brick masonry wall =	75	psf
Unit weight of 8" thick masonry wall =	84	psf
Unit weight of partition =	10	psf
Unit weight of glazing =	10	psf

Roof Dead Load - Building

Roofing =	2	psf
Insulation =	2	psf
Plywood sheathing =	1.5	psf
Framing	5	psf
Fire sprinkler =	2	psf
Ceiling =	1	psf
MEP =	2	psf
Misc. =	0.5	psf
	16	psf

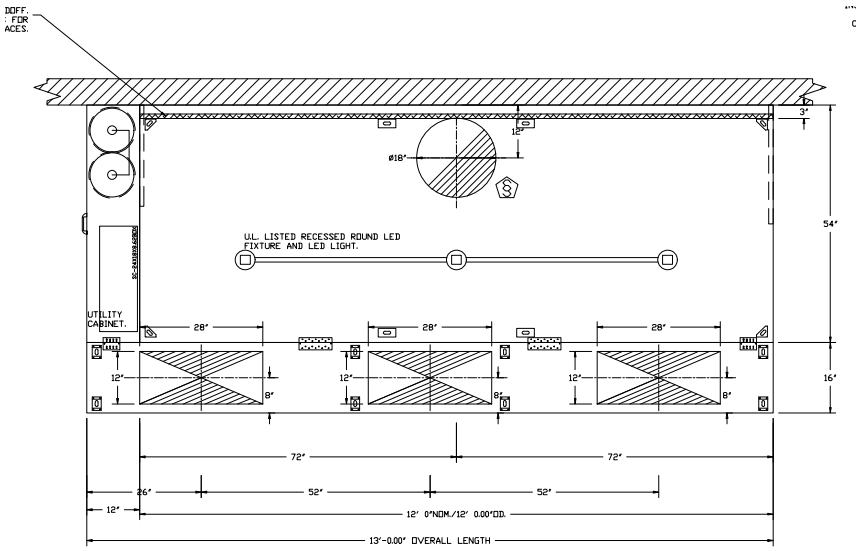
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Job No.:
Date:
Engineer:

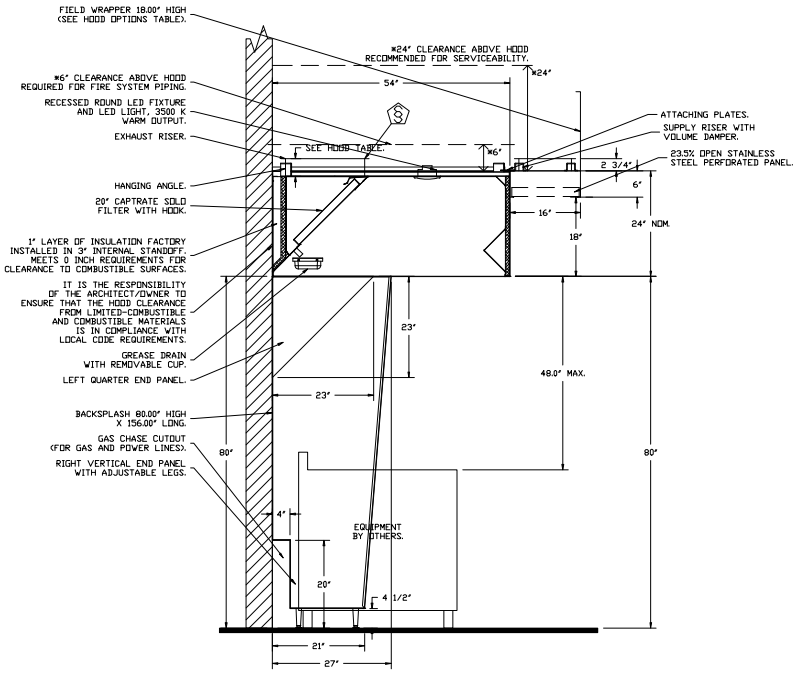
HOOD ANCHORAGE

IDS GROUP

Job No.:
Date:
Engineer:



PLAN VIEW – HOOD #1 (119/119A)
12' 0.00" LONG 5424ND-2-PSP-F
NOTE: ADDITIONAL HANGING ANGLES PROVIDED FOR HOODS 12' AND LONGER.



SECTION VIEW – MODEL 5424ND-2-PSP-F
HOOD – #1 (119/119A)

HOOD CUTSHEET

IDS GROUP

Job No.: 24X084.01
Date: 10/14/26
Engineer: DD

ASCE 7-22 CH. 13 HOOD CALCULATION

SEISMIC DESIGN PARAMETERS (CBC 2025 / ASCE 7-22)		
SDS — design spectral accel., short period	1.38	g
Ip — component importance factor	0.89	
z — attachment height above base	1.00	ft
h — average roof height of structure	1.00	ft
z/h (≤ 1.0)	1.00	
R — building SFRS response mod. coeff.	2.00	
Ω0 — building SFRS overstrength	2.50	
CAR — component resonance factor	1.40	
Rpo — component strength factor	2.00	
Ω0p — component overstrength	2.00	
Seismic Design Category	D	

COMPONENT / HANGER RODS		
Wp — component operating weight	940	lb
N — number of hanger rods	8	
Hanger rod diameter	0.500	in
Fy (ksi)	36	ksi
Fu (ksi)	60	ksi
Rod length (attachment to hood)	40.0	in
e — vert. distance, brace plane to hood C.G.	12.0	in
d — rod spacing (overturning couple arm)	54.0	in

OPTION B — RIGID TENSION/COMPRESSION STRUT BRACING		
θb — strut angle from horizontal	60	deg
φb — strut plan-skew angle	15	deg
nb — struts effective per direction	2	
Strut section	1-1/4 in Sch 40 pipe (A53 Gr B)	
A — strut area	0.669	in²
r — strut radius of gyration	0.540	in
Fy — strut yield	35	ksi
E — modulus	29,000	ksi
K — effective length factor	1.0	
Lb — strut length	40.0	in

SEISMIC DESIGN FORCE — ASCE 7-22 §13.3.1 (CBC 2025)

HORIZONTAL SEISMIC DESIGN FORCE Fp		
Hf = 1 + 2.5·(z/h)	3.50	
Rμ = √(1.1·R/(Ip·Ω0)) ≥ 1.3	1.30	
Fp (Eq. 13.3-1)	865	lb
Fp,max = 1.6·SDS·Ip·Wp (Eq. 13.3-2)	1,836	lb
Fp,min = 0.3·SDS·Ip·Wp (Eq. 13.3-3)	344	lb
Fp DESIGN (governing, strength level) 100%+30%	1,124	lb
Ev = 0.2·SDS·Wp (vertical, strength level)	259	lb

ASD DEMANDS (ASCE 7-22 §2.4.5 — 0.7·E)		
Eh,ASD = 0.7·Fp (horizontal)	787	lb
Combo 8: (1.0 + 0.14·SDS)·Wp (gravity + Ev↓)	1,121	lb
Combo 10: (0.6 − 0.14·SDS)·Wp (uplift case)	383	lb

IDS GROUP

Job No.: 24X084.01
Date: 10/14/26
Engineer: DD

ASCE 7-22 CH. 13 HOOD SUPPORT CALCULATION

STRUT DESIGN FORCE (ASD)		
Fh — horizontal seismic (ASD)	787	lb
P — strut axial (T or C) = Fh/(nb·cosθb·cosφb)	815	lb

COMPRESSION CAPACITY (AISC 360 §E3, ASD)		
KL/r	74.1	
Slenderness ≤ 200 ?	**OK**	
Fe = π²E/(KL/r)²	52.16	ksi
4.71·√(E/Fy)	135.6	
Fcr	26.43	ksi
Pn = Fcr·A	17,682	lb
Pc,allow = Pn / 1.67	10,588	lb
D/C ratio — compression	0.08	
STATUS — strut compression	**OK**	

TENSION CAPACITY (AISC 360 §D2, ASD)		
Pt,allow = Fy·A / 1.67	14,021	lb
D/C ratio — tension	0.06	
STATUS — strut tension	**OK**	

HANGER ROD CHECK		
ΔT — overturning axial per rod = Eh·e/(d·N/2)	44	lb
T,rod max = Combo8/N + ΔT	184	lb
Ab — rod nominal area	0.196	in²
Rod allowable tension = 0.75·Fu·Ab / 2.00	4,410	lb
D/C ratio — rod tension	0.04	
STATUS — rod tension	**OK**	

IDS GROUP

Job No.:
Date:
Engineer:

RTU UNITS
ANCHORAGE

Refrigeration Specification

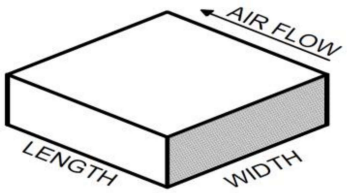
Quote #: E117905
Date: 06/12/2026

HEAVIEST UNIT USED FOR DESIGN

Item 0006 Indoor Cooler - Shared Cooler Holding

Room Temperature 35°F

Refrigeration system is designed to maintain 35°F at 80°F ambient temperature around box and 110°F outdoor ambient temperature assuming products enter at 35°F with moderate traffic.



28.25" 27.875"

Condensing Unit: RFON130E4AEANT

8695.5 BTUH

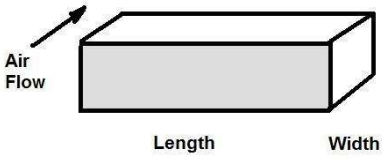
Ambient temp 110°F

NEXT-GEN MINI-CON: A2L: NO DEFROST PROVISIONS (ECONET)

Compressor Type	Refrigerant	HP	Electrical	RLA	LRA	MCA	MOPD
Scroll	**R-454A**	**1**	**208-230/3/60**	**7.2**	**55.4**	**15**	**15**

Receiver Capacity	Liquid	Suction		Length	Width	Height	Weight
10.8	**3/8 OD**	**5/8 OD**		**28.25**	**27.875**	**18.75**	**195**

Notice:Air cooled condensing units require a minimum of 1000 cfm of fresh air per horsepower



27.125" 15.5"

Unit Cooler: RAL6A081ADARSF

QTY 1

8100 BTUH

LOW PROFILE WITH DUAL SPEED EC MOTOR - A2L - ECONET ENABLED	**Heaters**		**Fans**	**Qty**	**1**
	Voltage	**N/A**	**Voltage**	**115/1/60**	
	Amps	**0**	**Amps Total**	**0.8**	

Drain Connection	Inlet	Suction	Equalizer	Length	Width	Height	Weight
3/4 x 1/2 NPT	**3/8 OD**	**5/8 OD**	**1/4 OD**	**27.125**	**15.5**	**18.125**	**47**

Standard Features:

Unit Cooler
Pressure Control
Liquid Line Filter/Drier
Sight Glass

Outdoor Systems Include:

Head Pressure Control

IDS GROUP

Job No.: 24X084.01
Date: 10/14/26
Engineer: DD

ASCE 7-22 CH. 13 RTU CALCULATION

SEISMIC DESIGN PARAMETERS

SDS — Design spectral response accel., short period (g)	1.38
Ip — Component importance factor	1.25
Wp — Component operating weight (lb)	195
z — Height of attachment point above base (ft)	1.00
h — Mean roof height of supporting structure (ft)	1.00
z/h (ratio, capped at 1.0)	1.00
Hf — Flexibility (height) amplification factor	3.50
Rμ — Building ductility reduction factor	1.30
CAR — Component amplification/resonance factor	1.40
Rpo — Component overstrength factor	2.00
R — building SFRS response mod. coeff.	2.00
Ω0 — building SFRS overstrength	2.50

Horizontal Seismic Design Force, Fp

Fp = 0.4 · SDS · Ip · Wp · (Hf/Rμ) · (CAR/Rpo) [ASCE 7-22 Eq. 13.3-1]	
Fp (calculated)	253
Fp,max = 1.6 · SDS · Ip · Wp (Eq. 13.3-2 upper limit)	537
Fp,min = 0.3 · SDS · Ip · Wp (Eq. 13.3-3 lower limit)	101
Fp — GOVERNING (lb)	253

Vertical Seismic Force

Fpv = 0.2 · SDS · Wp (per §12.4.2.2, applied ± vertically)	54

Anchor Overstrength Force (for concrete-controlled limit states)

Ω0 — Overstrength factor for the component/anchorage	2.00
Ω0·Fp — Amplified horizontal seismic force (lb)	506

IDS GROUP

Job No.: 24X084.01
Date: 10/14/26
Engineer: DD

ASCE 7-22 CH. 29 RTU CALCULATION

Velocity Pressure Inputs	
V — Basic (ultimate) design wind speed, Vult (mph)	95
Kz — Velocity pressure exposure coefficient at height z	0.90
Kzt — Topographic factor	1.00
Kd — Wind directionality factor	0.85
Ke — Ground elevation factor	1.00
qh (calculated)	17.67

Rooftop Equipment Geometry	
B — Horizontal building dimension normal to wind (ft)	180.00
h — Mean roof height of building (ft)	20.00
Af — Vertical projected (windward) area of unit, incl. clearance (ft²)	5.46
Ar — Horizontal projected (plan) area of unit (ft²)	5.83
0.1·B·h (ft²)	360.00
B·h (ft²)	3600.00

GCr — Pressure/Force Coefficient (linear interpolation per §29.4.1)	
GCr, lateral (Fh) — 1.9 if Af ≤ 0.1Bh; interpolate to 1.0 at Af = Bh	1.90
GCr, uplift (Fv) — fixed at 1.5	1.50

GCr — Pressure/Force Coefficient (linear interpolation per §29.4.1)	
Fh = qh · GCr(lateral) · Af (lb, horizontal)	183
Fv = qh · GCr(uplift) · Ar (lb, vertical uplift)	155

IDS GROUP

Job No.: 24X084.01
Date: 10/14/26
Engineer: DD

Governing Forces

Fp — Seismic horizontal force, governing (lb) PER 13.3.1 100%+30%	329
Ω0·Fp — Amplified seismic force for brittle (concrete) anchors (lb)	657
Fpv — Seismic vertical force, ± (lb)	54
Wp — Component operating weight (lb)	195
Fh — Wind horizontal (lateral) force, ultimate (lb)	183
Fv — Wind vertical (uplift) force, ultimate (lb)	155

Anchor Group Geometry

N — Number of anchors	4
L — Anchor spacing in direction of overturning (ft) Short Direction	1.96
h_cg — Height of lateral force application above anchor plane (ft)	0.92

Load Combination Basis (§2.3 / §2.4)	**ASD**	**LRFD**
Seismic Combinations		
Shear demand, V_E (ASD: 0.7·Fp | LRFD: 1.0·Fp)	230	329
Overturning moment, M_E = V_E · h_cg (ft-lb)	211	301
Tension per anchor, T_E = (net uplift)/N + 2·M_E/(N·L) (governs if > 0)	34	46
Wind Combinations		
Shear demand, V_W (ASD: 0.6·Fh | LRFD: 1.0·Fh)	110	183
Overturning moment, M_W = V_W · h_cg (ft-lb)	101	168
Tension per anchor, T_W = (net uplift)/N + 2·M_W/(N·L) (governs if > 0)	20	38
Governing Per-Anchor Demand		
Governing group shear, V_max = MAX(V_E, V_W) (lb)	230	329
Per-anchor shear demand, v_u = V_max / N (lb/anchor)	58	82
Per-anchor tension demand, t_u = MAX(T_E, T_W) (lb/anchor, incl. overturning)	34	46
Amplified per-anchor shear, concrete-controlled (ASD: Ω0Fp·0.7/N | LRFD: Ω0Fp/N)	115	164
Amplified per-anchor tension, concrete-controlled overturning (Ω0Fp basis)	88	123

EXISTING BEAM DESIGN



FRAMING NOTES

1. ROOF DECK AT SHALL BE VERCO PLB-36 (APMO ER-2018 OR APPROVED EQUAL. Ix+ = 0.302 in⁴/ft, Ix- = 0.306 in⁴/ft, Sx+ = 0.314 in³/ft, Sx+ = 0.331 in³/ft ; 18 GAGE (U.N.O.).
2. ALL STEEL BEAMS SHALL BE EQUALLY SPACED BETWEEN CONTROLLING COLUMNS, BEAMS, OR GRID LINES TYP. (U.N.O.)
3. ALL STEEL COLUMNS SHALL BE FULL BUILDING HEIGHT AS SIZED PER FOUNDATION PLAN TYP. (U.N.O.).
4. SEE ARCH DWGS, FOR ALL DIMENSIONS NOT SHOWN.
5. SEE ARCH DWGS, FOR ALL WALL ELEVATIONS.
6. SEE MEP DWGS, FOR EXACT LOCATION OF DUCT AND PIPE/CONDUIT PENETRATIONS.
7. SEE SHEETS S-001 TO S-016, FOR ADDITIONAL INFO, NOT SHOWN OR NOT INDICATED IN THIS DRAWING.
8. FIRE SPRINKLER MAIN LINES (4"ø MAX., SCH. 10) SHALL ONLY BE SUPPORTED BY (E) 5½ x OR 7x GLULAM BEAMS, OR STEEL BEAMS. WHEN SUPPORT FRAMING IS NOT PRESENT FOR FIRE SPRINKLER MAIN LINE HANGER ATTACHMENT POINTS. THE CONTRACTOR SHALL PROVIDE ADDITIONAL 4x12 MIN. DF#1 BEAMS WHERE REQUIRED.
9. FIRE SPRINKLER BRANCH LINES (2"ø MAX. SCH. 40) MAY BE SUPPORTED BY 5½ x OR 7x GLULAM BEAMS OR STEEL BEAMS. FIRE SPRINKLER BRANCH LINES SHALL NOT BE SUPPORTED BY A SINGLE 2x ROOF MEMBER; PROVIDE LAMINATED 2x ROOF JOIST TO MATCH (E) -/---
10. SHORING CONTRACTOR SHALL SUBMIT SHORING DESIGN WITH ALL CALCULATIONS INCLUDED IN SUBMITTAL AND SIGNED BY A CALIFORNIA REGISTERED CIVIL OR STRUCTURAL ENGINEER.
11. THE STRUCTURAL DESIGN FOR EQUIPMENT SUPPORT IS BASED ON THE WEIGHTS SHOWN ON DRAWINGS. IF THE CONTRACTOR CHOOSES OR IS ALLOWED TO SUBSTITUTE EQUIPMENT WEIGHTING MORE THAN THE INDICATED LOAD, ALL STRUCTURAL DESIGN REVISIONS AND ASSOCIATED APPROVALS SHALL BE BORNE AND EXECUTED BY THE CONTRACTOR OR AT HIS/ HER EXPENSE AND WITHOUT CHANGE IN CONTRACT AMOUNT OR TIME. SUCH CHANGED SHALL BE SUBMITTED TO SEOR AND CITY FOR REVIEW AND APPROVAL PRIOR TO EQUIPMENT INSTALLATION.
12. INFILL EXISTING OPENING W/ 2x STUD @ 16" O.C., DEPTH TO MATCH EXISTING JOIST. WHERE EXISTING BLKG INTERFERES WITH NEW JOIST/BEAM, CUT THE EXISTING BLKG AND RECONNECT TO THE NEW MEMBER WITH SIMPSON HU HANGER. PROVIDE NEW PLYWOOD AND NAILING PER SCHEDULE ON SHEET S-220. OPTION TO REMOVE EXISTING FRAMINGS AROUND THE OPENING AND PROVIDE NEW 2x JOIST DEPTH TO MATCH EXISTING.
13. REMOVE (E) 2x JOISTS FOR INSTALLATION OF (N) EQUIPMENT SUPPORT BEAMS W/ SIMPSON HU HANGER AT EA. END., AND USE GRADE NO. 1 AND BETTER FOR NEW SUPPORT BEAMS.
14. THE CONTRACTOR SHALL HAVE AN ALLOWANCE IN THE BID TO REPLACE 65 PLYWOOD SHEETS (1/2" THK. STRUCT 1) AT THE EXISTING ROOF SHOULD ANY SHEETS BE DAMAGED.
15. PROVIDE NEW PLYWOOD SHTG. BETWEEN GL 3-5 TO GL B-F, SEE S-200 AND -/--- FOR PLYWOOD NAILING.
16. SHORING CONTRACTOR SHALL SUBMIT SHORING DESIGN WITH ALL CALCULATIONS INCLUDED IN SUBMITTAL AND SIGNED BY A CALIFORNIA REGISTERED CIVIL OR STRUCTURAL ENGINEER.
17. STEEL FABRICATOR QUALIFICATIONS: A QUALIFIED FABRICATOR THAT PARTICIPATES IN THE AISC QUALITY CERTIFICATION PROGRAM AND IS DESIGNATED AN AISC-CERTIFIED PLANT, CATEGORY STD OR LA CITY CERTIFICATION.
18. STEEL ERECTOR QUALIFICATIONS: A QUALIFIED INSTALLER WHO PARTICIPATES IN THE AISC QUALITY CERTIFICATION PROGRAM AND IS DESIGNATED AN AISC-CERTIFIED ERECTOR, CATEGORY ACSE OR CSE.

LEGEND

- INDICATES PIPE COLUMN
- INDICATES HSS COL.
- INDICATES 8" THK. MASONRY WALL
- INDICATES (E) 8" THK. MASONRY WALL
- STEEL DECK, INDICATES DIRECTION OF STEEL DECKING) (U.N.O.)
- INDICATES EXTENT OF JOISTS
- INDICATES JOIST SPAN DIRECTION
- INDICATES OPENINGS
- INDICATES TSC/ W BEAM CAMBER

ROOF FRAMING PLAN | 1/8" = 1'-0" | 1

CLIENT

CITY OF CORONA EST. 1896

400 S VICENTIA AVE, CORONA, CA 92882

CORONA SOUTH MALL BUILDING "B" TENANT IMPROVEMENT

725 S MAIN ST, CORONA, CA 92882

ENGINEER/ARCHITECT

IDS

IDS GROUP

1 PETERS CANYON ROAD, SUITE 130
IRVINE, CA, 92606
TEL: 949-387-8500, FAX: 949-387-0800

STAMP

STAMP

ISSUE		
REV.	DESCRIPTION	DATE

PROJECT IDENTIFICATION

THIS DRAWING WAS CREATED IN REVIT V, 2023 UNLESS OTHERWISE NOTED.

THE ORIGINAL SIZE OF THIS SHEET IS 30" X 42"

THESE DRAWINGS AND SPECIFICATIONS ARE THE PROPERTY AND COPYRIGHT OF IDS GROUP AND SHALL NOT BE USED ON ANY OTHER PROJECT OR LOCATIONS EXCEPT AS DESCRIBED ON THE DRAWINGS, WITHOUT THE WRITTEN AGREEMENT WITH IDS GROUP.

IDS PROJECT NO.	24X084.01
DRAWN BY	CN
CHECKED BY	VM
APPROVED BY	SH

SHEET TITLE

ROOF FRAMING PLAN

SHEET NUMBER

TS-210

SHEET NO. OF

APPROVAL STAMP

PROGRESS SET - NOT FOR CONSTRUCTION OR BID

IDS GROUP - FILE LOCATION: C:\Users\calli.nguyen\Documents\24X084.01_Struct_BLDG B_R23_Phase 2_calli.nguyen.rvt PLOT DATE: 8/21/2026 3:10:19 PM USER: Author

Project Title:
Engineer:
Project ID:
Project Descr:

B1

Wood Beam

Project File: 24X084.01 City of Corona, South Mall Development - TI.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: (E) 2x16 W/ RTU SPANNING BTWN GRID A AND B

CODE REFERENCES

Calculations per NDS 2024, IBC 2024
Load Combination Set : ASCE 7-22 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-22 / IBC 2024

Wood Species : Douglas Fir-Larch
Wood Grade : Select Structural

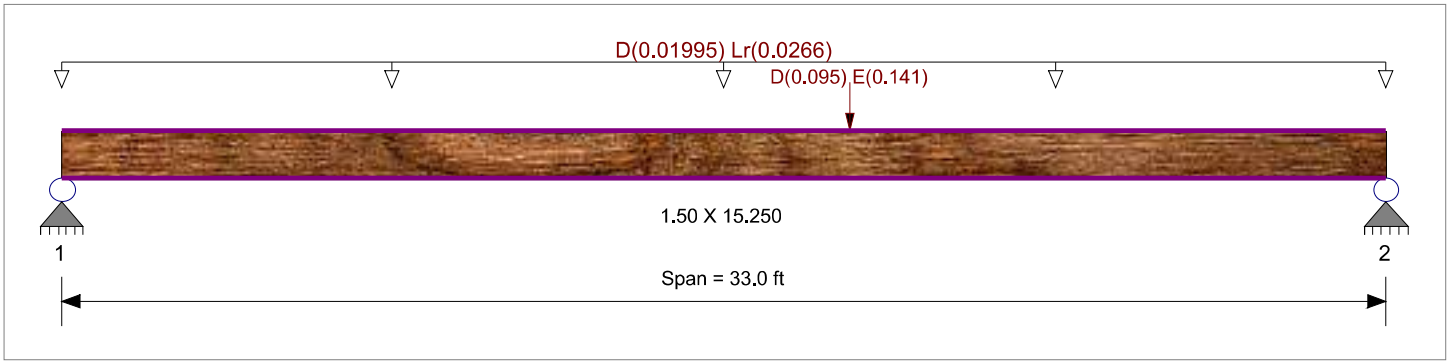
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1500 psi
Fb -	1500 psi
Fc - Prll	1700 psi
Fc - Perp	625 psi
Fv	180 psi
Ft	1000 psi

E : Modulus of Elasticity

Ebend- xx	1900 ksi
Eminbend - xx	690 ksi
Density	31.21 pcf

Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.0150, Lr = 0.020 ksf, Tributary Width = 1.330 ft
Point Load : D = 0.0950, E = 0.1410 k @ 19.660 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.815 : 1	Maximum Shear Stress Ratio	=	0.246 : 1
Section used for this span		1.50 X 15.250	Section used for this span		1.50 X 15.250
fb: Actual	=	1,580.88 psi	fv: Actual	=	55.37 psi
F'b	=	1,940.63 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	17.223 ft	Location of maximum on span	=	31.796 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.848 in	Ratio =	467 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	1.780 in	Ratio =	222 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 33.0 ft	1	0.599	0.177	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.15	4.05	836.7	1,397.3	0.44	28.7	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 33.0 ft	1	0.815	0.246	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.15	7.66	1,580.9	1,940.6	0.84	55.4	225.0
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 33.0 ft	1	0.719	0.216	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.15	6.76	1,394.5	1,940.6	0.74	48.7	225.0
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 33.0 ft	1	0.202	0.060	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	2.43	502.0	2,484.0	0.26	17.2	288.0

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 24X084.01 City of Corona, South Mall Development - TI.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: (E) 2x16 W/ RTU SPANNING BTWN GRID A AND B

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+0.70E					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 33.0 ft	1	0.399	0.113	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	4.81	991.9	2,484.0	0.50	32.5	288.0
+D+0.5250E					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 33.0 ft	1	0.383	0.110	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	4.61	951.8	2,484.0	0.48	31.6	288.0
+0.60D+0.70E					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 33.0 ft	1	0.266	0.073	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	3.20	659.8	2,484.0	0.32	21.1	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.7804	16.620		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.888	0.906
Max Upward from Load Combinations	0.888	0.906
Max Upward from Load Cases	0.449	0.468
D Only	0.449	0.468
+D+Lr	0.888	0.906
+D+0.750Lr	0.779	0.797
+0.60D	0.270	0.281
+D+0.70E	0.489	0.526
+D+0.5250E	0.479	0.512
+0.60D+0.70E	0.310	0.339
Lr Only	0.439	0.439
E Only	0.057	0.084

Project Title:
Engineer:
Project ID:
Project Descr:

B2

Wood Beam

Project File: 24X084.01 City of Corona, South Mall Development - TI.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: (E) TYP 2x14 W/ RTU

CODE REFERENCES

Calculations per NDS 2024, IBC 2024
Load Combination Set : ASCE 7-22 / IBC 2024

Material Properties

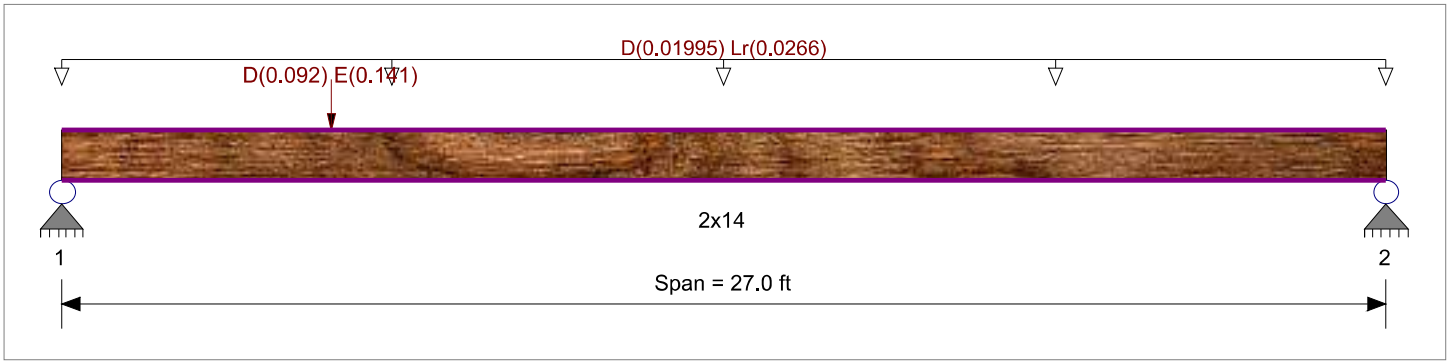
Analysis Method :	Allowable Stress Design
Load Combination :	ASCE 7-22 / IBC 2024
Wood Species :	Douglas Fir-Larch
Wood Grade :	Select Structural
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling

Fb +	1,500.0 psi
Fb -	1,500.0 psi
Fc - Prll	1,700.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	1,000.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	690.0 ksi
Density	31.210 pcf

Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.0150, Lr = 0.020 ksf, Tributary Width = 1.330 ft
Point Load : D = 0.0920, E = 0.1410 k @ 5.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.689 : 1	Maximum Shear Stress Ratio	=	0.236 : 1
Section used for this span		2x14	Section used for this span		2x14
fb: Actual	=	1,337.19 psi	fv: Actual	=	53.19 psi
F'b	=	1,940.63 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	13.106 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.579 in	Ratio =	559	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	1.176 in	Ratio =	275	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 26.901 ft	1	0.483	0.174	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.15	2.47	675.5	1,397.3	0.37	28.3	162.0
Length = 0.09854 ft	1	0.007	0.149	0.90	1.00	1.00	1.00	0.900	1.00	1.00	1.15	0.03	9.3	1,397.3	0.32	24.1	162.0
+D+Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 26.901 ft	1	0.689	0.236	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.15	4.89	1,337.2	1,940.6	0.70	53.2	225.0
Length = 0.09854 ft	1	0.010	0.218	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.15	0.07	18.9	1,940.6	0.65	49.1	225.0
+D+0.750Lr					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 26.901 ft	1	0.604	0.209	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.15	4.29	1,171.7	1,940.6	0.62	47.0	225.0

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 24X084.01 City of Corona, South Mall Development - TI.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: (E) TYP 2x14 W/ RTU

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 0.09854 ft	1	0.009	0.190	1.25	1.00	1.00	1.00	0.900	1.00	1.00	1.15	0.06	16.5	1,940.6	0.57	42.8	225.0
+0.60D					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 26.901 ft	1	0.163	0.059	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	1.48	405.3	2,484.0	0.22	17.0	288.0
Length = 0.09854 ft	1	0.002	0.050	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	0.02	5.6	2,484.0	0.19	14.5	288.0
+D+0.70E					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 26.901 ft	1	0.304	0.119	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	2.77	756.2	2,484.0	0.45	34.2	288.0
Length = 0.09854 ft	1	0.004	0.089	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	0.04	9.8	2,484.0	0.34	25.7	288.0
+D+0.5250E					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 26.901 ft	1	0.296	0.114	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	2.69	735.6	2,484.0	0.43	32.7	288.0
Length = 0.09854 ft	1	0.004	0.088	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	0.04	9.7	2,484.0	0.34	25.3	288.0
+0.60D+0.70E					1.00	1.00	1.00	0.900	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 26.901 ft	1	0.196	0.079	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	1.78	487.6	2,484.0	0.30	22.9	288.0
Length = 0.09854 ft	1	0.002	0.056	1.60	1.00	1.00	1.00	0.900	1.00	1.00	1.15	0.02	6.1	2,484.0	0.21	16.0	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.1759	13.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.760	0.705
Max Upward from Load Combinations	0.760	0.705
Max Upward from Load Cases	0.401	0.359
D Only	0.401	0.346
+D+Lr	0.760	0.705
+D+0.750Lr	0.670	0.616
+0.60D	0.240	0.208
+D+0.70E	0.479	0.366
+D+0.5250E	0.460	0.361
+0.60D+0.70E	0.319	0.228
Lr Only	0.359	0.359
E Only	0.112	0.029

Project Title:
Engineer:
Project ID:
Project Descr:

B3

Wood Beam

Project File: 24X084.01 City of Corona, South Mall Development - TI.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: (E) 2x12 W/ RTU SPANNING BTWN 1 TO 1.5 AND 6 TO 7

CODE REFERENCES

Calculations per NDS 2024, IBC 2024
Load Combination Set : ASCE 7-22 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-22 / IBC 2024

Wood Species : Douglas Fir-Larch
Wood Grade : Select Structural

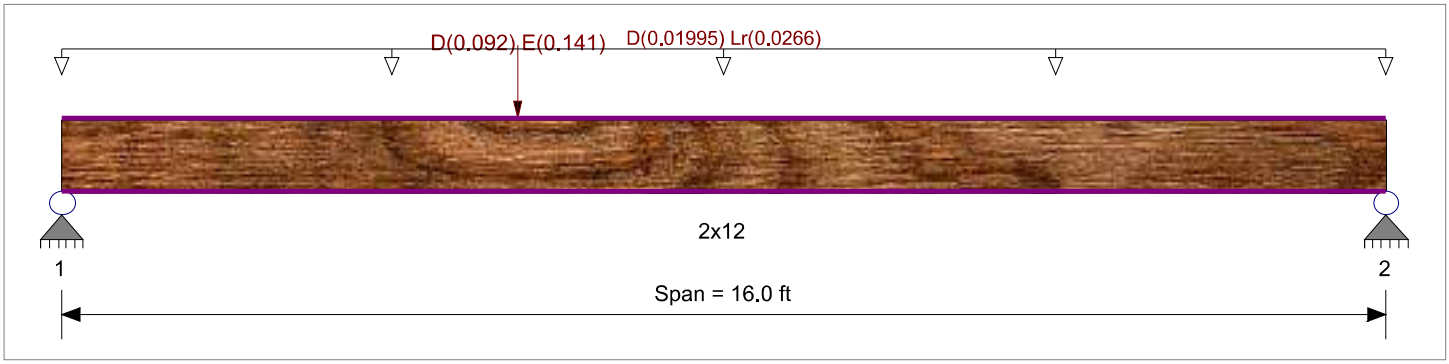
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1,500.0 psi
Fb -	1,500.0 psi
Fc - Prll	1,700.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	1,000.0 psi

E : Modulus of Elasticity

Ebend- xx	1,900.0 ksi
Eminbend - xx	690.0 ksi
Density	31.210 pcf

Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.0150, Lr = 0.020 ksf, Tributary Width = 1.330 ft
Point Load : D = 0.0920, E = 0.1410 k @ 5.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.329 : 1	Maximum Shear Stress Ratio	=	0.164 : 1
Section used for this span		2x12	Section used for this span		2x12
fb: Actual	=	709.06 psi	fv: Actual	=	36.90 psi
F'b	=	2,156.25 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	7.358 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.117 in	Ratio =	1645 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.255 in	Ratio =	751 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.252	0.125	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.15	1.03	390.5	1,552.5	0.23	20.2	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.329	0.164	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.15	1.87	709.1	2,156.3	0.42	36.9	225.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.292	0.145	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.15	1.66	628.9	2,156.3	0.37	32.7	225.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.085	0.042	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.15	0.62	234.3	2,760.0	0.14	12.1	288.0

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

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DESCRIPTION: (E) 2x12 W/ RTU SPANNING BTWN 1 TO 1.5 AND 6 TO 7

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+0.70E					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.188	0.090	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.15	1.37	519.4	2,760.0	0.29	26.0	288.0
+D+0.5250E					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.176	0.085	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.15	1.28	485.8	2,760.0	0.28	24.5	288.0
+0.60D+0.70E					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.132	0.062	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.15	0.96	365.5	2,760.0	0.20	17.9	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.2553	7.942		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.462	0.433
Max Upward from Load Combinations	0.462	0.433
Max Upward from Load Cases	0.249	0.220
D Only	0.249	0.220
+D+Lr	0.462	0.433
+D+0.750Lr	0.409	0.380
+0.60D	0.150	0.132
+D+0.70E	0.314	0.254
+D+0.5250E	0.298	0.246
+0.60D+0.70E	0.214	0.166
Lr Only	0.213	0.213
E Only	0.093	0.048

Project Title:
Engineer:
Project ID:
Project Descr:

B4

Wood Beam

Project File: 24X084.01 City of Corona, South Mall Development - TI.ec6

LIC# : KW-06018039, Build:20.25.04.16 INTEGRATED DESIGN SERVICES, INC (c) ENERCALC, LLC 1982-2026

DESCRIPTION: (E) 5 1/2 x 27 5/8 GLB w/ RTU

CODE REFERENCES

Calculations per NDS 2024, IBC 2024
Load Combination Set : ASCE 7-22 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-22 / IBC 2024

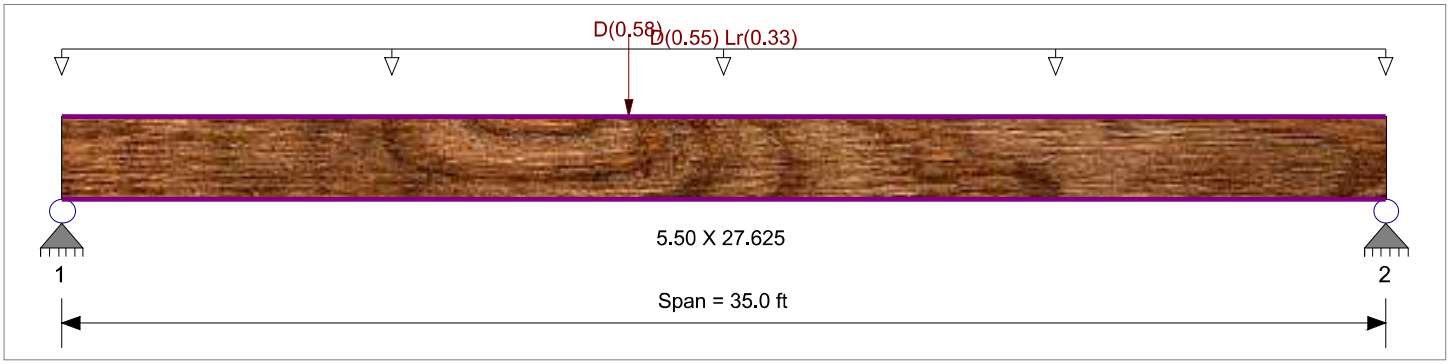
Wood Species : DF/DF
Wood Grade : 24F-V4

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

Fb +	2,400.0 psi
Fb -	1,850.0 psi
Fc - Prll	1,650.0 psi
Fc - Perp	650.0 psi
Fv	265.0 psi
Ft	1,100.0 psi

E : Modulus of Elasticity

Ebend- xx	1,800.0 ksi
Eminbend - xx	950.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.0120 ksf, Tributary Width = 27.50 ft
Point Load : D = 0.580 k @ 15.0 ft, (RTU)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.950 : 1	Maximum Shear Stress Ratio	=	0.423 : 1
Section used for this span		5.50 X 27.625	Section used for this span		5.50 X 27.625
fb: Actual	=	2,473.20 psi	fv: Actual	=	140.27 psi
F'b	=	2,604.10 psi	F'v	=	331.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	17.245 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.644 in	Ratio =	651	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	1.833 in	Ratio =	229	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 35.0 ft	1	0.857	0.381	0.90	1.00	1.00	1.00	0.868	1.00	1.00	1.00	93.66	1,606.7	1,875.0	9.19	90.8	238.5
+D+Lr					1.00	1.00	1.00	0.868	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 35.0 ft	1	0.950	0.423	1.25	1.00	1.00	1.00	0.868	1.00	1.00	1.00	144.18	2,473.2	2,604.1	14.21	140.3	331.3
+D+0.750Lr					1.00	1.00	1.00	0.868	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 35.0 ft	1	0.867	0.386	1.25	1.00	1.00	1.00	0.868	1.00	1.00	1.00	131.55	2,256.5	2,604.1	12.95	127.9	331.3
+0.60D					1.00	1.00	1.00	0.868	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 35.0 ft	1	0.289	0.128	1.60	1.00	1.00	1.00	0.868	1.00	1.00	1.00	56.20	964.0	3,333.2	5.52	54.5	424.0

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

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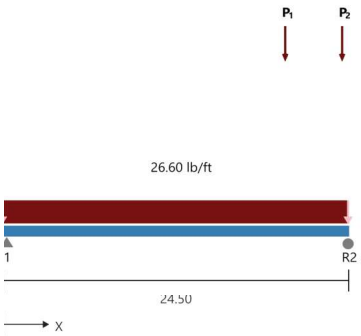
Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.8329	17.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	16.308	16.225
Max Upward from Load Combinations	16.308	16.225
Max Upward from Load Cases	10.533	10.450
D Only	10.533	10.450
+D+Lr	16.308	16.225
+D+0.750Lr	14.864	14.781
+0.60D	6.320	6.270
Lr Only	5.775	5.775



Section: 800S162-97 (50 ksi) Single C Stud (punched)
Maxo = 6058.7 ft-lb **Va =** 10885.3 lb **I =** 9.71 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	Full, 294.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1*	467.28	1.00	1607.9	0.0	0.15	NO
R2*	1570.42	1.00	1528.8	0.0	0.53	YES
P1	693.00	1.50	4976.0	4026.1	0.47	NO
P2	693.00	1.50	1717.4	766.3	0.29	NO

"*" after support means punched near support

Point Loads	P1	P2
Load(lb)	693.00	693.00
X-Dist.(ft)	20.00	24.00

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	1570.4	5937.9	26%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	4104.3	5738.3	72%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	4104.3	6058.7	68%	
	Shear/Moment	0.68	1.00	68%	Shear 0.0, Moment 4104.3
	Axial/Moment	0.72	1.00	72%	Axial 0.0(c), Moment 4104.3
	Deflection Span, in	1.512	--meets L/194--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	467.3	By Others & Anchorage Designed by Engineer	NA	NA
R2	0.0	1570.4	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements