ECLIPSE ENGINEERING

Reviewed 2/23/2023 DL
Subject to field inspectors approvals.

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

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

Structural Calculations

Steel Stockroom Shelving

By PIPP Mobile Storage Systems Inc.

PIPP PO #67120 SO #140336

Bath & Body Works #737 – Remote Storage

South Hill Mall
3500 S. Meridian Street – Space #142
Puyallup, Washington 98373

FEB 01 2023

ROLF ARMSTRONG
STATE OF WASHINGTON
39125
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

Digitally signed by Rolf Henry Armstrong
DN: E=rarmstrong@eeimt.com, CN=Rolf Henry Armstrong, O="Eclipse Engineering, P.C.", L=Bend, S=Oregon, C=US
Date: 2023.02.01 18:24:22-08'00'

Prepared For:

PIPP Mobile Storage Systems Inc.
2966 Wilson Drive NW
Walker, MI 49534

Please note the calculations contained within justify the seismic resistance of the shelving for both vertical and lateral forces as required by the 2018 IBC, ASCE 7-16, and ANSI/RMI-MH16.1-2012. These storage shelves are not accessible to the general public.

PRCTI20230176

PIPP MOBILE STORAGE SYSTEMS INC.

STEEL STORAGE SHELVING - LIGHT RETAIL

CODES: Current Editions of the: IBC & CBC & ASCE 7 & RMI

Design Inputs: Steel Storage Shelving: MOBILE UNITS

Shelving Geometry -

Height of Shelving Unit =	10.0	ft	Steel Yield Stress =	33	ksi
Width of Shelving Unit =	4.0	ft	Modulus of Elast. =	29000	ksi
Depth of Shelving Unit =	2.5	ft	Eff. Lx Factor =	1.7	
Number of Shelves/Unit =	6		Unbraced Length,x =	22.5	in
Vertical Shelf Spacing =	22.5	in	Unbraced Length,y =	22.5	in
Back to Back Unit?	NO		Type of Post?	14ga Upright Posts	
Unit Type:	MOBILE		Type of Beam?	DRZ Z-Beam	
Number of Units per Track?	1		Top Shelf Loaded?	YES	
Mobile Anchor Spacing?	22.50	in	Intermediate Anchor:	Double	
Wall Supported Unit?	NO				

Shelving Loading -

Live Load per Shelf =	16.00	psf	Display On Plaque Near Shelving Units
Maximum Weight per Shelf =	160	lbs	Per 48.00 in. x 30.00 in. shelf
Dead Load per Shelf =	2.5	psf	Particle Board Shelf Material
Weight of Each Post =	7.4	lbs	Shelving is NOT accessible to public
Weight of Mobile Carriage =	50	lbs	

Floor Load Calculations:

Total Load on Each Post =	285	lbs	Ground Flr Conc Slab
Total Load On Each Unit =	1190	lbs	2500 psi NWC Concrete
Floor Area Load =	12.0	ft2	3/8"ϕ KB-TZ2 w/ 2.000" Embedment
Allowable Floor Loading =	100	psf	
Floor Load Under Shelf =	99	psf	OK FOR 100psf RETAIL FLOOR LOADING

Seismic Information -

Risk Category =	II	Not Open to the Public	SDC: D
Seismic Importance Factor (I_E) =	1.0		
Site Class =	D	– Default	Worst Case Assumed

Mapped Accel. Parameters: ρ = 1.3

Ss = 1.264	Fa = 1.200	Sms = 1.517	Sds = 1.011
S1 = 0.436	Fv = 1.864	Sm1 = 0.813	Sd1 = 0.542

Structural System: ASCE 7 Section 15.5.1

Steel Storage Shelving: R = 4 ap = 2.5 Ip = 1.0

Average Roof Height =	20	ft	0'-0" For Ground Floor Location
Height of Base Attachment =	0	ft	Ground Floor

Shear Coeff Boundaries =	Vmin = 0.044	RMI, 2.6.3
	Vmax = 0.253	RMI, 2.6.3
Design Base Shear Coeff =	V_t = 0.230	Adjusted For ASD RMI, 2.6.3

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Lateral Force Distribution: per ASCE 7 Section 15.5.1

Total Dead Load per Level =	29.9	lbs
Total Live Load per Level =	160	lbs
Lateral DL Force per Level =	6.9	lbs
Lateral LL Force per Level =	36.8	lbs
67% of LL Force per Level =	24.7	lbs
Total DL Base Shear =	41.3	lbs
Total LL Base Shear =	220.8	lbs

LC1: Each Level, Loaded to 67% of its Live Weight — Cumulative Moment: 49570 in-lbs

Total Base Shear = 189.3 lbs — LC #1 Governs

LC 2: Top Level Only, Loaded to 100% of its Live Weight — Cumulative Moment: 29457 in-lbs

Total Base Shear = 78.1 lbs — LC #2 Does NOT Govern

	Load Case #1:		Load Case #2:		Lateral Force/Shelf:		
Shelf Heights:	Load:	% Per Shelf:	Load:	% Per Shelf:	Force #:	LC #1:	LC #2:
h1 = 4 in	137 lbs	1.1%	30 lbs	0.4%	F1 =	2.1 lbs	0.3 lbs
h2 = 27 in	137 lbs	7.3%	30 lbs	2.7%	F2 =	13.9 lbs	2.1 lbs
h3 = 49 in	137 lbs	13.6%	30 lbs	5.0%	F3 =	25.7 lbs	3.9 lbs
h4 = 72 in	137 lbs	19.8%	30 lbs	7.3%	F4 =	37.4 lbs	5.7 lbs
h5 = 94 in	137 lbs	26.0%	30 lbs	9.5%	F5 =	49.2 lbs	7.5 lbs
h6 = 117 in	137 lbs	32.2%	190 lbs	75.1%	F6 =	61.0 lbs	58.7 lbs
h7 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F7 =	0.0 lbs	0.0 lbs
h8 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F8 =	0.0 lbs	0.0 lbs
h9 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F9 =	0.0 lbs	0.0 lbs
h10 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F10 =	0.0 lbs	0.0 lbs
h11 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F11 =	0.0 lbs	0.0 lbs
h12 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F12 =	0.0 lbs	0.0 lbs
h13 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F13 =	0.0 lbs	0.0 lbs
h14 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F14 =	0.0 lbs	0.0 lbs
h15 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F15 =	0.0 lbs	0.0 lbs
h16 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F16 =	0.0 lbs	0.0 lbs
h17 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F17 =	0.0 lbs	0.0 lbs
h18 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F18 =	0.0 lbs	0.0 lbs
h19 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F19 =	0.0 lbs	0.0 lbs
h20 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F20 =	0.0 lbs	0.0 lbs
		Sum = 100%		Sum = 100%	Total =	189.3 lbs	78.1 lbs

By inspection, the force distribution for intermediate level without live load (case 2) is negligible. Calculate the moment for each column based on the total seismic base shear for each shelf being loaded to 67% of it's allowable live weight. The column at the center of the shelving system is the worst case for this condition.

Column Calculations - Combined Bending and Axial

Post Type: **Double Rivet "L" or "T" Post**

Width =	1.5	in	r_x =	0.470	in
Depth =	1.5	in	S_x =	0.044	in3
Thickness =	0.075	in	I_x =	0.049	in4
			A_p =	0.217	in2

Column Bending Calculations -

Max Column Moment =	43.9	ft-lbs	
Allowable Bending Stress =	19.8	ksi	
Bending Stress on Column =	11.9	ksi	**Bending Stress OK**

Column Deflection Calculations -

Max Deflection =	0.457	in	At Top of Unit
Deflection Ratio =	263		L/Δ
Allowable Deflection =	6	in	Max Deflection = 5% of Height
			Deflection OK

Column Axial Calculations - Per "L" Post

DL + PL =	285	lbs	RMI Load Combination #1
DL + PL + EQ =	414	lbs	RMI Load Combination #6

Column Capacity Calculations -

Controlling Buckling Stress =	9.6	ksi	
Allowable Comp. Stress =	9.6	ksi	
Factor of Safety for Comp. =	1.80		
Nominal Column Capacity =	2068	lbs	
Allowable Column Capacity =	1149	lbs	
Static Axial Load on Column =	285	lbs	**Axial Load OK**

Combined Bending And Axial Forces -

Critical Buckling Load =	9527	lbs		
Axial Stress Unity =	0.360		Magnification Factor =	0.946
Bending Stress Unity =	0.539		C_m =	0.85

Combined Stress Unity = 0.900

Column is Adequate

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Overturning and Anti-Tip Calculations

Overturning Forces On Anchors (LRFD)

Overstrength Factor, Ω = 2.00

Load Combination	Total Weight, W (lbs)	Base Shear, Eh (lbs)	Vertical Seismic Force, Ev (lbs)	Overturning Moment (Ω=2), Mo (ft-lbs)	Resisting Moment, Mr (ft-lbs)	Net Overturning Moment, Mn (ft-lbs)	Shear Force per Anchor, V (lbs)	Tension Force per Anchor, T (lbs)
LC1	823	208	166	1469	718	751	**104**	**301**
LC2	340	86	69	750	296	454	**43**	**182**

Per Side of Unit

$W(LC1) = \left(DL_{Shelf} + 0.67 * LL_{Shelf}\right) * \#\,of\,Shelves$
$W(LC2) = DL_{Shelf} * \#\,of\,Shelves + LL_{Shelf}$
$Eh = (Vt * W)/(0.7 * \rho)$
$Ev = 0.20 * Sds * W$
$Mo(LC1) = \Omega * \sum(hx * fx/0.7)$
$Mo(LC2) = \Omega * Vt/0.7 * \left(DL_{Total} * \left(\frac{H}{2} + S\right) + LL_{Shelf} * H\right)$

$Mr = (0.9 * W - Ev) * \frac{d}{2}$
$Mn = Mo - Mr$
$V = \frac{\Omega * Eh}{\#\,of\,Anchors}$
$T = \frac{\frac{Mn}{d}}{\#\,of\,Anchors}$

USE: POST INSTALLED ANCHOR BOLTS / LAG SCREWS AS REQUIRED FOR FLOOR NOTED BELOW

Fixed Units - Allowable Loads
Allowable Tension Force = 1327 lbs
Allowable Shear Force = 1165 lbs

2500 psi NWC Concrete
3/8"ϕ KB-TZ2 w/ 2.000" Embedment

Fixed Units - Combined Loading — LC1: 0.227; LC2: 0.137

Floor anchors are adequate

Mobile Unit Anchors - LC1: 0.415; LC2: 0.251

Mobile Units - Allowable Loads
Allowable Tension Force = 1448 lbs
Allowable Shear Force = 1541 lbs

Floor anchors are adequate

Anti-Tip Track Design -

Type of Anti-Tip Device = **Arm and Track**
Tension per Carriage Anchor = 601 lbs
Combined Loading = 0.415 — **Floor anchors are adequate**
Tension per Shelf Post = 346 lbs
Capacity of Screws to Carriage = 1200 lbs — **(2) 1/4-in screws are adequate**

Anti-Tip Peg Yield Stress = 40.275 ksi — Steel Fy = 53.700 ksi
Thickness Anti-Tip Peg Head = 0.09 in — Fu = 63.800 ksi
Width of Anti-Tip Peg Head = 0.43 in
Section Modulus of Peg Head = 0.0006 in3

Allowable Stress on Leg = 40.275 ksi
Bending Stress on Leg = 25.71 ksi
Anti-Tip Stress Unity = 0.638 — **Bending Stress OK**

Section Modulus of Track = 0.093 in3
Spacing of Track A.B's = 22.50 in

Allowable Alumn. Stress = 21 ksi — 6061-T6 Fty = 35.000 ksi
Bending Stress on Track = 10.47 ksi — Ftu = 38.000 ksi
Track Stress Unity = 0.498 — **Bending Stress OK**

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Shelf Beam Calculations

Shelf Beam Calculations: **DRZ Z-Beam**

Steel Yield Stress =	33	ksi	Shelf DL =	2.5	psf
Modulus of Elast. =	29000	ksi	Shelf LL =	16.00	psf

Beam Type: DRZ Z-Beam

Area of Beam =	0.234	in2
Section Modulus of Beam =	0.054	in3
Moment of Inertia of Beam =	0.058	in4

Shelf Width =	4.0	ft	Allowable Bending Stress =	19.8	ksi
Shelf Depth =	2.5	ft	Allowable Shear Stress =	13.2	ksi
Total Load/Shelf =	185	lbs			
Distributed Load =	23.125	plf			

Maximum Design Moment =	46.3	ft-lbs	
Maximum Design Shear =	46.3	lbs	
Beam Bending Stress =	10.3	ksi	
Bending Stress Unity =	0.519		**Bending Stress OK**
Beam Shear Stress =	0.20	ksi	
Shear Stress Unity =	0.015		**Shear Stress OK**
Max Allowable Deflection =	0.267	in	L/180
Maximum Beam Deflection =	0.079	in	**Deflection OK**

Shelf Beam Rivet Check:

Diameter of Rivet =	0.25	in	
Post Moment Shear on Rivet =	351.0	lbs	
Beam Shear on Rivet =	46.3	lbs	
Resultant Shear =	354.0	lbs	
Bearing Capacity of Rivet =	519.8	lbs	**Bearing Stress OK**
Allowable Shear Stress =	13.5	ksi	
Shear Stress on Rivet =	7.2	ksi	**Shear Stress OK**

Seismic Uplift on Shelves -

Vertical Seismic Component =	37.4	lbs	
Vertical Total Load per Shelf =	137.1	lbs	
Connection Points per Shelf =	4.0	(1) per Corner	
Net Uplift Load per Shelf =	-44.9	lbs	
Uplift Forcer per Connection =	-11.2	lbs	**Rivet Connection OK**

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Slab Bearing & Uplift Calculations

Slab Design Properties -

Minimum Concrete Strength =	2500	psi	Assumed
Thickness of Concrete Slab =	4	in	Assumed
Weight of Concrete Slab =	50	psf	
Allowable Bearing Pressure =	500	psf	Assumed
Bearing Loads On Post =	45	lbs	Dead Load
	240	lbs	Live Load
	294	lbs	EQ Load
Uplift Loads on Post =	358	lbs	Resultant Uplift

Slab Bearing Capacity -

Depth of Post on Slab =	1.5	in	
Factored Bearing Load =	858	lbs	
Required Bearing Area =	166.66	in2	12.91 inches per side
Critical Section =	3.70	in	For Bending
Soil Pressure on Crit. Section =	741.0	plf	Along Critical Length
Section Modulus =	32.0	in3	Plain Concrete per Foot
Shear Area =	22	in	
Conc. Shear Stress =	9.7	psi	
Allowable Shear Stress =	73.2	psi	**Shear Stress OK**
Conc. Bending Stress =	13.2	psi	
Allowable Bending Stress =	137.5	psi	**Bending Stress OK**

Slab Uplift Capacity -

Required Area to Resist Uplift =	6.68	ft2	
Length of Slab Req'd =	0.83	ft	Assume Required Area / Full Shelf Width
Worst Case Length of Slab =	1.25	ft	Maximum Length Required
Distance to Anchor Bolt =	0.63	ft	
Length of 1ft Strip =	1.25	ft	Length Safety Factor: 2.00
Shear Force on 1ft Strip =	87.5	lbs	
Allowable Shear Force =	1760.0	lbs	**Shear OK**
Bending Moment on 1ft Strip =	13.7	ft-lbs	
Allowable Bending Moment =	366.7	ft-lbs	**Bending OK**

PIPP MOBILE STORAGE SYSTEMS INC.

STEEL STORAGE SHELVING - LIGHT RETAIL

CODES: Current Editions of the: IBC & CBC & ASCE 7 & RMI

Design Inputs: Steel Storage Shelving: FIXED UNITS - 30" DEEP

Shelving Geometry -

Height of Shelving Unit =	10.0	ft	Steel Yield Stress =	33	ksi
Width of Shelving Unit =	4.0	ft	Modulus of Elast. =	29000	ksi
Depth of Shelving Unit =	2.5	ft	Eff. Lx Factor =	1.7	
Number of Shelves/Unit =	5		Unbraced Length,x =	21.0	in
Average Vertical Shelf Spacing =	22.5	in	Unbraced Length,y =	21.0	in
Back to Back Unit?	NO		Type of Post?	14ga Upright Posts	
Unit Type:	FIXED		Type of Beam?	DRZ Z-Beam	
Number of Units per Track?	NA		Top Shelf Loaded?	YES	
Mobile Anchor Spacing?		in	Intermediate Anchor:	Double	
Wall Supported Unit?	YES				

Shelving Loading -

Live Load per Shelf =	20.75	psf	Display On Plaque Near Shelving Units
Maximum Weight per Shelf =	205	lbs	Per 48.00 in. x 30.00 in. shelf
Dead Load per Shelf =	2.5	psf	Particle Board Shelf Material
Weight of Each Post =	7.4	lbs	Shelving is NOT accessible to public
Weight of Mobile Carriage =	0	lbs	

Floor Load Calculations:

Total Load on Each Post =	295	lbs	Ground Flr Conc Slab
Total Load On Each Unit =	1180	lbs	2500 psi NWC Concrete
Floor Area Load =	12.0	ft2	3/8"ϕ KB-TZ2 w/ 2.000" Embedment
Allowable Floor Loading =	100	psf	
Floor Load Under Shelf =	98	psf	OK FOR 100psf RETAIL FLOOR LOADING

Seismic Information -

Risk Category = II Not Open to the Public SDC: D

Seismic Importance Factor (I_E) = 1.0

Site Class = D – Default Worst Case Assumed

Mapped Accel. Parameters: ρ = 1.3

Ss = 1.264	Fa = 1.200	Sms = 1.517	Sds = 1.011
S1 = 0.436	Fv = 1.864	Sm1 = 0.813	Sd1 = 0.542

Structural System: ASCE 7 Section 15.5.1

Steel Storage Shelving: R = 4 ap = 2.5 Ip = 1.0

Average Roof Height =	20	ft	0'-0" For Ground Floor Location
Height of Base Attachment =	0	ft	Ground Floor

Shear Coeff Boundaries = Vmin = 0.044 RMI, 2.6.3

Vmax = 0.253 RMI, 2.6.3

Design Base Shear Coeff = V_t = 0.230 Adjusted For ASD RMI, 2.6.3

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Lateral Force Distribution: per ASCE 7 Section 15.5.1

Total Dead Load per Level =	30.9	lbs
Total Live Load per Level =	205	lbs
Lateral DL Force per Level =	7.1	lbs
Lateral LL Force per Level =	47.2	lbs
67% of LL Force per Level =	31.6	lbs
Total DL Base Shear =	35.6	lbs
Total LL Base Shear =	235.8	lbs

LC1: Each Level, Loaded to 67% of its Live Weight — Cumulative Moment: 59563 in-lbs

Total Base Shear = 193.5 lbs — LC #1 Governs

LC 2: Top Level Only, Loaded to 100% of its Live Weight — Cumulative Moment: 35541 in-lbs

Total Base Shear = 82.7 lbs — LC #2 Does NOT Govern

	Load Case #1:		Load Case #2:		Lateral Force/Shelf:		
Shelf Heights:	Load:	% Per Shelf:	Load:	% Per Shelf:	Force #:	LC #1:	LC #2:
h1 = 24 in	168 lbs	6.8%	31 lbs	2.1%	F1 =	13.1 lbs	1.7 lbs
h2 = 47 in	168 lbs	13.3%	31 lbs	4.1%	F2 =	25.7 lbs	3.4 lbs
h3 = 70 in	168 lbs	19.8%	31 lbs	6.1%	F3 =	38.3 lbs	5.0 lbs
h4 = 93 in	168 lbs	26.3%	31 lbs	8.1%	F4 =	50.8 lbs	6.7 lbs
h5 = 120 in	168 lbs	33.9%	236 lbs	79.7%	F5 =	65.6 lbs	65.9 lbs
h6 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F6 =	0.0 lbs	0.0 lbs
h7 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F7 =	0.0 lbs	0.0 lbs
h8 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F8 =	0.0 lbs	0.0 lbs
h9 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F9 =	0.0 lbs	0.0 lbs
h10 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F10 =	0.0 lbs	0.0 lbs
h11 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F11 =	0.0 lbs	0.0 lbs
h12 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F12 =	0.0 lbs	0.0 lbs
h13 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F13 =	0.0 lbs	0.0 lbs
h14 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F14 =	0.0 lbs	0.0 lbs
h15 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F15 =	0.0 lbs	0.0 lbs
h16 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F16 =	0.0 lbs	0.0 lbs
h17 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F17 =	0.0 lbs	0.0 lbs
h18 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F18 =	0.0 lbs	0.0 lbs
h19 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F19 =	0.0 lbs	0.0 lbs
h20 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F20 =	0.0 lbs	0.0 lbs
		Sum = 100%		Sum = 100%	Total =	193.5 lbs	82.7 lbs

By inspection, the force distribution for intermediate level without live load (case 2) is negligible. Calculate the moment for each column based on the total seismic base shear for each shelf being loaded to 67% of it's allowable live weight. The column at the center of the shelving system is the worst case for this condition.

Column Calculations - Combined Bending and Axial

Post Type: **Double Rivet "L" or "T" Post**

Width =	1.5	in	r_x =	0.470	in
Depth =	1.5	in	S_x =	0.044	in3
Thickness =	0.075	in	I_x =	0.049	in4
			A_p =	0.217	in2

Column Bending Calculations -

Max Column Moment =	45.1	ft-lbs	
Allowable Bending Stress =	19.8	ksi	
Bending Stress on Column =	12.2	ksi	**Bending Stress OK**

Column Deflection Calculations -

Max Deflection =	0.418	in	At Top of Unit
Deflection Ratio =	287		L/Δ
Allowable Deflection =	6	in	Max Deflection = 5% of Height
			Deflection OK

Column Axial Calculations - Per "L" Post

DL + PL =	295	lbs	RMI Load Combination #1
DL + PL + EQ =	426	lbs	RMI Load Combination #6

Column Capacity Calculations -

Controlling Buckling Stress =	10.4	ksi	
Allowable Comp. Stress =	10.4	ksi	
Factor of Safety for Comp. =	1.80		
Nominal Column Capacity =	2247	lbs	
Allowable Column Capacity =	1248	lbs	
Static Axial Load on Column =	295	lbs	**Axial Load OK**

Combined Bending And Axial Forces -

Critical Buckling Load =	10937	lbs		
Axial Stress Unity =	0.341		Magnification Factor =	0.951
Bending Stress Unity =	0.551		C_m =	0.85

Combined Stress Unity = 0.892

Column is Adequate

PRCTI20230176

Overturning and Anti-Tip Calculations

Overturning Forces On Anchors (LRFD)

Overstrength Factor, Ω = 2.00

Load Combination	Total Weight, W (lbs)	Base Shear, Eh (lbs)	Vertical Seismic Force, Ev (lbs)	Overturning Moment (Ω=2), Mo (ft-lbs)	Resisting Moment, Mr (ft-lbs)	Net Overturning Moment, Mn (ft-lbs)	Shear Force per Anchor, V (lbs)	Tension Force per Anchor, T (lbs)
LC1	841	213	170	1539	734	805	**106**	**322**
LC2	360	91	73	832	314	518	**45**	**207**

Per Side of Unit

$W(LC1) = \left(DL_{Shelf} + 0.67 * LL_{Shelf}\right) * \#\ of\ Shelves$

$W(LC2) = DL_{Shelf} * \#\ of\ Shelves + LL_{Shelf}$

$Eh = (Vt * W)/(0.7 * \rho)$

$Ev = 0.20 * Sds * W$

$Mo(LC1) = \Omega * \sum(hx * fx/0.7)$

$Mo(LC2) = \Omega * Vt/0.7 * \left(DL_{Total} * \left(\frac{H}{2} + S\right) + LL_{Shelf} * H\right)$

$Mr = (0.9 * W - Ev) * \frac{d}{2}$

$Mn = Mo - Mr$

$V = \frac{\Omega * Eh}{\#\ of\ Anchors}$

$T = \frac{\frac{Mn}{d}}{\#\ of\ Anchors}$

USE: POST INSTALLED ANCHOR BOLTS / LAG SCREWS AS REQUIRED FOR FLOOR NOTED BELOW

Fixed Units - Allowable Loads
Allowable Tension Force = 1327 lbs
Allowable Shear Force = 1165 lbs

2500 psi NWC Concrete
3/8"ϕ KB-TZ2 w/ 2.000" Embedment

Fixed Units - Combined Loading: LC1: 0.243; LC2: 0.156

Floor anchors are adequate

Mobile Units - Combined Loading: LC1: N/A; LC2: N/A

Mobile Units - Allowable Loads
Allowable Tension Force = 1448 lbs
Allowable Shear Force = 1541 lbs

Anti-Tip Track Design -

N/A

Type of Anti-Tip Device =	**NONE**		
Tension per Carriage Anchor =	N/A	lbs	
Combined Loading =	N/A		**N/A**
Tension per Shelf Post =	N/A	lbs	
Capacity of Screws to Carriage =	N/A	lbs	**N/A**
Anti-Tip Peg Yield Stress =	40.275	ksi	Steel Fy = 53.700 ksi
Thickness Anti-Tip Peg Head =	0.09	in	Fu = 63.800 ksi
Width of Anti-Tip Peg Head =	0.43	in	
Section Modulus of Peg Head =	0.0006	in3	
Allowable Stress on Leg =	40.275	ksi	
Bending Stress on Leg =	N/A	ksi	
Anti-Tip Stress Unity =	N/A		**N/A**
Section Modulus of Track =	0.093	in3	
Spacing of Track A.B's =	0.00	in	
Allowable Alumn. Stress =	21	ksi	6061-T6 Fty = 35.000 ksi
Bending Stress on Track =	N/A	ksi	Ftu = 38.000 ksi
Track Stress Unity =	N/A		**N/A**

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Shelf Beam Calculations

Shelf Beam Calculations: **DRZ Z-Beam**

Steel Yield Stress =	33	ksi	Shelf DL =	2.5	psf
Modulus of Elast. =	29000	ksi	Shelf LL =	20.75	psf

Beam Type: DRZ Z-Beam

Area of Beam =	0.234	in2
Section Modulus of Beam =	0.054	in3
Moment of Inertia of Beam =	0.058	in4

Shelf Width =	4.0	ft	Allowable Bending Stress =	19.8	ksi
Shelf Depth =	2.5	ft	Allowable Shear Stress =	13.2	ksi
Total Load/Shelf =	233	lbs			
Distributed Load =	29.0625	plf			

Maximum Design Moment =	58.1	ft-lbs	
Maximum Design Shear =	58.1	lbs	
Beam Bending Stress =	12.9	ksi	
Bending Stress Unity =	0.652		**Bending Stress OK**
Beam Shear Stress =	0.25	ksi	
Shear Stress Unity =	0.019		**Shear Stress OK**
Max Allowable Deflection =	0.267	in	L/180
Maximum Beam Deflection =	0.100	in	**Deflection OK**

Shelf Beam Rivet Check:

Diameter of Rivet =	0.25	in	
Post Moment Shear on Rivet =	360.8	lbs	
Beam Shear on Rivet =	58.1	lbs	
Resultant Shear =	365.5	lbs	
Bearing Capacity of Rivet =	519.8	lbs	**Bearing Stress OK**
Allowable Shear Stress =	13.5	ksi	
Shear Stress on Rivet =	7.4	ksi	**Shear Stress OK**

Seismic Uplift on Shelves -

Vertical Seismic Component =	47.0	lbs	
Vertical Total Load per Shelf =	168.3	lbs	
Connection Points per Shelf =	4.0	(1) per Corner	
Net Uplift Load per Shelf =	-53.9	lbs	
Uplift Forcer per Connection =	-13.5	lbs	**Rivet Connection OK**

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Wall Supported Unit Calculations

Seismic Force at Top of Units -

Average Roof Height =	20.0	ft
Height of Attachment =	10.0	ft
Shear Coeff Boundaries =	V_{min} = 0.303	
	V_{max} = 1.618	
Design Base Shear Coeff =	V_t = 0.460	Adjusted For ASD and "ρ"
Total Weight per Unit =	841	lbs
Lateral Force at Top/Bottom =	194	lbs
Standard Stud Spacing =	16	in
Wall Connections per Unit =	3	
Tek Screw Capacity =	84	lbs — Tension Cap. for #10 Screw in 20ga Stud
Force Per Connection =	65	lbs — **Screw Capacity OK**

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Light Gauge Steel Stud Wall Framing

Stud Design Data -

Height of Wall Studs =	16.0	ft	Int. Non-Brg - Worst Case Ht Assumed
Location of Point Load =	10.0	ft	
Design Lateral Load =	64.5	lbs	From Shelving Unit
Additional Lateral Load =	5.0	psf	Interior Seismic Force
Design Axial Load =	85.3	lbs	Dead Load of Wall Framing
Spacing of Studs =	16.0	in	

TRY: 3-5/8" x 1-5/8" x 20ga Studs @ 16" o.c. (Worst Case Assumed)

Width =	3.625	in	rx =	1.450	in
Depth =	1.625	in	ry =	0.616	in
Thickness =	0.035	in	Sx =	0.268	in3
Fy =	33	ksi	Ix =	0.551	in4
E =	29000	ksi	Ap =	0.262	in2
K =	1.0		Unbraced Length X =	16	ft
			Unbraced Length Y =	4	ft

Stud Capacity -

Buckling Stress, X =	16.32	ksi	
Buckling Stress, Y =	47.14	ksi	
Allowable Buckling Stress =	16.32	ksi	
Nominal Axial Strength =	4277	lbs	
Factor of Safety =	1.92		
Allowable Axial Load =	2228	lbs	
Maximum Design Moment =	455.3	ft-lbs	
Maximum Design Shear =	93.7	lbs	
Allowable Bending Stress =	21.78	ksi	
Actual Bending Stress =	20.38	ksi	**Bending Stress OK**
Allowable Shear Stress =	13.20	ksi	
Actual Shear Stress =	0.36	ksi	**Shear Stress OK**
Allowable Axial Stress =	8.50	ksi	
Actual Axial Stress =	0.33	ksi	**Axial Stress OK**
Combined Stress Unity =	0.97		**Combined Stress OK**

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Slab Bearing & Uplift Calculations

Slab Design Properties -

Minimum Concrete Strength =	2500	psi	Assumed	
Thickness of Concrete Slab =	4	in	Assumed	
Weight of Concrete Slab =	50	psf		
Allowable Bearing Pressure =	500	psf	Assumed	
Bearing Loads On Post =	39	lbs	Dead Load	
	256	lbs	Live Load	
	308	lbs	EQ Load	
Uplift Loads on Post =	381	lbs	Resultant Uplift	

Slab Bearing Capacity -

Depth of Post on Slab =	1.5	in		
Factored Bearing Load =	896	lbs		
Required Bearing Area =	173.56	in2	13.17	inches per side
Critical Section =	3.84	in	For Bending	
Soil Pressure on Crit. Section =	743.4	plf	Along Critical Length	
Section Modulus =	32.0	in3	Plain Concrete per Foot	
Shear Area =	22	in		
Conc. Shear Stress =	10.2	psi		
Allowable Shear Stress =	73.2	psi	**Shear Stress OK**	
Conc. Bending Stress =	14.3	psi		
Allowable Bending Stress =	137.5	psi	**Bending Stress OK**	

Slab Uplift Capacity -

Required Area to Resist Uplift =	0.00	ft2		
Length of Slab Req'd =	0.00	ft	Assume Required Area / Full Shelf Width	
Worst Case Length of Slab =	1.25	ft	Maximum Length Required	
Distance to Anchor Bolt =	0.63	ft		
Length of 1ft Strip =	1.25	ft	Length Safety Factor:	2.00
Shear Force on 1ft Strip =	87.5	lbs		
Allowable Shear Force =	1760.0	lbs	**Shear OK**	
Bending Moment on 1ft Strip =	13.7	ft-lbs		
Allowable Bending Moment =	366.7	ft-lbs	**Bending OK**	

PRCTI20230176

PIPP MOBILE STORAGE SYSTEMS INC.

STEEL STORAGE SHELVING - LIGHT RETAIL

CODES: Current Editions of the: IBC & CBC & ASCE 7 & RMI

Design Inputs: Steel Storage Shelving: FIXED UNITS - 18" DEEP

Shelving Geometry -

Height of Shelving Unit =	10.0 ft	Steel Yield Stress =	33	ksi
Width of Shelving Unit =	4.0 ft	Modulus of Elast. =	29000	ksi
Depth of Shelving Unit =	1.5 ft	Eff. Lx Factor =	1.7	
Number of Shelves/Unit =	5	Unbraced Length,x =	22.5	in
Average Vertical Shelf Spacing =	22.5 in	Unbraced Length,y =	22.5	in
Back to Back Unit?	NO	Type of Post?	14ga Upright Posts	
Unit Type:	FIXED	Type of Beam?	DRL Low Profile	
Number of Units per Track?	NA	Top Shelf Loaded?	YES	
Mobile Anchor Spacing?	in	Intermediate Anchor:	Double	
Wall Supported Unit?	YES			

Shelving Loading -

Live Load per Shelf =	23.00	psf	Display On Plaque Near Shelving Units
Maximum Weight per Shelf =	135	lbs	Per 48.00 in. x 18.00 in. shelf
Dead Load per Shelf =	2.5	psf	Particle Board Shelf Material
Weight of Each Post =	7.4	lbs	Shelving is NOT accessible to public
Weight of Mobile Carriage =	0	lbs	

Floor Load Calculations:

Total Load on Each Post =	195	lbs	Ground Flr Conc Slab
Total Load On Each Unit =	780	lbs	2500 psi NWC Concrete
Floor Area Load =	8.0	ft2	3/8"ϕ KB-TZ2 w/ 2.000" Embedment
Allowable Floor Loading =	100	psf	
Floor Load Under Shelf =	**97**	**psf**	OK FOR 100psf RETAIL FLOOR LOADING

Seismic Information -

Risk Category = II Not Open to the Public SDC: D

Seismic Importance Factor (I_E) = 1.0

Site Class = D – Default Worst Case Assumed

Mapped Accel. Parameters: ρ = 1.3

Ss = 1.264	Fa = 1.200	Sms = 1.517	**Sds = 1.011**
S1 = 0.436	Fv = 1.864	Sm1 = 0.813	**Sd1 = 0.542**

Structural System: ASCE 7 Section 15.5.1

Steel Storage Shelving: R = 4 ap = 2.5 Ip = 1.0

Average Roof Height = 20 ft 0'-0" For Ground Floor Location

Height of Base Attachment = 0 ft Ground Floor

Shear Coeff Boundaries = Vmin = 0.044 RMI, 2.6.3

Vmax = 0.253 RMI, 2.6.3

Design Base Shear Coeff = V_t = 0.230 Adjusted For ASD RMI, 2.6.3

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Lateral Force Distribution: per ASCE 7 Section 15.5.1

Total Dead Load per Level =	20.9	lbs
Total Live Load per Level =	135	lbs
Lateral DL Force per Level =	4.8	lbs
Lateral LL Force per Level =	31.1	lbs
67% of LL Force per Level =	20.8	lbs
Total DL Base Shear =	24.0	lbs
Total LL Base Shear =	155.3	lbs

LC1: Each Level, Loaded to 67% of its Live Weight — Cumulative Moment: 39420 in-lbs

Total Base Shear = 128.1 lbs — LC #1 Governs

LC 2: Top Level Only, Loaded to 100% of its Live Weight — Cumulative Moment: 23601 in-lbs

Total Base Shear = 55.1 lbs — LC #2 Does NOT Govern

	Load Case #1:		Load Case #2:		Lateral Force/Shelf:		
Shelf Heights:	Load:	% Per Shelf:	Load:	% Per Shelf:	Force #:	LC #1:	LC #2:
h1 = 24 in	111 lbs	6.8%	21 lbs	2.1%	F1 =	8.7 lbs	1.2 lbs
h2 = 47 in	111 lbs	13.3%	21 lbs	4.2%	F2 =	17.0 lbs	2.3 lbs
h3 = 70 in	111 lbs	19.8%	21 lbs	6.2%	F3 =	25.3 lbs	3.4 lbs
h4 = 93 in	111 lbs	26.3%	21 lbs	8.2%	F4 =	33.7 lbs	4.5 lbs
h5 = 120 in	111 lbs	33.9%	156 lbs	79.3%	F5 =	43.4 lbs	43.7 lbs
h6 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F6 =	0.0 lbs	0.0 lbs
h7 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F7 =	0.0 lbs	0.0 lbs
h8 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F8 =	0.0 lbs	0.0 lbs
h9 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F9 =	0.0 lbs	0.0 lbs
h10 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F10 =	0.0 lbs	0.0 lbs
h11 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F11 =	0.0 lbs	0.0 lbs
h12 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F12 =	0.0 lbs	0.0 lbs
h13 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F13 =	0.0 lbs	0.0 lbs
h14 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F14 =	0.0 lbs	0.0 lbs
h15 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F15 =	0.0 lbs	0.0 lbs
h16 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F16 =	0.0 lbs	0.0 lbs
h17 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F17 =	0.0 lbs	0.0 lbs
h18 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F18 =	0.0 lbs	0.0 lbs
h19 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F19 =	0.0 lbs	0.0 lbs
h20 = 0 in	0 lbs	0.0%	0 lbs	0.0%	F20 =	0.0 lbs	0.0 lbs
		Sum = 100%		Sum = 100%	Total =	128.1 lbs	55.1 lbs

By inspection, the force distribution for intermediate level without live load (case 2) is negligible. Calculate the moment for each column based on the total seismic base shear for each shelf being loaded to 67% of it's allowable live weight. The column at the center of the shelving system is the worst case for this condition.

Column Calculations - Combined Bending and Axial

Post Type: **Double Rivet "L" or "T" Post**

Width =	1.5	in	r_x =	0.470	in
Depth =	1.5	in	S_x =	0.044	in3
Thickness =	0.075	in	I_x =	0.049	in4
			A_p =	0.217	in2

Column Bending Calculations -

Max Column Moment =	29.9	ft-lbs	
Allowable Bending Stress =	19.8	ksi	
Bending Stress on Column =	8.1	ksi	**Bending Stress OK**

Column Deflection Calculations -

Max Deflection =	0.277	in	At Top of Unit
Deflection Ratio =	434		L/Δ
Allowable Deflection =	6	in	Max Deflection = 5% of Height
			Deflection OK

Column Axial Calculations - Per "L" Post

DL + PL =	195	lbs	RMI Load Combination #1
DL + PL + EQ =	342	lbs	RMI Load Combination #6

Column Capacity Calculations -

Controlling Buckling Stress =	9.6	ksi	
Allowable Comp. Stress =	9.6	ksi	
Factor of Safety for Comp. =	1.80		
Nominal Column Capacity =	2068	lbs	
Allowable Column Capacity =	1149	lbs	
Static Axial Load on Column =	195	lbs	**Axial Load OK**

Combined Bending And Axial Forces -

Critical Buckling Load =	9527	lbs		
Axial Stress Unity =	0.298		Magnification Factor =	0.963
Bending Stress Unity =	0.360		C_m =	0.85

Combined Stress Unity = 0.658

Column is Adequate

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Overturning and Anti-Tip Calculations

Overturning Forces On Anchors (LRFD)

Overstrength Factor, Ω = 2.00

Load Combination	Total Weight, W (lbs)	Base Shear, Eh (lbs)	Vertical Seismic Force, Ev (lbs)	Overturning Moment (Ω=2), Mo (ft-lbs)	Resisting Moment, Mr (ft-lbs)	Net Overturning Moment, Mn (ft-lbs)	Shear Force per Anchor, V (lbs)	Tension Force per Anchor, T (lbs)
LC1	557	141	113	1018	291	727	**70**	**485**
LC2	240	61	48	553	125	428	**30**	**285**

Per Side of Unit

$W(LC1) = \left(DL_{Shelf} + 0.67 * LL_{Shelf}\right) * \#\ of\ Shelves$
$W(LC2) = DL_{Shelf} * \#\ of\ Shelves + LL_{Shelf}$
$Eh = (Vt * W)/(0.7 * \rho)$
$Ev = 0.20 * Sds * W$
$Mo(LC1) = \Omega * \sum(hx * fx/0.7)$
$Mo(LC2) = \Omega * Vt/0.7 * \left(DL_{Total} * \left(\frac{H}{2} + S\right) + LL_{Shelf} * H\right)$

$Mr = (0.9 * W - Ev) * \frac{d}{2}$
$Mn = Mo - Mr$
$V = \frac{\Omega * Eh}{\#\ of\ Anchors}$
$T = \frac{\frac{Mn}{d}}{\#\ of\ Anchors}$

USE: POST INSTALLED ANCHOR BOLTS / LAG SCREWS AS REQUIRED FOR FLOOR NOTED BELOW

Fixed Units - Allowable Loads
Allowable Tension Force = 1327 lbs
Allowable Shear Force = 1165 lbs

2500 psi NWC Concrete
3/8"ϕ KB-TZ2 w/ 2.000" Embedment

Fixed Units - Combined Loading		
	LC1:	0.365
	LC2:	0.215

Floor anchors are adequate

Mobile Units - Allowable Loads
Allowable Tension Force = 1448 lbs
Allowable Shear Force = 1541 lbs

Mobile Units - Combined Loading		
	LC1:	N/A
	LC2:	N/A

N/A

Anti-Tip Track Design -

Type of Anti-Tip Device =	**NONE**		
Tension per Carriage Anchor =	N/A	lbs	
Combined Loading =	N/A		**N/A**
Tension per Shelf Post =	N/A	lbs	
Capacity of Screws to Carriage =	N/A	lbs	**N/A**
Anti-Tip Peg Yield Stress =	40.275	ksi	Steel: Fy = 53.700 ksi
Thickness Anti-Tip Peg Head =	0.09	in	Fu = 63.800 ksi
Width of Anti-Tip Peg Head =	0.43	in	
Section Modulus of Peg Head =	0.0006	in3	
Allowable Stress on Leg =	40.275	ksi	
Bending Stress on Leg =	N/A	ksi	
Anti-Tip Stress Unity =	N/A		**N/A**
Section Modulus of Track =	0.093	in3	
Spacing of Track A.B's =	0.00	in	
Allowable Alumn. Stress =	21	ksi	6061-T6: Fty = 35.000 ksi
Bending Stress on Track =	N/A	ksi	Ftu = 38.000 ksi
Track Stress Unity =	N/A		**N/A**

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Shelf Beam Calculations

Shelf Beam Calculations: **DRL Low Profile**

Steel Yield Stress =	33	ksi	Shelf DL =	2.5	psf
Modulus of Elast. =	29000	ksi	Shelf LL =	23.00	psf

Beam Type: DRL Low Profile

Area of Beam =	0.264	in2
Section Modulus of Beam =	0.098	in3
Moment of Inertia of Beam =	0.072	in4

Shelf Width =	4.0	ft	Allowable Bending Stress =	19.8	ksi
Shelf Depth =	1.5	ft	Allowable Shear Stress =	13.2	ksi
Total Load/Shelf =	153	lbs			
Distributed Load =	19.125	plf			

Maximum Design Moment =	38.3	ft-lbs	
Maximum Design Shear =	38.3	lbs	
Beam Bending Stress =	4.7	ksi	
Bending Stress Unity =	0.238		**Bending Stress OK**
Beam Shear Stress =	0.14	ksi	
Shear Stress Unity =	0.011		**Shear Stress OK**
Max Allowable Deflection =	0.267	in	L/180
Maximum Beam Deflection =	0.053	in	**Deflection OK**

Shelf Beam Rivet Check:

Diameter of Rivet =	0.25	in	
Post Moment Shear on Rivet =	238.8	lbs	
Beam Shear on Rivet =	38.3	lbs	
Resultant Shear =	241.9	lbs	
Bearing Capacity of Rivet =	519.8	lbs	**Bearing Stress OK**
Allowable Shear Stress =	13.5	ksi	
Shear Stress on Rivet =	4.9	ksi	**Shear Stress OK**

Seismic Uplift on Shelves -

Vertical Seismic Component =	30.9	lbs	
Vertical Total Load per Shelf =	111.4	lbs	
Connection Points per Shelf =	4.0	(1) per Corner	
Net Uplift Load per Shelf =	-35.9	lbs	
Uplift Forcer per Connection =	-9.0	lbs	**Rivet Connection OK**

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Wall Supported Unit Calculations

Seismic Force at Top of Units -

Average Roof Height =	20.0	ft
Height of Attachment =	10.0	ft
Shear Coeff Boundaries =	V_{min} = 0.303	
	V_{max} = 1.618	
Design Base Shear Coeff =	V_t = 0.460	Adjusted For ASD and "ρ"
Total Weight per Unit =	557	lbs
Lateral Force at Top/Bottom =	128	lbs
Standard Stud Spacing =	16	in
Wall Connections per Unit =	3	
Tek Screw Capacity =	84	lbs	Tension Cap. for #10 Screw in 20ga Stud
Force Per Connection =	43	lbs	**Screw Capacity OK**

Light Gauge Steel Stud Wall Framing

Stud Design Data -

Height of Wall Studs =	16.0	ft	Int. Non-Brg - Worst Case Ht Assumed
Location of Point Load =	10.0	ft	
Design Lateral Load =	42.7	lbs	From Shelving Unit
Additional Lateral Load =	5.0	psf	Interior Seismic Force
Design Axial Load =	85.3	lbs	Dead Load of Wall Framing
Spacing of Studs =	16.0	in	

TRY: 3-5/8" x 1-5/8" x 20ga Studs @ 16" o.c. (Worst Case Assumed)

Width =	3.625	in	rx =	1.450	in
Depth =	1.625	in	ry =	0.616	in
Thickness =	0.035	in	Sx =	0.268	in3
Fy =	33	ksi	Ix =	0.551	in4
E =	29000	ksi	Ap =	0.262	in2
K =	1.0		Unbraced Length X =	16	ft
			Unbraced Length Y =	4	ft

Stud Capacity -

Buckling Stress, X =	16.32	ksi	
Buckling Stress, Y =	47.14	ksi	
Allowable Buckling Stress =	16.32	ksi	
Nominal Axial Strength =	4277	lbs	
Factor of Safety =	1.92		
Allowable Axial Load =	2228	lbs	
Maximum Design Moment =	373.4	ft-lbs	
Maximum Design Shear =	80.0	lbs	
Allowable Bending Stress =	21.78	ksi	
Actual Bending Stress =	16.72	ksi	**Bending Stress OK**
Allowable Shear Stress =	13.20	ksi	
Actual Shear Stress =	0.31	ksi	**Shear Stress OK**
Allowable Axial Stress =	8.50	ksi	
Actual Axial Stress =	0.33	ksi	**Axial Stress OK**
Combined Stress Unity =	0.81		**Combined Stress OK**

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Slab Bearing & Uplift Calculations

Slab Design Properties -

Minimum Concrete Strength =	2500	psi	Assumed
Thickness of Concrete Slab =	4	in	Assumed
Weight of Concrete Slab =	50	psf	
Allowable Bearing Pressure =	500	psf	Assumed
Bearing Loads On Post =	26	lbs	Dead Load
	169	lbs	Live Load
	339	lbs	EQ Load
Uplift Loads on Post =	420	lbs	Resultant Uplift

Slab Bearing Capacity -

Depth of Post on Slab =	1.5	in	
Factored Bearing Load =	786	lbs	
Required Bearing Area =	153.89	in2	12.41 inches per side
Critical Section =	3.45	in	For Bending
Soil Pressure on Crit. Section =	735.8	plf	Along Critical Length
Section Modulus =	32.0	in3	Plain Concrete per Foot
Shear Area =	22	in	
Conc. Shear Stress =	8.9	psi	
Allowable Shear Stress =	73.2	psi	**Shear Stress OK**
Conc. Bending Stress =	11.4	psi	
Allowable Bending Stress =	137.5	psi	**Bending Stress OK**

Slab Uplift Capacity -

Required Area to Resist Uplift =	0.00	ft2	
Length of Slab Req'd =	0.00	ft	Assume Required Area / Full Shelf Width
Worst Case Length of Slab =	0.75	ft	Maximum Length Required
Distance to Anchor Bolt =	0.38	ft	
Length of 1ft Strip =	0.75	ft	Length Safety Factor: 2.00
Shear Force on 1ft Strip =	52.5	lbs	
Allowable Shear Force =	1760.0	lbs	**Shear OK**
Bending Moment on 1ft Strip =	4.9	ft-lbs	
Allowable Bending Moment =	366.7	ft-lbs	**Bending OK**

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ECLIPSE ENGINEERING, P.C.
nburnam
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Thursday, October 6, 2022 10:29 AM

Geometric Properties

Area	0.217	in^2
Ix	0.049	in^4
Ixy	-0.029	in^4
Iy	0.049	in^4
Sx+	0.044	in^3
Sx-	0.120	in^3
Sy+	0.044	in^3
Sy-	0.120	in^3
Xc	0.406	in
Yc	0.406	in
rx	0.473	in
ry	0.473	in

Polar Properties

Ip	0.097	in^4
rp	0.669	in

Principal Properties

I1	0.078	in^4
I2	0.019	in^4
S1+	0.074	in^3
S1-	0.074	in^3
S2+	0.035	in^3
S2-	0.036	in^3
r1	0.599	in
r2	0.296	in
α	45.000	deg

Overall Properties

Depth	1.500	in
Perimeter	5.920	in
Weight	0.001	K/ft
Width	1.500	in

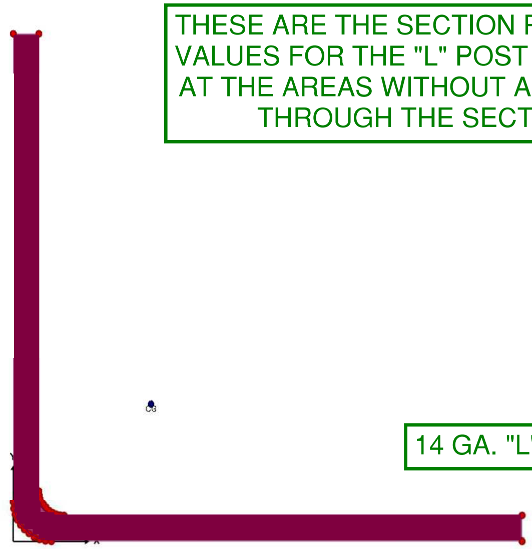
Torsion Properties

Cw	0.000	in^6
H	0.626	
J	0.000	in^4
Xsc	0.041	in
Ysc	0.041	in
ro	0.845	in
B1	0.000	in

Plastic Properties

Xpna	0.074	in
Ypna	0.074	in
Zx	0.080	in^3
Zy	0.080	in^3

THESE ARE THE SECTION PROPERTY VALUES FOR THE "L" POST MEMBERS AT THE AREAS WITHOUT ANY HOLES THROUGH THE SECTION.



14 GA. "L" POST

PRCTI20230176

ShapeBuilder 12.00.0001
www.iesweb.com

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nburnam
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Thursday, October 6, 2022 10:14 AM

Geometric Properties

Area	0.401	in^2
Ix	0.183	in^4
Ixy	0.000	in^4
Iy	0.192	in^4
Sx+	0.140	in^3
Sx-	0.291	in^3
Sy+	0.113	in^3
Sy-	0.113	in^3
Xc	1.693	in
Yc	0.629	in
rx	0.676	in
ry	0.692	in

Polar Properties

Ip	0.375	in^4
rp	0.967	in

Principal Properties

I1	0.192	in^4
I2	0.183	in^4
S1+	0.113	in^3
S1-	0.113	in^3
S2+	0.140	in^3
S2-	0.291	in^3
r1	0.692	in
r2	0.676	in
α	90.000	deg

Overall Properties

Depth	1.938	in
Perimeter	14.293	in
Weight	0.001	K/ft
Width	3.387	in

Torsion Properties

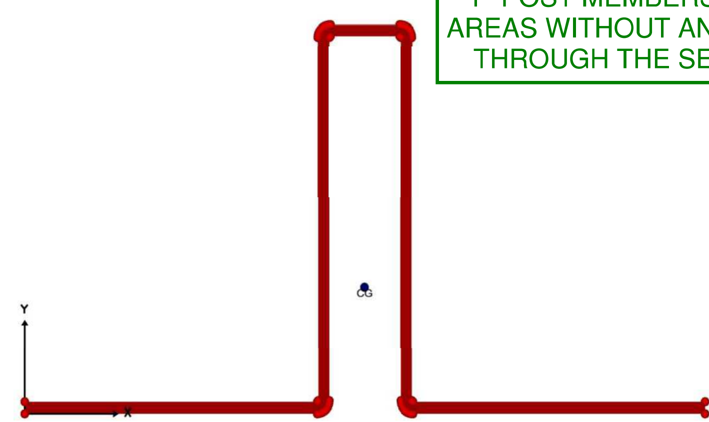
Cw	0.059	in^6
H	0.967	
J	0.000	in^4
Xsc	1.693	in
Ysc	0.807	in
ro	0.983	in
B1	0.000	in

Plastic Properties

Xpna	1.693	in
Ypna	0.328	in
Zx	0.231	in^3
Zy	0.205	in^3

16 GA. "T" POST

THESE ARE THE SECTION PROPERTY VALUES FOR THE "T" POST MEMBERS AT THE AREAS WITHOUT ANY HOLES THROUGH THE SECTION.



NOTE THAT THE "T" POST MEMBER SECTION PROPERTIES ARE APPROXIMATELY THREE TIMES THE VALUES FOR THE "L" POST MEMBERS, BUT ONLY SUPPORT ABOUT TWICE THE LOADS.

ShapeBuilder 12.00.0001
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Geometric Properties

Area	0.142 in^2
Ix	0.041 in^4
Ixy	-0.026 in^4
Iy	0.041 in^4
Sx+	0.040 in^3
Sx-	0.089 in^3
Sy+	0.040 in^3
Sy-	0.089 in^3
Xc	0.461 in
Yc	0.461 in
rx	0.539 in
ry	0.539 in

Principal Properties

I1	0.067 in^4
I2	0.016 in^4
S1+	0.063 in^3
S1-	0.063 in^3
S2+	0.034 in^3
S2-	0.026 in^3
r1	0.685 in
r2	0.332 in
α	45.000 deg

Polar Properties

Ip	0.083 in^4
rp	0.762 in

Overall Properties

Depth	1.500 in
Perimeter	4.218 in
Weight	0.000 K/ft
Width	1.500 in

Plastic Properties

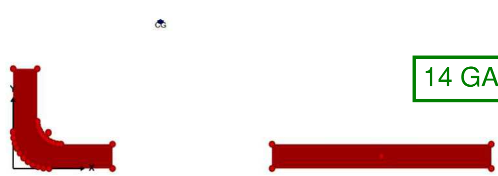
Xpna	0.074 in
Ypna	0.074 in
Zx	0.060 in^3
Zy	0.060 in^3

Shear Flow

Name	Qx in^3	Qy in^3	f(Vx) K/ft	f(Vy) K/ft
Part #2 0	0.036	0.022	0.000	0.000
Part #3 0	0.014	0.014	0.000	0.000
Part #4 0	0.022	0.036	0.000	0.000

THESE ARE THE SECTION PROPERTY VALUES FOR THE "L" POST MEMBERS AT THE AREAS WITH THE RIVET-CONNECTION HOLES THROUGH THE SECTION.

PRCTI20230176



14 GA. "L" POST

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Geometric Properties

Area	0.288 in^2
Ix	0.156 in^4
Ixy	0.000 in^4
Iy	0.120 in^4
Sx+	0.125 in^3
Sx-	0.226 in^3
Sy+	0.071 in^3
Sy-	0.071 in^3
Xc	1.693 in
Yc	0.689 in
rx	0.735 in
ry	0.646 in

Principal Properties

I1	0.156 in^4
I2	0.120 in^4
S1+	0.125 in^3
S1-	0.226 in^3
S2+	0.071 in^3
S2-	0.071 in^3
r1	0.735 in
r2	0.646 in
α	0.000 deg

Polar Properties

Ip	0.276 in^4
rp	0.978 in

Overall Properties

Depth	1.938 in
Perimeter	10.745 in
Weight	0.001 K/ft
Width	3.387 in

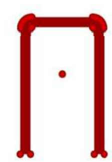
Plastic Properties

Xpna	1.693 in
Ypna	0.328 in
Zx	0.180 in^3
Zy	0.132 in^3

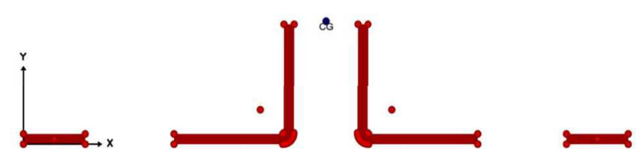
Shear Flow

Name	Qx in^3	Qy in^3	f(Vx) K/ft	f(Vy) K/ft
ClosedPoly 1	0.036	0.027	0.000	0.000
Part 1	0.013	0.030	0.000	0.000
Part 2	0.013	0.030	0.000	0.000
Part 3	0.097	0.000	0.000	0.000
Part 4	0.036	0.027	0.000	0.000

16 GA. "T" POST



THESE ARE THE SECTION PROPERTY VALUES FOR THE "T" POST MEMBERS AT THE AREAS WITH THE RIVET-CONNECTION HOLES THROUGH THE SECTION.



NOTE THAT THE "T" POST MEMBER SECTION PROPERTIES ARE APPROXIMATELY THREE TIMES THE VALUES FOR THE "L" POST MEMBERS, BUT ONLY SUPPORT ABOUT TWICE THE LOADS.

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Screw Capacities

Table Notes

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

Allowable Screw Connection Capacity (lbs)

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

SUPREME* Allowable Screw Connection Capacity (Pounds Per Screw)

See back cover for exclusive manufacturers of the Supreme Framing System.

Thickness (mil)	Design Thickness (in)	Fy Yield (ksi)	Fu Tensile (ksi)	#6 Screw (Pss = 643 lbs, Pts = 419 lbs) 0.138" Dia; 0.272" Head			#8 Screw (Pss= 1278 lbs, Pts = 586 lbs) 0.164" Dia; 0.272" Head			#10 Screw (Pss= 1644 lbs, Pts = 1158 lbs) 0.190" Dia; 0.340" Head			#12 Screw (Pss= 2330 lbs, Pts = 2325 lbs) 0.216" Dia; 0.340" Head			¼" Screw (Pss= 3048 lbs, Pts = 3201 lbs) 0.250" Dia; 0.409" Head		
				Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over
D25	0.0155	57	65	65	39	137	150	47	137	77	54	171	-	-	-	-	-	-
D20	0.0188	57	65	142 [1]	48	140	150 [1]	57	166	164 [1]	66	208	109	75	208	-	-	-
30EQD	0.0235	57	65	174 [1]	60	140	184 [1]	71	195	236 [1]	82	260	152	93	260	-	-	-
33EQD	0.0235	57	65	174 [1]	60	140	184 [1]	71	195	236 [1]	82	260	152	93	260	-	-	-
33EQS	0.0295	57	65	171	75	140	187	89	195	201	103	326	214	117	326	231	136	392
43EQS	0.0400	57	65	270	102	140	295	121	195	317	140	386	338	159	442	364	184	532

[1]Values are based on testing using AISI S100 procedures.

SUPREME products are only available from those SSMA members who are certified to produce SUPREME products.

PRCTI20230176

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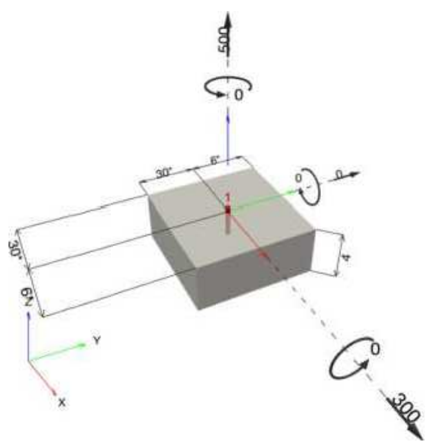
Company:	ECLIPSE ENGINEERING, INC.	Page:	1
Address:	376 SW Bluff Dr., Suite 8	Specifier:	nburnam
Phone I Fax:	541-389-9659 |	E-Mail:	
Design:	NWC- (1) KB-TZ2-0.375X2.0	Date:	8/6/2022
Fastening point:	Base of Shelving Unit		

Specifier's comments: Single Anchor Condition

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 318-14 / Mech
Stand-off installation:	
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))

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



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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Load case: Design loads	N = 500; V_x = 300; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	yes	35

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	500	300	300	0

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	500	4,869	11	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	500	1,448	35	OK

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

THIS IS THE LIMITING TENSION VALUE FOR A SINGLE ANCHOR. 1,448 LBS PER ANCHOR

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Design:	NWC- (1) KB-TZ2-0.375X2.0	Date:	8/6/2022
Fastening point:	Base of Shelving Unit		

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-4266
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

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	500

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]
2.000	6.000	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	500

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Fastening point:	Base of Shelving Unit		

4 Shear load

	Load V_{ua} [lb]	Capacity $\phi\ V_n$ [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	300	2,201	14	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	300	2,079	15	OK
Concrete edge failure in direction x+**	300	1,541	20	OK

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

THIS IS THE LIMITING SHEAR VALUE FOR A SINGLE ANCHOR. 1,541 LBS PER ANCHOR

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-4266
$\phi\ V_{steel} \geq V_{ua}$ 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}	$\phi_{nonductile}$	$\phi\ V_{sa,eq}$ [lb]	V_{ua} [lb]
3,386	0.650	1.000	2,201	300

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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)

$\psi_{cp,N} = MAX \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ 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	2.000	6.000	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	300

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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)

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

$\psi_{h,V} = \sqrt{\frac{1.5 c_{a1}}{h_a}} \geq 1.0$ 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.]
6.000	6.000	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]
60.00	162.00	0.900	1.500	4,403

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi \, V_{cb}$ [lb]	V_{ua} [lb]
2,201	0.700	1.000	1.000	1,541	300

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.345	0.195	5/3	24	OK

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

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Fastening point:	Base of Shelving Unit		

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!

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Design:	NWC- (1) KB-TZ2-0.375X2.0	Date:	8/6/2022
Fastening point:	Base of Shelving Unit		

7 Installation data

	Anchor type and diameter: Kwik Bolt TZ2 - CS 3/8 (2) hnom2
Profile: -	Item number: 2210236 KB-TZ2 3/8x3
Hole diameter in the fixture: -	Maximum installation torque: 361 in.lb
Plate thickness (input): -	Hole diameter in the base material: 0.375 in.
	Hole depth in the base material: 2.750 in.
Drilling method: Hammer drilled	Minimum thickness of the base material: 4.000 in.
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

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	30.000	6.000	30.000	6.000

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8 Remarks; Your Cooperation Duties

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- 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.

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Design:	NWC- (2) KB-TZ2-0.375X2.0	Date:	8/6/2022
Fastening point:	Base of Shelving Units		

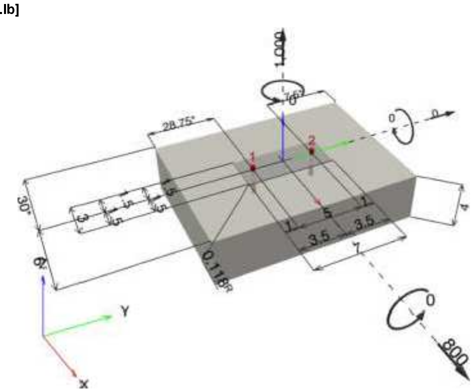
Specifier's comments: ANCHOR PAIR CONDITION

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 318-14 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.118 in.
Anchor plate[R] :	l_x x l_y x t = 3.000 in. x 7.000 in. x 0.118 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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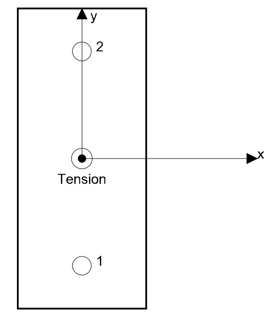
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Load case: Design loads	N = 1,000; V_x = 800; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	yes	38

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	500	400	400	0
2	500	400	400	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 1,000 [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*	500	4,869	11	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	1,000	2,654	38	OK

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

THIS IS THE LIMITING TENSION VALUE FOR A SINGLE ANCHOR. 1,327 LBS PER ANCHOR

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3.1 Steel Strength

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

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

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	500

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3.2 Concrete Breakout Failure

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

$\phi\ N_{cbg} \geq N_{ua}$ — 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)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — 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.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2.000	0.000	0.000	6.000	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}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
66.00	36.00	1.000	1.000	1.000	1.000	2,970

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
5,445	0.650	0.750	1.000	2,654	1,000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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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*	400	2,201	19	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	800	3,811	21	OK
Concrete edge failure in direction x+**	800	2,331	35	OK

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

THIS IS THE LIMITING SHEAR VALUE FOR A SINGLE ANCHOR. 1,165 LBS PER ANCHOR

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	400

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4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-14 Eq. (17.5.3.1b)
ϕ $V_{cpg} \geq V_{ua}$ 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)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N}{3\ h_{ef}}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}} \right) \leq 1.0$ 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

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
1	2.000	0.000	0.000	6.000

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

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
66.00	36.00	1.000	1.000	1.000	1.000	2,970

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕ V_{cpg} [lb]	V_{ua} [lb]
5,445	0.700	1.000	1.000	3,811	800

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4.3 Concrete edge failure in direction x+

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

$\phi\ V_{cbg} \geq V_{ua}$ 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)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_v}{3c_{a1}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.5)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ 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.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
6.000	7.500	0.000	1.000	4.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
2.000	1.000	0.375	2,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
86.00	162.00	1.000	0.950	1.500	4,403

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
3,330	0.700	1.000	1.000	2,331	800

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.377	0.343	5/3	37	OK

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

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- 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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7 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = 0.438 in.

Plate thickness (input): 0.118 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

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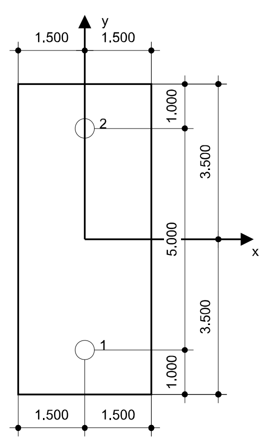

Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-0.000	-2.500	30.000	6.000	28.750	12.500
2	-0.000	2.500	30.000	6.000	33.750	7.500

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- 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.

MOBILE STORAGE SYSTEMS INC.
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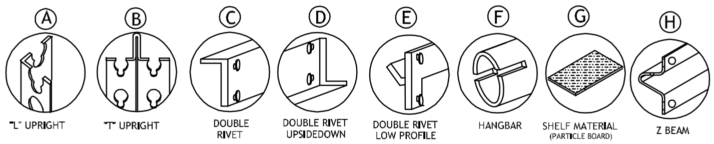
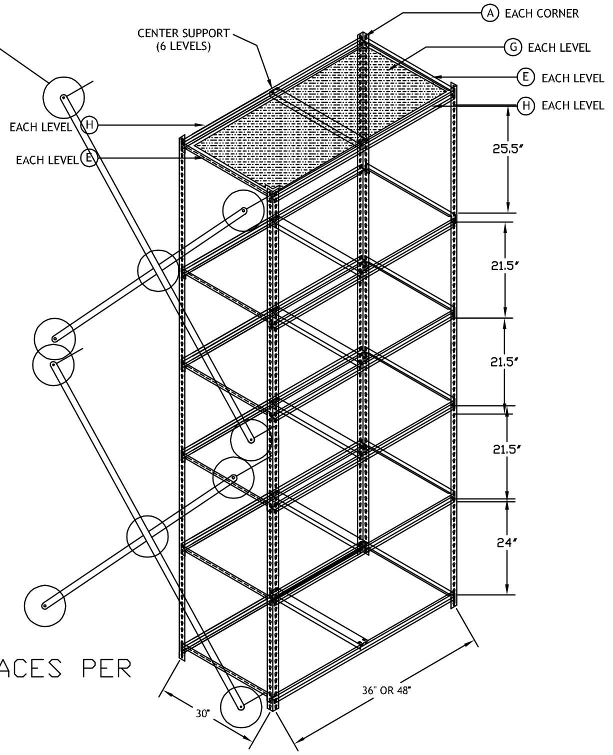
WWW.PIPPMOBILE.COM
E-MAIL: CUSTOMERSERV@PIPPMOBILE.COM

INSTALLATION INSTRUCTIONS FOR MOBILE UNITS

UNITS WILL BE 10' HIGH
WITH 6 SHELVES 5 OPENINGS

SHELVING UNIT PARTS:





(4) X-BRACES PER CARRIAGE

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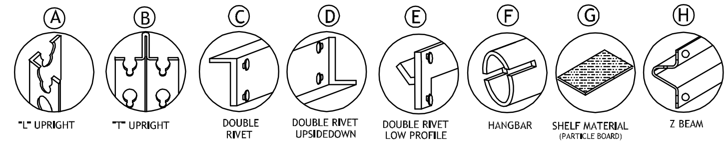
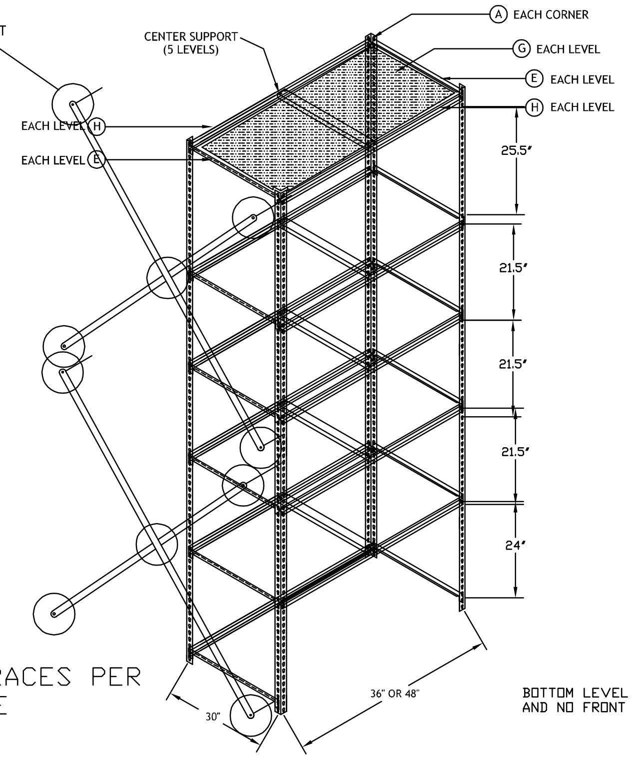
INSTALLATION INSTRUCTIONS FOR STATIONARY UNITS

UNITS WILL BE 10' HIGH
WITH 5 SHELVES 5 OPENINGS

UNITS MARKED WITH "/"

SHELVING UNIT PARTS:





(4) X-BRACES PER CARRIAGE

BOTTOM LEVEL NO SHELF AND NO FRONT SUPPORT

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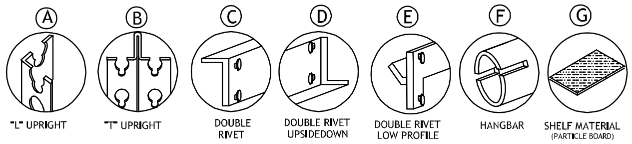

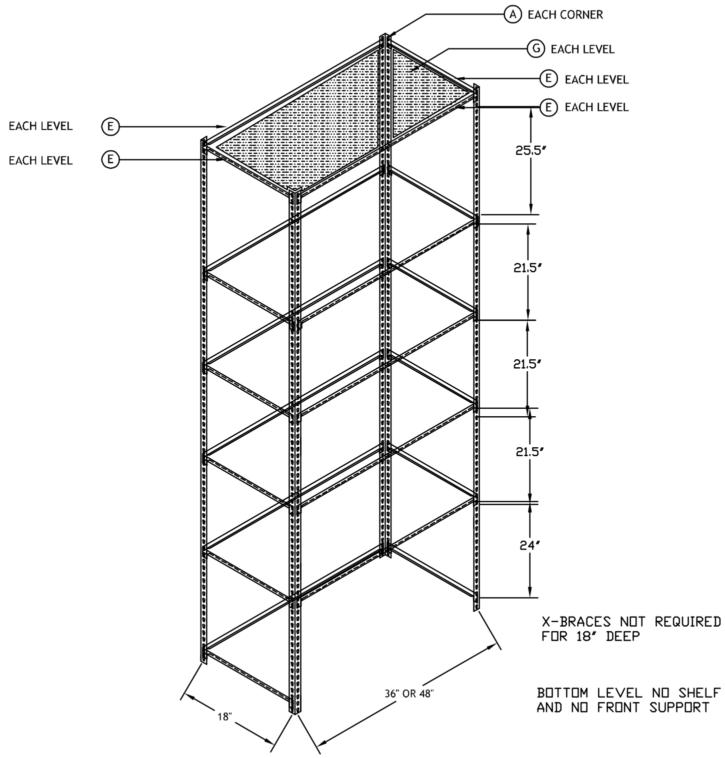

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