STRUCTURAL CALCULATIONS

FOR

SEISMIC BRACING OF FIXTURES

WILLIAMS SONOMA – BREWERY BLOCKS
PORTLAND, OR
AST PROJECT #OR 1050

REGISTERED PROFESSIONAL ENGINEER
93264PE
OREGON
7/8/22
ORIGINAL DATE OF REG JAN 9, 2018
MICHELLE ALICIA SCHMIDT
EXPIRES: 6/30/24

PERMIT SUBMITTAL
JULY 8, 2022

PREPARED BY:

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By	DB
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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 >	Williams Sonoma Free-standing fixture

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 =	0.885	(From USGS Website or Figure 22-1 to 22-18)
-SECOND PERIOD ACCELERATION:	S_1 =	0.396	(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 =	*1.9*	(Table 11.4-2)

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

RESPONSE SPECTRUM

S_{MS} =	*1.062*	$S_{MS} = F_a \times S_s$	(EQN 11.4-1)
S_{M1} =	*0.754*	$S_{M1} = F_v \times S_1$	(EQN 11.4-2)
S_{DS} =	*0.708*	$S_{DS} = (2/3)S_{MS}$	(EQN 11.4-3)
S_{D1} =	*0.503*	$S_{D1} = (2/3)S_{M1}$	(EQN 11.4-4)

Earthquake Design Criteria

SEISMIC DESIGN CATEGORY

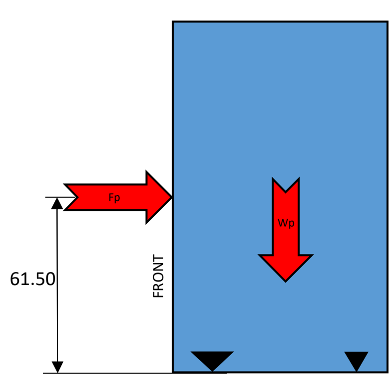
S_{DS} =	*D*	(Table 11.6-1)
S_{D1} =	*D*	(Table 11.6-2)
SDC FOR DESIGN =	*D*	

Seismic Weight

Self Weight of Fixture = 2000 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*	*2000*	61.50	*2000*	*21.20%*	*424*	26076.0
	SHELF #2	*0*	0.00	*0*	*21.20%*	*0*	0.0
	SHELF #3	*0*	0.00	*0*	*21.20%*	*0*	0.0
	SHELF #4	*0*	0.00	*0*	*21.20%*	*0*	0.0
	CONTENT #1	0	0.00	*0*	*21.20%*	*0*	0.0
	CONTENT #2	0	0.00	*0*	*21.20%*	*0*	0.0
	CONTENT #3	0	0.00	*0*	*21.20%*	*0*	0.0
	CONTENT #4	0	0.00	*0*	*21.20%*	*0*	0.0
				2000		***424***	***26076.0*** LBS-IN
				W_p		F_p	***2173.0*** LBS-FT



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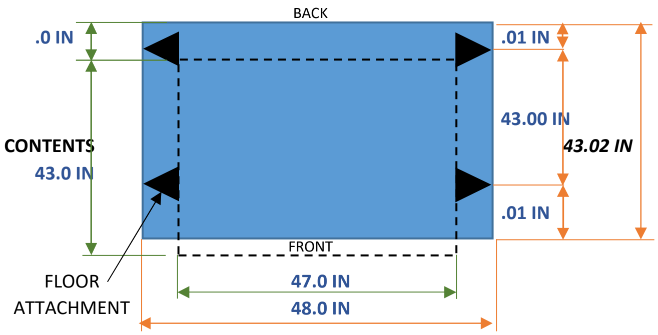
THIS SPREADSHEET IS USED TO CALCULATE THE SEISMIC DESIGN FORCES FOR NONSTRUCTURAL COMPONENTS USING ASCE 7-16, CHAPTER 13

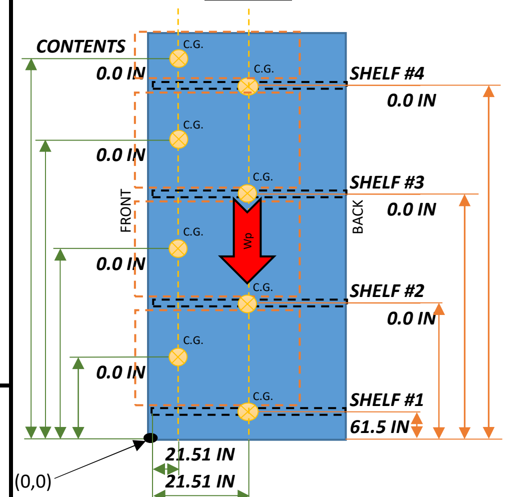
DESCRIPTION >	*Williams Sonoma* *Free-standing fixture*

THIS IS AN EXAMPLE LAYOUT ONLY

PLAN VIEW



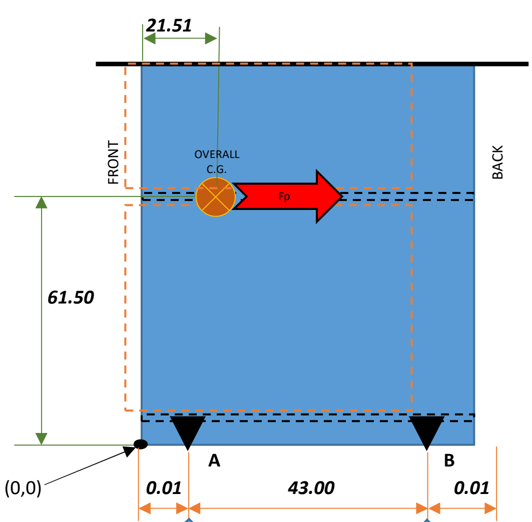
SIDE VIEW



X_{cm} =	*21.51* IN
Y_{cm} =	*61.50* IN

$W_{p\ Shelves}$ =	***2000 LBS***
$W_{p\ Component}$ =	***LBS***
F_p =	***424 LBS***

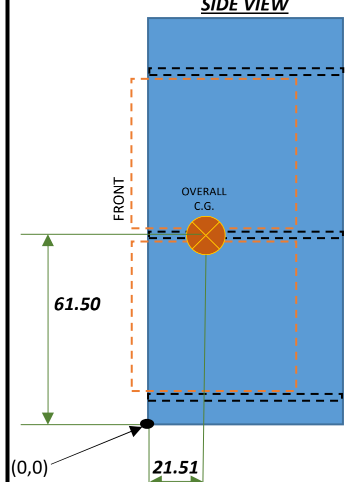
BASE CONNECTION FORCES



@ FRONT
1948 LBS
OR
152 LBS

@ BACK
1948 LBS
OR
152 LBS

SIDE VIEW



@ FRONT ANCHOR
974 LBS
OR
76 LBS

@ REAR ANCHOR
974 LBS
OR
76 LBS

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

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

NO NET UPLIFT. THEREFORE, FLOOR ANCHORS RESIST SHEAR FORCES ONLY

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

DESCRIPTION >	*Williams Sonoma* *Free-standing fixture*

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

$h =$ **102** 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

$F_p =$ ***0.113 * W_p*** (EQN 13.3-1)

$F_{p,max} =$ ***1.133 * W_p*** (EQN 13.3-2)

$F_{p,min} =$ ***0.212 * W_p*** (EQN 13.3-3)

$F_p =$ ***0.212 * W_p***

$W_p =$ ***2000.0*** LBS (Assembled fixture + attachments)

$F_p =$ ***424.0*** LBS

shear capacity per anchor = 245# (see following pages)

Required # anchors = 1.73

Per detail 4-5/S310, a min. of 6 anchors are provided per fixture

Hilti PROFIS Engineering 3.0.79

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Jul 8, 2022	Date:	7/8/2022
Fastening point:			

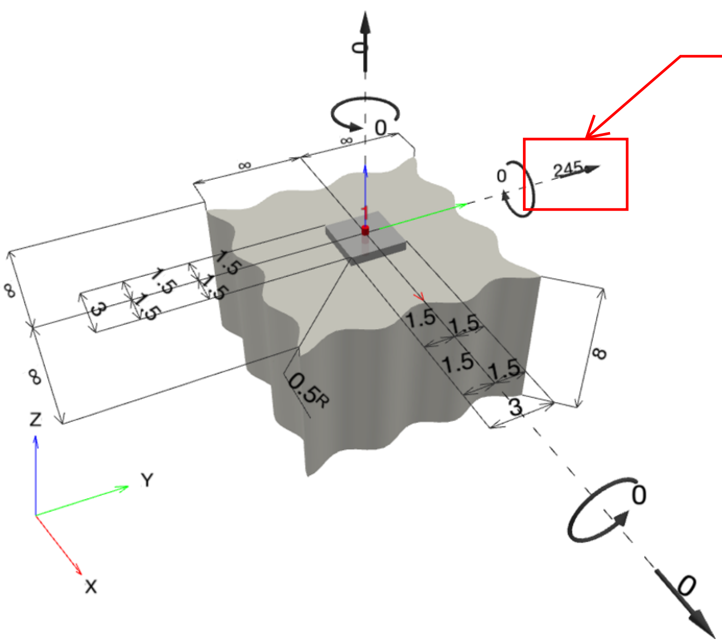
Specifier's comments:

1 Input data

Anchor type and diameter:	**HDI-P TZ 3/8**
Item number:	not available
Effective embedment depth:	h_{ef} = 0.750 in., h_{nom} = 0.750 in.
Material:	ASTM A 36
Evaluation Service Report:	ESR-4236
Issued I Valid:	7/1/2021 | 7/1/2023
Proof:	Design Method ACI 318 / AC193
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 3.000 in. x 3.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 8.000 in.
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)	no

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

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



CAPACITY PER ANCHOR

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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 = 245; M_x = 0; M_y = 0; M_z = 0;	no	99

2 Load case/Resulting anchor forces

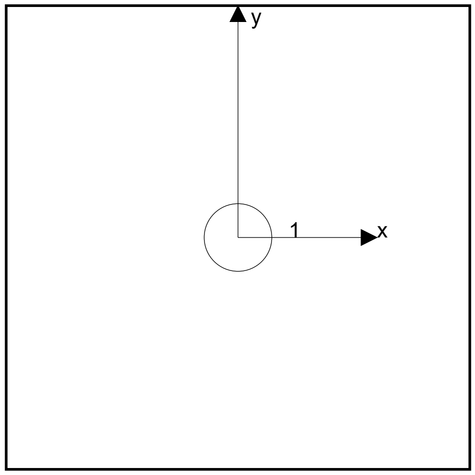
Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	245	0	245

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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Design:	Concrete - Jul 8, 2022	Date:	7/8/2022
Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	245	585	42	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	245	248	99	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

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

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-4236

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.07	58,000	1.000

Calculations

V_{sa} [lb]
975

Results

V_{sa} [lb]	ϕ_{steel}	ϕ V_{sa} [lb]	V_{ua} [lb]
975	0.600	585	245

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Address:		Specifier:	
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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 318-08 Eq. (D-30)

$\phi\ V_{cp} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-08 Eq. (D-6)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-08 Eq. (D-13)

$N_b = k_c\ \lambda\ \sqrt{f_c'}\ h_{ef}^{1.5}$ ACI 318-08 Eq. (D-7)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1	0.750	∞	1.000

c_{ac} [in.]	k_c	λ	f_c' [psi]
6.500	17	1	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
5.06	5.06	1.000	1.000	552

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
552	0.450	248	245

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/
- This anchor can only be used for the support of non structural elements
- This anchor can only be used for overhead applications, see figure 3 of ESR-4236

Fastening meets the design criteria!

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Design:	Concrete - Jul 8, 2022	Date:	7/8/2022
Fastening point:			

6 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = 0.438 in.

Plate thickness (input): 0.500 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: HDI-P TZ 3/8

Item number: not available

Maximum installation torque: -

Hole diameter in the base material: 0.562 in.

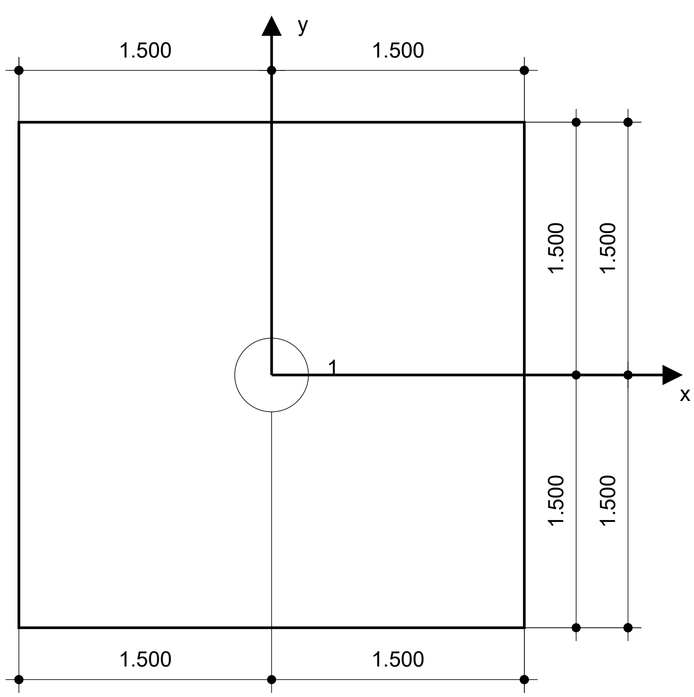
Hole depth in the base material: 0.750 in.

Minimum thickness of the base material: 4.000 in.

Hilti HDI flush anchor with 0.75 in embedment, 3/8, Carbon steel, installation per ESR-4236

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• -	• Drop-in anchor hand-setting or machine-setting tool • Torque wrench • Hammer



Coordinates Anchor [in.]

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

Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Jul 8, 2022	Date:	7/8/2022
Fastening point:			

7 Remarks; Your Cooperation Duties

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Project	
Date	7/8/2022
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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 >	Williams Sonoma Interior fixture (bar-glassware) - laterally braced at floor and light-ga wall

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 =	0.885	(From USGS Website or Figure 22-1 to 22-18)
-SECOND PERIOD ACCELERATION:	S_1 =	0.396	(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 =	*1.9*	(Table 11.4-2)

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

RESPONSE SPECTRUM

S_{MS} =	*1.062*	$S_{MS} = F_a \times S_s$	(EQN 11.4-1)
S_{M1} =	*0.754*	$S_{M1} = F_v \times S_1$	(EQN 11.4-2)
S_{DS} =	*0.708*	$S_{DS} = (2/3)S_{MS}$	(EQN 11.4-3)
S_{D1} =	*0.503*	$S_{D1} = (2/3)S_{M1}$	(EQN 11.4-4)

Earthquake Design Criteria

SEISMIC DESIGN CATEGORY

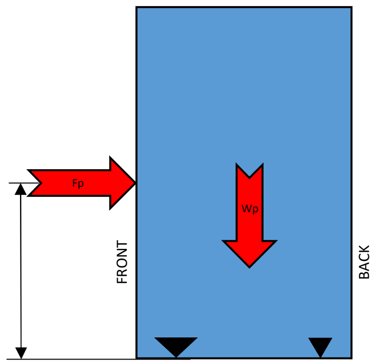
S_{DS} =	*D*	(Table 11.6-1)
S_{D1} =	*D*	(Table 11.6-2)
SDC FOR DESIGN =	*D*	

Seismic Weight

Self Weight of Fixture = 3800 Lbs. 3000 self wt. of fixture + 800 pounds product

Number of Shelves = 1

	Shelf	Weight of Contents (LBS)	Shelf Height Above Base (IN)	Total Weight	F_p Coefficient	Seismic Force
COMPONENT MAKEUP	*SHELF #1*	*3800*	72.00	*3800*	*21.20%*	*806*
	SHELF #2	*0*	0.00	*0*	*21.20%*	*0*
	SHELF #3	*0*	0.00	*0*	*21.20%*	*0*
	SHELF #4	*0*	0.00	*0*	*21.20%*	*0*
	CONTENT #1	0	0.00	*0*	*21.20%*	*0*
	CONTENT #2	0	0.00	*0*	*21.20%*	*0*
	CONTENT #3	0	0.00	*0*	*21.20%*	*0*
	CONTENT #4	0	0.00	*0*	*21.20%*	*0*
				3800 W_p		*806* F_p



Apply 0.6(806#) = 483# at base and top of fixture

required # floor anchors = 483/245 = 1.97
2 are provided each bay
(see previous pages for floor anchor capcity calc)

Fixture header connection to wall (using a S.F. = 8.0)
T/all per screw = 800#/8 = 100#
(2) screws per stud = 200#/stud

o.k. by inspection (studs are spaced at 24" o.c. max)

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

DESCRIPTION >	***Williams Sonoma*** ***Interior fixture (bar-glassware) - laterally braced at floor and light-ga wall***

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

I_p = **1.0** (Section 13.1.3)

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

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

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

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	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 = ***0.113*** $* W_p$ (EQN 13.3-1)
$F_{p,max}$ = ***1.133*** $* W_p$ (EQN 13.3-2)
$F_{p,min}$ = ***0.212*** $* W_p$ (EQN 13.3-3)

F_p = ***0.212*** $* W_p$
W_p = ***3800.0*** LBS (Assembled fixture + attachments)

F_p = ***805.6*** LBS

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.

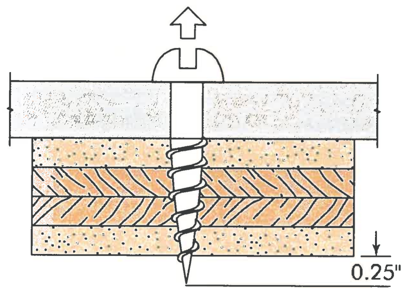
Values shown in Table 3 for wood screws are based on 1/4-in. protrusion of the wood screw from the back of the panel. This was to assure measurable length of thread embedment in the wood, since the tip of the tapered wood screw may be smaller than the pilot hole. This was not a factor for sheet metal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

TABLE 3

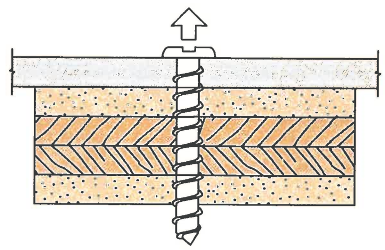
WOOD AND SHEET METAL SCREWS: METAL-TO-PLYWOOD CONNECTIONS(a)

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

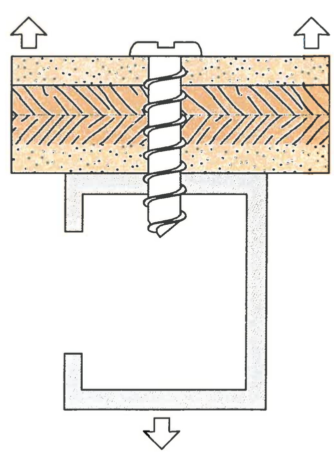
TABLE 4

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.



Project	
Date	7/8/2022
By	DB
Sheet	of

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 >	Williams Sonoma Cutlery/CT Bin - laterally braced at floor and added beam above

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 =	0.885	(From USGS Website or Figure 22-1 to 22-18)
-SECOND PERIOD ACCELERATION:	S_1 =	0.396	(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 =	1.9	(Table 11.4-2)

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

RESPONSE SPECTRUM

S_{MS} =	1.062	$S_{MS} = F_a \times S_s$	(EQN 11.4-1)
S_{M1} =	0.754	$S_{M1} = F_v \times S_1$	(EQN 11.4-2)
S_{DS} =	0.708	$S_{DS} = (2/3)S_{MS}$	(EQN 11.4-3)
S_{D1} =	0.503	$S_{D1} = (2/3)S_{M1}$	(EQN 11.4-4)

Earthquake Design Criteria

SEISMIC DESIGN CATEGORY

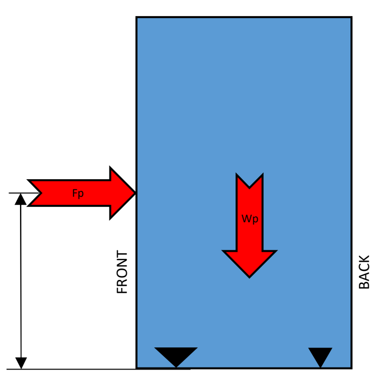
S_{DS} =	D	(Table 11.6-1)
S_{D1} =	D	(Table 11.6-2)
SDC FOR DESIGN =	D	

Seismic Weight

Self Weight of Fixture = 4000 Lbs. 3200 self wt. of fixture + 800 pounds product

Number of Shelves = 1

	Shelf	Weight of Contents (LBS)	Shelf Height Above Base (IN)	Total Weight	F_p Coefficient	Seismic Force
COMPONENT MAKEUP	SHELF #1	4000	72.00	4000	21.20%	848
	SHELF #2	0	0.00	0	21.20%	0
	SHELF #3	0	0.00	0	21.20%	0
	SHELF #4	0	0.00	0	21.20%	0
	CONTENT #1	0	0.00	0	21.20%	0
	CONTENT #2	0	0.00	0	21.20%	0
	CONTENT #3	0	0.00	0	21.20%	0
	CONTENT #4	0	0.00	0	21.20%	0
				4000 W_p		848 F_p



Apply 0.6(848#) = 508# at base and top of fixture

required # floor anchors = 508/245 = 2
2 are provided each bay
(see previous pages for floor anchor capcity calc)

Fixture header connection to beam (using a S.F. = 8.0)
T/allowable per screw = 800#/8 = 100#

min of (2) acrews per bay x 4 bays = 8 x 100# = 800#

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

DESCRIPTION >	*Williams Sonoma* *Cutlery/CT Bin - laterally braced at floor and added beam above*

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	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 = ***0.113*** $* W_p$ (EQN 13.3-1)

$F_{p,max}$ = ***1.133*** $* W_p$ (EQN 13.3-2)

$F_{p,min}$ = ***0.212*** $* W_p$ (EQN 13.3-3)

F_p = ***0.212*** $* W_p$

W_p = ***4000.0*** LBS (Assembled fixture + attachments)

F_p = ***848.0*** LBS

Project OR 1050 15

Date

By

Sheet ____ of ____

CHECK GIRT   L = 25'

$F_p = \frac{508\#}{20'} = 25\ plf$

HSS6×4×1/4 ← SEE FOLLOWING PAGES

CHECK SAG   HSS6×4×1/4

$I_{yy} = 11.1\ in^4$

$w_D = 16\ plf$

$\Delta = 0.44"$ SAG

USE HSS6×6×1/4

CHECK GIRT   L ≤ 16'

$F_p = 25\ plf$

HSS4×4×1/4 ← SEE FOLLOWING PAGES

$\Delta = 0.163"$

Gravity Beam Design

RAM SBeam v3.0
Licensed to: Advanced Structural Tech 07/08/22 11:04:10

STEEL CODE: AISC LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (16.00,0.00)

Beam Size (User Selected) = HSS4X4X1/4 Fy = 46.0 ksi
Total Beam Length (ft) = 16.00
Mp (kip-ft) = 17.98

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL
1	0.000	0.025	0.000
	16.000	0.025	0.000

SHEAR (Ultimate): Max Vu (1.4DL) = 0.28 kips 0.90Vn = 46.30 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.4DL	1.1	8.0	16.0	1.14	0.90	16.18
Controlling		1.4DL	1.1	8.0	16.0	1.14	0.90	16.18

REACTIONS (kips):

	Left	**Right**
DL reaction	0.20	0.20
Max +total reaction (factored)	0.28	0.28

DEFLECTIONS:

Dead load (in)	at	8.00 ft	=	-0.163	L/D	=	1178
Live load (in)	at	8.00 ft	=	-0.000			
Net Total load (in)	at	8.00 ft	=	-0.163	L/D	=	1178

Gravity Beam Design

RAM SBeam v3.0
Licensed to: Advanced Structural Tech 07/08/22 10:51:21

STEEL CODE: AISC LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (25.00,0.00)

Beam Size (User Selected) = HSS6X4X1/4 Fy = 46.0 ksi
Total Beam Length (ft) = 25.00
Mp (kip-ft) = 32.70

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL
1	0.000	0.025	0.000
	25.000	0.025	0.000

load entered as DL since seismic is ultimate

SHEAR (Ultimate): Max Vu (1.4DL) = 0.44 kips 0.90Vn = 69.45 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.4DL	2.7	12.5	25.0	1.14	0.90	29.43
Controlling		1.4DL	2.7	12.5	25.0	1.14	0.90	29.43

REACTIONS (kips):

	Left	**Right**
DL reaction	0.31	0.31
Max +total reaction (factored)	0.44	0.44

DEFLECTIONS:

Dead load (in)	at	12.50 ft	=	-0.363	L/D	=	828
Live load (in)	at	12.50 ft	=	-0.000			
Net Total load (in)	at	12.50 ft	=	-0.363	L/D	=	828

Project OR 1050 18

Date

By

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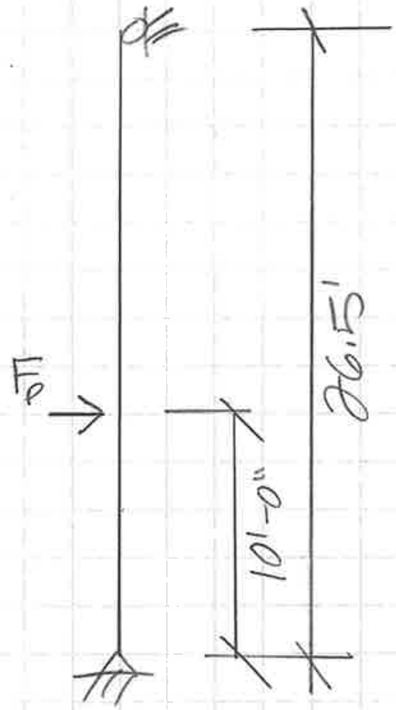
NEW COLUMNS



$F_p \cong (25 \text{ plf})\left(\frac{26'}{2}\right) = 325\#$

HSS 4x4x1/4 ← SEE FOLLOWING PAGES

Gravity Beam Design

RAM SBeam v3.0
Licensed to: Advanced Structural Tech 07/08/22 11:09:38

STEEL CODE: AISC LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (26.50,0.00)

Beam Size (User Selected)	=	HSS4X4X1/4	Fy	=	46.0 ksi
Total Beam Length (ft)	=	26.50			

Mp (kip-ft) = 17.98

POINT LOADS (kips):

Dist (ft)	DL	LL	Flange Bracing Top	Bottom
10.000	0.33	0.00	No	No

SHEAR (Ultimate): Max Vu (1.4DL) = 0.28 kips 0.90Vn = 46.30 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.4DL	2.8	10.0	26.5	1.40	0.90	16.18
Controlling		1.4DL	2.8	10.0	26.5	1.40	0.90	16.18

REACTIONS (kips):

	Left	**Right**
DL reaction	0.20	0.12
Max +total reaction (factored)	0.28	0.17

DEFLECTIONS:

Dead load (in)	at	12.32 ft	=	-0.888	L/D	=	358
Live load (in)	at	12.32 ft	=	-0.000			
Net Total load (in)	at	12.32 ft	=	-0.888	L/D	=	358