

Structural Calculations for Heartland Market

Lees Summit, MO

Prepared by:



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 J&S STRUCTURAL ENGINEERS 6640 W 143 rd St Suite 250 Overland Park, KS 66223 913.549.4701	Project Heartland Market – Lees Summit				Job Ref.	
	Section Wind Loading				Sheet no./rev. 1	
	Calc. by K	Date 3/29/2023	Chk'd by	Date	App'd by	Date

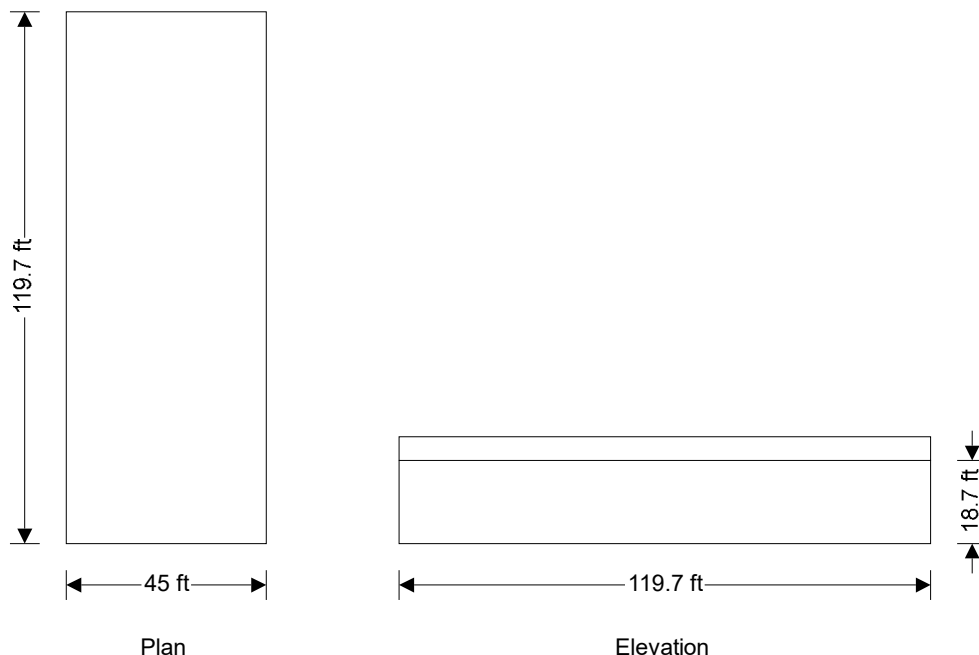
WIND LOADING (ASCE7)

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.09



Building data

Type of roof	Flat
Length of building	b = 45.00 ft
Width of building	d = 119.67 ft
Height to eaves	H = 18.67 ft
Height of parapet	h _p = 5.33 ft
Mean height	h = 18.67 ft

General wind load requirements

Basic wind speed	V = 115.0 mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K _d = 0.85
Ground elevation above sea level	z _{gl} = 0 ft
Ground elevation factor	K _e = exp(-0.0000362 × z _{gl} /1ft) = 1.00
Exposure category (cl 26.7.3)	B
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC _{pi_p} = 0.18
Internal pressure coef -ve (Table 26.13-1)	GC _{pi_n} = -0.18
Gust effect factor	G _f = 0.85
Minimum design wind loading (cl.27.4.7)	p _{min_r} = 8 lb/ft ²

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Topography

Topography factor not significant

$$K_{zt} = 1.0$$

Velocity pressure equation

$$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1 \text{ psf/mph}^2$$

Velocity pressures table

z (ft)	K _z (Table 26.10-1)	q _z (psf)
15.00	0.57	16.40
18.67	0.61	17.46
24.00	0.65	18.76

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) $q_i = 17.46$ psf

Parapet pressures and forces

Velocity pressure at top of parapet

$$q_p = 18.76 \text{ psf}$$

Combined net pressure coefficient, leeward

$$GC_{pnl} = -1.0$$

Combined net parapet pressure, leeward

$$p_{pl} = q_p \times GC_{pnl} = -18.76 \text{ psf}$$

Combined net pressure coefficient, windward

$$GC_{pnw} = 1.5$$

Combined net parapet pressure, windward

$$p_{pw} = q_p \times GC_{pnw} = 28.14 \text{ psf}$$

Wind direction 0 deg:

Leeward parapet force

$$F_{w,wpl_0} = p_{pl} \times h_p \times b = -4.5 \text{ kips}$$

Windward parapet force

$$F_{w,wpw_0} = p_{pw} \times h_p \times b = 6.8 \text{ kips}$$

Wind direction 90 deg:

Leeward parapet force

$$F_{w,wpl_90} = p_{pl} \times h_p \times d = -12 \text{ kips}$$

Windward parapet force

$$F_{w,wpw_90} = p_{pw} \times h_p \times d = 18 \text{ kips}$$

Pressures and forces

Net pressure

$$p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$$

Net force

$$F_w = p \times A_{ref}$$

Roof load case 1 - Wind 0, GC_{pi} 0.18, -C_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c _{pe}	Peak velocity pressure q _p (psf)	Net pressure p (psf)	Area A _{ref} (ft ²)	Net force F _w (kips)
A (-ve)	18.67	-0.90	17.46	-16.50	420.08	-6.93
B (-ve)	18.67	-0.90	17.46	-16.50	420.08	-6.93
C (-ve)	18.67	-0.50	17.46	-10.56	840.15	-8.87
D (-ve)	18.67	-0.30	17.46	-7.59	3704.85	-28.14

Total vertical net force

$$F_{w,v} = -50.87 \text{ kips}$$

Total horizontal net force

$$F_{w,h} = 0.00 \text{ kips}$$

Walls load case 1 - Wind 0, GC_{pi} 0.18, -C_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c _{pe}	Peak velocity pressure q _p (psf)	Net pressure p (psf)	Area A _{ref} (ft ²)	Net force F _w (kips)
A ₁	15.00	0.80	16.40	8.01	675.00	5.41
A ₂	18.67	0.80	17.46	8.73	165.15	1.44

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Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
B	18.67	-0.27	17.46	-7.11	840.15	-5.97
C	18.67	-0.70	17.46	-13.53	2234.24	-30.23
D	18.67	-0.70	17.46	-13.53	2234.24	-30.23

Overall loading

Projected vertical plan area of wall

$$A_{vert_w_0} = b \times (H + h_p) = 1080.00 \text{ ft}^2$$

Projected vertical area of roof

$$A_{vert_r_0} = 0.00 \text{ ft}^2$$

Minimum overall horizontal loading

$$F_{w_total_min} = p_{min_w} \times A_{vert_w_0} + p_{min_r} \times A_{vert_r_0} = 17.28 \text{ kips}$$

Leeward net force

$$F_l = F_{w_WB} + F_{w_wpl_0} = -10.5 \text{ kips}$$

Windward net force

$$F_w = F_{w_wA_1} + F_{w_wA_2} + F_{w_wpw_0} = 13.6 \text{ kips}$$

Overall horizontal loading

$$F_{w_total} = \max(F_w - F_l + F_{w_h}, F_{w_total_min}) = 24.1 \text{ kips}$$

Roof load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A (+ve)	18.67	-0.18	17.46	0.47	420.08	0.20
B (+ve)	18.67	-0.18	17.46	0.47	420.08	0.20
C (+ve)	18.67	-0.18	17.46	0.47	840.15	0.40
D (+ve)	18.67	-0.18	17.46	0.47	3704.85	1.75

Total vertical net force

$$F_{w_v} = 2.54 \text{ kips}$$

Total horizontal net force

$$F_{w_h} = 0.00 \text{ kips}$$

Walls load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A ₁	15.00	0.80	16.40	14.30	675.00	9.65
A ₂	18.67	0.80	17.46	15.02	165.15	2.48
B	18.67	-0.27	17.46	-0.82	840.15	-0.69
C	18.67	-0.70	17.46	-7.25	2234.24	-16.19
D	18.67	-0.70	17.46	-7.25	2234.24	-16.19

Overall loading

Projected vertical plan area of wall

$$A_{vert_w_0} = b \times (H + h_p) = 1080.00 \text{ ft}^2$$

Projected vertical area of roof

$$A_{vert_r_0} = 0.00 \text{ ft}^2$$

Minimum overall horizontal loading

$$F_{w_total_min} = p_{min_w} \times A_{vert_w_0} + p_{min_r} \times A_{vert_r_0} = 17.28 \text{ kips}$$

Leeward net force

$$F_l = F_{w_WB} + F_{w_wpl_0} = -5.2 \text{ kips}$$

Windward net force

$$F_w = F_{w_wA_1} + F_{w_wA_2} + F_{w_wpw_0} = 18.9 \text{ kips}$$

Overall horizontal loading

$$F_{w_total} = \max(F_w - F_l + F_{w_h}, F_{w_total_min}) = 24.1 \text{ kips}$$

Roof load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

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Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A (-ve)	18.67	-0.90	17.46	-16.50	1117.12	-18.43
B (-ve)	18.67	-0.90	17.46	-16.50	1117.12	-18.43
C (-ve)	18.67	-0.50	17.46	-10.56	2234.24	-23.60
D (-ve)	18.67	-0.30	17.46	-7.59	916.67	-6.96

Total vertical net force $F_{w,v} = -67.42$ kips

Total horizontal net force $F_{w,h} = 0.00$ kips

Walls load case 3 - Wind 90, $GC_{pi} 0.18, -c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A ₁	15.00	0.80	16.40	8.01	1795.05	14.38
A ₂	18.67	0.80	17.46	8.73	439.19	3.83
B	18.67	-0.50	17.46	-10.56	2234.24	-23.60
C	18.67	-0.70	17.46	-13.53	840.15	-11.37
D	18.67	-0.70	17.46	-13.53	840.15	-11.37

Overall loading

Projected vertical plan area of wall

$$A_{vert_w_90} = d \times (H + h_p) = 2872.08 \text{ ft}^2$$

Projected vertical area of roof

$$A_{vert_r_90} = 0.00 \text{ ft}^2$$

Minimum overall horizontal loading

$$F_{w,total_min} = p_{min_w} \times A_{vert_w_90} + p_{min_r} \times A_{vert_r_90} = 45.95 \text{ kips}$$

Leeward net force

$$F_l = F_{w,wB} + F_{w,wpl_90} = -35.6 \text{ kips}$$

Windward net force

$$F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wpw_90} = 36.2 \text{ kips}$$

Overall horizontal loading

$$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = 71.7 \text{ kips}$$

Roof load case 4 - Wind 90, $GC_{pi} -0.18, +c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A (+ve)	18.67	-0.18	17.46	0.47	1117.12	0.53
B (+ve)	18.67	-0.18	17.46	0.47	1117.12	0.53
C (+ve)	18.67	-0.18	17.46	0.47	2234.24	1.05
D (+ve)	18.67	-0.18	17.46	0.47	916.67	0.43

Total vertical net force $F_{w,v} = 2.54$ kips

Total horizontal net force $F_{w,h} = 0.00$ kips

Walls load case 4 - Wind 90, $GC_{pi} -0.18, +c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A ₁	15.00	0.80	16.40	14.30	1795.05	25.66
A ₂	18.67	0.80	17.46	15.02	439.19	6.59

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Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
B	18.67	-0.50	17.46	-4.28	2234.24	-9.56
C	18.67	-0.70	17.46	-7.25	840.15	-6.09
D	18.67	-0.70	17.46	-7.25	840.15	-6.09

Overall loading

Projected vertical plan area of wall

Projected vertical area of roof

Minimum overall horizontal loading

Leeward net force

Windward net force

Overall horizontal loading

$$A_{vert_w_90} = d \times (H + h_p) = \mathbf{2872.08 \text{ ft}^2}$$

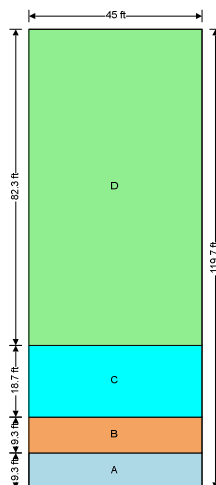
$$A_{vert_r_90} = \mathbf{0.00 \text{ ft}^2}$$

$$F_{w,total_min} = p_{min_w} \times A_{vert_w_90} + p_{min_r} \times A_{vert_r_90} = \mathbf{45.95 \text{ kips}}$$

$$F_l = F_{w,wB} + F_{w,wpl_90} = \mathbf{-21.5 \text{ kips}}$$

$$F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wpw_90} = \mathbf{50.2 \text{ kips}}$$

$$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = \mathbf{71.7 \text{ kips}}$$



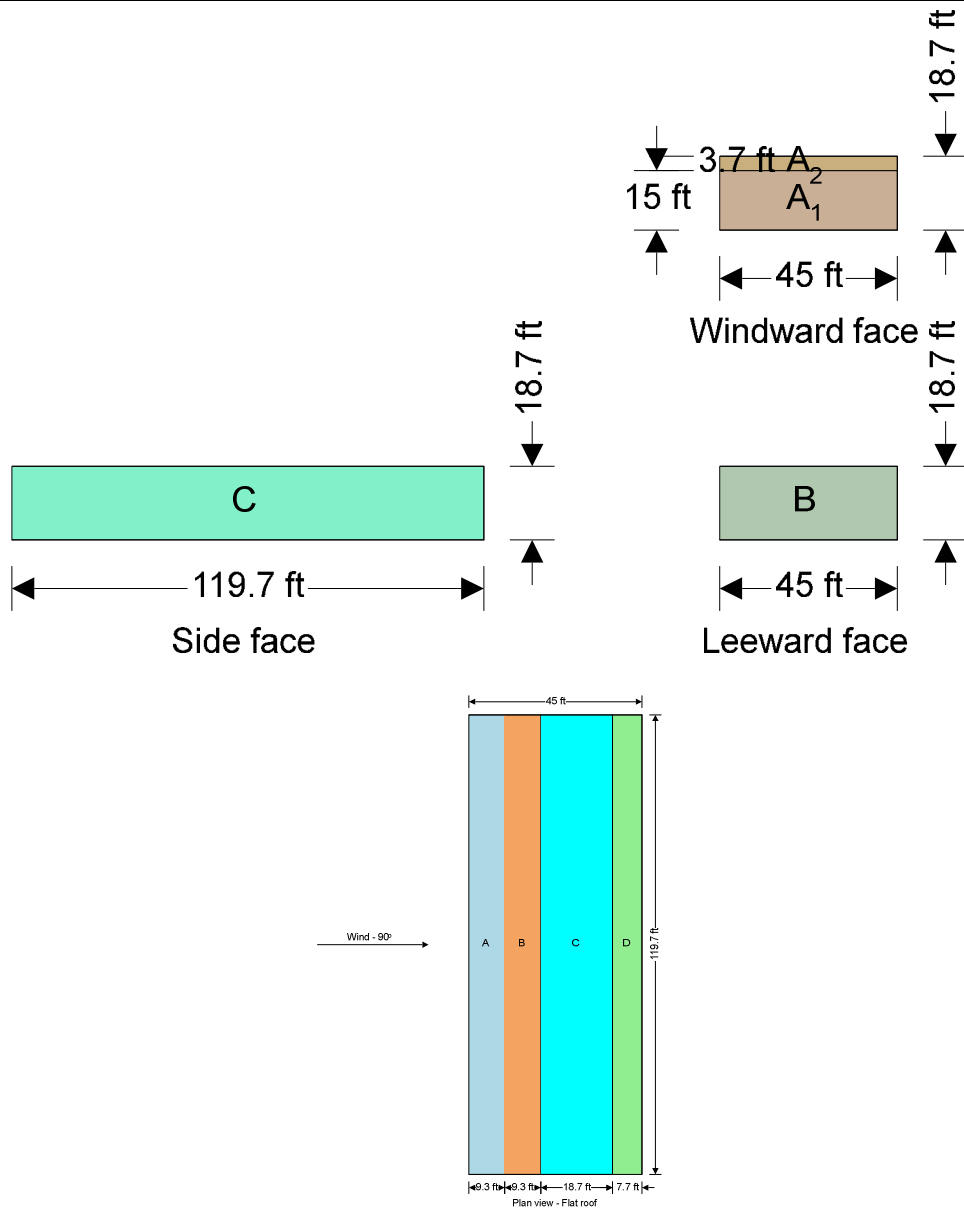
Wind - Cp
Plan view - Flat roof



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Project Heartland Market – Lees Summit				Job Ref.	
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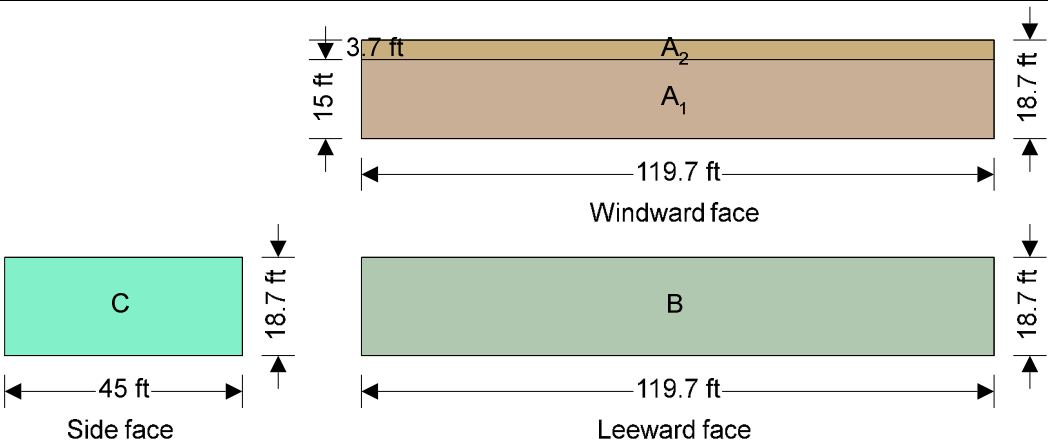




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Project				Job Ref.	
Heartland Market – Lees Summit					
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Calc. by	Date	Chk'd by	Date	App'd by	Date
K	3/29/2023				



NP FOUND design $40(22.5) + 55(12.67) + 10(11.33) = 1710 \text{ plf}$

$\frac{1500 \text{ psf}}{= 1.14' \text{ min Frg width}}$

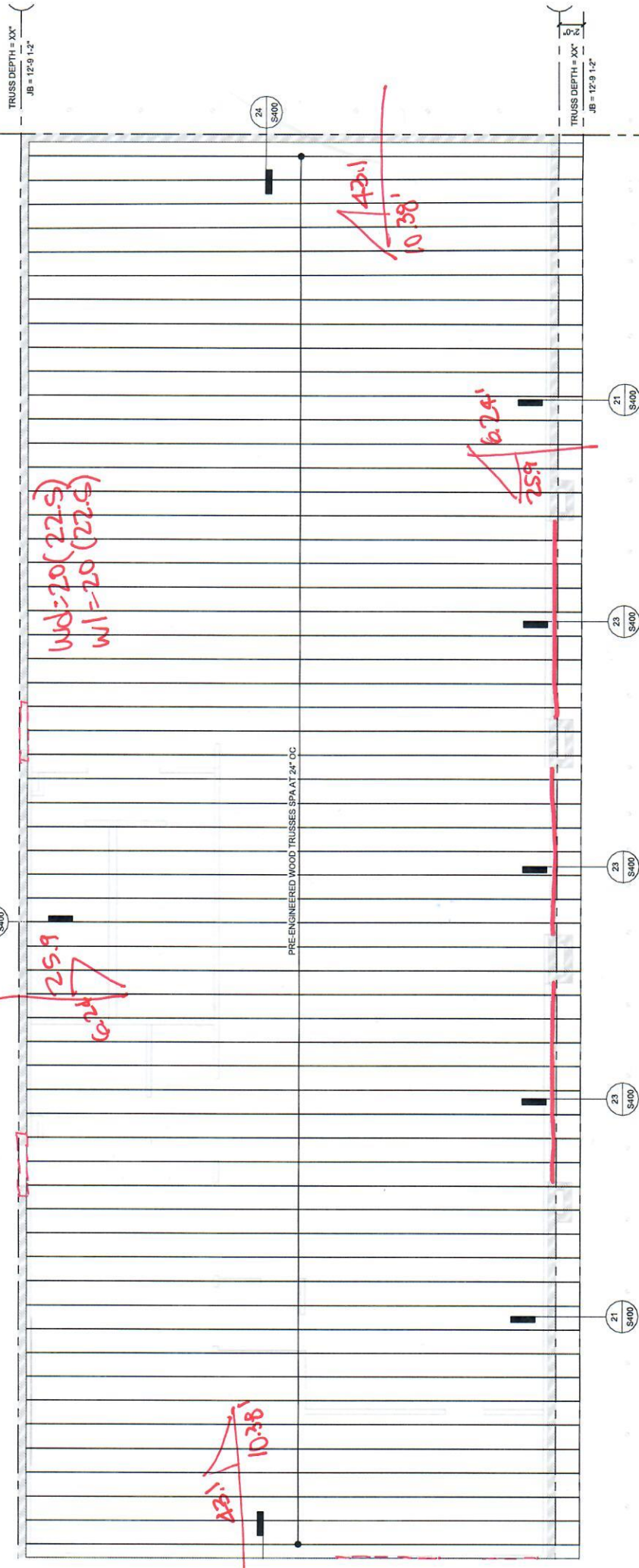
Wind: $20 \text{ psf}(9.33) + 20(1.5)(5.33) = 347 \text{ plf}$

X: $347(49) = 15.6 \text{ k}$

Y: $347(119.67) = 41.53$

Diaphragm: $41.53/2 / 45^\circ = 46 \text{ klf}(.6)$

277 klf



Project Title:
Engineer:
Project ID:
Project Descr:

ASCE 7-16 Seismic Base Shear

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

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DESCRIPTION: Seismic Base Shear Analysis

Specific Description:

Risk Category

Calculations per ASCE 7-16

Risk Category of Building or Other Structure : "II" : All Buildings and other structures except those listed as Category I, III, and IV [SCE 7-16, Page 4, Table 1.5-1](#)

Seismic Importance Factor = 1 [ASCE 7-16, Page 5, Table 1.5-2](#)

Gridded Ss & S1 values from ASCE 7-16

[ASCE 7-16 11.4.2](#)

Max. Ground Motions, 5% Damping

$S_S = 0.1005$ g, 0.2 sec response

$S_1 = 0.06858$ g, 1.0 sec response

Location Lees Summit, MO 64086

Latitude = 38.928 deg North

Longitude = 94.324 deg West

For the closest datapoint grid location . . .

Latitude = 38.930 deg North

Longitude = 94.320 deg West

Site Class, Site Coeff. and Design Category

Classification: "D" : Shear Wave Velocity 600 to 1,200 ft/sec = **D** (Based on Testing) [ASCE 7-16 Table 20.3-1](#)

Site Coefficients F_a & F_v $F_a = 1.60$ [ASCE 7-16 Table 11.4-1 & 11.4-2](#)
(using straight-line interpolation from table values) $F_v = 2.40$

Maximum Considered Earthquake Acceleration $S_{MS} = F_a * S_s = 0.161$ [ASCE 7-16 Eq. 11.4-1](#)

$S_{M1} = F_v * S_1 = 0.165$ [ASCE 7-16 Eq. 11.4-2](#)

Design Spectral Acceleration $S_{DS} = S_{MS} * 2/3 = 0.107$ [ASCE 7-16 Eq. 11.4-3](#)

$S_{D1} = S_{M1} * 2/3 = 0.110$ [ASCE 7-16 Eq. 11.4-4](#)

Seismic Design Category = **B** [ASCE 7-16 Table 11.6-1 & -2](#)

Resisting System

[ASCE 7-16 Table 12.2-1](#)

Basic Seismic Force Resisting System . . .

Bearing Wall Systems

8. Intermediate reinforced masonry shear walls

Response Modification Coefficient "R" = 3.50 [Building height Limits :](#)

System Overstrength Factor "Wo" = 2.50

Deflection Amplification Factor "Cd" = 2.25

Category "A & B" Limit: No Limit
Category "C" Limit: No Limit
Category "D" Limit: Not Permitted
Category "E" Limit: Not Permitted
Category "F" Limit: Not Permitted

NOTE! See ASCE 7-16 for all applicable footnotes

Lateral Force Procedure

[ASCE 7-16 Section 12.8.2](#)

Equivalent Lateral Force Procedure

[The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-16 12.8](#)

Determine Building Period

[Use ASCE 12.8-7](#)

Structure Type for Building Period Calculation: All Other Structural Systems

"Ct" value = 0.020 "hn" : Height from base to highest level = 18.670 ft

"x" value = 0.75

"Ta" Approximate fundamental period using Eq. 12.8-7 : $T_a = C_t * (h_n^x) = 0.180$ sec

"TL" : Long-period transition period per ASCE 7-16 Maps 22-14 -> 22-17 = 8.000 sec

Building Period "Ta" Calculated from Approximate Method selected = 0.180

"Cs" Response Coefficient

[ASCE 7-16 Section 12.8.1.1](#)

S_{DS} : Short Period Design Spectral Response = 0.107 From Eq. 12.8-2, Preliminary $C_s = 0.031$

"R" : Response Modification Factor = 3.50 From Eq. 12.8-3 & 12.8-4, C_s need not exceed = 0.175

"I" : Seismic Importance Factor = 1 From Eq. 12.8-5 & 12.8-6, C_s not be less than = 0.010

C_s : Seismic Response Coefficient = 0.0306

Project Title:
Engineer:
Project ID:
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ASCE 7-16 Seismic Base Shear

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

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DESCRIPTION: Seismic Base Shear Analysis

Seismic Base Shear

ASCE 7-16 Section 12.8.1

Cs = 0.0306 from 12.8.1.1

W (see Sum Wi below) = 0.00 k

Seismic Base Shear $V = C_s * W = 0.00 \text{ k}$

Vertical Distribution of Seismic Forces

ASCE 7-16 Section 12.8.3

"k": hx exponent based on $T_a = 1.00$

Table of building Weights by Floor Level...

Level #	Wi : Weight	Hi : Height	(Wi * Hi^k)	Cvx	Fx=Cvx * V	Sum Story Shear	Sum Story Moment
Sum Wi =	0.00 k	Sum Wi * Hi =	0.00 k-ft	Total Base Shear =		0.00 k	
						Base Moment =	0.0 k-ft

Diaphragm Forces : Seismic Design Category "B" to "F"

ASCE 7-16 12.10.1.1

Level #	Wi	Fi	Sum Fi	Sum Wi	Fpx : Calcd	Fpx : Min	Fpx : Max	Fpx	Dsgn. Force
Wpx	Weight at level of diaphragm and other structure elements attached to it.								
Fi	Design Lateral Force applied at the level.								
Sum Fi	Sum of "Lat. Force" of current level plus all levels above								
MIN Req'd Force @ Level	$0.20 * S_{DS} * I * W_{px}$								
MAX Req'd Force @ Level	$0.40 * S_{DS} * I * W_{px}$								
Fpx : Design Force @ Level	$W_{px} * \text{SUM}(x \rightarrow n) Fi / \text{SUM}(x \rightarrow n) wi, x = \text{Current level}, n = \text{Top Level}$								

Project Title:
 Engineer:
 Project ID:
 Project Descr:

ASCE 7-16 Snow Loads

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

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DESCRIPTION: TYPICAL SNOW

Flat Roof Snow Loads

Description :

per ASCE 7-16, Chapter 7

Ground Snow Load, per Fig 7.2	20.00 psf	Roof Slope, Sec .7.3.4	1.20
Terrain Category	B (see ASCE 7-16 Section 26.7)	Roof Configuration	Monoslope
Exposure of Roof	Fully Exposed		
Ce : Exposure Factor, Table 7.3-	0.90		
Ct : Thermal Factor	1.0 : All not otherwise defined	pm, Minimum required	20.00 psf
Risk Category, per Table 1.5-	II	pf, Calculated Snow Load per Equatio	12.60 psf
Importance Factor, Is, Table 1.5-2	1.00	pf, Design Snow Load Max(pm min, pf calc	20.00 psf

Snow Drifts on Roof Projections

Description :

per ASCE 7-16, Chapt

Balanced Snow Load	20.00 psf	hd : windward	1.56 ft
Ground Snow Load	20.00 psf	hd : Design	1.56 ft
lu-upwind	45.00 ft	pd : Max Drift Only	25.89 psf
Height of Projection	9.00 ft	pd + Balanced	45.89 psf
Snow Density	16.60 pcf	W : Drift Width	6.24 ft
hb : Balanced	1.21 ft		
hc : Ht. of Projection - hb	7.80 ft		
hc / hb	6.47		
Importance Factor	1.00		

Description :

per ASCE 7-16, Chapt

Balanced Snow Load	20.00 psf	hd : windward	2.59 ft
Ground Snow Load	20.00 psf	hd : Design	2.59 ft
lu-upwind	119.67 ft	pd : Max Drift Only	43.07 psf
Height of Projection	9.00 ft	pd + Balanced	63.07 psf
Snow Density	16.60 pcf	W : Drift Width	10.38 ft
hb : Balanced	1.21 ft		
hc : Ht. of Projection - hb	7.80 ft		
hc / hb	6.47		
Importance Factor	1.00		

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

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DESCRIPTION: Beam 1 16-8

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

Material Properties

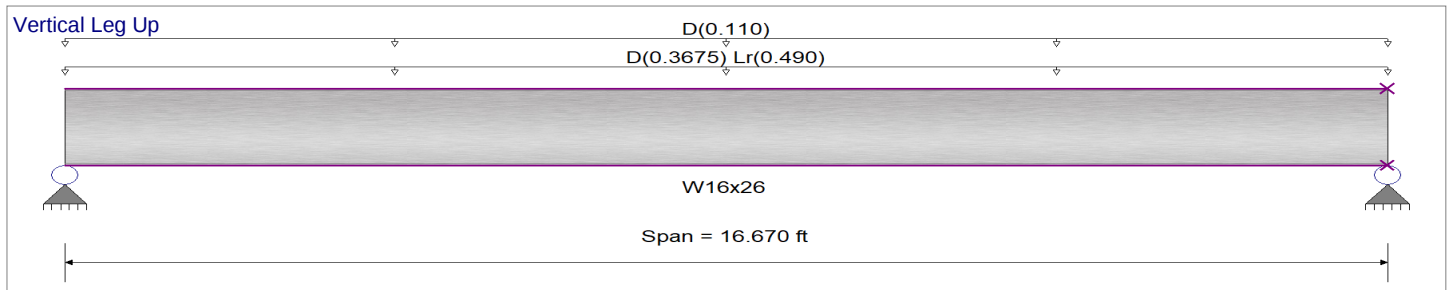
Analysis Method : Allowable Strength Design

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 NOT internally calculated and added

Uniform Load : D = 0.0150, Lr = 0.020 ksf, Tributary Width = 24.50 ft

Uniform Load : D = 0.0550 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.305 : 1	Maximum Shear Stress Ratio =	0.114 : 1
Section used for this span	W16x26	Section used for this span	W16x26
Ma : Applied	33.607 k-ft	Va : Applied	8.064 k
Mn / Omega : Allowable	110.279 k-ft	Vn/Omega : Allowable	70.509 k
Load Combination	+D+Lr	Load Combination	+D+Lr
Location of maximum on span	0.000 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Maximum Deflection			
Max Downward Transient Deflection	0.098 in	Ratio =	2.041 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.193 in	Ratio =	1034 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <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	Vnx	Vnx/Omega
D Only													
Dsgn. L = 16.67 ft	1	0.150	0.056	16.59		16.59	184.17	110.28	1.00	1.00	3.98	117.75	70.51
+D+Lr													
Dsgn. L = 16.67 ft	1	0.305	0.114	33.61		33.61	184.17	110.28	1.00	1.00	8.06	117.75	70.51
+D+0.750Lr													
Dsgn. L = 16.67 ft	1	0.266	0.100	29.35		29.35	184.17	110.28	1.00	1.00	7.04	117.75	70.51
+0.60D													
Dsgn. L = 16.67 ft	1	0.090	0.034	9.95		9.95	184.17	110.28	1.00	1.00	2.39	117.75	70.51

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.1935	8.383		0.0000	0.000

Vertical Reactions

Support notation : Far left is #:

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	8.064	8.064
Max Upward from Load Combinations	8.064	8.064
Max Upward from Load Cases	4.084	4.084
D Only	3.980	3.980
+D+Lr	8.064	8.064

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: [Beam 1 16-8](#)

Vertical Reactions	Support notation : Far left is #.		Values in KIPS
	Support 1	Support 2	
Load Combination			
+D+0.750Lr	7.043	7.043	
+0.60D	2.388	2.388	
Lr Only	4.084	4.084	

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: heartland market.ec6

LIC#: KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

(c) ENERCALC INC 1983-2022

DESCRIPTION: DOOR LINTEL LOAD BEARING

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

Material Properties

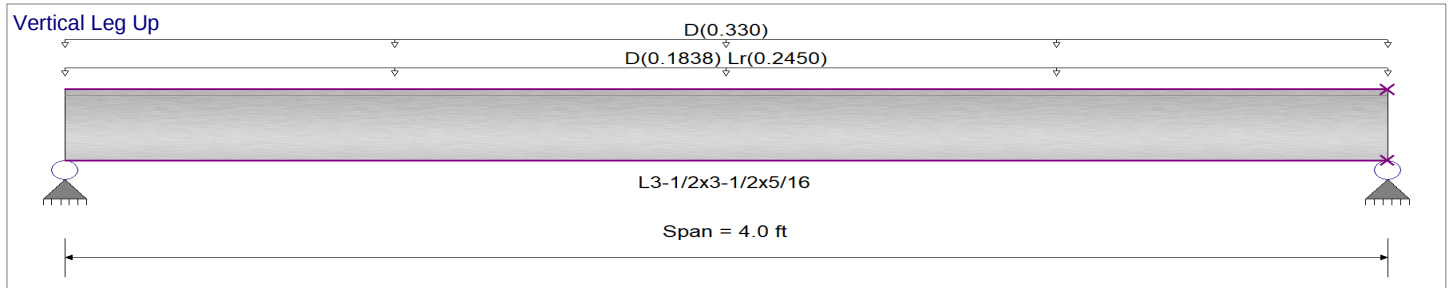
Analysis Method : Allowable Strength Design

Fy : Steel Yield : 36.0 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

Uniform Load : D = 0.0150, Lr = 0.020 ksf, Tributary Width = 12.250 ft

Uniform Load : D = 0.0550 ksf, Tributary Width = 6.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =			0.582 : 1			Maximum Shear Stress Ratio =			0.107 : 1		
Section used for this span			L3-1/2x3-1/2x5/16			Section used for this span			L3-1/2x3-1/2x5/16		
Ma : Applied			1.518 k-ft			Va : Applied			1.518 k		
Mn / Omega : Allowable			2.608 k-ft			Vn/Omega : Allowable			14.169 k		
Load Combination			+D+Lr			Load Combination			+D+Lr		
Location of maximum on span			0.000 ft			Location of maximum on span			0.000 ft		
Span # where maximum occurs			Span # 1			Span # where maximum occurs			Span # 1		
Maximum Deflection											
Max Downward Transient Deflection			0.020 in Ratio =			2,395			>=360		
Max Upward Transient Deflection			0.000 in Ratio =			0			<360		
									Span: 1 : Lr Only		
Max Downward Total Deflection			0.062 in Ratio =			774			>=180		
									Span: 1 : +D+Lr		
Max Upward Total Deflection			0.000 in Ratio =			0			<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	Vnx	Vnx/Omega
D Only													
Dsgn. L = 4.00 ft	1	0.394	0.073	1.03		1.03	4.36	2.61	1.00	1.00	1.03	23.66	14.17
+D+Lr													
Dsgn. L = 4.00 ft	1	0.582	0.107	1.52		1.52	4.36	2.61	1.00	1.00	1.52	23.66	14.17
+D+0.750Lr													
Dsgn. L = 4.00 ft	1	0.535	0.098	1.40		1.40	4.36	2.61	1.00	1.00	1.40	23.66	14.17
+0.60D													
Dsgn. L = 4.00 ft	1	0.236	0.044	0.62		0.62	4.36	2.61	1.00	1.00	0.62	23.66	14.17

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0620	2.011		0.0000	0.000

Vertical Reactions

Support notation : Far left is #:

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.518	1.518
Max Upward from Load Combinations	1.518	1.518
Max Upward from Load Cases	1.028	1.028
D Only	1.028	1.028
+D+Lr	1.518	1.518

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: DOOR LINTEL LOAD BEARING

Vertical Reactions	Support notation : Far left is #.		Values in KIPS
	Support 1	Support 2	
Load Combination			
+D+0.750Lr	1.395	1.395	
+0.60D	0.617	0.617	
Lr Only	0.490	0.490	

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

(c) ENERCALC INC 1983-2022

DESCRIPTION: WINDOW LINTEL NON LOAD BEARING

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Strength Design

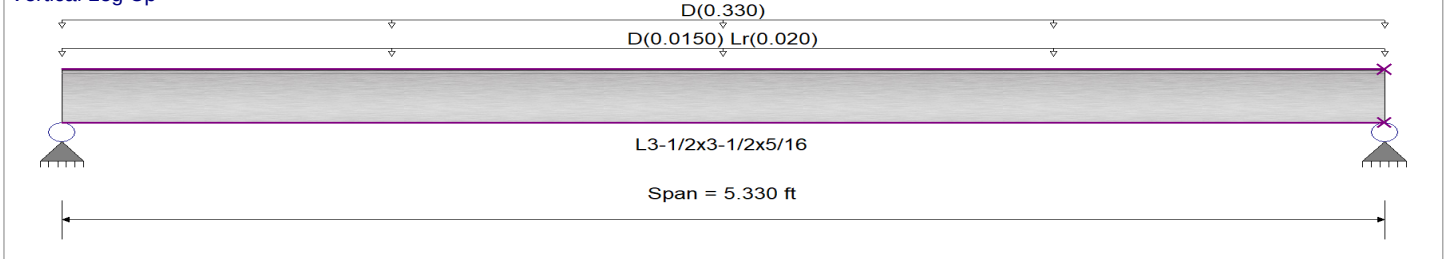
Fy : Steel Yield : 36.0 ksi

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

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending

Vertical Leg Up



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.0150, Lr = 0.020 ksf, Tributary Width = 1.0 ft

Uniform Load : D = 0.0550 ksf, Tributary Width = 6.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.497 : 1	Maximum Shear Stress Ratio =	0.069 : 1
Section used for this span	L3-1/2x3-1/2x5/16	Section used for this span	L3-1/2x3-1/2x5/16
Ma : Applied	1.296 k-ft	Va : Applied	0.9727 k
Mn / Omega : Allowable	2.608 k-ft	Vn/Omega : Allowable	14.169 k
Load Combination	+D+Lr	Load Combination	+D+Lr
Span # where maximum occurs	Span # 1	Location of maximum on span	0.000 ft
		Span # where maximum occurs	Span # 1
Maximum Deflection			
Max Downward Transient Deflection	0.005 in Ratio = 12.404		>=360
Max Upward Transient Deflection	0.000 in Ratio = 0		<360
Max Downward Total Deflection	0.094 in Ratio = 680		>=180
Max Upward Total Deflection	0.000 in Ratio = 0		<180

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
			M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only	Dsgn. L = 5.33 ft	1	0.470	0.065	1.23		1.23	4.36	2.61	1.00	1.00	0.92	23.66	14.17
+D+Lr	Dsgn. L = 5.33 ft	1	0.497	0.069	1.30		1.30	4.36	2.61	1.00	1.00	0.97	23.66	14.17
+D+0.750Lr	Dsgn. L = 5.33 ft	1	0.490	0.068	1.28		1.28	4.36	2.61	1.00	1.00	0.96	23.66	14.17
+0.60D	Dsgn. L = 5.33 ft	1	0.282	0.039	0.74		0.74	4.36	2.61	1.00	1.00	0.55	23.66	14.17

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0941	2.680		0.0000	0.000

Vertical Reactions

Support notation : Far left is #:

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.973	0.973
Max Upward from Load Combinations	0.973	0.973
Max Upward from Load Cases	0.919	0.919
D Only	0.919	0.919
+D+Lr	0.973	0.973

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: WINDOW LINTEL NON LOAD BEARING

Vertical Reactions

Support notation : Far left is #.

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750Lr	0.959	0.959
+0.60D	0.552	0.552
Lr Only	0.053	0.053

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Masonry Beam

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.22.10.25

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: WINDOW BOND BEAM

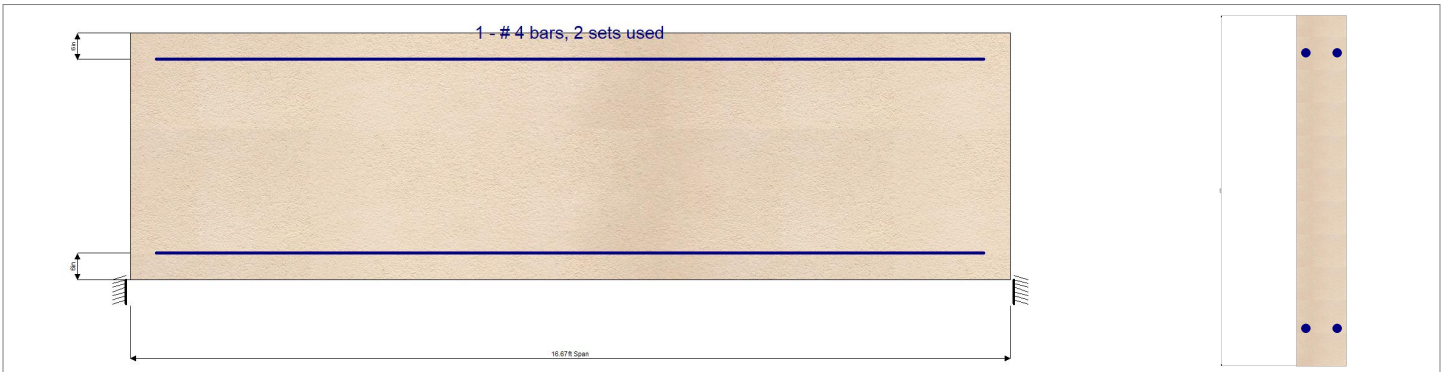
Code References

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combinations Used : ASCE 7-16

General Information

f'm	1,500.0 psi	Clear Span	16.670 ft	Rebar Size	4.0
Fs	24,000.0 psi	Beam Depth	4.670 ft	# Bars E/F	1
Em = f'm *	900.0	Thickness	8 in	Top Clear	6.0 in
Wall Wt Mult.	1.0	End Fixity	Fix-Fix	Btm Clear	6.0 in
Block Type	Normal Wt	Equiv. Solid Thick	7.60 in	# Bar Sets	2
Lateral Wind Load	25.0 psf	Wall Weight	86.0 psf	Bar Spacing	5.0 in
Beam is Fully Braced ?	Yes	E	1,350.0 ksi		
Lateral Wall Weight Seismic Factor	0.0330	n	21.481		
Calculate vertical beam weight ?	No				



DESIGN SUMMARY

Design OK

Maximum Stress Ratios...

	Vertical	Lateral	SRSS Combination
fb/Fb	0.0	0.3439	0.3439 : 1.00
fv/Fv	0.0	0.04262	0.04262 : 1.00

Maximum Moment

	Actual	Allowable
Vertical Loads	0.0 k-ft	0.0 k-ft
for Load Combination :		
Lateral Loads	1.622 k-ft	4.718
for Load Combination :		

Maximum Shear

	Actual	Allowable
Vertical Loads	0.0 psi	0.0 psi
for Load Combination :		
Lateral Loads	1.651 psi	38.730 psi
for Load Combination :		

Minimum Mn = 1.3 * Fcr * S = 43.236 k-ft

Vertical Strength

As	0.40 in^2
rho	0.001048
np	0.02252
k : ((np)^2+2np)^.5-np	0.1909
j = 1 - k/3	0.9364
M:mas=Fb k j b d^2/2	95.987 k-ft
M:Stl = Fs As j d	37.485 k-ft

Lateral Strength

(Checking lateral bending for span)

As	0.40 in^2
rho	0.001131
np	0.02429
k : ((np)^2+2np)^.5-np	0.1975
j = 1 - k/3	0.9342
M:mas=Fb k j b d^2/2	11.585 k-ft
M:Stl = Fs As j d	4.718 k-ft

Detailed Load Combination Results

Load Combinatic	Vertical				Lateral			
	Mmax k-ft	Mallow k-ft	fv : Vert psi	Fv : Vert psi	Mactual k-ft	Mallow k-ft	fv psi	Fv psi
	0.00	37.48	0.00	77.46	0.00	4.72	0.00	38.73
+0.60W	0.00	37.48	0.00	77.46	1.62	4.72	1.65	38.73
+0.450W	0.00	37.48	0.00	77.46	1.22	4.72	1.24	38.73

Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Beam

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.22.10.25

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: WINDOW BOND BEAM

Detailed Load Combination Results

Load Combinatic	Vertical				Lateral			
	Mmax k-ft	Mallow k-ft	fv : Vert psi	Fv : Vert psi	Mactual k-ft	Mallow k-ft	fv psi	Fv psi
E Only * 0.70	0.00	37.48	0.00	77.46	0.21	4.72	0.22	38.73
E Only * 0.5250	0.00	37.48	0.00	77.46	0.16	4.72	0.16	38.73

Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Slender Wall

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.22.10.25

J&S STRUCTURAL ENGINEERS, P.A.

(c) ENERCALC INC 1983-2022

DESCRIPTION: -TPYICAL MASONRY WALL

Code References

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combinations Used : ASCE 7-16

General Information

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16

Construction Type : Grouted Hollow Concrete Masonry

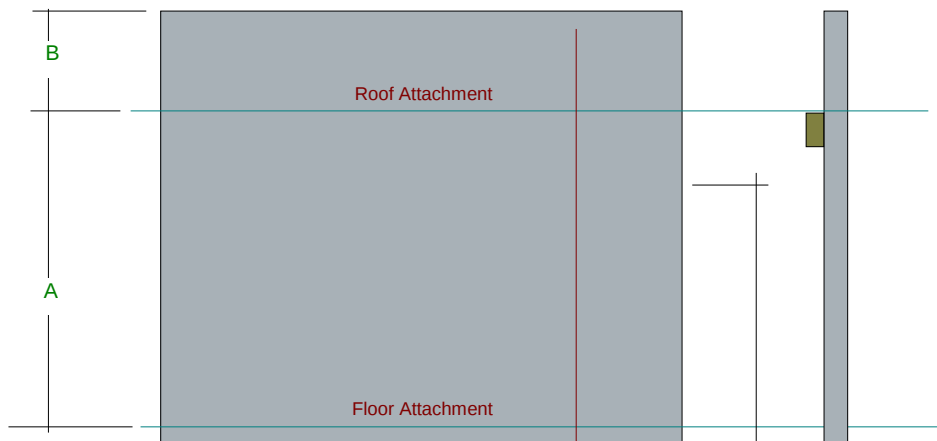
F'm	=	1.50 ksi	Nom. Wall Thickness	8 in	Temp Diff across thickness	=	deg F
Fy - Yield	=	60.0 ksi	Actual Thickness	7.625 in	Min Allow Out-of-plane Defl Ra	=	0.0
Fr - Rupture	=	61.0 psi	Rebar "d" distance	3.8125 in	Minimum Vertical Steel %	=	0.0020
Em = f'm *	=	900.0	Lower Level Rebar . . .				
Max % of ρ bal.	=	0.006824	Bar Size	# 5			
Grout Density	=	140 pcf	Bar Spacing	48 in			
Block Weight		Normal Weight					
Wall Weight	=	47.0 psf					

Wall is grouted at rebar cells only

One-Story Wall Dimensions

A Clear Height	=	12.67 ft
B Parapet height	=	ft

Wall Support ConditionTop & Bottom Pinned



Vertical Loads

Vertical Uniform Loads . . . (Applied per foot of Strip Width)

Ledger Load	Eccentricity	6.750 in	DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind
Concentric Load			.94	.94			k/ft

Lateral Loads

Wind Loads :

Full area WIND load 20 psf

Seismic Loads :

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

Seismic Wall Lateral Load 12 psf

	Fp	1.0	=	12.0 psf	
	D	Lr	L	E	W
Point Lateral Load					Height
Point Lateral Load					(Applied to full "STRIP Width")

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Masonry Slender Wall

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.22.10.25

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: -TPYICAL MASONRY WALL

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

Governing Load Combination . . .		Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +0.90D+W	Maximum Bending Stress Ratio0.2785			
		Max Mu	0.4028 k-ft	Phi * Mn	1.446 k-ft
PASS	Service Deflection Check W Only	Actual Defl. Ratio L/	5,874	Allowable Defl. Ratio	150.0
		Max. Deflection	0.02589 in		
PASS	Axial Load Check +1.20D+0.50Lr+W	Max Pu / Ag	48.651 psi	Max. Allow. Defl.	1.014 in
		Location	6.124 ft	0.2 * f'm	300.0 psi
	Reinforcing Limit Check				
		Actual As/bd	0.001694	Max Allow As/bd	0.006824
		Maximum Reactions for Load Combination...			
		Top Horizontal	W Only		0.1267 k
		Base Horizontal	W Only		0.1267 k
		Vertical Reaction	+D+Lr		2.475 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load		Mcr k-ft	Mu k-ft	Moment Values			As Ratio	0.6 * rho bal	Bar 'd'
	Pu k	0.2*f'm*b*t k			Phi	Phi Mn k-ft	As in^2			
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
+1.20D+1.60Lr+0.50W at 5.91 to 6.34	3.013	12.204	0.44	0.20	0.90	1.93	0.078	0.0017	0.0060	0.00
+1.20D+0.50W at 5.91 to 6.34	1.509	12.204	0.44	0.20	0.90	1.55	0.078	0.0017	0.0066	0.00
+1.20D+0.50Lr+W at 5.91 to 6.34	1.979	12.204	0.44	0.40	0.90	1.67	0.078	0.0017	0.0064	0.00
+1.20D+W at 5.91 to 6.34	1.509	12.204	0.44	0.40	0.90	1.55	0.078	0.0017	0.0066	0.00
+0.90D+W at 5.91 to 6.34	1.132	12.204	0.44	0.40	0.90	1.45	0.078	0.0017	0.0067	0.00
+1.20D+E at 5.91 to 6.34	1.509	12.204	0.44	0.24	0.90	1.55	0.078	0.0017	0.0066	0.00
+0.90D+E at 5.91 to 6.34	1.132	12.204	0.44	0.24	0.90	1.45	0.078	0.0017	0.0067	0.00

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load		Moment Values		Stiffness			Deflections	
	Pu k		Mcr k-ft	Mactual k-ft	I gross in^4	I cracked in^4	I effective in^4	Deflection in	Defl. Ratio
	0.000		0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000		0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000		0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.60W at 5.91 to 6.34	1.258		0.44	0.24	331.10	20.73	331.100	0.016	9,725.7
+D+0.750Lr+0.450W at 5.91 to 6.34	1.963		0.44	0.18	331.10	22.70	331.100	0.012	12,919.4
+D+0.450W at 5.91 to 6.34	1.258		0.44	0.18	331.10	20.73	331.100	0.012	12,967.6
+0.60D+0.60W at 5.91 to 6.34	0.755		0.44	0.24	331.10	19.28	331.100	0.016	9,751.1
+D+0.70E at 5.91 to 6.34	1.258		0.44	0.17	331.10	20.73	331.100	0.011	13,893.8
+D+0.5250E at 5.91 to 6.34	1.258		0.44	0.13	331.10	20.73	331.100	0.008	18,525.1
+0.60D+0.70E at 5.91 to 6.34	0.755		0.44	0.17	331.10	19.28	331.100	0.011	13,930.1
	0.000		0.00	0.00	0.00	0.00	0.000	0.000	0.0
W Only at 6.34 to 6.76	0.000		0.44	0.40	331.10	17.04	331.100	0.026	5,873.5
E Only at 6.34 to 6.76	0.000		0.44	0.24	331.10	17.04	331.100	0.016	9,789.2

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	1.535 k

Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Slender Wall

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.22.10.25

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: -TPYICAL MASONRY WALL

+D+Lr	0.0 k	0.00 k	2.475 k
+D+0.750Lr	0.0 k	0.00 k	2.240 k
+D+0.60W	0.1 k	0.08 k	1.535 k

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
+D+0.750Lr+0.450W	0.1 k	0.06 k	2.240 k
+D+0.450W	0.1 k	0.06 k	1.535 k
+0.60D+0.60W	0.1 k	0.08 k	0.921 k
+D+0.70E	0.1 k	0.05 k	1.535 k
+D+0.5250E	0.0 k	0.04 k	1.535 k
+0.60D+0.70E	0.1 k	0.05 k	0.921 k
Lr Only	0.0 k	0.00 k	0.940 k
W Only	0.1 k	0.13 k	0.000 k
E Only	0.1 k	0.08 k	0.000 k

Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Slender Wall

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.22.10.25

J&S STRUCTURAL ENGINEERS, P.A.

(c) ENERCALC INC 1983-2022

DESCRIPTION: TYP WINDOW JAMB

Code References

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combinations Used : ASCE 7-16

General Information

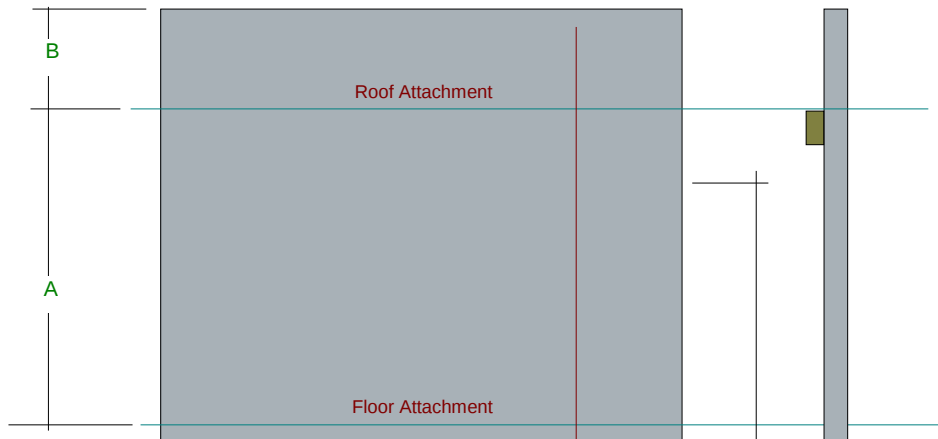
Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16

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 Ra	=	0.0
Fr - Rupture	=	61.0 psi	Rebar "d" distance	3.8125 in	Minimum Vertical Steel %	=	0.0020
Em = f'm *	=	900.0	Lower Level Rebar . . .				
Max % of ρ bal.	=	0.006819	Bar Size	# 5			
Grout Density	=	140 pcf	Bar Spacing	8 in			
Block Weight	=	Light Weight					
Wall Weight	=	78.0 psf					
Wall is Solid Grouted							

One-Story Wall Dimensions

A Clear Height	=	12.670 ft
B Parapet height	=	ft
Wall Support ConditionTop & 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					k/ft
Concentric Load		0.940	0.940			k/ft

Lateral Loads

Wind Loads :

Full area WIND load 20.0 psf

Seismic Loads :

Wall Weight Seismic Load Input Method : Direct entry of Lateral Wall Weight
Seismic Wall Lateral Load 12.0 psf

	Fp	1.0	=	12.0 psf	
	D	Lr	L	E	W
Point Lateral Load					Height (Applied to full "STRIP Width")
Point Lateral Load					.431 k
Point Lateral Load					4.67 ft
Point Lateral Load					.431 k
Point Lateral Load					10.17 ft

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Masonry Slender Wall

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.22.10.25

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: TYP WINDOW JAMB

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

Governing Load Combination . . .		Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +1.20D+0.50Lr+W	Maximum Bending Stress Ratio0.3639			
		Max Mu	2.111 k-ft	Phi * Mn	5.801 k-ft
PASS	Service Deflection Check W Only	Actual Defl. Ratio L/	278	Allowable Defl. Ratio	150.0
		Max. Deflection	0.5478 in		
PASS	Axial Load Check +1.20D+0.50Lr+W	Max Pu / Ag	25.641 psi	Max. Allow. Defl.	1.014 in
		Location	4.857 ft	0.2 * f'm	300.0 psi
	Reinforcing Limit Check				
		Actual As/bd	0.01016	Max Allow As/bd	0.006822
		Maximum Reactions for Load Combination...			
		Top Horizontal W Only			0.6315 k
		Base Horizontal W Only			0.4839 k
		Vertical Reaction +D+Lr			2.868 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load		Mcr k-ft	Mu k-ft	Moment Values			As Ratio	0.6 * rho bal	Bar 'd'
	Pu k	0.2*f'm*b*t k			Phi	Phi Mn k-ft	As in^2			
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
+1.20D+1.60Lr+0.50W at 4.65 to 5.07	3.383	27.468	0.59	1.05	0.90	5.85	0.465	0.0102	0.0059	0.00
+1.20D+0.50W at 4.65 to 5.07	1.879	27.468	0.59	1.04	0.90	5.78	0.465	0.0102	0.0065	0.00
+1.20D+0.50Lr+W at 4.65 to 5.07	2.348	27.468	0.59	2.11	0.90	5.80	0.465	0.0102	0.0063	0.00
+1.20D+W at 4.65 to 5.07	1.878	27.468	0.59	2.09	0.90	5.78	0.465	0.0102	0.0065	0.00
+0.90D+W at 4.65 to 5.07	1.408	27.468	0.59	2.08	0.90	5.75	0.465	0.0102	0.0066	0.00
+1.20D+E at 5.91 to 6.34	1.760	27.468	0.59	0.24	0.90	5.77	0.465	0.0102	0.0065	0.00
+0.90D+E at 5.91 to 6.34	1.320	27.468	0.59	0.24	0.90	5.75	0.465	0.0102	0.0067	0.00

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values		I gross in^4	Stiffness		Deflections	
		Mcr k-ft	Mactual k-ft		I cracked in^4	I effective in^4	Deflection in	Defl. Ratio
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.60W at 5.91 to 6.34	1.467	0.59	1.21	443.30	65.82	67.740	0.254	597.9
+D+0.750Lr+0.450W at 5.91 to 6.34	2.172	0.59	0.91	443.30	66.78	72.517	0.138	1,101.1
+D+0.450W at 5.91 to 6.34	1.467	0.59	0.90	443.30	65.82	71.707	0.137	1,112.1
+0.60D+0.60W at 5.91 to 6.34	0.880	0.59	1.19	443.30	65.01	66.979	0.253	601.7
+D+0.70E at 5.91 to 6.34	1.467	0.59	0.17	443.30	65.82	443.300	0.008	18,618.2
+D+0.5250E at 5.91 to 6.34	1.467	0.59	0.13	443.30	65.82	443.300	0.006	24,824.2
+0.60D+0.70E at 5.91 to 6.34	0.880	0.59	0.17	443.30	65.01	443.300	0.008	18,660.3
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
W Only at 6.34 to 6.76	0.000	0.59	1.93	443.30	63.77	64.155	0.548	277.5
E Only at 6.34 to 6.76	0.000	0.59	0.24	443.30	63.77	443.300	0.012	13,106.4

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	1.928 k

Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Slender Wall

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.22.10.25

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: TYP WINDOW JAMB

+D+Lr	0.0 k	0.00 k	2.868 k
+D+0.750Lr	0.0 k	0.00 k	2.633 k
+D+0.60W	0.3 k	0.38 k	1.928 k

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
+D+0.750Lr+0.450W	0.2 k	0.28 k	2.633 k
+D+0.450W	0.2 k	0.28 k	1.928 k
+0.60D+0.60W	0.3 k	0.38 k	1.157 k
+D+0.70E	0.1 k	0.05 k	1.928 k
+D+0.5250E	0.0 k	0.04 k	1.928 k
+0.60D+0.70E	0.1 k	0.05 k	1.157 k
Lr Only	0.0 k	0.00 k	0.940 k
W Only	0.5 k	0.63 k	0.000 k
E Only	0.1 k	0.08 k	0.000 k

Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Shear Wall

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

(c) ENERCALC INC 1983-2022

DESCRIPTION: worst case shearwall

Code References

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

General Information

Wall Material	MASONRY	f'm	1.50 ksi	Block Class	
Total Wall Height	12.670 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	13.50 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	13.50 ft
Effective Length 'd'	158.0 in
Nominal Block Thickness	8 in
Solid Grout?	Partial Groute

Reinforcing in Field of Wall

Vertical Bar Size #	5
Vertical Bar Spacing	48 in
Horiz. joint reinf. area (HJR)	.02 in
HJR Spacing	16 in
Bond beam reinf. area	.0006 in
Spacing of bond beams	48 in

In each chord cell:

Vertical rebar size #	5
# Chord Cells @ Each End	1.0

Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Shear Wall

Project File: heartland market.ec6

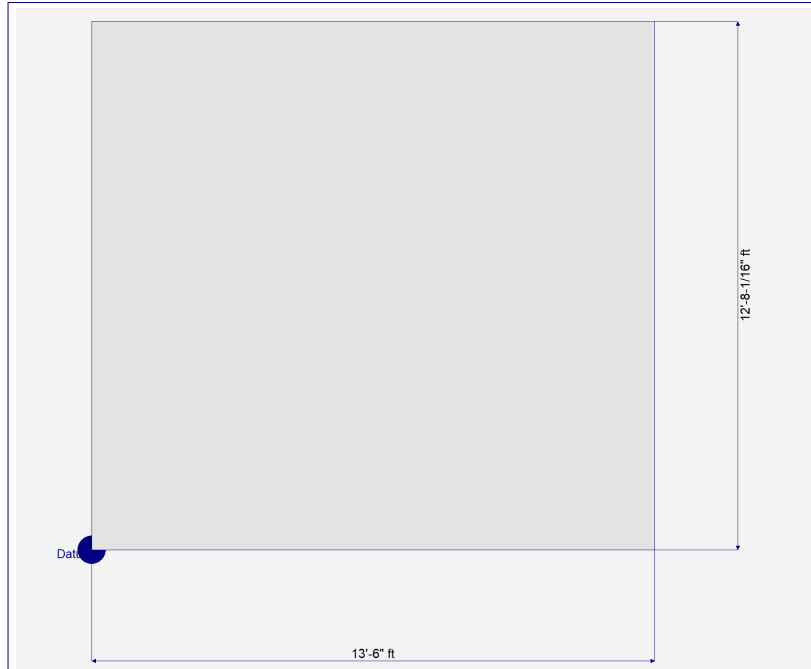
LIC# : KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: worst case shearwall

Wall Sketch



SHEAR ANALYSIS

Bottom Level

Special Boundary	
Elements Req'd?	Not Req'd
Vu : Story Shear	10.340 k
for Load Combination	+1.20D+0.50Lr+W
Controlling Mu/(Vud)	1.00
Vn Masonry	73.268 k
Vn Steel	5.367 k
Vn Masonry + Vn Steel	78.635 k
Vn Max	92.162 k
Phi Vn	62.908 k
Ratio: Vu/PhiVn (controlling)	0.1644
Vertical As >= Av/3	OK
Vertical Bar Spacing <= 96"	OK

AXIAL ANALYSIS

Bottom Level

H / d Ratio	0.96
Pu	16.588 k
for Load Combination	+1.20D+1.60Lr
Phi Pn	1.20D+1.60Lr k
Ratio: Pu/PhiPn (controlling)	0.01875

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Masonry Shear Wall

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: worst case shearwall

BENDING ANALYSIS

Bottom Level

"a" : Flexural compression 2.03 in
 Length of defined chord zone
 is >= the "a" dimension of the masonry (the compression zone) OK
 "d" : Eff depth to tension reinf 158.0
 As-flex < As-max ? 10 <= 10.676
 Mu 188.803 k
 for Load Combination +1.20D+0.50Lr+W
 Phi Mn 218.992 k
 Ratio: Mu/PhiMn (controlling) **0.8621**

Force Summary

Load Combination Wall Level	Values for Wall section			Resultant Ecc (ft)	Overturning Ratio	Uplift (k)	
	Vu (k)	Mu (k)	Pu (k)			Left	Right
+1.40D							
Wall Level : 1		22.050	13.753	1.603			
+1.20D+0.50Lr							
Wall Level : 1		26.775	13.288	2.015			
+1.20D							
Wall Level : 1		18.900	11.788	1.603			
+1.20D+1.60Lr							
Wall Level : 1		44.100	16.588	2.658			
+1.20D+1.60Lr+0.50W							
Wall Level : 1	5.170	125.114	16.588	7.542	0.724	3.612	3.612
+1.20D+0.50W							
Wall Level : 1	5.170	99.914	11.788	8.476	0.635	3.065	3.065
+1.20D+0.50Lr+W							
Wall Level : 1	10.340	188.803	13.288	14.208	0.332	9.389	9.389
+1.20D+W							
Wall Level : 1	10.340	180.928	11.788	15.348	0.318	9.218	9.218
+0.90D+W							
Wall Level : 1	10.340	176.203	8.841	19.930	0.309	9.115	9.115
+1.20D+E							
Wall Level : 1		18.900	11.788	1.603			
+0.90D+E							
Wall Level : 1		14.175	8.841	1.603			

Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Shear Wall

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

(c) ENERCALC INC 1983-2022

DESCRIPTION: worst case shearwall

Footing Information

Footing Dimensions

Dist. Left	1.0 ft	f'c	3.0 ksi	Rebar Cover	3.0 in
Wall Length	13.50 ft	Fy	60.0 ksi	Footing Thickness	32.0 in
Dist. Right	1.0 ft			Width	1.330 ft
Total Ftg Length	15.50 ft				

Max Factored Soil Pressures

@ Left Side of Footing	813.70 psf
.... governing load comb	+1.40D
@ Right Side of Footing	5,296.61 psf
.... governing load comb	+1.20D+1.60Lr+0.50W

Max UNfactored Soil Pressures

@ Left Side of Footing	581.21 psf
.... governing load comb	D Only
@ Right Side of Footing	15,458.8 psf
.... governing load comb	D+0.60W

Footing One-Way Shear Check...

vu @ Left End of Footing	0.0 psi
vu @ Right End of Footing	0.0 psi
vn * phi : Allowable	93.113 psi

Overturning Stability... @ Left End of Ftg

Overturning Moment	113.761 k-ft
Resisting Moment	85.805 k-ft
Stability Ratio	0.7543 : 1
.... governing load comb	+0.60D+0.60W

@ Right End of Ftg

Overturning Moment	113.761 k-ft
Resisting Moment	66.905 k-ft
Stability Ratio	0.5881 : 1
.... governing load comb	+0.60D+0.60W

Footing Bending Design...

@ Left End

Mu	0.5530 k-ft
Ru	0.5493 psi
As % Req'd	0.00180 in^2
As Req'd in Footing Width	0.9193 in^2

@ Right End

Mu	3.366 k-ft
Ru	3.344 psi
As % Req'd	0.00180 in^2
As Req'd in Footing Width	0.9193 in^2

Project Title:
Engineer:
Project ID:
Project Descr:

Wall Footing

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

(c) ENERCALC INC 1983-2022

DESCRIPTION: Typical footing

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16

Load Combinations Used : ASCE 7-16

General Information

Material Properties

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

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
AutoCalc Footing Weight as DL :	=	No

Soil Design Values

Allowable Soil Bearing	=	1.0 ksf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

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

Increases based on footing Width

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

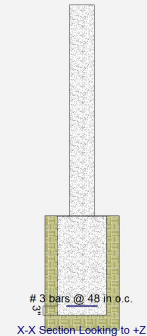
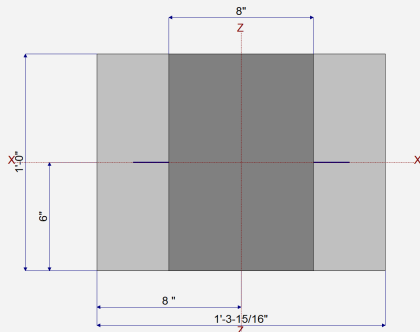
Adjusted Allowable Bearing Pressure

=	1.0 ksf
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Dimensions

Reinforcing

Footing Width	=	1.33 ft	Footing Thickness	=	32.0 in	Bars along X-X Axis	
Wall Thickness	=	8.0 in	Rebar Centerline to Edge of Concrete...			Bar spacing	= 48.00
Wall center offset			at Bottom of footing =	3.0 in		Reinforcing Bar Size	= # 3
from center of footing	=	0 in					



Applied Loads

	D	Lr	L	S	W	E	H
P : Column Load	=	0.450	0.450				k
OB : Overburden	=						ksf
V-x	=						k
M-zz	=						k-ft
Vx applied	=						in above top of footing

Applied pressure is
great than 1/3
therefore design ok

Project Title:
Engineer:
Project ID:
Project Descr:

Wall Footing

Project File: heartland market.ec6

LIC# : KW-06017929, Build:20.23.2.14

J&S STRUCTURAL ENGINEERS, P.A.

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DESCRIPTION: Typical footing

DESIGN SUMMARY

Design N.G.

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

	Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.6767	Soil Bearing	0.6767 ksf	1.0 ksf	+D+Lr
FAIL	As < Min	Z Flexure (+X)	0.02605 k-ft	3.585 k-ft	+1.40D
FAIL	As < Min	Z Flexure (-X)	0.02605 k-ft	3.585 k-ft	+1.40D
PASS	n/a	1-way Shear (+X)	0.0 psi	82.158 psi	n/a
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Actual Soil Bearing Stress		Actual / Allowable Ratio
			-X	+X	
, D Only	1.0 ksf	0.0 in	0.3383 ksf	0.3383 ksf	0.338
, +D+Lr	1.0 ksf	0.0 in	0.6767 ksf	0.6767 ksf	0.677
, +D+0.750Lr	1.0 ksf	0.0 in	0.5921 ksf	0.5921 ksf	0.592
, +0.60D	1.0 ksf	0.0 in	0.2030 ksf	0.2030 ksf	0.203

Units : k-ft

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
-------------------------------------	--------------------	------------------	-----------------	--------

Footing Has NO Overturning

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio	Status
--	---------------	-----------------	---------------------	--------

Footing Has NO Sliding

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Which Side ?	Tension @ Bot. or Top ?	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
, +1.40D	0.02605	-X	Bottom	0.6912	Min Temp %	0.0275	3.585	OK
, +1.40D	0.02605	+X	Bottom	0.6912	Min Temp %	0.0275	3.585	OK
, +1.20D+0.50Lr	0.03164	-X	Bottom	0.6912	Min Temp %	0.0275	3.585	OK
, +1.20D+0.50Lr	0.03164	+X	Bottom	0.6912	Min Temp %	0.0275	3.585	OK
, +1.20D	0.02233	-X	Bottom	0.6912	Min Temp %	0.0275	3.585	OK
, +1.20D	0.02233	+X	Bottom	0.6912	Min Temp %	0.0275	3.585	OK
, +1.20D+1.60Lr	0.05211	-X	Bottom	0.6912	Min Temp %	0.0275	3.585	OK
, +1.20D+1.60Lr	0.05211	+X	Bottom	0.6912	Min Temp %	0.0275	3.585	OK
, +0.90D	0.01675	-X	Bottom	0.6912	Min Temp %	0.0275	3.585	OK
, +0.90D	0.01675	+X	Bottom	0.6912	Min Temp %	0.0275	3.585	OK

Units : k

One Way Shear

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