Reviewed 2/28/2023 DL
Subject to field inspectors approvals.

STRUCTURAL CALCULATIONS

FOR

SEISMIC ANALYSIS OF STEEL STORAGE RACKS AND FIXTURES

BEST BUY STORE #0366 – PUYALLUP, WA
AST PROJECT #WA 1069

DAVID C. BUCHANAN
STATE OF WASHINGTON
54815
REGISTERED PROFESSIONAL ENGINEER
01/13/23

PERMIT SUBMITTAL
JANUARY 13, 2023

PREPARED BY:

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

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PRCTI20230074

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SECTION A

DESIGN NARRATIVE

DESIGN NARRATIVE

In retail facilities, the safety of the public during an earthquake event is not only dependent on the performance of the building structure, but also the structural performance of the steel storage rack systems and displays. As a result, steel storage rack systems and displays must be designed to prevent collapse or overturning when subjected to the code-prescribed gravity and seismic loads. The calculations contained in this package are provided to demonstrate the structural adequacy of the steel storage rack system and displays proposed at this location.

SCOPE

The scope of these calculations is limited to the structural analysis/design of the steel storage racks between 5'-9" and 12'-0" high subjected to gravity (weight of racks/displays and their contents) and code-prescribed seismic loads. Anchorage of the racking system or display to the slab on grade is also included as part of these structural calculations.

CODES

The seismic structural analysis/design of the racking system contained in these calculations is based on the following codes:

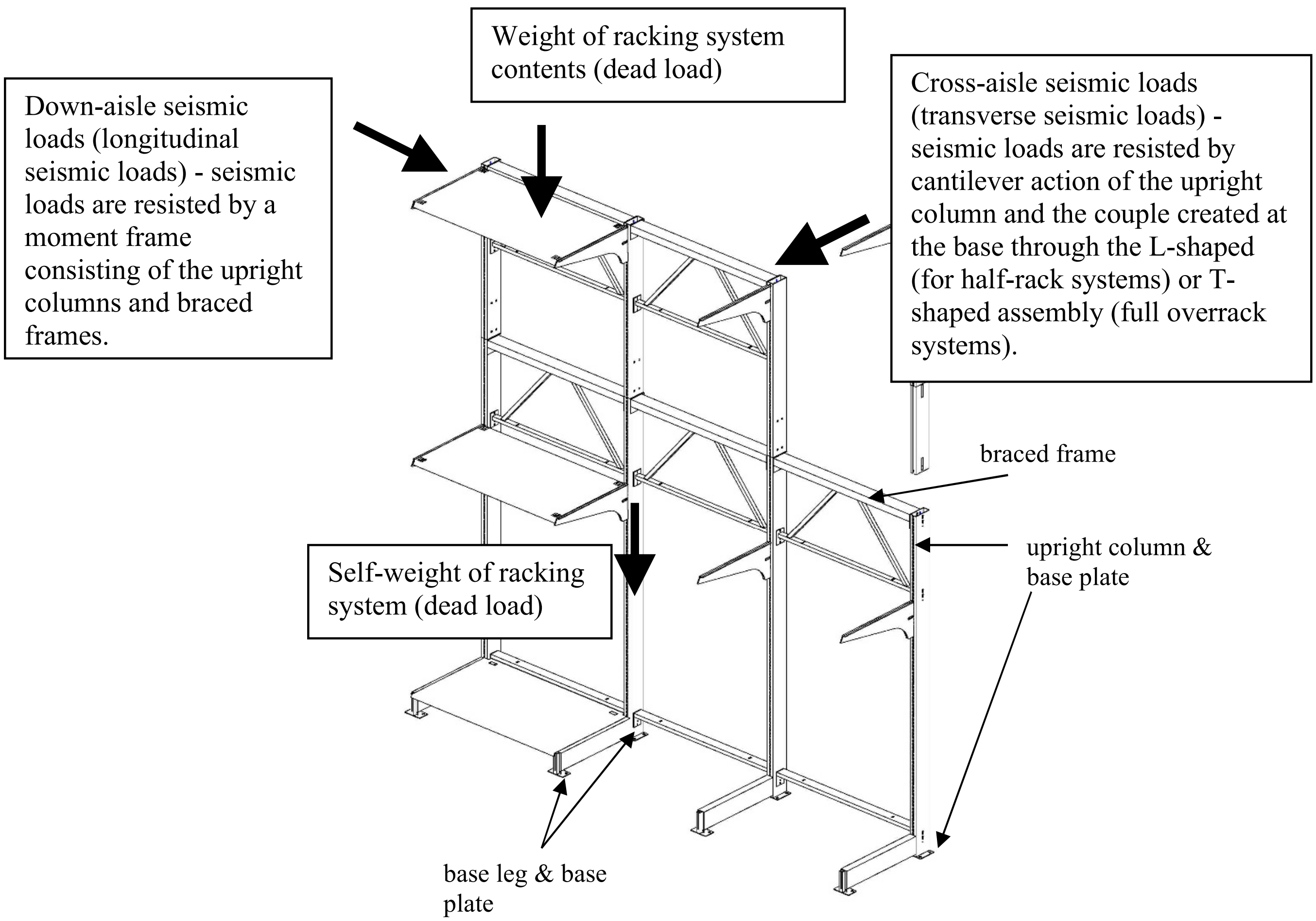
- 2018 International Building Code
- RMI Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks
- ASCE 7-16 – Minimum Design Loads for Buildings and Other Structures
- AISI North American Specification for the Design of Cold-Formed Steel Structural Members

RACK SYSTEMS

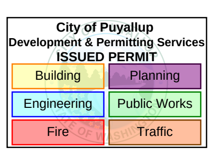
The racking consists of two main types: gondola overrack systems & warehouse rack systems. The rack systems are analyzed for two conditions of operating (seismic) weight: a) weight of the rack plus every storage level loaded to 67% of rated load capacity and b) weight of the rack plus the highest storage level only loaded to 100% of its rated capacity. See pages A2 & A3 and the appropriate sections of these calculations for additional information on these racking systems.

GONDOLA OVERRACK DESIGN (8'-0" High Half Gondola Overrack, 8'-0" Full Gondola Overrack & 12'-0" Half High Gondola Overrack)

1. Rack system is analyzed considering the loads shown using the Visual Analysis software program (version 19.00.0002). Note: the 12'-0" high half rack controls and all three racks are designed for these loads.



2. The system components are analyzed to determine their adequacy to support the design loads. The main seismic resisting components include the upright column, the base leg, the braced frame and the connections between these components.
3. The system is checked for overturning, the base plates analyzed and the anchorage of the rack system base plates to the concrete slab is designed.
4. The concrete slab and the supporting soil are then analyzed to determine if they can support the design loads.



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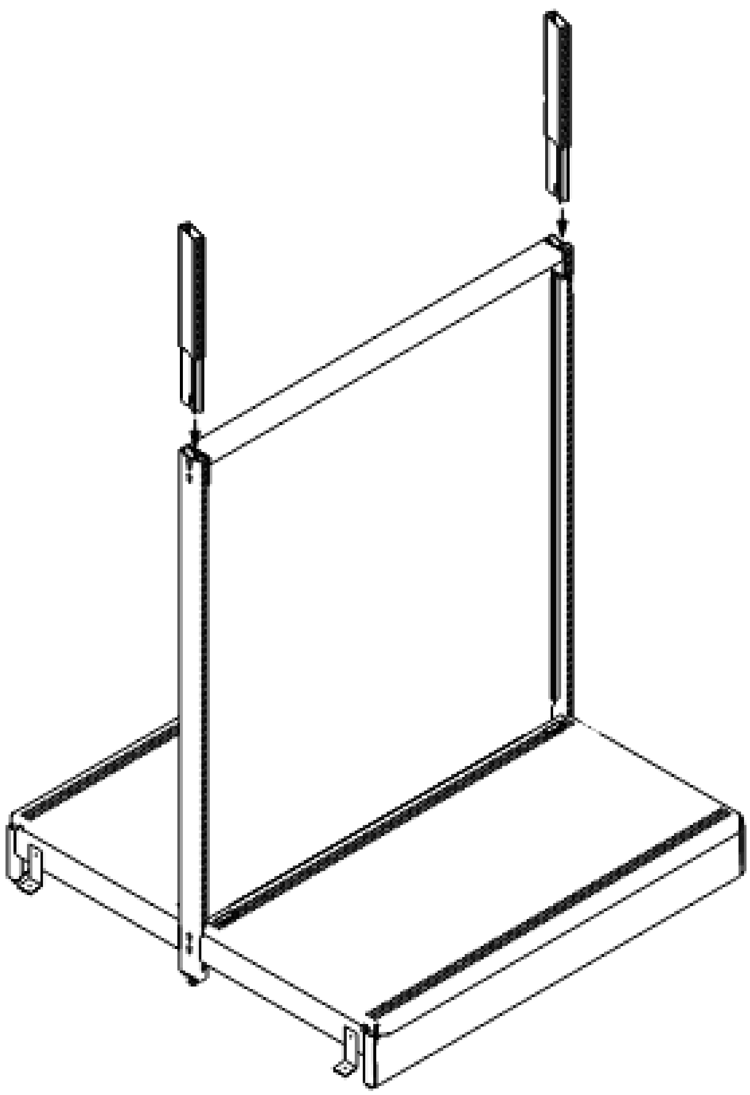


SECTION H

FIXTURE TYPE LBKH
(LOWBAY WITH EXTENDER)

Project	MN 1806
Date	11/11/2019
By	EAL
Sheet	1 of 3

THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCE(S) FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

Updated: 09/03/19
By: EAL

DESCRIPTION >	Lowbay rack with 18" extender

PROJECT INFORMATION

		SITE NAME:	TBD
CITY:	TBD	SITE ADDRESS:	TBD
COUNTY:	TBD	LATITUDE:	TBD
STATE:	TBD	LONGITUDE:	TBD

SEISMIC LOAD ANAYLSIS

OCCUPANCY CATEGORY:		II	(Table 1-1)
SITE CLASS:		D-default	
SHORT PERIOD ACCELERATION:	S_s =	2.484	(From USGS Website or Figure 22-1 to 22-18)
.-SECOND PERIOD ACCELERATION:	S_1 =	0.85	(From USGS Website or Figure 22-1 to 22-18)
IMPORATANCE FACTOR:	I =	*1*	(Table 1.5-2)
SITE	F_a =	*1.2*	(Table 11.4-1)
COEFICIENTS:	F_v =	*1.7*	(Table 11.4-2)

Note: fixture is designed for:
Ss = 2.484
S1 = 0.85

Actual Seismic coefficients are:
Ss = 1.261
S1 = 0.435

NOTE: scroll down to input other seismic coefficients (ap, Rp, z and h)

RESPONSE SPECTRUM

S_{MS} =	*2.981*	$S_{MS} = F_a \times S_s$	(EQN 11.4-1)
S_{M1} =	*1.445*	$S_{M1} = F_v \times S_1$	(EQN 11.4-2)
S_{DS} =	*1.987*	$S_{DS} = (2/3)S_{MS}$	(EQN 11.4-3)
S_{D1} =	*0.963*	$S_{D1} = (2/3)S_{M1}$	(EQN 11.4-4)

Earthquake Design Criteria

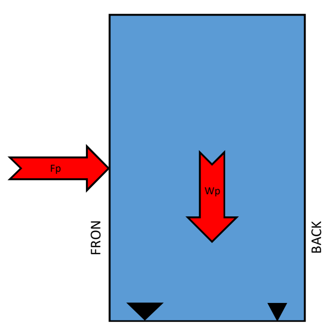
SEISMIC DESIGN CATEGORY

S_{DS} =	*D*	(Table 11.6-1)
S_{D1} =	*E*	(Table 11.6-2)
SDC FOR DESIGN =	*E*	

Seismic Weight

Self Weight of Fixture = 120 LBS (Fixture weight will be distributed to shelves)
Number of Shelves = 1

	Shelf	Weight of Contents (LBS)	Shelf Height Above Base (IN)	Total Weight	F_p Coefficient	Seismic Force	OTM at Base (LBS-IN)
COMPONENT MAKEUP	*SHELF #1*	*120*	36.00	*120*	*59.60%*	*72*	2574.7
	SHELF #2	*0*	0.00	*0*	*59.60%*	*0*	0.0
	SHELF #3	*0*	0.00	*0*	*59.60%*	*0*	0.0
	SHELF #4	*0*	0.00	*0*	*59.60%*	*0*	0.0
	CONTENT #1	200	78.00	*200*	*59.60%*	*119*	9297.6
	CONTENT #2	200	45.00	*200*	*59.60%*	*119*	5364.0
	CONTENT #3	0	0.00	*0*	*59.60%*	*0*	0.0
	CONTENT #4	0	0.00	*0*	*59.60%*	*0*	0.0
				520		*310*	*17236.3* LBS-IN
				W_p		**F_p**	*1436.4* LBS-FT



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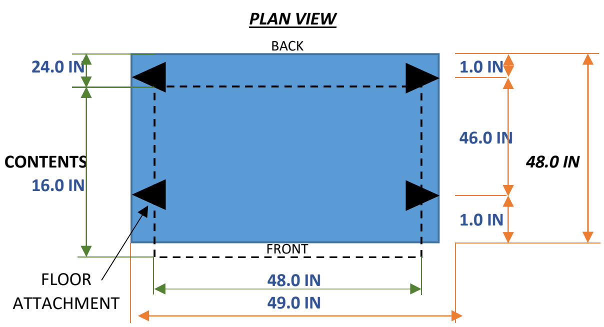
Project	MN 1806
Date	11/11/2019
By	EAL
Sheet	2 of 3

THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCE(S) FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

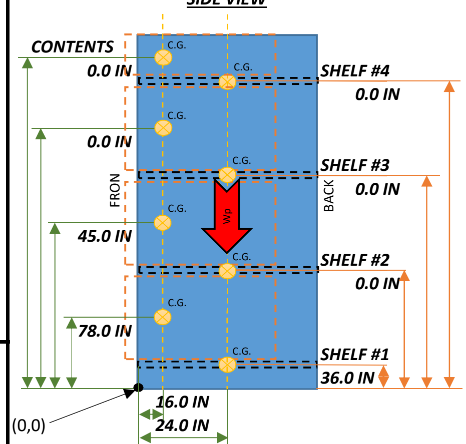
DESCRIPTION >	*Lowbay rack with 18" extender*

THIS IS AN EXAMPLE LAYOUT ONLY

PLAN VIEW



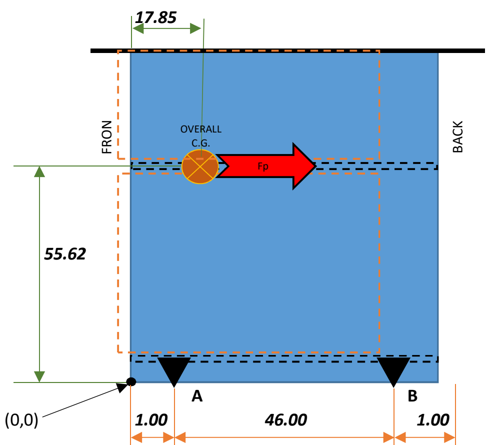
SIDE VIEW



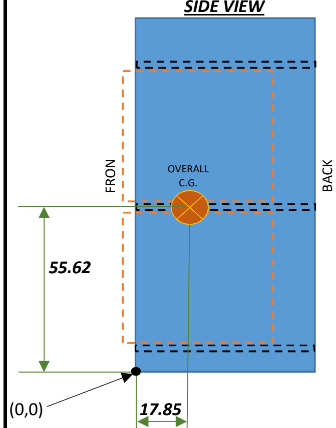
X_{cm} =	*17.85* IN
Y_{cm} =	*55.62* IN

$W_{p\ Shelves}$ =	*120 LBS*
$W_{p\ Component}$ =	*400 LBS*
F_p =	*310 LBS*

BASE CONNECTION FORCES



SIDE VIEW



@ FRONT	@ BACK	@ FRONT ANCHOR	@ REAR ANCHOR
901 LBS	***679 LBS***	450 LBS	339 LBS
OR	OR	OR	OR
-209 LBS	***-279 LBS***	-104 LBS	-139 LBS

THE ABOVE REACTIONS ARE PER SIDE USING THE FOLLOWING LRFD EQUATIONS	
(0.9-0.2SDS)*DL +/- Q_E	*Uplift (-)*
(0.503)*DL +/- Fp	*Uplift (-)*
(1.2+0.2SDS)*DL +/- *QE	*Compression (+)*
(1.597)*DL +/- Fp	*Compression (+)*

Project	MN 1806
Date	11/11/2019
By	EAL
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THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCE(S) FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

DESCRIPTION >	***Lowbay rack with 18" extender***

Seismic Design Requirements for Non-Structural Components (ASCE 7-16, Chapter 13)

I_p = **1.0** (Section 13.1.3)

$$F_p = \frac{0.4 * a_p * S_{DS} * W_p}{(R_p / I_p)} [1 + 2 (z/h)] \quad \text{(EQN 13.3-1)}$$

$F_{p,max}$ = $1.6 * S_{DS} * I_p * W_p$ (EQN 13.3-2)

$F_{p,min}$ = $0.3 * S_{DS} * I_p * W_p$ (EQN 13.3-3)

a_p = **1.0** (Table 13.5-1)

R_p = **2.5** (Table 13.5-1)

z = **0** IN

h = **120** IN

Table 13.5-1 Coefficients for Architectural Components

Architectural Component	a_p[a]	R_p[b]
Interior nonstructural walls and partitions[b]		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever elements (Unbraced or braced to structural frame below its center of mass)		
Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys where laterally braced or supported by the structural frame	2.5	2.5
Cantilever elements (Braced to structural frame above its center of mass)		
Parapets	1.0	2.5
Chimneys	1.0	2.5
Exterior nonstructural walls[b]	1.0[b]	2.5
Exterior nonstructural wall elements and connections[b]		
Wall element	1.0	2.5
Body of wall panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1.0
Veneer		
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Penthouses (except where framed by an extension of the building frame)	2.5	3.5
Ceilings		
All	1.0	2.5
Cabinets		
Permanent floor-supported storage cabinets over 6 ft (1,829 mm) tall, including contents	1.0	2.5
Permanent floor-supported library shelving, book stacks, and bookshelves over 6 ft (1,829 mm) tall, including contents	1.0	2.5
Laboratory equipment	1.0	2.5
Access floors		
Special access floors (designed in accordance with Section 13.5.7.2)	1.0	2.5
All other	1.0	1.5
Appendages and ornamentations	2.5	2.5
Signs and billboards	2.5	3.0
Other rigid components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability materials and attachments	1.0	1.5
Other flexible components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5
Egress stairways not part of the building structure	1.0	2.5

[a] A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for rigid components and rigidly attached components. The value of $a_p = 2.5$ is for flexible components and flexibly attached components.
[b] Where flexible diaphragms provide lateral support for concrete or masonry walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 12.11.2.

F_p = ***0.318*** **** W_p*** (EQN 13.3-1)

$F_{p,max}$ = ***3.18*** **** W_p*** (EQN 13.3-2)

$F_{p,min}$ = ***0.596*** **** W_p*** (EQN 13.3-3)

F_p = ***0.596*** **** W_p***

W_p = ***520.0*** LBS (Assembled wood cabinet + fixtures installed into display)

F_p = ***309.9*** LBS

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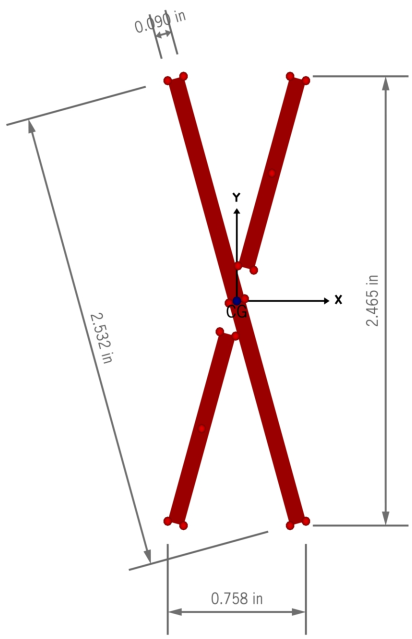

Geometric Properties

Area	0.422	in^2
Ix	0.226	in^4
Ixy	0.000	in^4
Iy	0.017	in^4
Sx+	0.183	in^3
Sx-	0.183	in^3
Sy+	0.046	in^3
Sy-	0.046	in^3
Xc	0.000	in
Yc	0.000	in
rx	0.732	in
ry	0.203	in

Principal Properties

I1	0.226	in^4
I2	0.017	in^4
S1+	0.183	in^3
S1-	0.183	in^3
S2+	0.046	in^3
S2-	0.046	in^3
r1	0.732	in
r2	0.203	in
⍰	0.028	deg

Polar Properties

Ip	0.243	in^4
rp	0.759	in

Overall Properties

Depth	2.465	in
Perimeter	9.918	in
Weight	0.001	K/ft
Width	0.758	in

Plastic Properties

Xpna	0.000	in
Ypna	0.000	in
Zx	0.275	in^3
Zy	0.076	in^3

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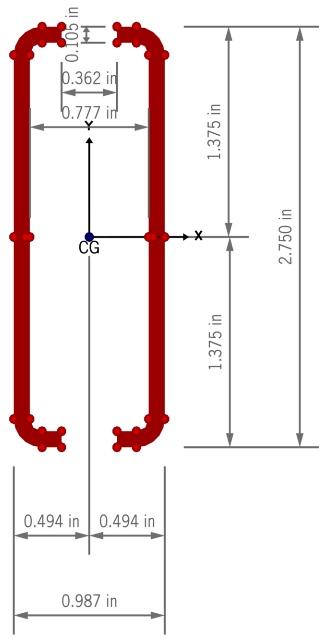

Geometric Properties

Area	0.641	in^2
Ix	0.473	in^4
Ixy	0.000	in^4
Iy	0.115	in^4
Sx+	0.344	in^3
Sx-	0.344	in^3
Sy+	0.233	in^3
Sy-	0.233	in^3
Xc	0.000	in
Yc	0.000	in
rx	0.859	in
ry	0.423	in

Principal Properties

I1	0.473	in^4
I2	0.115	in^4
S1+	0.344	in^3
S1-	0.344	in^3
S2+	0.233	in^3
S2-	0.233	in^3
r1	0.859	in
r2	0.423	in
□	0.000	deg

Polar Properties

Ip	0.588	in^4
rp	0.958	in

Overall Properties

Depth	2.750	in
Perimeter	12.607	in
Weight	0.002	K/ft
Width	0.987	in

Plastic Properties

Xpna	0.000	in
Ypna	0.000	in
Zx	0.480	in^3
Zy	0.268	in^3

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By EAL
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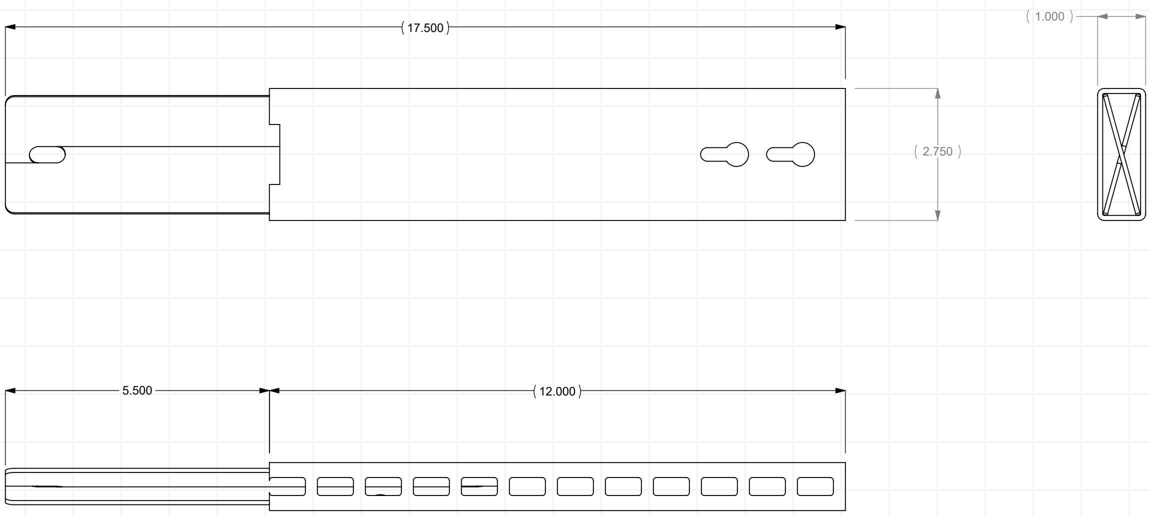
BEST BUY LOWBAY EXTENDER



Assumptions

$S_S = 2.484$

P = 250 lbs @ top of 12" extender

= 200 lbs @ top of 18" extender

Seismic Load

$S_{DS} = 1.987$

F_p = 0.596*P = (0.58)(250 lbs) = 149 lbs for 12" extender

F_p = 0.596*P = (0.58)(200 lbs) = 119 lbs for 18" extender

Bending Capacity

$M_n = M_p = F_yZ \leq 1.6M_y$

F_yZ = (33,000 psi)(0.0759 in^3) = 2,505 lb-in

$1.6M_y$ = (1.6)(33,000 psi)(0.0458 in^3) = 2,418 lb-in

ϕM_n = (0.9)(2,418 lb-in) = 2,176 lb-in

12" Extender:

M_u = (149 lbs)(12 in) = 1,788lb-in ≤ 2,176 lb-in --> OKAY

18" Extender:

Mu = (119 lbs)(18 in) = 2,142 lb-in ≤ 2,176 lb-in --> OKAY

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Phone I Fax:	|	E-Mail:	
Design:	Section H - S-OTR.3 (TZ2 Anchor Update)	Date:	4/14/2022
Fastening point:			

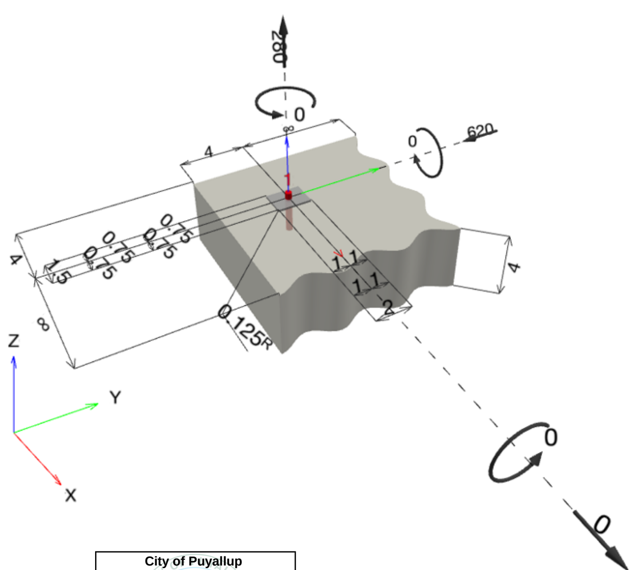
Specifier's comments:

1 Input data

Anchor type and diameter:	**Kwik Bolt TZ2 - CS 3/8 (2) hnom2**
Item number:	2210236 KB-TZ2 3/8x3
Effective embedment depth:	$h_{ef,act}$ = 2.000 in., h_{nom} = 2.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-4266
Issued I Valid:	12/17/2021 | 12/1/2023
Proof:	Design Method ACI 318-19 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.125 in.
Anchor plate^R :	l_x x l_y x t = 1.500 in. x 2.000 in. x 0.125 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 4.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d)) Shear load: yes (17.10.6.3 (c))

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

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



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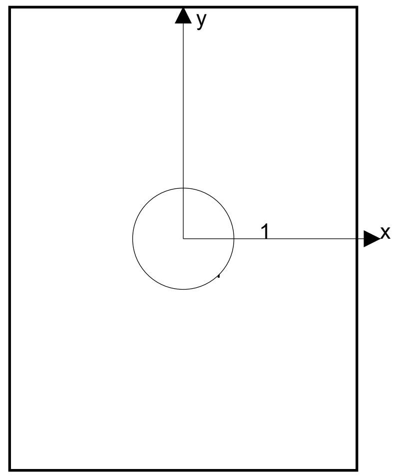
Design:	Section H - S-OTR.3 (TZ2 Anchor Update)	Date:	4/14/2022
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 280; V_x = 0; V_y = -620;$ $M_x = 0; M_y = 0; M_z = 0;$	yes	61

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	280	620	0	-620

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 280 [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

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	280	4,869	6	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	280	1,448	20	OK

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

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Design:	Section H - S-OTR.3 (TZ2 Anchor Update)	Date:	4/14/2022
Fastening point:			

3.1 Steel Strength

N_{sa} = ESR value — refer to ICC-ES ESR-4266

$\phi\ N_{sa} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.05	126,204

Calculations

N_{sa} [lb]
6,493

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
6,493	0.750	1.000	4,869	280

3.2 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

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

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
2.000	4.000	1.000	4.375	21	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
36.00	36.00	1.000	1.000	2,970

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
2,970	0.650	0.750	1.000	1,448	280

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Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	620	2,201	29	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	620	2,079	30	OK
Concrete edge failure in direction y-**	620	1,027	61	OK

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

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-4266

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

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.05	126,204	1.000

Calculations

$V_{sa,eq}$ [lb]
3,386

Results

$V_{sa,eq}$ [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi V_{sa,eq}$ [lb]	V_{ua} [lb]
3,386	0.650	1.000	2,201	620

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4.2 Pryout Strength

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ed,N} \ \psi_{c,N} \ \psi_{cp,N} \ N_b \right]$ ACI 318-19 Eq. (17.7.3.1a)

$\phi \ V_{cp} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 \ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

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

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \ \lambda_a \ \sqrt{f'_c} \ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1	2.000	4.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
4.375	21	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
36.00	36.00	1.000	1.000	2,970

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi \ V_{cp}$ [lb]	V_{ua} [lb]
2,970	0.700	1.000	1.000	2,079	620

4.3 Concrete edge failure in direction y-

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ ACI 318-19 Eq. (17.7.2.1a)

$\phi V_{cb} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vc0} = 4.5 c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = \left(7 \left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\, c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
4.000	4.000	1.000	4.000	2.000

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.375	2,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
40.00	72.00	0.900	1.225	2,396

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕV_{cb} [lb]	V_{ua} [lb]
1,468	0.700	1.000	1.000	1,027	620

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.193	0.604	5/3	50	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 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.
- 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/
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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Fastening point:			

7 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = 0.438 in.

Plate thickness (input): 0.125 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: Kwik Bolt TZ2 - CS 3/8 (2) hnom2

Item number: 2210236 KB-TZ2 3/8x3

Maximum installation torque: 361 in.lb

Hole diameter in the base material: 0.375 in.

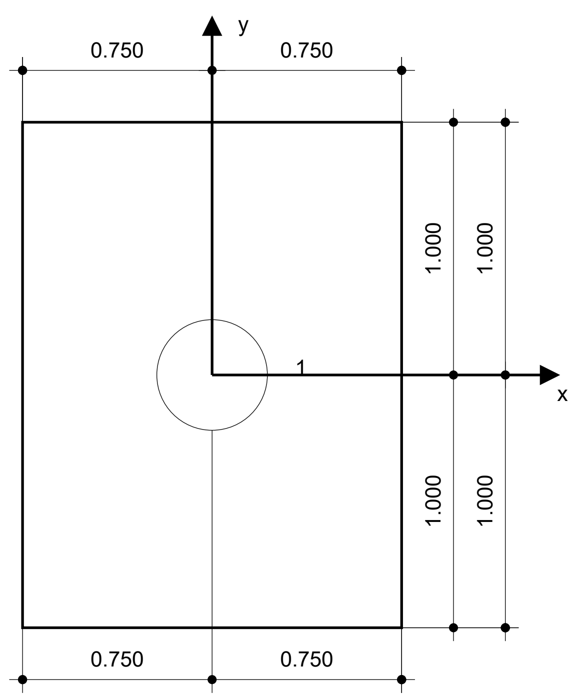
Hole depth in the base material: 2.750 in.

Minimum thickness of the base material: 4.000 in.

Hilti KB-TZ2 stud anchor with 2.5 in embedment, 3/8 (2) hnom2, Carbon steel, installation per ESR-4266

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hammer



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	-0.000	4.000	-	4.000	[illegible]

8 Remarks; Your Cooperation Duties

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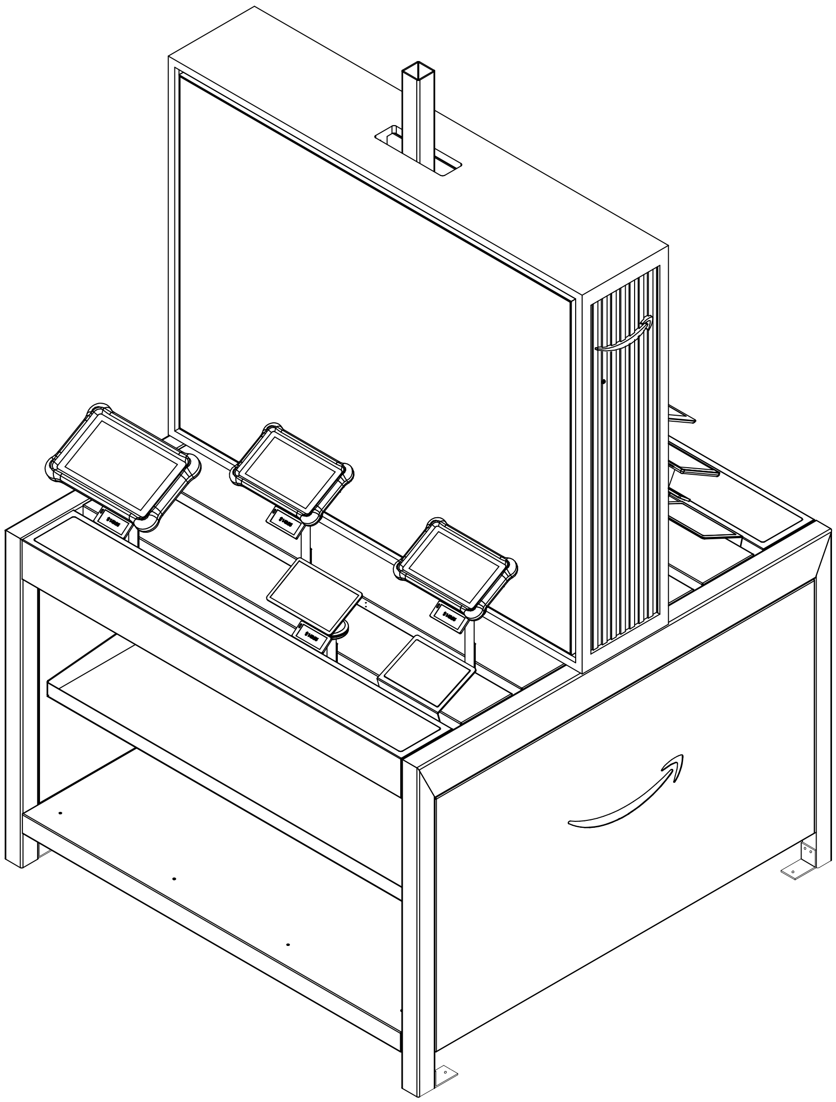
SECTION K

VENDOR PROVIDED DISPLAYS



S-AMZ.3

6'-0" HIGH AMAZON PLAY TABLE

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PRCTI20230074

K724

Project	MN 2033
Date	8/12/2022
By	KCS
Sheet	1 of 3

THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCE(S) FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

Updated: 08/10/22
By: KCS

DESCRIPTION >	**BBY Play Table**

PROJECT INFORMATION

		SITE NAME:	**TBD**
CITY:	**TBD**	SITE ADDRESS:	**TBD**
COUNTY:	**TBD**	LATITUDE:	**TBD**
STATE:	**TBD**	LONGITUDE:	**TBD**

SEISMIC LOAD ANAYLSIS

OCCUPANCY CATEGORY:		**II**	(Table 1-1)
SITE CLASS:		**D-default**	
SHORT PERIOD ACCELERATION:	S_s =	**2.484**	(From USGS Website or Figure 22-1 to 22-18)
L-SECOND PERIOD ACCELERATION:	S_1 =	**0.85**	(From USGS Website or Figure 22-1 to 22-18)
IMPORATANCE FACTOR:	I =	***1***	(Table 1.5-2)
SITE	F_a =	***1.2***	(Table 11.4-1)
COEFICIENTS:	F_v =	***1.7***	(Table 11.4-2)

Note: fixture is designed for:
Ss = 2.484
S1 = 0.85

Actual Seismic coefficients are:
Ss = 1.261
S1 = 0.435

NOTE: scroll down to input other seismic coefficients (ap, Rp, z and h)

RESPONSE SPECTRUM

S_{MS} =	***2.981***	$S_{MS} = F_a \times S_s$	(EQN 11.4-1)
S_{M1} =	***1.445***	$S_{M1} = F_v \times S_1$	(EQN 11.4-2)
S_{DS} =	***1.987***	$S_{DS} = (2/3)S_{MS}$	(EQN 11.4-3)
S_{D1} =	***0.963***	$S_{D1} = (2/3)S_{M1}$	(EQN 11.4-4)

Earthquake Design Criteria

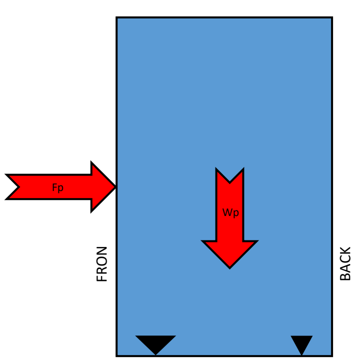
SEISMIC DESIGN CATEGORY

S_{DS} =	***D***	(Table 11.6-1)
S_{D1} =	***E***	(Table 11.6-2)
SDC FOR DESIGN =	***E***	

Seismic Weight

Self Weight of Fixture = **471** LBS (Fixture weight will be distributed to shelves)
Number of Shelves = **1**

	Shelf	Weight of Contents (LBS)	Shelf Height Above Base (IN)	Total Weight	F_p Coefficient	Seismic Force	OTM at Base (LBS-IN)
COMPONENT MAKEUP	***SHELF #1***	***471***	33.00	***471***	***59.60%***	***281***	9263.6
	SHELF #2	***0***	0.00	***0***	***59.60%***	***0***	0.0
	SHELF #3	***0***	0.00	***0***	***59.60%***	***0***	0.0
	SHELF #4	***0***	0.00	***0***	***59.60%***	***0***	0.0
	CONTENT #1	0	0.00	***0***	***59.60%***	***0***	0.0
	CONTENT #2	0	0.00	***0***	***59.60%***	***0***	0.0
	CONTENT #3	0	0.00	***0***	***59.60%***	***0***	0.0
	CONTENT #4	0	0.00	***0***	***59.60%***	***0***	0.0
				471		***281***	***9263.6*** LBS-IN
				W_p		**F_p**	***772.0*** LBS-FT



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K725

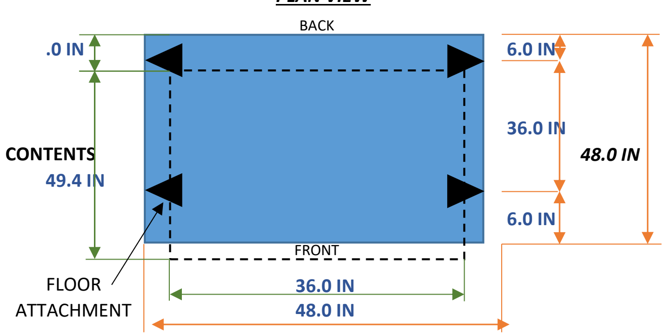
Project	MN 2033
Date	8/12/2022
By	KCS
Sheet	2 of 3

THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCE(S) FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

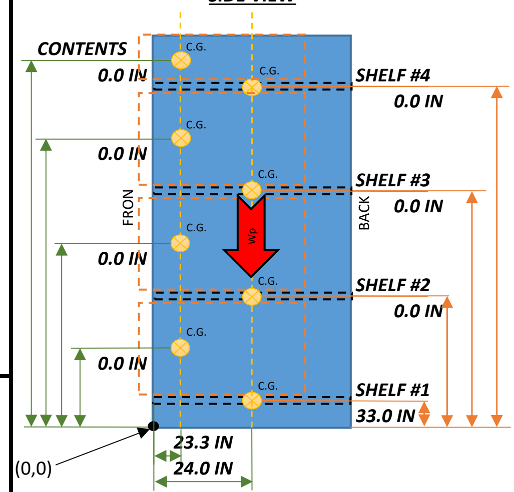
DESCRIPTION >	*BBY Play Table*

THIS IS AN EXAMPLE LAYOUT ONLY

PLAN VIEW



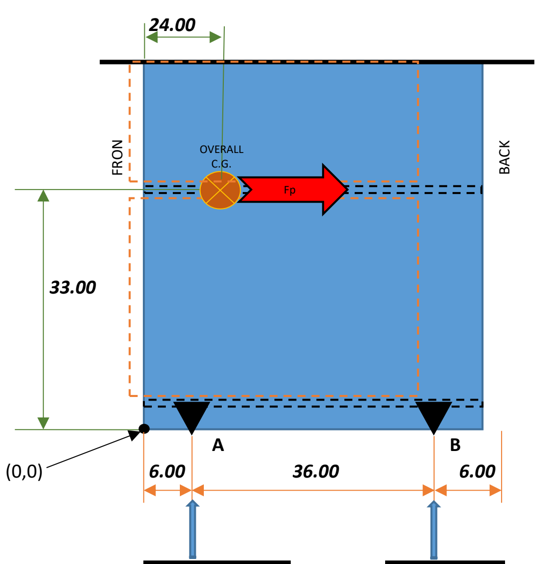
SIDE VIEW



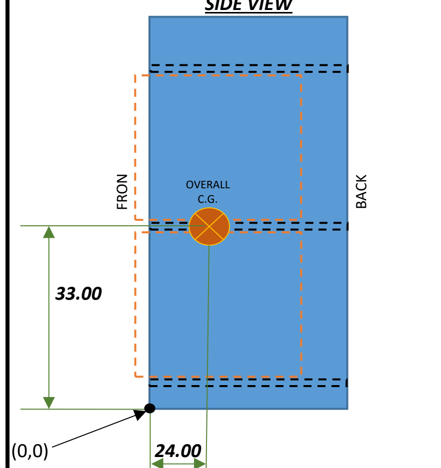
X_{cm} =	*24.00* IN
Y_{cm} =	*33.00* IN

$W_{p\ Shelves}$ =	*471 LBS*
$W_{p\ Component}$ =	*LBS*
F_p =	*281 LBS*

BASE CONNECTION FORCES



SIDE VIEW



@ FRONT	@ BACK
633 LBS	*633 LBS*
OR	OR
-139 LBS	*-139 LBS*

@ FRONT ANCHOR	@ REAR ANCHOR
317 LBS	317 LBS
OR	OR
-70 LBS	-70 LBS

THE ABOVE REACTIONS ARE PER SIDE USING THE FOLLOWING LRFD EQUATIONS	
(0.9-0.2SDS)*DL +/- Q_E	*Uplift (-)*
(0.503)*DL +/- Fp	*Uplift (-)*
(1.2+0.2SDS)*DL +/- *QE	*Compression (+)*
(1.597)*DL +/- Fp	*Compression (+)*

Project	MN 2033		
Date	8/12/2022		
By	KCS		
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THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCE(S) FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

DESCRIPTION >	***BBY Play Table***

Seismic Design Requirements for Non-Structural Components (ASCE 7-10, Chapter 13)

I_p = **1.0** (Section 13.1.3)

$$F_p = \frac{0.4 * a_p * S_{DS} * W_p}{(R_p / I_p)} [1 + 2 (z/h)] \quad \text{(EQN 13.3-1)}$$

$F_{p,max}$ = $1.6 * S_{DS} * I_p * W_p$ (EQN 13.3-2)

$F_{p,min}$ = $0.3 * S_{DS} * I_p * W_p$ (EQN 13.3-3)

a_p = **1.0** (Table 13.5-1)

R_p = **2.5** (Table 13.5-1)

z = **0** IN

h = **120** IN

Table 13.5-1 Coefficients for Architectural Components

Architectural Component	a_p[a]	R_p[b]
Interior nonstructural walls and partitions[b]		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever elements (Unbraced or braced to structural frame below its center of mass)		
Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys where laterally braced or supported by the structural frame	2.5	2.5
Cantilever elements (Braced to structural frame above its center of mass)		
Parapets	1.0	2.5
Chimneys	1.0	2.5
Exterior nonstructural walls[b]	1.0[b]	2.5
Exterior nonstructural wall elements and connections[b]		
Wall element	1.0	2.5
Body of wall panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1.0
Veneer		
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Penthouses (except where framed by an extension of the building frame)	2.5	3.5
Ceilings		
All	1.0	2.5
Cabinets		
Permanent floor-supported storage cabinets over 6 ft (1,829 mm) tall, including contents	1.0	2.5
Permanent floor-supported library shelving, book stacks, and bookshelves over 6 ft (1,829 mm) tall, including contents	1.0	2.5
Laboratory equipment	1.0	2.5
Access floors		
Special access floors (designed in accordance with Section 13.5.7.2)	1.0	2.5
All other	1.0	1.5
Appendages and ornamentations	2.5	2.5
Signs and billboards	2.5	3.0
Other rigid components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability materials and attachments	1.0	1.5
Other flexible components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5
Egress stairways not part of the building structure	1.0	2.5

[a]A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of a_p = 1 is for rigid components and rigidly attached components. The value of a_p = 2.5 is for flexible components and flexibly attached components.
[b]Where flexible diaphragms provide lateral support for concrete or masonry walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 12.11.2.

F_p = ***0.318 * W_p*** (EQN 13.3-1)

$F_{p,max}$ = ***3.18 * W_p*** (EQN 13.3-2)

$F_{p,min}$ = ***0.596 * W_p*** (EQN 13.3-3)

F_p = ***0.596 * W_p***

W_p = ***471.0*** LBS (Assembled wood cabinet + fixtures installed into display)

F_p = ***280.7*** LBS

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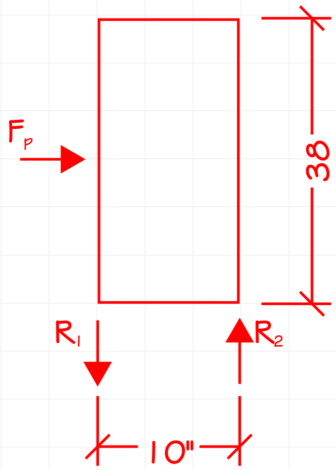
Tower Connection to Table



$F_p = 0.596W_p$ (see previous page)

$= (0.596)(200 \text{ lbs})$

$= 119.2 \text{ lbs}$

$OTM = (119.2 \text{ lbs})(38''/2) = 2264.8 \text{ lb-in}$

$$\text{Uplift} = \frac{2264.8 \text{ lb-in}}{10 \text{ in}} = 226.5 \text{ lbs}$$

$$\frac{226.5 \text{ lbs}}{3 \text{ screws}} = 75.5 \text{ lb/screw}$$

→ screw connection OK by inspection

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Design:	MN 2033 Base Connection	Date:	8/12/2022
Fastening point:			

Specifier's comments:

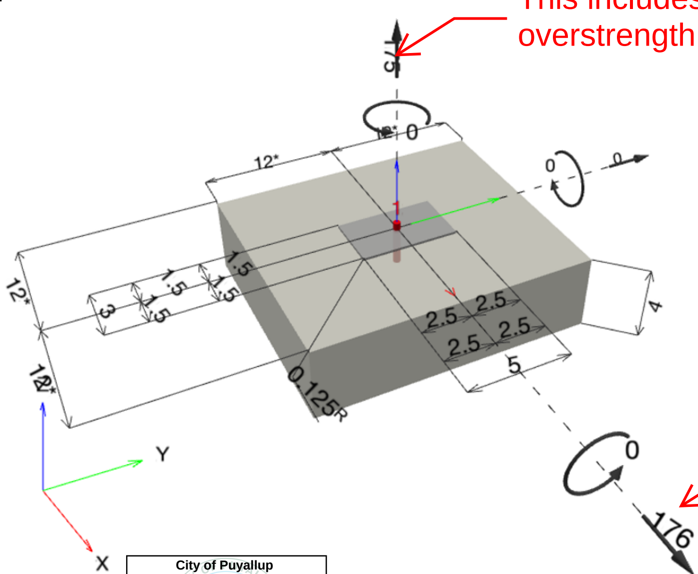
1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 3/8 (2 1/2)**
Item number:	418057 KH-EZ 3/8"x3"
Effective embedment depth:	$h_{ef,act}$ = 1.860 in., h_{nom} = 2.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	4/1/2022 | 12/1/2023
Proof:	Design Method ACI 318-14 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.125 in.
Anchor plate^R :	l_x x l_y x t = 3.000 in. x 5.000 in. x 0.125 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 4.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

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

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

This includes 2.5 overstrength factor



This includes 2.5 overstrength factor

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Address:		Specifier:	
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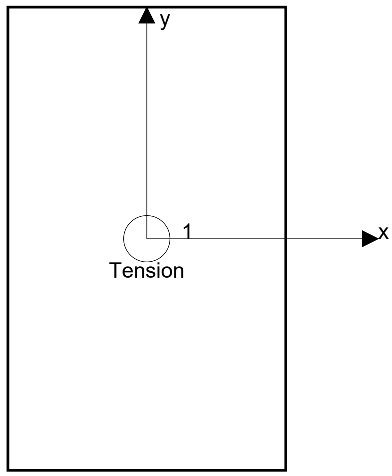
Design:	MN 2033 Base Connection	Date:	8/12/2022
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 175; V_x = 176; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	yes	17

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	175	176	176	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 175 [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

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	175	6,718	3	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	175	1,051	17	OK

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

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Fastening point:			

3.1 Steel Strength

N_{sa} = ESR value — refer to ICC-ES ESR-3027

$\phi\ N_{sa} \geq N_{ua}$ — ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.09	120,300

Calculations

N_{sa} [lb]
10,335

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
10,335	0.650	1.000	6,718	175

3.2 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-14 Eq. (17.4.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ — ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.860	12.000	1.000	2.920	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
31.14	31.14	1.000	1.000	2,156

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
2,156	0.650	0.750	1.000	1,051	175

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	176	1,866	10	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	176	1,509	12	OK
Concrete edge failure in direction x+**	176	2,700	7	OK

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

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-3027

ϕ $V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.09	120,300	0.600

Calculations

$V_{sa,eq}$ [lb]
3,110

Results

$V_{sa,eq}$ [lb]	ϕ $_{steel}$	$\phi_{nonductile}$	ϕ $V_{sa,eq}$ [lb]	V_{ua} [lb]
3,110	0.600	1.000	1,866	176

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4.2 Pryout Strength

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-14 Eq. (17.5.3.1a)

$\phi\ V_{cp} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1	1.860	12.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
2.920	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
31.14	31.14	1.000	1.000	2,156

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
2,156	0.700	1.000	1.000	1,509	176

4.3 Concrete edge failure in direction x+

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ ACI 318-14 Eq. (17.5.2.1a)

$\phi\ V_{cb} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\ c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
8.000	12.000	1.000	4.000	1.860

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.375	2,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
96.00	288.00	1.000	1.732	6,681

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
3,857	0.700	1.000	1.000	2,700	176

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.166	0.117	5/3	8	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 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.
- 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/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (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 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (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 17.2.3.4.3 (c) / Section 17.2.3.5.3 (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 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

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

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7 Installation data

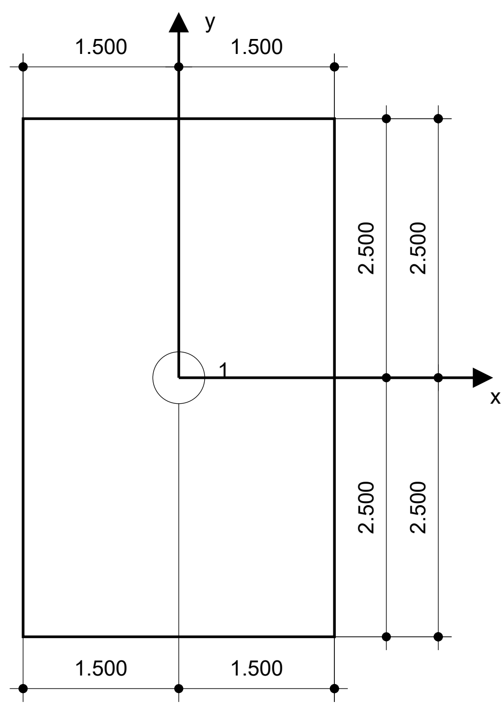
Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 3/8 (2 1/2)
Item number: 418057 KH-EZ 3/8"x3"
Maximum installation torque: 480 in.lb
Hole diameter in the base material: 0.375 in.
Hole depth in the base material: 2.750 in.
Minimum thickness of the base material: 4.000 in.

Profile: no profile
Hole diameter in the fixture: d_f = 0.500 in.
Plate thickness (input): 0.125 in.
Recommended plate thickness: not calculated
Drilling method: Hammer drilled
Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti KH-EZ screw anchor with 2.5 in embedment, 3/8 (2 1/2), Carbon steel, installation per ESR-3027

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	-0.000	12.000	12.000	12.000	12.000

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8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
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S-VPD.46

6'-7" HIGH SAMSUNG 3PC ENDCAP DSIPLAY

PRCTI20230074

Project: MN 2030
Date: 11/28/2022
By: BC
Sheet 1 of 3

THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCES FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

Updated: 03/01/21
By: SM

DESCRIPTION >	**BBY Samsung 3pc 4pc Modernspace**

PROJECT INFORMATION

		SITE NAME:	**TBD**
CITY:	**TBD**	SITE ADDRESS:	**TBD**
COUNTY:	**TBD**	LATITUDE:	**TBD**
STATE:	**TBD**	LONGITUDE:	**TBD**

SEISMIC LOAD ANAYLSIS

OCCUPANCY CATEGORY:		**II**	(Table 1.5-1)
SITE CLASS:		**D-default**	
SHORT PERIOD ACCELERATION:	S_s =	**2.484**	(From USGS Website or Figure 22-1 to 22-18)
1-SECOND PERIOD ACCELERATION:	S_1 =	**0.85**	(From USGS Website or Figure 22-1 to 22-18)
IMPORATANCE FACTOR:	I =	***1***	(Table 1.5-2)
SITE	F_a =	***1.2***	(Table 11.4-1)
COEFICIENTS:	F_v =	***1.7***	(Table 11.4-2)

Note: fixture is designed for:
Ss = 2.484
S1 = 0.85

Actual Seismic coefficients are:
Ss = 1.261
S1 = 0.435

NOTE: scroll down to input other seismic coefficients (ap, Rp, z and h)

RESPONSE SPECTRUM

S_{MS} =	***2.981***	$S_{MS} = F_a \times S_s$	(EQN 11.4-1)
S_{M1} =	***1.445***	$S_{M1} = F_v \times S_1$	(EQN 11.4-2)
S_{DS} =	***1.987***	$S_{DS} = (2/3)S_{MS}$	(EQN 11.4-3)
S_{D1} =	***0.963***	$S_{D1} = (2/3)S_{M1}$	(EQN 11.4-4)

Earthquake Design Criteria

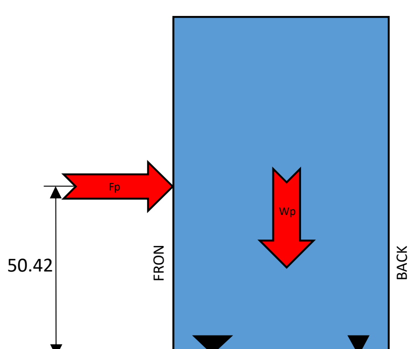
SEISMIC DESIGN CATEGORY

S_{DS} =	***D***	(Table 11.6-1)
S_{D1} =	***E***	(Table 11.6-2)
SDC FOR DESIGN =	***E***	

Seismic Weight

Self Weight of Fixture = **330** Lbs. (Fixture weight will be distributed to shelves)
Number of Shelves = **1**

	Shelf	Weight of Contents (LBS)	Shelf Height Above Base (IN)	Total Weight	F_p Coefficient	Seismic Force	OTM at Base (LBS-IN)
COMPONENT MAKEUP	***SHELF #1***	***330***	47.40	***330***	***59.60%***	***197***	9322.6
	SHELF #2	***0***	0.00	***0***	***59.60%***	***0***	0.0
	SHELF #3	***0***	0.00	***0***	***59.60%***	***0***	0.0
	SHELF #4	***0***	0.00	***0***	***59.60%***	***0***	0.0
	CONTENT #1	60	67.03	***60***	***59.60%***	***36***	2396.8
	CONTENT #2	0	0.00	***0***	***59.60%***	***0***	0.0
	CONTENT #3	0	0.00	***0***	***59.60%***	***0***	0.0
	CONTENT #4	0	0.00	***0***	***59.60%***	***0***	0.0
				390		***232***	***11719.4*** LBS-IN
				W_p		**F_p**	***976.6*** LBS-FT



City of Puyallup
Development & Permitting Services
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Fire | Traffic

PRCTI20230074

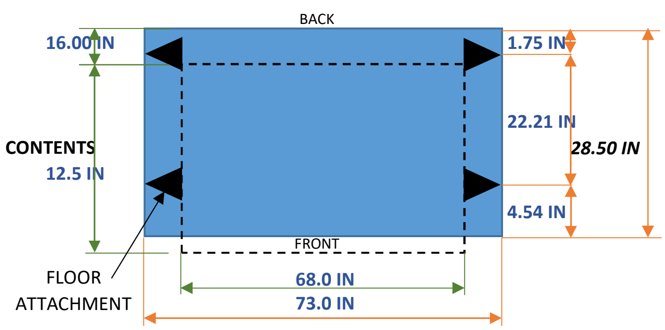
Project	MN 2030
Date	11/28/2022
By	BC
Sheet	2 of 3

THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCES FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

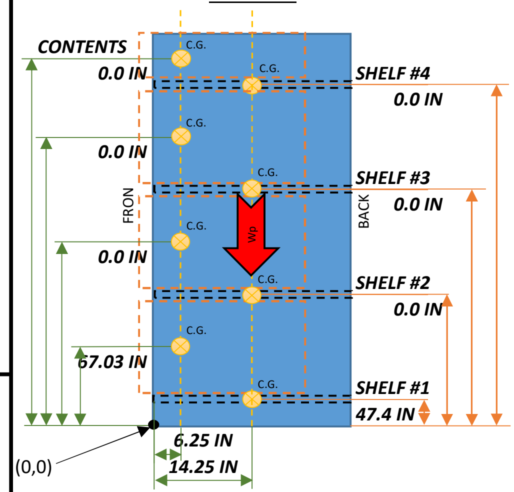
DESCRIPTION >	*BBY Samsung 3pc 4pc Modernspace*

THIS IS AN EXAMPLE LAYOUT ONLY

PLAN VIEW



SIDE VIEW

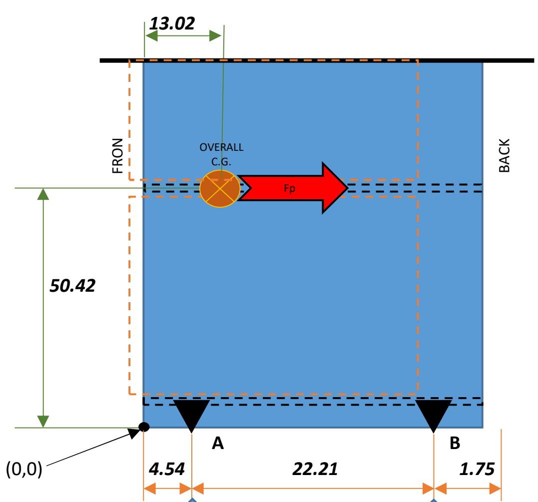


$W_{p\ Shelves}$ =	*330 LBS*
$W_{p\ Component}$ =	*60 LBS*
F_p =	*232 LBS*

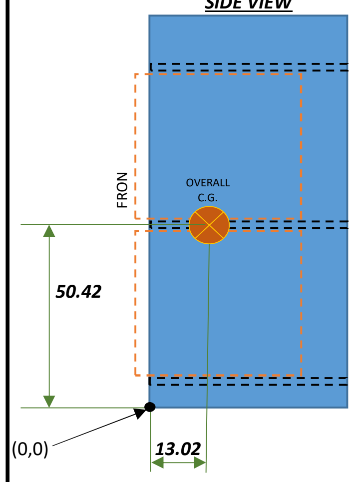
X_{cm} = ***13.02*** IN

Y_{cm} = ***50.42*** IN

BASE CONNECTION FORCES



SIDE VIEW



@ FRONT	@ BACK
913 LBS	*766 LBS*
OR	OR
-407 LBS	*-453 LBS*

@ FRONT ANCHOR	@ REAR ANCHOR
456 LBS	383 LBS
OR	OR
-102 LBS	-113 LBS

THE ABOVE REACTIONS ARE **PER SIDE** USING THE FOLLOWING LRFD EQUATIONS

$(0.9\text{-}0.2SDS)*DL +/- Q_E$	*Uplift (-)*
$(0.503)*DL +/- Fp$	*Uplift (-)*
$(1.2+0.2SDS)*DL +/- *QE$	*Compression (+)*
$(1.597)*DL +/- Fp$	*Compression (+)*

THE ABOVE REACTIONS ARE **PER ANCHOR BRACKET**, CONSIDERING **4** ANCHOR BRACKETS EACH SIDE

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

PRCTI20230074

Project	MN 2030
Date	11/28/2022
By	BC
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THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCES FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

DESCRIPTION >	***BBY Samsung 3pc 4pc Modernspace***

Seismic Design Requirements for Non-Structural Components (ASCE 7-16, Chapter 13)

I_p = **1.0** (Section 13.1.3)

$$F_p = \frac{0.4 * a_p * S_{DS} * W_p}{(R_p / I_p)} [1 + 2 (z/h)]$$ (EQN 13.3-1)

$F_{p,max}$ = $1.6 * S_{DS} * I_p * W_p$ (EQN 13.3-2)

$F_{p,min}$ = $0.3 * S_{DS} * I_p * W_p$ (EQN 13.3-3)

a_p = **1.0** (Table 13.5-1)

R_p = **2.5** (Table 13.5-1)

z = **0** IN

h = **79** IN

Table 13.5-1 Coefficients for Architectural Components

Architectural Component	a_p[a]	R_p	Ω_0[b]
Interior nonstructural walls and partitions[c]			
Plain (unreinforced) masonry walls	1	1½	1½
All other walls and partitions	1	2½	2
Cantilever elements (unbraced or braced to structural frame below its center of mass)			
Parapets and cantilever interior nonstructural walls	2½	2½	2
Chimneys where laterally braced or supported by the structural frame	2½	2½	2
Cantilever elements (braced to structural frame above its center of mass)			
Parapets	1	2½	2
Chimneys	1	2½	2
Exterior nonstructural walls[c]	1[b]	2½	2
Exterior nonstructural wall elements and connections[b]			
Wall element	1	2½	NA
Body of wall panel connections	1	2½	NA
Fasteners of the connecting system	1¼	1	1
Veneer			
Limited deformability elements and attachments	1	2½	2
Low-deformability elements and attachments	1	1½	2
Penthouses (except where framed by an extension of the building frame)	2½	3½	2
Ceilings			
All	1	2½	2
Cabinets			
Permanent floor-supported storage cabinets more than 6 ft (1,829 mm) tall, including contents	1	2½	2
Permanent floor-supported library shelving, book stacks, and bookshelves more than 6 ft (1,829 mm) tall, including contents	1	2½	2
Laboratory equipment	1	2½	2
Access floors			
Special access floors (designed in accordance with Section 13.5.7.2)	1	2½	2
All other	1	1½	1½
Appendages and ornamentations	2½	2½	2
Signs and Billboards	2½	3	2
Other rigid components			
High-deformability elements and attachments	1	3½	2
Limited-deformability elements and attachments	1	2½	2
Low-deformability materials and attachments	1	1½	1½
Other flexible components			
High-deformability elements and attachments	2½	3½	2½
Limited-deformability elements and attachments	2½	2½	2½
Low-deformability materials and attachments	2½	1½	1½
Egress stairways not part of the building seismic force-resisting system	1	2½	2
Egress stairs and ramp fasteners and attachments	2½	2½	2½

F_p = ***0.318 * W_p*** (EQN 13.3-1)

$F_{p,max}$ = ***3.18 * W_p*** (EQN 13.3-2)

$F_{p,min}$ = ***0.596 * W_p*** (EQN 13.3-3)

F_p = ***0.596 * W_p***

W_p = ***390.0*** LBS (Assembled fixture + attachments)

F_p = ***232.4*** LBS

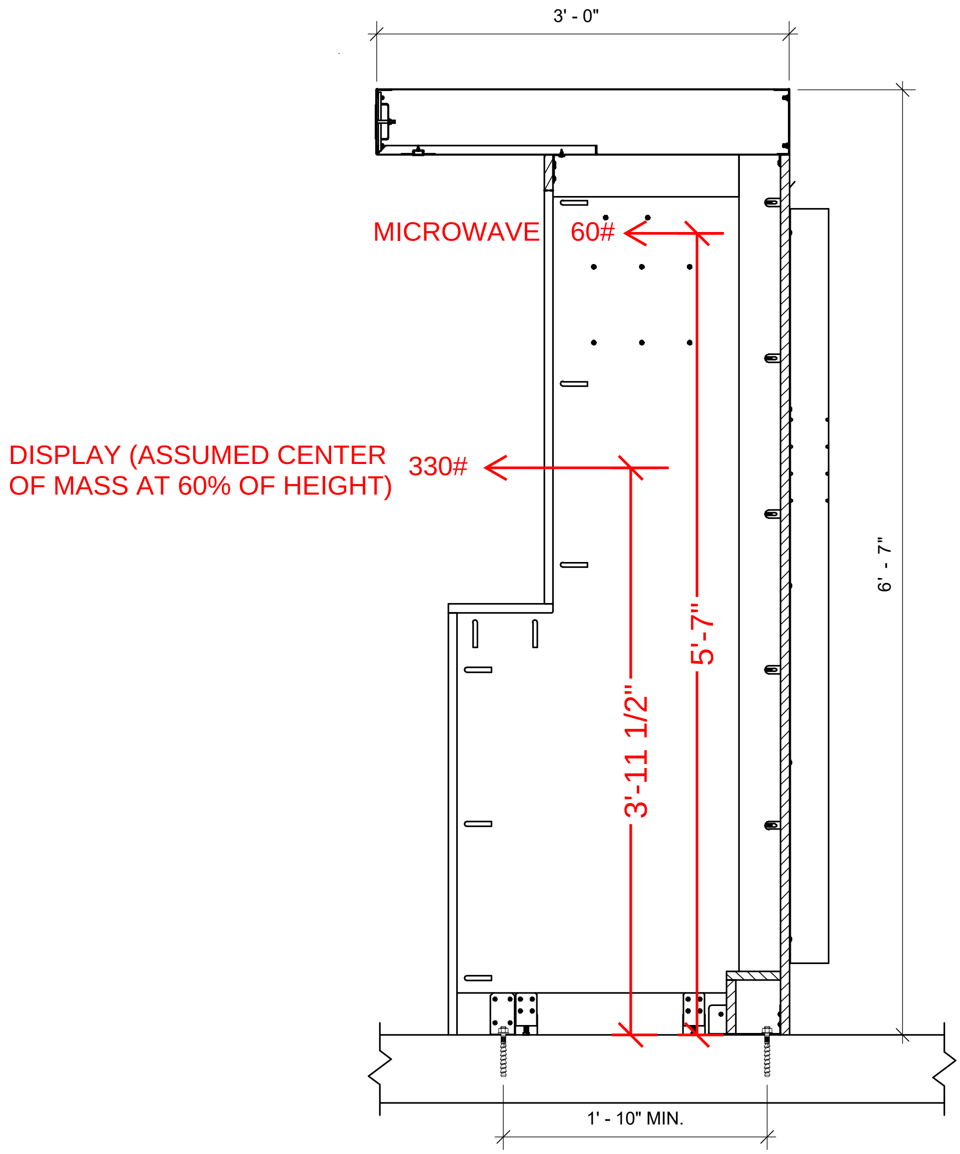
K740

Project____________________

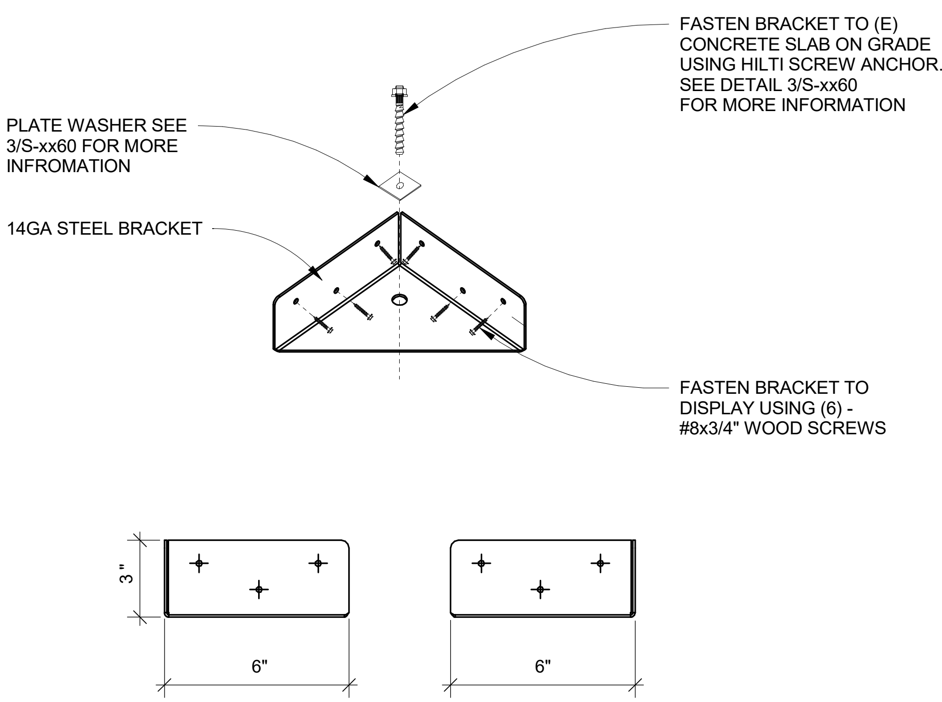
Date ____________________

By ____________________

Sheet ________ of ________



CORNER BRACKET DETAIL



Check connection to display:
V_{ASD} = (113#)(0.7) / 6 screws = 14#/screw
V_{allow} = 52#/screw
V_{ASD}/V_{allow} = 14# / 52# = 0.27 → OK

(see page XX)

Check anchorage to slab:
T_{LRFD} = (2.0)(113#) = 226#
V_{LRFD} = (2.0)(232# / 8) = 58#
Interaction = 22% (concrete breakout)

(see page XX for Hilti Profis Anchor Calculation)

this calculation is based on the base bracket anchorage forces, which are higher than the anchorage forces for the corner bracket

K742

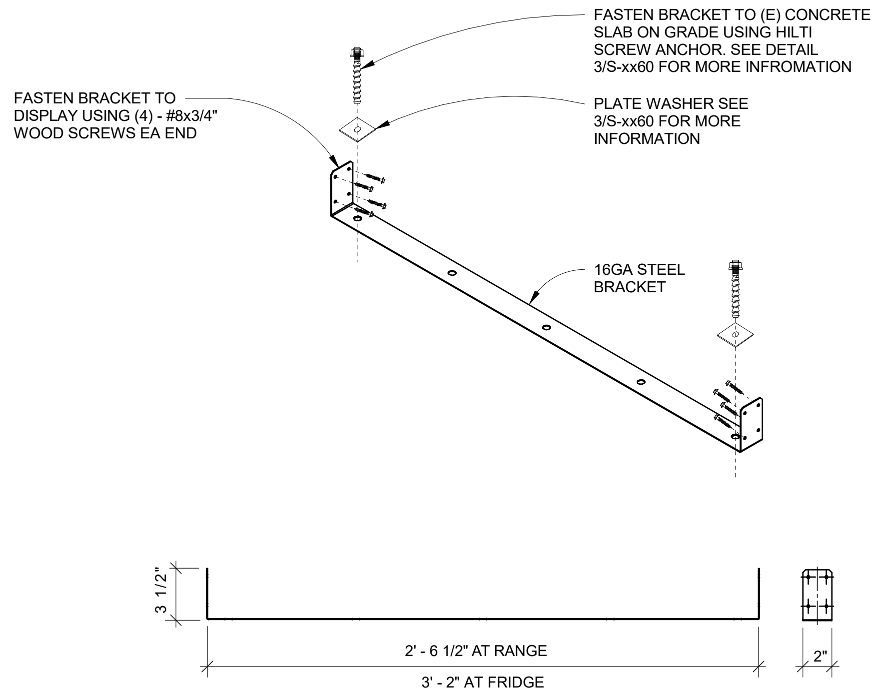
Project

Date

By

Sheet of

BASE BRACKET DETAIL



Check connection to display:

V_{ASD} = (113#)(0.7) / 4 screws = 20#/screw

V_{allow} = 52#/screw

V_{ASD}/V_{allow} = 20# / 52# = 0.39 → OK

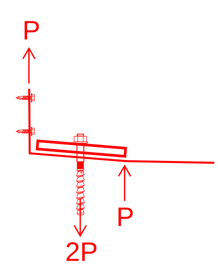
(see page XX)

Check strap capacity:

T_{LRFD} = 113#

ΦT_n = (0.9)(33 ksi)(0.060")(2") = 3,564#

$T_{LRFD}/\Phi T_n$ = 113# / 3,564# = 0.03 → OK



Check washer capacity

T_{LRFD} = 113#

M_u = (113#)(1") = 113 lb-in

ΦM_n = (0.9)(33 ksi)(2")(0.125")^2/4 = 232 lb-in

$M_u/\Phi M_n$ = 113# / 232# = 0.49 → OK

Check anchorage to slab:

T_{LRFD} = (2.0)(113#)(2) = 452#

V_{LRFD} = (2.0)(232# / 8) = 58#

Interaction = 22% (concrete breakout)

(see page XX for Hilti Profis Anchor Calculation)

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

PRCTI20230074

5/8" / No Pilot Hole / ASTM											
Sample Board Number	Initial Weights (grams)	Density (lb/ft^3)	Screw Penetration Depth	Pilot Hole?	Conditioning Room	Maximum Load (lbf) Screw Torque Range (lbf-in)			Weight (grams)		MC (%)
						30 -- 37	35 -- 47	54 -- 62	Wet	OD	
16	339.4	47.9	5/8 inch	No	ASTM	212.8	228.9	62.9	352.77	313.08	12.68%
19	335.7	47.4	5/8 inch	No	ASTM	244.6	246.1	58.5	347.34	308.83	12.47%
22	338.9	47.8	5/8 inch	No	ASTM	255.5	243.1	54.9	351.60	312.51	12.51%
23	338.1	47.7	5/8 inch	No	ASTM	231.6	253.2	52.4	350.34	311.49	12.47%
24	338.0	47.7	5/8 inch	No	ASTM	209.9	244.6	46.6	350.20	311.76	12.33%
41	342.9	48.4	5/8 inch	No	ASTM	236.2	276.8	35.7	355.31	318.86	11.43%
47	331.3	46.7	5/8 inch	No	ASTM	184.6	257.8	54.5	342.46	306.26	11.82%
48	327.9	46.3	5/8 inch	No	ASTM	194.7	264.2	49.9	340.04	304.20	11.78%
49	333.0	47.0	5/8 inch	No	ASTM	238.1	233.5	46.6	344.03	305.17	12.73%
58	337.5	47.6	5/8 inch	No	ASTM	246.3	231.2	61.0	350.97	311.96	12.50%
82	342.8	48.4	5/8 inch	No	ASTM	207.0	237.9	59.1	355.13	315.30	12.63%
84	341.3	48.2	5/8 inch	No	ASTM	226.2	232.9	46.8	353.14	314.42	12.31%
89	331.4	46.8	5/8 inch	No	ASTM	220.2	249.8	51.4	341.16	301.51	13.15%
Average		**47.5**				**223.7**	**246.2**	**52.3**			**12.4%**

Placed in oven on: Tuesday, June 30th @ 9:00am

Take out of oven: Wednesday, July 1st @ 9:00am

Per Oregon Wood Innovation Center, screws in 5/8" MDF have a capacity of 246# when properly installed. Conservatively, use 52# per screw which is the typical capacity of an over-torqued screw.

5/8" / Pil[illegible]											
Sample Board Number	Initial Weights (grams)	Density (lb/ft^3)	Screw Penetration Depth	Pilot Hole?	Co[illegible]	[illegible]			[illegible]ght [illegible]ns)		MC (%)
										OD	
11	341.6	48.2	5/8 inch	Yes	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	314.60	12.23%
20	339.6	47.9	5/8 inch	Yes	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	313.02	12.64%
26	334.0	47.1	5/8 inch	Yes	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	306.25	12.85%
29	335.6	47.4	5/8 inch	Yes	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	311.38	11.79%
31	341.0	48.1	5/8 inch	Yes	ASTM	262.1	289.1	300.0	354.00	316.79	11.75%
34	334.2	47.2	5/8 inch	Yes	ASTM	211.6	226.6	64.5	346.00	307.79	12.41%
38	333.9	47.1	5/8 inch	Yes	ASTM	171.7	280.1	47.2	346.22	308.86	12.10%
42	335.1	47.3	5/8 inch	Yes	ASTM	207.4	237.7	43.9	347.67	309.90	12.19%
51	335.1	47.3	5/8 inch	Yes	ASTM	201.8	227.5	57.7	347.81	308.48	12.75%
62	335.2	47.3	5/8 inch	Yes	ASTM	202.4	205.3	41.6	347.39	308.95	12.44%
81	342.9	48.4	5/8 inch	Yes	ASTM	242.1	253.6	45.1	354.62	313.23	13.21%
83	332.9	47.0	5/8 inch	Yes	ASTM	235.0	239.6	72.3	342.73	302.01	13.48%
85	343.1	48.4	5/8 inch	Yes	ASTM	157.9	258.4	53.7	354.70	313.74	13.06%
Average		**47.6**				**213.4**	**247.4**	**72.7**			**12.5%**

Placed in oven on: Tuesday, June 30th @ 8:30am

Take out of oven: Wednesday, July 1st @ 9:00am

Hilti PROFIS Engineering 3.0.81

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Design:	Front Connection	Date:	12/12/2022
Fastening point:			

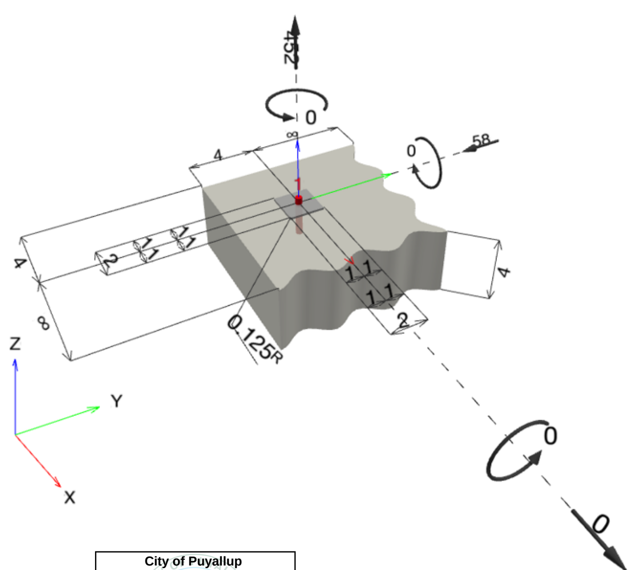
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 3/8 (2 1/2)**
Item number:	418057 KH-EZ 3/8"x3"
Effective embedment depth:	$h_{ef,act}$ = 1.860 in., h_{nom} = 2.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	4/1/2022 | 12/1/2023
Proof:	Design Method ACI 318-14 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.125 in.
Anchor plate^R :	l_x x l_y x t = 2.000 in. x 2.000 in. x 0.125 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 4.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

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

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



PRCTI20230074

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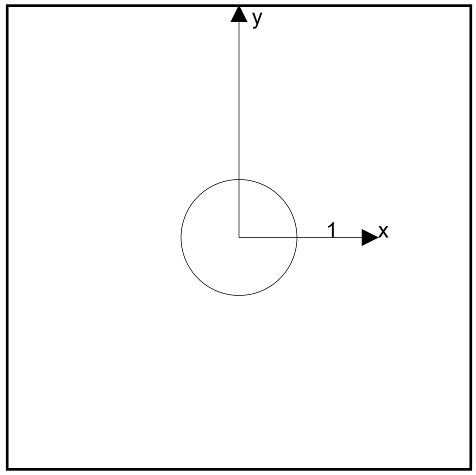
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 452; V_x = 0; V_y = -58; M_x = 0; M_y = 0; M_z = 0;	yes	44

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	452	58	0	-58

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 452 [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

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	452	6,718	7	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	452	1,051	44	OK

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

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3.1 Steel Strength

N_{sa} = ESR value — refer to ICC-ES ESR-3027

$\phi\ N_{sa} \geq N_{ua}$ — ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.09	120,300

Calculations

N_{sa} [lb]
10,335

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
10,335	0.650	1.000	6,718	452

3.2 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-14 Eq. (17.4.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ — ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.860	4.000	1.000	2.920	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
31.14	31.14	1.000	1.000	2,156

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
2,156	0.650	0.750	1.000	1,051	452

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	58	1,866	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	58	1,509	4	OK
Concrete edge failure in direction y-**	58	1,012	6	OK

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

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-3027

ϕ $V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.09	120,300	0.600

Calculations

$V_{sa,eq}$ [lb]
3,110

Results

$V_{sa,eq}$ [lb]	ϕ $_{steel}$	$\phi_{nonductile}$	ϕ $V_{sa,eq}$ [lb]	V_{ua} [lb]
3,110	0.600	1.000	1,866	58

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4.2 Pryout Strength

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ ACI 318-14 Eq. (17.5.3.1a)

$\phi\ V_{cp} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1	1.860	4.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
2.920	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
31.14	31.14	1.000	1.000	2,156

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
2,156	0.700	1.000	1.000	1,509	58

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4.3 Concrete edge failure in direction y-

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-14 Eq. (17.5.2.1a)

$\phi\ V_{cb} \geq V_{ua}$ — ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-14 Eq. (17.5.2.8)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\ c_{a1}^{1.5}$ — ACI 318-14 Eq. (17.5.2.2a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
4.000	4.000	1.000	4.000	1.860

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.375	2,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
40.00	72.00	0.900	1.225	2,362

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
1,446	0.700	1.000	1.000	1,012	58

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.430	0.057	5/3	26	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 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.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- Annular gap must be removed by e.g. filling the gaps with mortar of sufficient compressive strength.
- 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 318-14, Chapter 17, Section 17.2.3.4.3 (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 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (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 17.2.3.4.3 (c) / Section 17.2.3.5.3 (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 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

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7 Installation data

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 3/8 (2 1/2)

Item number: 418057 KH-EZ 3/8"x3"

Maximum installation torque: 480 in.lb

Hole diameter in the base material: 0.375 in.

Hole depth in the base material: 2.750 in.

Minimum thickness of the base material: 4.000 in.

Profile: no profile

Hole diameter in the fixture: d_f = 0.500 in.

Plate thickness (input): 0.125 in.

Recommended plate thickness: not calculated

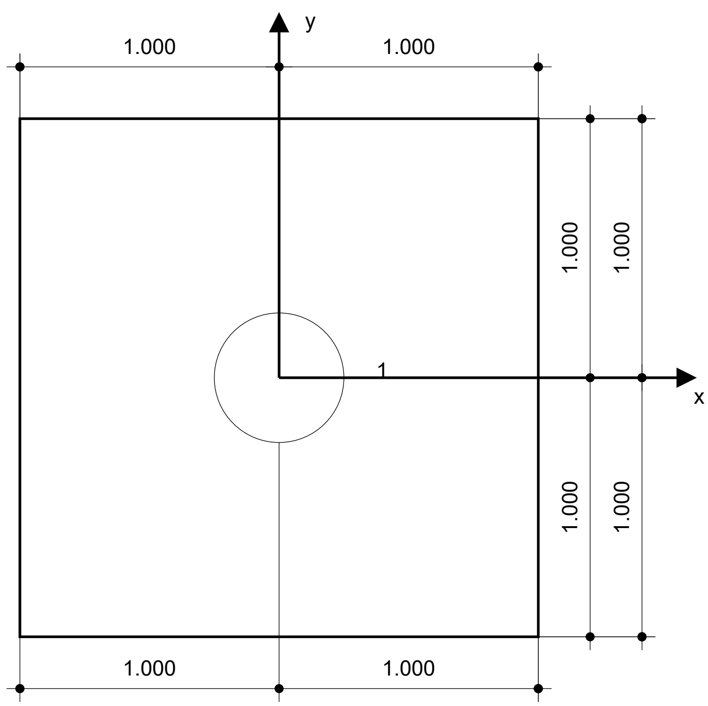
Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti KH-EZ screw anchor with 2.5 in embedment, 3/8 (2 1/2), Carbon steel, installation per ESR-3027

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	4.000	-	4.000	-

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Fastening point:			

8 Remarks; Your Cooperation Duties

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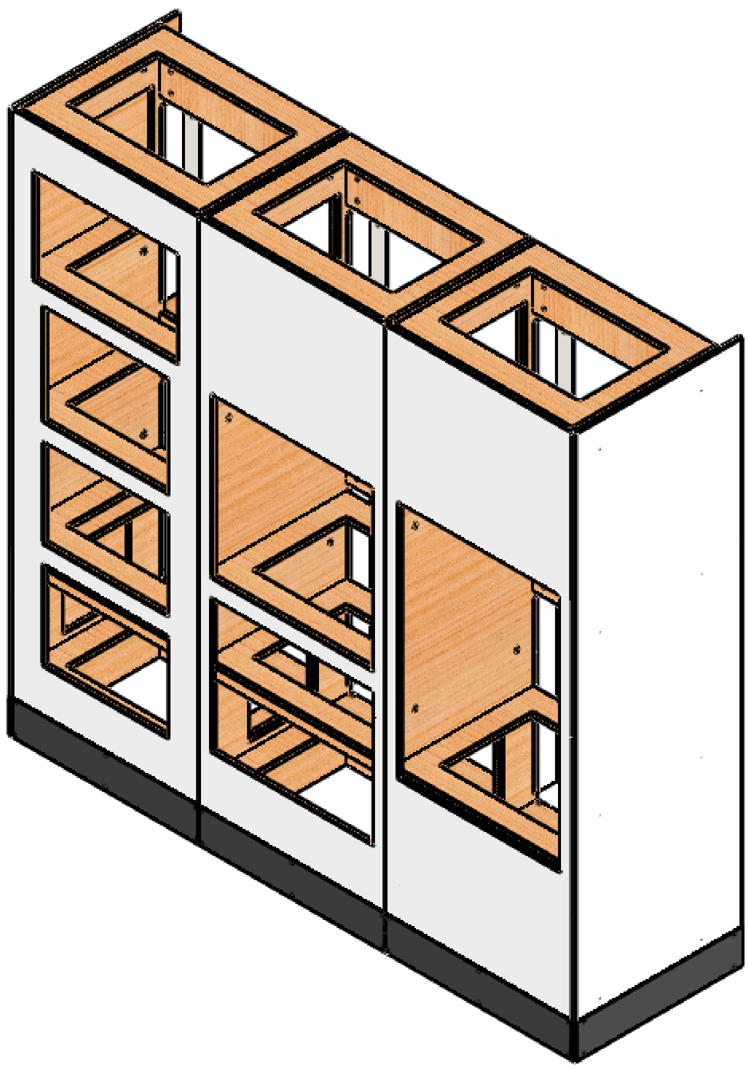
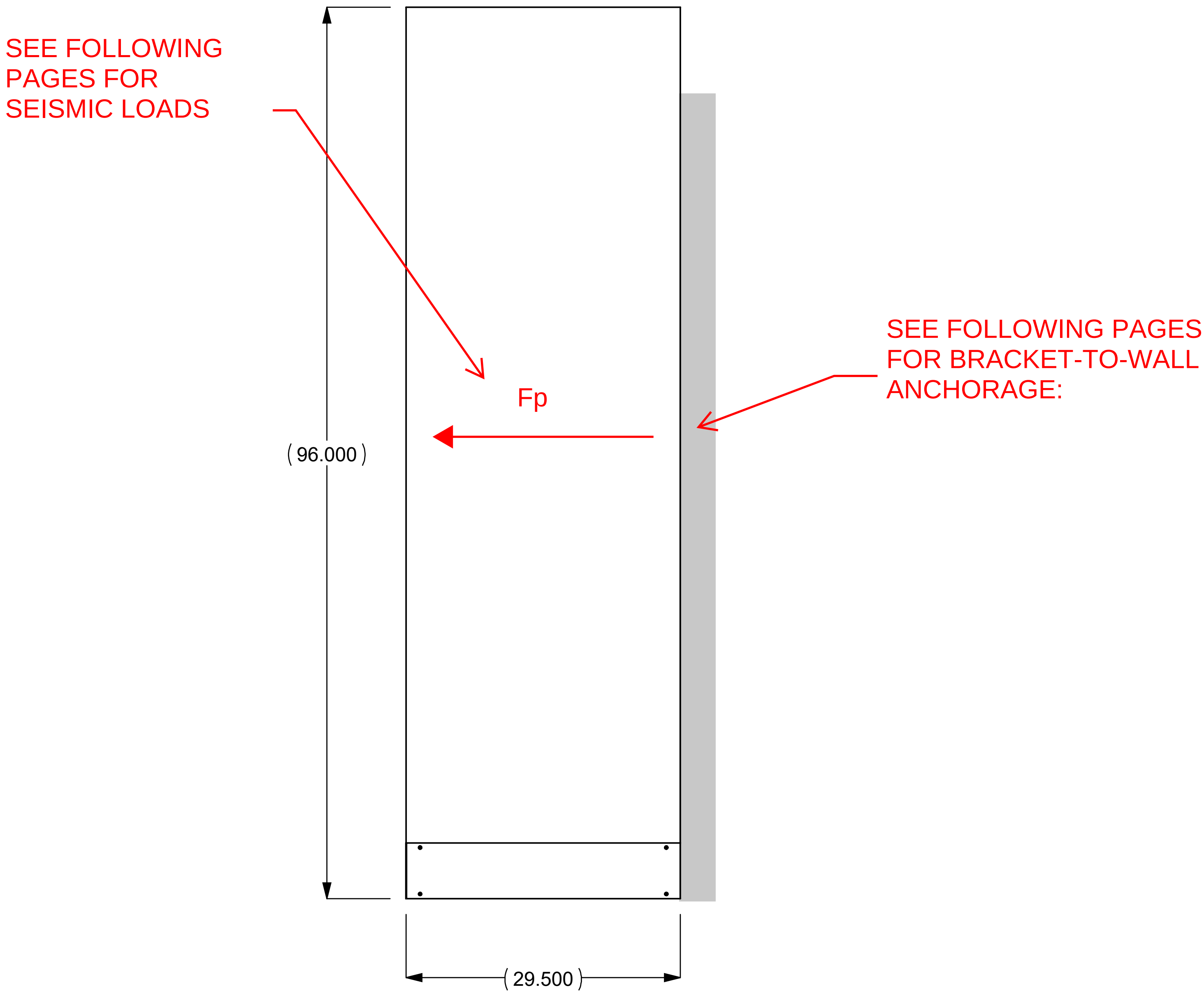
SECTION L

PAC SALES DISPLAYS



S-DOC.1 A (DOD 2.1W)

DOUBLE OVEN WALL DISPLAY


SEE FOLLOWING
PAGES FOR
SEISMIC LOADS
SEE FOLLOWING PAGES
FOR BRACKET-TO-WALL
ANCHORAGE:
Fp
(96.000)
(29.500)

Dave Buchanan

From: Le, Adam <Adam.Le@bestbuy.com>
Sent: Thursday, June 11, 2020 12:52 PM
To: Dave Buchanan
Subject: RE: [CAUTION! EXTERNAL] oven cabinet

Correct. To be precise, the overall weight is:

Cabinet & 1 front panel = ~120 lbs.
Oven Shelf x 2 = 15 lbs
Anchor Hardware = 15 lbs
Oven x 2 = 280

Adam Le
Mechanical Engineer
Store Design Team

Best Buy Co., Inc.
7601 Penn Avenue South, Richfield, MN 55423, USA | BestBuy.com
T: +1 (612) 291-5980 **M:** +1 (952) 406-2049 | adam.le@bestbuy.com

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From: Dave Buchanan <dbuchanan@asteng.com>
Sent: Thursday, June 11, 2020 12:33 PM
To: Le, Adam <Adam.Le@bestbuy.com>
Subject: RE: [CAUTION! EXTERNAL] oven cabinet

Thanks, Adam. Just to confirm, is this (2) 140 pound ovens plus a 150 pound self-wt for cabinet (total mass = 430 pounds)?

DAVID C. BUCHANAN
P.E., S.E., P.ENG. PRINCIPAL

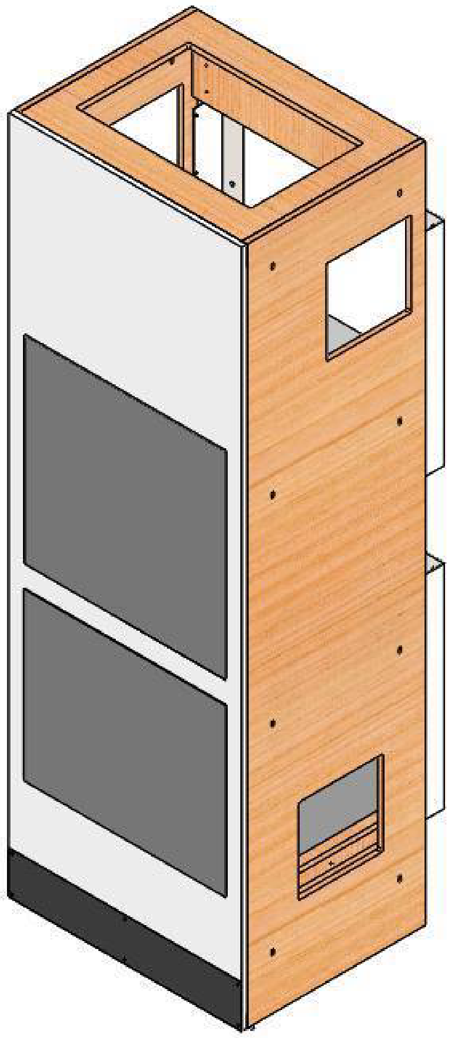
O 952.854.9302 x202
DBUCHANAN@ASTENG.COM

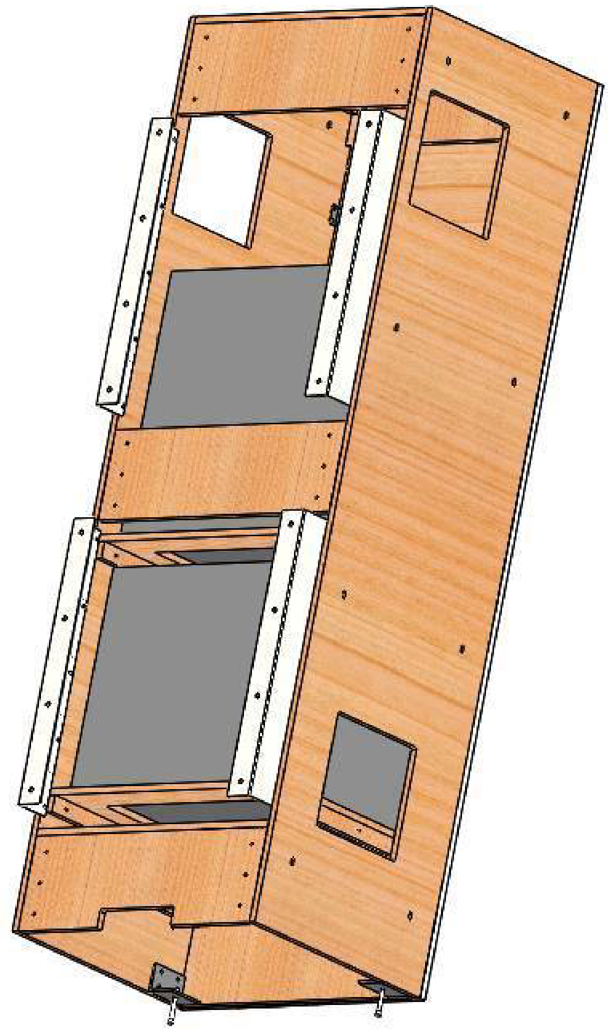
7301 OHMS LANE – SUITE 215
EDINA, MN 55439

From: Le, Adam <Adam.Le@bestbuy.com>
Sent: Thursday, June 11, 2020 12:22 PM
To: Dave Buchanan <dbuchanan@asteng.com>
Subject: RE: [CAUTION! EXTERNAL] oven cabinet

Hi Dave,

As requested, I have attached renderings of two ovens in a single cabinet with the wall anchors attached. Let me know if you need anything else.





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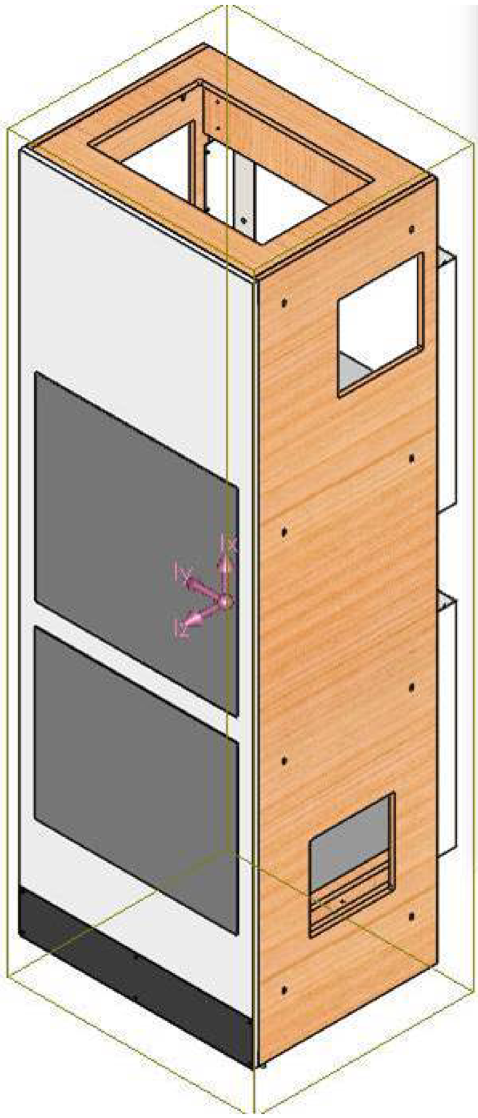
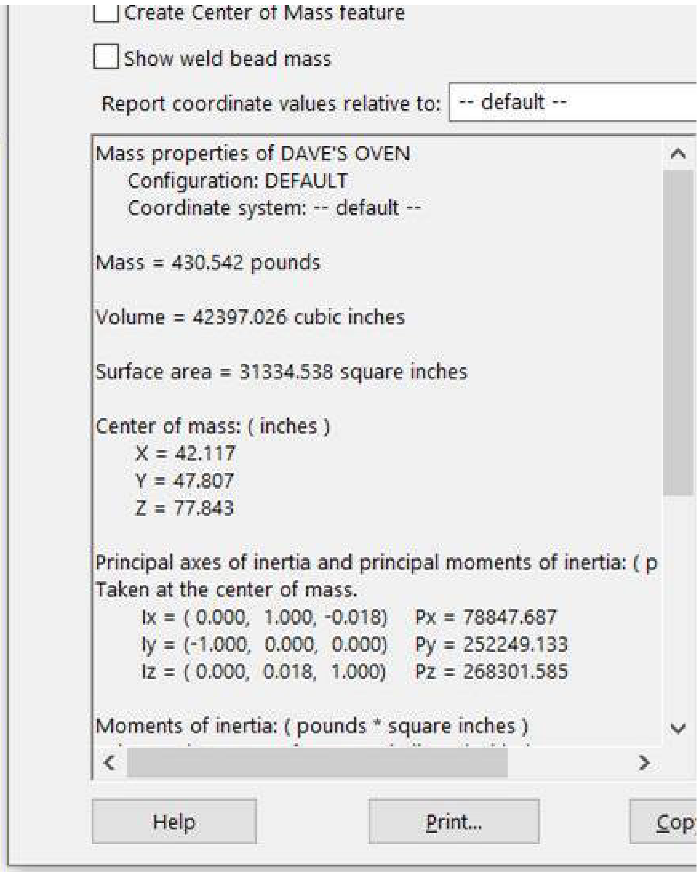
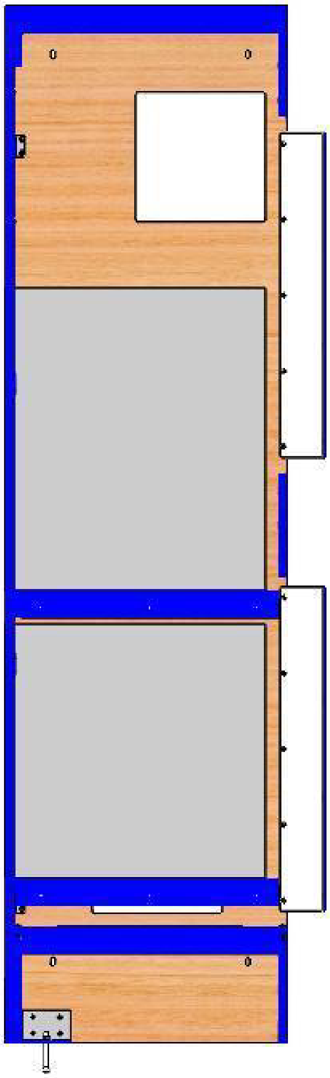
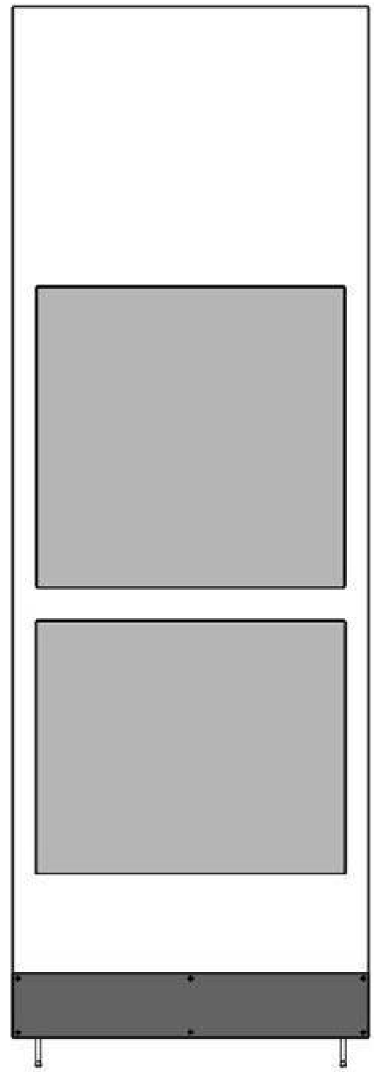


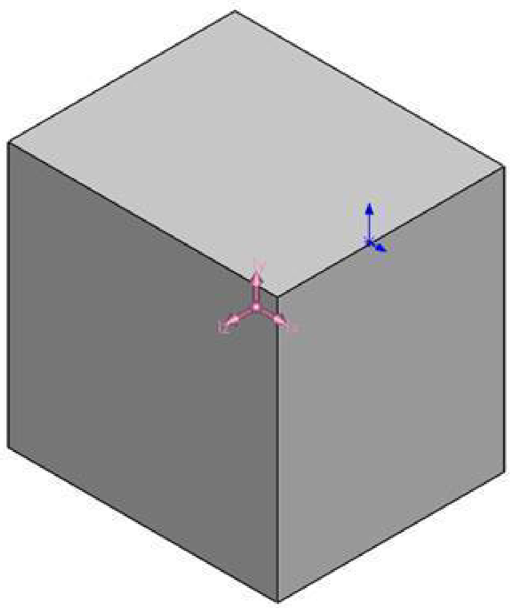
Ix
Iy
Iz

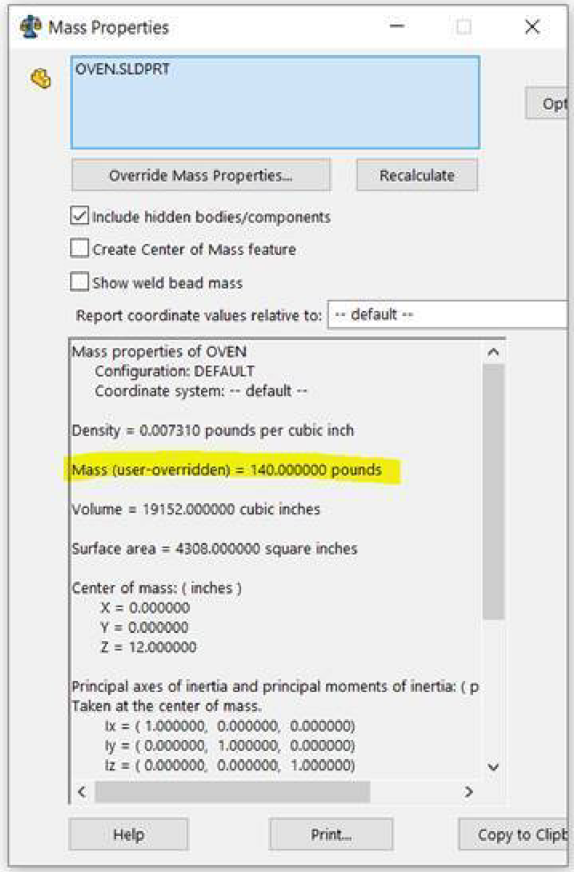


Create Center of Mass feature
Show weld bead mass
Report coordinate values relative to: -- default --
Mass properties of DAVE'S OVEN
Configuration: DEFAULT
Coordinate system: -- default --
Mass = 430.542 pounds
Volume = 42397.026 cubic inches
Surface area = 31334.538 square inches
Center of mass: (inches)
X = 42.117
Y = 47.807
Z = 77.843
Principal axes of inertia and principal moments of inertia: (p
Taken at the center of mass.
Ix = (0.000, 1.000, -0.018) Px = 78847.687
Iy = (-1.000, 0.000, 0.000) Py = 252249.133
Iz = (0.000, 0.018, 1.000) Pz = 268301.585
Moments of inertia: (pounds * square inches)
Help
Print...
Cop









Adam Le
Mechanical Engineer
Store Design Team

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T: +1 (612) 291-5980 **M:** +1 (952) 406-2049 | adam.le@bestbuy.com

---

From: Dave Buchanan <dbuchanan@asteng.com>
Sent: Thursday, June 11, 2020 11:33 AM
To: Le, Adam <Adam.Le@bestbuy.com>
Subject: [CAUTION! EXTERNAL] oven cabinet

Hi Adam –

Will you call me when you have a few minutes?

Thanks!

Dave

Project	MN 1806		
Date			
By	DB		
Sheet	1	of	3

THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCE(S) FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

Updated: 09/03/19
By: EAL

DESCRIPTION >	Wood double oven cabinets

PROJECT INFORMATION

		SITE NAME:	TBD
CITY:	TBD	SITE ADDRESS:	TBD
COUNTY:	TBD	LATITUDE:	TBD
STATE:	TBD	LONGITUDE:	TBD

SEISMIC LOAD ANAYLSIS

OCCUPANCY CATEGORY:		II	(Table 1-1)
SITE CLASS:		D-default	
SHORT PERIOD ACCELERATION:	S_s =	2.5	(From USGS Website or Figure 22-1 to 22-18)
-SECOND PERIOD ACCELERATION:	S_1 =	0.85	(From USGS Website or Figure 22-1 to 22-18)
IMPORATANCE FACTOR:	I =	*1*	(Table 1.5-2)
SITE	F_a =	*1.2*	(Table 11.4-1)
COEFICIENTS:	F_v =	*1.7*	(Table 11.4-2)

Note: fixture is designed for:
Ss = 2.5
S1 = 0.85

Actual Seismic coefficients are:
Ss = 1.261
S1 = 0.435

NOTE: scroll down to input other seismic coefficients (ap, Rp, z and h)

RESPONSE SPECTRUM

S_{MS} =	*3.000*	$S_{MS} = F_a \times S_s$	(EQN 11.4-1)
S_{M1} =	*1.445*	$S_{M1} = F_v \times S_1$	(EQN 11.4-2)
S_{DS} =	*2.000*	$S_{DS} = (2/3)S_{MS}$	(EQN 11.4-3)
S_{D1} =	*0.963*	$S_{D1} = (2/3)S_{M1}$	(EQN 11.4-4)

Earthquake Design Criteria

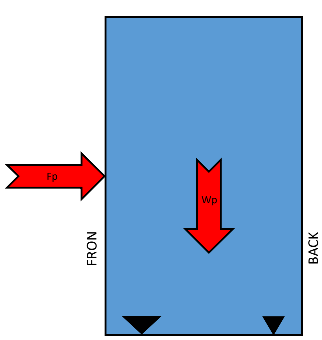
SEISMIC DESIGN CATEGORY

S_{DS} =	*D*	(Table 11.6-1)
S_{D1} =	*E*	(Table 11.6-2)
SDC FOR DESIGN =	*E*	

Seismic Weight

Self Weight of Fixture = 150 LBS (Fixture weight will be distributed to shelves)
Number of Shelves = 1

	Shelf	Weight of Contents (LBS)	Shelf Height Above Base (IN)	Total Weight	F_p Coefficient	Seismic Force	OTM at Base (LBS-IN)	
COMPONENT MAKEUP	*SHELF #1*	*150*	48.00	*150*	*60.00%*	*90*	4320.0	
	SHELF #2	*0*	0.00	*0*	*60.00%*	*0*	0.0	
	SHELF #3	*0*	0.00	*0*	*60.00%*	*0*	0.0	
	SHELF #4	*0*	0.00	*0*	*60.00%*	*0*	0.0	
	CONTENT #1	140	48.00	*140*	*60.00%*	*84*	4032.0	
	CONTENT #2	140	48.00	*140*	*60.00%*	*84*	4032.0	
	CONTENT #3	0	0.00	*0*	*60.00%*	*0*	0.0	
	CONTENT #4	0	0.00	*0*	*60.00%*	*0*	0.0	
				430		*258*	*12384.0*	LBS-IN
				W_p		**F_p**	*1032.0*	LBS-FT



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PRCTI20230074

Project	MN 1806
Date	
By	DB
Sheet	2 of 3

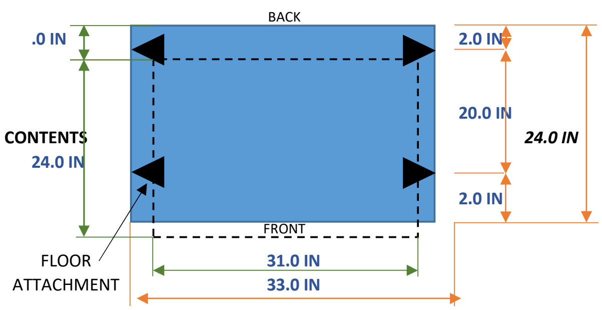
THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCE(S) FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

0

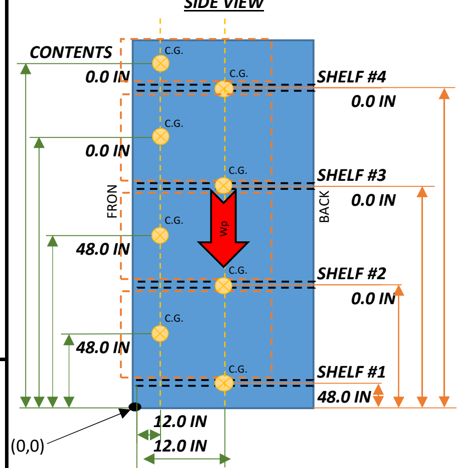
DESCRIPTION >	*Wood double oven cabinets*

THIS IS AN EXAMPLE LAYOUT ONLY

PLAN VIEW



SIDE VIEW

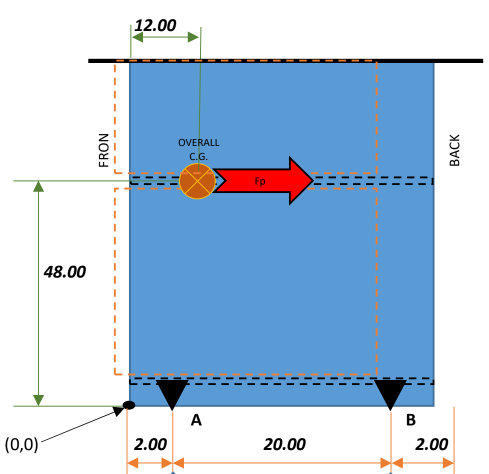


$W_{p\ Shelves}$ =	*150 LBS*
$W_{p\ Component}$ =	*280 LBS*
F_p =	*258 LBS*

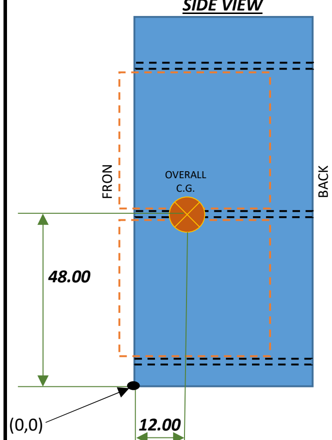
X_{cm} = *12.00* IN

Y_{cm} = *48.00* IN

BASE CONNECTION FORCES



SIDE VIEW



@ FRONT	@ BACK
963 LBS	*963 LBS*
OR	OR
-511 LBS	*-511 LBS*

@ FRONT ANCHOR	@ REAR ANCHOR
482 LBS	482 LBS
OR	OR
-256 LBS	-256 LBS

THE ABOVE REACTIONS ARE PER SIDE USING THE FOLLOWING LRFD EQUATIONS

(0.9-0.2SDS)*DL +/- Q_E	*Uplift (-)*
(0.5)*DL +/- Fp	*Uplift (-)*
(1.2+0.2SDS)*DL +/- *QE	*Compression (+)*
(1.6)*DL +/- Fp	*Compression (+)*

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Engineering	Public Works
Fire	Traffic

PRCTI20230074

Project	MN 1806
Date	
By	DB
Sheet	3 of 3

THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCE(S) FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

0

DESCRIPTION >	***Wood double oven cabinets***

Seismic Design Requirements for Non-Structural Components (ASCE 7-10, Chapter 13)

I_p = **1.0** (Section 13.1.3)

$F_p = \dfrac{0.4 * a_p * S_{DS} * W_p}{(R_p / I_p)}$ [1 + 2 (z/h)] (EQN 13.3-1)

$F_{p,max}$ = $1.6 * S_{DS} * I_p * W_p$ (EQN 13.3-2)

$F_{p,min}$ = $0.3 * S_{DS} * I_p * W_p$ (EQN 13.3-3)

a_p = **1.0** (Table 13.5-1)

R_p = **2.5** (Table 13.5-1)

z = **0** IN

h = **120** IN

Table 13.5-1 Coefficients for Architectural Components

Architectural Component	a_p[a]	R_p[b]
Interior nonstructural walls and partitions[b]		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever elements (Unbraced or braced to structural frame below its center of mass)		
Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys where laterally braced or supported by the structural frame	2.5	2.5
Cantilever elements (Braced to structural frame above its center of mass)		
Parapets	1.0	2.5
Chimneys	1.0	2.5
Exterior nonstructural walls[b]	1.0[b]	2.5
Exterior nonstructural wall elements and connections[b]		
Wall element	1.0	2.5
Body of wall panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1.0
Veneer		
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Penthouses (except where framed by an extension of the building frame)	2.5	3.5
Ceilings		
All	1.0	2.5
Cabinets		
Permanent floor-supported storage cabinets over 6 ft (1,829 mm) tall, including contents	1.0	2.5
Permanent floor-supported library shelving, book stacks, and bookshelves over 6 ft (1,829 mm) tall, including contents	1.0	2.5
Laboratory equipment	1.0	2.5
Access floors		
Special access floors (designed in accordance with Section 13.5.7.2)	1.0	2.5
All other	1.0	1.5
Appendages and ornamentations	2.5	2.5
Signs and billboards	2.5	3.0
Other rigid components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability materials and attachments	1.0	1.5
Other flexible components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5
Egress stairways not part of the building structure	1.0	2.5

[a] A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for rigid components and rigidly attached components. The value of $a_p = 2.5$ is for flexible components and flexibly attached components.
[b] Where flexible diaphragms provide lateral support for concrete or masonry walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 12.11.2.

F_p = ***0.32 * W_p*** (EQN 13.3-1)

$F_{p,max}$ = ***3.2 * W_p*** (EQN 13.3-2)

$F_{p,min}$ = ***0.6 * W_p*** (EQN 13.3-3)

F_p = ***0.6 * W_p***

W_p = ***430.0*** LBS (Assembled wood cabinet + fixtures installed into display)

F_p = ***258.0*** LBS

ast

Project MN 1806

Date

By DB

Sheet of

BRACKET-TO-SIDE PANEL CONNECTION

(8) #10 WOOD SCREWS (2 PER BRACKET/ 4 brackets per panel)

$$LOAD = \frac{258\ \#}{(2)(4)} = 22\ \#$$

0.7(22#) = 15#

12 GA. PLATE/BRACKET

$Z = 96\ \#$

$C_D = 1.6$

$Z' = 154\ \#$ ✓ O.K.

ast

Project_ MN 1806
Date
By DB
Sheet ____ of ____

CONNECTION TO HOHT-OR WALL

APPLY 75% OF LOAD @ FLOOR AND 75% AT WALL ANCHORAGE

LOAD = (0.75)(258#) = 194#

USE (3) ANCHORS PER BRACKET

$$LOAD = \frac{194\#}{(3)(4\ brackets)} = 16\#/fastener$$

PLYWOOD-TO-STUD:

W = 680# ✓ O.K.

BRACKET-TO-PLYWOOD:

W = 275# ✓ O.K.

Withdrawal:
Tables 3 and 4 present average ultimate withdrawal loads for wood and sheet metal screws in plywood-and-metal joints, based on analysis of test results. Wood screws have a tapered shank and are threaded for only 2/3 of their length. Sheet metal screws typically have higher ultimate load than wood screws in the smaller gages, because of their uniform shank diameter and full-length thread. The difference is not as apparent in the larger gages and lengths because the taper is not as significant.

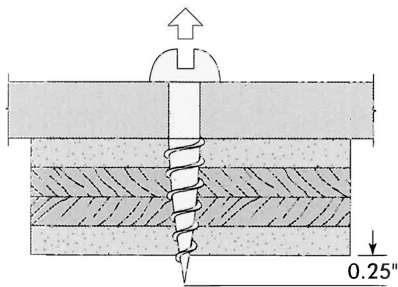
Values shown in Table 3 for wood screws are based on 1/4-in. protrusion of the wood screw from the back of the panel. This was to assure measurable length of thread embedment in the wood, since the tip of the tapered wood screw may be smaller than the pilot hole. This was not a factor for sheet metal screws due to their uniform shanks.

TABLE 3

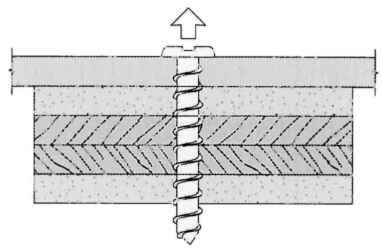
WOOD AND SHEET METAL SCREWS: METAL-TO-PLYWOOD CONNECTIONS(a)

Depth of Threaded Penetration (in.)	Average Ultimate Withdrawal Load (lbf)					
	Screw Size					
	#6	#8	#10	#12	#14	#16
3/8	150	180	205	–	–	–
1/2	200	240	275	315	350	–
5/8	250	295	345	390	440	–
3/4	300	355	415	470	525	–
1	–	–	–	625	700	775
1-1/8	–	–	–	705	790	875
2-1/4	–	–	–	–	1580	–

(a) Plywood was C-D grade with exterior glue (all plies Group 1).



Wood Screw



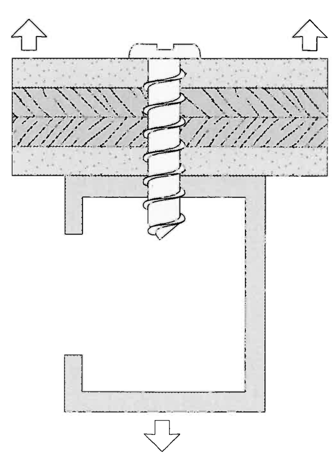
Sheet Metal Screw

TABLE 4

SHEET METAL SCREWS: PLYWOOD-TO-METAL CONNECTIONS(a)

Framing	Plywood Thickness (in.)	Ultimate Withdrawal Load (lbf)(b)				
		Screw Size				1/4"-20 Self Tapping Screw
		#8	#10	#12	#14	
0.080-in. Aluminum	1/4	130	150	170	180	220
	1/2	350	470	500	520	500
	3/4	660	680	790	850*	790*
0.078-in. Galvanized Steel (14 gage)	1/4	130	150	170	180	220
	1/2	350	470	500	520	500
	3/4	660	680	800	900	850

(a) Plywood was A-C EXT (all plies Group 1).
(b) Loads denoted by an asterisk(*) were limited by screw-to-framing strength; others were limited by plywood strength.



Hilti PROFIS Engineering 3.0.73

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Best Buy - Seismic Racks_Section L - S-DOC.1 A - hi	Date:	11/9/2021
Fastening point:			

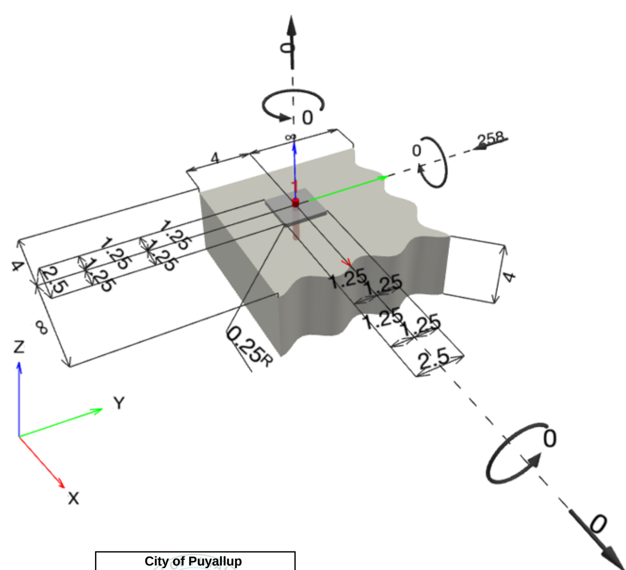
Specifier's comments:

1 Input data

Anchor type and diameter:	**Kwik Bolt TZ2 - CS 3/8 (2) hnom2**
Item number:	2210237 KB-TZ2 3/8x3 1/2
Effective embedment depth:	$h_{ef,act}$ = 2.000 in., h_{nom} = 2.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-4266
Issued I Valid:	7/1/2021 | 12/1/2021
Proof:	Design Method ACI 318-14 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.250 in.
Anchor plate^R :	l_x x l_y x t = 2.500 in. x 2.500 in. x 0.250 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 4.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

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

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



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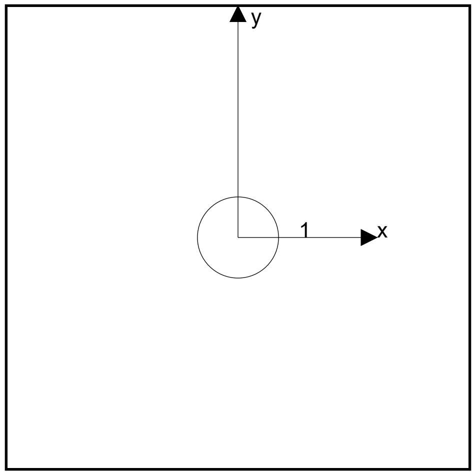
Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Best Buy - Seismic Racks_Section L - S-DOC.1 A - hi	Date:	11/9/2021
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0; V_x = 0; V_y = -258;$ $M_x = 0; M_y = 0; M_z = 0;$	yes	26

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	258	0	-258

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]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout 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)

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	258	2,201	12	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	258	2,079	13	OK
Concrete edge failure in direction y-**	258	1,027	26	OK

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

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-4266

ϕ $V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.05	126,204	1.000

Calculations

$V_{sa,eq}$ [lb]
3,386

Results

$V_{sa,eq}$ [lb]	ϕ steel	$\phi_{nonductile}$	ϕ $V_{sa,eq}$ [lb]	V_{ua} [lb]
3,386	0.650	1.000	2,201	258

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4.2 Pryout Strength

$V_{cp} = k_{cp}\left[\left(\frac{A_{Nc}}{A_{Nc0}}\right)\psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ ACI 318-14 Eq. (17.5.3.1a)

$\phi\ V_{cp} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1	2.000	4.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
4.375	21	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
36.00	36.00	1.000	1.000	2,970

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
2,970	0.700	1.000	1.000	2,079	258

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4.3 Concrete edge failure in direction y-

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ ACI 318-14 Eq. (17.5.2.1a)

$\phi\ V_{cb} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\ c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
4.000	4.000	1.000	4.000	2.000

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.375	2,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
40.00	72.00	0.900	1.225	2,396

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
1,468	0.700	1.000	1.000	1,027	258

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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.
- 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/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (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 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (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 17.2.3.4.3 (c) / Section 17.2.3.5.3 (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 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

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6 Installation data

Anchor type and diameter: Kwik Bolt TZ2 - CS 3/8 (2) hnom2

Item number: 2210237 KB-TZ2 3/8x3 1/2

Maximum installation torque: 361 in.lb

Hole diameter in the base material: 0.375 in.

Hole depth in the base material: 2.750 in.

Minimum thickness of the base material: 4.000 in.

Profile: no profile

Hole diameter in the fixture: d_f = 0.438 in.

Plate thickness (input): 0.250 in.

Recommended plate thickness: not calculated

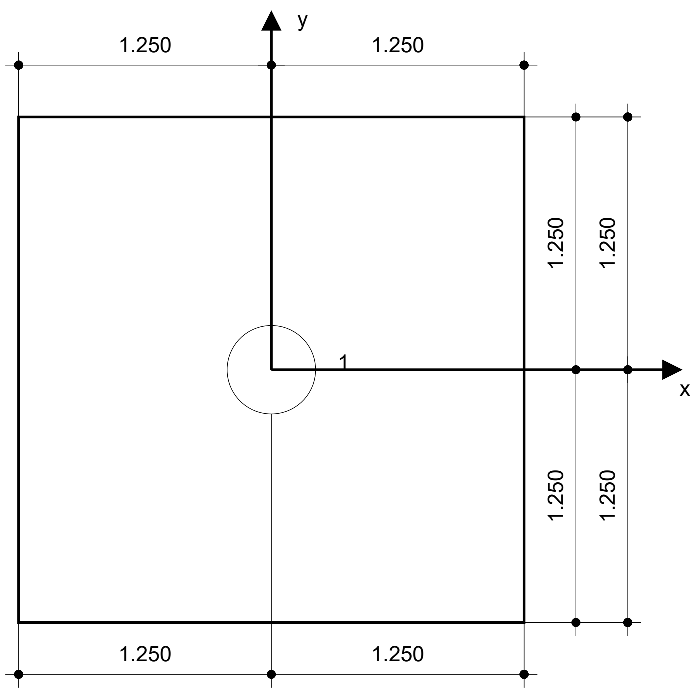
Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti KB-TZ2 stud anchor with 2.5 in embedment, 3/8 (2) hnom2, Carbon steel, installation per ESR-4266

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hammer



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	4.000	-	4.000	

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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.

ast

Project MN 1806
Date
By DB
Sheet of

12 GA. BRACKET CONNECTION TO 'HARD' WALL

HILTI X-U ($\phi = 0.157"$)

ULT. LOAD

LOAD/FASTENER = 258# / (3)(4 brackets)

= 22 #

1) CONCRETE

CAPACITY/FASTENER = 165# ✓ O.K

2) CMU

CAPACITY/FASTENER = 70 # ✓ O.K.

3) PULL-OVER TENSILE CAPACITY (12 GA. BRACKET)

CAPACITY/FASTENER = 825# ✓ O.K

TABLE 3—ALLOWABLE LOADS FOR FASTENERS DRIVEN INTO NORMAL-WEIGHT CONCRETE[1,2,4]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	MINIMUM EMBEDMENT DEPTH (inches)	ALLOWABLE LOADS (lbf)							
Concrete Compressive Strength:				2500 psi		4000 psi		6000 psi		8000 psi	
Load Direction:				Tension	Shear	Tension	Shear	Tension	Shear	Tension	Shear
Universal Knurled Shank	X-U	0.157	3/4	100	125	100	125	105	205	–	–
			1	165	190	170	225	110[3]	280[3]	–	–
			1 1/4	240	310	280	310	180	425	–	–
			1 1/2	275	420	325	420	–	–	–	–
Smooth Shank	X-P	0.157	3/4[5]	100	155	100	175	105	205	135	205
			1[5]	165	220	180	225	150	300	150	215
			1 1/4[5]	240	310	280	310	180	425	–	–
			1 1/2[5]	310	420	–	–	–	–	–	–

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 psi = 6895 Pa.

[1]Unless otherwise noted, values apply to normal weight cast-in-place concrete. Fasteners must not be driven until the concrete has reached the designated minimum compressive strength.
[2]Unless otherwise noted, concrete thickness must be a minimum of 3 times the embedment depth of the fastener.
[3]This allowable load value for the X-U fastener also applies to normal weight hollow core concrete slabs with f'_c of 6600 psi and minimum dimensions shown in Figure 7, when installed in accordance with Section 4.2.4.
[4]The fasteners listed in the table above may be used for static load conditions and for the seismic load conditions described in Section 4.1.6, as applicable. The tabulated allowable loads apply to static load conditions. For seismic load conditions, the allowable loads must be limited in accordance with Section 4.1.6, Items 2 and 3, as applicable.
[5]Applies to fastening of cold-formed steel up to 54 mil thick using the X-P 22, X-P 27, X-P 34 and X-P 40 fasteners, respectively, for the 3/4, 1, 1 1/4 and 1 1/2 inch embedment depths.

TABLE 4—ALLOWABLE LOADS FOR FASTENERS DRIVEN INTO NORMAL-WEIGHT CONCRETE USING DX-KWIK[1,2,3,4]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	MINIMUM EMBEDMENT (inches)	ALLOWABLE LOADS (lbf)			
Concrete Compressive Strength:				4,000 psi		6,000 psi	
Load Direction:				Tension	Shear	Tension	Shear
Universal Knurled Shank	X-U 47 P8 w/ DX-KWIK	0.157	1 1/2	395	405	360	570

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 psi = 6895 Pa.

[1]X-U Fastener is installed using the DX-KWIK drilled pilot hole installation procedure described in Section 4.2.5.
[2]Pilot holes must not be drilled until the concrete has reached the designated minimum compressive strength.
[3]Concrete thickness must be a minimum of 3 times the embedment depth of the fastener.
[4]The fasteners listed in the table above may be used for static load conditions and for the seismic load conditions described in Section 4.1.6, as applicable. The tabulated allowable loads apply to static load conditions. For seismic load conditions, the allowable loads must be limited in accordance with Section 4.1.6, Items 2 and 3, as applicable.

TABLE 5—ALLOWABLE LOADS FOR FASTENERS DRIVEN INTO MINIMUM f'_c = 3000 psi STRUCTURAL SAND-LIGHTWEIGHT CONCRETE WITH OR WITHOUT METAL DECK[1,6]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	MINIMUM EMBEDMENT (inch)	ALLOWABLE LOADS (lbf)							
Fastener Location:				Installed into Concrete[4]		Installed Through Metal Deck Panel into Concrete[5]					
						3-inch deep composite floor deck panel[2]			$1^1/_2$-inch deep composite floor deck panel[3]		
Load Direction:				Tension	Shear	Tension		Shear	Tension		Shear
						Upper Flute	Lower Flute	Lower Flute	Upper Flute	Lower Flute	Lower Flute
Universal Knurled Shank	X-U	0.157	$^3/_4$	125	115	130	95	245	95	95	370
			1	205	260	215	155	330	125	125	415
			$1^1/_4$	315	435	295	200	375	–	–	–
			$1^1/_2$	425	475	400	260	430	–	–	–
Smooth Shank	X-P	0.157	$^3/_4$[7]	155	165	130	105	285	140	130	335
			1[7]	225	300	215	165	340	215	215	385
			$1^1/_4$[7]	325	445	295	230	375	–	270	465
			$1^1/_2$[7]	425	480	400	330	365	–	–	–

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 psi = 6895 Pa.

[1]Fasteners must not be driven until the concrete has reached the designated minimum compressive strength.
[2]The steel deck profile for the 3-inch deep composite floor deck panel has a minimum thickness of 0.0359 inch (0.91 mm) and a minimum F_y of 50 ksi and a minimum tensile strength of 65 ksi. Lower and upper flute width must be a minimum of $3^7/_8$ inches. Figure 4 shows the nominal flute dimensions, fastener locations, and load orientations for the deck panel profile. Sand-lightweight concrete fill above top of steel deck panel must be minimum $3^1/_4$ inches thick.
[3]The steel deck profile for the $1^1/_2$-inch deep composite floor deck panel has a minimum thickness of 0.0359 inch (0.91 mm) and a minimum F_y of 33 ksi and a minimum tensile strength of 45 ksi. Lower flute and upper flute widths must be a minimum of $1^3/_4$ inch and $3^1/_2$ inch, respectively. This deck panel may also be inverted as shown in Figure 6. Figures 5 and 6 show the nominal flute dimensions, fastener locations, and load orientations for the deck panel profile. Sand-lightweight concrete fill above top of steel deck panel must be minimum $2^1/_2$ inches thick.
[4]Concrete thickness must be a minimum of 3 times the embedment depth of the fastener.
[5]Minimum allowable spacing parallel to the deck flutes is 5.1 inches.
[6]The fasteners listed in the table above may be used for static load conditions and for the seismic load conditions described in Section 4.1.6, as applicable. The tabulated allowable loads apply to static load conditions. For seismic load conditions, the allowable loads must be limited in accordance with Section 4.1.6, Items 2 and 3, as applicable.
[7]Applies to fastening of cold-formed steel up to 54 mil thick using the X-P 22, X-P 27, X-P 34 and X-P 40 fasteners, respectively, for the $^3/_4$, 1, $1^1/_4$ and $1^1/_2$ inch embedment depths.

TABLE 6—ALLOWABLE LOADS FOR FASTENERS DRIVEN INTO CONCRETE MASONRY[3,10]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	MINIMUM EMBEDMENT DEPTH (inch)	ALLOWABLE LOADS (lbf)									
Masonry Type:				Hollow CMU				Grouted CMU					
Fastener Location:				Face Shell[4]		Mortar Joint[5]		Face Shell[4]		Mortar Joint[5]		Top of Grouted Cell[8]	
Load Direction:				Tension	Shear[9]	Tension	Shear[6]	Tension	Shear[9]	Tension	Shear[6]	Tension	Shear[9]
Universal Knurled Shank[1]	X-U	0.157	1	70	85	25	70	215	210	150	190	165	240
Smooth Shank[2]	X-P	0.157	1	70	105	85	70	150	145	150	155[7]	165	240

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N

[1]The tabulated allowable load values are for fasteners installed in masonry conforming to the requirements of Section 3.2.2 of this report. CMUs must be normal weight. Mortar must be Type S.
[2]The tabulated allowable load values are for fasteners installed in masonry conforming to the requirements of Section 3.2.2 of this report. CMUs must be medium weight. Mortar must be Type S.
[3]No more than one low-velocity fastener may be installed in an individual concrete masonry unit cell. The fastener must be installed a minimum of 4 inches from the top, bottom and edges of the wall.
[4]Fastener must be located a minimum of 1 inch from the mortar joints.
[5]Fasteners must not be installed in the head joints. Fasteners installed in the bed joints must be installed a minimum of 8 inches from the end of the wall. Multiple fasteners in a bed joint must be spaced a minimum of 8 inches.
[6]Unless otherwise noted, shear load direction can be horizontal or vertical along the CMU wall plane.
[7]When installed at the intersection of head joint and bed joint (T-joint), the shear load can be applied parallel to the bed joint. When installed away from the T-joint, the shear load direction can be horizontal or vertical along the CMU wall plane.
[8]Fastener located in center of grouted cell installed vertically.
[9]Shear load can be in any direction.
[10]The fasteners listed in the table above may be used for static load conditions and for the seismic load condition described in Item 1 of Section 4.1.6.

Allowable loads in minimum ASTM A36 ($F_y \geq 36$ ksi, $F_u \geq 58$ ksi) steel[1,2,4,5]

Fastener description	Fastener	Shank diameter in. (mm)	Steel thickness (in.)											
			1/8		3/16		1/4		3/8		1/2		≥3/4	
			Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)
Universal knurled shank*	X-U[6]	**0.157** (4.0)	–	–	**535** (2.38)	**720** (3.20)	**775** (3.45)	**720** (3.20)	**935** (4.16)	**720** (3.20)	**900** (4.00)	**720** (3.20)	**350** (1.56)	**375** (1.67)
Stepped-shank knurling-lengthwise	X-U 15[7]	**0.145** (3.7)	–	–	**155** (0.69)	**395** (1.76)	**230** (1.02)	**395** (1.76)	**420** (1.86)	**450** (2.00)	**365** (1.62)	**500** (2.22)	**365** (1.62)	**400** (1.78)
Standard knurled shank	X-S13	**0.145** (3.7)	**140** (0.62)	**300** (1.33)	**300** (1.33)	**450** (2.00)	**300** (1.33)	**450** (2.00)	**300** (1.33)	**450** (2.00)	–	–	–	–
Drywall smooth shank w/metal top hat washer	X-S16	**0.145** (3.7)	–	–	**225** (1.00)	**420** (1.87)	**225** (1.00)	**430** (1.91)	**225** (1.00)	**430** (1.91)	**225** (1.00)	**430** (1.91)	–	–
Heavy duty knurled shank	EDS[3]	**0.177** (4.5)	–	–	**305** (1.36)	**615** (2.67)	**625** (2.78)	**870** (3.87)	**715** (3.18)	**870** (3.87)	**890** (3.96)	**960** (4.27)	**400** (1.78)	**655** (2.91)
Heavy duty smooth shank	DS	**0.177** (4.5)	–	–	**365** (1.62)	**725** (3.22)	**580** (2.58)	**725** (3.22)	**695** (3.09)	**725** (3.22)	**735** (3.27)	**860** (3.83)	–	–
Stainless steel smooth shank	X-R[10]	**0.145** (3.7)	–	–	**460** (2.05)	**460** (2.05)	**615** (2.74)	**500** (2.22)	–	–	–	–	–	–
	X-R[8,10]	**0.145** (3.7)	**300** (1.33)	**190** (0.85)	**615** (2.74)	**495** (2.20)	**760** (3.38)	**500** (2.22)	**220** (0.98)	**325** (1.45)	**225** (1.00)	**335** (1.49)	–	–
Standard gas fastener for steel	X-EGN 14[9], X-S 14 B3	**0.118** (3.0)	**140** (0.6)	**230** (1.0)	**220** (1.0)	**245** (1.1)	**225** (1.0)	**290** (1.3)	**280** (1.2)	**330** (1.5)	**280** (1.2)	**330** (1.5)	**280** (1.2)	**330** (1.5)
Standard gas fastener for steel	X-EGN 14[8,9] X-S 14 B38	**0.118** (3.0)	–	–	**220** (1.0)	**295** (1.3)	**260** (1.2)	**355** (1.6)	**280** (1.2)	**385** (1.7)	**280** (1.2)	**385** (1.7)	**280** (1.2)	**385** (1.7)
Premium gas fastener	X-GHP, X-P G3, X-P B3	**0.118** (3.0)	**125** (0.6)	**230** (1.0)	**170** (0.8)	**245** (1.1)	**200** (0.9)	**230** (1.0)	**250** (1.1)	**255** (1.1)	–	–	–	–

1 The tabulated allowable load values are for the low-velocity fasteners only, using a safety factor that is greater than or equal to 5.0, calculated in accordance with ICC-ES AC70. Wood or steel members connected to the substrate must be investigated in accordance with accepted design criteria.
2 Low-velocity fasteners shall be driven to where the point of the fastener penetrates through the steel base material in accordance with Section 3.2.2.3, except as noted in this table.
3 EDS fasteners installed into greater than 1/2" thick steel require 1/2" minimum penetration.
4 Multiple fasteners are recommended for any attachment.
5 Refer to guidelines for fastening to steel, Section 3.2.2, for application limits.
6 Tabulated allowable load values provided for 3/4" steel are based upon minimum point penetration of 1/2" into the steel. If 1/2" point penetration into the steel is not achieved, but a point penetration of at least 3/8" is obtained, the tabulated tension value should be reduced by 20 percent and the tabulated shear load should be reduced by 8 percent.
7 X-U 15 fasteners installed into greater than 3/8" thick steel require 15/32" minimum penetration into the steel.
8 Based on testing with F_y = 50 ksi base material.
9 Fasteners installed into 3/8" or thicker base steel require 0.320" minimum penetration depth into the steel.
10 Fasteners installed into 3/8" or thicker base require 0.38" minimum penetration depth into the steel.

Allowable tensile pullover and shear bearing load capacities for steel framing with power driven fasteners[1,2,3,4]

Fastener description	Fastener	Head dia. in. (mm)	Sheet steel thickness													
			14 ga.		16 ga.		18 ga.		20 ga.		22 ga.		24 ga.		25/26 ga.	
			Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)	Tension lb (kN)	Shear lb (kN)
0.157" shank with or w/o plastic washers or MX collation	X-U, X-P	**0.322** (8.2)	**825** (3.67)	**1,085** (4.83)	**685** (3.05)	**720** (3.20)	**490** (2.18)	**525** (2.34)	**360** (1.60)	**445** (1.98)	**300** (1.33)	**330** (1.47)	**205** (0.91)	**255** (1.13)	**120** (0.53)	**145** (0.64)
0.145" shank with or w/o plastic washers or MX collation	X-C, X-R	**0.322** (8.2)	–	**985** (4.38)	**685** (3.05)	**720** (3.20)	**490** (2.18)	**515** (2.29)	**360** (1.60)	**440** (1.96)	**300** (1.33)	**310** (1.38)	**205** (0.91)	**235** (1.05)	**120** (0.53)	**145** (0.64)
0.177" shank without washer	DS, EDS	**0.322** (8.2)	**965** (4.29)	**1,085** (4.83)	**810** (3.60)	**815** (3.63)	**625** (2.78)	**535** (2.38)	**460** (2.05)	**465** (2.07)	**360** (1.60)	**350** (1.56)	**300** (1.33)	**260** (1.16)	**240** (1.07)	**180** (0.80)
0.145" shank with plastic top hat washers	X-S13 THP X-S16 TH	**0.322** (8.2)	–	**985** (4.38)	**685** (3.05)	**720** (3.20)	**490** (2.18)	**515** (2.29)	**360** (1.60)	**440** (1.96)	**300** (1.33)	**310** (1.38)	**205** (0.91)	**235** (1.05)	**120** (0.53)	**145** (0.64)
0.118" shank with MX collation	X-EGN, X-GN, X-GHP	**0.276** (6.8)	–	–	–	–	**325** (1.45)	**390** (1.73)	**265** (1.18)	**335** (1.49)	**250** (1.11)	**235** (1.05)	**170** (0.76)	**185** (0.82)	**100** (0.44)	**125** (0.56)

1 Allowable load values are based on a safety factor of 3.0.
2 Allowable pullover capacities of sheet steel should be compared to the allowable fastener tensile load capacities in concrete, steel, and masonry to determine controlling resistance load.
3 Allowable shear bearing capacities of sheet steel should be compared to allowable fastener shear capacities in concrete, steel and masonry to determine controlling resistance load.
4 Data is based on the following minimum sheet steel properties, F_y = 33 ksi, F_u = 45 ksi (ASTM A653 material).

* More details about the innovative X-U fastener can be found in Section 3.2.6.