STRUCTURAL CALCULATIONS

WALMART EXPANSION, STORE #2403-278

PUYALLUP, WA

WD PROJECT NO: WALGP0402

MARCH 16, 2022

EXPIRES: 10-23-2023

PREPARED BY

WD PARTNERS

7007 DISCOVERY BLVD

DUBLIN OH 43017

614.634.7000 T

614.634.7777 F

Adopted Codes

New Building and Fire Code Effective Date Extended

The State of Washington has extended the date for adoption of the 2018 International Building and Fire Codes from July 1, 2020 to February 1, 2021. Customers who submit complete permit applications prior to February 1, 2021, will be reviewed to the current 2015 code requirements. Permit applications submitted after that date will need to comply with the new 2018 updated codes.

Building Department Codes

The State of Washington has adopted and amended the following building construction codes, effective February 1, 2021:

- 2018 International Building Code
- 2018 International Residential Code
- 2018 International Mechanical Code
- 2018 Uniform Plumbing Code
- 2018 International Fire Code
- 2018 Washington State Energy Code
- 2016 NFPA Standard 72
- 2016 NFPA Standard 13, 13-D, and 13-R

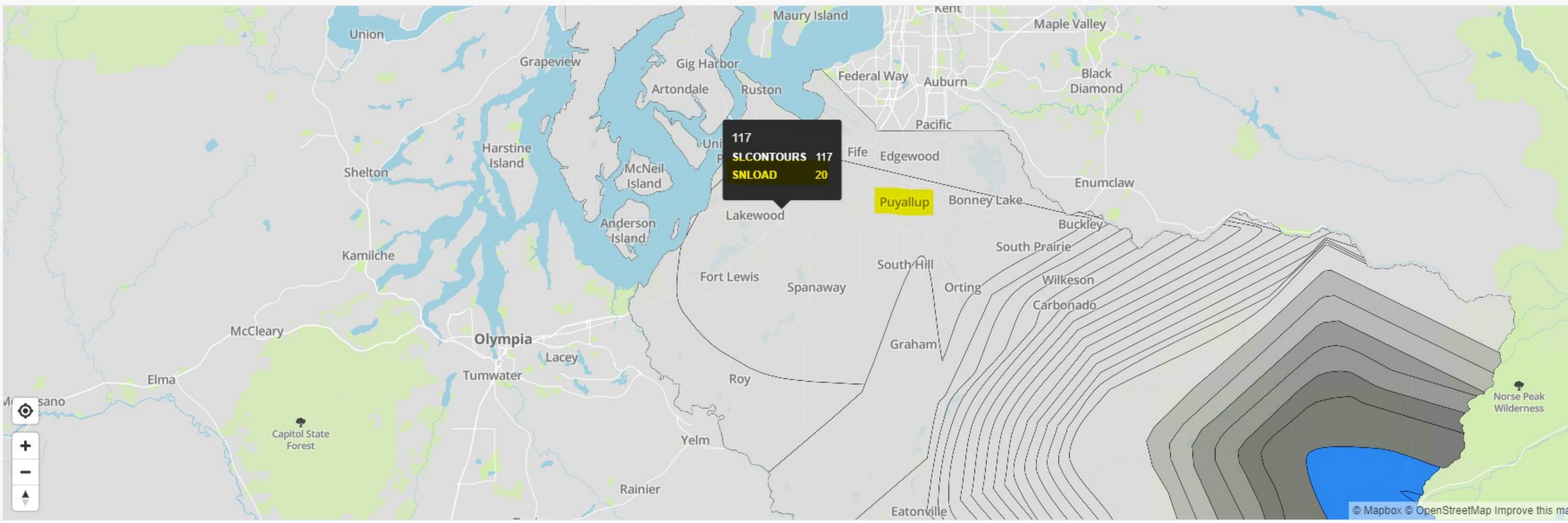
Online Link to International Codes with Washington State Amendments
See Puyallup Municipal Code for local amendments.

The City of Puyallup is in the process of implementing these codes locally, which includes the creation of an adopting and amending ordinance, updating department forms and brochures, and staff training.

This site will be providing information regarding these implementation processes and identifying some of the more significant differences between the new codes and our current building construction codes. Look for more information in the near future as it becomes available.

Snow Load

More Info



ATC Hazards by Location

Search Information

Address:	310 31st Ave SE, Puyallup, WA 98374, USA
Coordinates:	47.1610458, -122.2888112
Elevation:	442 ft
Timestamp:	2021-08-30T16:11:56.723Z
Hazard Type:	Wind

ASCE 7-16

MRI 10-Year	67 mph
MRI 25-Year	73 mph
MRI 50-Year	78 mph
MRI 100-Year	82 mph
Risk Category I	92 mph
Risk Category II	97 mph
Risk Category III	104 mph
Risk Category IV	108 mph

ASCE 7-10

MRI 10-Year	72 mph
MRI 25-Year	79 mph
MRI 50-Year	85 mph
MRI 100-Year	91 mph
Risk Category I	100 mph
Risk Category II	110 mph
Risk Category III-IV	115 mph

ASCE 7-05

ASCE 7-05 Wind Speed	85 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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ATC Hazards by Location

Search Information

Address: 310 31st Ave SE, Puyallup, WA 98374, USA

Coordinates: 47.1610458, -122.2888112

Elevation: 442 ft

Timestamp: 2021-09-08T09:21:10.416Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default

Basic Parameters

Name	Value	Description
S_S	1.261	MCE_R ground motion (period=0.2s)
S_1	0.435	MCE_R ground motion (period=1.0s)
S_{MS}	1.513	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.009	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.914	Coefficient of risk (0.2s)
CR_1	0.898	Coefficient of risk (1.0s)
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.6	Site modified peak ground acceleration

T_L	6	Long-period transition period (s)
SsRT	1.261	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.38	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.435	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.484	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

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Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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DESIGN LOADS

1. BUILDING CODE	
A. BUILDING CODE	2018 WASHINGTON STATE BUILDING CODE (ASCE 7-16)
2. GRAVITY LOADS	
A. ROOF DEAD LOAD	17.8 PSF (PER EXISTING DRAWINGS)
B. ROOF LIVE LOADS	
1. ROOF	20 PSF (MIN OR SNOW LOAD)
C. ROOF SNOW LOADS	
1. GROUND SNOW LOAD (Pg)	20 PSF
2. IMPORTANCE FACTOR (I)	1.0
3. SNOW EXPOSURE FACTOR (Ce)	1.0
4. ROOF THERMAL FACTOR (Ct)	1.0
5. FLAT ROOF SNOW LOAD (Pf) (PER CODE)	14.0
3. LATERAL LOADS	
A. WIND LOADS	
1. BASIC WIND SPEED (3-SECOND GUST)	
- ULTIMATE DESIGN WIND SPEED	97 MPH
- BASIC DESIGN WIND SPEED (SERVICE)	75.14 MPH
2. WIND EXPOSURE CATEGORY	C
3. RISK CATEGORY	II
B. SEISMIC LOADS (SERVICE)	
1. 5% DAMPED MAPPED ACCELERATION PARAMETER (Ss)	1.261
2. 1-SEC PERIOD MAPPED ACCELERATION PARAMETER (S1)	0.435
3. 5% DAMPED SPECTRAL RESPONSE COEFF. (Sds)	0.841
4. 1-SEC PERIOD SPECTRAL RESPONSE COEFF. (Sd1)	0.541 (REFER TO EXCEPTION 2 OF ASCE 7-16 SECTION 11.4.8 FOR MINIMUM PERIOD T USED FOR Cs EQUATIONS IN SECTION 12.8)
5. SITE CLASS	D (SOILS REPORT)
6. RISK CATEGORY	II
7. IMPORTANCE FACTOR (Ie)	1.0
8. SEISMIC DESIGN CATEGORY	D
9. SEISMIC RESISTING SYSTEM	SPECIAL REINFORCED MASONRY SHEAR WALLS

1

WD Partners	JOB TITLE	Walmart Renovation	
7007 Dsicovery Blvd		Store 2403-278	
Dublin, OH	JOB NO.	WALGP0402	SHEET NO.
wdpartners.com	CALCULATED BY		DATE
	CHECKED BY		DATE

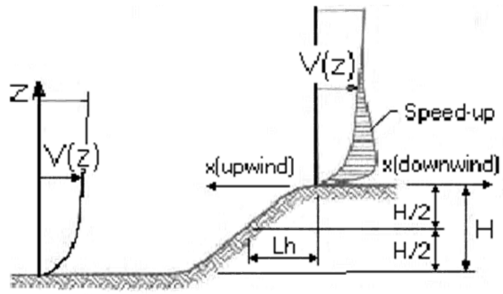
Wind Loads : ASCE 7- 16

Ultimate Wind Speed	97 mph
Nominal Wind Speed	75.1 mph
Risk Category	II
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.879
Kh case 2	0.879
Type of roof	Monoslope

Topographic Factor (Kzt)

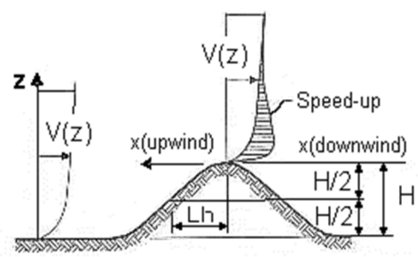
Topography	Flat
Hill Height (H)	0.0 ft
Half Hill Length (Lh)	0.0 ft
Actual H/Lh =	0.00
Use H/Lh =	0.00
Modified Lh =	0.0 ft
From top of crest: x =	0.0 ft
Bldg up/down wind?	downwind

H< 15ft;exp C
∴ Kzt=1.0



ESCARPMENT

H/Lh= 0.00 K_1 = 0.000
x/Lh = 0.00 K_2 = 0.000
z/Lh = 0.00 K_3 = 1.000
At Mean Roof Ht:
$K_{zt} = (1+K_1K_2K_3)^2$ = 1.00



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h = 17.7 ft
B = 45.7 ft
/z (0.6h) = 15.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
If building h/B>4 then may be flexible and should be investigated.
h/B = 0.39 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure default

Rigid Structure	
ē =	0.20
ℓ =	500 ft
z_{min} =	15 ft
c =	0.20
g_Q, g_v =	3.4
L_z =	427.1 ft
Q =	0.92
I_z =	0.23
G =	0.88 use G = 0.85

Flexible or Dynamically Sensitive Structure					
34 ıcy (η_1) =	0.0 Hz				
Damping ratio (β) =	0				
/b =	0.65				
/α =	0.15				
Vz =	81.9				
N_1 =	0.00				
R_n =	0.000				
R_h =	28.282	η =	0.000	h =	17.7 ft
R_B =	28.282	η =	0.000		
R_L =	28.282	η =	0.000		
g_R =	0.000				
R =	0.000				
Gf =	0.000				

Enclosure Classification

Test for Enclosed Building: Ao < 0.01Ag or 4 sf, whichever is smaller

Test for Open Building: All walls are at least 80% open.
Ao ≥ 0.8Ag

Test for Partially Enclosed Building: Predominately open on one side only

	Input			Test	
Ao	500.0	sf	Ao ≥ 1.1Aoi	NO	
Ag	600.0	sf	Ao > 4' or 0.01Ag	YES	
Aoi	1000.0	sf	Aoi / Agi ≤ 0.20	YES	Building is NOT
Agi	10000.0	sf			Partially Enclosed

Conditions to qualify as Partially Enclosed Building. Must satisfy all of the following:
Ao ≥ 1.1Aoi
Ao > smaller of 4' or 0.01 Ag
Aoi / Agi ≤ 0.20
Where:
Ao = the total area of openings in a wall that receives positive external pressure.
Ag = the gross area of that wall in which Ao is identified.
Aoi = the sum of the areas of openings in the building envelope (walls and roof) not including Ao.
Agi = the sum of the gross surface areas of the building envelope (walls and roof) not including Ag.

Test for Partially Open Building: A building that does not qualify as open, enclosed or partially enclosed.
(This type building will have same wind pressures as an enclosed building.

Reduction Factor for large volume partially enclosed buildings (Ri) :

If the partially enclosed building contains a single room that is unpartitioned , the internal pressure coefficient may be multiplied by the reduction factor Ri.

Total area of all wall & roof openings (Aog): 0 sf
Unpartitioned internal volume (Vi) : 0 cf
Ri = 1.00

Ground Elevation Factor (Ke)

Grd level above sea level = 0.0 ft
Constant = 0.00256 Adj Constant = 0.00256
Ke = 1.0000

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JOB TITLE	Walmart Renovation		
	Store 2403-278		
JOB NO.	WALGP0402	SHEET NO.	
CALCULATED BY		DATE	
CHECKED BY		DATE	

Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

Kz = Kh (case 1) =	0.88	Edge Strip (a) =	4.6 ft
Base pressure (qh) =	**18.0 psf**	End Zone (2a) =	9.1 ft
GCpi =	+/-0.18	Zone 2 length =	22.8 ft

Wind Pressure Coefficients

	CASE A			CASE B		
		θ = 1.2 deg				
Surface	GCpf	w/-GCpi	w/+GCpi	GCpf	w/-GCpi	w/+GCpi
1	0.40	0.58	0.22	-0.45	-0.27	-0.63
2	-0.69	-0.51	-0.87	-0.69	-0.51	-0.87
3	-0.37	-0.19	-0.55	-0.37	-0.19	-0.55
4	-0.29	-0.11	-0.47	-0.45	-0.27	-0.63
5				0.40	0.58	0.22
6				-0.29	-0.11	-0.47
1E	0.61	0.79	0.43	-0.48	-0.30	-0.66
2E	-1.07	-0.89	-1.25	-1.07	-0.89	-1.25
3E	-0.53	-0.35	-0.71	-0.53	-0.35	-0.71
4E	-0.43	-0.25	-0.61	-0.48	-0.30	-0.66
5E				0.61	0.79	0.43
6E				-0.43	-0.25	-0.61

Ultimate Wind Surface Pressures (psf)

Surface	Case A w/-GCpi	Case A w/+GCpi	Case B w/-GCpi	Case B w/+GCpi
1	10.4	4.0	-4.9	-11.3
2	-9.2	-15.7	-9.2	-15.7
3	-3.4	-9.9	-3.4	-9.9
4	-2.0	-8.5	-4.9	-11.3
5			10.4	4.0
6			-2.0	-8.5
1E	14.2	7.7	-5.4	-11.9
2E	-16.0	-22.5	-16.0	-22.5
3E	-6.3	-12.8	-6.3	-12.8
4E	-4.5	-11.0	-5.4	-11.9
5E			14.2	7.7
6E			-4.5	-11.0

Parapet

Windward parapet = 28.9 psf (GCpn = +1.5)
Leeward parapet = -19.3 psf (GCpn = -1.0)

Windward roof overhangs = 12.6 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

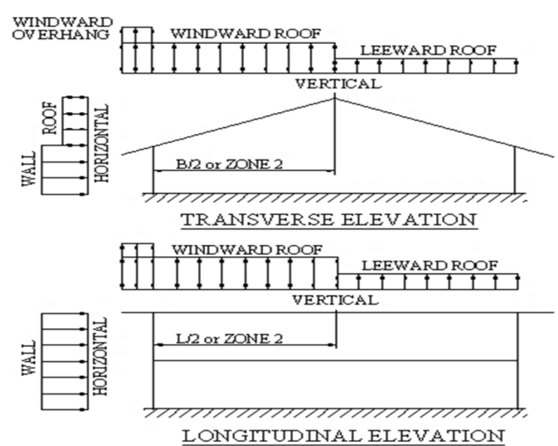
Interior Zone:	Wall	12.4 psf
	Roof	-5.8 psf **
End Zone:	Wall	18.7 psf
	Roof	-9.7 psf **

Longitudinal direction (parallel to L)

Interior Zone:	Wall	12.4 psf
End Zone:	Wall	18.7 psf

** NOTE: Total horiz force shall not be less than that determined by neglecting roof forces (except for MWFRS moment frames).

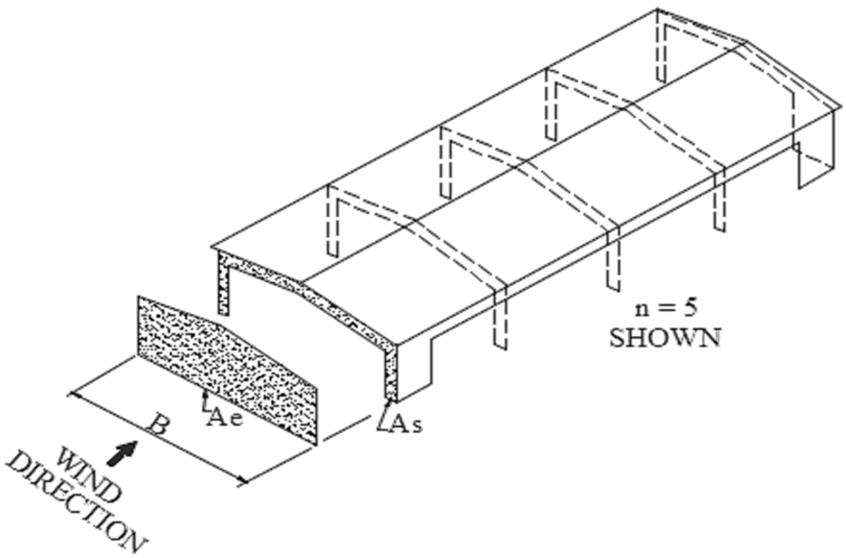
The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.



Wind Loads - h≤60' Longitudinal Direction MWFRS On Open or Partially Enclosed Buildings with Transverse Frames and Pitched Roofs

ASCE 7-16 procedure

Base pressure (qh) = **18.0 psf**
GCpi = +/-0.18 Enclosed bldg, procdure doesn't apply
Roof Angle (θ) = 1.2 deg



B=	45.7	ft
# of frames (n) =	5	
Solid are of end wall including fascia (As) =	1,500.0	sf
Roof ridge height =	18.1	ft
Roof eave height =	17.7	ft
Total end wall area if soild (Ae) =	817.9	sf

Longidinal Directional Force (F) = pAe
p= qh [(GCpf)windward -(GCpf)leeward] K_B K_S

Solidarity ratio (Φ) =	1.834	
n =	5	
KB =	0.8	
KS =	4.470	
Zones 5 & 6 area =	736.6	sf
5E & 6E area =	81.2	sf
(GCpf) windward - (GCpf) leeward] =	0.725	
p =	46.6	psf

Total force to be resisted by MWFRS (F) = **38.1 kips** applied at the centroid of the end wall area Ae

Note: The longidudinal force acts in combination with roof loads calculated elsewhere for an open or partially enclosed building.

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CALCULATED BY		DATE	
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Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 1) =	0.88	h =	17.7 ft	0.2h =	3.5 ft
Base pressure (qh) =	**18.0 psf**	0.6h =	10.6 ft		
Minimum parapet ht =	6.0 ft	GCpi =	+/-0.18		
Roof Angle (θ) =	1.2 deg	qi = qh =	18.0 psf		
Type of roof =	Monoslope				

Roof

	Surface Pressure (psf)								User input	
Area	10 sf	20 sf	50 sf	100 sf	200 sf	350 sf	500 sf	1000 sf	260 sf	300 sf
Negative Zone 1	-33.8	-31.6	-28.6	-26.4	-24.2	-22.4	-21.2	-21.2	-23.3	-22.9
Negative Zone 1'	-19.4	-19.4	-19.4	-19.4	-16.7	-16.0	-16.0	-16.0	-16.0	-16.0
Negative Zone 2	-44.6	-41.7	-38	-35.1	-32.2	-29.9	-28.4	-28.4	-31.1	-30.5
Negative Zone 3	-44.6	-41.7	-38	-35.1	-32.2	-29.9	-28.4	-28.4	-31.1	-30.5
Positive Zone 1 & 1'	16	16	16	16	16.0	16.0	16.0	16.0	16.0	16.0
Positive Zones 2 & 3	19.4	18.6	17.4	16.6	16.0	16.0	16.0	16.0	16.0	16.0
Overhang Zone 1&1'	-30.6	-30	-29.3	-28.8	-24.1	-20.4	-18.0	-18.0	-22.4	-21.4
Overhang Zone 2	-41.4	-37.6	-32.5	-28.7	-24.8	-21.8	-19.8	-19.8	-23.4	-22.6
Overhang Zone 3	-41.4	-37.6	-32.5	-28.7	-24.8	-21.8	-19.8	-19.8	-23.4	-22.6

Negative zone 3 = zone 2, since parapet >= 3ft.
Overhang pressures in the table above assume an internal pressure coefficient (Gcpi) of 0.0
Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 3.2 psf)

Parapet

qp = 19.3 psf

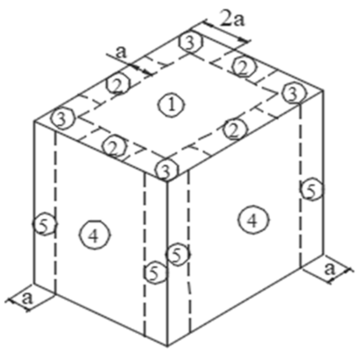
		Surface Pressure (psf)						User input
Solid Parapet Pressure		10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	40 sf
CASE A:	Zone 2 :	61.8	57.8	52.5	48.5	44.5	39.2	53.8
	Zone 3 :	61.8	57.8	52.5	48.5	44.5	39.2	53.8
CASE B :	Interior zone :	-36.5	-34.6	-32.2	-30.3	-28.5	-26.1	-32.8
	Corner zone :	-41.7	-38.9	-35.3	-32.5	-29.7	-26.1	-36.1

Walls

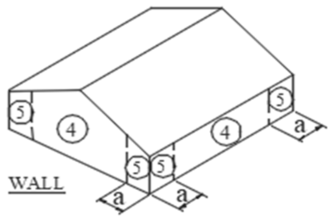
	GCp +/- GCpi				Surface Pressure at h				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	20 sf	200 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-19.4	-18.2	-17.3	-16.2	-20.2	-17.3
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-35.6	-20.2	-18.5	-16.2	-24.2	-18.5
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	19.4	16.6	16.0	16.0	18.6	16.0

Note: GCp reduced by 10% due to roof angle <= 10 deg.

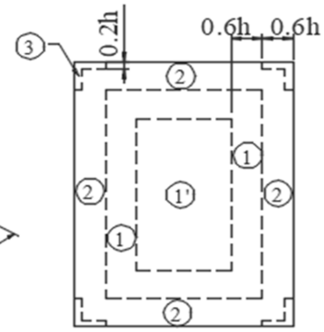
Location of C&C Wind Pressure Zones - ASCE 7-16



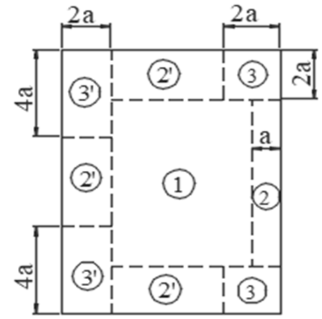
Roofs w/ θ ≤ 10°
and all walls
h > 60'



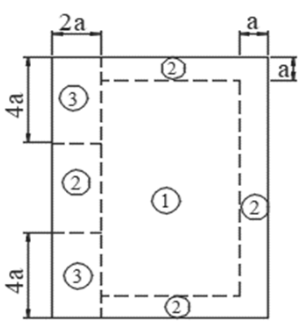
Walls h ≤ 60'
& alt design h<90'



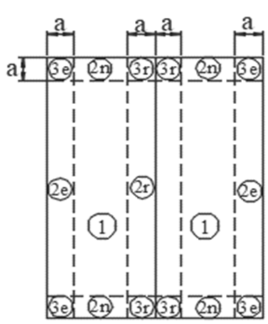
Gable, Sawtooth and
Multispan Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



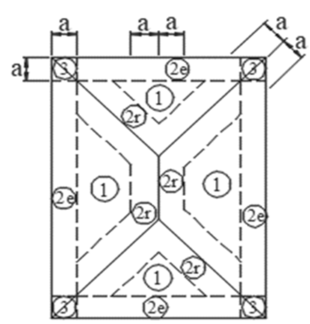
Monoslope roofs
3° < θ ≤ 10°
h ≤ 60' & alt design h<90'



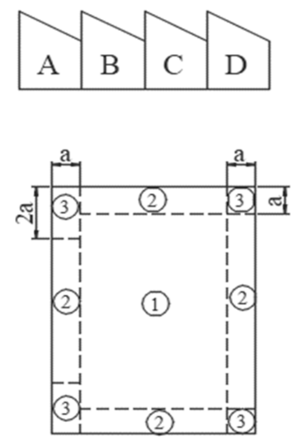
Monoslope roofs
10° < θ ≤ 30°
h ≤ 60' & alt design h<90'



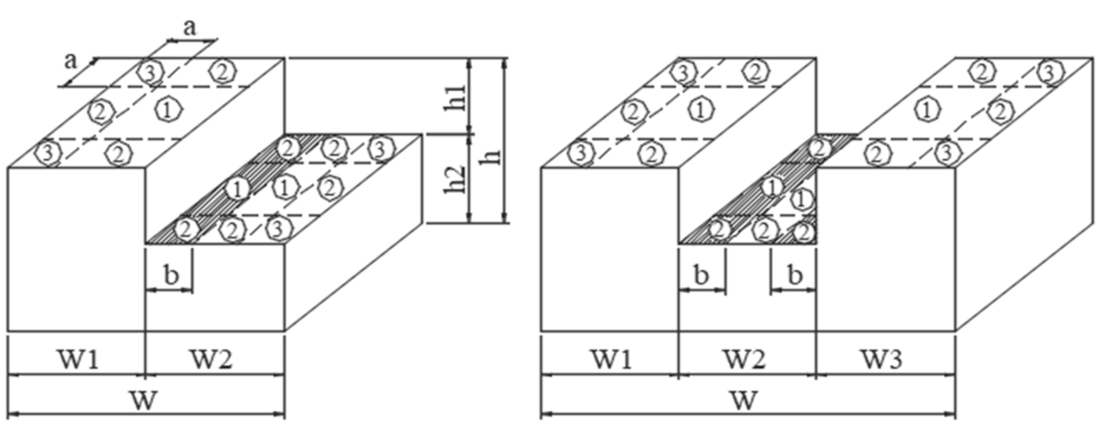
Multispan Gable &
Gable 7° < θ ≤ 45°



Hip 7° < θ ≤ 27°



Sawtooth 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'



Stepped roofs θ ≤ 3°
h ≤ 60' & alt design h<90'

Snow Loads : ASCE 7- 16

Nominal Snow Forces

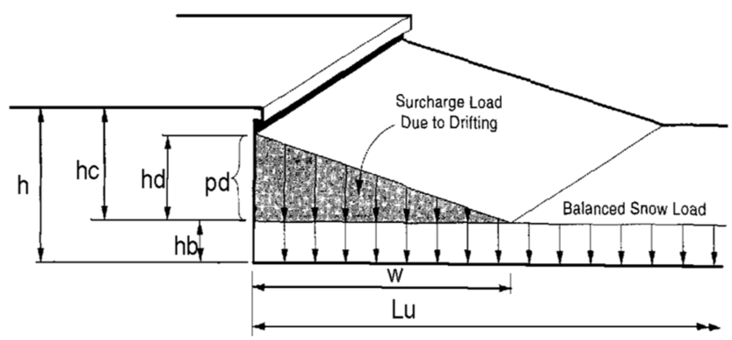
Roof slope		=	1.2 deg
Horiz. eave to ridge dist (W)		=	45.7 ft
Roof length parallel to ridge (L)		=	70.7 ft
Type of Roof			Monoslope
Ground Snow Load	Pg	=	20.0 psf
Risk Category		=	II
Importance Factor	I	=	1.0
Thermal Factor	Ct	=	1.00
Exposure Factor	Ce	=	1.0
Pf = 0.7*Ce*Ct*I*Pg		=	14.0 psf
Unobstructed Slippery Surface			yes
Sloped-roof Factor	Cs	=	1.00
Balanced Snow Load		**=**	**14.0 psf**
Rain on Snow Surcharge Angle			0.91 deg
Code Maximum Rain Surcharge			5.0 psf
Rain on Snow Surcharge		=	0.0 psf
Ps plus rain surcharge		=	14.0 psf
Minimum Snow Load	Pm	=	20.0 psf
Uniform Roof Design Snow Load		=	**20.0 psf**

Near ground level surface balanced snow load = **20.0 psf**

NOTE: Alternate spans of continuous beams shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code for loading diagrams and exceptions for gable roofs..

Windward Snow Drifts 1 - Against walls, parapets, etc

Upwind fetch	lu	=	603.3 ft
Projection height	h	=	6.0 ft
Snow density	g	=	16.6 pcf
Balanced snow height	hb	=	0.84 ft
	hd	=	5.25 ft
	hc	=	5.16 ft
hc/hb >0.2 = 6.1	**Therefore, design for drift**		
Drift height (hc)		=	5.16 ft
Drift width	w	=	21.40 ft
Surcharge load:	pd = γ*hd	=	**85.6 psf**
Balanced Snow load:		=	14.0 psf
			99.6 psf



Windward Snow Drifts 2 - Against walls, parapets, etc

Upwind fetch	lu	=	78.0 ft
Projection height	h	=	6.0 ft
Snow density	g	=	16.6 pcf
Balanced snow height	hb	=	0.84 ft
	hd	=	2.10 ft
	hc	=	5.16 ft
hc/hb >0.2 = 6.1	**Therefore, design for drift**		
Drift height (hd)		=	2.10 ft
Drift width	w	=	8.40 ft
Surcharge load:	pd = γ*hd	=	**34.9 psf**
Balanced Snow load:		=	14.0 psf
			48.9 psf

Note: If bottom of projection is at least 2 feet above hb then snow drift is not required.

WD Partners	JOB TITLE	Walmart Renovation		
7007 Dsicovery Blvd		Store 2403-278		
Dublin, OH	JOB NO.	WALGP0402	SHEET NO.	
wdpartners.com	CALCULATED BY		DATE	
	CHECKED BY		DATE	

Snow Loads - from adjacent building or roof:

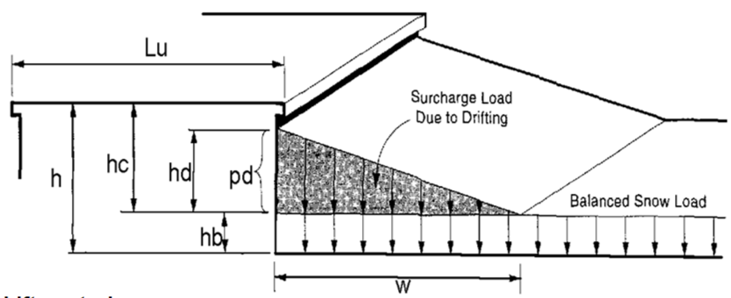
ASCE 7- 16 — Nominal Snow Forces

		Higher Roof	Lower Roof
Roof slope	=	1.2 deg	0.25 / 12 = 1.2 deg
Horiz. eave to ridge dist (W)	=	346.8 ft	45.7 ft
Roof length parallel to ridge (L)	=	603.3 ft	70.7 ft
Projection height (roof step) h	=		6.0 ft
Building separation s	=		0.0 ft
Type of Roof		Monoslope	Monoslope
Ground Snow Load Pg	=	20.0 psf	20.0 psf
Risk Category	=	II	II
Importance Factor I	=	1.0	1.0
Thermal Factor Ct	=	1.00	1.00
Exposure Factor Ce	=	1.0	1.0
Pf = 0.7*Ce*Ct*I*Pg	=	14.0 psf	14.0 psf
Unobstructed Slippery Surface		yes	yes
Sloped-roof Factor Cs	=	1.00	1.00
Balanced Snow Load Ps	**=**	**14.0 psf**	**14.0 psf**
Rain on Snow Surcharge Angle		6.94 deg	0.91 deg
Code Maximum Rain Surcharge		5.0 psf	5.0 psf
Rain on Snow Surcharge	=	5.0 psf	0.0 psf
Ps plus rain surcharge	=	19.0 psf	14.0 psf
Minimum Snow Load Pm	=	20.0 psf	20.0 psf
Uniform Roof Design Snow Load	=	**20.0 psf**	**20.0 psf**
Building Official Minimum	=		

NOTE: Alternate spans of continuous beams and other areas shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code.

Leeward Snow Drifts - from adjacent higher roof

Upper roof length	lu	=	603.3 ft
Snow density	γ	=	16.6 pcf
Balanced snow height	hb	=	0.84 ft
	hc	=	5.16 ft
hc/hb >0.2 = 6.1	**Therefore, design for drift**		
Adj structure factor		=	1.00
Drift height (hc)		=	5.16 ft
Drift width	w	=	27.93 ft
Surcharge load:	pd = γ*hd	=	**85.6 psf**
Balanced Snow load:		=	14.0 psf
			99.6 psf **Leeward drift controls**

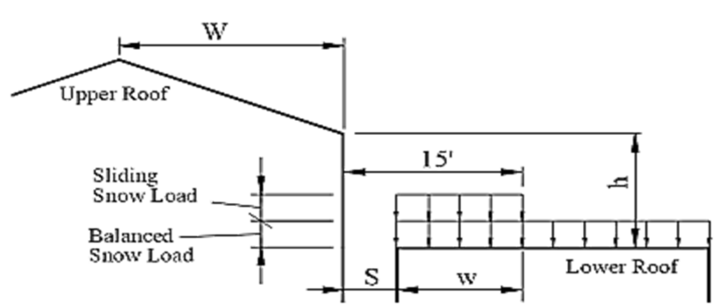


Windward Snow Drifts - from low roof against high roof

Lower roof length	lu	=	45.7 ft
Adj structure factor		=	1.00
Drift height	hd	=	1.57 ft
Drift width	w	=	6.29 ft
Surcharge load:	pd = γ*hd	=	**26.1 psf**
Balanced Snow load:		=	14.0 psf
			40.1 psf

Sliding Snow - onto lower roof

Sliding snow = 0.4 Pf W =	1942.2 plf
Distributed over 15 feet =	129.5 psf
hd + hb =	8.64 ft
hd + hb > h therefore sliding snow =	**85.6 psf**
Balanced snow load =	14.0 psf
Uniform snow load within 15' of higher roof =	99.6 psf
w =	15.00 ft



Seismic Loads: ASCE 7- 16 Strength Level Forces

Risk Category : II
Importance Factor (I) : 1.00

Site Class : D

Ss (0.2 sec) = 126.10 %g
S1 (1.0 sec) = 43.50 %g

Fa =	1.000	Sms =	1.261	S_{DS} =	0.841	Design Category =	D
Fv =	1.865	Sm1 =	0.811	S_{D1} =	0.541	Design Category =	D

Seismic Design Category = **D**

Redundancy Coefficient ρ = 1.30
Number of Stories: 1
Structure Type: All other building system:
Horizontal Struct Irregularities:No plan Irregularity
Vertical Structural Irregularities:No vertical Irregularity

Flexible Diaphragms: Yes
Building System: **Building Frame Systems**
Seismic resisting system: **Special reinforced masonry shear walls**
System Structural Height Limit: **160 ft**
Actual Structural Height (hn) = 18.1 ft
See ASCE7 Section 12.2.5 for exceptions and other system limitations

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 5.5
Over-Strength Factor (Ωo) = 2
Deflection Amplification Factor (Cd) = 4
S_{DS} = 0.841
S_{D1} = 0.541

Seismic Load Effect (E) = Eh +/-Ev = ρ Q_E +/- 0.2S_{DS} D = 1.3Qe +/- 0.168D Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = Ωo Q_E +/- 0.2S_{DS} D = 2Qe +/- 0.168D D = dead load

PERMITTED ANALYTICAL PROCEDURES

Simplified Analysis - Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted
Building period coef. (C_T) = 0.020 Cu = 1.40
Approx fundamental period (Ta) = $C_T h_n^x$ = 0.176 sec x= 0.75 Tmax = CuTa = 0.246
User calculated fundamental period (T) = sec Use T = 0.176
Long Period Transition Period (TL) = ASCE7 map = 6
Seismic response coef. (Cs) = S_{DS}I/R = 0.153
need not exceed Cs = Sd1 I /RT = 0.560
but not less than Cs = 0.044SdsI = 0.037
USE Cs = 0.153
Design Base Shear V = 0.153W

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures
Allowable story drift Δa = 0.020hsx where hsx is the story height below level x

Seismic Loads - cont. :

Strength Level Forces

Seismic Design Category (SDC)= D
Ie = 1.00
Sds = 1.009

CONNECTIONS

Force to connect smaller portions of structure to remainder of structure

$F_p = 0.133 S_{ds} w_p$ = 0.134 w_p
or $F_p = 0.05 w_p$ = 0.05 w_p Use Fp = 0.13 w_p w_p = weight of smaller portion

Beam, girder or truss connection for resisting horizontal force parallel to member

F_P = no less than 0.05 times dead plus live load vertical reaction

Anchorage of Structural Walls to elements providing lateral support

Fp = not less than 0.2KaIeWp

Flexible diaphragm span Lf =

Enter Lf to calculate Fp for flexible diaphragm
Fp =0.4SdskaIeWp = 0.404 Wp, but not less than 0.2Wp (rigid diaphragm)

ka= 1 Fp = 0.404 Wp
but Fp shall not be less than 5 psf

MEMBER DESIGN

Bearing Walls and Shear Walls (out of plane force)

Fp = 0.4SdsIeWw = 0.404 w_w
but not less than 0.10 w_w Use Fp = 0.40 w_w

Diaphragms

Fp = (Sum Fi / Sum Wi)Wpx + Vpx = (Sum Fi / Sum Wi)Wpx + Vpx
need not exceed 0.4 SdsIeWpx + Vpx = 0.404 Wpx + Vpx
but not less than 0.2 SdsIeWpx + Vpx = 0.202 Wpx + Vpx

ARCHITECTURAL COMPONENTS SEISMIC COEFFICIENTS

Architectural Component : Cantilever Elements (Unbraced or Braced to Structural Frame Below Its Center of Mass): Parapets and cantilever interior nonstructural walls

Importance Factor (Ip) : 1.0
Component Amplification Factor (a_p) = 2.5 h= 18.1 feet
Comp Response Modification Factor (R_p) = 2.5 z= 22.1 feet z/h = 1.00
Over-Strength Factor (Ωo) = 2

$F_p = 0.4 a_p S_{ds} I_p W_p (1+2z/h)/R_p$ = 1.211 Wp
not greater than Fp = 1.6SdsIpWp = 1.614 Wp
but not less than Fp = 0.3SdsIpWp = 0.303 Wp use Fp = 1.211 Wp

MECH AND ELEC COMPONENTS SEISMIC COEFFICIENTS

Seismic Design Category D & Ip=1.0, therefore
see ASCE7 Section 13.1.4 for exceptions

Mech or Electrical Component : Air-side HVAC, fans, air handlers, ac units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing.

Importance Factor (Ip) : 1.0
Component Amplification Factor (a_p) = 2.5 h= 18.1 feet
Comp Response Modification Factor (R_p) = 6 z= 22.1 feet z/h = 1.00
Over-Strength Factor (Ωo) = 2

$F_p = 0.4 a_p S_{ds} I_p W_p (1+2z/h)/R_p$ = 0.504 Wp
not greater than Fp = 1.6SdsIpWp = 1.614 Wp
but not less than Fp = 0.3SdsIpWp = 0.303 Wp use Fp = 0.504 Wp

WD Partners
7007 Discovery Blvd
Dublin, OH 43017
wdpartners.com

Project Title:
Engineer:
Project ID:
Project Descr:

ASCE Seismic Demands on Nonstructural Components

Seismic Demands on Nonstructural Components per ASCE 7-16 Section 13.3.1 and 13.4.1

h = 17.50 ft S_{DS} = 0.8410

Description : **RTU**

a_P = 2.50	W_P = 1,000.0	z = 17.50 ft	R_P = 6.00	I_P = 1.00	z/h : Actual = 1.00	z/h Design = 1.00	
Fp Upper Limit = 1,345.60	Fp Calc'd : Components =	420.50	**Fp DESIGN** : Components =	**420.50**			
Fp Lower Limit = 252.30	Connections =	420.50	Connections =	**420.50**			

Description : **Air-Curtain**

a_P = 2.50	W_P = 169.0(	z = 9.00 ft	R_P = 6.00	I_P = 1.00	z/h : Actual = 0.51	z/h Design = 0.51	
Fp Upper Limit = 227.41	Fp Calc'd : Components =	48.05	**Fp DESIGN** : Components =	**48.05**			
Fp Lower Limit = 42.64	Connections =	48.05	Connections =	**48.05**			

Description : **RCU1**

a_P = 2.50	W_P = 230.0(	z = 17.50 ft	R_P = 3.00	I_P = 1.00	z/h : Actual = 1.00	z/h Design = 1.00	
Fp Upper Limit = 309.49	Fp Calc'd : Components =	193.43	**Fp DESIGN** : Components =	**193.43**			
Fp Lower Limit = 58.03	Connections =	193.43	Connections =	**193.43**			

Description : **RCU2**

a_P = 2.50	W_P = 1,250.0	z = 17.50 ft	R_P = 3.00	I_P = 1.00	z/h : Actual = 1.00	z/h Design = 1.00	
Fp Upper Limit = 1,682.00	Fp Calc'd : Components =	1,051.25	**Fp DESIGN** : Components =	**1,051.25**			
Fp Lower Limit = 315.38	Connections =	1,051.25	Connections =	**1,051.25**			

a_P : Component amplification factor, Table 13.5-1 or Table 13.6-1
W_P : Component operating weight
Z : Height of point of attachment
R_P : Component response modification factor (Table 13.5-1 or Table 13.6-1)
I_P : Component importance factor (Section 13.1.3)

Fp - Calc'd : Calculated Fp, Eq (13.3-1), Same units as Wp
Fp - Upper : Upper Limit on Fp, Eq (13.3-2) ,(Same units as Wp
Fp - Lower : Lower Limit on Fp, Eq (13.3-3), Same units as Wp
Fp - Design : Fp for design purposes, Same units as Wp

1 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

The following report presents the results of PSI's geotechnical investigation performed for the proposed Walmart Store #2403 expansion located at 310 31st Avenue Southeast in Puyallup, Washington. A Site Vicinity Map is presented in Figure 1. This investigation was performed for Galloway and Company, Inc, (Galloway) in general accordance with PSI proposal number 0704-353378, dated September 5, 2021. Project authorization was provided by Mr. Ryan James of Galloway in an email September 10, 2021.

1.2 PROJECT DESCRIPTION

Project information was provided by Mr. Ryan James, in an email dated September 3, 2021. The provided information included the following:

- A Utility Plan Titled "Walmart, Puyallup, WA", dated June 30, 2005 by Pacland

Based on the provided information, PSI understands the improvements at the existing Walmart store will include a 3,500 square foot addition on shallow foundations located at the southwest corner of the existing Walmart Superstore. Structural loads planned were not provided, however, based on similar projects, PSI estimates column and wall loads will be on the order of a maximum of 100 kips and 2 kips per linear foot, respectively. The ground floor will remain at grade and consist of a reinforced concrete slab with floor loads less than 150 psf.

Should any of the above information or design basis made by PSI be inconsistent with the planned construction, it is requested that you contact us immediately to allow us to make any necessary modifications to this report. PSI will not be held responsible for changes to the project if not provided the opportunity to review the information and provide modifications to our recommendations.

1.3 PURPOSE AND SCOPE OF SERVICES

Based on correspondence with Mr. Ryan James and PSI proposal number 0704-353378, the purpose of this exploration was to evaluate the subsurface at the site and to develop geotechnical foundation design criteria for support of the proposed addition.

The scope of the exploration included a reconnaissance of the project site and completion of two test borings using hollow stem auger drilling methods. The project analysis included laboratory testing of samples collected from the borings, an engineering analysis and evaluation of the subsurface materials encountered, and preparation of this report.

2 SITE AND SUBSURFACE CONDITIONS

2.1 SITE DESCRIPTION

The site is located at 310 31st Avenue Southeast in Puyallup, Washington. It consists of a single parcel that contains Walmart Store #2403 and its associated parking and drive lanes. The site is bound on all sides by commercial and residential properties. Highway 512 is located to the west and Bradley Lake is located to the east.

2.2 TOPOGRAPHY

Based on The National Map developed by the United States Geological Survey, the property for the existing Walmart is relatively flat, at an elevation of about 440 to 443 feet (NAVD88). In the location of the proposed addition, the elevation is approximately 443 feet.

2.3 GEOLOGY

Based on a review of soils on the United States National Geologic Map Database, the site is mapped as Pleistocene glacial recessional outwash consisting of silt, clay, sand, and gravel.

2.4 GROUNDWATER

Groundwater was observed during drilling processes in boring B1 and B2 at a depth of approximately 18 feet bgs. Based on a review of public well log information from the Washington Department of Natural Resources, groundwater was anticipated to be 20 to 25 feet below grade

2.4.1 LOCAL FAULTING AND SEISMIC DESIGN PARAMETERS

PSI has reviewed the USGS Quaternary Fault and Fold Database of the United States and the following have been mapped within about 15 miles of the project site.

Table 1 – Local Faulting

Fault	Distance (Miles)
Tacoma Fault Zone	6.3, North

Based on site explorations and geologic mapping, we recommend using Site Class D to evaluate the seismic design of the structure. Site coefficients and spectral acceleration parameters for structural design are provided in Table 1.

Table 2 - Seismic Design Parameters
(47.16012° N, 122.28943° W) – SITE CLASS D

ASCE 7-16 CODE BASED RESPONSE SPECTRUM MCER GROUND MOTION - 5% DAMPING 1% IN 50 YEARS PROBABILITY OF COLLAPSE	
S_S	1.126
S_1	0.435
MAPPED MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION PARAMETER (SITE CLASS D)	
F_A	1.2
F_V	Null – See Section 11.4.8
S_{MS}	1.513
S_{M1}	Null – See Section 11.4.8
DESIGN SPECTRAL RESPONSE ACCELERATION PARAMETER	
S_{DS}	1.009
S_{D1}	Null – See Section 11.4.8

Notes: S_S = Short period (0.2 second) Mapped Spectral Acceleration
S_1 = 1.0 second period Mapped Spectral Acceleration
S_{MS} = Spectral Response adjusted for site class effects for short period = $F_A \bullet S_S$
S_{M1} = Spectral Response adjusted for site class effects for 1-second period = $F_v \bullet S_1$
S_{DS} = Design Spectral Response Acceleration for short period = $2/3 \bullet S_{MS}$
S_{D1} = Design Spectral Response Acceleration for 1-second period =$2/3 \bullet S_{M1}$
F_A = Short Period Site Coefficients
F_V = Long Period Site Coefficients

2.4.2 GEOLOGIC HAZARDS

The following table presents a qualitative assessment of geologic hazards considering the site class, the subsurface soil properties, the groundwater elevation, and probabilistic ground motions:

Table 3 – Qualitative Seismic Site Assessments

Liquefaction	Low	Based on the subsurface conditions encountered in the soil borings drilled on site, the area has a low risk of liquefaction.
Earthquake Shaking	Strong	The area is mapped as being in a zone of Strong Earthquake Shaking, based on the Seismic Response of the Washington Geologic Information Portal.
Slope Stability	Low	The site and surrounding areas are relatively flat and thus are at low risk of landslide.
Surface Rupture	Low	No known active faults underlie the site, based on the USGS Quaternary Faults Map

2.5 SUBSURFACE CONDITIONS

A detailed description of the Field Exploration Program can be found in Appendix A. Laboratory test results are presented on the exploration logs and in detail in Appendix B.

In borings B1 and B2, three inches of asphalt was observed overlying three inches of aggregate base rock. Subsurface conditions at the site generally consist of poorly graded gravelly sand with trace silt. Based on SPT blow counts, the sand had a relative density of medium dense to vary dense and moisture percentages of 7 to 29%. The sand extended to the termination depth of 26½ feet in boring B1. Underlying the sand at a depth of 20 feet bgs in boring B2 is poorly grade sandy gravel with trace silt. The gravel had a relative density of very dense and extended to the termination depth of 26½ in boring B2.

3 GEOTECHNICAL RECOMMENDATIONS

In our opinion, the proposed Walmart addition can be supported on shallow foundations, provided the geotechnical engineering recommendations in this report are followed.

3.1 SITE PREPARATION

PSI recommends that organics, loose, and otherwise unsuitable soils at the project site be stripped and removed from the building areas. Buried piping, where encountered, must be completely removed and rerouted from below proposed building foundations. Concrete structures and remnants of previous structures encountered during site excavation and site construction operations should be completely removed beneath the planned foundations and replaced with an engineered fill.

After the surficial materials have been stripped and completely removed from proposed development areas, PSI should observe the subgrade to identify any loose or unsuitable areas. Where organic, loose, or otherwise unsuitable soils are identified, within structural areas of the project, these soils should be completely removed and replaced with structural fill.

3.2 WET WEATHER CONSTRUCTION

It has been PSI's experience that during warm, dry weather, the moisture content of the upper few feet of soil will decrease; however, below the upper few feet, the moisture content of the soil tends to remain relatively unchanged and often well above the optimum moisture content for compaction.

As a result, the subcontractor must use care to protect exposed subgrade from disturbance by construction traffic, particularly during wet weather. The Contractor must employ construction equipment and procedures that prevent disturbance and softening of the subgrade soils. The use of excavation equipment equipped with smooth-edged buckets for excavation with the concurrent placement of granular work pads tends to minimize the potential for subgrade disturbance.

3.3 EXCAVATION CONSIDERATIONS

Open excavations exceeding four feet are not anticipated; however, if they do occur, excavations should be performed in accordance with OSHA regulations as stated in 29 CFR Part 1926. The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor should evaluate the soil exposed in the excavations as part of the required safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified by local, state, and federal safety regulations. PSI is providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations. Temporary excavations above the water table can be sloped at 1H:1V.

3.4 STRUCTURAL FILL MATERIALS

3.4.1 GENERAL

Structural fill materials should be compacted to at least 95% of the maximum dry density, at a moisture content within about 3% of optimum, as determined by ASTM D1557. Coarse granular fill should be compacted until well keyed. No brush, roots, construction debris, or other deleterious material should be placed within the structural fills. The earthwork contractor's compactive effort should be evaluated based on field observations, and lift thicknesses should be adjusted accordingly to meet compaction requirements. Additional information regarding specific types of fill is provided below.

3.4.2 IMPORTED GRANULAR FILL

Imported granular fill materials should consist of sand, gravel, or fragmental rock with a maximum size on the order of 4 inches and with not more than 8% passing the No. 200 sieve (washed analysis). Material satisfying these requirements can usually be placed during periods of wet weather. The first lift of granular fill placed over a fine-grained subgrade should be about 18 inches thick and subsequent lifts about 12 inches thick when using medium to heavy weight vibratory rollers. Granular structural fill should be limited to a maximum size of about 1½ inches when compacted with hand-operated equipment. We also recommend that lift thicknesses be limited to less than 8 inches when using hand-operated vibratory plate compactors.

3.4.3 UTILITY TRENCH BACKFILL

Utility trench backfill should consist of granular fill limited to a maximum size of about 1½ inches. The granular trench backfill should be compacted to at least 95% of the maximum dry density as determined by ASTM D1557 in the upper 4 feet of the trench and to at least 90% of this density below this depth. Lift thicknesses should be evaluated based on field density tests; however, particular care should be taken when operating hoe-mounted compactors to prevent damage to the newly placed conduits. Flooding or jetting to compact the trench backfill should not be permitted.

3.5 FOUNDATION RECOMMENDATIONS

3.5.1 SPREAD FOOTINGS

Based on the loads discussed earlier in this report, the proposed structure can be supported on conventional spread footing foundations constructed in accordance with the following design criteria. Footings should be established at a minimum depth of 1½ feet below the lowest adjacent finished grade. In addition, isolated column and continuous footings should have a minimum width of at least 3 and 1½ feet, respectively.

We recommend the use of a smooth-edged excavator to make the footing excavations. A geotechnical engineer should observe the footing subgrade at the time of excavation and prior to placing the reinforcing steel and concrete. Footings established in accordance with these criteria can be designed on the basis of an allowable soil bearing pressure of 3,000 psf. This value applies to the total of dead load plus frequently and/or permanently applied live loads and can be increased by one third for the total of all loads; dead, live, and wind or seismic.

If fill and/or other unsuitable soils are encountered at footing depth, the unsuitable material should be over excavated to firm subgrade material and replaced with granular structural fill. The total width of the over excavation area beneath the design footing elevation should be increased one foot in plan area for every foot of depth of over excavation. The over excavated areas should be backfilled with clean crushed rock and compacted to at least 95% of the maximum dry density as determined by ASTM D1557.

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of spread footings and the underlying soil. The total shearing resistance between the foundation footprint and the soil can be computed as the normal force, i.e., the sum of all vertical forces (dead load plus real live load), times the coefficient of friction equal to 0.40 (ultimate value). If additional lateral resistance is required, passive earth resistance against embedded footings or walls can be computed using a pressure based on an equivalent fluid with a unit weight of 300 pcf. This design passive earth pressure assumes granular structural fill is used to backfill the footing excavation or the footings will be neat formed in situ.

3.5.2 SHALLOW FOUNDATION CONSTRUCTION CONSIDERATIONS

The foundation excavations should be observed by a representative of PSI prior to steel or concrete placement to assess that the foundation materials are consistent with the materials discussed in this report. Soft or loose soil zones encountered at the bottom of the footing excavations should be recompacted or removed and replaced with properly compacted structural fill as directed by the geotechnical engineer. Cavities formed as a result of excavation of soft or loose soil zones should be backfilled with dense graded compacted crushed stone.

After opening, footing excavations should be observed and concrete placed as quickly as possible to avoid exposure of the footing bottoms to wetting and drying. Surface runoff water should be drained away from the excavations and not be allowed to pond. If possible, the foundation concrete should be placed during the same day the excavation is made. If it is required that footing excavations be left open for more than one day, the soils in the excavation should be protected to reduce evaporation or entry of moisture.

3.5.3 FLOOR SLAB SUPPORT

PSI recommends the slab-on-grade be underlain by at least 8 inches of angular, free-draining rock with less than 5% fines. The drain rock should be compacted until it is well keyed. In addition, it may be appropriate to install a durable vapor-retarding membrane to limit the risk of damp floors in areas that will have moisture-sensitive materials placed directly on the floor. The vapor-retarding membrane should be installed in accordance with the manufacturer's recommendations.

In our opinion, a coefficient of subgrade reaction, k, of 100 pci can be used to characterize the support with a minimum thickness of 8 inches of "drain rock" (based on a 1x1-foot plate load). However, depending on how the slab load is applied, the value will have to be geometrically modified. The value should be adjusted for larger areas using the following expression for cohesionless soil:

Modulus of Subgrade Reaction, for $k_s = k\left(\frac{B+1}{2B}\right)^2$ for cohesionless soil,

where: ks = coefficient of vertical subgrade reaction for loaded area;

k = coefficient of vertical subgrade reaction for 1x1 square foot area; and,

B = width of area loaded, in feet.

3.6 PAVEMENT

Prior to pavement construction, the pavement subgrade should be prepared as indicated previously in this report. PSI has provided the following pavement subgrade parameters based on the California Bearing Ratio (CBR) associated with the soils found at the site:

- Native Gravelly Sand Subgrade California Bearing Ratio (CBR) – 10
- Native Gravelly Sand Subgrade Resilient Modulus (MR) – 9,388 psi

PSI has provided the following estimated pavement design parameters based on experience in the general area of the project site with similar subgrade soils. Table 3 below contains our pavement section recommendations.

- Design Life = 20 years
- Reliability = 95%
- Initial Serviceability Index = 4.2 for asphalt, 4.5 for concrete
- Terminal Serviceability Index = 2.5
- Estimated Traffic Volumes
  - o Light-Duty – 30,000 ESALs
  - o Heavy-Duty – 80,000 ESALs

Table 4 - Recommended Pavement Sections

	FLEXIBLE **Light-Duty**	**FLEXIBLE** **Heavy-Duty**	**RIGID**
Asphalt / Concrete Course	3 inches Asphalt	4 inches Asphalt	4 inches Concrete
Gravel Base Course	8 inches	8 inches	6 inches

The recommended pavement sections in Table 4 are based on the AASHTO design methods for flexible and rigid pavement design, and a design life of 20 years. In addition, the ranges also represent typical light-duty and heavy-duty type pavement sections for use in preliminary design.

Governing Code: 2018 Washington State Building Code (ASCE 7-16)

DL - 15 psf
LL - 20 psf
Snow:
SL - Based on P_g = 20 psf, $I_s = 1.0$
Wind:
WL - Wind based on V_{ult} = 97 mph (3-sec gusts)

V_{ASD} = 75.1 mph

Exposure C
Risk Cat II

Seismic:

- $S_S = 1.261$; $S_1 = 0.435$
  $S_{DS} = .841$; $S_{D1} = 0.541$

Site Class D (stiff soil) per soils report (PSI 10/20/2021)

- Seismic Design Category = D

ACTION ITEMS

project Walmart Renovation - Puyallup, WA
project no. WALGP0402 date 3/14/2022

wd thinkers that do™

Gravity

- Typical joist (uniform loading)

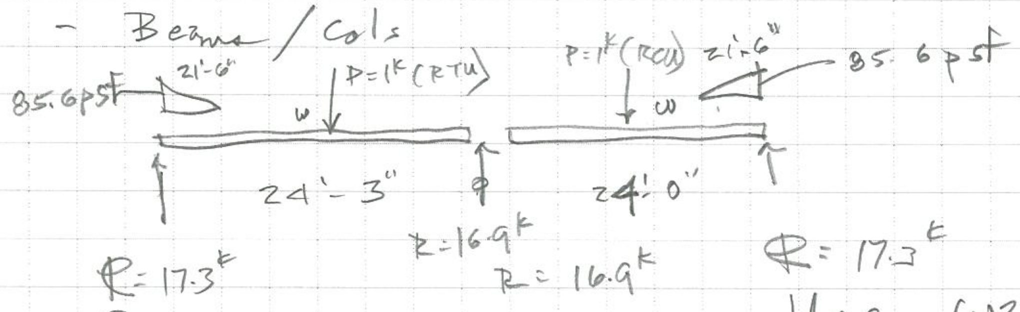
DL - 15 psf × 5' = 75 plf
SL/LL - 20 psf × 5' = 100 plf

175 plf

Use 28K (175/100) joists, l = 35'-6"

+ Snow Drift

- Beams/Cols



Use W24×55

See computer output

- Cols @ int

P = 16.9^k × 2
= 33.8^k

l = 17'-6"

Use HSS 6×6×1/4

See Computer Output

Ftg @ sides:

$\frac{17.3^k}{1.5^{ksf}} = 11.5^{\square'}$

or 2'-6" × 5'-0" Ftg
w/ (3)#5 LW T&B.
& (5)#5 SW T&B

Ftg: $\frac{40^k}{1.5^{ksf}} = 26.7^{\square'}$ or 5.18'

Use 6'-0" × 6'-0" × 1'-6"
w/ 6 #6 Each Way Bottom

WDPARTNERS.COM
+1 888 335 0014

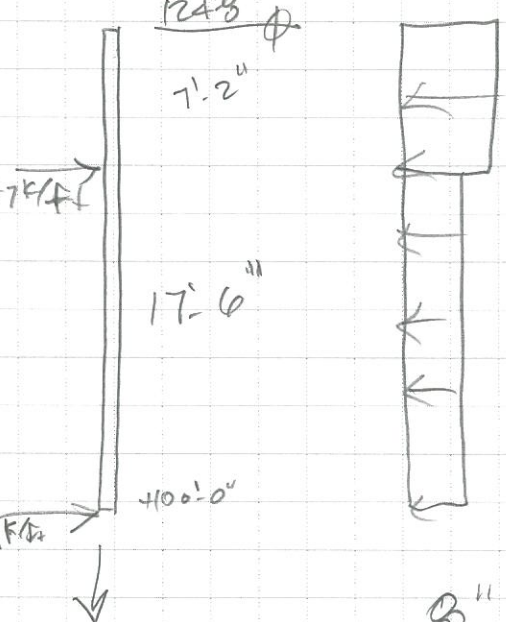
project Walmart Renovation - Puyallup, W
project no. WALGP0402 date 3/16/2022

Lateral

- Walls: w/ parapet

- Rear wall has no parapet (gutter)

- S_{DS} = 0.841
- Hor Bond Beams w/ (1)#5 @ 48" o.c.
- solid grouted



R=0.347 k/ft

R=0.141 k/ft

39.2 psf

16.2 psf

R_{max} = 2.852 k/ft

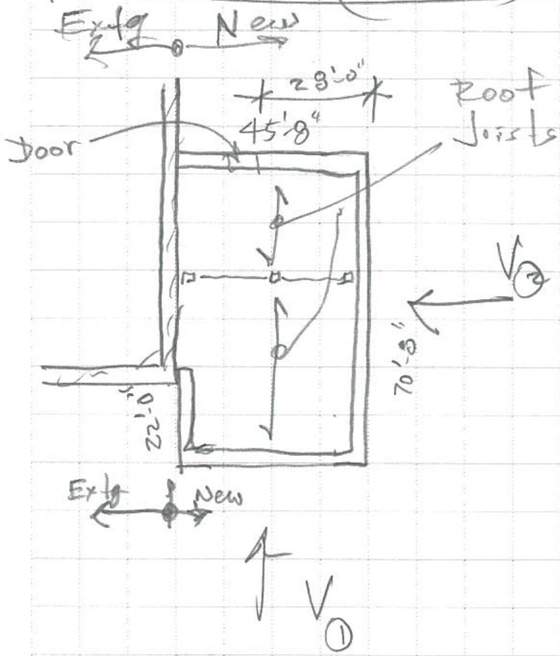
$\frac{2.852^k}{1.5 ksf}$ = 1.9 ft, 2'x1' cont ftg OK w/(2)#5 cont

8" Masonry w/ #6@32"oc

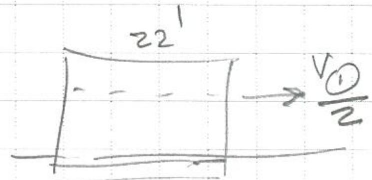
See Computer Output

ACTION ITEMS

Lateral (Cont)



- $S_{DS} = 0.841$
- $V = 0.153\,W$
- $f'_m = 2.0^{ksi}$
- Special Masonry Shear Walls.



① $W_{roof} = 15\,psf \times (45' \times 70') = 47.3^k$

Non participating walls:

$W_{wall} = 86\,psf \times 24'\text{-}8" \times 45'\text{-}8 \times 2 = 193.8^k$

$V = 0.153(47.3^k + 193.8^k) = 36.9^k$

$V_{ASD} = 36.9^k \times 0.7 = 25.8^k$

$\llcorner$ LC

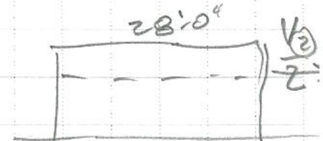
Shear Walls, 22^{ft} (min) long OK
See Computer output

② Non participating walls

$W_{wall} = 86\,psf \times 24'8" \times (70'\text{-}8 + 22'\text{-}0") = 196.6^k$

$V_{②} = 0.153(47.3^k + 196.6^k) = 37.32^k$.

$V_{② ASD} = 26.13^k$



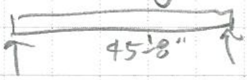
Shear Walls, 28'-0" (min) long OK
See computer Output.

project Walmart Renovation - Puyallup, WA
project no. WALGP0402 date 3/16/2022

Lateral (Cont)

Roof Diaphragm

(1)



ASD: $M = \frac{25.8^k \times 45'\text{-}8}{8} = 294.6^{k\text{-}ft}$

$R = \frac{25.8^k}{2} = 12.9^k$

$\text{Shear} = \frac{12.9^k}{69'\text{-}0"} = \underline{187\ plf}$

$\boxed{\text{Diaphragm cap} = \frac{1190\ plf}{\Omega = 2.15} = 553\ plf}$

OK

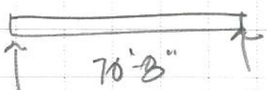
Chords:

$T/C = \frac{294.6^{k\text{-}ft}}{69'\text{-}0"} = 4.27^k$

$A_{req} = \frac{4.27^k}{24^{ksi}} = 0.1779\ \square"$, $L4 \times 4 \times 5/16$ OK $(A = 2.40\ \square")$
or $L6 \times 4 \times 5/16$ $(A = 3.03\ \square")$

---

(2)



$R = 26.13^k/2 = 13^k$

$M = 26.13^k \frac{(70'\text{-}8")}{4} = 461.7^{k\text{-}ft}$

$\text{shear} = \frac{13^k}{44'} = 295.5\ plf < 553\ plf$ OK

Chords:

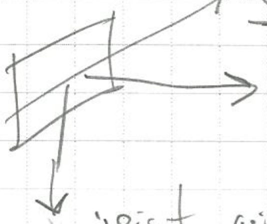
$T/C = \frac{461.7^{k\text{-}ft}}{44'} = 10.5^k$; $A_{req} = \frac{10.5^k}{24^{ksi}} = 0.4372\ \square"$ OK

Re-entrant corner

$F = 10.5^k \longrightarrow$ $\frac{10.5^k}{\underbrace{0.925(4)}_{1/4"\text{ welds}}} = 2.9"$ OK

ASCE 7-16
12.11.2.2.1

Embed plates (@ 32" oc)



$\frac{\left(\frac{25.8^{k}}{2}\right)}{70'} \times \frac{32''}{12} = 490^{lbs} \times 1.4 = 688^{lbs}$

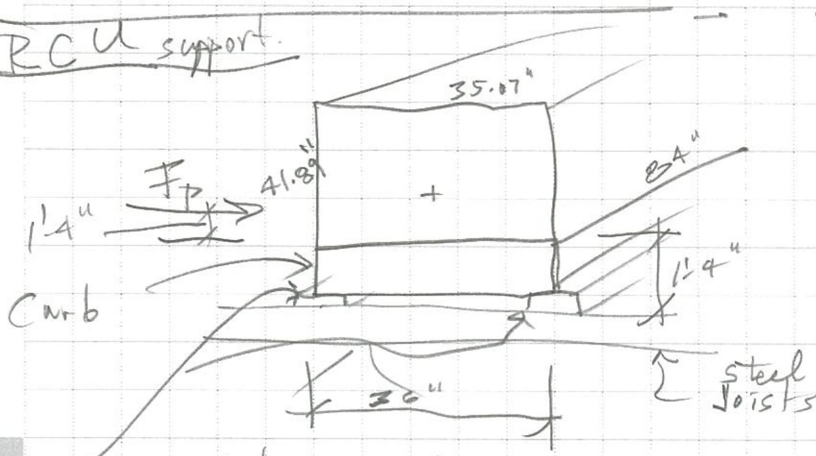
$347\ plf \times \frac{32}{12} = 925^{lbs} = T$

joist will support vertical
∴ $V = 0^{lbs}$

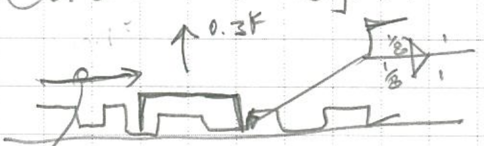
Plates ok @ 32" oc

See Computer Output.

RCU support



C6x8.2 between joist



$0.7\left(\frac{F_p}{4}\right) = \frac{1051^{lbs} \times 0.7}{4\ supports} = 0.184^{K}$

$W_p = 1250^{lbs}$; $I_p = 1.0$
$S_{DS} = 0.841$
$z/h = 1.0$

$F_p = 1051^{lbs}$

load combination

$M_o = 0.7(1051^{lbs})(1'4'' + 1'4'') = 1.96$

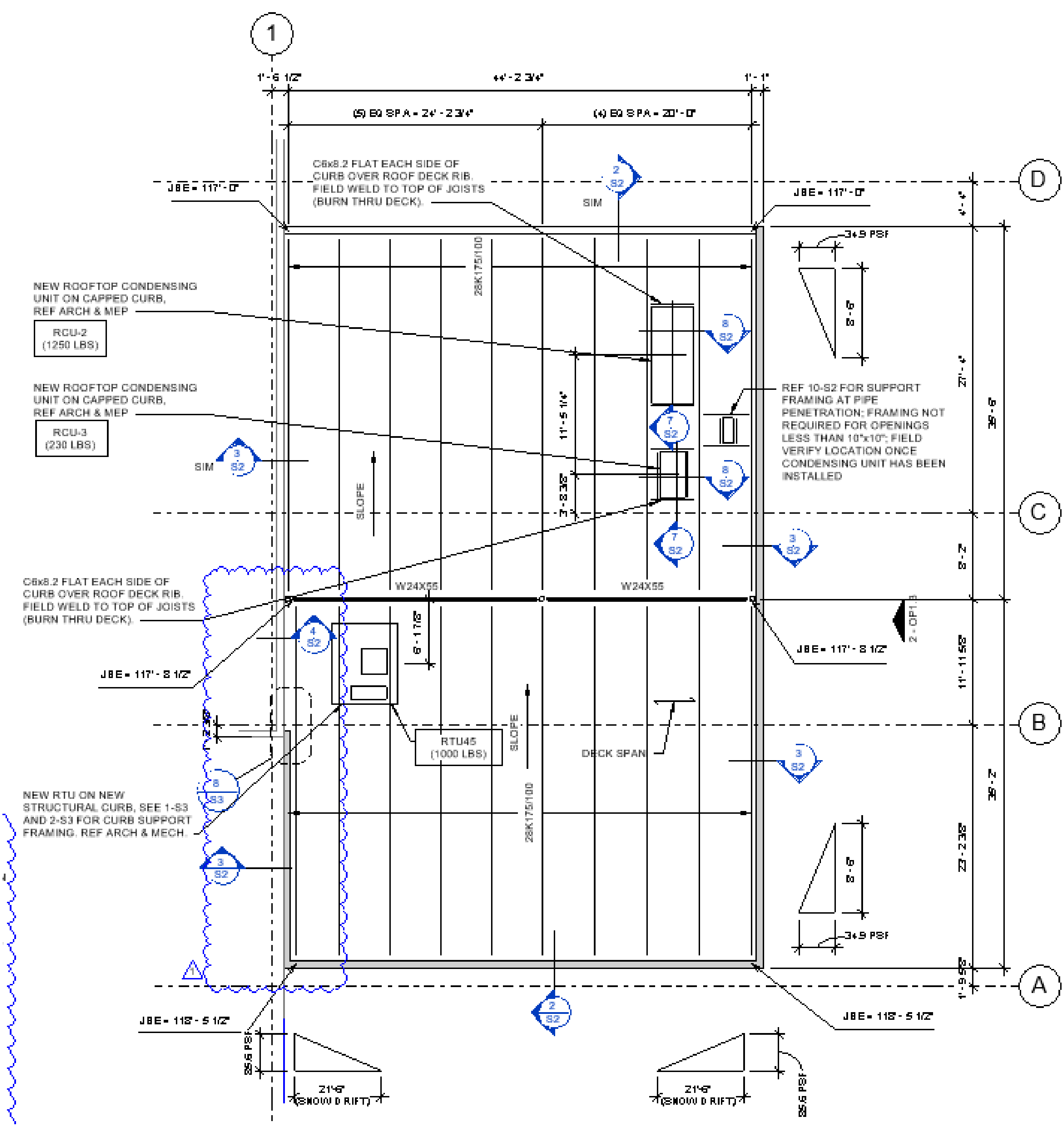
$M_r = 0.6\left(1250 \times \frac{3'}{2}\right) = 1.13^{k-ft}$

$T/C = \frac{1.96^{k-ft} - 1.13^{k-ft}}{3'-0''} = 0.3^{k}$

Welds: $0.925(2) = 2.77^{k/in}$ ok

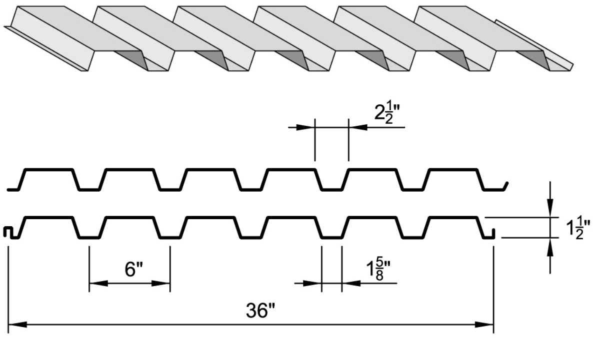
1/8"

ACTION ITEMS



1
1'-6 1/2"
44'-2 3/4"
1'-1"
(5) EQ SPA = 24'-2 3/4"
(4) EQ SPA = 20'-0"
C6x8.2 FLAT EACH SIDE OF CURB OVER ROOF DECK RIB. FIELD WELD TO TOP OF JOISTS (BURN THRU DECK).
JBE = 117'-0"
JBE = 117'-0"
SIM
D
C
B
A
4'-4"
34.9 PSF
8'-6"
28K175/100
NEW ROOFTOP CONDENSING UNIT ON CAPPED CURB, REF ARCH & MEP
RCU-2 (1250 LBS)
NEW ROOFTOP CONDENSING UNIT ON CAPPED CURB, REF ARCH & MEP
RCU-3 (230 LBS)
REF 10-S2 FOR SUPPORT FRAMING AT PIPE PENETRATION; FRAMING NOT REQUIRED FOR OPENINGS LESS THAN 10"x10"; FIELD VERIFY LOCATION ONCE CONDENSING UNIT HAS BEEN INSTALLED
11'-5 1/4"
3'-8 3/8"
27'-4"
36'-6"
SLOPE
8'-2"
W24X55
W24X55
C6x8.2 FLAT EACH SIDE OF CURB OVER ROOF DECK RIB. FIELD WELD TO TOP OF JOISTS (BURN THRU DECK).
6'-1 7/8"
JBE = 117'-8 1/2"
JBE = 117'-8 1/2"
2 - OP1.3
11'-11 5/8"
RTU45 (1000 LBS)
SLOPE
DECK SPAN
NEW RTU ON NEW STRUCTURAL CURB, SEE 1-S3 AND 2-S3 FOR CURB SUPPORT FRAMING. REF ARCH & MECH.
28K175/100
36'-2"
23'-2 3/8"
8'-6"
34.9 PSF
1'-9 5/8"
JBE = 118'-5 1/2"
JBE = 118'-5 1/2"
25.6 PSF
21'-6" (SNOW DRIFT)
21'-6" (SNOW DRIFT)
25.6 PSF

User Defined Criteria	
Deck Application:	Roof
Design Method:	ASD
Deck Type:	B, BI, BV, BIV
Gage:	Standard
Yield Stress (ksi):	80



Roof Decks - Types B, BI, BV, BIV

- Type B (Wide Rib) provides the best balance of strength and economy of all the 1 1/2” deep roof decks. Where rigid roofing insulation is used with B deck, a minimum 1” thickness is required.
- Available with nested side laps, types B and BV or with interlocking side laps, types BI and BIV.
- Available as a vented deck, types BV and BIV are manufactured with slot vents in the bottom flutes. The openings can be specified from 0.5% up to 1.5% of total surface. Types BV and BIV are to be specified when venting is required for cementitious insulation fill.
- Also available with rolled-in hanger tabs (non-vented types only).

See load tables on page 2

Roof Decks - Types B, BI, BV, BIV

Properties

Gage	Thickness (in)	Coverage (in)	Weight (psf)
22	0.0295	36	1.63
20	0.0358		1.98
18	0.0474		2.62
16	0.0598		3.30

ASD Section Properties / Design Strengths

Gage	F_y (ksi)	I_D 1 Span (in^4/ft)	I_D 2+ Span (in^4/ft)	I_p (in^4/ft)	I_n (in^4/ft)	S_p (in^3/ft)	S_n (in^3/ft)	Mn,p/Ω (in-lb/ft)	Mn,n/Ω (in-lb/ft)	Vn/Ω (lb/ft)	*Rbe/Ω (lb/ft)	*Rbi/Ω (lb/ft)
22	60	0.157	0.173	0.147	0.172	0.160	0.171	5756	6160	3013	981	1771
20	60	0.198	0.212	0.190	0.211	0.212	0.220	7623	7909	3642	1398	2544
18	60	0.274	0.281	0.271	0.281	0.298	0.310	10690	11127	4788	2336	4288
16	40	0.355	0.355	0.355	0.355	0.390	0.395	9332	9454	3996	2381	4396

Notes:

1. Section properties are calculated in accordance with the AISI S100-16.
2. Web crippling design strengths* are based on minimum bearing lengths of 1 1/2'' for end bearing and 3'' for interior bearing.

Allowable Uniform Loads and Maximum Construction Spans

Span Condition	Gage	Allowable Uniform Total Load (psf) / Load that Produces L/240 Deflection (psf)										Max. Constr. Span (ctr / ctr)
		Center to Center Span (ft - in)										
		5 - 0	5 - 6	6 - 0	6 - 6	7 - 0	8 - 0	9 - 0	10 - 0	11 - 0	12 - 0	
Single	22	153 / 82	127 / 62	107 / 48	91 / 37	78 / 30	60 / 20	47 / 14	38 / 10	- / -	- / -	9 - 7
	20	203 / 104	168 / 78	141 / 60	120 / 47	104 / 38	79 / 25	63 / 18	51 / 13	42 / 10	35 / 8	12 - 8
	18	285 / 144	236 / 108	198 / 83	169 / 66	145 / 52	111 / 35	88 / 25	71 / 18	59 / 14	49 / 10	17 - 9
	16	249 / 186	206 / 140	173 / 108	147 / 85	127 / 68	97 / 45	77 / 32	62 / 23	51 / 17	43 / 13	15 - 6
Double	22	162 / 219	134 / 164	113 / 127	96 / 100	83 / 80	64 / 53	50 / 38	41 / 27	34 / 21	28 / 16	11 - 9
	20	208 / 268	172 / 201	145 / 155	124 / 122	107 / 98	82 / 65	65 / 46	53 / 33	43 / 25	37 / 19	15 - 7
	18	291 / 356	242 / 267	203 / 206	174 / 162	150 / 130	115 / 87	91 / 61	74 / 44	61 / 33	51 / 26	21 - 11
	16	247 / 448	205 / 337	173 / 260	147 / 204	127 / 163	98 / 109	77 / 77	63 / 56	52 / 42	44 / 32	19 - 1
Triple	22	201 / 171	167 / 129	141 / 99	120 / 78	104 / 62	80 / 42	63 / 29	51 / 21	42 / 16	36 / 12	11 - 11
	20	258 / 210	214 / 158	180 / 121	154 / 95	133 / 76	102 / 51	81 / 36	66 / 26	54 / 20	46 / 15	15 - 10
	18	361 / 278	300 / 209	253 / 161	216 / 127	187 / 101	143 / 68	114 / 48	92 / 35	76 / 26	64 / 20	22 - 3
	16	307 / 351	255 / 264	215 / 203	183 / 160	159 / 128	122 / 86	96 / 60	78 / 44	65 / 33	54 / 25	19 - 5

Notes:

3. Allowable Uniform Loads and maximum construction spans shown are based on the following criteria:
   - ANSI/SDI RD-2017 Standard for Steel Roof Deck
   - Minimum bearing lengths of 1 1/2'' for end bearing and 3'' for interior bearing. Check web crippling if minimums are not met.
4. Maximum construction spans shown include a check for a nominal 200 lbs. concentrated load supported by a one foot section of deck per SDI criteria, which exceeds the IBC requirement of a 300 lbs. roof maintenance load distributed over an area of 2 1/2 feet by 2 1/2 feet per Section 1607.4 and Table 1607.1.
5. Values in RED are shown for use in determining deck capacity under deflection limits more stringent than Span/240. The total loads shown are not to be exceeded.
6. See website at www.newmill.com for Factory Mutual approved deck types and maximum FM construction spans.

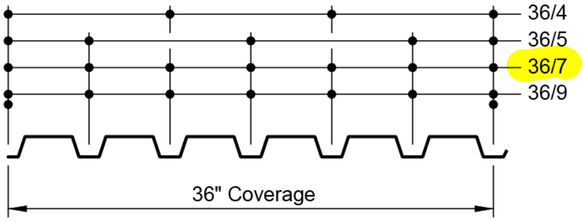
Maximum Cantilever Spans

Gage	F_y (ksi)	Back-Span Condition		
		Single	Double	Triple
22	60	1 - 8	1 - 7	1 - 7
20	60	1 - 11	1 - 10	1 - 10
18	60	2 - 5	2 - 3	2 - 3
16	40	3 - 1	2 - 7	2 - 7

Notes:

7. Maximum cantilever spans shown are based on the following criteria:
   - ANSI/SDI RD-2017 Standard for Steel Roof Deck
   - Adjacent span assumed to be at least 3 times longer than the cantilever and no greater than the maximum design or construction spans shown in table above
   - Bearing width at perimeter support assumed to be 3'' minimum
   - Design total uniform load of 45 psf in conjunction with a 100 lb. concentrated load.

User Defined Criteria	
Deck Application:	Roof
Deck Type:	B
Gage:	20
Yield Stress (ksi):	80
Net Uplift (psf):	25
Support Fasteners:	Arc Spot Welds Visible Diameter: 0.625
Side Lap Fasteners:	Screws #10



B, BV 20 GA. Diaphragm Design

The values shown in the tables are nominal strengths and are not to be used without applying the proper safety or resistance factor as shown above the top-right-hand corner of table. The factors are to be applied as follows:

- **LRFD** – The table values must be multiplied by the Φ resistance factor when comparing to forces calculated using Load and Resistance Factor Design.
- **ASD** – The table values must be divided by the Ω safety factor when comparing to forces calculated using Allowable Strength Design.

When diaphragm design includes net uplift only ASD Wind and Buckling safety factors are provided.

The shear strength values indicated are calculated based on the number of side lap connections being equally spaced per span. When maximum spacing of side lap connections is preferred in lieu of number equally spaced per span, due to the fasteners being placed at one-half space from each support with whole spaces between, divide span by required fastener spacing and round up to nearest integer. Use values in the table row corresponding to calculated number of fasteners per span.

Average spacing of support connections parallel to deck flutes is assumed equal to the side lap connection spacing. It may be possible to achieve greater diaphragm shear strengths by decreasing the spacing of support connections parallel to deck flutes. Please contact New Millennium for details.

The shear strength fields shown as blank (-) are conditions that do not meet minimum Steel Deck Institute side lap connection requirements. The values shown in RED indicate conditions with 0 side lap fasteners not in compliance with minimum Steel Deck Institute requirements. These values can be used to determine diaphragm shear strengths when properly spaced side lap connections are ignored for conservatism as part of the diaphragm shear design.

Designs for bare deck (no concrete fill) show nominal diaphragm shear values due to buckling in table below the shear strength table, for use in determining when conditions may be limited by panel buckling. An asterisk (*) is shown following the value in the shear strength table indicating conditions where panel buckling could potentially govern over connector strength. Nominal shear strength values flagged in this manner denote that a factored shear strength value exceeds the factored buckling shear strength value. The Designer should compare the factored values to determine which design requirement is governing.

See Diaphragm table on page 2

B, BV 20 GA. Diaphragm Design

Fy = 60 ksi, Fu = 62 ksi, Fxx = 60 ksi, Design Thickness = 0.0358 in.
Support Fasteners: 5/8'' Arc Spot Welds
Side Lap Fasteners: #10 Screws

Ω (Wind): 2.15
Ω (Buckling): 2.00

Support Fastener Pattern	Side Lap Conn./ Span	Bare Deck Nominal Shear Strength (plf) in Presence of 25 psf Net Uplift														K_1 (ft^{-1})
		Deck Span (ft - in)														
		3 - 0	3 - 6	4 - 0	4 - 6	5 - 0	5 - 6	6 - 0	6 - 6	7 - 0	7 - 6	8 - 0	8 - 6	9 - 0	9 - 6	
36/9 Dn = 68	0	2938	2585	2302	2056	1839	1660	1512	1386	1278	1185	1103	1031	967	911	0.357
	1	3120	2755	2459	2218	1992	1800	1640	-	-	-	-	-	-	-	0.299
	2	3294	2918	2612	2359	2146	1940	1768	1623	1498	1390	1296	1212	1138	-	0.258
	3	3459	3076	2760	2498	2278	2080	1896	1741	1608	1493	1392	1303	1223	1154	0.226
	4	3617	3227	2903	2633	2405	2211	2024	1859	1718	1595	1488	1393	1309	1235	0.202
	5	3766	3372	3042	2764	2529	2328	2153	1978	1828	1698	1584	1484	1394	1315	0.182
	6	3908	3511	3175	2891	2649	2442	2262	2096	1937	1800	1680	1574	1480	1396	0.166
	7	4042	3644	3305	3015	2767	2553	2368	2206	2047	1903	1776	1664	1565	1477	0.152
	8	4169	3771	3429	3135	2882	2663	2472	2305	2157	2005	1872	1755	1651	1558	0.141
	9	4289	3893	3549	3251	2994	2770	2574	2402	2251	2108	1968	1845	1736	1639	0.131
	10	4402	4009	3664	3364	3102	2874	2674	2498	2342	2203	2064	1936	1821	1720	0.122
36/7 Dn = 68	0	1880	1635	1431	1265	1132	1024	934	857	791	735	685	641	537	501	0.535
	1	2092	1825	1616	1436	1286	1164	1062	-	-	-	-	-	-	-	0.415
	2	2294	2009	1784	1602	1440	1304	1190	1094	1011	940	877	822	708	-	0.340
	3	2487	2187	1947	1751	1590	1443	1318	1212	1121	1042	973	912	793	744	0.287
	4	2670	2357	2104	1897	1725	1581	1446	1330	1231	1145	1069	1003	879	825	0.249
	5	2843	2520	2256	2039	1857	1704	1573	1448	1341	1247	1165	1093	964	906	0.219
	6	3007	2676	2403	2177	1986	1824	1686	1566	1450	1350	1262	1184	1050	987	0.196
	7	3161	2825	2545	2310	2112	1942	1797	1671	1560	1452	1358	1274	1135	1068	0.178
	8	3306	2967	2681	2439	2234	2058	1906	1774	1658	1555	1454	1365	1220	1148	0.162
	9	3442	3101	2811	2564	2352	2170	2012	1874	1753	1646	1550	1455	1306	1229	0.149
	10	3570	3229	2936	2684	2467	2280	2116	1973	1848	1736	1637	1546	1391	1310	0.138
36/5 Dn = 428	0	1664	1463	1303	1170	1047	946	766	691	626	570	520	476	437	402	0.642
	1	1844	1632	1459	1317	1199	1086	894	-	-	-	-	-	-	-	0.477
	2	2010	1789	1607	1455	1328	1220	1022	927	846	775	713	657	608	-	0.380
	3	2161	1936	1747	1588	1453	1337	1135	1044	956	877	809	748	693	644	0.315
	4	2299	2072	1878	1713	1572	1450	1241	1144	1058	980	905	838	779	725	0.270
	5	2424	2198	2002	1832	1686	1559	1342	1240	1150	1070	997	929	864	806	0.236
	6	2538	2314	2117	1945	1795	1664	1440	1333	1239	1154	1078	1010	947	887	0.209
	7	2641	2421	2225	2051	1898	1764	1533	1423	1325	1237	1157	1085	1019	959	0.188
	8	2734	2519	2325	2151	1997	1859	1622	1509	1407	1316	1233	1158	1090	1027	0.171
	9	2818	2610	2418	2245	2090	1951	1707	1591	1487	1393	1307	1229	1158	1093	0.156
	10	2894	2692	2505	2333	2178	2038	1788	1670	1563	1466	1378	1298	1224	1157	0.144
36/4 Dn = 608	0	1274	1122	999	782	682	599	530	471	419	374	334	298	266	237	0.802
	1	1449	1286	1152	936	836	739	658	-	-	-	-	-	-	-	0.561
	2	1605	1436	1295	1068	961	868	786	707	639	579	526	479	437	-	0.431
	3	1742	1572	1426	1191	1077	978	892	816	748	681	622	569	522	480	0.350
	4	1864	1694	1547	1304	1186	1082	990	909	837	772	713	660	608	560	0.294
	5	1970	1804	1657	1407	1286	1178	1083	998	922	853	790	733	681	633	0.254
	6	2063	1902	1757	1501	1378	1268	1169	1081	1001	929	864	804	748	697	0.224
	7	2144	1990	1848	1587	1463	1351	1250	1159	1077	1002	933	870	812	758	0.200
	8	2215	2068	1930	1664	1540	1427	1325	1232	1147	1070	999	933	872	816	0.180
	9	2278	2138	2005	1734	1611	1498	1394	1300	1213	1134	1060	992	929	870	0.164
	10	2332	2200	2072	1798	1675	1562	1459	1363	1275	1194	1118	1048	983	922	0.151
	I (in4/ft)	Nominal Shear Due to Panel Buckling, Sn (plf) / Deck Span (ft - in)														
		3 - 0	3 - 6	4 - 0	4 - 6	5 - 0	5 - 6	6 - 0	6 - 6	7 - 0	7 - 6	8 - 0	8 - 6	9 - 0	9 - 6	
	0.2127	20959	15399	11790	9315	7545	6236	5240	4465	3850	3353	2947	2611	2329	2090	

Notes:

1. Diaphragm shear and stiffness values are based on the Steel Deck Institute Diaphragm Design Manual, Fourth Edition (DDM04) and AISI S310-2016 (S310-16).
2. Diaphragm shear and stiffness values are based on minimum 3 span condition.
3. An asterisk (*) denotes span may be limited by shear buckling. See bottom table.
4. Shear strength values shown in RED do not comply with minimum SDI side lap connection requirements and shall not be used except with properly spaced side lap connections.

Diaphragm Stiffness, G' (k/in.)

K_2 = 1056 kip/in.
K_4 = 3.518
L_v = Span (ft)

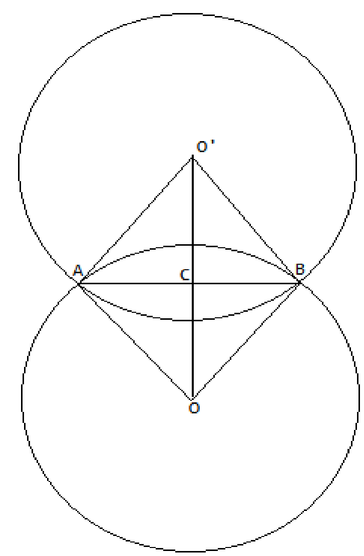
$$G' = \frac{K_2}{K_4 + \frac{0.30Dn}{L_v} + 3K_1L_v}$$

Project:	Walmart - Puyallup, WA	3/18/2022
Proj #	WALGP0402	

Anchor Rods Design in Solid-grouted Masonry (ACI 530-13)

(Section 8.1.3 - ASD Approach)
(Reference: NCMA TEK 12-3)

Actual Tensile load, lbs =	925.0 lbs	
Actual Shear load, lbs=	688.0 lbs	
Number of bolts (n) =	2	
Spacing (if more than one rod)	7.0 in	
Diameter of the bolt =	0.5 in	
Area of the bolt (A_b)=	0.196 in^2	
Embedment length (*lb*)=	5.0 in	
Compressive strength of masonry, f'm =	2000 psi	
Yield strength of anchor bolt, fy =	50000 psi	
Masonry wall width or spacing for in-plane shear =	7.625 in	
Anchor bolt edge distance, lbe =	40. in	3.813 in
	No shear reduction required	



Allowable load in Tenson:

Ap=	78.54 in^2
or	
Ap=	5026.548 in^2
Ap (overlapped areas)=	71.009 in^2
Ap (min of above) x n=	142.019 in^2
Ba (masonry breakout)=	7939.10 lbs
or	
Ba (steel yielding)=	5890.49 lbs
Allowable load in tension, Ba =	5890.49 lbs
ba/Ba=	.16

Angle between OA and OB
t = 1.590798
area of overlap = 15.061 in^2

Allowable load in Shear:
Bv (min of)=

(masonry breakout)	7939.10 lbs
(masonry crushing)	1558.06 lbs
(anchor pryout)	15878.20 lbs
(steel yielding)	3534.29 lbs

Allowable load in shear, Bv = 1558.06 lbs
bv/Bv= .44

Combined shear and tension (CSI): .60 ***<1 OK!***

WD Partners
7007 Discovery Blvd
Dublin, OH 43017
wdpartners.com

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

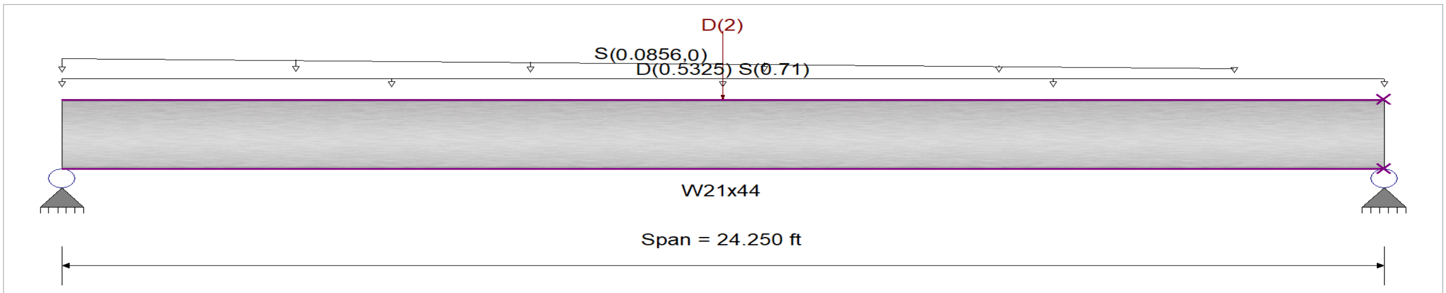
DESCRIPTION: Girders

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 calculated and added to loading

Uniform Load : D = 0.0150, S = 0.020 ksf, Tributary Width = 35.50 ft

Varying Uniform Load : S= 0.08560->0.0 k/ft, Extent = 0.0 -->> 21.50 ft, Trib Width = 1.0 ft, (Snow Drift)

Point Load : D = 2.0 k @ 12.125 ft, (RTU)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.460** : 1	Maximum Shear Stress Ratio =	**0.119** : 1
Section used for this span	**W21x44**	Section used for this span	**W21x44**
Ma : Applied	109.462 k-ft	Va : Applied	17.250 k
Mn / Omega : Allowable	238.024 k-ft	Vn/Omega : Allowable	144.90 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	12.125 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	0.239 in	Ratio =	1,217 >=240.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240.0
Max Downward Total Deflection	0.466 in	Ratio =	625 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

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 = 24.25 ft	1	0.229	0.055	54.52		54.52	397.50	238.02	1.00	1.00	7.99	217.35	144.90
+D+S													
Dsgn. L = 24.25 ft	1	0.460	0.119	109.46		109.46	397.50	238.02	1.00	1.00	17.25	217.35	144.90
+D+0.750S													
Dsgn. L = 24.25 ft	1	0.402	0.103	95.73		95.73	397.50	238.02	1.00	1.00	14.94	217.35	144.90
+0.60D													
Dsgn. L = 24.25 ft	1	0.137	0.033	32.71		32.71	397.50	238.02	1.00	1.00	4.80	217.35	144.90

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.4656	12.125		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	17.250	16.874
Overall MINimum	4.796	4.796
D Only	7.993	7.993

Steel Beam

DESCRIPTION: Girders

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+S	17.250	16.874
+D+0.750S	14.936	14.654
+0.60D	4.796	4.796
S Only	9.257	8.881

Steel Section Properties : W21x44

Depth	=	20.700	in	I xx	=	843.00	in^4	J	=	0.770	in^4
Web Thick	=	0.350	in	S xx		81.60	in^3	Cw	=	2,110.00	in^6
Flange Width	=	6.500	in	R xx	=	8.060	in				
Flange Thick	=	0.450	in	Zx	=	95.400	in^3				
Area	=	13.000	in^2	I yy	=	20.700	in^4				
Weight	=	44.252	plf	S yy	=	6.370	in^3	Wno	=	32.900	in^2
Kdesign	=	0.950	in	R yy	=	1.260	in	Sw	=	24.100	in^4
K1	=	0.813	in	Zy	=	10.200	in^3	Qf	=	14.000	in^3
rts	=	1.600	in					Qw	=	46.800	in^3
Ycg	=	10.350	in								

Steel Column

DESCRIPTION: Int column

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS6x6x1/4**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 46 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 17.5 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 17.5 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 17.5 ft, K = 1.0

Applied Loads

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

Column self weight included : 332.261 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 17.50 ft, Xecc = 3.0 in, D = 33.80 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.6967** : 1
Load Combination D Only
Location of max.above base 17.383 ft
At maximum location values are . . .
Pa : Axial 34.132 k
Pn / Omega : Allowable 83.962 k
Ma-x : Applied 0.0 k-ft
Mn-x / Omega : Allowable 25.709 k-ft
Ma-y : Applied -8.393 k-ft
Mn-y / Omega : Allowable 25.709 k-ft

PASS Maximum Shear Stress Ratio = **0.01183** : 1
Load Combination D Only
Location of max.above base 0.0 ft
At maximum location values are . . .
Va : Applied 0.4829 k
Vn / Omega : Allowable 40.826 k

Maximum Load Reactions . .
Top along X-X 0.4829 k
Bottom along X-X 0.4829 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 -0.3489 in at 10.218 ft above base
for load combination : D Only

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	0.697	PASS	17.38 ft	1.00	1.66	89.74	89.74	0.012	PASS	0.00 ft
+0.60D	0.418	PASS	17.38 ft	1.00	1.66	89.74	89.74	0.007	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx @ Top	My - End Moments @ Base	My @ Top
D Only	34.132	0.483	0.483						
+0.60D	20.479	0.290	0.290						

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx @ Top	My - End Moments @ Base	My @ Top
Axial @ Base	Maximum	34.132	0.483	0.483						
"	Minimum	20.479	0.290	0.290						
Reaction, X-X Axis Base	Maximum	34.132	0.483	0.483						
"	Minimum	20.479	0.290	0.290						
Reaction, Y-Y Axis Base	Maximum	34.132	0.483	0.483						
"	Minimum	34.132	0.483	0.483						
Reaction, X-X Axis Top	Maximum	34.132	0.483	0.483						

Steel Column

DESCRIPTION: Int column

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ 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
"	Minimum	20.479	0.290	0.290						
Reaction, Y-Y Axis Top	Maximum	20.479	0.290	0.290						
"	Minimum	34.132	0.483	0.483						
Moment, X-X Axis Base	Maximum	34.132		0.483						
"	Minimum	34.132		0.483						
Moment, Y-Y Axis Base	Maximum	34.132	0.483	0.483						
"	Minimum	34.132	0.483	0.483						
Moment, X-X Axis Top	Maximum	34.132	0.483	0.483						
"	Minimum	34.132	0.483	0.483						
Moment, Y-Y Axis Top	Maximum	34.132	0.483	0.483						
"	Minimum	34.132	0.483	0.483						

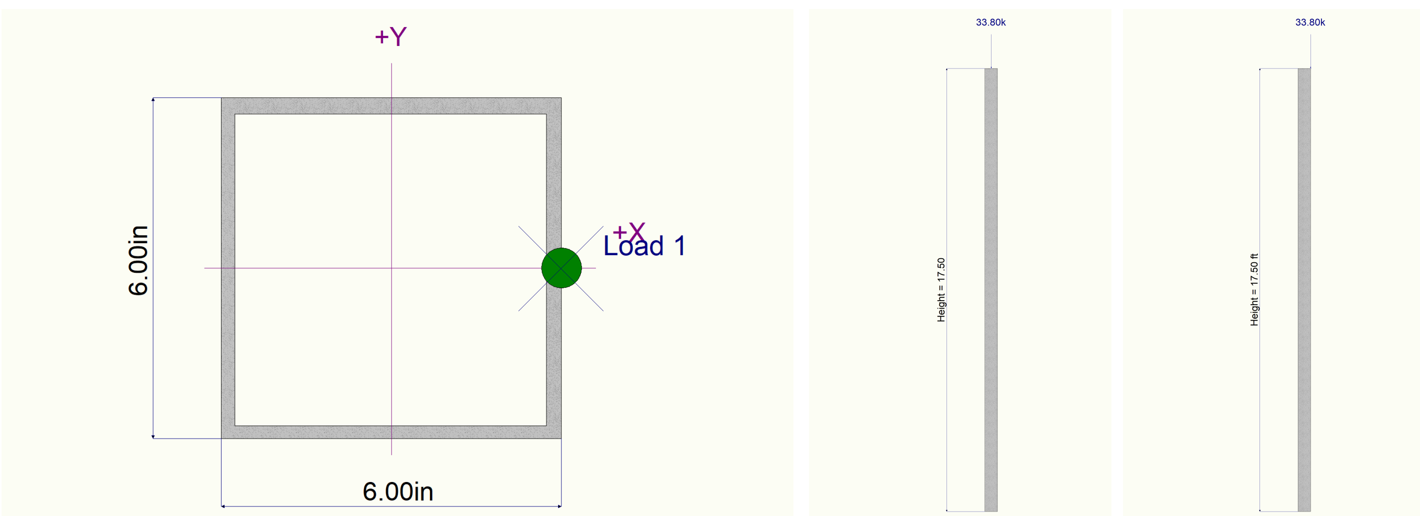
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	-0.3489	in	10.218	ft	0.000	in	0.000	ft
+0.60D	-0.2093	in	10.218	ft	0.000	in	0.000	ft

Steel Section Properties : HSS6x6x1/4

Depth	=	6.000	in	I xx	=	28.60	in^4	J	=	45.600	in^4
Design Thick	=	0.233	in	S xx	=	9.54	in^3				
Width	=	6.000	in	R xx	=	2.340	in				
Wall Thick	=	0.250	in	Zx	=	11.200	in^3				
Area	=	5.240	in^2	I yy	=	28.600	in^4	C	=	15.400	in^3
Weight	=	18.986	plf	S yy	=	9.540	in^3				
				R yy	=	2.340	in				
Ycg	=	0.000	in								

Sketches



Masonry Slender Wall

DESCRIPTION: Walls

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

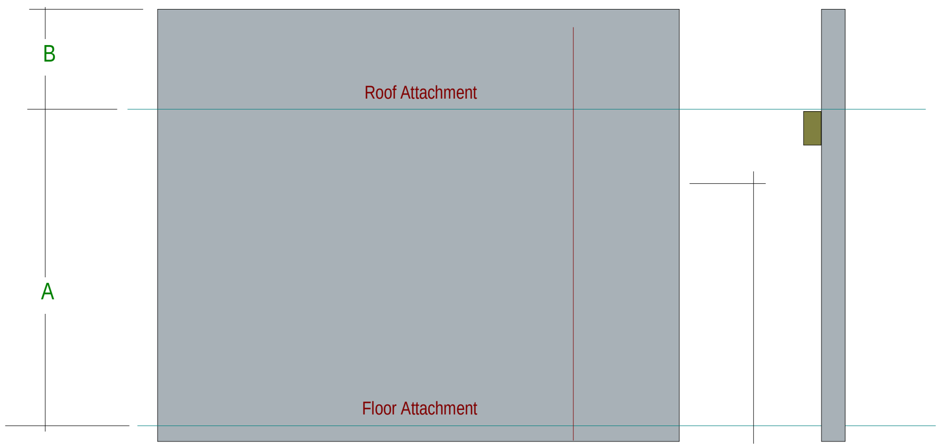
F'm	=	1.50 ksi	Nom. Wall Thickness	8 in	Temp Diff across thickness	=	deg F
Fy - Yield	=	60.0 ksi	Actual Thickness	7.625 in	Min Allow Out-of-plane Defl Ratio	=	0.0
Fr - Rupture	=	61.0 psi	Rebar "d" distance	3.8125 in			
Em = f'm *	=	900.0	Lower Level Rebar . . .		Minimum Vertical Steel %	=	0.0020
Max % of ρ bal.	=	0.008678	Bar Size #	6			
Grout Density	=	140 pcf	Bar Spacing	32 in			
Block Weight		Normal Weight					
Wall Weight	=	84.0 psf					

Wall is Solid Grouted

One-Story Wall Dimensions

A	Clear Height	=	17.5 ft
B	Parapet height	=	7.17 ft

Wall Support Condition Top & Bottom Pinned



Vertical Loads

Vertical Uniform Loads . . . *(Applied per foot of Strip Width)*			DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind	
Ledger Load	Eccentricity	3.81 in	.27			.36		k/ft
Concentric Load								k/ft

Vertical Concentrated Loads . . . *(Applied to full "Strip Width")*			DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind	
Beam Load #1	Eccentricity	3.81 in				.15		k
	Dist. from Base	17.5 ft						

Lateral Loads

Wind Loads :
Full area WIND load 16.2 psf

Seismic Loads :
Wall Weight Seismic Load Input Method : ASCE seismic factors entered

SDS Value per ASCE 12.11.1 $S_{DS} * I$ = .841

Fp = Wall Wt. * 0.3364 = 28.258 psf

(Applied to full "STRIP Width")

	D	Lr	L	E	W	Endpoints from Base top	bottom
Distributed Lateral Load					.023 k/ft	24.67	17.5 ft

Masonry Slender Wall

Lic. # : KW-06003498 **WDA&E**

DESCRIPTION: Walls

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +1.20D+0.50S-W	Maximum Bending Stress Ratio =	0.4389		
		Max Mu	1.168 k-ft	Phi * Mn	2.661 k-ft
PASS	Service Deflection Check E Only	Actual Defl. Ratio L/	1,059	Allowable Defl. Ratio	150.0
		Max. Deflection	0.1983 in		
PASS	Axial Load Check +1.20D+0.50S-W	Max Pu / Ag	14.103 psi	Max. Allow. Defl.	1.40 in
		Location	17.208 ft	0.2 * f'm	300.0 psi
PASS	Reinforcing Limit Check	Actual As/bd	0.003607	Max Allow As/bd	0.008678

Maximum Reactions . . .	for Load Combination....	
Top Horizontal	E Only	0.4914 k
Base Horizontal	E Only	0.2057 k
Vertical Reaction	+D+S	2.852 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	Mcr k-ft	Moment Values Mu k-ft	Phi	Phi Mn k-ft	As in^2	As Ratio	0.6 * rho bal
+1.40D at 16.92 to 17.50	1.290	27.360	0.59	0.12	0.90	2.66	0.165	0.0036	0.0085
+1.20D at 16.92 to 17.50	1.106	27.360	0.59	0.10	0.90	2.62	0.165	0.0036	0.0085
+1.20D+0.50S at 15.75 to 16.33	1.478	27.360	0.59	0.17	0.90	2.71	0.165	0.0036	0.0084
+1.20D+0.50W at 16.92 to 17.50	1.106	27.360	0.59	0.40	0.90	2.62	0.165	0.0036	0.0085
+1.20D-0.50W at 16.92 to 17.50	1.106	27.360	0.59	0.61	0.90	2.62	0.165	0.0036	0.0085
+1.20D+1.60S at 15.75 to 16.33	2.039	27.360	0.59	0.33	0.90	2.84	0.165	0.0036	0.0082
+1.20D+1.60S+0.50W at 7.58 to 8.17	2.862	27.360	0.59	0.25	0.90	3.03	0.165	0.0036	0.0079
+1.20D+1.60S-0.50W at 16.92 to 17.50	1.682	27.360	0.59	0.79	0.90	2.75	0.165	0.0036	0.0083
+1.20D+W at 16.92 to 17.50	1.106	27.360	0.59	0.90	0.90	2.62	0.165	0.0036	0.0085
+1.20D-W at 16.92 to 17.50	1.106	27.360	0.59	1.11	0.90	2.62	0.165	0.0036	0.0085
+1.20D+0.50S+W at 16.92 to 17.50	1.286	27.360	0.59	0.85	0.90	2.66	0.165	0.0036	0.0085
+1.20D+0.50S-W at 16.92 to 17.50	1.286	27.360	0.59	1.17	0.90	2.66	0.165	0.0036	0.0085
+0.90D+W at 16.92 to 17.50	0.829	27.360	0.59	0.93	0.90	2.55	0.165	0.0036	0.0086
+0.90D-W at 16.92 to 17.50	0.830	27.360	0.59	1.08	0.90	2.55	0.165	0.0036	0.0086
+1.368D+0.20S+E at 7.00 to 7.58	2.502	27.360	0.59	0.87	0.90	2.94	0.165	0.0036	0.0080
+1.368D+0.20S-E at 16.92 to 17.50	1.332	27.360	0.59	0.87	0.90	2.67	0.165	0.0036	0.0084
+0.7318D+E at 7.00 to 7.58	1.284	27.360	0.59	0.80	0.90	2.66	0.165	0.0036	0.0085
+0.7318D-E at 16.92 to 17.50	0.674	27.360	0.59	0.79	0.90	2.52	0.165	0.0036	0.0087

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 9.92 to 10.50	1.509	0.59	0.05	443.30	34.57	443.300	0.005	42,769.5
+D+S at 9.92 to 10.50	2.019	0.59	0.15	443.30	35.71	443.300	0.014	14,764.9
+D+0.750S at 9.92 to 10.50	1.892	0.59	0.12	443.30	35.43	443.300	0.012	17,664.4
+D+0.60W at 5.83 to 6.42	1.852	0.59	0.16	443.30	35.34	443.300	0.007	29,100.3
+D-0.60W at 13.42 to 14.00	1.215	0.59	0.29	443.30	33.91	443.300	0.010	21,776.2
+D+0.450W at 5.83 to 6.42	1.852	0.59	0.13	443.30	35.34	443.300	0.006	32,843.7
+D-0.450W at 12.83 to 13.42	1.264	0.59	0.20	443.30	34.02	443.300	0.008	27,053.9
+D+0.750S+0.450W at 7.58 to 8.17	2.088	0.59	0.17	443.30	35.86	443.300	0.013	16,742.8
+D+0.750S-0.450W at 12.25 to 12.83	1.696	0.59	0.25	443.30	34.99	443.300	0.014	14,870.6

Masonry Slender Wall

DESCRIPTION: Walls

+0.60D+0.60W at 5.25 to 5.83	1.141	0.59	0.15	443.30	33.74	443.300	0.006	37,027.0
+0.60D-0.60W at 13.42 to 14.00	0.729	0.59	0.26	443.30	32.79	443.300	0.008	26,616.4
+D+0.70E at 8.17 to 8.75	1.656	0.59	0.56	443.30	34.90	443.300	0.047	4,457.2
+D-0.70E at 7.58 to 8.17	1.705	0.59	0.49	443.30	35.01	443.300	0.038	5,539.6

Masonry Slender Wall

DESCRIPTION: Walls

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+0.750S+0.5250E at 8.17 to 8.75	2.039	0.59	0.49	443.30	35.75	443.300	0.043	4,846.0
+D+0.750S-0.5250E at 7.00 to 7.58	2.137	0.59	0.31	443.30	35.97	443.300	0.021	9,995.0
+0.60D+0.70E at 7.58 to 8.17	1.023	0.59	0.55	443.30	33.47	443.300	0.045	4,663.2
+0.60D-0.70E at 7.58 to 8.17	1.023	0.59	0.50	443.30	33.47	443.300	0.040	5,308.5
S Only at 9.92 to 10.50	0.510	0.59	0.09	443.30	32.28	443.300	0.009	22,846.0
W Only at 14.58 to 15.17	0.000	0.59	0.54	443.30	31.08	443.300	0.025	8,291.6
-W at 14.58 to 15.17	0.000	0.59	0.54	443.30	31.08	443.300	0.025	8,291.6
E Only at 7.58 to 8.17	0.000	0.59	0.74	443.30	31.08	38.587	0.198	1,058.9
E Only * -1.0 at 7.58 to 8.17	0.000	0.59	0.74	443.30	31.08	38.587	0.198	1,058.9

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	2.342 k
+D+S	0.0 k	0.01 k	2.852 k
+D+0.750S	0.0 k	0.01 k	2.725 k
+D+0.60W	0.1 k	0.28 k	2.342 k
+D-0.60W	0.0 k	0.29 k	2.342 k
+D+0.450W	0.0 k	0.21 k	2.342 k
+D-0.450W	0.0 k	0.22 k	2.342 k
+D+0.750S+0.450W	0.0 k	0.20 k	2.725 k
+D+0.750S-0.450W	0.0 k	0.23 k	2.725 k
+0.60D+0.60W	0.1 k	0.29 k	1.405 k
+0.60D-0.60W	0.0 k	0.29 k	1.405 k
+D+0.70E	0.1 k	0.34 k	2.342 k
+D-0.70E	0.1 k	0.35 k	2.342 k
+D+0.750S+0.5250E	0.1 k	0.25 k	2.725 k
+D+0.750S-0.5250E	0.1 k	0.27 k	2.725 k
+0.60D+0.70E	0.1 k	0.34 k	1.405 k
+0.60D-0.70E	0.1 k	0.35 k	1.405 k
S Only	0.0 k	0.01 k	0.510 k
W Only	0.1 k	0.48 k	0.000 k
-W	0.1 k	0.48 k	0.000 k
E Only	0.2 k	0.49 k	0.000 k
E Only * -1.0	0.2 k	0.49 k	0.000 k

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Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Slender Wall

DESCRIPTION: Walls, no parapet

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

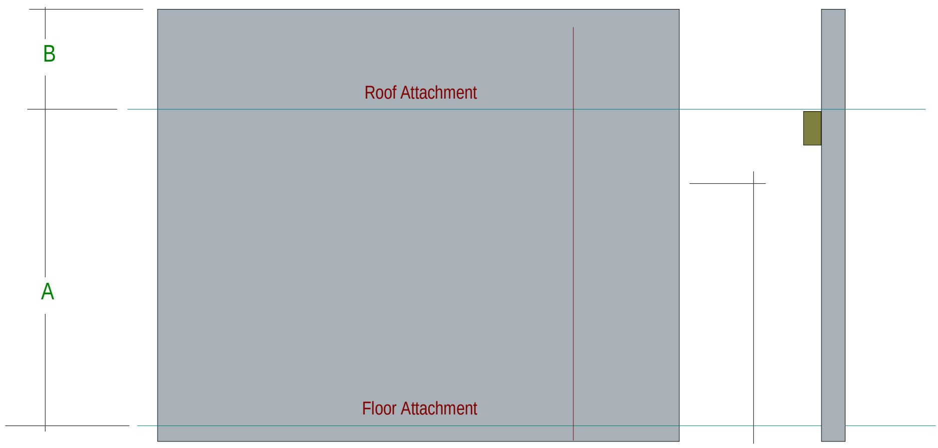
F'm	= 1.50 ksi	Nom. Wall Thickness	8 in	Temp Diff across thickness	=	deg F
Fy - Yield	= 60.0 ksi	Actual Thickness	7.625 in	Min Allow Out-of-plane Defl Ratio	=	0.0
Fr - Rupture	= 61.0 psi	Rebar "d" distance	3.8125 in			
Em = f'm *	= 900.0	Lower Level Rebar . . .		Minimum Vertical Steel %	=	0.0020
Max % of ρ bal.	= 0.008839	Bar Size #	6			
Grout Density	= 140 pcf	Bar Spacing	32 in			
Block Weight	Normal Weight					
Wall Weight	= 84.0 psf					

Wall is Solid Grouted

One-Story Wall Dimensions

A Clear Height = 17.50 ft
B Parapet height = 0 ft

Wall Support Condition Top & Bottom Pinned



Vertical Loads

<u>Vertical Uniform Loads . . .</u> *(Applied per foot of Strip Width)*

			DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind	
Ledger Load	Eccentricity	3.810 in	0.270			0.360		k/ft
Concentric Load								k/ft

<u>Vertical Concentrated Loads . . .</u> *(Applied to full "Strip Width")*

			DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind	
Beam Load #1	Eccentricity	3.810 in				0.150		k
	Dist. from Base	17.50 ft						

Lateral Loads

Wind Loads :
Full area WIND load 16.20 psf

Seismic Loads :
Wall Weight Seismic Load Input Method : ASCE seismic factors entered

SDS Value per ASCE 12.11.1 $S_{DS} * I$ = 0.8410

Fp = Wall Wt. * 0.3364 = 28.258 psf

(Applied to full "STRIP Width")

	D	Lr	L	E	W	Endpoints from Base top	bottom
Distributed Lateral Load					0.0230 k/ft	24.670	17.50 ft

Masonry Slender Wall

DESCRIPTION: Walls, no parapet

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +1.368D+0.20S+E	Maximum Bending Stress Ratio =	0.4663		
		Max Mu	1.262 k-ft	Phi * Mn	2.706 k-ft
PASS	Service Deflection Check E Only	Actual Defl. Ratio L/	283	Allowable Defl. Ratio	150.0
		Max. Deflection	0.7408 in		
PASS	Axial Load Check +1.368D+0.20S+E	Max Pu / Ag	16.196 psi	Max. Allow. Defl.	1.40 in
		Location	9.042 ft	0.2 * f'm	300.0 psi
PASS	Reinforcing Limit Check	Actual As/bd	0.003607	Max Allow As/bd	0.008839

Maximum Reactions . . .	*for Load Combination....*	
Top Horizontal	E Only	0.2473 k
Base Horizontal	E Only	0.2473 k
Vertical Reaction	+D+S	2.250 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)	Mcr (k-ft)	Mu (k-ft)	Phi	Phi Mn (k-ft)	As (in^2)	As Ratio	0.6 * rho bal
+1.40D at 16.92 to 17.50	0.447	27.360	0.59	0.12	0.90	2.46	0.165	0.0036	0.0088
+1.20D at 16.92 to 17.50	0.383	27.360	0.59	0.10	0.90	2.45	0.165	0.0036	0.0088
+1.20D+0.50S at 15.75 to 16.33	0.755	27.360	0.59	0.17	0.90	2.53	0.165	0.0036	0.0086
+1.20D+0.50W at 9.33 to 9.92	1.147	27.360	0.59	0.37	0.90	2.63	0.165	0.0036	0.0085
+1.20D-0.50W at 7.58 to 8.17	1.324	27.360	0.59	0.26	0.90	2.67	0.165	0.0036	0.0084
+1.20D+1.60S at 15.75 to 16.33	1.316	27.360	0.59	0.33	0.90	2.67	0.165	0.0036	0.0084
+1.20D+1.60S+0.50W at 11.08 to 11.67	1.787	27.360	0.59	0.52	0.90	2.78	0.165	0.0036	0.0083
+1.20D+1.60S-0.50W at 16.92 to 17.50	0.959	27.360	0.59	0.29	0.90	2.58	0.165	0.0036	0.0086
+1.20D+W at 8.75 to 9.33	1.206	27.360	0.59	0.69	0.90	2.64	0.165	0.0036	0.0085
+1.20D-W at 8.17 to 8.75	1.265	27.360	0.59	0.57	0.90	2.66	0.165	0.0036	0.0085
+1.20D+0.50S+W at 8.75 to 9.33	1.461	27.360	0.59	0.74	0.90	2.70	0.165	0.0036	0.0084
+1.20D+0.50S-W at 7.58 to 8.17	1.579	27.360	0.59	0.54	0.90	2.73	0.165	0.0036	0.0083
+0.90D+W at 8.75 to 9.33	0.905	27.360	0.59	0.67	0.90	2.57	0.165	0.0036	0.0086
+0.90D-W at 8.17 to 8.75	0.949	27.360	0.59	0.59	0.90	2.58	0.165	0.0036	0.0086
+1.368D+0.20S+E at 8.75 to 9.33	1.477	27.360	0.59	1.26	0.90	2.71	0.165	0.0036	0.0084
+1.368D+0.20S-E at 8.17 to 8.75	1.544	27.360	0.59	1.08	0.90	2.72	0.165	0.0036	0.0084
+0.7318D+E at 8.75 to 9.33	0.735	27.360	0.59	1.16	0.90	2.53	0.165	0.0036	0.0087
+0.7318D-E at 8.17 to 8.75	0.771	27.360	0.59	1.09	0.90	2.54	0.165	0.0036	0.0086

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu (k)	Mcr (k-ft)	Mactual (k-ft)	I gross (in^4)	I cracked (in^4)	I effective (in^4)	Deflection (in)	Defl. Ratio
D Only at 9.92 to 10.50	0.907	0.59	0.05	443.30	33.20	443.300	0.005	42,960.4
+D+S at 9.92 to 10.50	1.417	0.59	0.15	443.30	34.37	443.300	0.014	14,831.1
+D+0.750S at 9.92 to 10.50	1.290	0.59	0.12	443.30	34.08	443.300	0.012	17,743.5
+D+0.60W at 8.75 to 9.33	1.005	0.59	0.42	443.30	33.43	443.300	0.039	5,347.7
+D-0.60W at 8.17 to 8.75	1.054	0.59	0.33	443.30	33.54	443.300	0.030	7,058.9
+D+0.450W at 8.75 to 9.33	1.005	0.59	0.33	443.30	33.43	443.300	0.031	6,850.4
+D-0.450W at 8.17 to 8.75	1.054	0.59	0.24	443.30	33.54	443.300	0.021	9,936.0
+D+0.750S+0.450W at 8.75 to 9.33	1.387	0.59	0.39	443.30	34.30	443.300	0.038	5,589.4
+D+0.750S-0.450W at 7.58 to 8.17	1.486	0.59	0.18	443.30	34.52	443.300	0.015	14,392.3

Masonry Slender Wall

DESCRIPTION: Walls, no parapet

+0.60D+0.60W at 8.75 to 9.33	0.603	0.59	0.40	443.30	32.50	443.300	0.037	5,640.3
+0.60D-0.60W at 8.17 to 8.75	0.632	0.59	0.35	443.30	32.57	443.300	0.032	6,658.9
+D+0.70E at 8.75 to 9.33	1.005	0.59	0.83	443.30	33.43	38.127	0.336	625.8
+D-0.70E at 8.17 to 8.75	1.054	0.59	0.73	443.30	33.54	42.349	0.195	1,079.5

Masonry Slender Wall

DESCRIPTION: Walls, no parapet

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	I gross in^4	Stiffness I cracked in^4	I effective in^4	Deflections Deflection in	Defl. Ratio
+D+0.750S+0.5250E at 8.75 to 9.33	1.387	0.59	0.69	443.30	34.30	46.954	0.155	1,355.6
+D+0.750S-0.5250E at 8.17 to 8.75	1.436	0.59	0.47	443.30	34.41	443.300	0.041	5,080.1
+0.60D+0.70E at 8.75 to 9.33	0.603	0.59	0.80	443.30	32.50	37.963	0.298	704.7
+0.60D-0.70E at 8.17 to 8.75	0.632	0.59	0.74	443.30	32.57	40.465	0.215	975.4
S Only at 9.92 to 10.50	0.510	0.59	0.09	443.30	32.28	443.300	0.009	22,846.0
W Only at 8.75 to 9.33	0.000	0.59	0.62	443.30	31.08	63.890	0.073	2,870.1
-W at 8.75 to 9.33	0.000	0.59	0.62	443.30	31.08	63.890	0.073	2,870.1
E Only at 8.75 to 9.33	0.000	0.59	1.08	443.30	31.08	32.574	0.741	283.5
E Only * -1.0 at 8.75 to 9.33	0.000	0.59	1.08	443.30	31.08	32.574	0.741	283.5

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	1.740 k
+D+S	0.0 k	0.01 k	2.250 k
+D+0.750S	0.0 k	0.01 k	2.122 k
+D+0.60W	0.1 k	0.08 k	1.740 k
+D-0.60W	0.1 k	0.09 k	1.740 k
+D+0.450W	0.1 k	0.06 k	1.740 k
+D-0.450W	0.1 k	0.07 k	1.740 k
+D+0.750S+0.450W	0.1 k	0.05 k	2.123 k
+D+0.750S-0.450W	0.1 k	0.08 k	2.122 k
+0.60D+0.60W	0.1 k	0.08 k	1.044 k
+0.60D-0.60W	0.1 k	0.09 k	1.044 k
+D+0.70E	0.2 k	0.17 k	1.740 k
+D-0.70E	0.2 k	0.18 k	1.740 k
+D+0.750S+0.5250E	0.1 k	0.12 k	2.123 k
+D+0.750S-0.5250E	0.1 k	0.14 k	2.123 k
+0.60D+0.70E	0.2 k	0.17 k	1.044 k
+0.60D-0.70E	0.2 k	0.18 k	1.044 k
S Only	0.0 k	0.01 k	0.510 k
W Only	0.1 k	0.14 k	0.000 k
-W	0.1 k	0.14 k	0.000 k
E Only	0.2 k	0.25 k	0.000 k
E Only * -1.0	0.2 k	0.25 k	0.000 k

Masonry Shear Wall

DESCRIPTION: V1

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	2.0 ksi	Block Class	
Total Wall Height	24.670 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	22.0 ft	Fy - HJR	70.0 ksi	Min. Bending As %	0.00180
R: Resp. Mod Factor	5.0	Em	3,120.0 ksi		
Ie: Seismic Import. Factor	1.0	Phi - Shear	0.80	Phi : Axial & Flexure	0.90

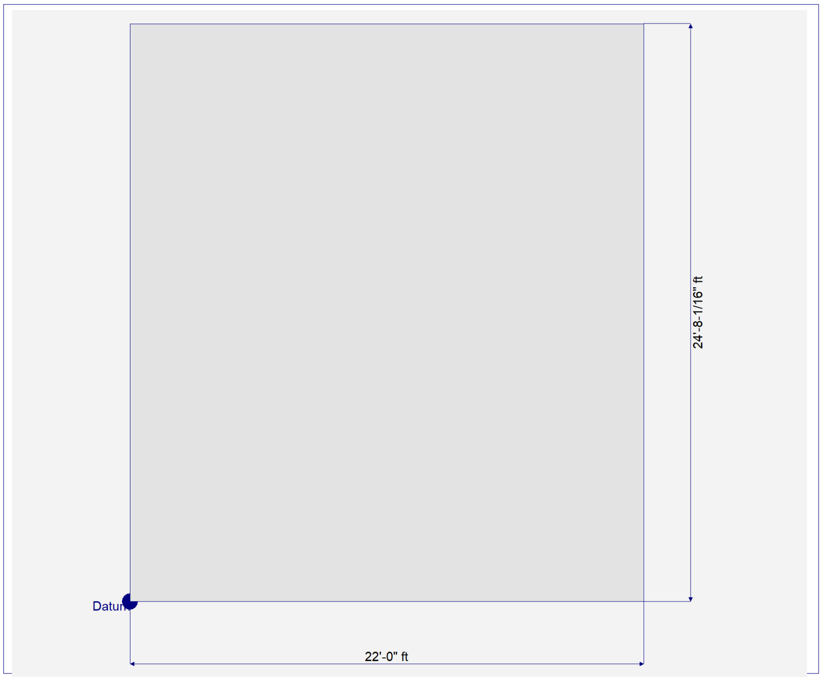
Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	22.0 ft
Effective Length 'd'	256.0 in
Nominal Block Thickness	8 in
Solid Grout?	Solid Grouted
Reinforcing in Field of Wall	
Vertical Bar Size #	6
Vertical Bar Spacing	32 in
Horiz. joint reinf. area (HJR)	0 in
HJR Spacing	24 in
Bond beam reinf. area	.31 in
Spacing of bond beams	48 in
In each chord cell:	
Vertical rebar size #	5
# Chord Cells @ Each End	2

Masonry Shear Wall

DESCRIPTION: V1

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
17.50	0.0	0.0	0.0	0.0	18.50	0.0

SHEAR ANALYSIS

	Bottom Level
Special Boundary Elements Req'd?	Not Req'd
Vu : Story Shear	43.987 k
for Load Combination	+1.368D+1.30E
Controlling Mu/(Vud)	0.69
Vn Masonry	266.675 k
Vn Steel	51.150 k
Vn Masonry + Vn Steel	317.825 k
Vn Max	433.321 k
Phi Vn	254.260 k
Ratio: Vu/PhiVn (controlling)	0.1730
Vertical As >= Av/3	OK
Vertical Bar Spacing <= 96"	OK

Masonry Shear Wall

DESCRIPTION: V1

AXIAL ANALYSIS

	Bottom Level
H / d Ratio	1.16
Pu	63.826 k
for Load Combination	+1.40D
Phi Pn	+1.40D k
Ratio: Pu/PhiPn (controlling)	0.02214

BENDING ANALYSIS

	Bottom Level
"a" : Flexural compression	3.05 in
Length of defined chord zone is >= the "a" dimension of the masonry (the compression zone)	OK
"d" : Eff depth to tension reinf	256.0
As-flex < As-max ?	0.620 <= 23.064
Mu	666.80 k
for Load Combination	+1.368D+1.30E
Phi Mn	709.99 k
Ratio: Mu/PhiMn (controlling)	0.9392

Force Summary

Load Combination	Values for Wall section			Resultant	Overturning	Uplift (k)	
Wall Level	Vu (k)	Mu (k)	Pu (k)	Ecc (ft)	Ratio	Left	Right
+1.40D							
Wall Level : 1			63.826				
+1.20D							
Wall Level : 1			54.708				
+0.90D							
Wall Level : 1			41.031				
+1.368D+1.30E							
Wall Level : 1	43.987	666.804	62.376	6.747	1.192		
+1.368D-1.30E							
Wall Level : 1	43.987	666.804	62.376	6.747	1.192		
+0.7318D+1.30E							
Wall Level : 1	43.987	666.804	33.363	12.615	1.192		
+0.7318D-1.30E							
Wall Level : 1	43.987	666.804	33.363	12.615	1.192		

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Project ID:
Project Descr:

Masonry Shear Wall

DESCRIPTION: V1

Footing Information

Footing Dimensions

Dist. Left	0.0 ft	f'c	3.0 ksi	Rebar Cover	3.0 in
Wall Length	22.0 ft	Fy	60.0 ksi	Footing Thickness	12.0 in
Dist. Right	0.0 ft			Width	2.0 ft
Total Ftg Length	22.0 ft				

Max Factored Soil Pressures

@ Left Side of Footing	22,635.5 psf
.... governing load comb	+1.368D-1.30E
@ Right Side of Footing	22,635.5 psf
.... governing load comb	+1.368D+1.30E

Max UNfactored Soil Pressures

@ Left Side of Footing	4,741.68 psf
.... governing load comb	+D-0.70E
@ Right Side of Footing	4,741.68 psf
.... governing load comb	+D+0.70E

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	@ Right End of Ftg
Overturning Moment	382.734 k-ft	382.734 k-ft
Resisting Moment	335.743 k-ft	335.743 k-ft
Stability Ratio	0.8772 : 1	0.8772 : 1
.... governing load comb	+0.60D+0.70E	+0.60D+0.70E

Footing Bending Design...

	@ Left End	@ Right End
Mu	0.0 k-ft	0.0 k-ft
Ru	0.0 psi	0.0 psi
As % Req'd	0.00180 in^2	0.00180 in^2
As Req'd in Footing Width	0.5184 in^2	0.5184 in^2

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Project Descr:

Masonry Shear Wall

DESCRIPTION: V2

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	2.0 ksi	Block Class	
Total Wall Height	24.670 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	28.0 ft	Fy - HJR	70.0 ksi	Min. Bending As %	0.00180
R: Resp. Mod Factor	5.0	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	28.0 ft
Effective Length 'd'	328.0 in
Nominal Block Thickness	8 in
Solid Grout?	Solid Grouted
Reinforcing in Field of Wall	
Vertical Bar Size #	6
Vertical Bar Spacing	32 in
Horiz. joint reinf. area (HJR)	in
HJR Spacing	24 in
Bond beam reinf. area	.31 in
Spacing of bond beams	48 in
In each chord cell:	
Vertical rebar size #	5
# Chord Cells @ Each End	2

Masonry Shear Wall

DESCRIPTION: V2

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
17.50	0.0	0.0	0.0	0.0	18.70	0.0

SHEAR ANALYSIS

	Bottom Level
Special Boundary Elements Req'd?	Not Req'd
Vu : Story Shear	49.685 k
for Load Combination	+1.368D+1.30E
Controlling Mu/(Vud)	0.53
Vn Masonry	371.031 k
Vn Steel	57.358 k
Vn Masonry + Vn Steel	428.389 k
Vn Max	599.69 k
Phi Vn	342.711 k
Ratio: Vu/PhiVn (controlling)	0.1450
Vertical As >= Av/3	OK
Vertical Bar Spacing <= 96"	OK

Masonry Shear Wall

DESCRIPTION: V2

AXIAL ANALYSIS

	Bottom Level
H / d Ratio	0.90
Pu	81.233 k
for Load Combination	+1.40D
Phi Pn	+1.40D k
Ratio: Pu/PhiPn (controlling)	0.02214

BENDING ANALYSIS

	Bottom Level
"a" : Flexural compression	3.05 in
Length of defined chord zone is >= the "a" dimension of the masonry (the compression zone)	OK
"d" : Eff depth to tension reinf	328.0
As-flex < As-max ?	0.620 <= 29.550
Mu	738.43 k
for Load Combination	+1.368D+1.30E
Phi Mn	910.87 k
Ratio: Mu/PhiMn (controlling)	0.8107

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			81.233				
+1.20D							
Wall Level : 1			69.629				
+0.90D							
Wall Level : 1			52.221				
+1.368D+1.30E							
Wall Level : 1	49.685	738.425	79.388	5.359	1.909		
+1.368D-1.30E							
Wall Level : 1	49.685	738.425	79.388	5.359	1.909		
+0.7318D+1.30E							
Wall Level : 1	49.685	738.425	42.462	10.019	1.909		
+0.7318D-1.30E							
Wall Level : 1	49.685	738.425	42.462	10.019	1.909		

Masonry Shear Wall

DESCRIPTION: V2

Footing Information

Footing Dimensions

Dist. Left	ft	f'c	3.0 ksi	Rebar Cover	3.0 in
Wall Length	28.0 ft	Fy	60.0 ksi	Footing Thickness	12.0 in
Dist. Right	ft			Width	2.0 ft
Total Ftg Length	28.0 ft				

Max Factored Soil Pressures

@ Left Side of Footing	5,683.36 psf
.... governing load comb	+1.368D-1.30E
@ Right Side of Footing	5,683.36 psf
.... governing load comb	+1.368D+1.30E

Max UNfactored Soil Pressures

@ Left Side of Footing	3,957.70 psf
.... governing load comb	+0.60D-0.70E
@ Right Side of Footing	3,957.70 psf
.... governing load comb	+0.60D+0.70E

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	@ Right End of Ftg
Overturning Moment	424.367 k-ft	424.367 k-ft
Resisting Moment	543.85 k-ft	543.85 k-ft
Stability Ratio	1.282 : 1	1.282 : 1
.... governing load comb	+0.60D+0.70E	+0.60D+0.70E

Footing Bending Design...

	@ Left End	@ Right End
Mu	0.0 k-ft	0.0 k-ft
Ru	0.0 psi	0.0 psi
As % Req'd	0.00180 in^2	0.00180 in^2
As Req'd in Footing Width	0.5184 in^2	0.5184 in^2

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Steel Beam

File: WALGP0402 - Calculations.ec6

Lic. # : KW-06003498 WDA&E

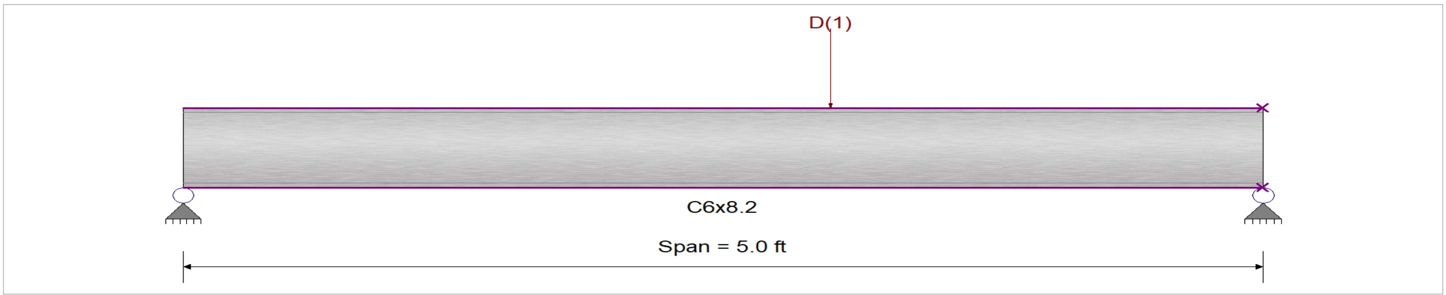
DESCRIPTION: Equipment support channel (spans between joists).

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 :	Minor Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Load(s) for Span Number 1
Point Load : D = 1.0 k @ 3.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.976** : 1	Maximum Shear Stress Ratio =	**0.081** : 1
Section used for this span	**C6x8.2**	Section used for this span	**C6x8.2**
Ma : Applied	1.369 k-ft	Va : Applied	0.6936 k
Mn / Omega : Allowable	1.403 k-ft	Vn/Omega : Allowable	8.518 k
Load Combination	+1.118D	Load Combination	+1.118D
Location of maximum on span	3.000 ft	Location of maximum on span	5.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.221 in	Ratio =	272 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

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	Mny	Mny/Omega	Cb	Rm	Va Max	Vny	Vny/Omega
D Only													
Dsgn. L = 5.00 ft	1	0.873	0.073	1.22		1.22	2.34	1.40	1.00	1.00	0.62	14.22	8.52
+0.60D													
Dsgn. L = 5.00 ft	1	0.524	0.044	0.73		0.73	2.34	1.40	1.00	1.00	0.37	14.22	8.52
+1.118D													
Dsgn. L = 5.00 ft	1	0.976	0.081	1.37		1.37	2.34	1.40	1.00	1.00	0.69	14.22	8.52
+1.088D													
Dsgn. L = 5.00 ft	1	0.950	0.079	1.33		1.33	2.34	1.40	1.00	1.00	0.68	14.22	8.52
+0.4823D													
Dsgn. L = 5.00 ft	1	0.421	0.035	0.59		0.59	2.34	1.40	1.00	1.00	0.30	14.22	8.52

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.2208	2.643		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.421	0.621
Overall MINimum	0.252	0.372
D Only	0.421	0.621
+0.60D	0.252	0.372

Steel Beam

DESCRIPTION: Equipment support channel (spans between joists).

Steel Section Properties : C6x8.2

Depth	=	6.000	in	I xx	=	13.10	in^4	J	=	0.074	in^4
Web Thick	=	0.200	in	S xx		4.35	in^3	Cw	=	4.70	in^6
Flange Width	=	1.920	in	R xx	=	2.340	in	Ro	=	2.650	in
Flange Thick	=	0.343	in	Zx	=	5.160	in^3	H	=	0.824	in
Area	=	2.390	in^2	I yy	=	0.687	in^4				
Weight	=	8.200	plf	S yy	=	0.488	in^3	Wno	=	3.170	in^2
Kdesign	=	0.813	in	R yy	=	0.536	in	Sw	=	0.610	in^4
				Zy	=	0.987	in^3	Qf	=	1.720	in^3
rts	=	0.643	in					Qw	=	2.620	in^3
Ycg	=	3.000	in					Wn2	=	1.980	
Xcg	=	0.512	in					Sw2	=	0.370	
Xp	=	0.199	in					Sw3	=	0.190	
Eo	=	0.599	in								

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Project Title:
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Steel Beam

Lic. # : KW-06003498 **WDA&E**

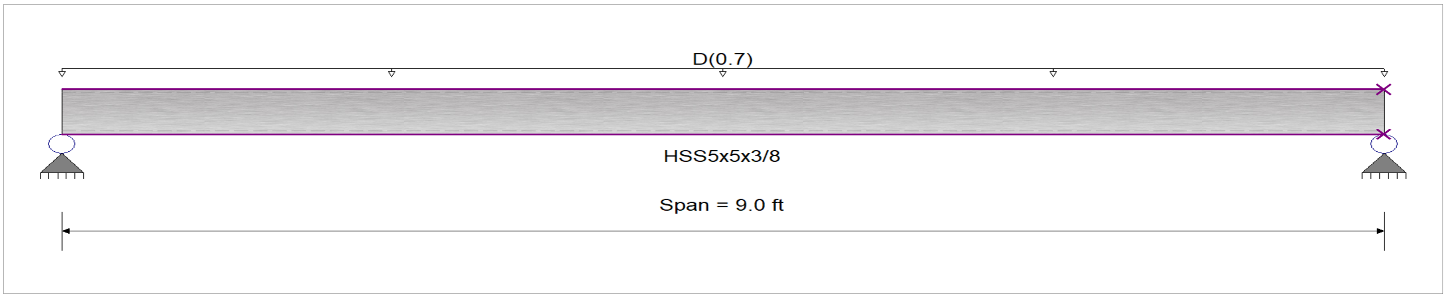
DESCRIPTION: lintel (gravity loads)

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 :	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 calculated and added to loading
Uniform Load : D = 0.70 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.301** : 1	Maximum Shear Stress Ratio =	**0.071** : 1
Section used for this span	**HSS5x5x3/8**	Section used for this span	**HSS5x5x3/8**
Ma : Applied	7.313 k-ft	Va : Applied	3.250 k
Mn / Omega : Allowable	24.331 k-ft	Vn/Omega : Allowable	45.601 k
Load Combination	D Only	Load Combination	D Only
Location of maximum on span	4.500 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	0.000 in	Ratio =	0 <360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.170 in	Ratio =	634 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

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 = 9.00 ft	1	0.301	0.071	7.31		7.31	40.63	24.33	1.00	1.00	3.25	76.15	45.60
+0.60D													
Dsgn. L = 9.00 ft	1	0.180	0.043	4.39		4.39	40.63	24.33	1.00	1.00	1.95	76.15	45.60

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.1702	4.526		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.250	3.250
Overall MINimum	1.950	1.950
D Only	3.250	3.250
+0.60D	1.950	1.950

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Steel Beam

DESCRIPTION: lintel (gravity loads)

Steel Section Properties : HSS5x5x3/8

Depth	=	5.000	in	I xx	=	21.70	in^4	J	=	36.100	in^4
				S xx		8.68	in^3				
Width	=	5.000	in	R xx	=	1.870	in				
Wall Thick	=	0.349	in	Zx	=	10.600	in^3				
Area	=	6.180	in^2	I yy	=	21.700	in^4	C	=	0.000	in^3
Weight	=	22.302	plf	S yy	=	8.680	in^3				
				R yy	=	1.870	in				
Ycg	=	2.500	in								

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Steel Beam

File: WALGP0402 - Calculations.ec6

Lic. # : KW-06003498 **WDA&E**

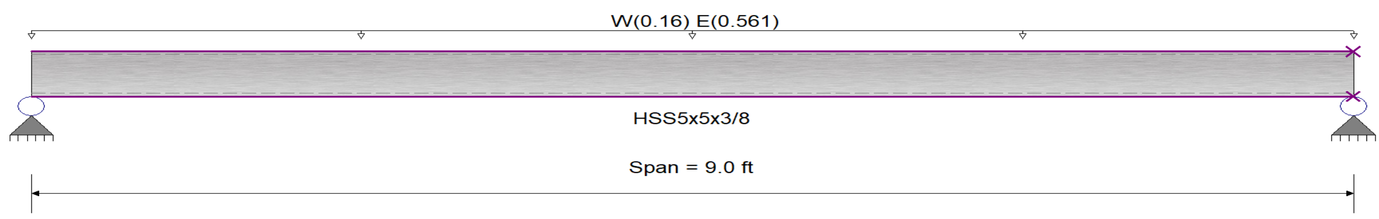
DESCRIPTION: lintel (lateral loads)

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 :	46.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : W = 0.0160, E = 0.05610 ksf, Tributary Width = 10.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.173** : 1	Maximum Shear Stress Ratio =	**0.041** : 1
Section used for this span	**HSS5x5x3/8**	Section used for this span	**HSS5x5x3/8**
Ma : Applied	4.202 k-ft	Va : Applied	1.868 k
Mn / Omega : Allowable	24.331 k-ft	Vn/Omega : Allowable	45.601 k
Load Combination	+D+0.70E	Load Combination	+D+0.70E
Location of maximum on span	4.500 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	0.132 in	Ratio =	816 >=360
Max Upward Transient Deflection	-0.132 in	Ratio =	816 >=360
Max Downward Total Deflection	0.098 in	Ratio =	1104 >=240.
Max Upward Total Deflection	-0.089 in	Ratio =	1208 >=240.

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	Mny	Mny/Omega	Cb	Rm	Va Max	Vny	Vny/Omega
D Only													
Dsgn. L = 9.00 ft	1	0.009	0.002	0.23		0.23	40.63	24.33	1.00	1.00	0.10	76.15	45.60
+D+0.60W													
Dsgn. L = 9.00 ft	1	0.049	0.012	1.20		1.20	40.63	24.33	1.00	1.00	0.53	76.15	45.60
+D-0.60W													
Dsgn. L = 9.00 ft	1	0.031	0.007		-0.75	0.75	40.63	24.33	1.00	1.00	0.33	76.15	45.60
+D+0.450W													
Dsgn. L = 9.00 ft	1	0.039	0.009	0.95		0.95	40.63	24.33	1.00	1.00	0.42	76.15	45.60
+D-0.450W													
Dsgn. L = 9.00 ft	1	0.021	0.005		-0.50	0.50	40.63	24.33	1.00	1.00	0.22	76.15	45.60
+0.60D+0.60W													
Dsgn. L = 9.00 ft	1	0.046	0.011	1.11		1.11	40.63	24.33	1.00	1.00	0.49	76.15	45.60
+0.60D-0.60W													
Dsgn. L = 9.00 ft	1	0.034	0.008		-0.84	0.84	40.63	24.33	1.00	1.00	0.37	76.15	45.60
+D+0.70E													
Dsgn. L = 9.00 ft	1	0.173	0.041	4.20		4.20	40.63	24.33	1.00	1.00	1.87	76.15	45.60
+D-0.70E													
Dsgn. L = 9.00 ft	1	0.154	0.037		-3.75	3.75	40.63	24.33	1.00	1.00	1.67	76.15	45.60
+D+0.5250E													
Dsgn. L = 9.00 ft	1	0.132	0.031	3.21		3.21	40.63	24.33	1.00	1.00	1.43	76.15	45.60
+D-0.5250E													
Dsgn. L = 9.00 ft	1	0.113	0.027		-2.76	2.76	40.63	24.33	1.00	1.00	1.23	76.15	45.60
+0.60D+0.70E													
Dsgn. L = 9.00 ft	1	0.169	0.040	4.11		4.11	40.63	24.33	1.00	1.00	1.83	76.15	45.60

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Steel Beam

DESCRIPTION: lintel (lateral loads)

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
+0.60D-0.70E													
Dsgn. L = 9.00 ft	1	0.158	0.037		-3.84	3.84	40.63	24.33	1.00	1.00	1.71	76.15	45.60

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	E Only * -1.0	-0.1322	4.526

Vertical Reactions

Support notation : Far left is #1   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-2.525	-2.525
Overall MINimum	0.100	0.100
D Only	0.100	0.100
+D+0.60W	0.532	0.532
+D-0.60W	-0.332	-0.332
+D+0.450W	0.424	0.424
+D-0.450W	-0.224	-0.224
+0.60D+0.60W	0.492	0.492
+0.60D-0.60W	-0.372	-0.372
+D+0.70E	1.868	1.868
+D-0.70E	-1.667	-1.667
+D+0.5250E	1.426	1.426
+D-0.5250E	-1.225	-1.225
+0.60D+0.70E	1.827	1.827
+0.60D-0.70E	-1.707	-1.707
W Only	0.720	0.720
-W	-0.720	-0.720
E Only	2.525	2.525
E Only * -1.0	-2.525	-2.525

Steel Section Properties : HSS5x5x3/8

Depth	=	5.000	in	I xx	=	21.70	in^4	J	=	36.100	in^4
				S xx		8.68	in^3				
Width	=	5.000	in	R xx	=	1.870	in				
Wall Thick	=	0.349	in	Zx	=	10.600	in^3				
Area	=	6.180	in^2	I yy	=	21.700	in^4	C	=	0.000	in^3
Weight	=	22.302	plf	S yy	=	8.680	in^3				
				R yy	=	1.870	in				
Ycg	=	2.500	in								

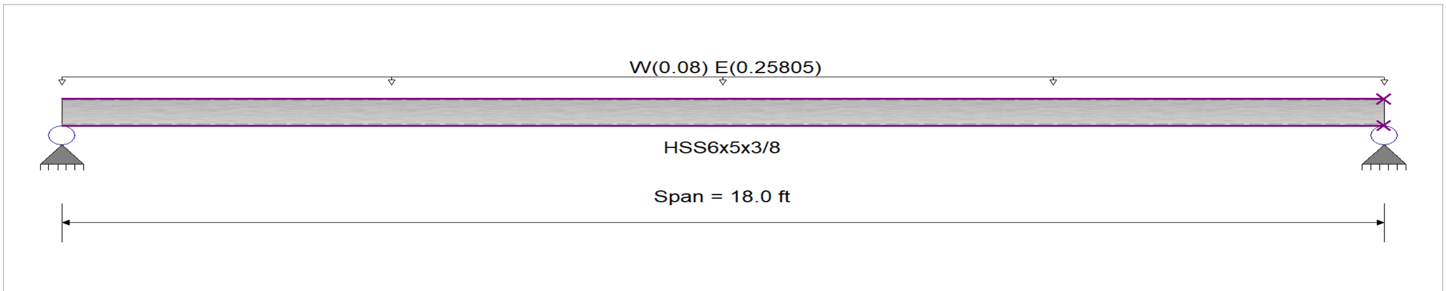
DESCRIPTION: Jamb (lateral loads)

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 :	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 calculated and added to loading
Uniform Load : W = 0.0160, E = 0.05161 ksf, Tributary Width = 5.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.270 : 1	Maximum Shear Stress Ratio =	0.033 : 1
Section used for this span	HSS6x5x3/8	Section used for this span	HSS6x5x3/8
Ma : Applied	8.539 k-ft	Va : Applied	1.898 k
Mn / Omega : Allowable	31.677 k-ft	Vn/Omega : Allowable	57.137 k
Load Combination	+1.215D+0.70E	Load Combination	+1.215D+0.70E
Location of maximum on span	9.000 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	0.000 in	Ratio =	0 <360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.496 in	Ratio =	436 >=240.
Max Upward Total Deflection	-0.400 in	Ratio =	540 >=240.

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 = 18.00 ft	1	0.032	0.004	1.01		1.01	52.90	31.68	1.00	1.00	0.22	95.42	57.14
+D+0.60W													
Dsgn. L = 18.00 ft	1	0.093	0.011	2.95		2.95	52.90	31.68	1.00	1.00	0.66	95.42	57.14
+D-0.60W													
Dsgn. L = 18.00 ft	1	0.030	0.004		-0.94	0.94	52.90	31.68	1.00	1.00	0.21	95.42	57.14
+D+0.450W													
Dsgn. L = 18.00 ft	1	0.078	0.010	2.46		2.46	52.90	31.68	1.00	1.00	0.55	95.42	57.14
+D-0.450W													
Dsgn. L = 18.00 ft	1	0.014	0.002		-0.45	0.45	52.90	31.68	1.00	1.00	0.10	95.42	57.14
+0.60D+0.60W													
Dsgn. L = 18.00 ft	1	0.080	0.010	2.55		2.55	52.90	31.68	1.00	1.00	0.57	95.42	57.14
+0.60D-0.60W													
Dsgn. L = 18.00 ft	1	0.042	0.005		-1.34	1.34	52.90	31.68	1.00	1.00	0.30	95.42	57.14
+1.215D+0.70E													
Dsgn. L = 18.00 ft	1	0.270	0.033	8.54		8.54	52.90	31.68	1.00	1.00	1.90	95.42	57.14
+1.215D-0.70E													
Dsgn. L = 18.00 ft	1	0.192	0.024		-6.09	6.09	52.90	31.68	1.00	1.00	1.35	95.42	57.14
+1.161D+0.5250E													
Dsgn. L = 18.00 ft	1	0.210	0.026	6.66		6.66	52.90	31.68	1.00	1.00	1.48	95.42	57.14
+1.161D-0.5250E													
Dsgn. L = 18.00 ft	1	0.136	0.017		-4.32	4.32	52.90	31.68	1.00	1.00	0.96	95.42	57.14
+0.3850D+0.70E													
Dsgn. L = 18.00 ft	1	0.243	0.030	7.70		7.70	52.90	31.68	1.00	1.00	1.71	95.42	57.14

WD Partners
7007 Discovery Blvd
Dublin, OH 43017
wdpartners.com

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

File: WALGP0402 - Calculations.ec6

Lic. # : KW-06003498 WDA&E

DESCRIPTION: Jamb (lateral loads)

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
+0.3850D-0.70E													
Dsgn. L = 18.00 ft	1	0.219	0.027		-6.93	6.93	52.90	31.68	1.00	1.00	1.54	95.42	57.14

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.70E	1	0.4960	9.051		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-2.322	-2.322
Overall MINimum	-0.100	-0.100
D Only	0.224	0.224
+D+0.60W	0.656	0.656
+D-0.60W	-0.208	-0.208
+D+0.450W	0.548	0.548
+D-0.450W	-0.100	-0.100
+0.60D+0.60W	0.566	0.566
+0.60D-0.60W	-0.298	-0.298
+D+0.70E	1.849	1.849
+D-0.70E	-1.402	-1.402
+D+0.5250E	1.443	1.443
+D-0.5250E	-0.996	-0.996
+0.60D+0.70E	1.760	1.760
+0.60D-0.70E	-1.492	-1.492
W Only	0.720	0.720
-W	-0.720	-0.720
E Only	2.322	2.322
E Only * -1.0	-2.322	-2.322

Steel Section Properties : HSS6x5x3/8

Depth	=	6.000	in	I xx	=	33.90	in^4	J	=	48.100	in^4
				S xx		11.30	in^3	Cw	=	18.20	in^6
Width	=	5.000	in	R xx	=	2.220	in				
Wall Thick	=	0.349	in	Zx	=	13.800	in^3				
Area	=	6.880	in^2	I yy	=	25.500	in^4	C	=	0.000	in^3
Weight	=	24.854	plf	S yy	=	10.200	in^3				
				R yy	=	1.920	in				
				Zy	=	12.200	in^3				
Ycg	=	3.000	in								

City of Puyallup
Building Division
333 S. Meridian, Puyallup, WA 98371
(253) 864-4165
www.cityofpuyallup.org

Comment Notice

Permit Application # B-21-0905

The City has completed the review of the above-mentioned permit submittal. Below please find the permit submittal review comments from your review team. Should you have any questions regarding the review comments, please contact the plan reviewer associated with the comment listed below.

Engineering Review (Reviewed By: Linda Lian, (253)841-5577, LindaL@PuyallupWA.gov)

- B-21-0905Fruitland Mutual Water Company. Please provide a Certificate of Water Availability. Plumbing Fixture Plan Review, WATER
- This permit application will be placed on hold by engineering until civil plans have been submitted, reviewed , and then approved by the Engineering Division as outline in the email sent to the applicant on January 27,2022. Cover Sheet C-1

Building Review (Reviewed By: David Leahy, (253)435-3618, DavidL@PuyallupWA.gov)

Letter below sent to applicant 11/23/2021

Permit: B-21-0905 Addition to Walmart #02403

Some items that need to be addressed on these plans before this application can be completely reviewed:

1.Will need to provide allowable area calculations since you are adding another 3,232 sq.ft. to this building.

2.The COMcheck information shown on the plans is not compatible with the requirements of the 2018 WSEC codes so not acceptable for this permit.

3.Need to show how ALL new requirements of the 2018 WSEC code is being met in the changes and most certainly in the additional new area, including all required commissioning items.

4.Provide an Engineering packet for the proposed racks per 2018 IBC section 2209 and related standards.

5.Provide an Engineering packet for the proposed new area and all changes in the existing building for this seismic zone.

6.Need to provide engineering for the new roof top units being proposed and make sure to include all seismic attachments for the curbs and units.

7.Provide seismic details for all the hanging units inside the building. None other than Air curt
(169 lbs)

8.Provide calculations for the roof drainage per the 2018 Uniform Plumbing Code on the plans. detail 7-S3

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9.Show the R value for the ridged insulation shown on the plans and how it meets all the requirements of the 2018 WSEC codes on the plans not just reference the specs. AND show how this meets the 2018 WSEC requirements for cooler and freezer areas, as well as other heated space if being installed under the floor area.

10.There is a note on page MP1, 28 that indicates trap primers if required by local codes. Per 2018 UPC section 1007 it would appear to be needed in this application. Please amend note and plans.

11.Provide the specs for the roof framing being proposed and make sure engineer of record also approves the framing being used for this project. (Deferred Submittal)

12.Special inspection statement on page SO indicates the CBC. That would not be applicable in Washington State. Please amend.

13.Show how this new area and area being remodeled meets the required ventilation requirements on the 2018 International Mechanical Code along with all the Washington State amendments on the plans.

14.Show all requirements for the roof ladder on page OP1.2 per the 2018 IMC section 306.5 and the Washington State amendments in detail on the plans.

15.All new doors being installed must have all information added to page A8 as this is what we review too and the inspectors use in the field, so must be complete and detailed.

16.If new suspended ceilings are being installed then plans must have details on the installation for this seismic zone for all ceilings.

To resubmit, you must address all comments and complete the resubmittal form. When you are ready to resubmit, you may do so by uploading a "new version" of the submittal requirement in the customer portal. In addition, you will need to pay the resubmittal fee at the time of your resubmittal. Your permit resubmittal will not be processed for review until this fee is paid. Please note, partial resubmittals will be deemed incomplete and returned.

If you need assistance with resubmitting your corrections, please contact the Permit Center.

Sincerely,
City of Puyallup Permit Center
(253) 864-4165 option 1
permitcenter@puyallupwa.gov