wallace design collective

COVENANT SHELL BUILDING
LEE'S SUMMIT, MO
PROJECT NO. 2220068

STRUCTURAL CALCULATIONS

04/05/2022
Missouri COA #001268

JAMES M. GRANICH, P.E.
ENGINEER OF RECORD

wallace design collective, pc
structural · civil · landscape · survey
1741 mcgee street
kansas city, missouri 64108
816.421.8282 · 800.364.5858
wallace.design

wallace design collective

CODE LOADING

wallace design collective, pc
structural · civil · landscape · survey
1741 mcgee street
kansas city, missouri 64108
816.421.8282 · 800.364.5858
wallace.design

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CODE CHECK

DATE: 3/10/22

TO: City of Lee's Summit, MO

PHONE: 8169691200 FAX:

ATTN: EMAIL:

PROJECT: # 2220068 Covenant Group Shell Building -- Lee's Summit, Missouri

BY: PHONE x VISIT OTHER TIME:

ITEM	DESCRIPTION	RESPONSE	
1. GOVERNING CODE			
A.	Local Building Code:	2018	IBC -- International Building Code
B.	Local Amendments:		
C.	Do State Building Code Requirements Differ?		
D.	Observations Required to be performed by EOR?	No	
E.	Special Inspections Final Report Required for Certificate of Occupancy?	Yes	
2. ROOF LIVE LOAD			
A.	Minimum Roof Live Load:	20 psf	
3. SNOW LOAD			
A.	Ground Snow Load, Pg:	20 psf	
B.	Can the roof snow be reduced below Pg as allowed by code?:	Yes	
4. WIND LOAD			
A.	Design Wind Speed:	109 mph	
B.	Risk Category	II	
5. SEISMIC LOAD			
A.	Mapped Spectral Response Acceleration, Ss:	.099	(short period, 0.2s)
B.	Mapped Spectral Response Acceleration, S1:	.068	(long period, 1.0s)
6. FROST DEPTH			
A.	Minimum Bearing Depth:	36 in.	

REMARKS:

Please notify the undersigned if the above information is incorrect or incomplete.

FROM: Tyler Monnett, P.E.

CC:

wallace design collective, pc
structural · civil · landscape · survey
123 north martin luther king jr. boulevard
tulsa, oklahoma 74103
918.584.5858 · 800.364.5858
wallace.design

ATC Hazards by Location

Search Information

Address:	400 NW Chipman Rd, Lee's Summit, MO 64063, USA
Coordinates:	38.92551679999999, -94.3894651
Elevation:	1000 ft
Timestamp:	2022-03-10T21:03:04.513Z
Hazard Type:	Wind

ASCE 7-16

MRI 10-Year	76 mph
MRI 25-Year	83 mph
MRI 50-Year	88 mph
MRI 100-Year	94 mph
Risk Category I	103 mph
Risk Category II	109 mph
Risk Category III	117 mph
Risk Category IV	122 mph

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	96 mph
Risk Category I	105 mph
Risk Category II	115 mph
Risk Category III-IV	120 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 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:	400 NW Chipman Rd, Lee's Summit, MO 64063, USA
Coordinates:	38.92551679999999, -94.3894651
Elevation:	1000 ft
Timestamp:	2022-03-10T21:03:35.967Z
Hazard Type:	Snow

ASCE 7-16		**ASCE 7-10**		**ASCE 7-05**	
Ground Snow Load	20 lb/sqft	Ground Snow Load	20 lb/sqft	Ground Snow Load	20 lb/sqft

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Disclaimer

azard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer.

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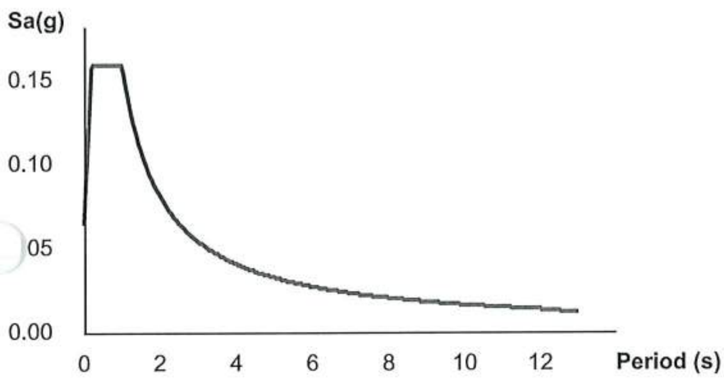
ATC Hazards by Location

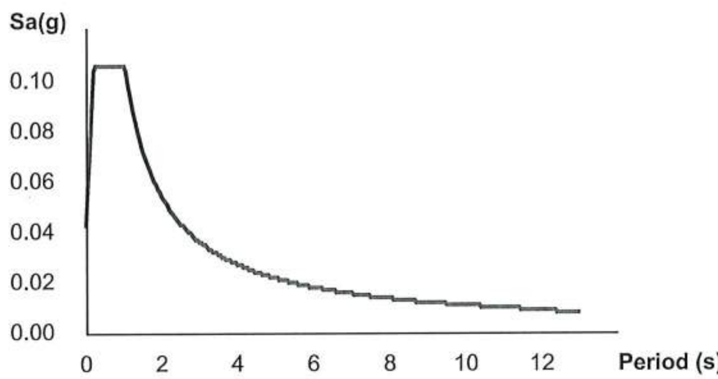
Search Information

Address:	400 NW Chipman Rd, Lee's Summit, MO 64063, USA
Coordinates:	38.92551679999999, -94.3894651
Elevation:	1000 ft
Timestamp:	2022-03-10T21:03:46.277Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.099	MCE_R ground motion (period=0.2s)
S_1	0.068	MCE_R ground motion (period=1.0s)
S_{MS}	0.159	Site-modified spectral acceleration value
S_{M1}	0.163	Site-modified spectral acceleration value
S_{DS}	0.106	Numeric seismic design value at 0.2s SA
S_{D1}	0.109	Numeric seismic design value at 1.0s SA

▾Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s

CR_S	0.927	Coefficient of risk (0.2s)
CR_1	0.877	Coefficient of risk (1.0s)
PGA	0.047	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.075	Site modified peak ground acceleration
T_L	12	Long-period transition period (s)
SsRT	0.099	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.107	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.068	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.078	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)

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

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Date	3/29/2022	Sheet No.	of
Job	2220068		
Subject	Dead Live Loads		

DEAD AND LIVE LOADS

ASCE 7-16 Table C3.1-1a and Table 4.3.1

ROOF LOADS

DEAD

ITEM	DESCRIPTION	UNIT WEIGHT (PSF)		TOTAL
Roof Covering	EPDM Membrane	1.0	x 0.50	0.5
Insulation	Polyisocyanurate Insulation (per inch thickness)	5.0	x 0.25	1.3
Deck	Plywood (per inch)	1.0	x 3.20	3.2
Ceiling	Acoustical Fiberboard	1.0	x 1.00	1.0
HVAC	HVAC Allowance (except sprinklers)			4.0
Sprinklers				2.0
Fire-Proofing				0.00
Waterproofing				0.00
Miscellaneous				0.00
Sub-Total				**12.0**
Secondary Frami	Wood Trusses at 2'-0" O.C.			3.0
Sub-Total				**15.0**
Primary Framing	Wood Girders (20' max. span)			1.0
Total				**16.0**

USE 20psf

ROOF LIVE/SNOW LOADS

ITEM	LOAD (PSF)
Roof Live Load	**20.0**
Factored Roof Snow Load, Pf	**30.0**

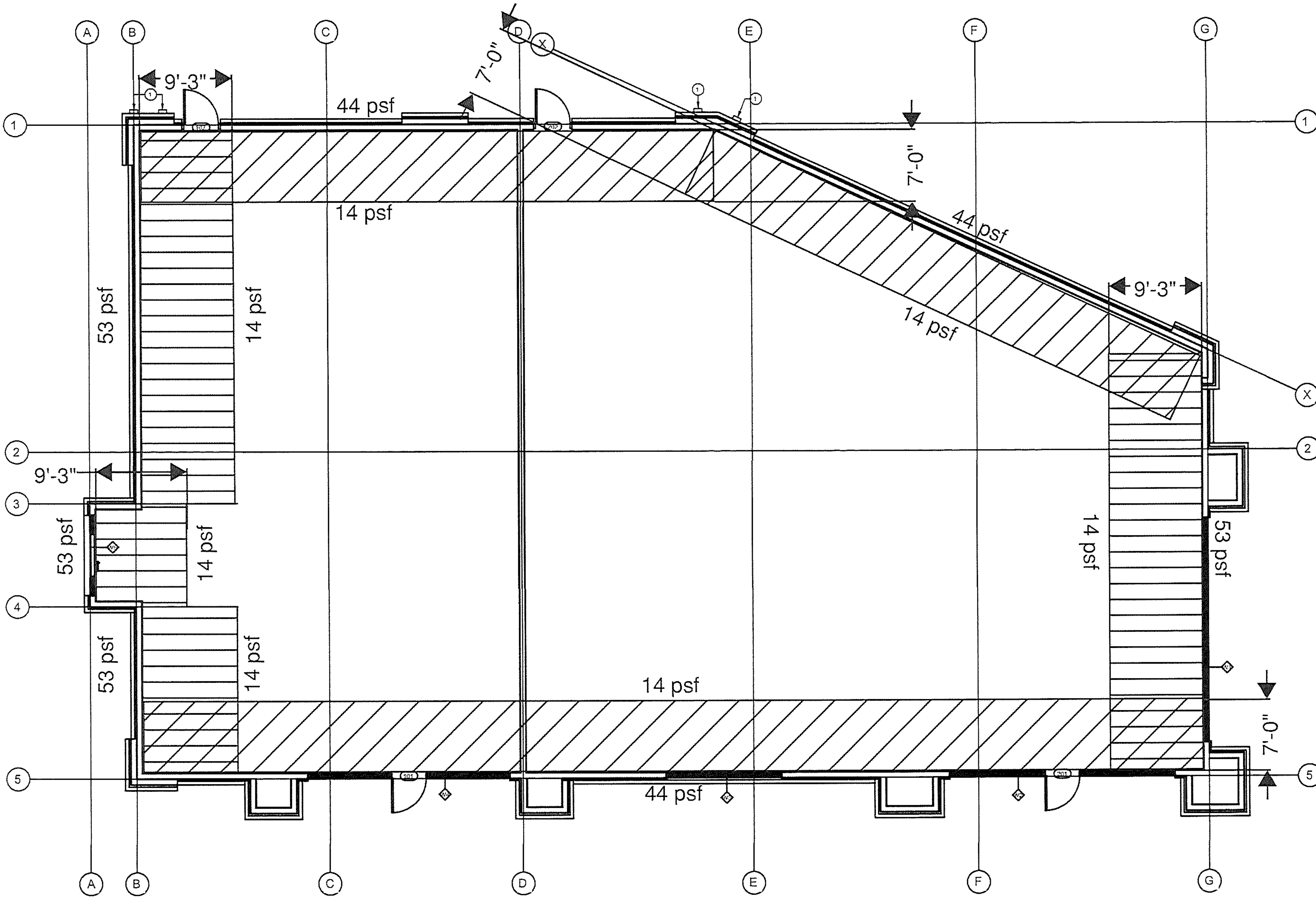
Date	3/29/2022	Sheet No.	of
Job	5/4/7982		
Subject	Dead Live Loads		

EXTERIOR WALL LOADS

DEAD

ITEM	DESCRIPTION	WEIGHT (PSF)
Wall 1	2x6 at 16" O.C., 5/8" Gypsum, Insulated, 3/8" Siding	12.00
Wall 2	Exterior Stud Walls with Brick Veneer	48.00
Wall 3	Select Exterior Wall Material	-
Wall 4	Select Exterior Wall Material	-
Wall 5	Select Exterior Wall Material	-
Wall 6	Select Exterior Wall Material	-

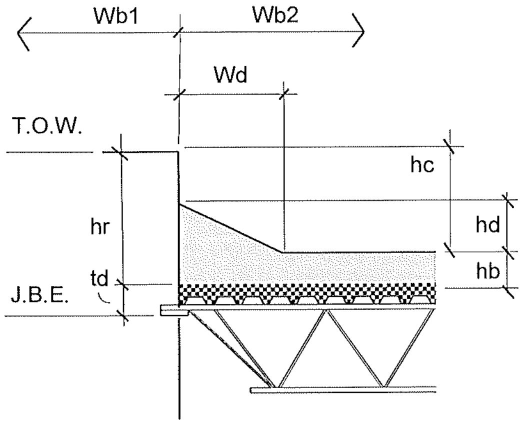
SNOW DRIFT SUMMARY



Revised 05/20/19, Carrie Johnson

Date	3/29/2022	Sheet No.	of
Project	2220068		
Subject	Perpendicular Snow Drift T1		

FLAT ROOF SNOW DRIFT - Joists Perpendicular to Wall
ASCE 7-16



Configuration

Note: For projections, input width of projection for Wb1 and input maximum of leeward and windward drift length for Wb2. For parapets, input the parapet width as Wb1.

1. **Input**

Dead Load =	**10**	psf
Roof Live Load =	**20**	psf
Pg, Ground Snow Load =	**20**	psf
Drift for parapet, projection, or upper roof?	**P**	(P), (PR) or (U)
Risk Category (I, II, III, or IV) =	**II**	Table 1.5-1
Ce, Exposure Factor =	**1.00**	Table 7.3-1
Ct, Thermal Factor =	**1.00**	Table 7.3-2
Use Pg minimum for drift calc's (Pf = Pg)?	**N**	(Y or N)

Geometry

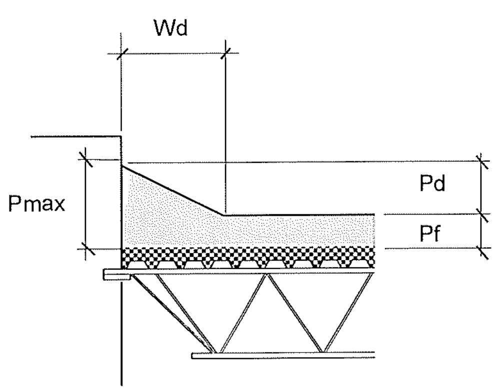
T.O.W., Top of Parapet Elevation =	**20.00**	feet
J.B.E., Joist Bearing Elevation =	**15.00**	feet
td, Thickness of Joist, Deck, and Insulation =	**0.00**	inches
Wb1, length of upper roof =	**5.00**	feet
Wb2, length of lower roof =	**57.00**	feet
S, Joist Spacing =	**2.00**	feet
L, Joist Span =	**28.50**	feet

2. **Balanced Snow Load Check**

Is, Importance Factor =	**1.00**	Table 1.5-2
Pf = 0.7 Ce Ct Is Pg =	**14.00**	psf (7.3-1)
Pm = Is Pg =	**20.00**	psf (7.3.4)
Rain on snow surcharge =	**5.00**	psf (7.10)
Pmin =	**20.00**	psf

3. **Drifted Snow Load Check**

Pf = 0.7 Ce Ct Is Pg =	**14.00**	psf
D = 0.13 Pg + 14.0 ≤ 30 pcf =	**16.60**	pcf
hb = Pf/D =	**0.84**	ft
Wb =	**57.00**	ft
hd = 0.75[0.43 Wb2^1/3 (Pg+10)^1/4-1.5] Is^	**1.78**	ft
hd + hb =	**2.62**	ft
hr =	**5.00**	ft
hc = hr - hb =	**4.16**	ft
Wd = 4 hd or 4 [hd^2/hc] ≤ 8 hc =	**7.10**	ft
Pmax = D (hd + hb) ≤ D hr =	**43.48**	psf
Pd =D hd ≤ D hc =	**29.48**	psf



Snow Drift

4. **Uniform Load Summary**

Drifted Snow Load

	Snow	Total	
R left =	**591.0**	**876.0**	lbs
R right =	**416.4**	**701.4**	lbs
M max =	**3096.2**	**5124.5**	ft-lbs
w base =	**28.0**	**48.0**	plf
w drift =	**59.0**	**79.0**	plf
w equiv =	**41.5**	**61.5**	plf *

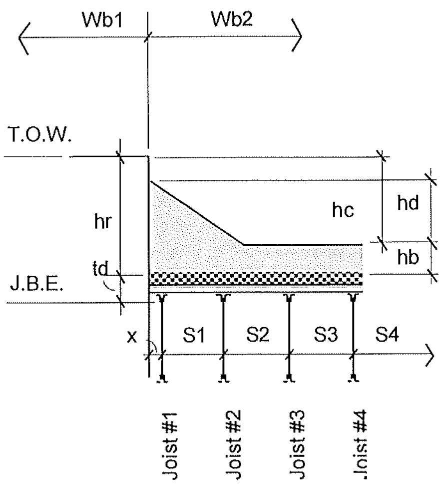
Load Without Drift

	Live	Total	
w (Live = 20 psf) =	**40.0**	**60.0**	plf

* indicates controlling load (drifted vs. undrifted)

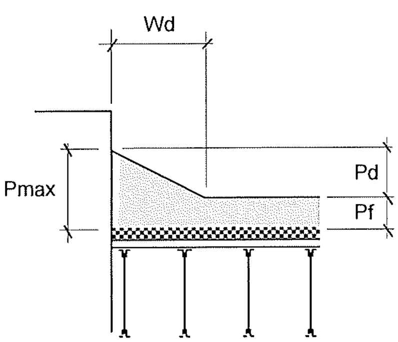
Date	3/29/2022	Sheet No.	of
Project	2220068		
Subject	Parallel Drift T2		

FLAT ROOF SNOW DRIFT - Joists Parallel to Wall
ASCE 7-16



Configuration

Note: For projections, input width of projection for Wb1 and input maximum of leeward and windward drift length for Wb2. For parapets, input the parapet width as Wb1.



Snow Drift

1. **Input**

Dead Load =	**10**	psf
Roof Live Load =	**20**	psf
Pg, Ground Snow Load =	**20**	psf
Drift for parapet, projection, or upper roof?	**P**	(P), (PR) or (U)
Risk Categoy (I, II, III, or IV) =	**II**	Table 1.5-1
Ce, Exposure Factor =	**1.0**	Table 7.3-1
Ct, Thermal Factor =	**1.0**	Table 7.3-2
Use Pg minimum for drift calc's (Pf = Pg)?	**N**	(Y or N)

Geometry

T.O.W., Top of Parapet Elevation =	**20.00**	feet
J.B.E., Joist Bearing Elevation =	**15.00**	feet
td, Thickness of Joist, Deck, and Insulation =	**0.00**	inches
Wb1, length of upper roof =	**5.00**	feet
Wb2, length of lower roof =	**97.00**	feet
x, Joist #1 dist. from wall =	**2.00**	feet
S1, First Joist Spacing =	**2.00**	feet
S2, Second Joist Spacing =	**2.00**	feet
S3, Third Joist Spacing =	**2.00**	feet
S4, Fourth Joist Spacing =	**2.00**	feet
S5, Fifth Joist Spacing =	**2.00**	feet

2. **Balanced Snow Load Check**

Is, Importance Factor =	**1.0**	Table 1.5-2
Pf = 0.7 Ce Ct Is Pg =	**14.00**	psf (7.3-1)
Pm = Is Pg =	**20.00**	psf (7.3.4)
Rain on snow surcharge =	**5.00**	psf (7.10)
Pmin =	**20.00**	psf

3. **Drifted Snow Load Check**

Pf = 0.7 Ce Ct Is Pg =	**14.00**	psf (7.3-1)
D = 0.13 Pg + 14.0 ≤ 30 pcf =	**16.60**	pcf (7.7-1)
hb = Pf/D =	**0.84**	feet
Wb =	**97.00**	feet
hd = 0.75[0.43 Wb2^1/3 (Pg+10)^1/4-1.5] Is^1	**2.34**	feet (Fig. 7.6-1)
hd + hb =	**3.18**	feet
hr =	**5.00**	feet
hc = hr - hb =	**4.16**	feet
Wd = 4 hd or 4 [hd^2/hc] ≤ 8 hc =	**9.35**	feet
Pmax = D (hd + hb) ≤ D hr =	**52.80**	psf
Pd =D hd ≤ D hc =	**38.80**	psf

4. **Uniform Load Summary**

Drifted Snow Load

	Snow	Total
w, wall	**50.7**	**60.7** plf *
w, Joist #1	**89.0**	**109.0** plf *
w, Joist #2	**72.4**	**92.4** plf *
w, Joist #3	**55.8**	**75.8** plf *
w, Joist #4	**39.2**	**59.2** plf
w, Joist #5	**29.5**	**49.5** plf

Balanced Load Check

	Pmin (20 psf)	Total
w, wall	**20.0**	**30.0** plf
w, Joist #1	**40.0**	**60.0** plf
w, Joist #2	**40.0**	**60.0** plf
w, Joist #3	**40.0**	**60.0** plf
w, Joist #4	**40.0**	**60.0** plf *
w, Joist #5	**40.0**	**60.0** plf *

* indicates controlling load (drifted vs. undrifted)

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Date 3/11/22 Sheet No. of

Job 2220068

Subject WIND LOADS

N/S ELEV

TOWER - 32.5' + 28' = 60.5'
MED PAR - 14' + 16.5' = 30.5'
LOW PAR - 10'

N/S WIND LOAD (1.0 W)

WALL (97')(15'/2)(24.3 psf) = 17.7 K

PARAPET (60.5')(9'/2)(60.1 psf) = 16.4 K
(30.5')(7'/2)(59.5 psf) = 6.4 K
(10')(5'/2)(58.2 psf) = 1.5 K

24.3 K (LRFD)
14.6 K (ASD)

E/W ELEV

TOWER - 32.5'
MED PAR - 0'
LOW PAR - 26.5'

E/W WIND LOAD (1.0 W)

WALL (57')(15'/2)(21.7 psf) = 9.3 K

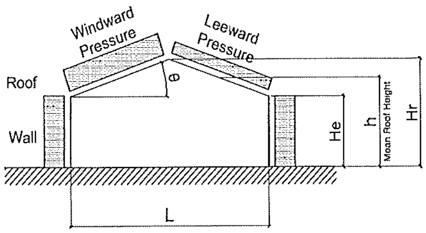
PARAPET (32.5')(9'/2)(60.1 psf) = 8.8 K
(26.5')(5'/2)(58.2 psf) = 3.9 K

22.0 K (LRFD)
13.2 K (ASD)

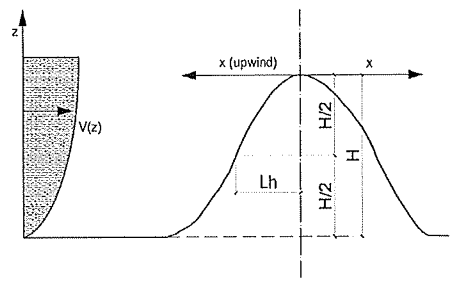
WALLACE DESIGN PROGRAM
Revised: 02/12/2019
Author: Katie Faulkner

Date	3/11/2022	Sheet	of
Job	2220068		
Subject	Wind NS/Walls EW Low Parapet		

WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD
ASCE 7-16, Chapters 26, 27 and 30



REFER TO FIGURE 27.3-1



2-D Ridge or Axisymmetrical Hill

REFER TO FIGURE 26.8-1

1. Input

Design Parameters

Basic Wind Speed, V =	**109** mph (Section 26.5, Fig. 1A-2D)
Exposure Category (B, C, or D) =	**C** (Section 26.7)
Building Risk Category (I, II, III, IV) =	**II** (Table 1.5-1)
Civil finished floor elevation (if unknown input 0) =	**0.00** feet (Sect. 26.9, Table 26.9-1)
Eave Height, He =	**15.00** feet
Max Building Height or Ridge Height above ground level, Hr =	**15.00** feet
Parapet Height above ground level, Hp =	**20.00** feet
Building Width Perpendicular to Wind, B =	**97.00** feet (max bldg dim)
Building Width Parallel to Wind, L =	**57.00** feet
Enclosure Classification =	**Enclosed Buildings** (Section 26.12)
Roof Configuration =	**Gabled, Hipped or Monoslope Roofs (ø ≤ 7)**
Angle of Plane of Roof From Horizontal, θ =	**1.19** degrees
Is building on or near a hill, ridge, or escarpment?	**N** (Y or N) (Section 26.8)
Height of Hill or Escarpment relative to upwind terrain, H =	**10.00** feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. Upwind to Point Where Elevation = H/2, Lh =	**10.00** feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. from Crest to Building Site, x =	**10.00** feet (Section 26.8, Fig. 26.8-1)
2D Ridge, 2D Escarpment, or Axisymmetrical Hill =	**E** (R, E, or H)
Is the building site upwind or downwind of the crest?	**DOWN** (up, down)

2. Calculations - Main Wind Force Resisting System

Equivalent Allowable Stress Design Wind Speed, Vasd =	84.43 mph (IBC 2018, 1609.3.1)	
Mean roof height, h =	**15.00** feet	
Kz, velocity pressure exposure coefficient at hz = 15ft =	**0.85** Table 26.10-1	(use with qz)
Kz, velocity pressure exposure coefficient at hh = 15ft =	**0.85** Table 26.10-1	(use with qh)
Kz, velocity pressure exposure coefficient at hp = 20ft =	**0.90** Table 26.10-1	(use with qp)
Kzt,topographic factor at hz = 15ft =	**1.00** Figure 26.8-1	(use with qz)
Kzt,topographic factor at hh = 15ft =	**1.00** Figure 26.8-1	(use with qh)
Kzt,topographic factor at hp = 20ft =	**1.00** Figure 26.8-1	(use with qp)
Kd, wind directionality factor =	**0.05** Table 26.6-1	
Ke, ground elevation factor at	**1.00** Table 26.9-1	
G, gust factor =	0.85 Section 26.11.4	
qz, velocity pressure at hz = 15ft =	**21.98** psf (Eq. 26.10-1)	
qh, velocity pressure at hh = 15ft =	**21.98** psf (Eq. 26.10-1)	
qp, velocity pressure at hp = 20ft =	**23.27** psf (Eq. 26.10-1)	

Walls: P = q(GCpf-GCpi) Eqn. 27.3-1	qz	GCp	GCpi	(1.0)P	(0.6)P
Windward pressure	21.98	0.68		14.9 psf	9 psf
	qh	GCp	GCpi	(1.0)P	(0.6)P
Leeward Pressure	21.98	-0.43		-9.3 psf	-5.6 psf
Sidewall pressure	21.98	-0.60	0.18	-17 psf	-10.2 psf
Internal Pressure	21.90		0.18	4 psf	2.4 psf

(1.0)W = (1.0)(Windward + Leeward Pressure) =	14.94 psf + 9.34 psf =	24.3 psf
(0.6)W =(0.6)(Windward + Leeward Pressure) =	8.97 psf + 5.6 psf =	14.6 psf

Parapets: Pp =qp(GCpn) Eqn. 27.3-3	qp	GCpn	(1.0)Pp	(0.6)Pp
Windward parapet pressure	23.27	1.5	34.9 psf	20.9 psf
Leeward parapet pressure	23.27	-1.0	-23.3 psf	-14 psf
Windward + Leeward Pressure	23.27	2.50	58.2 psf	**34.9 psf**

Roof Normal to Ridge (θ≥10 degrees)		qh	GCp	GCpi	(1.0)P	(0.6)P
Windward Pressure	case i	21.98	-0.60	0.18	**-17.2 psf**	**-10.3 psf**
	case ii	21.98	-0.15	0.18	**-7.3 psf**	**-4.4 psf**
Leeward Pressure		21.98	-0.26	0.18	**-9.8 psf**	**-5.9 psf**

Roof All Other Conditions	qh	GCp	GCpi	(1.0)P	(0.6)P
For 0 to h/2 = 0 ft to 7.5 ft	21.98	-0.77	0.18	**-20.8 psf**	**-12.5 psf**
h/2 to h = 7.5 ft to 15 ft	21.98	-0.77	0.18	**-20.8 psf**	**-12.5 psf**
h to 2h = 15 ft to 30 ft	21.98	-0.43	0.18	**-13.3 psf**	**-8 psf**
>2h = >30 ft	21.98	-0.26	0.18	**-9.6 psf**	**-5.7 psf**

Roof Overhangs Section 27.3.3	qh	GCp	GCpi	(1.0)P	(0.6)P
Maximum pressures	21.98	-0.77	0.68	**-31.8 psf**	**-19.1 psf**

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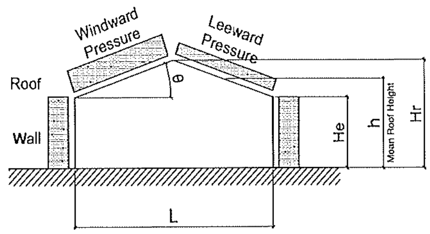
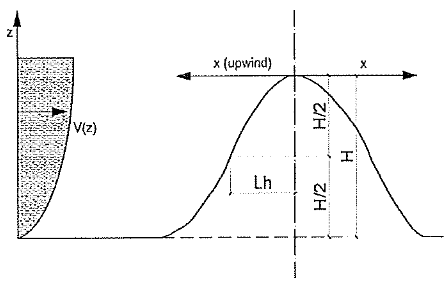
WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD
ASCE 7-16, Chapters 26, 27 and 30



REFER TO FIGURE 27.3-1



2-D Ridge or Axisymmetrical Hill

REFER TO FIGURE 26.8-1

1. Input

Design Parameters

Basic Wind Speed, V =	109	mph (Section 26.5, Fig. 1A-2D)
Exposure Category (B, C, or D) =	C	(Section 26.7)
Building Risk Category (I, II, III, IV) =	II	(Table 1.5-1)
Civil finished floor elevation (if unknown input 0) =	0.00	feet (Sect. 26.9, Table 26.9-1)
Eave Height, He =	15.00	feet
Max Building Height or Ridge Height above ground level, Hr =	15.00	feet
Parapet Height above ground level, Hp =	22.00	feet
Building Width Perpendicular to Wind, B =	97.00	feet (max bldg dim)
Building Width Parallel to Wind, L =	57.00	feet
Enclosure Classification =	Enclosed Buildings	(Section 26.12)
Roof Configuration =	Gabled, Hipped or Monoslope Roofs (ø ≤ 7)	
Angle of Plane of Roof From Horizontal, θ =	1.19	degrees
Is building on or near a hill, ridge, or escarpment?	N	(Y or N) (Section 26.8)
Height of Hill or Escarpment relative to upwind terrain, H =	10.00	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. Upwind to Point Where Elevation = H/2, Lh =	10.00	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. from Crest to Building Site, x =	10.00	feet (Section 26.8, Fig. 26.8-1)
2D Ridge, 2D Escarpment, or Axisymmetrical Hill =	E	(R, E, or H)
Is the building site upwind or downwind of the crest?	DOWN	(up, down)

2. Calculations - Main Wind Force Resisting System

Equivalent Allowable Stress Design Wind Speed, Vasd =	84.43	mph (IBC 2018, 1609.3.1)	
Mean roof height, h =	15.00	feet	
Kz, velocity pressure exposure coefficient at hz = 15ft =	0.85	Table 26.10-1	(use with qz)
Kz, velocity pressure exposure coefficient at hh = 15ft =	0.85	Table 26.10-1	(use with qh)
Kz, velocity pressure exposure coefficient at hp = 22ft =	0.92	Table 26.10-1	(use with qp)
Kzt,topographic factor at hz = 15ft =	1.00	Figure 26.8-1	(use with qz)
Kzt,topographic factor at hh = 15ft =	1.00	Figure 26.8-1	(use with qh)
Kzt,topographic factor at hp = 22ft =	1.00	Figure 26.8-1	(use with qp)
Kd, wind directionality factor =	0.85	Table 26.6-1	
Ke, ground elevation factor at	1.00	Table 26.9-1	
G, gust factor =	0.85	Section 26.11.4	
qz, velocity pressure at hz = 15ft =	21.98	psf (Eq. 26.10-1)	
qh, velocity pressure at hh = 15ft =	21.98	psf (Eq. 26.10-1)	
qp, velocity pressure at hp = 22ft =	23.78	psf (Eq. 26.10-1)	

Walls: P = q(GCpf-GCpi) Eqn. 27.3-1	**qz**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Windward pressure	21.98	0.68		14.9 psf	9 psf
	qh	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Leeward Pressure	21.98	-0.43		-9.3 psf	-5.6 psf
Sidewall pressure	21.98	-0.60	0.18	-17 psf	-10.2 psf
Internal Pressure	21.98		0.18	4 psf	2.4 psf

(1.0)W = (1.0)(Windward + Leeward Pressure) =	**14.94 psf + 9.34 psf =**	**24.3 psf.**
(0.6)W =(0.6)(Windward + Leeward Pressure) =	**8.97 psf + 5.6 psf =**	**14.6 psf**

Parapets: Pp =qp(GCpn) Eqn. 27.3-3	qp	GCpn	(1.0)Pp	(0.6)Pp
Windward parapet pressure	23.78	1.5	35.7 psf	21.4 psf
Leeward parapet pressure	23.78	-1.0	-23.8 psf	-14.3 psf
Windward + Leeward Pressure	23.78	2.50	**59.5 psf**	**35.7 psf**

Roof Normal to Ridge (θ≥10 degrees)		**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Windward Pressure	case i	21.98	-0.60	0.18	**-17.2 psf**	**-10.3 psf**
	case ii	21.98	-0.15	0.18	**-7.3 psf**	**-4.4 psf**
Leeward Pressure		21.98	-0.26	0.18	**-9.8 psf**	**-5.9 psf**

Roof All Other Conditions	**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
For 0 to h/2 = 0 ft to 7.5 ft	21.98	-0.77	0.18	**-20.8 psf**	**-12.5 psf**
h/2 to h = 7.5 ft to 15 ft	21.98	-0.77	0.18	**-20.8 psf**	**-12.5 psf**
h to 2h = 15 ft to 30 ft	21.98	-0.43	0.18	**-13.3 psf**	**-8 psf**
>2h = >30 ft	21.98	-0.26	0.18	**-9.6 psf**	**-5.7 psf**

Roof Overhangs Section 27.3.3	qh	GCp	GCpi	(1.0)P	(0.6)P
Maximum pressures	21.98	-0.77	0.68	**-31.8 psf**	**-19.1 psf**

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Author: Katie Faulkner

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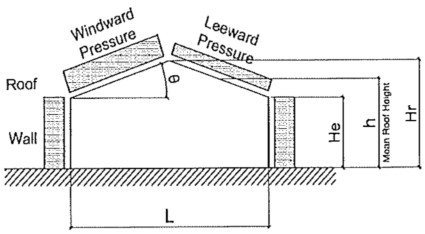
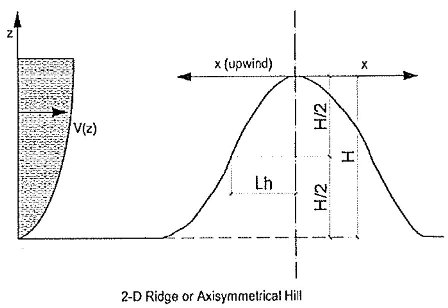
WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD
ASCE 7-16, Chapters 26, 27 and 30



REFER TO FIGURE 27.3-1



REFER TO FIGURE 26.8-1

1. Input

Design Parameters

Basic Wind Speed, V =	**109**	mph (Section 26.5, Fig. 1A-2D)
Exposure Category (B, C, or D) =	**C**	(Section 26.7)
Building Risk Category (I, II, III, IV) =	**II**	(Table 1.5-1)
Civil finished floor elevation (if unknown input 0) =	**0.00**	feet (Sect. 26.9, Table 26.9-1)
Eave Height, He =	**15.00**	feet
Max Building Height or Ridge Height above ground level, Hr =	**15.00**	feet
Parapet Height above ground level, Hp =	**24.00**	feet
Building Width Perpendicular to Wind, B =	**97.00**	feet (max bldg dim)
Building Width Parallel to Wind, L =	**57.00**	feet
Enclosure Classification =	**Enclosed Buildings**	(Section 26.12)
Roof Configuration =	**Gabled, Hipped or Monoslope Roofs (ø ≤ 7)**	
Angle of Plane of Roof From Horizontal, θ =	**1.19**	degrees
Is building on or near a hill, ridge, or escarpment?	**N**	(Y or N) (Section 26.8)
Height of Hill or Escarpment relative to upwind terrain, H =	**10.00**	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. Upwind to Point Where Elevation = H/2, Lh =	**10.00**	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. from Crest to Building Site, x =	**10.00**	feet (Section 26.8, Fig. 26.8-1)
2D Ridge, 2D Escarpment, or Axisymmetrical Hill =	**E**	(R, E, or H)
Is the building site upwind or downwind of the crest?	**DOWN**	(up, down)

2. Calculations - Main Wind Force Resisting System

Equivalent Allowable Stress Design Wind Speed, Vasd =	84.43	mph (IBC 2018, 1609.3.1)	
Mean roof height, h =	15.00	feet	
Kz, velocity pressure exposure coefficient at hz = 15ft =	0.85	Table 26.10-1	(use with qz)
Kz, velocity pressure exposure coefficient at hh = 15ft =	0.85	Table 26.10-1	(use with qh)
Kz, velocity pressure exposure coefficient at hp = 24ft =	0.93	Table 26.10-1	(use with qp)
Kzt,topographic factor at hz = 15ft =	1.00	Figure 26.8-1	(use with qz)
Kzt,topographic factor at hh = 15ft =	1.00	Figure 26.8-1	(use with qh)
Kzt,topographic factor at hp = 24ft =	1.00	Figure 26.8-1	(use with qp)
Kd, wind directionality factor =	0.85	Table 26.6-1	
Ke, ground elevation factor at	1.00	Table 26.9-1	
G, gust factor =	0.85	Section 26.11.4	
qz, velocity pressure at hz = 15ft =	21.98	psf (Eq. 26.10-1)	
qh, velocity pressure at hh = 15ft =	21.98	psf (Eq. 26.10-1)	
qp, velocity pressure at hp = 24ft =	24.04	psf (Eq. 26.10-1)	

Walls: P = q(GCpf-GCpi) Eqn. 27.3-1	**qz**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Windward pressure	21.98	0.68		14.9 psf	9 psf
	qh	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Leeward Pressure	21.98	-0.43		-9.3 psf	-5.6 psf
Sidewall pressure	21.98	-0.60	0.18	-17 psf	-10.2 psf
Internal Pressure	21.98		0.18	4 psf	2.4 psf

(1.0)W = (1.0)(Windward + Leeward Pressure) = **14.94 psf + 9.34 psf = 24.3 psf**
(0.6)W =(0.6)(Windward + Leeward Pressure) = **8.97 psf + 5.6 psf = 14.6 psf**

Parapets: Pp =qp(GCpn) Eqn. 27.3-3	**qp**	**GCpn**	**(1.0)Pp**	**(0.6)Pp**
Windward parapet pressure	24.04	1.5	36.1 psf	21.6 psf
Leeward parapet pressure	24.04	-1.0	-24 psf	-14.4 psf
Windward + Leeward Pressure	24.04	2.50	**60.1 psf**	**36.1 psf**

Roof Normal to Ridge (θ≥10 degrees)		**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Windward Pressure	case i	21.98	-0.60	0.18	**-17.2 psf**	**-10.3 psf**
	case ii	21.98	-0.15	0.18	**-7.3 psf**	**-4.4 psf**
Leeward Pressure		21.98	-0.26	0.18	**-9.8 psf**	**-5.9 psf**

Roof All Other Conditions	**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
For 0 to h/2 = 0 ft to 7.5 ft	21.98	-0.77	0.18	**-20.8 psf**	**-12.5 psf**
h/2 to h = 7.5 ft to 15 ft	21.98	-0.77	0.18	**-20.8 psf**	**-12.5 psf**
h to 2h = 15 ft to 30 ft	21.98	-0.43	0.18	**-13.3 psf**	**-8 psf**
>2h = >30 ft	21.98	-0.26	0.18	**-9.6 psf**	**-5.7 psf**

Roof Overhangs Section 27.3.3	**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Maximum pressures	21.98	-0.77	0.68	**-31.8 psf**	**-19.1 psf**

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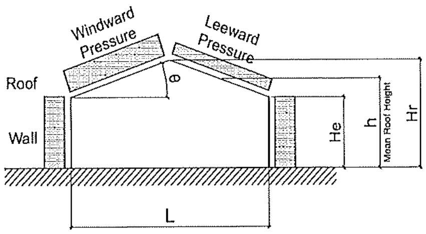
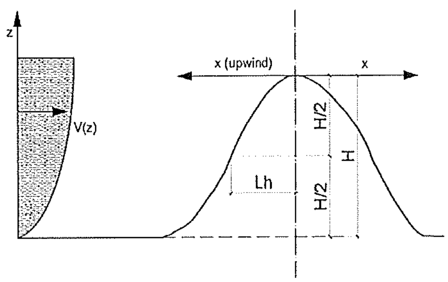
WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD
ASCE 7-16, Chapters 26, 27 and 30



REFER TO FIGURE 27.3-1



2-D Ridge or Axisymmetrical Hill

REFER TO FIGURE 26.8-1

1. Input

Design Parameters

Basic Wind Speed, V =	109	mph (Section 26.5, Fig. 1A-2D)
Exposure Category (B, C, or D) =	C	(Section 26.7)
Building Risk Category (I, II, III, IV) =	II	(Table 1.5-1)
Civil finished floor elevation (if unknown input 0) =	0.00	feet (Sect. 26.9, Table 26.9-1)
Eave Height, He =	15.00	feet
Max Building Height or Ridge Height above ground level, Hr =	15.00	feet
Parapet Height above ground level, Hp =	20.00	feet
Building Width Perpendicular to Wind, B =	57.00	feet (max bldg dim)
Building Width Parallel to Wind, L =	97.00	feet
Enclosure Classification =	**Enclosed Buildings**	(Section 26.12)
Roof Configuration =	**Gabled, Hipped or Monoslope Roofs (ø ≤ 7)**	
Angle of Plane of Roof From Horizontal, θ =	1.19	degrees
Is building on or near a hill, ridge, or escarpment?	N	(Y or N) (Section 26.8)
Height of Hill or Escarpment relative to upwind terrain, H =	10.00	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. Upwind to Point Where Elevation = H/2, Lh =	10.00	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. from Crest to Building Site, x =	10.00	feet (Section 26.8, Fig. 26.8-1)
2D Ridge, 2D Escarpment, or Axisymmetrical Hill =	E	(R, E, or H)
Is the building site upwind or downwind of the crest?	DOWN	(up, down)

2. Calculations - Main Wind Force Resisting System

Equivalent Allowable Stress Design Wind Speed, Vasd =	84.43	mph (IBC 2018, 1609.3.1)	
Mean roof height, h =	15.00	feet	
Kz, velocity pressure exposure coefficient at hz = 15ft =	0.85	Table 26.10-1	(use with qz)
Kz, velocity pressure exposure coefficient at hh = 15ft =	0.85	Table 26.10-1	(use with qh)
Kz, velocity pressure exposure coefficient at hp = 20ft =	0.90	Table 26.10-1	(use with qp)
Kzt,topographic factor at hz = 15ft =	1.00	Figure 26.8-1	(use with qz)
Kzt,topographic factor at hh = 15ft =	1.00	Figure 26.8-1	(use with qh)
Kzt,topographic factor at hp = 20ft =	1.00	Figure 26.8-1	(use with qp)
Kd, wind directionality factor =	0.85	Table 26.6-1	
Ke, ground elevation factor at	1.00	Table 26.9-1	
G, gust factor =	0.85	Section 26.11.4	
qz, velocity pressure at hz = 15ft =	21.98	psf (Eq. 26.10-1)	
qh, velocity pressure at hh = 15ft =	21.98	psf (Eq. 26.10-1)	
qp, velocity pressure at hp = 20ft =	23.27	psf (Eq. 26.10-1)	

Walls: P = q(GCpf-GCpi) Eqn. 27.3-1	**qz**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Windward pressure	21.98	0.68		14.9 psf	9 psf
	qh	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Leeward Pressure	21.98	-0.31		-6.7 psf	-4 psf
Sidewall pressure	21.98	-0.60	0.18	-17 psf	-10.2 psf
Internal Pressure	21.98		0.18	4 psf	2.4 psf

(1.0)W = (1.0)(Windward + Leeward Pressure) =	**14.94 psf + 6.72 psf =**	**21.7 psf**
(0.6)W =(0.6)(Windward + Leeward Pressure) =	**8.97 psf + 4.03 psf =**	**13 psf**

Parapets: Pp =qp(GCpn) Eqn. 27.3-3	**qp**	**GCpn**	**(1.0)Pp**	**(0.6)Pp**
Windward parapet pressure	23.27	1.5	34.9 psf	20.9 psf
Leeward parapet pressure	23.27	-1.0	-23.3 psf	-14 psf
Windward + Leeward Pressure	23.27	2.50	**58.2 psf**	**34.9 psf**

Roof Normal to Ridge (θ≥10 degrees)		**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Windward Pressure	case i	21.98	-0.60	0.18	**-17 psf**	**-10.2 psf**
	case ii	21.98	-0.15	0.18	**-7.3 psf**	**-4.4 psf**
Leeward Pressure		21.98	-0.26	0.18	**-9.6 psf**	**-5.7 psf**

Roof All Other Conditions	**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
For 0 to h/2 = 0 ft to 7.5 ft	21.98	-0.77	0.18	**-20.8 psf**	**-12.5 psf**
h/2 to h = 7.5 ft to 15 ft	21.98	-0.77	0.18	**-20.8 psf**	**-12.5 psf**
h to 2h = 15 ft to 30 ft	21.98	-0.43	0.18	**-13.3 psf**	**-8 psf**
>2h = >30 ft	21.98	-0.26	0.18	**-9.6 psf**	**-5.7 psf**

Roof Overhangs Section 27.3.3	**qh**	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Maximum pressures	21.98	-0.77	0.68	**-31.8 psf**	**-19.1 psf**

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WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD
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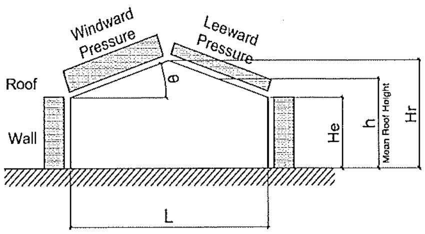
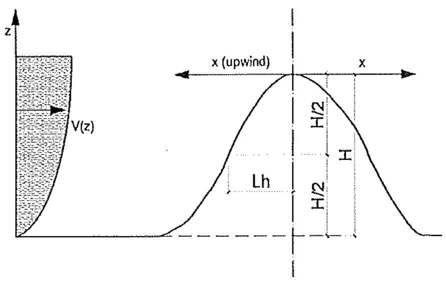


REFER TO FIGURE 27.3-1



2-D Ridge or Axisymmetrical Hill

REFER TO FIGURE 26.8-1

1. Input

Design Parameters

Basic Wind Speed, V =	109	mph (Section 26.5, Fig. 1A-2D)
Exposure Category (B, C, or D) =	C	(Section 26.7)
Building Risk Category (I, II, III, IV) =	II	(Table 1.5-1)
Civil finished floor elevation (if unknown input 0) =	0.00	feet (Sect. 26.9, Table 26.9-1)
Eave Height, He =	15.00	feet
Max Building Height or Ridge Height above ground level, Hr =	15.00	feet
Parapet Height above ground level, Hp =	22.00	feet
Building Width Perpendicular to Wind, B =	57.00	feet (max bldg dim)
Building Width Parallel to Wind, L =	97.00	feet
Enclosure Classification =	Enclosed Buildings	(Section 26.12)
Roof Configuration =	Gabled, Hipped or Monoslope Roofs (ø ≤ 7)	
Angle of Plane of Roof From Horizontal, θ =	1.19	degrees
Is building on or near a hill, ridge, or escarpment?	N	(Y or N) (Section 26.8)
Height of Hill or Escarpment relative to upwind terrain, H =	10.00	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. Upwind to Point Where Elevation = H/2, Lh =	10.00	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. from Crest to Building Site, x =	10.00	feet (Section 26.8, Fig. 26.8-1)
2D Ridge, 2D Escarpment, or Axisymmetrical Hill =	E	(R, E, or H)
Is the building site upwind or downwind of the crest?	DOWN	(up, down)

2. Calculations - Main Wind Force Resisting System

Equivalent Allowable Stress Design Wind Speed, Vasd =	84.43	mph (IBC 2018, 1609.3.1)	
Mean roof height, h =	15.00	feet	
Kz, velocity pressure exposure coefficient at hz = 15ft =	0.85	Table 26.10-1	(use with qz)
Kz, velocity pressure exposure coefficient at hh = 15ft =	0.85	Table 26.10-1	(use with qh)
Kz, velocity pressure exposure coefficient at hp = 22ft =	0.92	Table 26.10-1	(use with qp)
Kzt,topographic factor at hz = 15ft =	1.00	Figure 26.8-1	(use with qz)
Kzt,topographic factor at hh = 15ft =	1.00	Figure 26.8-1	(use with qh)
Kzt,topographic factor at hp = 22ft =	1.00	Figure 26.8-1	(use with qp)
Kd, wind directionality factor =	0.85	Table 26.6-1	
Ke, ground elevation factor at	1.00	Table 26.9-1	
G, gust factor =	0.85	Section 26.11.4	
qz, velocity pressure at hz = 15ft =	21.98	psf (Eq. 26.10-1)	
qh, velocity pressure at hh = 15ft =	21.98	psf (Eq. 26.10-1)	
qp, velocity pressure at hp = 22ft =	23.78	psf (Eq. 26.10-1)	

Walls: P = q(GCpf-GCpi) Eqn. 27.3-1	qz	GCp	GCpi	(1.0)P	(0.6)P
Windward pressure	21.98	0.68		14.9 psf	9 psf
	qh	GCp	GCpi	(1.0)P	(0.6)P
Leeward Pressure	21.98	-0.31		-6.7 psf	-4 psf
Sidewall pressure	21.98	-0.60	0.18	-17 psf	-10.2 psf
Internal Pressure	21.98		0.18	4 psf	2.4 psf

(1.0)W = (1.0)(Windward + Leeward Pressure) =	14.94 psf + 6.72 psf =	21.7 psf
(0.6)W =(0.6)(Windward + Leeward Pressure) =	8.97 psf + 4.03 psf =	13 psf

Parapets: Pp =qp(GCpn) Eqn. 27.3-3	qp	GCpn	(1.0)Pp	(0.6)Pp
Windward parapet pressure	23.78	1.5	35.7 psf	21.4 psf
Leeward parapet pressure	23.78	-1.0	-23.8 psf	-14.3 psf
Windward + Leeward Pressure	23.78	2.50	59.5 psf	35.7 psf

Roof Normal to Ridge (θ≥10 degrees)		qh	GCp	GCpi	(1.0)P	(0.6)P
Windward Pressure	case i	21.98	-0.60	0.18	-17 psf	-10.2 psf
	case ii	21.98	-0.15	0.18	-7.3 psf	-4.4 psf
Leeward Pressure		21.98	-0.26	0.18	-9.6 psf	-5.7 psf

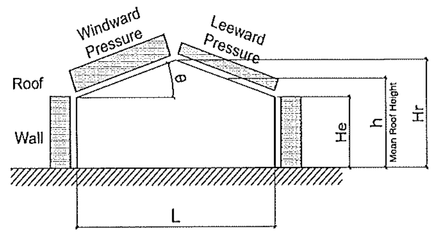
Roof All Other Conditions	qh	GCp	GCpi	(1.0)P	(0.6)P
For 0 to h/2 = 0 ft to 7.5 ft	21.98	-0.77	0.18	-20.8 psf	-12.5 psf
h/2 to h = 7.5 ft to 15 ft	21.98	-0.77	0.18	-20.8 psf	-12.5 psf
h to 2h = 15 ft to 30 ft	21.98	-0.43	0.18	-13.3 psf	-8 psf
>2h = >30 ft	21.98	-0.26	0.18	-9.6 psf	-5.7 psf

Roof Overhangs Section 27.3.3	qh	GCp	GCpi	(1.0)P	(0.6)P
Maximum pressures	21.98	-0.77	0.68	-31.8 psf	-19.1 psf

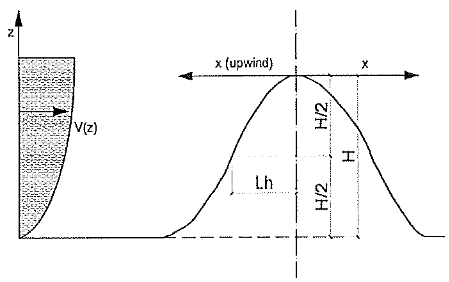
WALLACE DESIGN PROGRAM
Revised: 02/12/2019
Author: Katie Faulkner

Date	3/11/2022	Sheet	of
Job	220068		
Subject	Wind EW/Wall NS Tower		

WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD
ASCE 7-16, Chapters 26, 27 and 30



REFER TO FIGURE 27.3-1



2-D Ridge or Axisymmetrical Hill

REFER TO FIGURE 26.8-1

1. Input

Design Parameters

Basic Wind Speed, V =	109	mph (Section 26.5, Fig. 1A-2D)
Exposure Category (B, C, or D) =	C	(Section 26.7)
Building Risk Category (I, II, III, IV) =	II	(Table 1.5-1)
Civil finished floor elevation (if unknown input 0) =	0.00	feet (Sect. 26.9, Table 26.9-1)
Eave Height, He =	15.00	feet
Max Building Height or Ridge Height above ground level, Hr =	15.00	feet
Parapet Height above ground level, Hp =	24.00	feet
Building Width Perpendicular to Wind, B =	57.00	feet (max bldg dim)
Building Width Parallel to Wind, L =	97.00	feet
Enclosure Classification =	Enclosed Buildings	(Section 26.12)
Roof Configuration =	Gabled, Hipped or Monoslope Roofs (ø ≤ 7)	
Angle of Plane of Roof From Horizontal, θ =	1.19	degrees
Is building on or near a hill, ridge, or escarpment?	N	(Y or N) (Section 26.8)
Height of Hill or Escarpment relative to upwind terrain, H =	10.00	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. Upwind to Point Where Elevation = H/2, Lh =	10.00	feet (Section 26.8, Fig. 26.8-1)
Horiz. Dist. from Crest to Building Site, x =	10.00	feet (Section 26.8, Fig. 26.8-1)
2D Ridge, 2D Escarpment, or Axisymmetrical Hill =	E	(R, E, or H)
Is the building site upwind or downwind of the crest?	DOWN	(up, down)

2. Calculations - Main Wind Force Resisting System

Equivalent Allowable Stress Design Wind Speed, Vasd =	84.43	mph (IBC 2018, 1609.3.1)	
Mean roof height, h =	15.00	feet	
Kz, velocity pressure exposure coefficient at hz = 15ft =	0.85	Table 26.10-1	(use with qz)
Kz, velocity pressure exposure coefficient at hh = 15ft =	0.85	Table 26.10-1	(use with qh)
Kz, velocity pressure exposure coefficient at hp = 24ft =	0.93	Table 26.10-1	(use with qp)
Kzt,topographic factor at hz = 15ft =	1.00	Figure 26.8-1	(use with qz)
Kzt,topographic factor at hh = 15ft =	1.00	Figure 26.8-1	(use with qh)
Kzt,topographic factor at hp = 24ft =	1.00	Figure 26.8-1	(use with qp)
Kd, wind directionality factor =	0.85	Table 26.6-1	
Ke, ground elevation factor at	1.00	Table 26.9-1	
G, gust factor =	0.85	Section 26.11.4	
qz, velocity pressure at hz = 15ft =	21.98	psf (Eq. 26.10-1)	
qh, velocity pressure at hh = 15ft =	21.98	psf (Eq. 26.10-1)	
qp, velocity pressure at hp = 24ft =	24.04	psf (Eq. 26.10-1)	

Walls: P = q(GCpf-GCpi) Eqn. 27.3-1	qz	GCp	GCpi	(1.0)P	(0.6)P
Windward pressure	21.98	0.68		14.9 psf	9 psf
	qh	**GCp**	**GCpi**	**(1.0)P**	**(0.6)P**
Leeward Pressure	21.98	-0.31		-6.7 psf	-4 psf
Sidewall pressure	21.98	-0.60	0.18	-17 psf	-10.2 psf
Internal Pressure	21.98		0.18	4 psf	2.4 psf

(1.0)W = (1.0)(Windward + Leeward Pressure) =	14.94 psf + 6.72 psf =	21.7 psf
(0.6)W =(0.6)(Windward + Leeward Pressure) =	8.97 psf + 4.03 psf =	13 psf

Parapets: Pp =qp(GCpn) Eqn. 27.3-3	qp	GCpn	(1.0)Pp	(0.6)Pp
Windward parapet pressure	24.04	1.5	36.1 psf	21.6 psf
Leeward parapet pressure	24.04	-1.0	-24 psf	-14.4 psf
Windward + Leeward Pressure	24.04	2.50	60.1 psf	36.1 psf

Roof Normal to Ridge (θ≥10 degrees)		qh	GCp	GCpi	(1.0)P	(0.6)P
Windward Pressure	case i	21.98	-0.60	0.18	-17 psf	-10.2 psf
	case ii	21.98	-0.15	0.18	-7.3 psf	-4.4 psf
Leeward Pressure		21.98	-0.26	0.18	-9.6 psf	-5.7 psf

Roof All Other Conditions	qh	GCp	GCpi	(1.0)P	(0.6)P
For 0 to h/2 = 0 ft to 7.5 ft	21.98	-0.77	0.18	-20.8 psf	-12.5 psf
h/2 to h = 7.5 ft to 15 ft	21.98	-0.77	0.18	-20.8 psf	-12.5 psf
h to 2h = 15 ft to 30 ft	21.98	-0.43	0.18	-13.3 psf	-8 psf
>2h = >30 ft	21.98	-0.26	0.18	-9.6 psf	-5.7 psf

Roof Overhangs Section 27.3.3	qh	GCp	GCpi	(1.0)P	(0.6)P
Maximum pressures	21.98	-0.77	0.68	-31.8 psf	-19.1 psf

WALLACE DESIGN PROGRAM
Revised: 02/12/2019
Author: Katie Faulkner

Date	3/22/2022	Sheet	of
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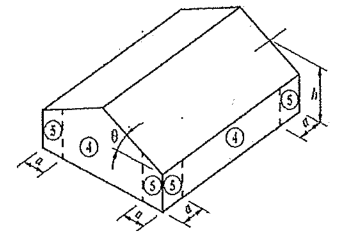
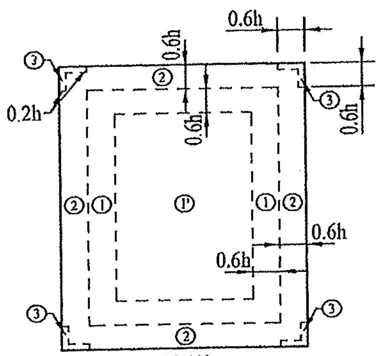
Subject	Wind EW/Walls NS Med Parapet		

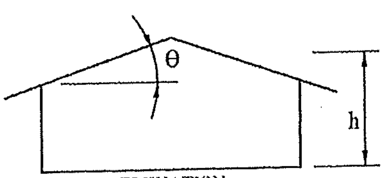
WIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD
ASCE 7-16, Chapters 27 and 30



ELEVATION



PLAN



ELEVATION

REFER TO FIGURE 30.3-2A

3. Input - Component and Cladding Elements

Tributary Area for Wall Components, 1 =	10.00	square feet
Tributary Area for Wall Components, 2 =	500.00	square feet
Tributary Area for Parapet Components, 1 =	10.00	square feet
Tributary Area for Parapet Components, 2 =	50.00	square feet
Tributary Area for Roof Components, 1 =	10.00	square feet
Tributary Area for Roof Components, 2 =	100.00	square feet
Tributary Area for Overhangs or Canopies, 1 =	10.00	square feet
Tributary Area for Overhangs or Canopies, 2 =	50.00	square feet

4. Calculations - Component and Cladding Elements

Kh, velocity pressure exposure coefficient at hh = 15ft =	0.85	Table 26.10-1	(use with qh)
Kh, velocity pressure exposure coefficient at hp = 22ft =	0.92	Table 26.10-1	(use with qp)
Kzt,topographic factor at hh = 15ft =	1.00	Figure 26.8-1	(use with qh)
Kzt,topographic factor at hp = 22ft =	1.00	Figure 26.8-1	(use with qp)
Kd, wind directionality factor =	0.85	Table 26.6-1	
Ke, ground elevation factor at	1.00	Table 26.9-1	
G, gust factor =	0.85	Section 26.11.4	
qh, velocity pressure at hh = 15ft =	21.98	psf (Eq. 26.10-1)	
qp, velocity pressure at hp = 22ft =	23.78	psf (Eq. 26.10-1)	

Walls: trib. Area = 10 sq. ft.		qh	GCp	GCpi	(1.0)P	(0.6)P
Zone 4	Interior Zone	21.98	-0.99	0.18	-25.7 psf	-15.4 psf
Zone 5	End Zone	21.98	-1.26	0.18	-31.6 psf	-19 psf
Zone 4 and 5		21.98	0.90	-0.18	23.7 psf	14.2 psf

Walls: trib. Area = 500 sq. ft.		qh	GCp	GCpi	(1.0)P	(0.6)P
Zone 4	Interior Zone	21.98	-0.72	0.18	-19.8 psf	-11.9 psf
Zone 5	End Zone	21.98	-0.72	0.18	-19.8 psf	-11.9 psf
Zone 4 and 5		21.98	0.63	-0.18	17.8 psf	10.7 psf

Parapets: trib. Area = 10 sq. ft.			qp	GCp	GCpi	(1.0)P	(0.6)P
Case A	Zone 4	Interior Zone	23.78	2.97	0.00	70.6 psf	42.4 psf
	Zone 5	End Zone	23.78	3.78	0.00	89.9 psf	53.9 psf
Case B	Zone 4	Interior Zone	23.78	1.89	0.00	45 psf	27 psf
	Zone 5	End Zone	23.78	2.16	0.00	51.4 psf	30.8 psf

Parapets: trib. Area = 50 sq. ft.			qp	GCp	GCpi	(1.0)P	(0.6)P
Case A	Zone 4	Interior Zone	23.78	2.53	0.00	60.1 psf	36 psf
	Zone 5	End Zone	23.78	3.00	0.00	71.4 psf	42.8 psf
Case B	Zone 4	Interior Zone	23.78	1.67	0.00	39.7 psf	23.8 psf
	Zone 5	End Zone	23.78	1.83	0.00	43.4 psf	26.1 psf

Roofs: trib. Area = 10 sq. ft.	qh	GCp	GCpi	(1.0)P	(0.6)P
Corner Zone (3)	21.98	-3.20	0.18	-74.3 psf	-44.6 psf
End Zone (2)	21.98	-2.30	0.18	-54.5 psf	-32.7 psf
Interior Zone (1)	21.98	-1.70	0.18	-41.3 psf	-24.8 psf
Interior Zone (1')	21.98	-0.90	0.18	-23.7 psf	-14.2 psf
Positive (All Zones)	21.98	0.30	-0.18	16 psf	9.6 psf

Roofs: trib. Area = 100 sq. ft.	qh	GCp	GCpi	(1.0)P	(0.6)P
Corner Zone (3)	21.98	-2.14	0.18	-51 psf	-30.6 psf
End Zone (2)	21.98	-1.77	0.18	-42.9 psf	-25.7 psf
Interior Zone (1)	21.98	-1.29	0.18	-32.3 psf	-19.4 psf
Interior Zone (1')	21.98	-0.90	0.18	-23.7 psf	-14.2 psf
Positive (All Zones)	21.98	0.20	-0.18	16 psf	9.6 psf

Overhangs: trib. Area = 10 sq. ft.	qh	GCpn	(1.0)Pp	(0.6)Pp
Corner Zone (3)	21.98	-3.20	-70.3 psf	-42.2 psf
End Zone (2)	21.98	-2.30	-50.5 psf	-30.3 psf
Interior Zone (1)	21.98	-1.70	-37.4 psf	-22.4 psf
Interior Zone (1')	21.98	-1.70	-37.4 psf	-22.4 psf
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Overhangs: trib. Area = 50 sq. ft.	qh	GCpn	(1.0)Pp	(0.6)Pp
Corner Zone (3)	21.98	-2.34	-51.3 psf	-30.8 psf
End Zone (2)	21.98	-1.81	-39.7 psf	-23.8 psf
Interior Zone (1)	21.98	-1.63	-35.8 psf	-21.5 psf
Interior Zone (1')	21.98	-1.63	-35.8 psf	-21.5 psf
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a, end zone width = Min. of 10% L and .4H but not < 4% L or 3' = 5.7 feet (Fig. 30.3-1)

Notes:
1. The gust factor of 0.85 is based on a building with a natural frequency of > 1 Hz. For other buildings, the gust factor must be calculated.
2. GCp for walls include a 10% reduction when angle of roof is 10 deg or less. (Figure 30.3-1, Footnote 5)
3. If a parapet equal to 3 ft or higher is provided around the perimeter of a roof with a slope of ≤ 7°, the roof corner zones may be treated as end zones. (Fig. 30.3-2A, Footnote 5)

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Subject SEISMIC LOAD SUMMARY

BUILDING WEIGHT

ROOF - 20psf

(4878sf)(20psf) = 97.56K

BRICK - 48psf

(105sf + 45sf + 100sf + 98sf + 116sf + 84sf + 103sf + 113sf + 100sf + 100sf + 74sf + 35sf + 64sf + 136sf + 62sf)(48psf) = (1335sf)(48psf) = 64.08K

STUD WALL - 12psf

(825sf + 340sf + 996sf + 598sf + 563sf)(12psf) = (3322sf)(12psf) = 39.86K

TOTAL WEIGHT

97.56K + 64.08K + 39.86K = 201.5K

SEISMIC LOAD

(201.5K)(0.016) = 3.2K

Date	3/29/2022	Sheet No.	of
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SEISMIC LOAD SUMMARY

2018 IBC (Ch: 16) and ASCE 7-16 (Ch: 11 to 13)

1. Input

Spectral Response Acceleration for Short Periods, Ss =	0.099	
Spectral Response Acceleration for 1-second Periods, S1 =	0.068	
Risk Category =	II	(IBC Table 1604.5 & ASCE: Table 1-5-1)
Site Classification (A,B,C,D,E,F) =	D	(ASCE 7 Ch.20 Table 20.3-1)
Is Site Class Assumed or Known?	Known	(ASCE 7 Section 11.4.4)
Basic Structural System	BEARING WALL SYSTEMS	(Table 12.2-1)
Lateral Force Resisting System	Light Frame (wood) Walls w/ wood structural panels rated for shear resistance	(Table 12.2-1)
ρx , redundancy in x-dir.= (Redundancy is either 1.0 or 1.3)	1.00	(ASCE 7 Section 12.3.4)
ρy , redundancy in y-dir.= (Redundancy is either 1.0 or 1.3)	1.00	(ASCE 7 Section 12.3.4)

r, =1.0 for Seismic Design Category B and C, RE: ASCE 7 Section 12.3.4.1 for additional exceptions.

Is Structure regular with a period < .5 sec?	Yes	(Yes or No, ASCE 7 Section 12.8.1.3)
Is Structure short period with a rigid diaphragm?	No	(Yes or No, ASCE 7 Section 11.6)
Is Structure short period w/ non-rigid diaphragm & vertical elements of seismic force-resisting system spaced at 40' oc	No	(Yes or No, ASCE 7 Section 11.6)
Does Structure have a flexible diaphragm?	N	(Yes or No, ASCE 7 Section 11.6)

(For Wall anchorage requirements per Section 12.11.2.1)

Span length of flexible diaphragm -x dir. =	115	feet (input 0 for rigid diaphragm)
Span length of flexible diaphragm -y-dir. =	63	feet

2. Determine Design Spectral Response Accelerations and Seismic Design Category, Section 11.6:

Response Modification Factor, R =	6.5	(Table 12.2-1)
Overstrength Factor, Ωo = (refer to footnote b for .5 reduction for Flexible Diaphragms)	3	(Table 12.2-1)
Deflection Amplification Factor, Cd =	4	(Table 12.2-1)

Acceleration for Short Period

Site Coefficient, Fa =	1.60	(IBC Table 1613.2.3(1), ASCE 7 Table 11.4-1)
Site Adjusted Spectral Response Acceleration for Short Periods, Sms =	0.158	(IBC Section 1613.2.3, ASCE 7 Section 11.4.4)

Acceleration for 1-Second Period

Site Coefficient, Fv =	2.40	(IBC Table 1613.2.3(2), ASCE 7 Table 11.4-2)
Site Adjusted Spectral Response Acceleration for 1-second Periods, Sm1 =	0.163	(IBC Section 1613.2.3, ASCE 7 Section 11.4.4)
Design Spectral Response Acceleration for Short Periods, Sds =	0.106	(IBC Section 1613.2.4 and ASCE 7 Section 11.4.5)
Seismic Design Category based on short period =	A	
Design Spectral Response Acceleration for 1-second Periods, Sd1 =	0.109	(IBC Section 1613.2.4 and ASCE 7 Section 11.4.5)
Seismic Design Category based on 1-second period =	B	
Design Response Spectrum, Ts =	1.030	seconds (Section 11.4.6)
Approximate Fundamental Period, Ta =	0.500	seconds (Section 12.8.2.1)
Fundamental Period, T, shall not exceed Ta * Cu =	0.800	seconds (Section 12.8.2)
Can the Seismic Design Category be based on the short period alone?	No	(IBC Section 1613.2.5.1, ASCE 7 Section 11.6)
Seismic Design Category =	B	(Most severe case except as allowed by Sect 11.6)

3. Seismic Base Shear for the Lateral Force Resisting System using the Equivalent Lateral Force Procedure, Section 12.8:

a. Calculation of Seismic Base Shear Coefficient:

Seismic Importance Factor, I_e =	1.00	(ASCE 7 Table 1.5-2)
Cs = (Sds/(R / I_e)) =	0.016	(ASCE Equation 12.8-2, Section 12.8.1.3)

	Strength (1.0E)	ASD (0.7E)
b. Seismic Base Shear, Section 12.8.1:		
V = Cs W =	0.016 W	0.011 W
c. Horizontal Seismic Load, Section 12.4.2.1=		
For the X-direction: Eh=	0.016 W	0.011 W
For the Y-direction: Eh=	0.016 W	0.011 W
d. Vertical Seismic Load Component, Section 12.4.2.2:		
Ev = 0.2 Sds D =	0.021 D	0.015 D

For structures in SDC B and for the design of foundations using ASD, Ev may be taken as zero. (Section 12.4.2.2)

	Strength (1.0E)	ASD (0.7E)
e. Find the Design Seismic Shear for the Diaphragm, Section 12.10.1.1:		
Force shall not be less than 0.2*Sds*Ie*wpx =	0.021 W	0.015 W
but need not exceed 0.4*Ip*Sds wpx =	0.042 W	0.030 W
For a one story building, Fpx =	0.021 W	0.015 W
f. For collector elements in Seismic Design Categories C through F, Section 12.10.2		
Emh = Ωo V =	0.049 W	0.034 W

Notes:

1 A building that is low rise (one or two story) building with a short period is assumed for calculation of Seismic Response Coefficient, Cs.
2 The values for design spectral response acceleration assume a regular structure of 5 stories or less with a period, T < 0.5 seconds
3 The values for design forces for the diaphragm assume no offsets or changes in the stiffness of the vertical components
4 Section 1613.1 of 2018 IBC excludes the detailing requirements of Chapter 14 of ASCE 7.
5 Per Section 1613.2.2 and 11.4.3, if site investigations performed per ASCE 7 Chpt 20 reveal rock conditions consistent with Site Class B, but site-specific velocity measurements are not made, Fa and Fv shall = 1.0.

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SEISMIC LOAD SUMMARY
2018 IBC (Ch: 16) and ASCE 7-16 (Ch: 11 to 13)

4. Minimum Continuous Load Path, Interconnection and Connection to supports, Section 12.1.3 and 12.1.4:

	Strength (1.0E)	ASD (0.7E)	
a. Continuous Load Path and Interconnections, Section 12.1.3:			
F_p= 0.133 S_{ds} W_p or .05 W_p min. =	**0.050 Wp**	**0.035 Wp**	(Section 12.1.3)
b. Connection to Supports, Section 12.1.4 :			
F_p= .05 * dead + live reaction =	**0.050 Rd+l**	**0.035 Rd+l**	(Section 12.1.4)

5. Structural Walls and Anchorage, Section 12.11

	Strength (1.0E)	ASD (0.7E)	
a. Minimum Out-of-Plane Forces on Structural Walls, Section 12.11.1:			
F_p= **0.40** Ie Sds wp or .10 wp min =	**0.100 Wp**	**0.070 Wp**	(Section 12.11.1)

b. Minimum anchorage connection of structural walls to supporting construction, Section 12.11.2.1 and 12.11.2:

Per 12.11.2.2, the strength design force for steel elements with the exception of anchor bolts and reinforcing steel shall be increased by 1.4 times.

			Strength (1.0E)	ASD (0.7E)
For loading in the x-direction:				
k_a= 1.0+L_f/100 or max 2.0 =	2.00			
Fp= 0.4 S_{ds} k_a I_e W_p or .2 k_a I_e Wp min. =		Connections at Flexible Diaphragms:	**0.400 Wp**	**0.280 Wp**
F_p * 1.4 for steel elements per 12.11.2.2.2			**0.560 Wp**	**0.392 Wp**
k_a= **1.0**				
Fp= 0.4 S_{ds} k_a I_e W_p or .2 k_a I_e Wp min. =		For Connections not at Flexible Diaphragms:	**0.200 Wp**	**0.140 Wp**
Fp * 1.4 for steel elements per 12.11.2.2.2			**0.280 Wp**	**0.196 Wp**
For loading in the y-direction:				
k_a= 1.0+L_f/100 or max 2.0 =	1.63			
Fp= 0.4 S_{ds} k_a I_e W_p or .2 k_a I_e Wp min. =		For Connections at Flexible Diaphragms:	**0.326 Wp**	**0.228 Wp**
Fp * 1.4 for steel elements per 12.11.2.2.2			**0.456 Wp**	**0.319 Wp**
k_a= **1.0**				
Fp= 0.4 S_{ds} k_a I_e W_p or .2 k_a I_e Wp min. =		For Connections not at Flexible Diaphragms:	**0.200 Wp**	**0.140 Wp**
Fp * 1.4 for steel elements per 12.11.2.2.2			**0.280 Wp**	**0.196 Wp**

The minimum wall anchorage load for concrete or masonry walls is 0.2* the wall weight or 5 psf per 1.4.4.

6. Horizontal Seismic Design Force on Nonstructural Architectural Components, Section 13.3:

	For Ip =1.0	For Ip=1.5	
Fp max = 1.6 Sds Ip Wp=	**0.169 Wp**	**0.263 Wp**	(Equation 13.3-2)
Fp min = 0.3 Sds Ip Wp=	**0.032 Wp**	**0.048 Wp**	(Equation 13.3-3)

The Seismic Design Force is based on Equation 13.3-1, with the minimum and maximum limits noted above.
Fp= **0.4** ap Sds Wp (1 + 2 z/h)/(Rp/Ip)

Seismic Design Force Summary on Architectural Components, Section 13.5:

	ap=	Rp=	Ip=	z/h=	Strength (1.0E)	ASD (0.7E)
1. Cantilevered (Unbraced) Parapets and Chimneys	**2.50**	**2.50**	**1.00**	**1.00**	**0.127 Wp**	**0.089 Wp** (Table 13.5-1)
2. Braced Interior Non-masonry walls and partitions						
Fp at floor=	**1.00**	**2.50**	**1.00**	**0.00**	**0.032 Wp**	**0.022 Wp** (Table 13.5-1)
Fp at roof=	**1.00**	**2.50**	**1.00**	**1.00**	**0.051 Wp**	**0.035 Wp** (Table 13.5-1)
Fp average at roof and floor:					**0.041 Wp**	**0.029 Wp**
3. Braced Interior Unreinforced masonry walls and partitions						
Fp at floor=	**1.00**	**1.50**	**1.00**	**0.00**	**0.032 Wp**	**0.022 Wp** (Table 13.5-1)
Fp at roof=	**1.00**	**1.50**	**1.00**	**1.00**	**0.084 Wp**	**0.059 Wp** (Table 13.5-1)
Fp average at roof and floor:					**0.058 Wp**	**0.041 Wp**
4. Cantilevered (Unbraced) Interior Nonstructural walls	**2.50**	**2.50**	**1.00**	**0.00**	**0.042 Wp**	**0.030 Wp** (Table 13.5-1)
5. Braced Parapets and Chimneys	**1.00**	**2.50**	**1.00**	**1.00**	**0.051 Wp**	**0.035 Wp** (Table 13.5-1)
6. Exterior Nonstructural Wall Elements						
Fp at floor=	**1.00**	**2.50**	**1.00**	**0.00**	**0.032 Wp**	**0.022 Wp** (Table 13.5-1)
Fp at roof=	**1.00**	**2.50**	**1.00**	**1.00**	**0.051 Wp**	**0.035 Wp** (Table 13.5-1)
Fp average at roof and floor:					**0.041 Wp**	**0.029 Wp**
For the Body of the Wall Panel Connection:						
Fp at floor=	**1.00**	**2.50**	**1.00**	**0.00**	**0.032 Wp**	**0.022 Wp** (Table 13.5-1)
Fp at roof=	**1.00**	**2.50**	**1.00**	**1.00**	**0.051 Wp**	**0.036 Wp** (Table 13.5-1)
For the fasteners of the connecting system:						
Fp at floor=	**1.25**	**1.00**	**1.00**	**0.00**	**0.053 Wp**	**0.037 Wp** (Table 13.5-1)
Fp at roof=	**1.25**	**1.00**	**1.00**	**1.00**	**0.158 Wp**	**0.111 Wp** (Table 13.5-1)
7. Appendages and Ornamentation	**2.50**	**2.50**	**1.00**	**1.00**	**0.127 Wp**	**0.089 Wp** (Table 13.5-1)

Notes:
1 Refer to Section 13.4.2 for additional requirements for anchors in concrete and masonry.
2 Section 1613.1 of 2018 IBC excludes the detailing requirements of Chapter 14 of ASCE 7.

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SEISMIC LOAD SUMMARY

2018 IBC (Ch: 16) and ASCE 7-16 (Ch: 11 to 13)

Table 11.4-1 and IBC 1613.2.3(1)
Site Coefficient, Fa

Site	Mapped Spectral Response Acceleration at Short Periods (Ss)						Distance
Class	Ss<=0.25	0.5	0.75	1	1.25	Ss>=1.5	Value
A	0.80	0.80	0.80	0.80	0.80	0.80	0.80
B	0.90	0.90	0.90	0.90	0.90	0.90	0.90
C	1.30	1.30	1.20	1.20	1.20	1.20	1.30
D	1.60	1.40	1.20	1.10	1.00	1.00	1.60
E	2.40	1.70	1.30	1.20	1.20	1.20	2.40
F	---	---	---	---	---	---	---

Minimum of 1.2 per Section 11.4.4 considered. Exceptions per Section 11.4.8 included.

Table 11.4-2 and IBC 1613.2.3(2)
Site Coefficient, Fv

Site	Mapped Spectral Response Acceleration at 1 Second Period (S1)						Distance
Class	S1<=0.1	0.2	0.3	0.4	0.5	S1>=0.6	Value
A	0.80	0.80	0.80	0.80	0.80	0.80	0.80
B	0.80	0.80	0.80	0.80	0.80	0.80	0.80
C	1.50	1.50	1.50	1.50	1.50	1.40	1.50
D	2.40	2.20	2.00	1.90	1.80	1.70	2.40
E	4.20	3.30	2.80	2.40	2.20	2.00	4.20
F	---	---	---	---	---	---	---

IBC Table 1613.2.5(1) and 11.6-1
ıismic Design Category based on Short Period Response Acceleratio

Value of	Occupancy Category			Design	
Sds	I or II	III	IV	Category	Category
Sds <= 0.167	A	A	A	A	A
0.167 <= Sds < 0.33	B	B	C	B	
0.33 <= Sds < 0.5	C	C	D	C	
0.5 <= Sds	D	D	D	D	
S1 >= 0.75	E	E	F	E	

Table 1613.2.5(2) and 11.6-2
Seismic Design Category Based on 1-Second Period Response Acceleration

Value of	Occupancy Category			Design	
Sd1	I or II	III	IV	Category	Category
Sd1 <= 0.067	A	A	A	A	B
0.067 <= Sd1 < 0.13ː	B	B	C	B	
0.133 <= Sd1 < 0.2	C	C	D	C	
0.2 <= Sd1	D	D	D	D	
S1 >= 0.75	E	E	F	E	

Bedding material should be graded to provide a continuous support beneath all points of the pipe and joints. Embedment material should be deposited and compacted uniformly and simultaneous on each side of the pipe to prevent lateral displacement. Compacted control fill material will be required for the full depth of the trench above the embedment material except in area landscape area with the compaction may be reduced to 90% Standard Proctor ASTM D 698. No backfill should be deposited or compacted in standing water.

Permanent slopes greater than 3 horizontals to 1 vertical should not be used unless additional testing and slope analysis is performed.

6.5 Drainage and Dewatering

Normal seasonal weather conditions should be anticipated and planned for during earthwork. It is recommended that the Contractor determine the actual groundwater levels at the site at the time of the construction activities to assess the impact groundwater may have on construction. Water should not be allowed to collect in the foundation excavation, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of collected rainwater, groundwater, or surface runoff. Positive site drainage should be provided to reduce infiltration of surface water around the perimeter of the building and beneath the floor slabs. The grades should be sloped away from the building and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and floor slab areas of the building.

The site should be graded such that positive drainage (normally 2% minimum) is provided away from any structures. Where sidewalks or paving do not immediately adjoin the building, protective slopes of at least 5% for a minimum of 10 feet from the perimeter walls are recommended. Roof drains and downpours should also be directed away from the building. Open-graded stone is not recommended for use under sidewalks unless the stone is adequately drained to prevent collection of water under the walks.

The site should also be graded to avoid water flows, concentrations, or pools behind retaining walls, curbs or similar structures. When swales are designed at the top of the walls, proper line and slope should be considered to avoid any flow down behind walls. Special attention is needed for sources of storm water from slopes, building roofs, gutter downspouts and paved areas draining to one point.

6.6 Landscaping

Landscaping and irrigation should be limited adjacent to buildings and pavements to reduce the potential for large moisture changes. Trees and large bushes can develop intricate root systems that can draw moisture from the subgrade, resulting in shrinkage of the bearing material during dry periods of the year. Desiccation of bearing material below foundations may result in foundation settlement.

Landscaped areas near pavements and sidewalks should include a drainage system that prevents over saturation of the subgrade beneath asphalt and concrete surfaces. Drainage systems in irrigation areas should be incorporated into the storm drain system.

7 Geotechnical Engineering Recommendations

7.1 Foundations Recommendations

Conventional spread and continuous wall footings are, generally, most economical when the existing soil conditions allow them to be founded at shallow depths on existing materials. Based on the materials encountered during this exploration, it is CFS Engineers' opinion that the planned structure can be supported by a shallow foundation system, such as spread and/or trench footings bearing in native clay and/or shale soils. Please reference the following table for recommended design parameters.

Table 5: Shallow Foundation Design Parameters

DESIGN PARAMETER	RECOMMENDED VALUE	COMMENTS
Allowable Bearing Capacity [1] (shallow foundations)	2,500 psf	Evaluated based on field and laboratory testing results [1].
Recommended Bearing Material [2]	FAT CLAY AND/OR SHALE	Suitable bearing material required beneath entirety of foundation system [2].
Anticipated Total Settlement	< 1-inch	Maximum
Anticipated Differential Settlement	< ¾ -inch	Maximum per 100 feet of linear footing
Minimum Recommended width	24 and 16 inches	Spread and trench, respectively
Minimum Recommended Depth	36-inches	Based on seasonal freeze-thaw cycles

(1) If over excavation of any footing is required to reach design bearing capacity, backfill of the footing should be done with lean concrete.
(2) A uniform bearing condition should exist beneath the entirety of the foundation system for a given structure. A representative of the Geotechnical Engineer should test the materials in the footing excavations to verify the material and design bearing pressure.

If over excavation of footings becomes necessary to achieve the desired bearing pressure or a uniform bearing condition, backfill of the footing should be done with lean concrete. Footings should be suitably reinforced to reduce the effects of differential movement that may occur due to variations in the properties of the supporting soils. Top and bottom reinforcing steel is recommended for continuous wall footings to reduce differential settlement due to possible varying bearing capacities of the existing fill soils.

Every effort should be made to keep the footing excavations dry as the soils will tend to soften when exposed to free water. Footing bottoms should be free of loose soil and concrete should be placed as soon as possible to prevent drying of the foundation soils.

7.2 Seismic Analysis

The determination of the seismic class is based on ASCE Standard 7: Minimum Design Loads for Building and Other Structures. Based upon this information, the seismic properties of the soil were interpolated from the standard penetration test values. A Seismic Site Class "D" was determined for this site. In addition, there is no significant risk of liquefaction or mass movement of the on-site soils due to a seismic event.

7.3 Slab on Grade Recommendations

In its current state, the overburden materials (i.e., Fat Clay) encountered during this exploration are unsuitable for direct support of the planned slab on grade. CFS recommends all concrete slabs on grade be supported by a minimum of 24-inches of Low Volume Change (LVC) material. LVC material should consist of lean clay (CL), KDOT AB3, crushed limestone screenings or equivalent. A low volume change material is defined as a material with a liquid limit less than 45 and a plasticity index less than 25. The subgrade can be constructed as outlined below.

1. Cut the subgrade to a minimum depth of 24-inches beneath the planned bottom of slab elevation. The exposed material at this depth should be moisture conditioned and re-compacted, as necessary, to pass a proof roll as specified in Section 6.1, "Site Preparation" of this report.
2. Twenty (20) inches of a compacted LVC material should be placed atop the exposed slab subgrade. The LVC should be placed in lifts no greater than 8-inches-thick (compacted thickness) and compacted to 95% of the maximum dry density as determined by ASTM 698. Limestone based LVC material should be compacted at a moisture content sufficient to achieve the desired compaction, and lean clay (CL) material should be compacted at a moisture content between 0 and +4% of optimum. Please note, if lean clay is utilized as LVC, CFS recommends it be capped with 6-inches of limestone based LVC to ease construction and protect the subgrade from excessive drying and wetting.
3. A 4-inch-thick layer of open graded stone (ASTM C33 or equivalent material) should be placed atop the 20-inches of compacted LVC material to return the subgrade to the original bottom of slab elevation. The open-graded stone will ease construction and provide a capillary break between the LVC and concrete slab.

Based on the materials encountered, 100 psi/in can be used as a modulus of subgrade reaction (k_s) for fat or lean clay soils. A subgrade reaction modulus value of 150 psi/in can be used for 20-inches of compacted granular fill such as KDOT AB3, MODOT Type 5 or equivalent.

Every floor slab should be evaluated to determine if a vapor retarder under the concrete floor is required. The slab designer should refer to ACI 302 and/or ACI 360 for procedures regarding the use and placement of a vapor retarder.

To reduce the effects of differential movement, slabs-on-grade should not be rigidly connected to columns, walls, or foundations unless it is designed to withstand the additional resultant forces. Floor slabs should not extend beneath exterior doors or over foundation grade beams, unless saw cut at the beam after construction. Expansion joints may be used to allow unrestrained vertical movement of the slabs. The floor slabs should be designed to have an adequate number of joints to reduce cracking resulting from differential movement and shrinkage. CFS suggests joints be provided on a minimum spacing of twelve (12) feet on center. For additional recommendations refer to the ACI Design Manual.

The requirements for the slab reinforcement should be established by the designer based on experience and the intended slab use.

7.4 Lateral Earth Pressures

Lateral earth pressures are determined by multiplying the vertical applied pressure by the appropriate lateral earth pressure coefficient. If the foundation walls are rigidly attached to the building and not free to rotate or deflect at the top, CFS recommends designing the walls for the *at-rest* earth pressure coefficient. Walls that are permitted to rotate and deflect at the top can be designed for the *active* lateral earth pressure condition. Horizontal loads acting on shallow foundations are resisted by friction along the foundation base and by *passive* pressure against the footing face that is perpendicular to the line of applied force.

It is recommended that all walls be backfilled with open graded stone (such as No. 57 as referenced in ASTM C33) extending to two (2) feet behind the wall for the entire height of the wall to within 12-inches of the surface to allow for proper drainage and relief of any hydrostatic pressure build-ups that may occur in the native clay. The use of stone to backfill behind the walls will expedite construction, reduce potential settlement between the wall and the floor slab and lower the pressure induced on the wall from the backfill thus potentially reducing the thickness of the walls.

Table 6: Earth Pressure and Friction Coefficients

MATERIAL	ACTIVE (K_a)	PASSIVE (K_p)	AT-REST (K_o)	ALLOWABLE BASE FRICTION	UNIT WEIGHT (pcf)
Open-graded crushed limestone	0.27	3.69	0.43	0.47	130-140
In-situ lean clay soils	0.40	2.5	0.68	0.32	120-125
In-situ fat clay soils	0.49	2.04	0.66	0.24	120-125
Lean clay – conditioned and compacted	0.32	3.12	0.48	0.35	120-125
Fat clay/Weathered Shale – conditioned and compacted	0.45	2.2	0.63	0.27	120-130
Limestone Bedrock	-	-	-	0.55	140-150

These earth pressure coefficients do not include the effect of surcharge loads, hydrostatic loading, or a sloping backfill. Nor do they incorporate a factor of safety. Also, these earth pressure coefficients do not account for high lateral pressures that may result from volume changes when expansive clay soils are used as backfill behind walls with unbalanced fill depths. In addition, any disturbed soils that are relied upon to provide some level of passive resistance should be placed in lifts not exceeding six (6) inches in thickness and compacted to a minimum density of 95% of the Standard Proctor (ASTM D698) maximum dry density at a moisture content within +- 3% of the optimum moisture content. It is recommended that a representative of CFS should verify the compaction of any such materials relied upon to provide passive pressure.

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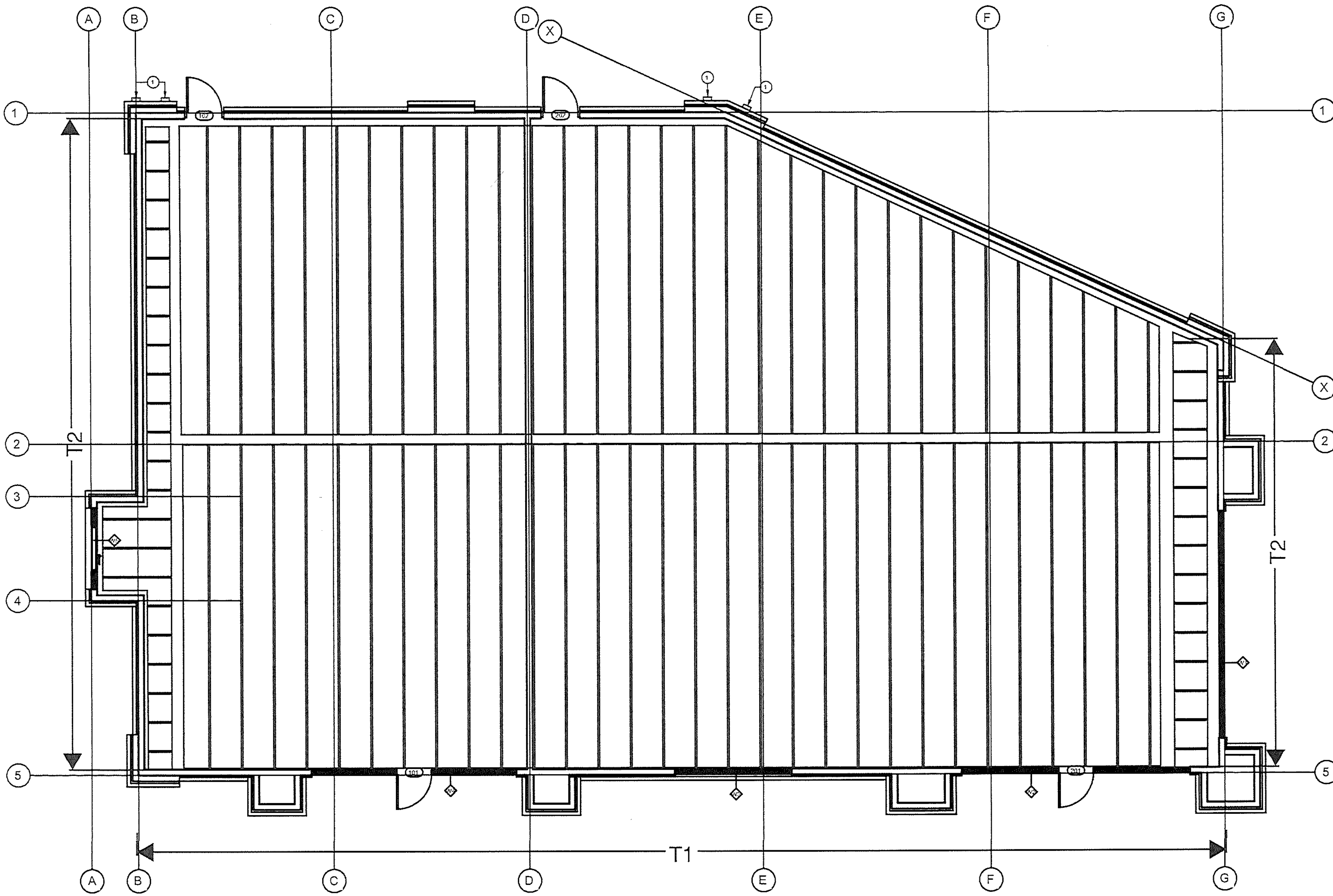
GRAVITY SYSTEM

wallace design collective, pc
structural · civil · landscape · survey
1741 mcgee street
kansas city, missouri 64108
816.421.8282 · 800.364.5858
wallace.design



T1 - 24" DEEP TRUSS, 28'-6", SPACED 2'-0" O.C.
T2 - 20" DEEP TRUSS, MAX 9'-0", SPACED 2'-0" O.C.

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Subject ROOF TRUSS

SPAN

- 28'-6"

(0.07)(28.5) = 2.00' = 24"

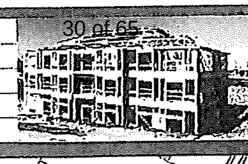
USE 24" DEPTH @ 24" OC

- TOTAL LOAD

- 20psf DEAD

- 20psf LIVE/SNOW

Roof Truss Span Tables

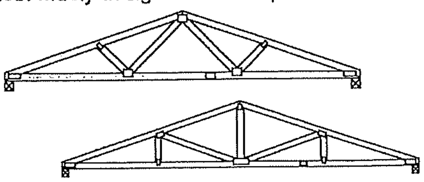


Alpine truss designs are engineered to meet specific span, configuration and load conditions. The shapes and spans shown here represent only a fraction of the millions of designs produced by Alpine Engineers.

Total load(PSF)	55	47	40	40
Duration factor	1.15	1.15	1.15	1.25
Live load(PSF)	40 snow	30 snow	20 snow	20 **
Roof type	shingle	shingle	shingle	shingle
	55 / 1.15 / 30 snow / tile			**construction or rain, not snow load

Top Chord	2x4	2x6	2x6	2x4	2x6	2x6	2x4	2x6	2x6	2x4	2x6	2x6
Bottom Chord	2x4	2x4	2x6	2x4	2x4	2x6	2x4	2x4	2x6	2x4	2x4	2x6

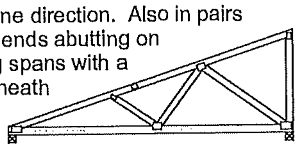
Common -- Truss configurations for the most widely designed roof shapes.



Spans in feet to out of bearing

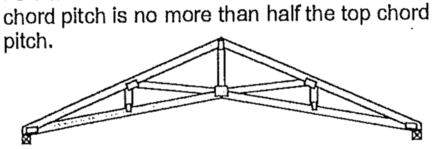
Pitch												
2/12	24	24	33	27	27	37	31	31	43	33	33	46
2.5/12	29	29	39	33	33	45	37	38	52	39	40	55
3/12	34	34	46	37	39	53	40	44	60	43	46	64
3.5/12	39	39	53	41	44	61	44	50	65	47	52	70
4/12	41	43	59	43	49	64	46	56	69	49	57	74
5/12	44	52	67*	46	58	69*	49	66	74*	53	66	80*
6/12	46	60*	69*	47	67*	71*	51	74*	76*	55	74*	82*
7/12	47	67*	70*	48*	72*	72*	52*	77*	77*	56*	80*	83*

Mono -- Used where the roof is required to slope only in one direction. Also in pairs with their high ends abutting on extremely long spans with a support underneath the high end.



2/12	24	24	33	25	27	38	27	31	41	29	32	44
2.5/12	28	29	40	29	32	43	31	37	46	33	37	49
3/12	30	33	45	31	37	47	34	42	50	36	42	54
3.5/12	33	37	49*	34	41	51*	36	46	54*	39	46	58*
4/12	35	41	52*	36	45*	54*	39	50*	58*	42*	49*	62*
5/12	38*	47*	57*	39*	51*	59*	42*	56*	63*	45*	54*	68*

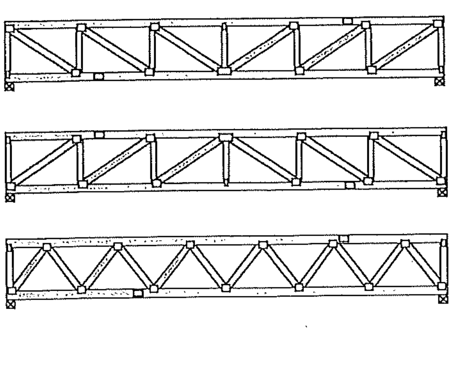
Scissors -- Provides a cathedral or vaulted ceiling. Most economical when the difference in slope between the top and bottom chords is at least 3/12 or the bottom chord pitch is no more than half the top chord pitch.



6/12 - 2/12 ‡	40	43	59*	42	49	62*	45	56*	66	48	57*	71*
6/12 - 2.5/12 ‡	37	38	52	38	44	57*	41	50	61*	44	52	66*
6/12 - 3/12 ‡	33	33	45	35	38	52	38	43	56*	40	46	60*
6/12 - 3.5/12 ‡	28	28	38	32	32	44	34	37	50	36	39	54
6/12 - 4/12 ‡	22	22	31	26	26	36	30	30	41	32	32	44

‡ Other pitch combinations available with these spans
For Example, a 5/12 - 2/12 combination has approx. the same allowable span as a 6/12 - 3/12

Flat -- The most economical flat truss for a roof is provided when the depth of the truss in inches is approximately equal to 7% of the span in inches.



Total load(PSF)	55	47	40	40
Duration factor	1.15	1.15	1.15	1.25
Live load(PSF)	40 snow	30 snow	20 snow	20 rain or constn.

Spans in feet to out of bearing

Depth												
Top Chord	2x4	2x6	2x6	2x4	2x6	2x6	2x4	2x6	2x6	2x4	2x6	2x6
Bottom Chord	2x4	2x4	2x6	2x4	2x4	2x6	2x4	2x4	2x6	2x4	2x4	2x6
16"	23	24	25 §	25 §	25 §	25 §	25 §	25 §	25 §	25 §	25 §	25 §
18"	25	27	28	27	27	29 §	29 §	29 §	29 §	29 §	29 §	29 §
20"	27	28	30	28	28	32	31	30	33 §	32	31	33 §
24"	29	30	33	31	31	35	34	33	38	35	34	40
28"	32	32	36	34	33	39	37	36	42	38	37	44
30"	33	33	38	35	35	40	38	37	44	40	39	45
32"	34	34	39	36	36	42	39	39	45	41	40	47
36"	36	36	42	39	38	45	42	41	48	43	43	50
42"	39	39	45	41	41	48	44	44	52	45	46	54
48"	40	42	49	43	44	52	46	47	56	46	49	58
60"	44	47	55	46	49	58	48	53	63	49	55	65
72"	45	51	60	48	54	64	51	57	68	51	59	69

§ = Span Limited by length to depth ratio of 24

NOTES: These overall spans are based on NDS '01 with 4" nominal bearing each end, 24" o.c. spacing, a live load deflection limited to L/240 maximum and use lumber properties as follows: 2x4 f_b =2000 psi f_t=1100 psi E=1.8x10^6 2x6 f_b=1750 psi f_t=950 psi f_c=1900 psi E=1.8x10^6. Allowable spans for 2x4 top chord trusses using sheathing other than plywood (e.g. spaced sheathing or 1x boards) may be reduced slightly. Trusses must be designed for any special loading such as concentrated loads from hanging partitions or air conditioning units, and snow loads caused by drifting near parapet or slide-off from higher roofs. To achieve maximum indicated spans, trusses may require six or more panels. Trusses with an asterisk (*) that exceed 14' in height may be shipped in two pieces. Contact your Alpine truss manufacturer for more information.

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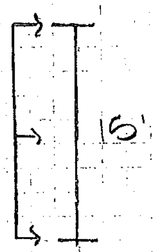
Subject WALL STUD CAPACITIES

MAX BENDING MOMENT



$f = 1925.6\ \text{psi}$

$S = \frac{(1.5'')(5.5'')^2}{6} = 7.56\ \text{in}^3$

$M = (1925.6\ \text{psi})(7.56\ \text{in}^3) = 14{,}557.5\ \text{lb·in}$

$M = 1.21\ \text{k·ft}$

$\frac{w(15)^2}{8} = 1.21\ \text{k·ft}$

$w = 0.043\ \text{klf}$

$= 43\ \text{plf}$

$\frac{43\ \text{plf}}{16''/12} = 32.25\ \text{psf}$

WALL STUDS 2x6

	LOAD CAPACITY
BENDING	32.25 psf
COMPRESSION PARALLEL	2.85 K
COMPRESSION PERPENDICULAR	6.14 K
SHEAR WIND	2.38 K
ELASTIC MODULUS	1,400 ksi

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Subject WALL STUD STRENGTH

Reference

BENDING F_b
COMPRESSION PERP $F_{c\perp}$
COMPRESSION PARALLEL F_c

BENDING F_b

C_D = 0.9(D), 1.0(L), 1.15(S), 1.6(W)
C_M = 1.0 (NOT EXCEEDING 19%)
C_t = 1.0 (T ≤ 100°F)
C_L = 1.0 (d ≥ b, 6" > 2", IN ACCORDANCE W/ 4.4.1)
C_F = 1.3 (d = 6", b = 2")
C_{fu} = 1.15 (d = 6", b = 2")
C_i = 1.0 (NO INCISING)
C_r = 1.15 (16" OC)
F_b = 700 psi (DOUGLAS FIR-LARCH, STUD)

WIND LOAD F'_b = (700 psi)(1.6)(1.0)(1.0)(1.0)(1.3)(1.15)(1.0)(1.15) = 1925.6 psi
LIVE F'_b = (700 psi)(1.3)(1.15) = 1046.5 psi

COMPRESSION PERPENDICULAR $F_{c\perp}$

C_M = 1.0 (NOT EXCEEDING 19%)
C_t = 1.0 (T ≤ 100°F)
C_i = 1.0 (NO INCISING)
C_b = 1.19 (2" BEARING)
F_b = 625 psi (DOUGLAS FIR-LARCH, STUD)
F'_b = (625 psi)(1.0)(1.0)(1.0)(1.19) = 743.8 psi

COMPRESSION PARALLEL F_c

C_D = 0.9 (D), 1.0(L), 1.15(S), 1.6(W)
C_M = 1.0 (NOT EXCEEDING 19%)
C_t = 1.0 (T ≤ 100°F)
C_F = 1.1 (d = 6", b = 2")
C_i = 1.0 (NO INCISING)
C_P = 0.34

wallace design collective

Date 3/11/22 Sheet No. of

Job 2220068

Subject WALL STUD STRENGTH

Reference

STRONG AXIS

$C_p =$ $\ell = 15'\text{-}0''$

$\ell_e = (1.0)(15'\text{-}0'') = 15'\text{-}0''$

$F_{ce} = \dfrac{0.822(510{,}000)}{(15'\text{-}0''/5.5'')^2} = 391.4$

$F_c = 850\,psi$

$$C_p = \frac{1+(391.4/850)}{2(0.8)} - \sqrt{\left[\frac{1+(391.4/850)}{2(0.8)}\right]^2 - \frac{391.4/850}{0.85}}$$

$C_p = 0.37$

$F_c = 850\,psi$

DEAD LOAD $F'_c = (850psi)(0.9)(1.0)(1.0)(1.1)(1.0)(0.37) = 311.4\ psi$

LIVE LOAD $F'_c = (850psi)(1.0)(1.0)(1.0)(1.1)(1.0)(0.37) = 346.0\ psi$

WEAK AXIS

$C_p =$ $\ell = 4'\text{-}0''$

$\ell_e = (1.0)(4'\text{-}0'') = 4'\text{-}0''$

$F_{ce} = \dfrac{0.822(510{,}000)}{(4'\text{-}0''/1.5'')^2} = 409.4\ psi$

$F_c = 850\,psi$

$$C_p = \frac{1+(409.4/850)}{2(0.8)} - \sqrt{\left[\frac{1+(409.4/850)}{2(0.8)}\right]^2 - \frac{409.4/850}{0.85}}$$

$C_p = 0.39$

$F_c = 850\,psi$

DEAD LOAD $F'_c = (850psi)(0.9)(1.0)(1.0)(1.1)(1.0)(0.39) = 328.2\ psi$

LIVE LOAD $F'_c = (850psi)(1.0)(1.0)(1.0)(1.1)(1.0)(0.39) = 364.7\ psi$

SHEAR F_v

$C_D = 0.9(D),\ 1.0(L),\ 1.15(S),\ 1.6(W)$

$C_M = 1.0$ (NOT EXCEEDING 19%)

$C_t = 1.0$ ($T \leq 100°F$)

$C_i = 1.0$ (NO INCISING)

$F_v = 180\,psi$

DEAD LOAD $F'_v = (0.9)(1.0)(1.0)(1.0)(180psi) = 162\,psi$

LIVE LOAD $F'_v = (1.0)(1.0)(1.0)(1.0)(180psi) = 180\,psi$

WIND LOAD $F'_v = (1.6)(1.0)(1.0)(1.0)(180psi) = 288\,psi$

wallace design collective

Date 3/11/22 Sheet No. of

Job 2220068

Subject WALL STUD STRENGTH

Reference

ELASTIC MODULUS E'

C_M = 1.0 (NOT EXCEEDING 19%)

C_t = 1.0 (T ≤ 100°F

C_i = 1.0 (NO INCISING)

E = 1,400,000 psi

E' = 1,400,000 psi

wallace design collective

Date 3/11/22 Sheet No. of

Job 2220068

Subject LOAD ON WALLS

DEAD LOAD

20psf

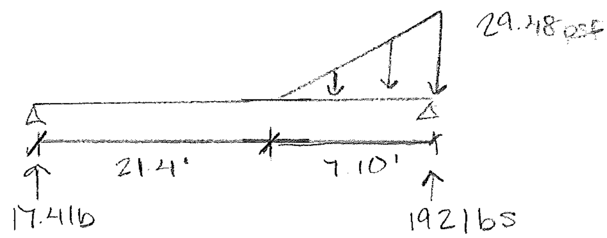
LIVE LOAD

20psf

SNOW LOAD

$P_{MIN} = 20psf$

$P_g = 14psf + DRIFT$



TA

(1/2 JOIST + OVERHANG) · 2

$\left(\frac{28.6'}{2} + 5'\right) 16''/12 = 25.67sf$

TOTAL LOAD ON WALL

$D+L = (20psf + 20psf)\frac{33.0'}{2}sf = 660\ lb = 0.66\ K$

$D+S = (20psf + 14psf)\frac{33.0}{2}sf + 192lb = 753\ lb = 0.75\ K$ ← CONTROLS

STUD CAPACITY

$\frac{P_n}{\Omega} = 2.85K$

$P_u = 0.75\ K$

$0.75\ K < 2.85K$ OK

wallace design collective

Date 3/31/22 Sheet No. of

Job 2220068

Subject LOAD ON WALLS

WIND LOAD (1.0)

CC ZONE 5 - 29.5psf (15')(16"/12) = 20psf

STUD CAPACITY

BENDING - 32.25psf

ASD LOADS

0.6D + 0.6W

1.0D + 0.6W

1.0D + 0.45W + 0.75S

UNITY CHECK

- 1.0D + 0.6W

$$\frac{(20psf)(33'/2)(16"/12)}{2.85K} + \frac{17.7psf}{32.25psf} = 0.15 + 0.55 = 0.7 < 1.0 \text{ OK}$$

- 1.0D + 0.45W + 0.75S

$$\frac{(20psf)(33'/2)(16"/12)}{2.85K} + \frac{0.45(29.5psf)}{32.25psf} + \frac{0.75((14psf)(33'/2)(16"/12) + 192lb)}{2.85K}$$

$$= 0.15 + 0.41 + 0.13 = 0.69 < 1.0 \text{ OK}$$

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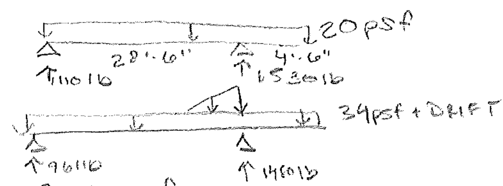
Date 3/28/22 Sheet No. of

Job 2220068

Subject 19'-6" HEADER - SOUTH WALL

LOADING



GRAVITY - 7624 lb/2' = 382 plf (S)

- [illegible]0/2' = [illegible]lf (S_D)

WALL GRAVITY - 12 psf · 5' = 60 plf

WIND C&C · 13.1 psf · 7.5' · 0.7 = 68.7 plf = 70 plf

DEFLECTION

TRY (3) 18" LVL

$I = \frac{3(1.5")(20")^3}{12} = 3000 in^4$

$\Delta = \frac{5 \cdot (442 plf/12)(19.5' \cdot 12)^4}{384(2 \cdot 10^6 psi)(2187 in^4)} = 0.329 in - L/710$

$L/500 = \frac{19.5 \cdot 12}{500} = 0.468 in$

LATERAL DEFLECTION

$I = \frac{(18")(4.5")^3}{12} = 136.68 in^4$

$\Delta = \frac{5 \cdot (70 plf/12)(19.5' \cdot 12)^4}{384(2 \cdot 10^6 psi)(182.25 in^4)} = 0.83 in$

$\Delta_{COL} \cdot 0.14 in$

$\Delta_{TOT} = (0.83 in + 0.14 in) = 0.97 in$

$L/240 + 0.25 in = \frac{19.5 \cdot 12}{240} + 0.25 in = 1.225 in > 0.97 in$ OK

STRENGTH

$V_x = \frac{(825 plf)(19.5')}{2} = 8044 lb$

$M_x = \frac{(825 plf)(19.5)^2}{8} = 39,214 lb \cdot ft$

$V_u = 3 \cdot 5130 lb = 15,390 lb$ - OK

$M_u = 3 \cdot 20,003 lb \cdot ft = 60,009 lb \cdot ft$ · OK

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Date 3/28/22 Sheet No. of

Job 2220068

Subject 10' HEADER

LOADING

BOD - 15'
TOW - 10'
HEADER FT = (1/2)(15' - 10') + (1/2)(10' - 3' - 8") = 5.86'
SF = (5.86')(10') = 58.6sf

WALL C&C = 13.8psf (0.6 ASD)

GRAVITY - 28.50'(40psf) = 1140plf (D+S NO DRIFT)
- 28.50'(34psf) = 969 plf + 195lb/2' = 1067 plf (D + S DRIFT)

WALL GRAVITY - 48psf · 5' = 240plf

TOTAL GRAVITY = 1140 + 240 = 1380plf

HEADER STRENGTH - GRAVITY

$V_x = \frac{(1380plf)(10')}{2} = 6900 lb$

$M_x = \frac{(1380plf)(10^2)}{8} = 17,250 lb\text{-}ft$

TRY (3) 9-1/2 x 1-1/2 LVL

$V_u = 3(2708 lb) = 8124 lb > 6900 lb$

$M_u = 3(5982 lb\text{-}ft) = 17946 lb\text{-}ft > 17,250 lb\text{-}ft$

DEFLECTION

$L/500 = \frac{10 \cdot 12}{500} = 0.24 in$

$I = \frac{(3 \cdot 1.5")(9.5)^3}{12} = 321.51 in^4$

$\Delta = \frac{5(1380plf/12)(10' \cdot 12)^4}{384(2 \cdot 10^6 psi)(321.51 in^4)} = 0.48 in$

TRY (3) 14" x 1-1/2"

$I = \frac{(3 \cdot 1.5")(14")^3}{12} = 1029 in^4$

$\Delta = \frac{5(1380plf/12)(10' \cdot 12)^4}{384(2 \cdot 10^6 psi)(1029 in^4)} = 0.15 in < 0.24 in$ OK

USE (3) 14" x 1-1/2" LVL

LATERAL DEFLECTION

$I = \frac{(14")(4.5")^3}{12} = 106.3125 in^4$

$\Delta = \frac{5(13.8psf \cdot 5.86'/12)(10' \cdot 12)^4}{384(2 \cdot 10^6 psi)(106.31 in^4)} = 0.08 in$ OK

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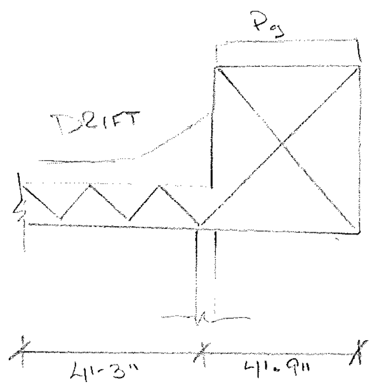
Date 3/23/22 Sheet No. of

Job 2220068

Subject EAST WALL HEADER GRAVITY

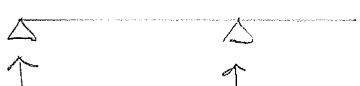
LOAD ON WALL

- JOISTS - 2'-0" OC
- SNOW - $P_g = 20psf$
- DRIFT - 52.8psf, $W_d = 9.35'$
- LIVE - 20psf
- DEAD - 20psf



BEAMBOY RESULTS

D+S



$\frac{434lb}{2'} = 217plf$ (ROOF)

WALL DEAD LOAD

- EXTERIOR STUD WALL W/ BRICK VENEER - 48psf

48psf · 5' = 240plf

TOTAL LOAD

(240plf + 217plf) ≈ 460plf

$M_u = 20,800 lb\text{-}ft$
$V_u = 4370 lb$

TRY (3) 20" x 1"2" LVL

M: 3·24,408 lb-ft > 20,800 lb-ft OK

V: 3·5700 lb > 4370 lb

DEFLECTION

$$\frac{5(460plf/12in)(19\cdot 12)^4}{384(2\cdot 10^6 psi)(3\cdot 1000 in^4)} = 0.23 in$$

$$\frac{L}{500} = \frac{19\cdot 12}{500} = 0.456in$$

0.23in < 0.456in OK

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Date 3/23/22 Sheet No. of

Job 22200068

Subject EAST WALL HEADER LATERAL

WALL LOAD

WIND C&C

- (1/2)(5') + (1/2)(10') = 7.5'
- TA = (19')(7.5') = 142.5sf
- 1.0W - 21.7psf
- 0.6W - 13.0psf
- (13psf)(7.5') = 97.5 plf

DEFLECTION

$$\frac{5(97.5\,plf/12")(19.5\cdot 12)^4}{384(2\cdot 10^6\,psi)(332.75\,in^4)}$$

$= 0.57in$

$I = \frac{(20)(5.5)^3}{12} = 0.57in$

$I_{req} = \frac{5wl^4}{384E\Delta_{MAX}}$

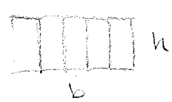
$\Delta_{MAX} = \Delta_{COL} + \Delta_{HEADER}$

$\Delta_{MAX} = \frac{19\cdot 12}{500} = 0.456$

Δ COL

5 STUDS @ 15'

$I = \frac{(5\cdot 1.5")(5.5")^3}{12} = 103.98in^4$



$V = \frac{(97.5plf)(19')}{2} = 926.25lb$

$$\Delta = \frac{(0.926K)(5\cdot 12)^2(10'\cdot 12)^2}{3(1400ksi)(103.98in^4)(15'\cdot 12)} = 0.61in$$

USE HSS 5x5x3/16 - 0.14in

Δ TOTAL

0.57in + 0.14in = 0.71in

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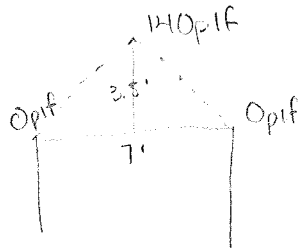
Date 3/3/22 Sheet No. of

Job 2220068

Subject DRIVE THROUGH HEADER

HEADER OVER OPENING

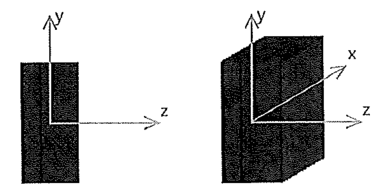


(40psf)(3.5') = 140plf

-SEE RISA OUTPUT FOR (3) 2x10 MEMBER · OK

Company	: <Licensed Company>	3/31/2022
Designer	: Seth Law	4:07:16 PM
Job Number	:	Checked By : ________
Model Name	:	

Detail Report: M1 Unity Check: 0.378 (shear) Load Combination: LC1: LOAD



Input Data:

Shape:	3-2X10 (nominal)	I Node:	N1
Member Type:	Beam	J Node:	N2
Length (ft):	7	I Release:	Fixed
Material Type:	Wood	J Release:	Fixed
Design Rule:	Typical	I Offset (in):	N/A
Number of Internal Sections:	97	J Offset (in):	N/A

Material Properties:

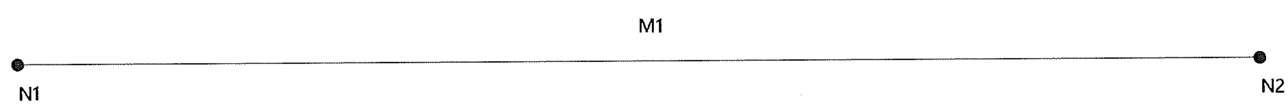
Material:	DF	Grade:	No.1	Nu:	0.3
Type:	Solid Sawn	Cm:	No	Therm. Coeff. ($1e^{5}$°F^{-1}):	0.3
Database:	Visually Graded	Emod:	1	Density (k/ft^3):	0.035
Species:	Douglas Fir-Larch				

Shape Properties:

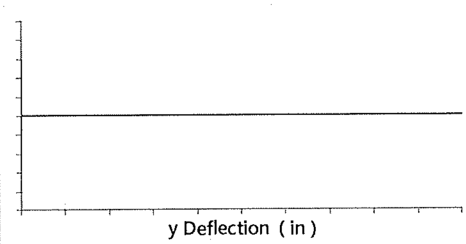
F_b (ksi):	1	E (ksi):	1700	b (actual) (in):	4.5
F_t (ksi):	0.675	E mod:	1	d (actual) (in):	9.25
F_v (ksi):	0.18	COV_E (Table F1):	0.25	# of Plies:	3
F_c (ksi):	1.5	E_{min} (ksi):	621.025	K_f:	0.6

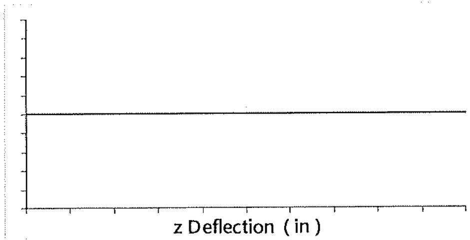
Design Properties:

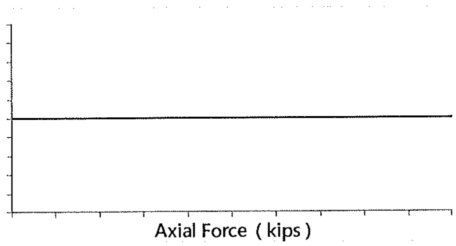
le2 (ft) :	N/A	C_D:	1	Max Defl Ratio:	L/10000
le1 (ft) :	N/A	R_B:	6.194	Max Defl Location:	0
le-bend top (ft) :	N/A	C_L:	0.997	Span:	N/A
le-bend bot (ft) :	N/A	C_r:	1		
K_{y-y}:	1	C_{fu}:	1.2		
K_{z-z}:	1	C_P:	0.41		
y sway:	No	K_f:	0.6		
z sway:	No				

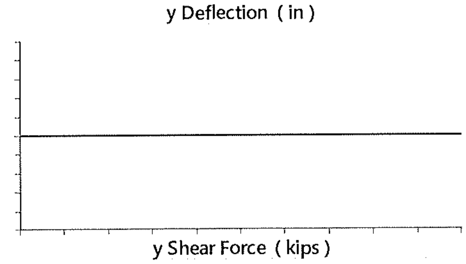


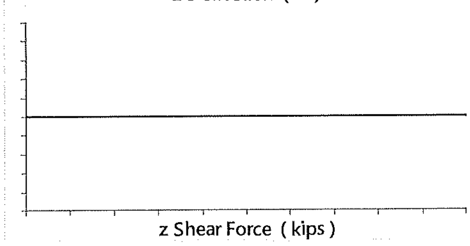
Diagrams:

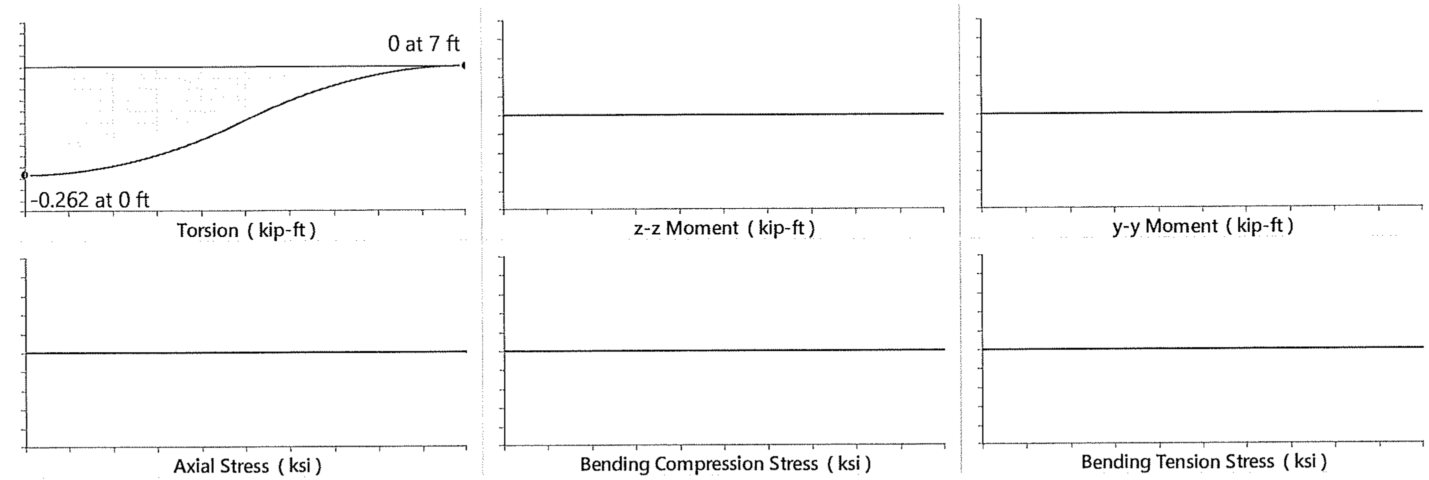












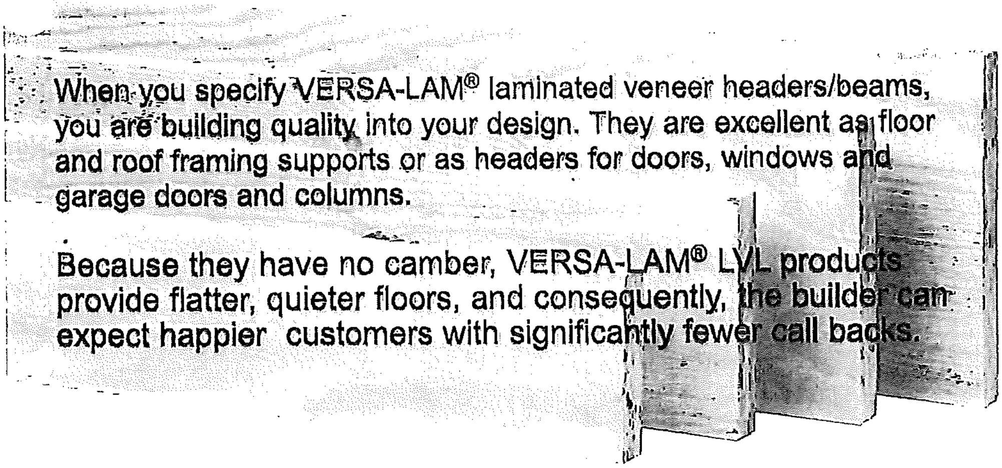
AWC NDS-18: ASD Code Check

Limit State	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-
Applied Loading - Shear + Torsion	-	-	-	-
Axial Compression Analysis	0.000 ksi	0.614 ksi	-	-
Axial Tension Analysis	0.000 ksi	0.743 ksi	-	-
Flexural Analysis, Fb1'	0.000 ksi	1.097 ksi	-	-
Flexural Analysis, Fb2'	0.000 ksi	1.32 ksi	-	-
Bending & Axial Compression Analysis	-	-	0.000	Pass
Bending & Axial Tension Analysis	-	-	0.000	Pass
Shear Analysis	0.068 ksi	0.18 ksi	0.378	Pass

2

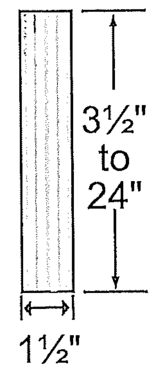
VERSA-LAM® Products

An Introduction to VERSA-LAM® Products



When you specify VERSA-LAM® laminated veneer headers/beams, you are building quality into your design. They are excellent as floor and roof framing supports or as headers for doors, windows and garage doors and columns.

Because they have no camber, VERSA-LAM® LVL products provide flatter, quieter floors, and consequently, the builder can expect happier customers with significantly fewer call backs.



1½" VERSA-LAM® 2.0 3100 Design Values

Design Property		
Grade		2.0 3100
Modulus of Elasticity	E(x 10^6 psi)(1)	2.0
Bending	F_b (psi)(2)(3)	3100
Horizontal Shear	F_v (psi)(2)(4)	285
Tension Parallel to Grain	F_t (psi)(2)(5)	2150
Compression Parallel to Grain	$F_{c\parallel}$ (psi)(2)	3000
Compression Perpendicular to Grain	$F_c\perp$ (psi)(1)(6)	750
Equivalent Specific Gravity for Fastener Design	(SG)	0.5

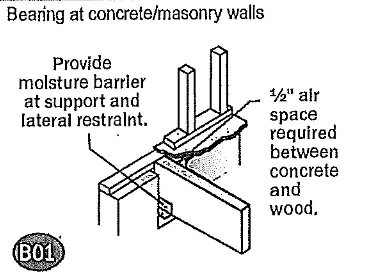
1. This value cannot be adjusted for load duration.
2. This value is based upon a load duration of 100% and may be adjusted for other load durations.
3. Fiber stress bending value shall be multiplied by the depth factor, $(12/d)^{1/9}$ where d = member depth [in].
4. Stress applied perpendicular to the gluelines.
5. Tension value shall be multiplied by a length factor, $(4/L)^{1/8}$ where L = member length [ft]. Use L = 4 for members less than four feet long.
6. Stress applied parallel to the gluelines.

* Design properties are limited to dry conditions of use where the maximum moisture content of the material will not exceed 16%.

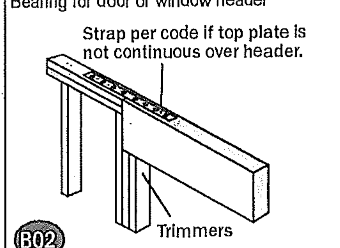
Width [in]	Depth [in]	Weight [lb/ft]	Allowable Shear [lb]	Allowable Moment [ft-lb]	Moment of Inertia [in⁴]
1½	3½	1.4	998	907	5.4
	5½	2.2	1568	2131	20.8
	7¼	2.9	2066	3590	47.6
	9¼	3.8	2636	5688	98.9
	9½	3.9	2708	5982	107.2
	11¼	4.6	3206	8233	178.0
	11⅞	4.8	3384	9118	209.3
	14	5.7	3990	12443	343.0
	16	6.5	4560	16013	512.0
	18	7.3	5130	20003	729.0
	20	8.1	5700	24408	1000.0
	24	9.8	6840	34443	1728.0

VERSA-LAM® Beam Details

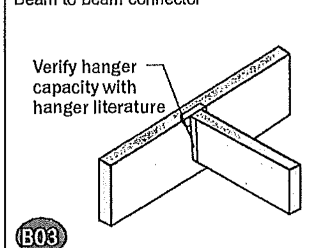
Bearing at concrete/masonry walls



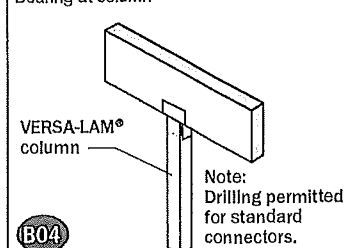
Bearing for door or window header



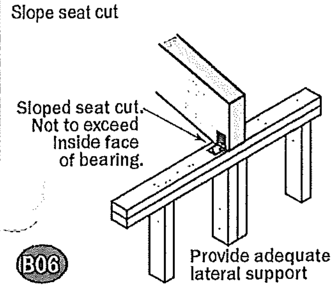
Beam to beam connector



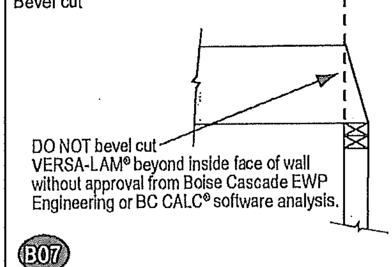
Bearing at column



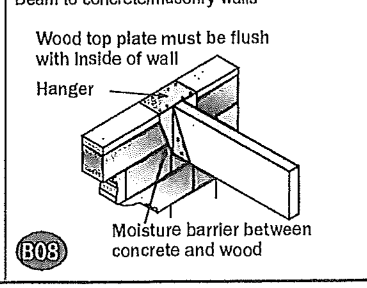
Slope seat cut



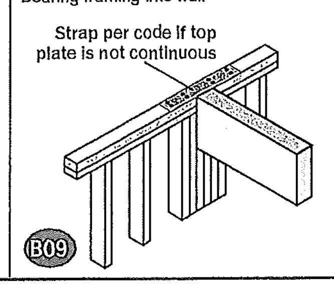
Bevel cut



Beam to concrete/masonry walls



Bearing framing into wall



VERSA-LAM® Installation Notes

- Minimum of ½" air space between beam and wall pocket or adequate barrier must be provided between beam and concrete/masonry.
- Adequate bearing shall be provided. If not shown on plans, please refer to load tables in your region's Specifier Guide.
- VERSA-LAM® beams are intended for interior applications only and should be kept as dry as possible during construction.
- Continuous lateral support of top of beam shall be provided (side or top bearing framing).

wallace design collective

Date 3/24/22 Sheet No. of

Job 2220068

Subject COL HEADER DEFLECTION

COL VS PSL

WIND LOAD - 13.0psf (ASD), 97.5plf, 926lb @ 10' ON COL

1/4" STEEL COL

$$\Delta = \frac{(0.926K)(5 \cdot 12)^2 (10' \cdot 12)^2}{3(29000 KSI)(16.0 in^4)(15' \cdot 12)} = 0.19 in$$

3/8" STEEL COL

$$\Delta = \frac{(0.926K)(5 \cdot 12)^2 (10' \cdot 12)^2}{(3)(29000 KSI)(21.7 in^4)(15' \cdot 12)} = 0.14 in$$

1/2" STEEL COL

$$\Delta = \frac{(0.926K)(5 \cdot 12)^2 (10' \cdot 12)^2}{(3)(29,000 KSI)(26.0 in^4)(15' \cdot 12)} = 0.12 in$$

5 1/4" x 9 1/4" PSL

$$I = \frac{(9.25)(5.25)^3}{12} = 111.54 in^4$$

$$\Delta = \frac{(0.926K)(5 \cdot 12)^2 (10 \cdot 12)^2}{(3)(2000 KSI)(111.54 in^4)(15' \cdot 12)} = 0.40 in$$

5 1/4" x 14" PSL

$$I = \frac{(14")(5.25")^3}{12} = 168.82 in^4$$

$$\Delta = \frac{(0.926K)(5 \cdot 12)^2 (10 \cdot 12)^2}{(3)(2000 KSI)(168.82 in^4)(15 \cdot 12)} = 0.26 in$$

5 1/4" x 18" PSL

$$I = \frac{(18")(5.25")^3}{12} = 217.05 in^4$$

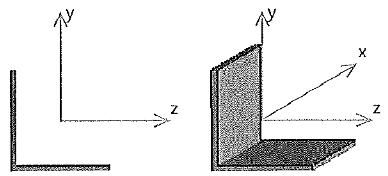
$$\Delta = \frac{(0.926K)(5 \cdot 12)^2 (10 \cdot 12)^2}{(3)(2000 KSI)(217.05 in^4)(15 \cdot 12)} = 0.20 in$$

RISA
A NEMETSCHEK COMPANY

Company : <Licensed Company>
Designer : Seth Law
Job Number :
Model Name :

3/31/2022
2:20:46 PM
Checked By : __________

etail Report: M1 | Unity Check: 0.788 (axial/bending) | Load Combination: LC 1: LOAD



Input Data:

Shape:	L5X5X5	I Node:	N1
Member Type:	Beam	J Node:	N2
Length (ft):	7	I Release:	Fixed
Material Type:	Hot Rolled Steel	J Release:	Fixed
Design Rule:	Typical	I Offset (in):	N/A
Number of Internal Sections:	97	J Offset (in):	N/A

Material Properties:

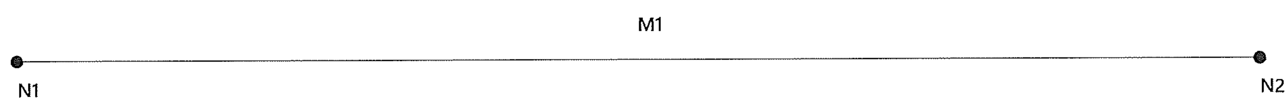
Material:	A36 Gr.36	Therm. Coeff. ($1e^{5}°F^{-1}$):	0.65	R_y:	1.5
E (ksi):	29000	Density (k/ft^3):	0.49	F_u (ksi):	58
G (ksi):	11154	F_y (ksi):	36	R_t:	1.2
Nu:	0.3				

Shape Properties:

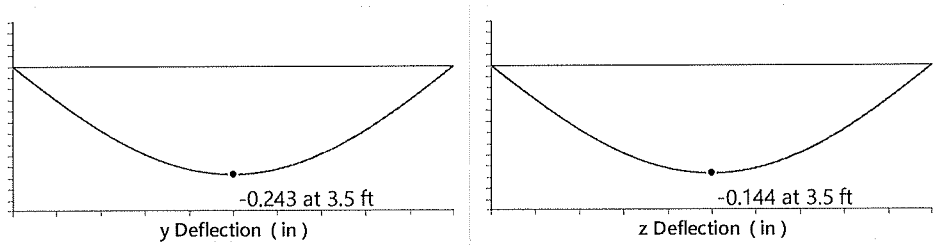
d (in):	5	Area (in^2):	3.07	J (in^4):	0.108
b_f (in):	5	I_{yy} (in^4):	7.44	r_z (in):	0.99
t (in):	0.313	I_{zz} (in^4):	7.44	k* (in):	0.813

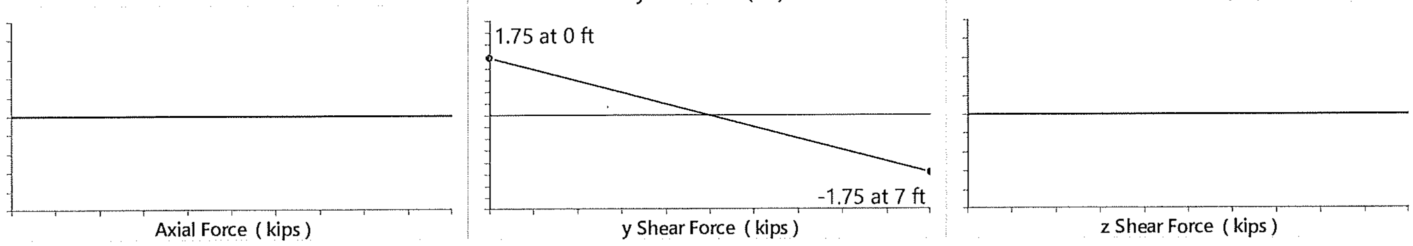
Design Properties:

$L_{b\ y\text{-}y}$ (ft):	N/A	$K_{y\text{-}y}$:	1	Max Defl Ratio:	L/346
$L_{b\ z\text{-}z}$ (ft):	N/A	$K_{z\text{-}z}$:	1	Max Defl Location:	3.5
$L_{comp\ top}$ (ft):	N/A	y sway:	No	Span:	1
$L_{comp\ bot}$ (ft):	N/A	z sway:	No		
L_{torque} (ft):	N/A	Function:	Lateral		
		Seismic DR:	None		



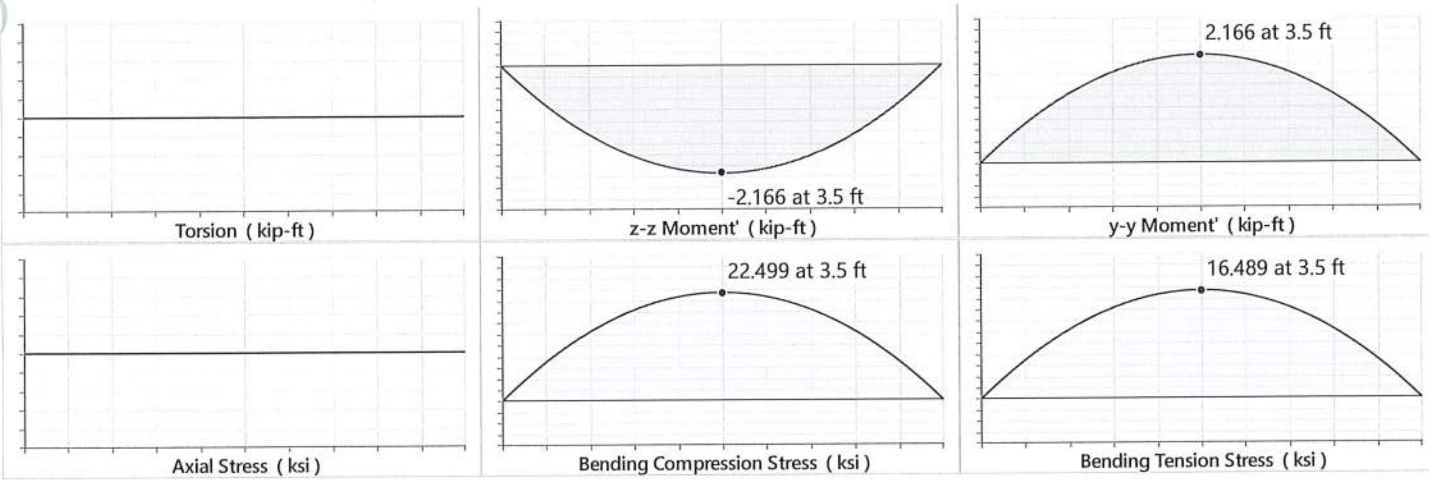
Diagrams:





Company : <Licensed Company>
Designer : Seth Law
Job Number :
Model Name :

3/31/2022
2:20:47 PM
Checked By : __________



AISC 15th (360-16): ASD Code Check

Limit State	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial				
Applied Loading - Shear + Torsion	-	-	-	-
Axial Tension Analysis	0.000 k	66.18 k	-	-
Axial Compression Analysis	0.000 k	44.065 k	-	-
Flexural Analysis (Strong Axis)	2.166 k-ft	7.794 k-ft	-	-
Flexural Analysis (Weak Axis)	2.166 k-ft	4.247 k-ft	-	-
Shear Analysis (Major Axis y)	1.75 k	20.242 k	0.086	Pass
Shear Analysis (Minor Axis z)	0.000 k	20.242 k	0.000	Pass
Bending & Axial Interaction Check (UC Bending Max)	-	-	0.788	Pass

$0.788 < 1.0$
ANGLE IS SUFFICIENT

LATERAL SYSTEM

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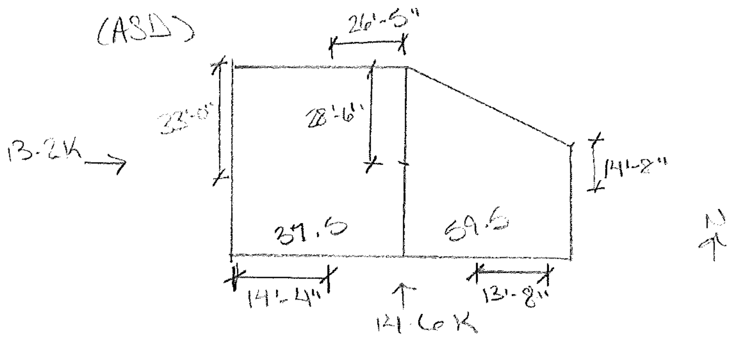
Date 3/11/22 Sheet No. of

Job 2220068

Subject DIAPHRAGM / SHEAR WALL LOADS

(ASD)



SHEAR IN S WALL

$\frac{6.6K}{28.0} \cdot 1000 = 236 \text{ plf}$

SHEAR IN N WALL

$\frac{6.6K}{26.42} \cdot 1000 = 250 \text{ plf}$

SHEAR IN E WALL

$\frac{(29.75/97)(14.6K)}{14.67'} = 305 \text{ plf}$

SHEAR IN CENTRAL WALL

$\frac{7.3K}{28.67'}$ 256 plf

SHEAR IN W WALL

$\frac{(18.75/97)(14.6K)}{33'} = 86 \text{ plf}$

wallace design collective

Date 3/11/22 Sheet No. of

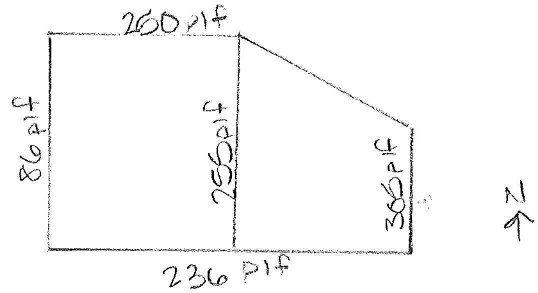
Job 2220068

Subject DIAPHRAGM SELECTION

EAST SIDE

CASE 1 - (305plf)(2) - 610plf

CASE 3 - (250plf)(2) - 500plf



USE 8d 1-3/8 w/ 15/32 PANEL THICKNESS 2" NAILED FACE

CASE 1 - $\frac{670 plf}{2}$ = 335 plf > 305 plf OK

CASE 3 - $\frac{505 plf}{2}$ = 252.5 plf > 250 plf OK

WEST SIDE

CASE 1 - (255 plf)(2) - 510 plf

CASE 3 - (250 plf)(2) - 500plf

SAME SHEAR WALL DESIGNATION AS ABOVE

CASE 1 - $\frac{607 plf}{2}$ = 335 plf > 255 plf OK

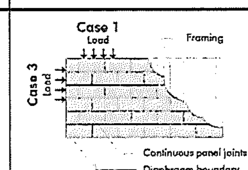
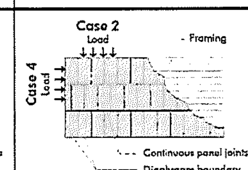
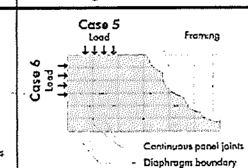
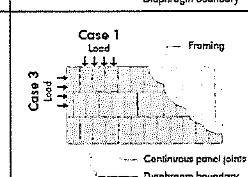
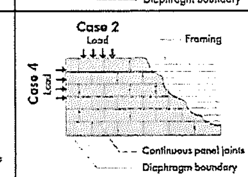
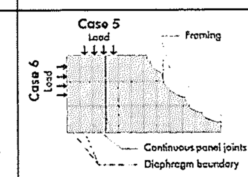
CASE 2 - $\frac{505 plf}{2}$ = 252.5plf > 250 plf OK

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

Unblocked Wood Structural Panel Diaphragms[1,2,3,4,5]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A SEISMIC – 6 in. Nail Spacing at diaphragm boundaries and supported panel edges – Case 1 – v_s (plf)	Case 1 – G_a (kips/in.) OSB	Case 1 – G_a (kips/in.) PLY	Cases 2,3,4,5,6 – v_s (plf)	Cases 2,3,4,5,6 – G_a (kips/in.) OSB	Cases 2,3,4,5,6 – G_a (kips/in.) PLY	B WIND – 6 in. Nail Spacing at diaphragm boundaries and supported panel edges – Case 1 – v_w (plf)	Cases 2,3,4,5,6 – v_w (plf)
Structural I	6d	1-1/4	5/16	2	330	9.0	7.0	250	6.0	4.5	460	350
				3	370	7.0	6.0	280	4.5	4.0	520	390
	8d	1-3/8	3/8	2	480	8.5	7.0	360	6.0	4.5	670	505
				3	530	7.5	6.0	400	5.0	4.0	740	560
	10d	1-1/2	15/32	2	570	14	10	430	9.5	7.0	800	600
				3	640	12	9.0	480	8.0	6.0	895	670
Sheathing and Single-Floor	6d	1-1/4	5/16	2	300	9.0	6.5	220	6.0	4.0	420	310
				3	340	7.0	5.5	250	5.0	3.5	475	350
			3/8	2	330	7.5	5.5	250	5.0	4.0	460	350
				3	370	6.0	4.5	280	4.0	3.0	520	390
	8d	1-3/8	3/8	2	430	9.0	6.5	320	6.0	4.5	600	450
				3	480	7.5	5.5	360	5.0	3.5	670	505
			7/16	2	460	8.5	6.0	340	5.5	4.0	645	475
				3	510	7.0	5.5	380	4.5	3.5	715	530
			15/32	2	480	7.5	5.5	360	5.0	4.0	670	505
				3	530	6.5	5.0	400	4.0	3.5	740	560
	10d	1-1/2	15/32	2	510	15	9.0	380	10	6.0	715	530
				3	580	12	8.0	430	8.0	5.5	810	600
			19/32	2	570	13	8.5	430	8.5	5.5	800	600
				3	640	10	7.5	480	7.0	5.0	895	670

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

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

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

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Date 3/11/22 Sheet No. of

Job 2220062

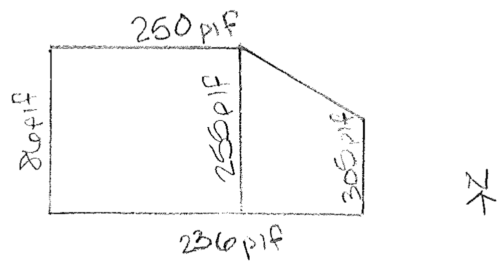
Subject SHEAR WALL SELECTION

SOUTH WALL

$w = 225\,plf \cdot 2 = 450\,plf$

6" SPACE 5/16, 1-1/4 6d - $\frac{w_u}{\Omega} = \frac{505}{2}\,plf = 253\,plf$

$253\,plf > 225\,plf$ OK



NORTH, WEST, WALLS

MAX $w_u = 250\,plf$

SAME DESIGNATION AS ABOVE

$253\,plf > 250\,plf$ OK

EAST WALL, CENTRAL

MAX $w_u = 305\,plf \cdot 2 = 610\,plf$

4" SPACE 5/16, 1-1/4, 6d - $\frac{w_u}{\Omega} = \frac{755}{2}\,plf = 377.5\,plf > 305\,plf$

Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1,3,6,7]

Wood-based Panels[4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimun Fastener Penetration in Framing Member or Blocking (in.)	Fastener Type & Size	A SEISMIC – Panel Edge Fastener Spacing (in.) – 6 – v_s (plf)	6 – G_a (kips/in.) OSB	6 – G_a (kips/in.) PLY	4 – v_s (plf)	4 – G_a (kips/in.) OSB	4 – G_a (kips/in.) PLY	3 – v_s (plf)	3 – G_a (kips/in.) OSB	3 – G_a (kips/in.) PLY	2 – v_s (plf)	2 – G_a (kips/in.) OSB	2 – G_a (kips/in.) PLY	B WIND – Panel Edge Fastener Spacing (in.) – 6 – v_w (plf)	4 – v_w (plf)	3 – v_w (plf)	2 – v_w (plf)
Wood Structural Panels - Structural I[4,5]			Nail (common or galvanized box)																
	5/16	1-1/4	6d	400	13	10	600	18	13	780	23	16	1020	35	22	560	840	1090	1430
	3/8[2]	1-3/8	8d	460	19	14	720	24	17	920	30	20	1220	43	24	645	1010	1290	1710
	7/16[2]			510	16	13	790	21	16	1010	27	19	1340	40	24	715	1105	1415	1875
	15/32			560	14	11	860	18	14	1100	24	17	1460	37	23	785	1205	1540	2045
	15/32	1-1/2	10d	680	22	16	1020	29	20	1330	36	22	1740	51	28	950	1430	1860	2435
Wood Structural Panels – Sheathing[4,5]	5/16	1-1/4	6d	360	13	9.5	540	18	12	700	24	14	900	37	18	505	755	980	1260
	3/8			400	11	8.5	600	15	11	780	20	13	1020	32	17	560	840	1090	1430
	3/8[2]	1-3/8	8d	440	17	12	640	25	15	820	31	17	1060	45	20	615	895	1150	1485
	7/16[2]			480	15	11	700	22	14	900	28	17	1170	42	21	670	980	1260	1640
	15/32			520	13	10	760	19	13	980	25	15	1280	39	20	730	1065	1370	1790
	15/32	1-1/2	10d	620	22	14	920	30	17	1200	37	19	1540	52	23	870	1290	1680	2155
	19/32			680	19	13	1020	26	16	1330	33	18	1740	48	22	950	1430	1860	2435
Plywood Siding			Nail (galvanized casing)																
	5/16	1-1/4	6d	280	13		420	16		550	17		720	21		390	590	770	1010
	3/8	1-3/8	8d	320	16		480	18		620	20		820	22		450	670	870	1150
Particleboard Sheathing - (M-S "Exterior Glue" and M-2 "Exterior Glue")			Nail (common or galvanized box)																
	3/8		6d	240	15		360	17		460	19		600	22		335	505	645	840
	3/8		8d	260	18		380	20		480	21		630	23		365	530	670	880
	1/2			280	18		420	20		540	22		700	24		390	590	755	980
	1/2		10d	370	21		550	23		720	24		920	25		520	770	1010	1290
	5/8			400	21		610	23		790	24		1040	26		560	855	1105	1455
Structural Fiberboard Sheathing	1/2		Nail (galvanized roofing) 11 ga. galv. roofing nail (0.120" x 1-1/2" long x 7/16" head)				340	4.0		460	5.0		520	5.5			475	645	730
	25/32		11 ga. galv. roofing nail (0.120" x 1-3/4" long x 3/8" head)				340	4.0		460	5.0		520	5.5			475	645	730

1. Nominal unit shear capacities sball be adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.3.6. For specific requirements, see 4.3.7.1 for wood structural panel shear walls, 4.3.7.2 for particleboard sbear walls, and 4.3.7.3 for fiberboard shear walls. See Appendix A for common and box nail dimensions.
2. Shears are permitted to be increased to values shown for 15/32 inch (nominal) sheathing with same nailing provided (a) studs are spaced a maximum of 16 incbes on center, or (b) panels are applied with long dimension across studs.
3. For species and grades of framing other than Douglas-Fir-Larcb or Southern Pine, reduced nominal unit sbear capacities sball be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1-(0.5-G)], where G = Specific Gravity of the framing lumber from the NDS (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
4. Apparent shear stiffness values G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for shear walls constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values sball be permitted to be multiplied by 1.2.
5. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
6. Where panels are applied on both faces of a shear wall and nail spacing is less than 6" on center on either side, panel joints sball be offset to fall on different framing members. Alternatively, the width of the nailed face of framing members shall be 3" nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.
7. Galvanized nails shall be hot-dipped or tumbled.

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Date 3/28/22 Sheet No. of

Job 2220068

Subject TOP PLATES

EAST WALL

V=305plf (WORST CASE)

(305plf)(24"/12) = 610lb (BLOCKING TRUSS SPACING)

10d NAILS ϕ=0.148
-CAPACITY -118lbs TABLE 12N

Z' =(118lbs)(1.6) = 189lbs

$\frac{610lb}{189lb}$ = 3.22 NAILS ≈ 4 NAILS

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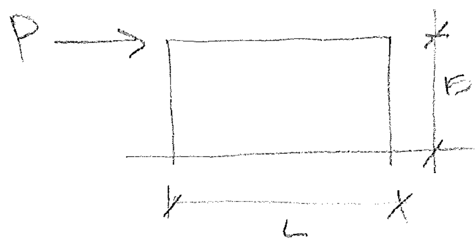
Date 3/28/22 Sheet No. of

Job 2220068

Subject SILL PLATE HOLDOWN

CENTRAL WALL

$\frac{(7.3K)(15')}{28.5'} = 3842 \text{ lb } T/C$



$\frac{15' \cdot P}{L} = T/C$

EAST WALL

$\frac{(4.47K)(15')}{14.67'} = 4575 \text{ lb } T/C$

HDU5-SDS2.5 - 5,645 lb - OK

NORTH WALL

$\frac{(6.791K)(15')}{27.2'} = 3745 \text{ lb } T/C$

HDU4-SDS2.5 - 4,565 lb OK

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FOUNDATIONS

wallace design collective, pc
structural · civil · landscape · survey
1741 mcgee street
kansas city, missouri 64108
816.421.8282 · 800.364.5858
wallace.design

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Date 2/2/22 Sheet No. of

Job 2120625

Subject GRADE BEAM

b=18", W=30" TRY 2(#5)

$A_s = 0.31 in^2 \cdot 2 = 0.62 in^2$

d=27"

$\phi M_n = F_y A_s (d - a/2)$

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{(0.62 in^2)(60 ksi)}{0.85(4.5ksi)(18")}$

a = 0.54"

$\phi M_n = (60ksi)(0.62in^2)(27" - 0.54"/2)(0.9)$

$\phi M_n = (0.9)\frac{(994.4 k\cdot in)}{12}$

$\phi M_n = 74.6 k\cdot ft$

LOAD - 10' SPAN

ROOF TA - 14'

1.2(20psf) + 1.6(20psf) = 56psf

(56psf)(14') = 784 plf

WALL TA - 15.5'

1.2(48psf) = 58psf

(58psf)(15.5') = 900plf

784plf + 900plf = 1684 plf

$M_u = \frac{(1684 plf)(10^2)}{8}$

M_u = 21,050 lb ft = 21.1 k·ft

(2) #5 GOOD!

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Date 2/11/22 Sheet No. of

Job 2220068

Subject COLUMN FOUNDATION

LOADS

D - 20psf
S - 20psf

(40psf)(39.5'/2)(57.0'/2) = 22.52 K

FOOTING SIZE

2000psf BEARING

$\frac{22.52K}{2KSF} = 11.26SF$

$\sqrt{11.26SF} = 3.35' \approx 4'-0"$

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Date 3/11/22 Sheet No. of

Job 2200068

Subject SHEAR WALL FOUNDATIONS

NORTH WALL

$\frac{(6.6K)(13.0')}{26.42'} = 3.25K$

$\frac{3.25K}{2ksf} = 1.62sf$

$\sqrt{1.62sf} = 1.27' \approx 3'-0''$ MIN

EAST WALL

$\frac{(305plf)(14.67')(13.0')}{14.67'} = 4.0K$

$\frac{4.0K}{2KSF} = 2sf$

$\sqrt{2sf} = 1.41' \approx 3'-0''$ MIN

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Subject HEADER FOUNDATIONS

GRAVITY

- 1530lb/2 = 765plf
- 12psf · 5' = 60plf
- 765plf + 60plf = 825plf

$V_u = \frac{(825plf)(19.5')}{2} = 8.0K$

FOUNDATION SIZE

$\frac{8.0K}{2ksf} = 4.02sf$

$\sqrt{4.02sf} = 2.00$ - USE 3' x 3' FTG

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MISCELLANEOUS

wallace design collective, pc
structural · civil · landscape · survey
1741 mcgee street
kansas city, missouri 64108
816.421.8282 · 800.364.5858
wallace.design

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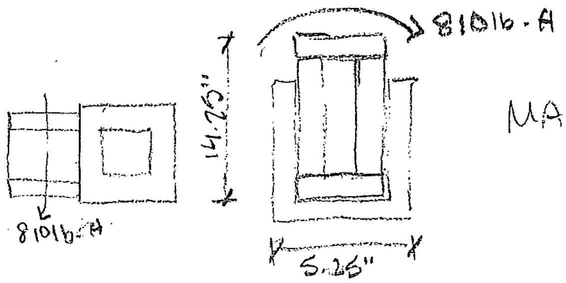
Subject HEADER SEAT STRENGTH



MAX PLATE THICKNESS = 0.75"

$\frac{6" - 3(1.5")}{2} = 0.75"$

$M = 135 \text{lb-ft/ft} \cdot 12' = 1620 \text{lb-ft}$ TOTAL

$\frac{1620 \text{lb-ft}}{2} = 810 \text{ lb-ft / SIDE} = 9.72 \text{ k-in / SIDE} \rightarrow 4.86 \text{ k-in / PLATE}$

$T/C = \frac{810 \text{lb-ft}}{(11.25"/12)} = 864 \text{lbs} \qquad \frac{864 \text{lbs}}{2} = 432 \text{lb / PLATE}$

$Z = \frac{bd^2}{4}$ TRY 3/8" PLATE

$= \frac{(3/8)(4")^2}{4} = 1.5 \text{in}^2$

$\frac{M_y}{\Omega} = \frac{F_y \cdot Z}{1.67} = \frac{(36 \text{ksi})(1.5 \text{in}^2)}{1.67} = 32.3 \text{ k-in / PLATE}$

32.3 k-in / PLATE > 4.86 k-in / PLATE OK

WELD

$\frac{432 \text{lb}}{1000} = 0.928(3)(l)$

$l = 0.16"$

USE 15" PLATE & WELD

BACK PLATE

$V = 432 \text{lb} \cdot 2 = 864 \text{lb}$ (T & B)

$864 \text{lb} \cdot 2 = 1728 \text{lb}$

$0.6(36 \text{ksi})\frac{(3/8" \cdot 15")}{1.67} = 72.4 \text{K}$ (OK)

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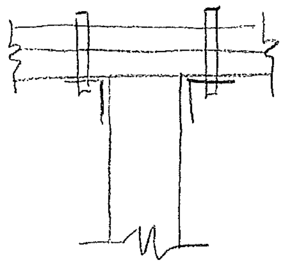
Subject TOP OF COLUMN CONNECTION



LOADS OUT OF PLANE ON COLUMN

WIND C&C - 132 plf
D+S TORSION - 136 lb/ft

$V = \frac{(132\ plf)(12')}{2} = 792\ lb/\ BOLT$

AWC CONNECTION CALCULATOR
- SINGLE SHEAR
- (2) 2x6 = 3" THICK
- (1) L6x4x1/4

- $C_D = 1.6$ (WIND)
- $C_M = 1.0$
- $C_t = 1.0$

$\phi = 3/8"$ - ASD = 841 lbs

Design Method	Allowable Stress Design (ASD)
Connection Type	Lateral loading
Fastener Type	Bolt
Loading Scenario	Single Shear - Wood Main Member

Main Member Type	Glulam Douglas Fir-Larch
Main Member Thickness	3 in.
Main Member: Angle of Load to Grain	0
Side Member Type	Steel
Side Member Thickness	1/4 in.
Side Member: Angle of Load to Grain	0
Fastener Diameter	3/8 in.
Load Duration Factor	C_D = 1.6
Wet Service Factor	C_M = 1.0
Temperature Factor	C_t = 1.0

Connection Yield Modes

Im	2520 lbs.
Is	3262 lbs.
II	1185 lbs.
IIIm	1370 lbs.
IIIs	841 lbs.
IV	883 lbs.

Adjusted ASD Capacity	**841 lbs.**

- Bolt bending yield strength of 45,000 psi is assumed.
- The Adjusted ASD Capacity is only applicable for bolts with adequate end distance, edge distance and spacing per NDS chapter 11.
- ASTM A36 Steel is assumed for steel side members 1/4 in. thick, and ASTM A653 Grade 33 Steel is assumed for steel side members less than 1/4 in. thick.

The Connection Calculator was designed and created by Cameron Knudson, Michael Dodson and David Pollock at Washington State University. Support for development of the Connection Calculator was provided by American Wood Council.

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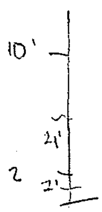
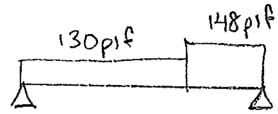
Subject SILL PLATE CONNECTION

OUT OF PLANE LOADS



5' · 12' = 60'
25.9 psf (1.0W) - 9'
29.5 psf (1.0W) · 3'



RESULTS

SEE BEAMBOY REPORT
M_u = 2.38 k·ft
V_u = 827 lb AT END

2x6 CAPACITY

(1) 2x6 - 32.25 psf > 29.5 psf OK
- 6.14 K > 0.83 K OK

(2) STUDS IS SUFFICIENT

CONNECTION

- SIMPSON A34 W/ (8) 8d NAILS BY 1½" - T & B · 545 lb · 2 = 1090 lb > 827 lb OK
- POWDERED ACTUATED FASTENERS - HILTI XU
720 lb / FASTENER (2)