PRCTI20230247

City of Puyallup Development & Permitting Services ISSUED PERMIT
Building | Planning | Engineering | Public Works | Fire | Traffic

BSE

Brienen **S**tructural **E**ngineers, P.S.

City of Puyallup Building ACCEPTED
JMontgomery
04/06/2023
9:50:39 AM

Enterprise TI
733 River Road
Puyallup, WA 98371

Tenant Improvement
Structural Calculations

THE APPROVED CONSTRUCTION PLANS, DOCUMENTS AND ALL ENGINEERING MUST BE POSTED ON THE JOB AT ALL INSPECTIONS IN A VISIBLE AND READILY ACCESSIBLE LOCATION.

FULL SIZED LEDGIBLE COLOR PLANS ARE REQUIRED TO BE PROVIDED BY THE PERMITEE ON SITE FOR INSPECTION

PAUL J. BRIENEN
STATE OF WASHINGTON
32114
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

Project Number 22460
2/17/2023

Phone: (206) 397-0000   1316 Central Avenue South, Suite 200, Kent, WA 98032   www.bse-ps.com

BSE

Brienen **S**tructural **E**ngineers, P.S.

Calculation Index

PRCTI20230247

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

BUILDING CODE

The 2018 edition of the 'international existing building code (IEBC), as adopted or amended by the city of Puyallup, shall govern design and construction.

SCOPE OF WORK

Design of the wall door cutouts on the west and north sides of the existing building. The scope of this package is not a seismic upgrade of the entire building, our is limited to replacing the seismic strength lost by wall cutouts. A full seismic upgrade is beyond the scope of this tenant improvement. The west wall requires additional seismic capacity and a concrete shearwall was added. The north wall of the building is approximately 190-feet long and did not require additional seismic capacity. Additional out of plane wall seismic bracing was added along the west wall to accommodate the additional out of plane loads at the location of the new concrete shear wall. New cutouts were framed with structural steel to support roof framing and coil doors. The 190-foot east-west length of the building spans numerous tenants and the tenant improvement associated with this submittal covers the western 50-feet of this building. We did not observe the interior framing of the eastern 140-feet of this building, it is assumed to be in similar condition to the western 50-foot of the building that we did observe.

WIND LOADS

⚠ This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

ⓘ The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Address: 733 River Rd, Puyallup, WA 98371, USA

Coordinates: 47.2020095, -122.3033086

Elevation: 37 ft

Timestamp: 2022-12-28T17:01:07.916Z

Hazard Type: Wind

ASCE 7-16

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

ASCE 7-10

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

ASCE 7-05

ASCE 7-05 Wind Speed	85 mph

97MPH RISK CAT II

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.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

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.

While the information presented on this website is believed to be correct, ATC and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in the report should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. ATC does not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the report provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the report.

N-S WIND LOADING

(MAIN FORCE RESISTING SYSTEM)

$P_{w_{ULT}}$ = 16psf PER WIND ANALYSIS OUTPUT.
MIN CONTROLLED

$V_{w_{ULT}} = 16psf \left(\frac{16'}{2} \times 190'\right) = 24.3^k \ll V_{SEISMIC_{ULT}} = 205^k$

SEISMIC CONTROLS —

BY INSPECTION SEISMIC WILL CONTROL E-W DIR AS WELL.

MecaWind v2405

Calculations Prepared by:
Client: 0
Date: Jan 19, 2023
Designer: 0

Calculations Prepared For:
Project #: 0
Location: 0

File Location:
G:\2022\22460 Enterprise Puyallup TI\Calcs\Enterprise Puyallup TI Wind Design.wnd

Basic Wind Parameters

Wind Load Standard	= ASCE 7-16	Exposure Category	= B
Wind Design Speed	= 97.0 mph	Risk Category	= II
Structure Type	= Building	Building Type	= Enclosed

General Wind Settings

Incl_LF	= Include ASD Load Factor of 0.6 in Pressures	= False
DynType	= Dynamic Type of Structure	= Rigid
Zg	= Altitude (Ground Elevation) above Sea Level	= 39.000 ft
Bdist	= Base Elevation of Structure	= 0.000 ft
SDB	= Simple Diaphragm Building	= True
Reacs	= Show the Base Reactions in the output	= False
MWFRSType	= MWFRS Method Selected	= Ch 27 Pt 1

Topographic Factor per Fig 26.8-1

Topo	= Topographic Feature	= None
Kzt	= Topographic Factor	= 1.000

Building Inputs

RoofType:	Building Roof Type	= Flat	RfHt	: Roof Height	= 16.500 ft
W	: Building Width	= 100.000 ft	L	: Building Length	= 380.000 ft
Par	: Is there a Parapet	= False			

Exposure Constants per Table 26.11-1:

Alpha:	Table 26.11-1 Const	= 7.000	Zg:	Table 26.11-1 Const	= 1200.000 ft
At:	Table 26.11-1 Const	= 0.143	Bt:	Table 26.11-1 Const	= 0.840
Am:	Table 26.11-1 Const	= 0.250	Bm:	Table 26.11-1 Const	= 0.450
C:	Table 26.11-1 Const	= 0.300	Eps:	Table 26.11-1 Const	= 0.333

Overhang Inputs:

Std	= Overhangs on all sides are the same	= True
OHType	= Type of Roof Wall Intersections	= None

Main Wind Force Resisting System (MWFRS) Calculations per Ch 27 Part 1:

h	= Mean Roof Height above grade	= 16.500 ft
Kh	= 15 ft [4.572 m]< Z <Zg -->(2.01*(Z/zg)^(2/Alpha) {Table 26.10-1}	= 0.591
Kzt	= Topographic Factor is 1 since no Topographic feature specified	= 1.000
Kd	= Wind Directionality Factor per Table 26.6-1	= 0.85
Zg	= Elevation above Sea Level	= 39.000 ft
Ke	= Ground Elevation Factor: Ke = e^-(0.0000362*Zg) {Table 26.9-1}	= 0.999
GCPi	= Ref Table 26.13-1 for Enclosed Building	= +/-0.18
RA	= Roof Area	= 38000.00 sq ft
LF	= Load Factor based upon STRENGTH Design	= 1.00
qh	= (0.00256 * Kh * Kzt * Kd * Ke * V^2) * LF	= 12.07 psf
qin	= For Negative Internal Pressure of Enclosed Building use qh*LF	= 12.07 psf
qip	= For Positive Internal Pressure of Enclosed Building use qh*LF	= 12.07 psf

Gust Factor Calculation:

Gust Factor Category I Rigid Structures - Simplified Method

G1	= For Rigid Structures (Nat. Freq.>1 Hz) use 0.85	= 0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Zm	= Max(0.6 * Ht, Zmin)	= 30.000 ft
Izm	= Cc * (33 / Zm) ^ 0.167	= 0.305
Lzm	= L * (Zm / 33) ^ Eps	= 309.993
B	= Structure Width Normal to Wind	= 380.000 ft
Q	= (1 / (1 + 0.63 * ((B + Ht) / Lzm)^0.63))^0.5	= 0.759
G2	= 0.925*((1+0.7*Izm*3.4*Q)/(1+0.7*3.4*Izm))	= 0.783

Gust Factor Used in Analysis

G	= Lessor Of G1 Or G2	= 0.783

MWFRS Wind Normal to Ridge (Ref Fig 27.3-1)

h	= Mean Roof Height Of Building	= 16.500 ft
RHt	= Ridge Height Of Roof	= 16.500 ft
B	= Horizontal Dimension Of Building Normal To Wind Direction	= 380.000 ft
L	= Horizontal Dimension Of building Parallel To Wind Direction	= 100.000 ft
L/B	= Ratio Of L/B used For Cp determination	= 0.263
h/L	= Ratio Of h/L used For Cp determination	= 0.165
Slope	= Slope of Roof	= 0.0 Deg
Roof	= Roof Coeff (0 to h/2) (0.000 ft to 8.250 ft)	= -0.18, -0.9
Roof	= Roof Coeff (h/2 to h) (8.250 ft to 16.500 ft)	= -0.18, -0.9
Roof	= Roof Coeff (h to 2h) (16.500 ft to 33.000 ft)	= -0.18, -0.5
Roof	= Roof Coeff (>2h) (>33.000 ft)	= -0.18, -0.3
Cp_WW	= Windward Wall Coefficient (All L/B Values)	= 0.80
Cp_LW	= Leward Wall Coefficient using L/B	= -0.50
Cp_SW	= Side Wall Coefficient (All L/B values)	= -0.70
GCpn_WW	= Parapet Combined Net Pressure Coefficient (Windward Parapet)	= 1.50
GCpn_LW	= Parapet Combined Net Pressure Coefficient (Leeward Parapet)	= -1.00

Wall Wind Pressures based On Positive Internal Pressure (+GCPi) - Normal to Ridge
All wind pressures include a load factor of 1.0

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
16.50	0.591	1.000	12.07	0.18	5.39	-6.90	-8.79	12.29	16.00

Wall Wind Pressures based on Negative Internal Pressure (-GCPi) - Normal to Ridge
All wind pressures include a load factor of 1.0

MFRS

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
16.50	0.591	1.000	12.07	-0.18	9.74	-2.55	-4.44	12.29	16.00

Notes Wall Pressures:
Kz = Velocity Press Exp Coeff   Kzt = Topographical Factor
qz = 0.00256*Kz*Kzt*Kd*V^2   GCPi = Internal Press Coefficient
Side = qh * G * Cp_SW - qip * +GCPi   Windward = qz * G * Cp_WW - qip * +GCPi
Leeward = qh * G * Cp_LW - qip * +GCPi   Total = Windward Press - Leeward Press
* Minimum Pressure: Para 27.1.5 no less than 16.00 psf (Incl LF) applied to Walls
+ Pressures Acting TOWARD Surface   - Pressures Acting AWAY from Surface

Roof Wind Pressures for Positive & Negative Internal Pressure (+/- GCPi) - Normal to Ridge
All wind pressures include a load factor of 1.0

Roof Var	Start Dist ft	End Dist ft	Cp_min	Cp_max	GCPi	Pressure Pn_min* psf	Pressure Pp_min* psf	Pressure Pn_max psf	Pressure Pp_max psf
Roof (All)	0.000	8.250	-0.180	-0.900	0.180	0.47	-3.87	-6.33	-10.68
Roof (All)	8.250	16.500	-0.180	-0.900	0.180	0.47	-3.87	-6.33	-10.68
Roof (All)	16.500	33.000	-0.180	-0.500	0.180	0.47	-3.87	-2.55	-6.90
Roof (All)	33.000	100.000	-0.180	-0.300	0.180	0.47	-3.87	-0.66	-5.01

Notes Roof Pressures:
Start Dist = Start Dist from Windward Edge   End Dist = End Dist from Windward Edge
Cp_Max = Largest Coefficient Magnitude   Cp_Min = Smallest Coefficient Magnitude
Pp_max = qh*G*Cp_max - qip*(+GCPi)   Pn_max = qh*G*Cp_max - qin*(-GCpi)
Pp_min* = qh*G*Cp_min - qip*(+GCPi)   Pn_min* = qh*G*Cp_min - qin*(-GCPi)
OH = Overhang   X = Dir along Ridge   Y = Dir Perpendcular to Ridge   Z = Vertical
* The smaller uplift pressures due to Cp_Min can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7
+ Pressures Acting TOWARD Surface   - Pressures Acting AWAY from Surface

MWFRS Wind Parallel to Ridge (Ref Fig 27.3-1)

h = Mean Roof Height Of Building = 16.500 ft

RHt	= Ridge Height Of Roof	= 16.500 ft
B	= Horizontal Dimension Of Building Normal To Wind Direction	= 100.000 ft
L	= Horizontal Dimension Of building Parallel To Wind Direction	= 380.000 ft
L/B	= Ratio Of L/B used For Cp determination	= 3.800
h/L	= Ratio Of h/L used For Cp determination	= 0.043
Slope	= Slope of Roof	= 0.0 Deg
Roof	= Roof Coeff (0 to h/2) (0.000 ft to 8.250 ft)	= -0.18, -0.9
Roof	= Roof Coeff (h/2 to h) (8.250 ft to 16.500 ft)	= -0.18, -0.9
Roof	= Roof Coeff (h to 2h) (16.500 ft to 33.000 ft)	= -0.18, -0.5
Roof	= Roof Coeff (>2h) (>33.000 ft)	= -0.18, -0.3

Cp_WW	= Windward Wall Coefficient (All L/B Values)	= 0.80
Cp_LW	= Leward Wall Coefficient using L/B	= -0.21
Cp_SW	= Side Wall Coefficient (All L/B values)	= -0.70
GCpn_WW	= Parapet Combined Net Pressure Coefficient (Windward Parapet)	= 1.50
GCpn_LW	= Parapet Combined Net Pressure Coefficient (Leeward Parapet)	= -1.00

Wall Wind Pressures based On Positive Internal Pressure (+GCPi) - Parallel to Ridge
All wind pressures include a load factor of 1.0

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
16.50	0.591	1.000	12.07	0.18	5.39	-4.16	-8.79	9.55	16.00

MFRS

Wall Wind Pressures based on Negative Internal Pressure (-GCPi) - Parallel to Ridge
All wind pressures include a load factor of 1.0

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
16.50	0.591	1.000	12.07	-0.18	9.74	0.19	-4.44	9.55	16.00

Notes Wall Pressures:
Kz = Velocity Press Exp Coeff Kzt = Topographical Factor
qz = 0.00256*Kz*Kzt*Kd*V^2 GCPi = Internal Press Coefficient
Side = qh * G * Cp_SW - qip * +GCPi Windward = qz * G * Cp_WW - qip * +GCPi
Leeward = qh * G * Cp_LW - qip * +GCPi Total = Windward Press - Leeward Press
* Minimum Pressure: Para 27.1.5 no less than 16.00 psf (Incl LF) applied to Walls
+ Pressures Acting TOWARD Surface - Pressures Acting AWAY from Surface

Roof Wind Pressures for Positive & Negative Internal Pressure (+/- GCPi) - Parallel to Ridge
All wind pressures include a load factor of 1.0

Roof Var	Start Dist ft	End Dist ft	Cp_min	Cp_max	GCPi	Pressure Pn_min* psf	Pressure Pp_min* psf	Pressure Pn_max psf	Pressure Pp_max psf
Roof (All)	0.000	8.250	-0.180	-0.900	0.180	0.47	-3.87	-6.33	-10.68
Roof (All)	8.250	16.500	-0.180	-0.900	0.180	0.47	-3.87	-6.33	-10.68
Roof (All)	16.500	33.000	-0.180	-0.500	0.180	0.47	-3.87	-2.55	-6.90
Roof (All)	33.000	380.000	-0.180	-0.300	0.180	0.47	-3.87	-0.66	-5.01

Notes Roof Pressures:
Start Dist = Start Dist from Windward Edge End Dist = End Dist from Windward Edge
Cp_Max = Largest Coefficient Magnitude Cp_Min = Smallest Coefficient Magnitude
Pp_max = qh*G*Cp_max - qip*(+GCPi) Pn_max = qh*G*Cp_max - qin*(-GCpi)
Pp_min* = qh*G*Cp_min - qip*(+GCPi) Pn_min* = qh*G*Cp_min - qin*(-GCpi)
OH = Overhang X = Dir along Ridge Y = Dir Perpendcular to Ridge Z = Vertical
* The smaller uplift pressures due to Cp_Min can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7
+ Pressures Acting TOWARD Surface - Pressures Acting AWAY from Surface

Components and Cladding (C&C) Zone Summary per Ch 30 Pt 1:

h/W	= Ratio of mean roof height to building width	= 0.165
h/L	= Ratio of mean roof height to building length	= 0.043
h	= Mean Roof Height above grade	= 16.500 ft
Kh	= 15 ft [4.572 m]< Z <Zg -->(2.01*(Z/zg)^(2/Alpha) {Table 26.10-1}	= 0.591

PRCTI20230247

City of Puyallup Development & Permitting Services ISSUED PERMIT
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```
Kzt     = Topographic Factor is 1 since no Topographic feature specified    = 1.000
Kd      = Wind Directionality Factor per Table 26.6-1                       = 0.85
GCPi    = Ref Table 26.13-1 for Enclosed Building                           = +/-0.18
LF      = Load Factor based upon STRENGTH Design                            = 1.00
qh      = (0.00256 * Kh * Kzt * Kd * Ke * V^2) * LF                         = 12.07 psf
LHD     = Least Horizontal Dimension:  Min(B, L)                            = 100.000 ft
a1      = Min(0.1 * LHD,  0.4 * h)                                          = 6.600 ft
a       = Max(a1, 0.04 * LHD, 3 ft [0.9 m])                                 = 6.600 ft
h/B     = Ratio of mean roof height to least hor dim: h / B                 = 0.165
0.2*h   = Parameter used to define Zone 3                                   = 3.300 ft
0.6*h   = Parameter used to define Zones 1 and 2                            = 9.900 ft
```

Wind Pressure Summary for C&C Zones based Upon Areas Ch 30 Pt 1 (Table 1 of 2)
All wind pressures include a load factor of 1.0

Zone	Figure	A <= 10.00 sq ft psf	A = 20.00 sq ft psf	A = 50.00 sq ft psf	A = 100.00 sq ft psf
1	30.3-2A	16.00 -22.70	16.00 -21.20	16.00 -19.22	16.00 -17.73
1'	30.3-2A	16.00 -16.00	16.00 -16.00	16.00 -16.00	16.00 -16.00
2	30.3-2A	16.00 -29.94	16.00 -28.02	16.00 -25.47	16.00 -23.55
3	30.3-2A	16.00 -40.81	16.00 -36.96	16.00 -31.87	16.00 -28.02
4	30.3-1	16.00 -16.00	16.00 -16.00	16.00 -16.00	16.00 -16.00
5	30.3-1	16.00 -17.39	16.00 -16.23	16.00 -16.00	16.00 -16.00

Wind Pressure Summary for C&C Zones based Upon Areas Ch 30 Pt 1 (Table 2 of 2)
All wind pressures include a load factor of 1.0

Zone	Figure	A = 200.00 sq ft psf	A = 500.00 sq ft psf	A > 1000.00 sq ft psf
1	30.3-2A	16.00 -16.23	16.00 -16.00	16.00 -16.00
1'	30.3-2A	16.00 -16.00	16.00 -16.00	16.00 -16.00
2	30.3-2A	16.00 -21.62	16.00 -19.08	16.00 -19.08
3	30.3-2A	16.00 -24.17	16.00 -19.08	16.00 -19.08
4	30.3-1	16.00 -16.00	16.00 -16.00	16.00 -16.00
5	30.3-1	16.00 -16.00	16.00 -16.00	16.00 -16.00

* A is effective wind area for C&C: Span Length * Effective Width
* Effective width need not be less than 1/3 of the span length
* Maximum and minimum values of pressure shown.
* + Pressures acting toward surface, - Pressures acting away from surface
* Per Para 30.2.2 the Minimum Pressure for C&C is 16.00 psf [0.766 kPa] {Includes LF}
* Interpolation can be used for values of A that are between those values shown.

COMPONENTS & CLADDING —

WALL WIND LOAD ULTIMATE IS:

ZONE 4 ±16psf

ZONE 5 +16psf -18psf

SEISMIC ANALYSIS

PRCTI20230247

This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Address:	733 River Rd, Puyallup, WA 98371, USA
Coordinates:	47.2020095, -122.3033086
Elevation:	37 ft
Timestamp:	2022-12-28T16:59:56.174Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default

DEFAULT SITE CLASS

Basic Parameters

Name	Value	Description
S_S	1.279	MCE_R ground motion (period=0.2s)
S_1	0.44	MCE_R ground motion (period=1.0s)
S_{MS}	1.535	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.023	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

$S_{DS} = 1.023$

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.914	Coefficient of risk (0.2s)
CR_1	0.899	Coefficient of risk (1.0s)
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.6	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.279	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.399	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.44	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.49	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)

$F_a = 1.2$

* See Section 11.4.8

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Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

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

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PRCTI20230247

SEISMIC FORCES

IEBC 2018 –

SECTION 806.3 –

USE OF REDUCED SEISMIC FORCES IS PERMITTED FOR ALTERATION.

REDUCED SEISMIC FORCE IS 75% OF OF CODE PRESCRIBED FORCE PER IEBC SECTION 303.3.2

SEISMIC SYSTEM –

N-S DIRECTION –

WEST WALL (NEW) CONCRETE SHEARWALL (SPECIAL)
$R = 5$ $\Omega = 2\frac{1}{2}$ $C_d = 5$

EAST WALL (EXIST) ORDINARY CMU WALL
$R = 2$ $\Omega = 2\frac{1}{2}$ $C_d = 1\frac{3}{4}$

MUST USE SMALLER OF THE TWO

$\therefore$ $R = 2$ $\Omega = 2\frac{1}{2}$ $C_d = 1\frac{3}{4}$

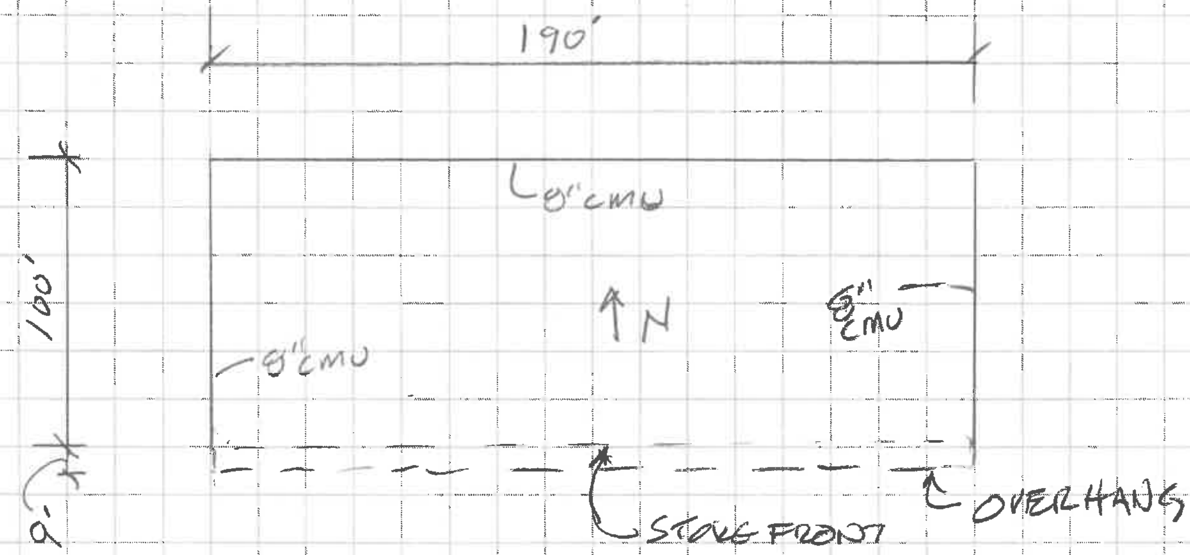
E-W DIRECTION –

NORTH WALL – ORDINARY CMU WALL
SAME AS ABOVE.

BSE
Brienen Structural Engineers, P.S.

Project: ______________ Date: ______________

SEISMIC WT



PLAN VIEW

$\frac{1}{2}(16')(190'-12'-12')(61psf) = 81.0^k$ (EXIST) NORTH WALL

$\frac{1}{2}(18' \text{ TALL})(100'-18')(61psf) = 40.0^k$ (E) WEST WALL Includes 2' Parapet

$\frac{1}{2}(13')(50')(75psf) = 24.4^k$ (NEW) WEST WALL

$\frac{1}{2}(16')(100')(61psf \text{ CMU}) = 48.8^k$ (E) EAST WALL

$109' \times 190' \times 15psf = 310.6^k$ ROOF + CEILING

$\frac{1}{2}(12')(500LF)(10psf) = 30^k$ INTERIOR PARTITIONS

TOTAL $= 535^k$

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Brienen Structural Engineers, P.S.

Project: ______________ Date: ______________

BASE SHEAR CALCULATION

ORDINARY MASONRY SHEARWALL –

$R = 2$ $\Omega = 2\frac{1}{2}$ $C_d = 1\frac{3}{4}$ $I_e = 1$ $S_{DS} = 1.02$

$C_s = \frac{S_{DS} I_e}{R}(.75) = \frac{1.02(1)(.75)}{2} = .383W$

$V_{ULT} = .383(535^k) = 205^k$

$V_{ASD} = .7(V_{ULT} = 205^k) = 143^k$

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NORTH WALL

PRCTI20230247

TABLE 2 **Net Section for Shear Calculations in Running Bond Type of Construction**

(2 Core Concrete Block Units)

ASSUMED FOR SEISMIC MASS

Nominal Wall Thickness		6 in.	8 in.	10 in.	12 in.
		Net Section in.^{2}/in.			
Solid Grouted		5.60	7.60	9.6	11.60
	16 in. O.C.	3.20	5.00	5.2	6.40
	24 in. O.C.	2.80	3.80	4.5	5.30
	32 in. O.C.	3.60	3.45	4.1	4.70
	40 in. O.C.	2.45	3.25	3.9	4.40
	48 in. O.C.	2.40	3.15	3.7	4.15
No Grout in Wall		2.0	2.5	3.0	3.0

ASSUMED FOR DESIGN WALL

NORTH CMU WALL

EXIST WALL LENGTH = 190' − (8×4') = 158'
↑ DOOR OPENINGS

L_{exist} = 158'

LENGTH TO BE REMOVED = 12' + (12' − 4') = 20'

L_{NEW} = 158' − 20' = 138'

$\frac{L_{EXIST}}{L_{NEW}} = \frac{158'}{138'} = 1.14$ 14% STRESS INCREASE

STRESS − $\rho V_{ASD} = \frac{(1.3)(143^k)}{138'} = 1347$ PLF

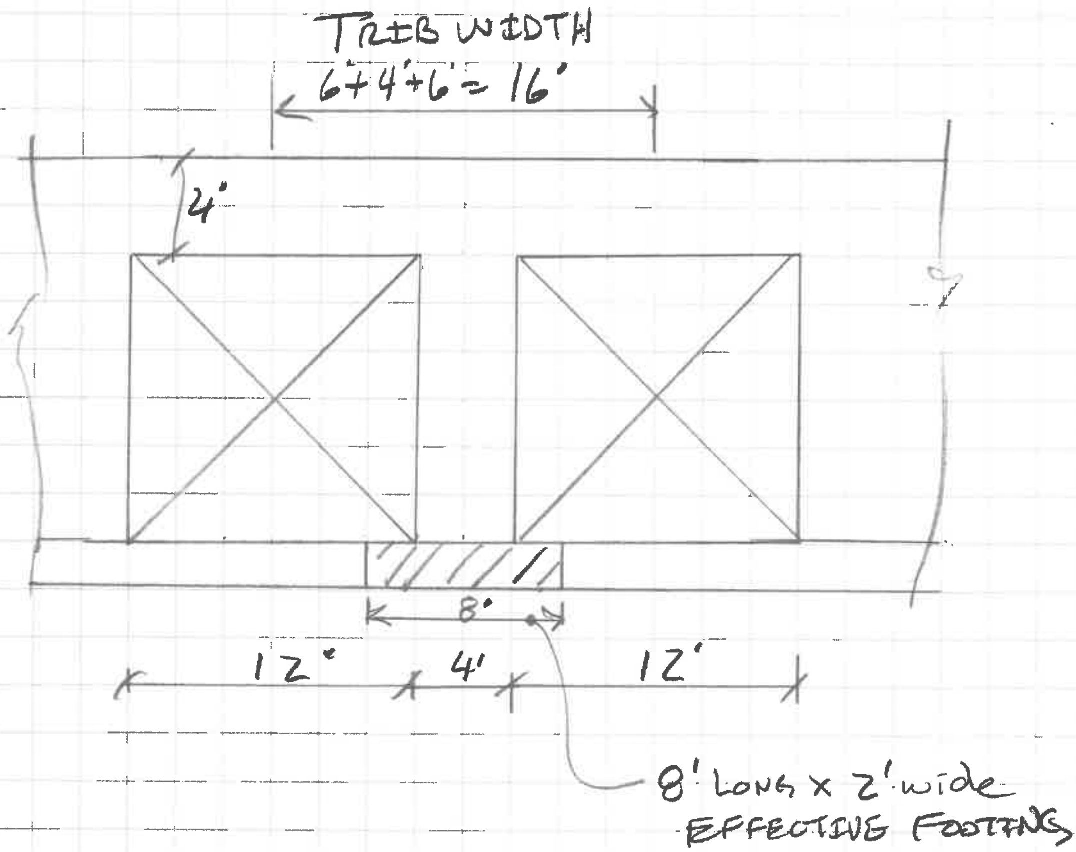
$A_{net} = 3.15"(12)"$
48" oc Grout

$\rho V_{ASD} = \frac{1347}{(3.15)(12)} = 36$ psi < 37 psi ok
↑ EFFECTIVE t GROUTED @ 48" OC

V_{allow} = 37 psi PER SECTION 8.2.6.2 (c)
Laid in running Bond

NORTH WALL IS STRUCTURALLY ACCEPTABLE WITH NEW OPENINGS.

CHECK FTG AT NORTH WALL



$$D = (6'+6')(4')(61^{psf}) + (16')(4')(61psf) + (6'+6')(100plf) + (\tfrac{25'}{2} \times 16' \times 15psf)$$

CMU HEADERS — CMU WALL — DOORS — ROOF FRAMING

$$D = 11032^{\#}$$

$$L = \frac{25'}{2}(16')(25psf) = 5000^{\#} \qquad P_{D+L} = 16032^{\#}$$

$$q = \frac{16032}{8' \times 2'} = 1000psf < 3000psf \text{ OK}$$

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WEST WALL

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MASONRY WALL – WEST

$$\rho V_{ASD} = \left(\frac{(1.3)\,205^{k}}{2}\right)\left(\frac{1}{50\,ft\ \text{long}}\right) = 2665\ PLF$$

$$\rho V_{ASD} = 2665 / (3.15")(12") = 70.5\,psi < 15\,psi$$

↖ Effective thickness Grouted @ 48" oc

V_{allow} = 15 psi PER SECTION 8.2.6.2 (f)
not laid in running bond.

EXISTING WEST MASONRY WALL IS OVERSTRESSED

ADD A NEW CONCRETE WALL –

SEE NEXT PAGE.

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NEW CONCRETE WALL (WEST SIDE)

$L = 50'$ LONG

$T = 6''$ Thick

$f'_c = 3000$ psi

$$\rho V_{ULT} = \frac{(1.3)(205^k/2)}{(50' \times 12)(6'')} = 37 \text{ psi}$$

$\phi V_n = \phi(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$ ACI 18.10.4.1

$h/\ell = 13'/50' = .9 < 1.5 \therefore \alpha_c = 3.0$ $\lambda = 1.0$ NORMAL WT CONC.

$\rho_t = .0025$ (min) $f_y = 60^{ksi}$

$\phi V_n = (.6)((3)(1)\sqrt{3000} + (.0025)(60000))$

$\phi V_n = 188$ psi $< \rho V_{ULT} = 74$ psi OK

$\therefore$ USE MIN REINF OF $\rho_t = .0025$

#4 @ 12 $\rho = \frac{.20''}{12'' \times 6''} = .0028 > .0025$ OK

USE #4 @ 12" OC EW

CHECK FOR BOUNDARY ELEMENTS –

$$S = \frac{bh^2}{6} = \frac{(6")(50' \times 12"/ft)^2}{6} = 360000 \text{ in}^3$$

$$f_c = \frac{PM}{S} = \frac{(1.3)(\frac{205^k}{2})(16')(12"/ft)}{360,000} = 71 \text{ psi} < 600 \text{ psi}$$

OK

$$F_{c\,ALLOWED} = .2f'_c = .2(3000\text{psi}) = 600^{psi}$$

↑ ACI 18.10.6.3

∴ NO BOUNDARY ELEMENTS REQUIRED.

CHECK FOOTING STRESS

WEST WALL

$M_{OT\,ASD} = \rho V_{ASD}\, h = 1.3\left(\frac{143^k}{2}\right)(16') = 1487^{\,k\text{-}ft}$

$P_{min} = (.6 - .14S_{ds})D =$

$(.56)\left[(75psf \times 13' \times 50') + (60psf \times 18' \times 50') + (15psf \times 3' \times 50') + (300PLF \times 52')\right]$

CONC WALL — CMU WALL — ROOF TRIB — FTG WT

$D = 121.5^k$

$P_{min} = 68^k$

$P_{MAX} = (1 + .14S_{ds})D + L$

$= 1.14\left[(121.5^k) + (25psf \times 3' \times 50')\right] =$

D — Roof Live

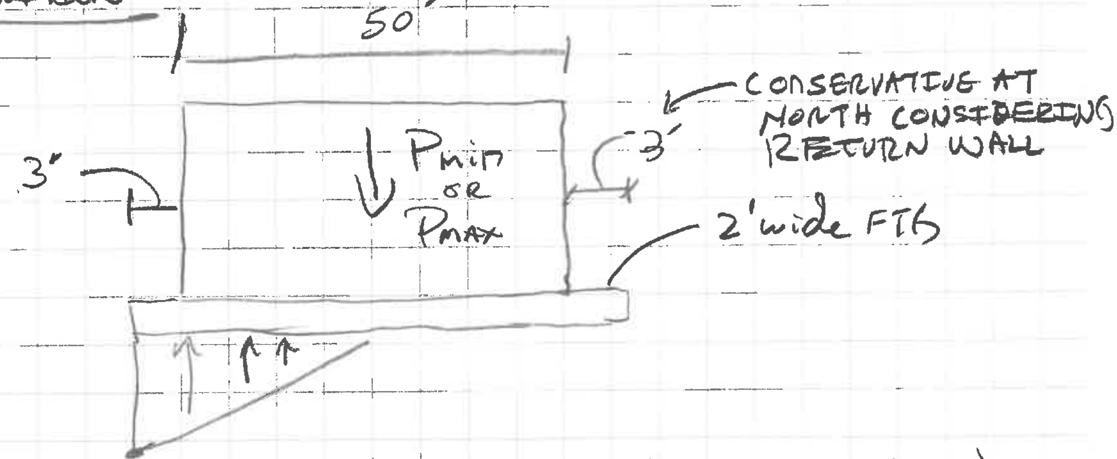
$P_{MAX} = 143^k$

ASSUMED ALLOWABLE BEARING PRESSURE

3000 psf AS SOIL HAVE BEEN COMPRESSING FOR SOME TIME NOW WITH NO SIGNS OF SETTLEMENT ISSUES

WEST WALL FTG (CONTINUED)

PMIN CONDITION



$$e = \frac{M_{OT}}{P_{min}} = \frac{1487'}{68^k} = 21.9'$$

$$q_{MAX} = \frac{4P_{min}}{3w(L-2e)} = \frac{4(68^k)}{3(2')(56'-2(21.9'))}$$

$$q_{MAX} = \underline{\underline{3716psf}} < 4/3(3000) = 4000psf \text{ OK}$$

PMAX CONDITION

$$e = \frac{M_{OT}}{P_{MAX}} = \frac{1487^{k\text{-}ft}}{143^k} = 10.4'$$

$$q_{MAX} = \frac{4P_{MAX}}{3w(L-2e)} = \frac{4(143^k)}{3(2')(56'-2(10.4))}$$

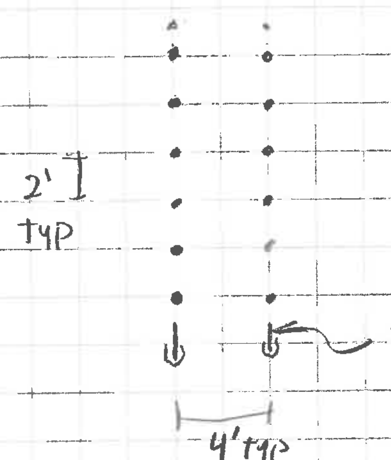
$$q_{MAX} = \underline{\underline{2708psf}} < 4/3(3000) = 4000psf \text{ } \underline{\underline{OK}}$$

TRANSFER OF SHEAR LOAD INTO WALL

$F_{px\,max} = 0.4\, S_{DS}\, I_E\, W_{px} = 0.4(1.023)(1)\left(\frac{535^k}{2}\right) = 110^k$

V PER #4 ADHESIVE DOWEL = 1.4^k/DOWEL ← SEE ATTACHED HILTI OUTPUT

$N = \frac{110}{1.4} = 79$ DOWELS



7/vert Col × 13 vert Col's = 91 DOWELS > 79 DOWEL

OK

DOWELS TO FND V PER DOWEL = 4^k/DOWEL ← SEE HILTI OUTPUT

$N_{REQ'D} = \frac{110^k}{4^k} = 28$

50 PROVIDED TO MATCH VERTS & PROVIDE MIN REINFORCING PER SHEARWALL DESIGN CRITERIA.

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Enterprise Concrete Wall Anchor	Date:	2/15/2023
Fastening point:			

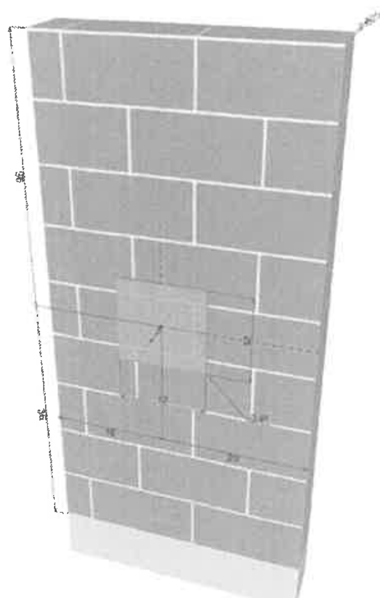
Specifier's comments:

1 Input data

Anchor type and diameter:	**HY 270 + Rebar A615 Gr.60 #4**
Item number:	not available (element) / 2194247 HIT-HY 270 (adhesive)
Effective embedment depth:	h_{ef} = 4.500 in.
Material:	ASTM A 615 GR.60
Evaluation Service Report:	ESR-4143
Issued I Valid:	3/1/2021 | 1/1/2022
Proof:	Design Method ASD Masonry
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.400 in.
Anchor plate[R] :	l_x x l_y x t = 12.000 in. x 12.000 in. x 0.400 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Joints: vertical: 0.375 in.; horizontal: 0.375 in. Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	no

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

Geometry [in.]



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

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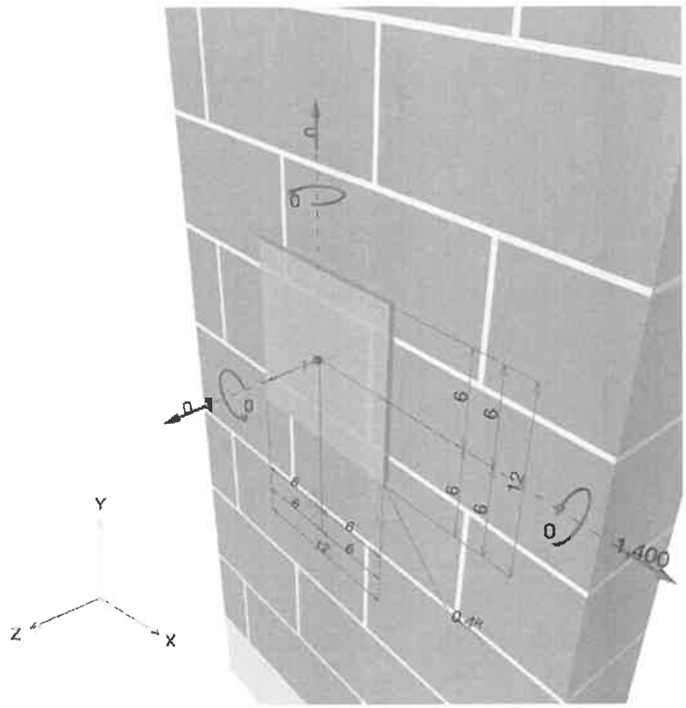
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 1,400; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	94

2 Load case/Resulting anchor forces

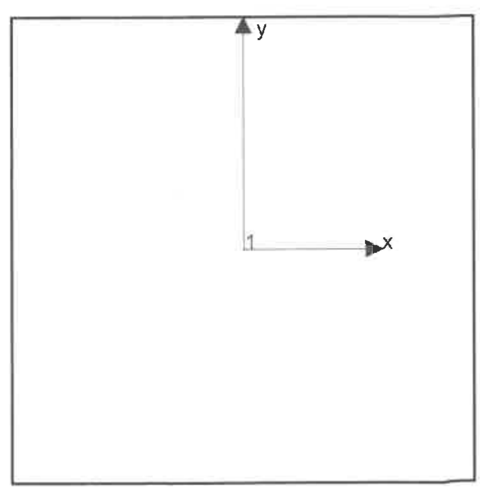
Load case: Service loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	1,400	1,400	0

max. compressive strain: - [‰]
max. compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]



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

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Design:	Enterprise Concrete Wall Anchor	Date:	2/15/2023
Fastening point:			

3 Tension load (Most utilized anchor 1)

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

4 Shear load (Most utilized anchor 1)

	Load V_s [lb]	Capacity V_t [lb]	Utilization $\beta_V = V_s/V_t$ [%]	Status
Steel strength	1,400	3,060	46	OK
Bond strength para and perp, (Dir. x+)[1]	-	-	94	OK

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

4.1 Steel strength

$V_{t,s}$ = ESR Value refer to ICC-ES ESR-4143

$V_{t,s} \geq V_s$

Results

$V_{t,s}$ [lb]	V_s [lb]
3,060	1,400

4.2 Bond strength parallel

$V_{t,b,Base,\parallel}$ = ESR Value refer to ICC-ES ESR-4143

$V_{t,b,\parallel} = V_{t,b,Base,\parallel} \cdot f_{red,E,\parallel} \cdot f_{red,s,\parallel} \cdot f_{red,Temp}$

$V_{t,b,\parallel} \geq V_{s,\parallel}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	12.000	4.000	18.000	68

Results

$V_{t,b,\parallel}$ [lb]	$V_{t,b,Base,\parallel}$ [lb]	$V_{s,\parallel}$ [lb]	$f_{red,E,\parallel}$	$f_{red,S,\parallel}$	$f_{red,Temp}$	Utilization $\beta_{V,\parallel}$ [%]
0	1,495	0	0.000	0.000	1.000	0

4.3 Bond strength perpendicular

$V_{t,b,Base,\perp}$ = ESR Value refer to ICC-ES ESR-4143

$V_{t,b,\perp} = V_{t,b,Base,\perp} \cdot f_{red,E,\perp} \cdot f_{red,s,\perp} \cdot f_{red,Temp}$

$V_{t,b,\perp} \geq V_{s,\perp}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	12.000	4.000	18.000	68

Results

$V_{t,b,\perp}$ [lb]	$V_{t,b,Base,\perp}$ [lb]	$V_{s,\perp}$ [lb]	$f_{red,E,\perp}$	$f_{red,S,\perp}$	$f_{red,Temp}$	Utilization $\beta_{V,\perp}$ [%]
1,495	1,495	1,400	1.000	1.000	1.000	94

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Enterprise Concrete Wall Anchor	Date:	2/15/2023
Fastening point:			

4.4 Shear interaction

$\beta_{V,\parallel} = \frac{V_{s,\parallel}}{V_{t,\parallel}}$	$\beta_{V,\perp} = \frac{V_{s,\perp}}{V_{t,\perp}}$	δ	Utilization β_V [%]	Status
0.000	0.936	1.667	94	OK

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

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry needs to be built in a regular way in accordance with state-of the art guidelines!
- Warnings/Notes - OST in Masonry HNA!

Fastening meets the design criteria!

6 Installation data

Anchor type and diameter: HY 270 + Rebar A615 Gr.60 #4

Profile: no profile

Item number: not available (element) / 2194247 HIT-HY 270 (adhesive)

Hole diameter in the fixture: d_f = - in.

Maximum installation torque: - in.lb

Plate thickness (input): 0.400 in.

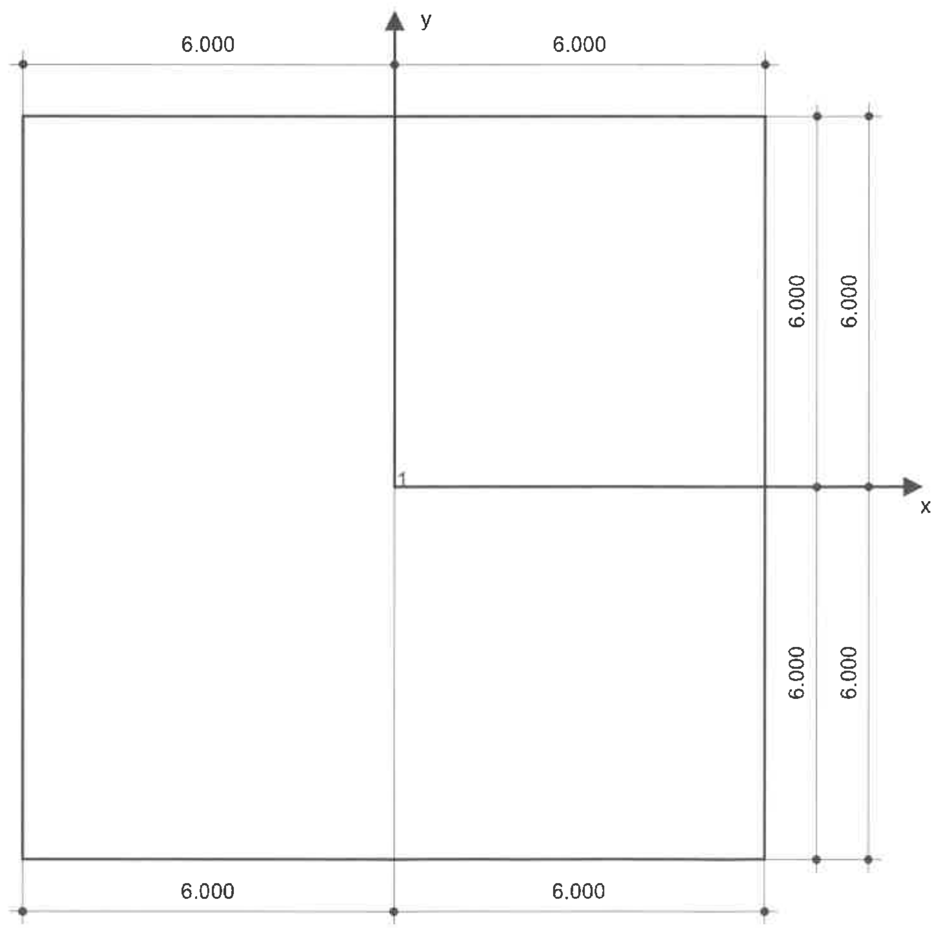
Hole diameter in the base material: 0.625 in.

Hole depth in the base material: 4.500 in.

Drilling method: Drilled in hammer mode

Minimum thickness of the base material: 7.625 in.

Rebar with HIT-HY 270 injection mortar with 4.5 in embedment h_ef, #4, Hammer drilled installation per ESR-4143



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	16.000	20.000	36.000	36.000

Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Enterprise Concrete Wall Anchor	Date:	2/15/2023
Fastening point:			

7 Remarks; Your Cooperation Duties

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BUILDING CODE

The 2018 edition of the 'international existing building code (IEBC), as adopted or amended by the city of Puyallup, shall govern design and construction.

SCOPE OF WORK

Design of the wall door cutouts on the west and north sides of the existing building. The scope of this package is not a seismic upgrade of the entire building, our is limited to replacing the seismic strength lost by wall cutouts. A full seismic upgrade is beyond the scope of this tenant improvement. The west wall requires additional seismic capacity and a concrete shearwall was added. The north wall of the building is approximately 190-feet long and did not require additional seismic capacity. Additional out of plane wall seismic bracing was added along the west wall to accommodate the additional out of plane loads at the location of the new concrete shear wall. New cutouts were framed with structural steel to support roof framing and coil doors. The 190-foot east-west length of the building spans numerous tenants and the tenant improvement associated with this submittal covers the western 50-feet of this building. We did not observe the interior framing of the eastern 140-feet of this building, it is assumed to be in similar condition to the western 50-foot of the building that we did observe.

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#4 DOWEL FROM WALL TO FND

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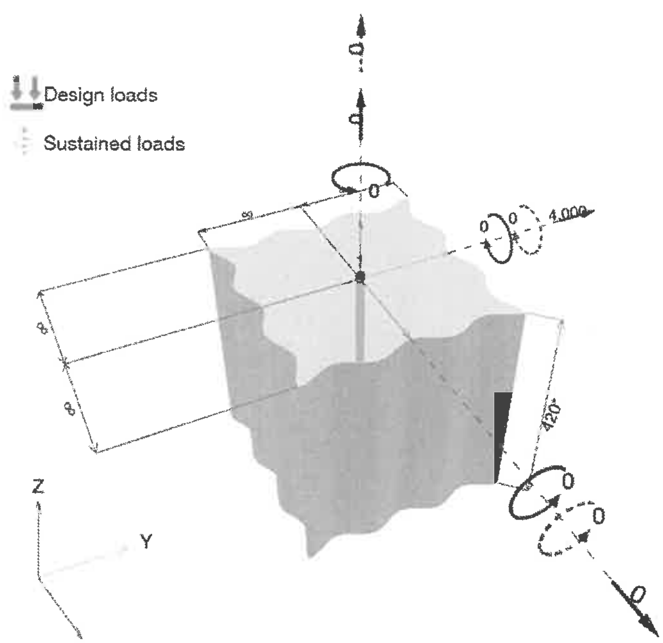
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Feb 15, 2023 (1)	Date:	2/15/2023
Fastening point:			

Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-RE 500 V3 + Rebar A615 Gr.60 #4**
Item number:	not available (element) / 2123401 HIT-RE 500 V3 (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 6.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM A 615 Gr.60
Evaluation Service Report:	ESR-3814
Issued I Valid:	3/1/2021 | 1/1/2023
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 420.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d)) Shear load: yes (17.10.6.3 (c))

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



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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Feb 15, 2023 (1)	Date:	2/15/2023
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0; V_x = 0; V_y = 4{,}000;$ $M_x = 0; M_y = 0; M_z = 0;$	yes	100

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	4,000	0	4,000

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Bond Strength**	N/A	N/A	N/A	N/A
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	4,000	4,032	100	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)**	4,000	16,598	25	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-3814

ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.20	80,000	0.700

Calculations

$V_{sa,eq}$ [lb]
6,720

Results

$V_{sa,eq}$ [lb]	ϕ $_{steel}$	ϕ $V_{sa,eq}$ [lb]	V_{ua} [lb]
6,720	0.600	4,032	4,000

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4.2 Pryout Strength (Bond Strength controls)

V_{cp}	$= k_{cp}\left[\left(\frac{A_{Na}}{A_{Na0}}\right)\psi_{ed,Na}\,\psi_{cp,Na}\,N_{ba}\right]$	ACI 318-19 Eq. (17.7.3.1a)
$\phi\ V_{cp}$	$\geq V_{ua}$	ACI 318-19 Table 17.5.2
A_{Na}	see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)	
A_{Na0}	$= (2\ c_{Na})^2$	ACI 318-19 Eq. (17.6.5.1.2a)
c_{Na}	$= 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$	ACI 318-19 Eq. (17.6.5.1.2b)
$\psi_{ed,Na}$	$= 0.7 + 0.3\left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.4.1b)
$\psi_{cp,Na}$	$= \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.5.1b)
N_{ba}	$= \lambda_a \cdot \tau_{k,c} \cdot \alpha_{N,seis} \cdot \pi \cdot d_a \cdot h_{ef}$	ACI 318-19 Eq. (17.6.5.2.1)

Variables

k_{cp}	$\alpha_{overhead}$	$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{k,c}$ [psi]
2	1.000	1,821	0.500	6.000	∞	1,398

c_{ac} [in.]	λ_a	$\alpha_{N,seis}$
10.061	1.000	0.900

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
6.404	164.07	164.07	1.000

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	11,856

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
23,712	0.700	1.000	1.000	16,598	4,000

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5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

6 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-RE 500 V3 + Rebar A615 Gr.60 #4

Item number: not available (element) / 2123401 HIT-RE 500 V3 (adhesive)

Maximum installation torque: -

Hole diameter in the base material: 0.625 in.

Hole depth in the base material: 6.000 in.

Minimum thickness of the base material: 7.250 in.

#4 Rebar with Hilti HIT-RE 500 V3

6.1 Recommended accessories

Drilling
- Suitable Rotary Hammer
- Properly sized drill bit

Cleaning
- Compressed air with required accessories to blow from the bottom of the hole
- Proper diameter wire brush

Setting
- Dispenser including cassette and mixer
- For deep installations, a piston plug is necessary
- Torque wrench

Coordinates Anchor in.

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

7 Remarks; Your Cooperation Duties

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

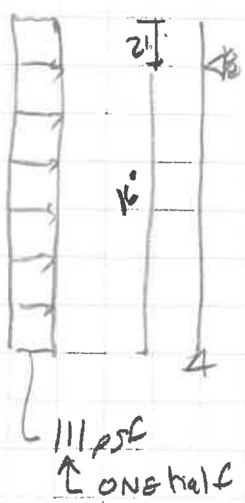
OUT OF PLANE WALL SUPPORT

8" CMU + 6" CONC = 61 psf + 75 psf = 136 psf

$S_{DS} = 1.023$ $k_a = 1.0 + \frac{L_f}{100} = 1.0 + \frac{100'}{100} = 2$

MIN ANCHORAGE FORCE PER ASCE 7-16 SECTION 12.11.1

$F_p = 0.4 S_{DS} k_a I_e W_p = .4(1.02)(2)(1)(136 psf) = 111 psf$

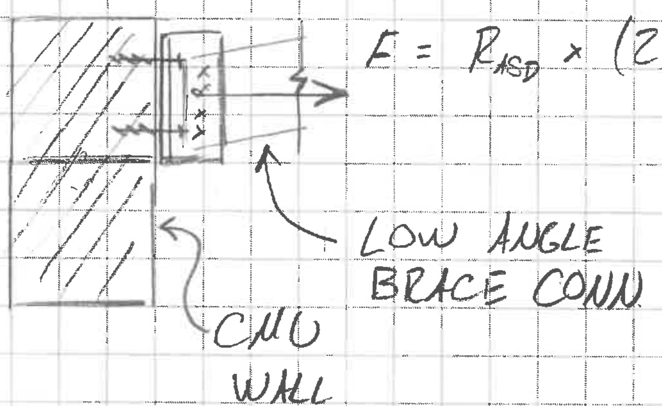


← R_{ULT} = 1110 PLF R_{ASD} = .7(1110) = 777 PLF

← R_{ULT} = 888 PLF R_{ASD} = .7 (888 PLF) = 622 PLF

Out of Plane Wall Support Cont.

$R_{ASD} = 777$ plf



$F = R_{ASD} \times (2ft) = 1554\#$ per brace

Try #10 SMS into 43mil brace

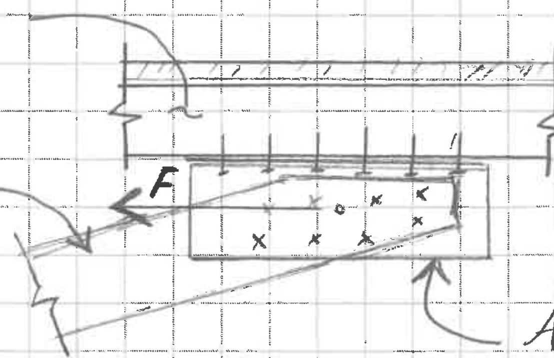
$V_{all} = 263\#$ ea per SSMA

$F / V_{all} = 1554\# / 263\# = 5.9$ screws

∴ Use (6) #10 SMS @ each brace connection

See Hilti printout for brace anchor design to CMU wall.

Out of Plane Wall Support Cont.



$F = 1554\#$ per brace

Try #10 SMS into 22ga roof deck (30 mil)

$V_{all} = 111\#$ ea per SSMA

$F/V_{all} = 1554\# / 111\# = 14$

See following Simpson CFS Designer printout for brace in compression check.

Screw Capacities

Table Notes

1. Capacities based on AISI S100 Section E4.
2. When connecting materials of different steel thicknesses or tensile strengths, use the lowest values. Tabulated values assume two sheets of equal thickness are connected.
3. Capacities are based on Allowable Strength Design (ASD) and include safety factor of 3.0.
4. Where multiple fasteners are used, screws are assumed to have a center-to-center spacing of at least 3 times the nominal diameter (d).
5. Screws are assumed to have a center-of-screw to edge-of-steel dimension of at least 1.5 times the nominal diameter (d) of the screw.
6. Pull-out capacity is based on the lesser of pull-out capacity in sheet closest to screw tip or tension strength of screw.
7. Pull-over capacity is based on the lesser of pull-over capacity for sheet closest to screw header or tension strength of screw.
8. Values are for pure shear or tension loads. See AISI Section E4.5 for combined shear and pull-over.
9. Screw Shear (Pss), tension (Pts), diameter, and head diameter are from CFSEI Tech Note (F701-12).
10. Screw shear strength is the average value, and tension strength is the lowest value listed in CFSEI Tech Note (F701-12).
11. Higher values for screw strength (Pss, Pts), may be obtained by specifying screws from a specific manufacturer.

Allowable Screw Connection Capacity (lbs)

Thickness (Mils)	Design Thickness	Fy Yield (ksi)	Fu Tensile (ksi)	#6 Screw (Pss = 643 lbs, Pts = 419 lbs) 0.138" dia, 0.272" Head			#8 Screw (Pss= 1278 lbs, Pts = 586 lbs) 0.164" dia, 0.272" Head			#10 Screw (Pss= 1644 lbs, Pts = 1158 lbs) 0.190" dia, 0.340" Head			#12 Screw (Pss= 2330 lbs, Pts = 2325 lbs) 0.216" dia, 0.340" Head			¼" Screw (Pss= 3048 lbs, Pts = 3201 lbs) 0.250" dia, 0.409" Head		
				Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over
18	0.0188	33	33	44	24	84	48	29	84	52	33	105	55	38	105	60	44	127
27	0.0283	33	33	82	37	127	89	43	127	96	50	159	102	57	159	110	66	191
30	0.0312	33	33	95	40	140	103	48	140	111	55	175	118	63	175	127	73	211
33	0.0346	33	45	151	61	140	164	72	195	177	84	265	188	95	265	203	110	318
43	0.0451	33	45	214	79	140	244	94	195	263	109	345	280	124	345	302	144	415
54	0.0566	33	45	214	100	140	344	118	195	370	137	386	394	156	433	424	180	521
68	0.0713	33	45	214	125	140	426	149	195	523	173	386	557	196	545	600	227	656
97	0.1017	33	45	214	140	140	426	195	195	548	246	386	777	280	775	1,016	324	936
118	0.1242	33	45	214	140	140	426	195	195	548	301	386	777	342	775	1,016	396	1,067
54	0.0566	50	65	214	140	140	426	171	195	534	198	386	569	225	625	613	261	752
68	0.0713	50	65	214	140	140	426	195	195	548	249	386	777	284	775	866	328	948
97	0.1017	50	65	214	140	140	426	195	195	548	356	386	777	405	775	1,016	468	1,067
118	0.1242	50	65	214	140	140	426	195	195	548	386	386	777	494	775	1,016	572	1,067

Weld Capacities

Table Notes

1. Capacities based on the AISI S100 Specification Sections E2.4 for fillet welds and E2.5 for flare groove welds.
2. When connecting materials of different steel thicknesses or tensile strengths, use the lowest values.
3. Capacities are based on Allowable Strength Design (ASD).
4. Weld capacities are based on E60 electrodes. For material thinner than 68 mil, 0.030" to 0.035" diameter wire electrodes may provide best results.
5. Longitudinal capacity is considered to be loading in the direction of the length of the weld.
6. Transverse capacity is loading in perpendicular direction of the length of the weld.
7. For flare groove welds, the effective throat of weld is conservatively assumed to be less than 2t.
8. For longitudinal fillet welds, a minimum value of EQ E2.4-1, E2.4-2, and E2.4-4 was used.
9. For transverse fillet welds, a minimum value of EQ E2.4-3 and E2.4-4 was used.
10. For longitudinal flare groove welds, a minimum value of EQ E2.5-2 and E2.5-3 was used.

Allowable Weld Capacity (lbs / in)

Thickness (Mils)	Design Thickness	Fy Yield (ksi)	Fu Tensile (ksi)	Fillet Welds		Flare Groove Welds	
				Longitudinal	Transverse	Longitudinal	Transverse
43	0.0451	33	45	499	864	544	663
54	0.0566	33	45	626	1084	682	832
68	0.0713	33	45	789	1365	859	1048
97	0.1017	33	45	1125	1269	-[1]	-[1]
54	0.0566	50	65	905	1566	985	1202
68	0.0713	50	65	1140	1972	1241	1514
97	0.1017	50	65	1269	1269	-[1]	-[1]

[1] *Weld capacity for material thickness greater than 0.10" requires engineering judgment to determine leg of welds, W1 and W2.*

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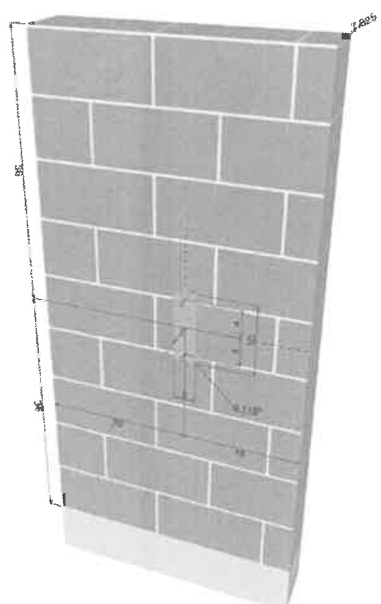
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 1/4 (2 1/2)**
Item number:	418046 KH-EZ 1/4"x3"
Effective embedment depth:	h_{ef} = 2.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3056
Issued I Valid:	1/1/2021 | 10/1/2022
Proof:	Design Method ASD Masonry
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.118 in.
Anchor plate^R :	l_x x l_y x t = 3.000 in. x 10.000 in. x 0.118 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.;
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
	Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	no

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.]



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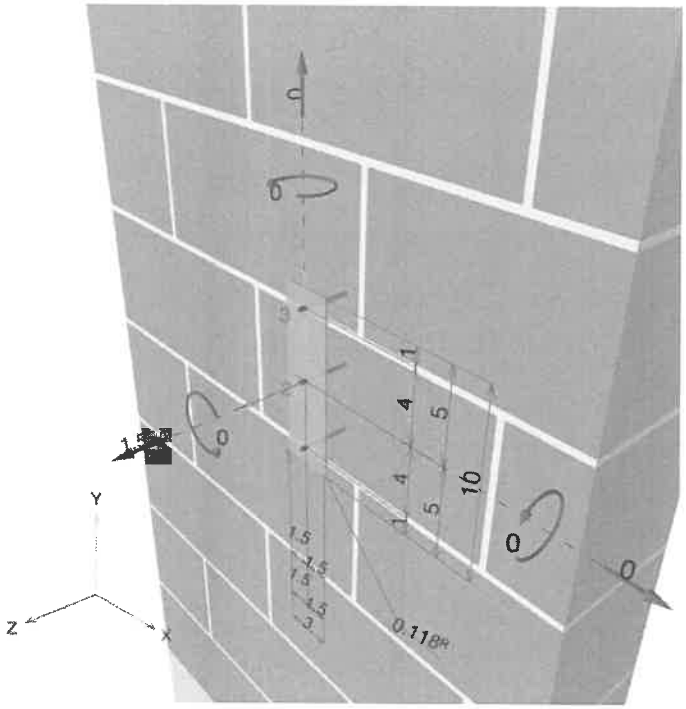
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 1,554; V_x = 0; V_y = 0;$ $M_x = 0; M_y = 0; M_z = 0;$	no	82

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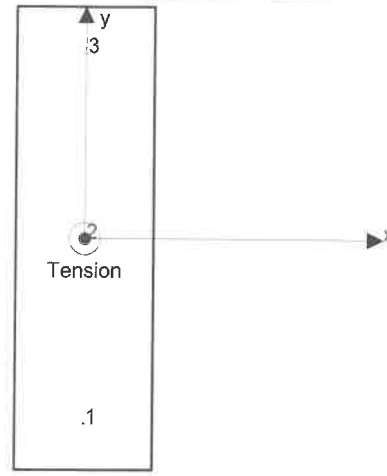
2 Load case/Resulting anchor forces

Load case: Service loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	518	0	0	0
2	518	0	0	0
3	518	0	0	0



max. compressive strain: - [‰]
max. compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 1,554 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

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

3 Tension load (Most utilized anchor 1)

	Load P_s [lb]	**Capacity P_t [lb]**	**Utilization $\beta_P = P_s/P_t$ [%]**	**Status**
Overall strength	518	633	82	OK

3.1 Overall strength

$P_{t,Base}$ = ESR Value refer to ICC-ES ESR-3056

$P_t = P_{t,Base} \cdot f_{red,E} \cdot f_{red,s} \cdot f_{red,Temp} \cdot f_{red,Bedjoint}$

$P_t \geq P_s$

Variables

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

Results

P_t [lb]	$P_{t,Base}$ [lb]	P_s [lb]	$f_{red,E}$	$f_{red,S}$	$f_{red,Temp}$	$f_{red,Bedjoint}$
633	728	518	1.000	1.000	1.000	0.870

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4 Shear load (Most utilized anchor 1)

	Load V_s [lb]	Capacity V_t [lb]	Utilization $\beta_V = V_s/V_t$ [%]	Status
Overall strength	N/A	N/A	N/A	N/A

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry needs to be built in a regular way in accordance with state-of the art guidelines!
- Warnings/Notes - OST in Masonry HNA!

Fastening meets the design criteria!

6 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = - in.

Plate thickness (input): 0.118 in.

Drilling method: Drilled in hammer mode

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 1/4 (2 1/2)

Item number: 418046 KH-EZ 1/4"x3"

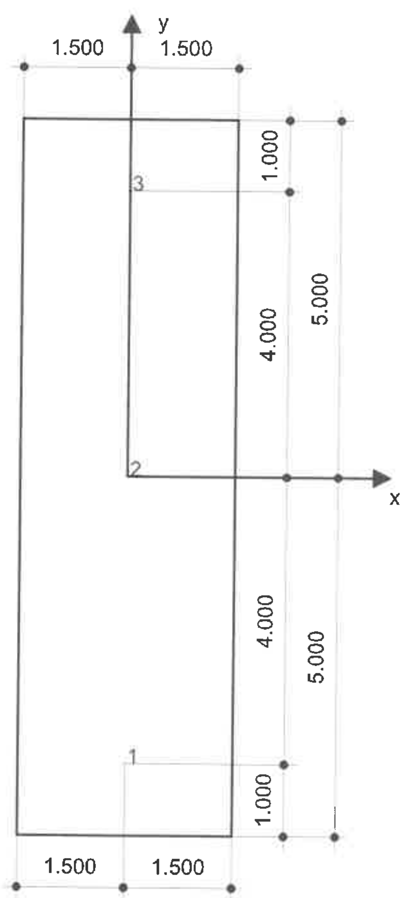
Maximum installation torque: 252 in.lb

Hole diameter in the base material: 0.250 in.

Hole depth in the base material: 2.875 in.

Minimum thickness of the base material: 7.625 in.

Hilti KH-EZ screw anchor with 2.5 in embedment, 1/4 (2 1/2), Steel galvanized, installation per ESR-3056



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	-4.000	20.000	16.000	32.000	40.000
2	0.000	0.000	20.000	16.000	36.000	36.000
3	0.000	4.000	20.000	16.000	40.000	32.000

7 Remarks; Your Cooperation Duties

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Screw Capacities

Table Notes

1. Capacities based on AISI S100 Section E4.
2. When connecting materials of different steel thicknesses or tensile strengths, use the lowest values. Tabulated values assume two sheets of equal thickness are connected.
3. Capacities are based on Allowable Strength Design (ASD) and include safety factor of 3.0.
4. Where multiple fasteners are used, screws are assumed to have a center-to-center spacing of at least 3 times the nominal diameter (d).
5. Screws are assumed to have a center-of-screw to edge-of-steel dimension of at least 1.5 times the nominal diameter (d) of the screw.
6. Pull-out capacity is based on the lesser of pull-out capacity in sheet closest to screw tip or tension strength of screw.
7. Pull-over capacity is based on the lesser of pull-over capacity for sheet closest to screw header or tension strength of screw.
8. Values are for pure shear or tension loads. See AISI Section E4.5 for combined shear and pull-over.
9. Screw Shear (Pss), tension (Pts), diameter, and head diameter are from CFSEI Tech Note (F701-12).
10. Screw shear strength is the average value, and tension strength is the lowest value listed in CFSEI Tech Note (F701-12).
11. Higher values for screw strength (Pss, Pts), may be obtained by specifying screws from a specific manufacturer.

Allowable Screw Connection Capacity (lbs)

Thickness (Mils)	Design Thickness	Fy Yield (ksi)	Fu Tensile (ksi)	#6 Screw (Pss = 643 lbs, Pts = 419 lbs) 0.138" dia, 0.272" Head			#8 Screw (Pss= 1278 lbs, Pts = 586 lbs) 0.164" dia, 0.272" Head			#10 Screw (Pss= 1644 lbs, Pts = 1158 lbs) 0.190" dia, 0.340" Head			#12 Screw (Pss= 2330 lbs, Pts = 2325 lbs) 0.216" dia, 0.340" Head			¼" Screw (Pss= 3048 lbs, Pts = 3201 lbs) 0.250" dia, 0.409" Head		
				Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over
18	0.0188	33	33	44	24	84	48	29	84	52	33	105	55	38	105	60	44	127
27	0.0283	33	33	82	37	127	89	43	127	96	50	159	102	57	159	110	66	191
30	0.0312	33	33	95	40	140	103	48	140	111	55	175	118	63	175	127	73	211
33	0.0346	33	45	151	61	140	164	72	195	[illegible]	84	265	188	95	265	203	110	318
43	0.0451	33	45	214	79	140	244	94	195	263	109	345	280	124	345	302	144	415
54	0.0566	33	45	214	100	140	344	118	195	370	137	386	394	156	433	424	180	521
68	0.0713	33	45	214	125	140	426	149	195	523	173	386	557	196	545	600	227	656
97	0.1017	33	45	214	140	140	426	195	195	548	246	386	777	280	775	1,016	324	936
118	0.1242	33	45	214	140	140	426	195	195	548	301	386	777	342	775	1,016	396	1,067
54	0.0566	50	65	214	140	140	426	171	195	534	198	386	569	225	625	613	261	752
68	0.0713	50	65	214	140	140	426	195	195	548	249	386	777	284	775	866	328	948
97	0.1017	50	65	214	140	140	426	195	195	548	356	386	777	405	775	1,016	468	1,067
118	0.1242	50	65	214	140	140	426	195	195	548	386	386	777	494	775	1,016	572	1,067

#10 Vall for Screw to stud

#10 Vall for Screw to roof deck

Weld Capacities

Table Notes

1. Capacities based on the AISI S100 Specification Sections E2.4 for fillet welds and E2.5 for flare groove welds.
2. When connecting materials of different steel thicknesses or tensile strengths, use the lowest values.
3. Capacities are based on Allowable Strength Design (ASD).
4. Weld capacities are based on E60 electrodes. For material thinner than 68 mil, 0.030" to 0.035" diameter wire electrodes may provide best results.
5. Longitudinal capacity is considered to be loading in the direction of the length of the weld.
6. Transverse capacity is loading in perpendicular direction of the length of the weld.
7. For flare groove welds, the effective throat of weld is conservatively assumed to be less than 2t.
8. For longitudinal fillet welds, a minimum value of EQ E2.4-1, E2.4-2, and E2.4-4 was used.
9. For transverse fillet welds, a minimum value of EQ E2.4-3 and E2.4-4 was used.
10. For longitudinal flare groove welds, a minimum value of EQ E2.5-2 and E2.5-3 was used.

Allowable Weld Capacity (lbs / in)

Thickness (Mils)	Design Thickness	Fy Yield (ksi)	Fu Tensile (ksi)	Fillet Welds		Flare Groove Welds	
				Longitudinal	Transverse	Longitudinal	Transverse
43	0.0451	33	45	499	864	544	663
54	0.0566	33	45	626	1084	682	832
68	0.0713	33	45	789	1365	859	1048
97	0.1017	33	45	1125	1269	-[1]	-[1]
54	0.0566	50	65	905	1566	985	1202
68	0.0713	50	65	1140	1972	1241	1514
97	0.1017	50	65	1269	1269	-[1]	-[1]

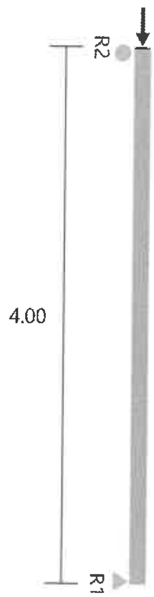
[1] *Weld capacity for material thickness greater than 0.10" requires engineering judgment to determine leg of welds, W1 and W2.*

PRCTI20230247

City of Puyallup Development & Permitting Services ISSUED PERMIT
Building | Planning | Engineering | Public Works | Fire | Traffic

Project Name: Out of Plane Brace
Model: Out of Plane Brace
Code: 2012 NASPEC [AISI S100-2012]

Date: 02/15/2023
Simpson Strong-Tie® CFS Designer™ 4.2.0.13



Section : 400S250-43 (33 ksi) @ 24" o.c. Single C Stud (unpunched)
Maxo = 888.0 ft-lb **Va =** 1739.1 lb **I =** 1.22 in^4

Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	None, None	None, 48.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	0.0	--Shear Connection w/ clip--				NO
R1	0.0	--Stud/Track Design, Ref Connectors--				NO

Gravity Load

Type	Load (lb)
Uniform	0.00plf
P1y	1554.00lb @ 4.00ft

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	1554.0(c)	4998.0(c)	31%	KΦ=0.00 lb-in/in Max KL/r = 51
	Max. Shear, lbs	0.0	1739.1	0%	
	Max. Moment (MaFy, Ma-dist), ft-lbs	0.0	867.9	0%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	0.0	888.0	0%	
	Shear/Moment	0.00	1.00	0%	Shear 0.0, Moment 0.0
	Axial/Moment	0.31	1.00	31%	Axial 1554.0(c), Moment 0.0
	Deflection Span, in	0.000	--meets L/0--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	0.0	0.0	SCB45.5(2) & Anchorage Designed by Engineer	0.00 %	NA
R1	0.0	1554.0	400T125-33 (33) & Anchorage Designed by Engineer	0.00 %	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

PRCTI20230247

City of Puyallup
Development & Permitting Services
ISSUED PERMIT
Building | Planning
Engineering | Public Works
Fire | Traffic

BSE

Brienen Structural Engineers, P.S.

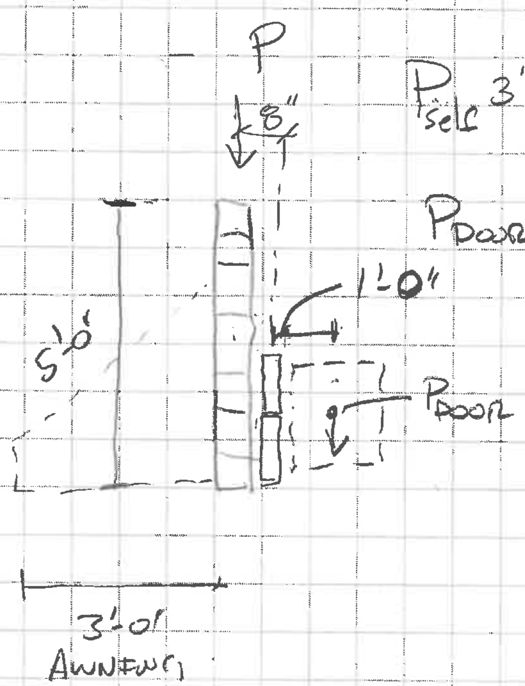
Project: ______________ Date: ______________

COILING DOOR HEADER

BSE
Brienen Structural Engineers, P.S.

Project: ____________ Date: ____________

HEADER @ WEST OPENING (NORTH SMALLER)



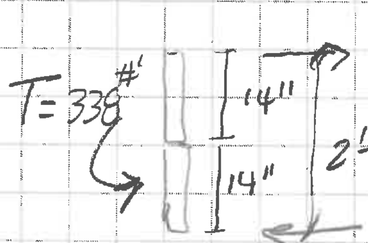
$P_{self} = 3'(10\,psf) + 3'(25\,psf) + 5'(80\,psf) = 505\,plf$

(Roof) (Snow) (CMU)

$P_{DOOR} = 80\,plf$

$T_{self} = 505\,plf \times \frac{8"}{12"} = 338\,\#'$

$T_{DOOR} = 80\,plf \times 1' = 80\,\#'$ Counters self wt (conservative to ignore)



$R_{TORSION} = \frac{338\,\#'}{2'-8"} = 145\,\#$

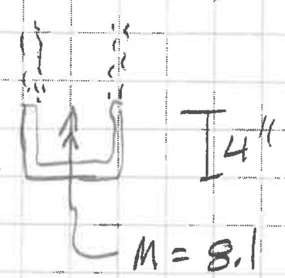
$R_{Torsion} = \frac{338\,\#'}{2'-8"} = 145\,\#$

BOTTOM ANGLE + TUBE —

OUT OF PLANE LOAD = $P = 145^{\#} + 30^{\#} + 50^{\#} = 225^{\#}$

$L = 18'\text{-}6"$ $M_{ASD} = \frac{w\ell^2}{8} = \frac{.225 k/ft (18.5')^2}{8} = 8.1^{k\text{-}ft}$

check Bot of Hss



$S = \frac{1}{2}$ of HSS 8x4x1/4 Weak Axis

$S = \frac{7.2}{2} = 3.6 \text{ in}^3$

$f_{b_y} = \frac{M}{S} = \frac{(8.1)(12)}{3.6} = 27^{ksi} < .67F_y = .67(46^{ksi}) = 31^{ksi}$ OK

↑ A LITTLE CONSV, AS IT DOES NOT INCLUDE THE ANGLE

PRCTI20230247

BSE Brienen Structural Engineers, P.S.

City of Puyallup Development & Permitting Services ISSUED PERMIT
Building | Planning | Engineering | Public Works | Fire | Traffic

Project: ____________ Date: ____________

HEADER DESIGN (vertical Load)

$L = 18'$ TRIB AREA $= \frac{40'}{2} = 20'$

USE GRAVITY LOADS FROM NORTH HEADER

$D = \frac{25'}{2} \times 15psf + 120PLF + 320 + 50 = 677$ PLF
(coil Door; CMU HEADER; steel Header)
USE 800PLF

$L = 25psf$ SNOW $\times \frac{25'}{2} = 313$ PLF
USE 500PLF

$W_{ASD} = 800_{PLF}$ (D) $+ 500_{PLF}$ (L) $= 1.3^{k}/ft$

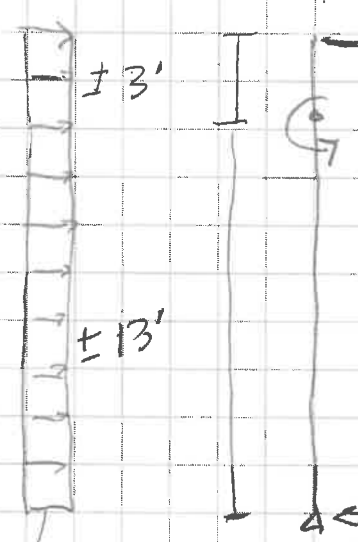
$M_{ASD} = \frac{w\ell^2}{8} = \frac{1.3(18')^2}{8} = 53^{k\text{-}ft}$

$f_b = \frac{M}{S} = \frac{53^{k\text{-}ft}(12)}{25.4\ in^3} = 25^{ksi} < .67 \times F_y = 31^{ksi}$
(46ksi) OK

↑ HSS 14x4x 1/4

NOTE THAT A DOUBLE TUBE IS USED TO PROVIDE SURFACE FOR COILING DOOR

JAMB STEEL @ WEST OPENING



$R_{wind} = 616^{\#}$ $R_{mom} = 169^{\#}$ $R_{TOTAL\,ASD} = 785^{\#}$

$M = \frac{16'}{2} \times 338^{\#'}/ft = 2.7^{kft}$

opening width TORSION

$R_{wind} = \frac{1}{2}(16')(77_{PLF}) = 616^{\#}$ $R_{mom} = \frac{2.7^{kft}}{16'} = 169^{\#}$

$R_{TOTAL\,ASD} = 785^{\#}$

$W = 8' \times (.6 \times 16_{PSF}) = 77_{PLF}$

ASD TRIB ZONE 4 WIND ASD

CHECK HSS5x5x1/4

$f_b = \frac{M}{S} = \frac{(2.7^{kft})(12)}{6.41\ in^3} = 5.1^{ksi} < .67F_y = 31^{ksi}$

OK

PRCTI20230247

JAMBS AT WINDOW OPENING

W_{wind} (ASD) = 4' x (17.4 psf x .6) = 42 PLF

$M = \frac{w\ell^2}{8} = \frac{(42\,PLF)(16)^2}{8} = 1.34^{kft}$

$f = \frac{(1.34^{kft})(12)}{1.73\,in^3} = 9.3^{ksi} < .6F_y = 22ksi$ OK

(F_y = 36 ksi)

USE AN MC6 x 15.1
OR MC 8 x 18.7