

Addendum No. 1 to REBID IFB 24-43



CITY OF SOMERVILLE, MASSACHUSETTS
Department of Procurement and Contracting Services
KATJANA BALLANTYNE
MAYOR

To: Bidders of REBID IFB 24-43 Installation of Three (3) Standalone Portland Loo Prefabricated Toilets
From: Andrea Caruth, Deputy Chief Procurement Officer
Date: March 21, 2024 (Updated to include bid price form referenced in response to question #1)
Re: Responses to Requests for information

Addendum No. 1 to REBID IFB 24-43

This addendum responds to requests for information.
This addendum adds the structural drawings and structural calculations noted on page 4.
This addendum updates the bid price form.
This addendum updates that the School Street dropbox is no longer available.

Please note: the City will receive submissions by mail, hand delivery, and BidExpress.com.

**** Failure to acknowledge this addendum may result in bid disqualification.****

NAME OF COMPANY / INDIVIDUAL: _____

ADDRESS: _____

CITY/STATE/ZIP: _____

TELEPHONE/FAX/EMAIL: _____

SIGNATURE OF AUTHORIZED INDIVIDUAL: _____

ACKNOWLEDGEMENT OF ADDENDA:

Addendum #1 _____ #2 _____ #3 _____ #4 _____

Addendum No. 1 to REBID IFB 24-43

Question	Answer
1. Please provide the unit price bid form with estimated quantities of work listed within the technical specifications shown on pages 113 to page 153 of the IFB document.	See attached document titled "Bid Form Portland Loo Installation."
2. Section 1.2 states contract completion date as 04/03/25, section 2.2 appears to incorrectly state date of substantial completion as 06/30/2024 and final completion of 08/31/2024. Can you please revise?	We regularly delay estimated contract completion to the spring following final completion to accommodate unexpected delays that are impacted by winter conditions. If the extra time is unneeded, we would end the contract at the final completion date.
3. Page 10 of 159 states that <i>"The Contractor will be responsible for obtaining EVERSOURCE work orders and for all costs and fees associated with EVERSOURCE."</i> a. We have experienced very long lead times securing Eversource Work Orders for electrical connections and have no way to determine prior to bid how much time will be required. This lengthy process would make a substantial completion date of 06/30/2024 and final completion date of 08/31/2024 unattainable. As these units will be owned & managed by the City, we would recommend the City begin the process of applying for the Work Orders independently of the contractor and prior to the start of construction.	We have a very collaborative relationship with Eversource, which usually results in reasonable work order timing. In addition, we have already notified Eversource that we will be executing this work over the summer. We do not expect any substantial delays that would push us beyond the final completion date. We expect the selected contractor to apply for the Eversource Work Order promptly upon notice to proceed.
4. If the work order process cannot be stated prior to construction, would the City consider creating an allowance item to cover all permit, connection & engineering fees Eversource will be assessing the project as they cannot be determined	An allowance item cannot be provided. Our experience is that there is sufficient time between Est Contract Commencement and Substantial Completion to accomplish the Eversource connection. If this work exceeds the Est Contract Completion date, we would consider an addendum for additional time and effort.

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prior to bid.	
<p>5. “Key Project Information” and “Project Background” tables both list liquidated damages as \$250.00 per day, page 117 of 159 of project specifications shows a different amount, at \$4,000.00 per week.</p> <ul style="list-style-type: none"> a. Can you please clarify which amount is correct? b. Can you please clarify when liquidated damages listed will be assessed onto the contractor? 	<p>The \$4000 per week is correct.</p> <p>Liquidated damages would result from the failure to install the public loos, sidewalks and other ancillary items within 90 days of receipt of public loos; it does not include final operations due to any delays in utility connections.</p>
<p>6. Has the City of Somerville pre-purchased the portable structures?</p> <ul style="list-style-type: none"> a. If so, can you please provide an estimated delivery date of the structures to the project site? b. Can you please clarify how the units will be shipped and what equipment will be needed to unload? 	<p>Yes, the Portland Loo’s have been pre-purchased and have been ordered.</p> <p>Please refer to Project Specification Item 1.1. The selected contractor may choose their yard for delivery and storage. We anticipate delivery during the month of May, but we have not received confirmation from the manufacturer.</p> <p>The units are each shipped on a pallet and can be unloaded with a forklift. The units are shipped wrapped in a thick vinyl wrap, and weighs just over 6,000 lbs.</p>
<p>7. We were unable to access the designer’s website and gather any information about the Portable Loo units, could you please provide complete set of installation instructions, shop drawings and specifications so we can review prior to bid?</p>	<p>See attached installation instructions and structural drawings. A video demonstrating installation can be found here: https://vimeo.com/141186536</p>
<p>8. Utility plans show electrical conduit running from the pre-fabricated structures up to existing buildings, has the City coordinated getting power and making final connections inside these existing structures?</p> <ul style="list-style-type: none"> a. If the electrical connections have not been finalized, would the City consider creating an allowance item for this work? 	<p>Design plans for electrical connections reflect proposed locations to existing utility poles. Final location will require coordination with provider.</p> <p>No.</p>

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9. Utility plans show proposed invert elevations of sewer connections but do not show any local rim elevations, can you please provide rim elevations close to the connection point so we can determine depth of proposed sewer connections?	<p>a. Davis Square – Approx. Depth: 8' Local Rim 29.3' Invert 21.3'</p> <p>b. Union Square - Approx. Depth: 8.5' Local Rim 17.6' Invert 9' (contractor will be installing the main and terminal manhole at this location.</p> <p>c. 165 Broadway - Approx. Depth: 8' (estimated from downstream manhole internal photo)</p>
10. Please provide foundation details and specifications for the pre-fabricated structures.	See Sheet S3.0 of the attached document titled "Portland Loo Structural Drawings."
11. IFB Specifications include descriptions for item 201.5 catch basin - municipal standard and includes a detail for an infiltration basin, we see no proposed catch basins or infiltration basins on the utility plans provided, can you clarify if we will be installing any of these items as part of the project?	There is an infiltrating catch basin to be installed at the 165 Broadway Street location, shown on Sheet E of the design plans.
12. IFB Specifications include descriptions for item 202.1 manhole - municipal standard, we see no proposed manholes on the utility plans provided, can you clarify if we will be installing any of these items as part of the project?	The Union Square installation location includes extending an 8" PVC sewer main from an existing manhole 80', and installing a terminal manhole, to which the Portland Loo will have its sanitary service connected to.
13. IFB Specifications include descriptions for item 707.96 install pre-fabricated steel sign, and shows a detail sheet from "sign bracket store". Can you please clarify pre-fabricated sign model number, post and sign material type, post and sign material colors, sign wording, fonts and colors so we are able to determine sign cost.	It is the intention to replace the existing sign in the Union Square location. Item 707.96 shows the item "Channel Post and Panel" from the Sign Bracket Store (Model number 368PP-30C-96-BF-ARC). A similar sign and post, with prior review and approval from the project engineer may also be used.

BID PRICING PAGE
2024 PORTLAND LOO INSTALLATION PROJECT, SOMERVILLE, MA
VARIOUS SIDEWALK RECONSTRUCTION, UTILITY INSTALLATION, AND PORTLAND LOO INSTALLATION

Item #	Estimate Quantity	Unit	Item Description with Unit Bid Price written in words	Unit Price		Amount	
				Dollars	Cents	Dollars	Cents
1.100	12	Week	RECEIVE & STORE PORTLAND LOO				
			Unit Price in words:				
1.200	3	EACH	INSTALL 12'6" CONCRETE FOUNDATION				
			Unit Price in words:				
1.	3	EACH	INSTALL PORTLAND LOO				
			Unit Price in words:				
103.	4	EACH	TREE PROTECTION				
			Unit Price in words:				
120.	75	CY	UNCLASSIFIED EXCAVATION				
			Unit Price in words:				
141.1	75	CY	TEST PIT FOR EXPLORATION				
			Unit Price in words:				
142.0	75	CY	CLASS B TRENCH EXCAVATION				
			Unit Price in words:				
151.	30	CY	GRAVEL BORROW				
			Unit Price in words:				
201.	157	LF	PLASTIC (PVC) PIPE				
			Unit Price in words:				
202.	1	EACH	CATCH BASIN (INC. INFILTRATING BASIN)				
			Unit Price in words:				
202.1	1	EACH	SEWER MANHOLE				
			Unit Price in words:				
220.00	6	EACH	STRUCTURE ADJUST				
			Unit Price in words:				
				Subtotal			
				Amount from Page 1			

Item #	Estimate Quantity	Unit	Item Description with Unit Bid Price written in words	Unit Price		Amount	
				Dollars	Cents	Dollars	Cents
358.20	9	EACH	GATE BOX ADJUSTED (SIDEWALK)				
			Unit Price in words:				
360.	67	LF	COPPER TUBING (WATER SERVICE)				
			Unit Price in words:				
415.1	133	SY	PAVEMENT STANDARD MILLING				
			Unit Price in words:				
460.0	38	TON	HOT MIX ASPHALT FOR LOCAL STREETS				
			Unit Price in words:				
482.3	40	LF	SAWCUTTING ASPHALT PAVEMENT				
			Unit Price in words:				
504.0	45	LF	GRANITE CURB TYPE VA4 - STRAIGHT				
			Unit Price in words:				
580.0	135	LF	GRANITE CURB REMOVED & RESET				
			Unit Price in words:				
701.0	165	SY	CEMENT CONCRETE SIDEWALK				
			Unit Price in words:				
701.2	7	SY	CEMENT CONCRETE PEDESTRIAN CURB RAMP				
			Unit Price in words:				
701.50	2	EACH	DETECTABLE WARNING PANEL				
			Unit Price in words:				
707.15	2	EACH	REMOVE & RESET PARK BENCH				
			Unit Price in words:				
707.96	1	EACH	INSTALL PREFABRICATED STEEL SIGN				
			Unit Price in words:				
734.00	5	EACH	SIGN POST REMOVE & RESET				
			Unit Price in words:				
751.00	4	CY	LOAM BORROW				
			Unit Price in words:				
				Subtotal			
				Amount from Page 2			

Item #	Estimate Quantity	Unit	Item Description with Unit Bid Price written in words	Unit Price		Amount	
				Dollars	Cents	Dollars	Cents
765.00	15	SY	SEEDING				
			Unit Price in words:				
854.03	70	LF	4" TEMPORARY PAVEMENT MARKINGS (REMOVEABLE TAPE)				
				Subtotal Amount from Page 3			

BID SUMMARY			
			Amount
			Dollars Cents
Subtotal Amount from Page 1			
Subtotal Amount from Page 2			
Subtotal Amount from Page 3			
BID TOTAL			

Company Name: _____
Signature: _____
Signature Name & Title: _____
Telephone #: _____ Fax #: _____
Date: _____

THE PORTLAND LOO

377 SUMMER ST. SOMMERVILLE, MA 02144
90 UNION SQUARE SOMMERVILLE, MA 02143
165 BTOADWAY SOMMERVILLE, MA 02145

DESIGN INFORMATION

- 9th ED MASSACHUSETTS STATE BUILDING CODE
- LIVE LOAD = 20 PSF
- WIND LOAD:
Vult = 127 MPH
Vasd = 98.4 MPH
Iw = 1.0
Exposure = C
Risk Category = II
Internal Pressure = +/- 0.18
- SEISMIC LOAD:
Ie = 1.0
Ss = 0.28
S1 = 0.07
Sds = 0.294
Sd1 = 0.112
Site Class D
SDC = D
Seismic Base Shear = 0.19 kip
Light Frame (Cold Formed Steel) Wall w/ steel sheets
Analysis = Equivalent Lateral Force
R = 7.0
Cs = 0.042
- SNOW LOAD:
Pg = 40 psf
Is = 1.0
Ct = 1.0
Ce = 1.0
Pf = 30 psf
- NET ALLOWABLE SOIL BEARING PRESSURE: 1500 PSF
FC=4000 PSI
- CONTRACTOR TO VERIFY DESIGN PARAMETERS SHOWN WITH ACTUAL SITE CONDITIONS AND ENSURE ACTUAL SITE PARAMETERS DO NOT EXCEED DESIGN

GENERAL NOTES

- WEIGHT : 6,013 lbs
- SQUARE FOOTAGE : 51.5
- ALL STRUCTURAL STEEL TO BE FABRICATED AND ERECTED IN ACCORDANCE WITH AISC MANUAL 14TH EDITION & A.W.S. ALL PANELS, LOUVERS, AND ROOF TO BE 304 STAINLESS STEEL Fy = 30 ksi
- STRUCTURAL TUBING ASTM 500 GR. B
- ANCHOR BOLTS 1/2" X 5" SIMPSON TITEN HD ANCHORS
- ALL WELDING TO BE DONE BY CERTIFIED WELDERS. WELD FILLER ELECTRODE MATERIAL TO BE 70 ksi LOW HYDROGEN
- ALL EXPOSED BOLTS AND SCREWS TO BE TAMPER RESISTANT FOR HEX PIN BITS.
- RIVET NUTS TO BE C.F.T. SERIES AND STAINLESS STEEL 302.
- ALL MATERIALS TO BE SAND BLASTED AND POWDER COATED
- ALL STRUCTURES TO BE FABRICATED OFF SITE SHALL BE DONE IN SHOPS OR FABRICATORS LICENSED OR APPROVED BY THE BUILDING AND ENGINEERING DIVISION OF THE CITY FOR WHICH THE INSTALLATION WILL OCCUR
- NO SPECIAL INSPECTION NEEDED

SHEET INDEX:

A0.0	COVER SHEET
A1.0	FLOOR PLAN
A2.0	EXTERIOR ELEVATIONS FRONT & RIGHT SIDE
A3.0	EXTERIOR ELEVATIONS FRONT & RIGHT SIDE
A4.0	ROOF LAYOUT
A5.0	INTERIOR SECTION VIEW
A6.0	INTERIOR SECTION VIEWS
S1.0	COLUMN AND BEAM LAYOUT
S2.0	FRONT DOOR
S3.0	FOUNDATION & ANCHOR BOLT LOCATIONS
P1.0	PLUMBING FLOOR PLAN
P2.0	PLUMBING DETAIL
E1.0	ELECTRICAL SCHEMATIC

Digitally signed by Trevor G.
Wickie
Date: 2023.12.20 11:07:23-06'00'



Larson
Engineering Inc
1488 Bond Street, Suite 100
Naperville, Illinois 60563-6503
(P) 630.357.0540 (F) 630.357.0164
LEI Proj. # 21230893.000

1.	11/14/23	INITIAL ISSUE	AYK	MADDEN FABRICATION	SOMMERVILLE, MA			
2.	12/19/23	REVISED PER REDLINE	AYK		COVER SHEET			
REV	DATE	DESCRIPTION	DRN	APPD				
2550 NW 25th Place Portland, OR. 97210 Phone: (503) 226-3968 Fax: (503) 242-2446					NAME	DATE	SERIAL NO.	DRW. NO.
					EVAN	08/08/2022		A0.0

BABY CHANGE TABLE
KOALA KARE KB101

LATERAL SUPPORTS
SEE S1.0
TYPICAL COLUMNS

FRONT DOOR COMPLIES
WITH 2016 CBC 11B
5# MAXIMUM FORCE

HSS 2 1/2 x 2 1/2
x 3/16" (BEVEL EDGES 45°)

TYP 3/16"
4 1/2"
F.F.E.
1 1/2"

HSS 3x3x.188 POST

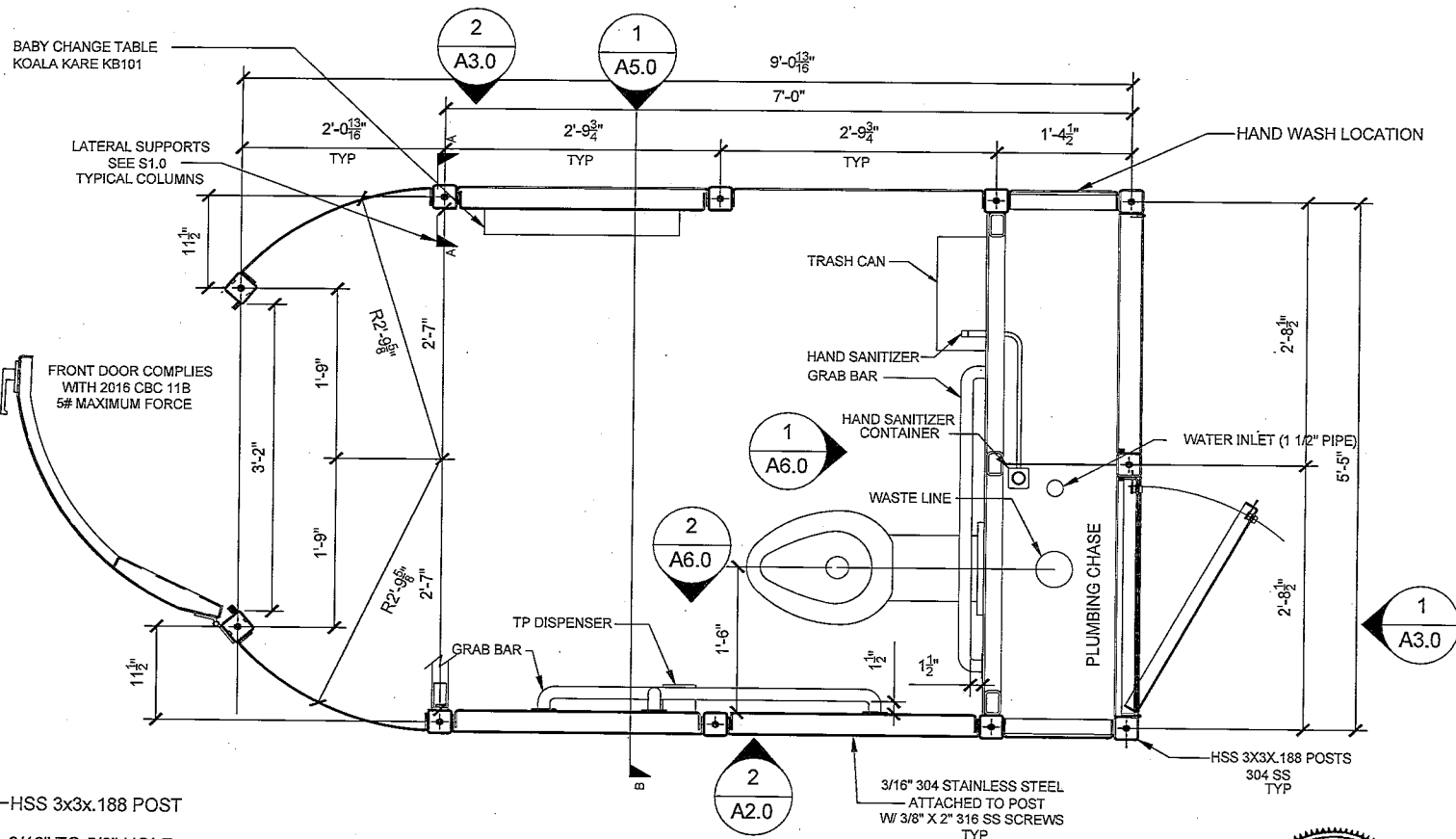
9/16" TO 5/8" HOLE
1/2" x 4" BOLT
FIELD LOCATE
1 1/4" PLUS ELEVATION

5/8" HOLE FOR
1/2" x 5" TITAN
4" EMBED MIN

FB 1/2" SS

SECTION A-A

Typ anchoring system
(11) REQ'D



INTERNAL TOP VIEW AND ANCHOR BOLT LOCATIONS

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Engineering Inc.

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REV	DATE	DESCRIPTION	DRN	APPD

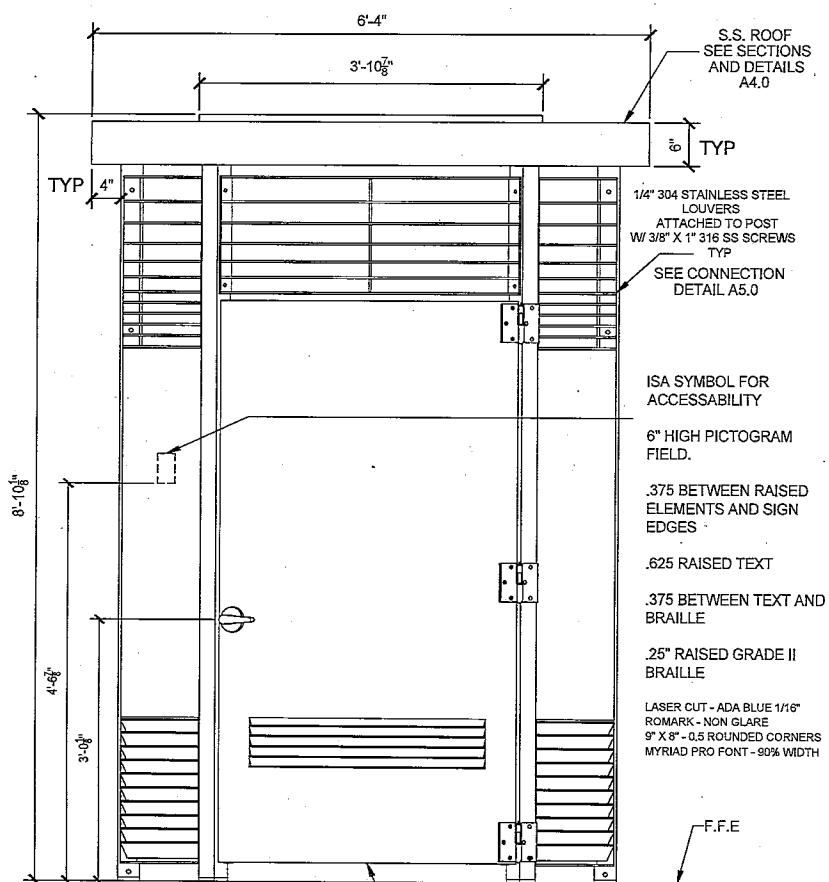
MADDEN
FABRICATION

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SOMMERVILLE, MA

FLOOR PLAN

NAME	DATE	SERIAL NO.	DRW. NO.
EVAN	08/08/2022		A1.0

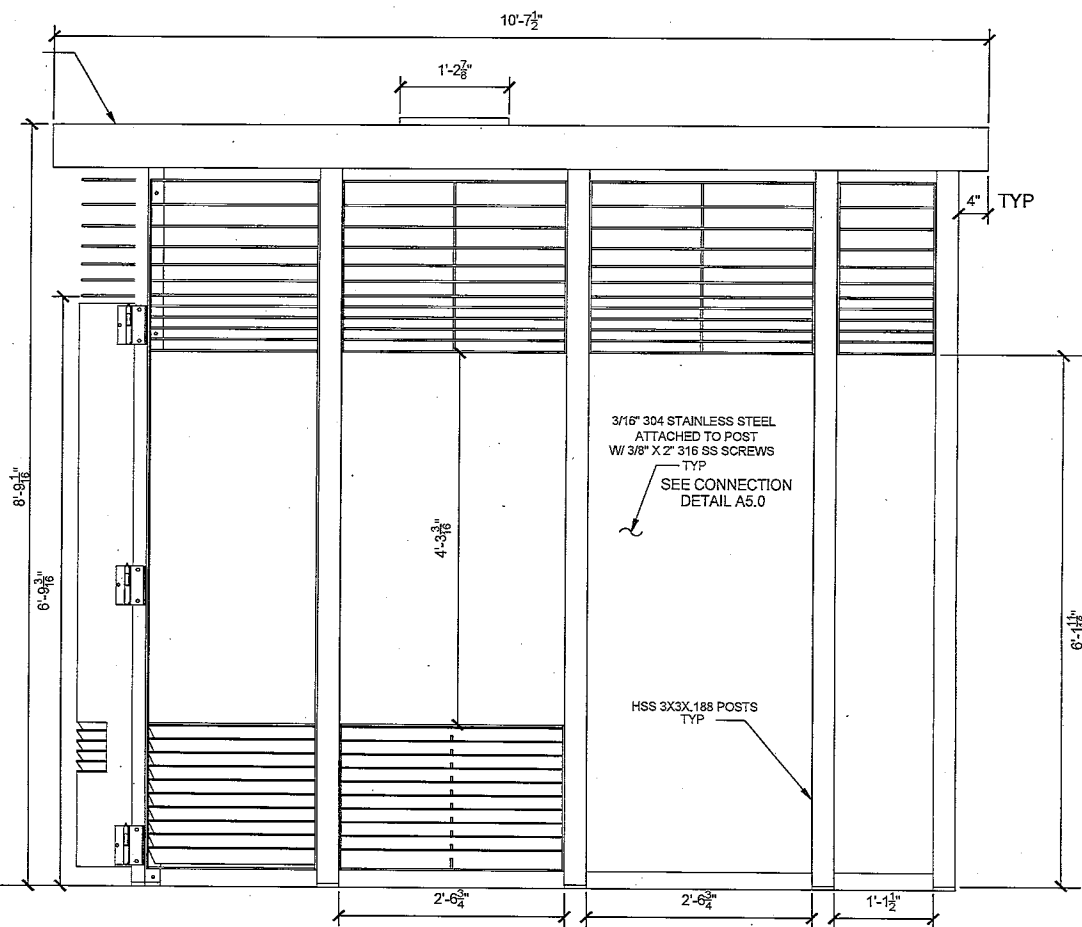
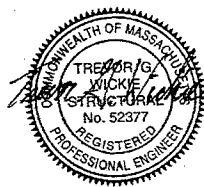


1 FRONT ELEVATION VIEW
A2.0



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2 RIGHT ELEVATION VIEW
A2.0

1	11/14/23	INITIAL ISSUE	AYK
2	12/19/23	REVISED PER REDLINE	AYK
REV	DATE	DESCRIPTION	DRN

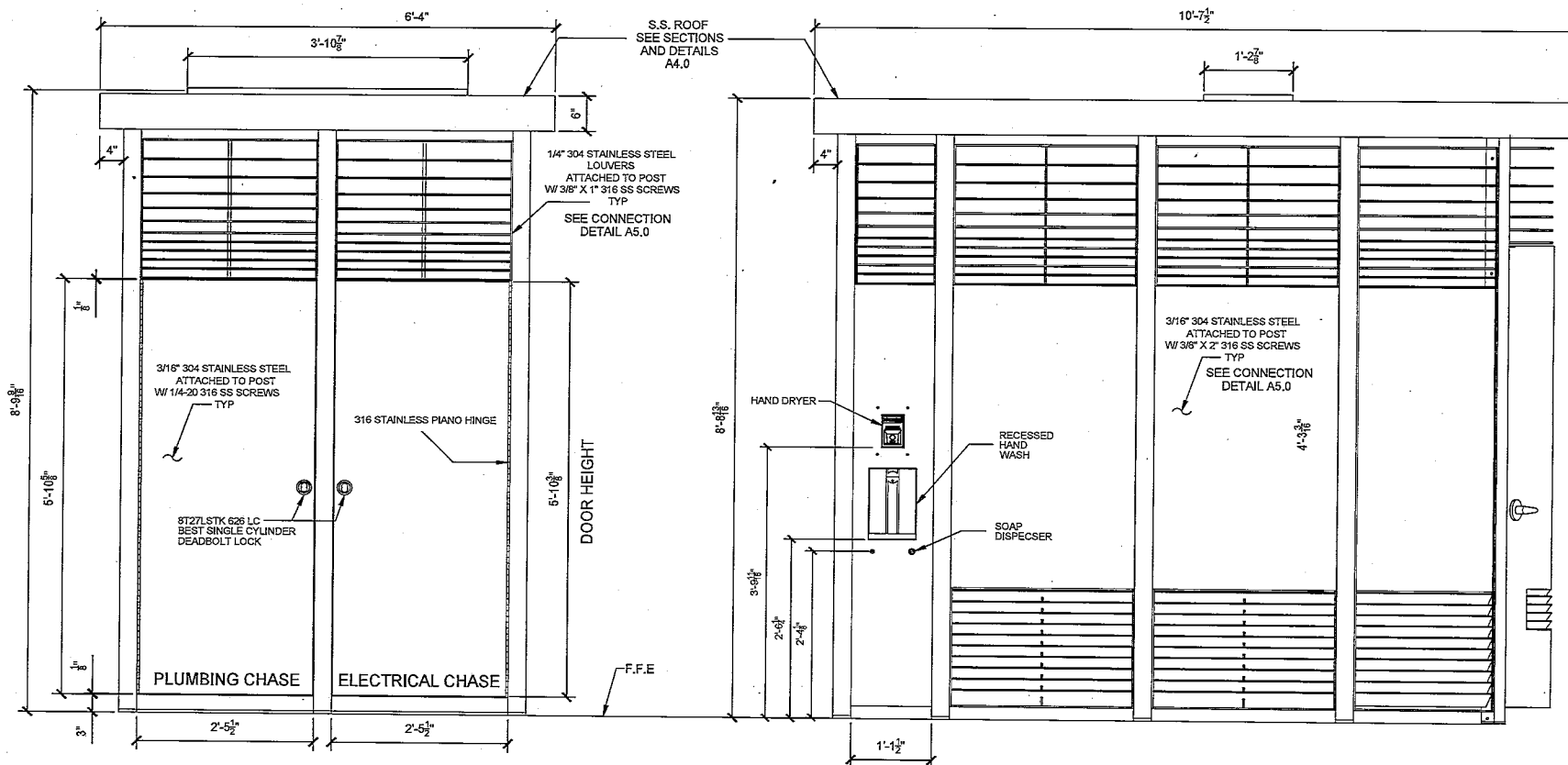
MADDEN FABRICATION

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SOMMERVILLE, MA

EXTERIOR ELEVATIONS FRONT & RIGHT SIDES

NAME	DATE	SERIAL NO.	DRW. NO.
EVAN	08/08/2022		A2.0



1 BACK ELEVATION VIEW
A3.0

2 LEFT ELEVATION VIEW
A3.0

***PLUMBING CHASE GIVES ACCESS TO WATERSEWER CONNECTIONS, PLUMBING MANIFOLD, AND HAND SANITIZER

***ELECTRICAL CHASE GIVES ACCESS TO SHARPS DISPOSAL, SOAP DISPENSER AND HAND WASH COMPONENTS



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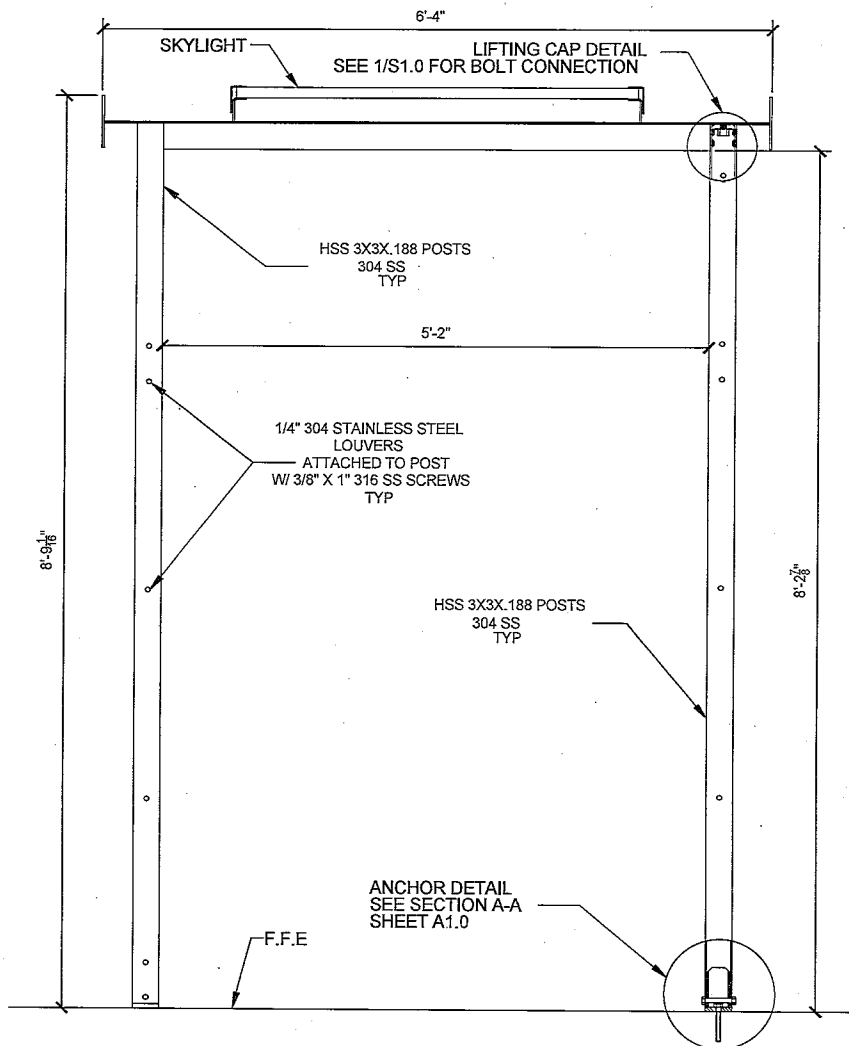
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2.	12/19/23	REVISED PER REDLINE	AYK	
REV	DATE	DESCRIPTION	DRN	APPD

MADDEN FABRICATION

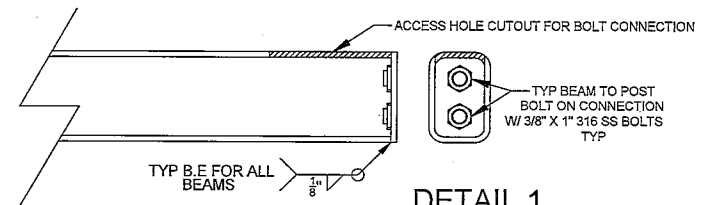
2550 NW 25th Place Portland, OR 97210
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SOMMERVILLE, MA			
EXTERIOR ELEVATIONS FRONT & RIGHT SIDES			
NAME	DATE	SERIAL NO.	DRW. NO.
EVAN	08/08/2022		A3.0

DETAIL 3



B-B
A1.0 INTERIOR ELEVATION VIEW



DETAIL 1

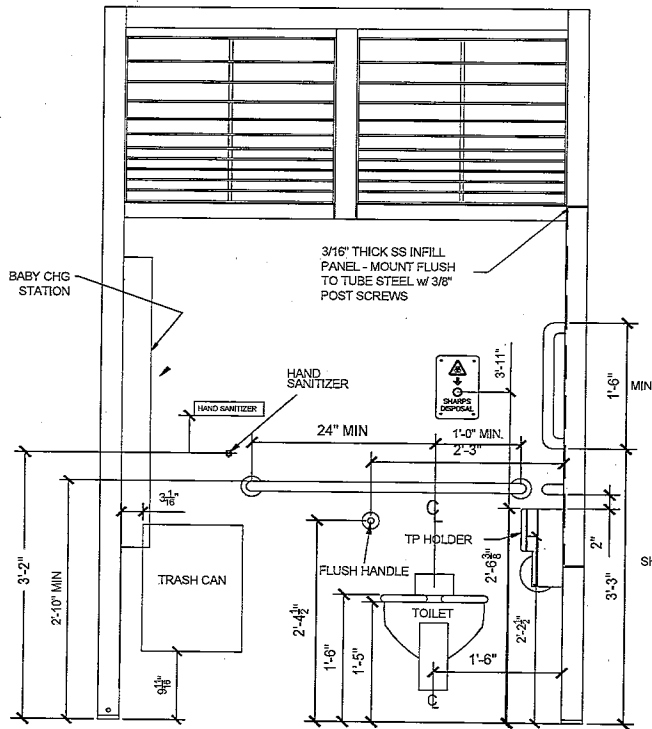
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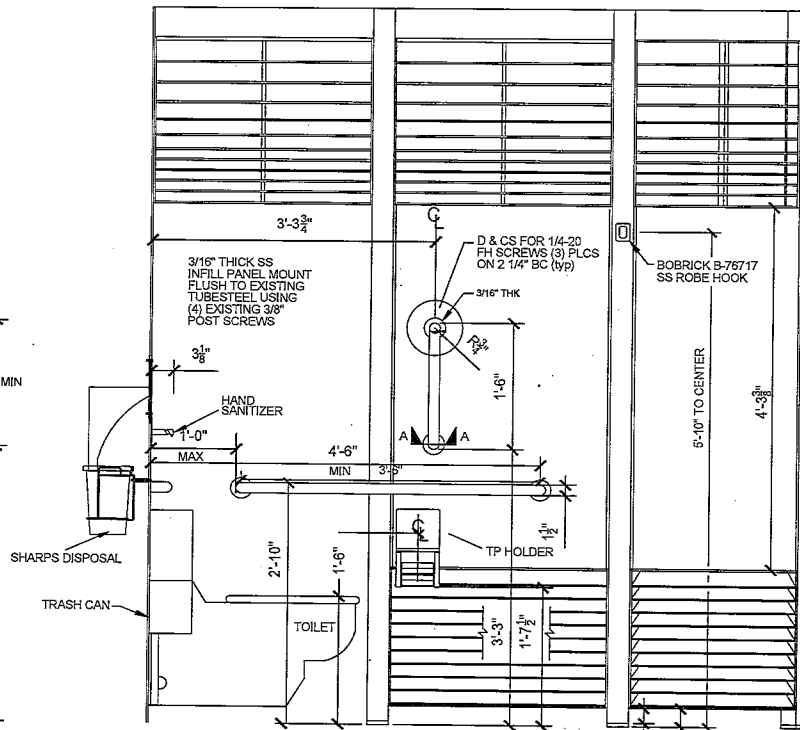
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2	12/19/23	REVISED PER REDLINE	AYK						
REV	DATE	DESCRIPTION	DRN	APPD					

MADDEN FABRICATION		SOMMERVILLE, MA	
		INTERIOR SECTION VIEW	
NAME	DATE	SERIAL NO.	DRW. NO.
EVAN	08/08/2022		A5.0

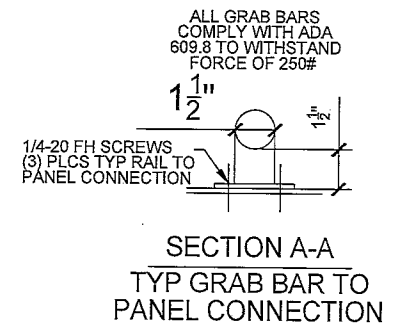
2550 NW 25th Place Portland, OR. 97210
Phone: (503) 226-3968 Fax: (503) 242-2446



1 WATER CLOSET RAILING
 A6.0 1 1/2"Øx120" wall ss
 PL 3/16" 304 SS
 (1) Assm REQ'D



2 WATER CLOSET RAILING
 A6.0 1 1/2"Øx120" wall ss
 PL 3/16" 304SS
 (1) Assm REQ'D

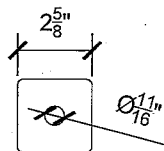


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2	12/19/23	REVISED PER REDLINE	AYK				
				MADDEN FABRICATION			
				SOMMERVILLE, MA			
				INTERIOR SECTION VIEWS			
REV	DATE	DESCRIPTION	DRN	APPO	NAME	DATE	SERIAL NO.
					EVAN	08/08/2022	A6.0

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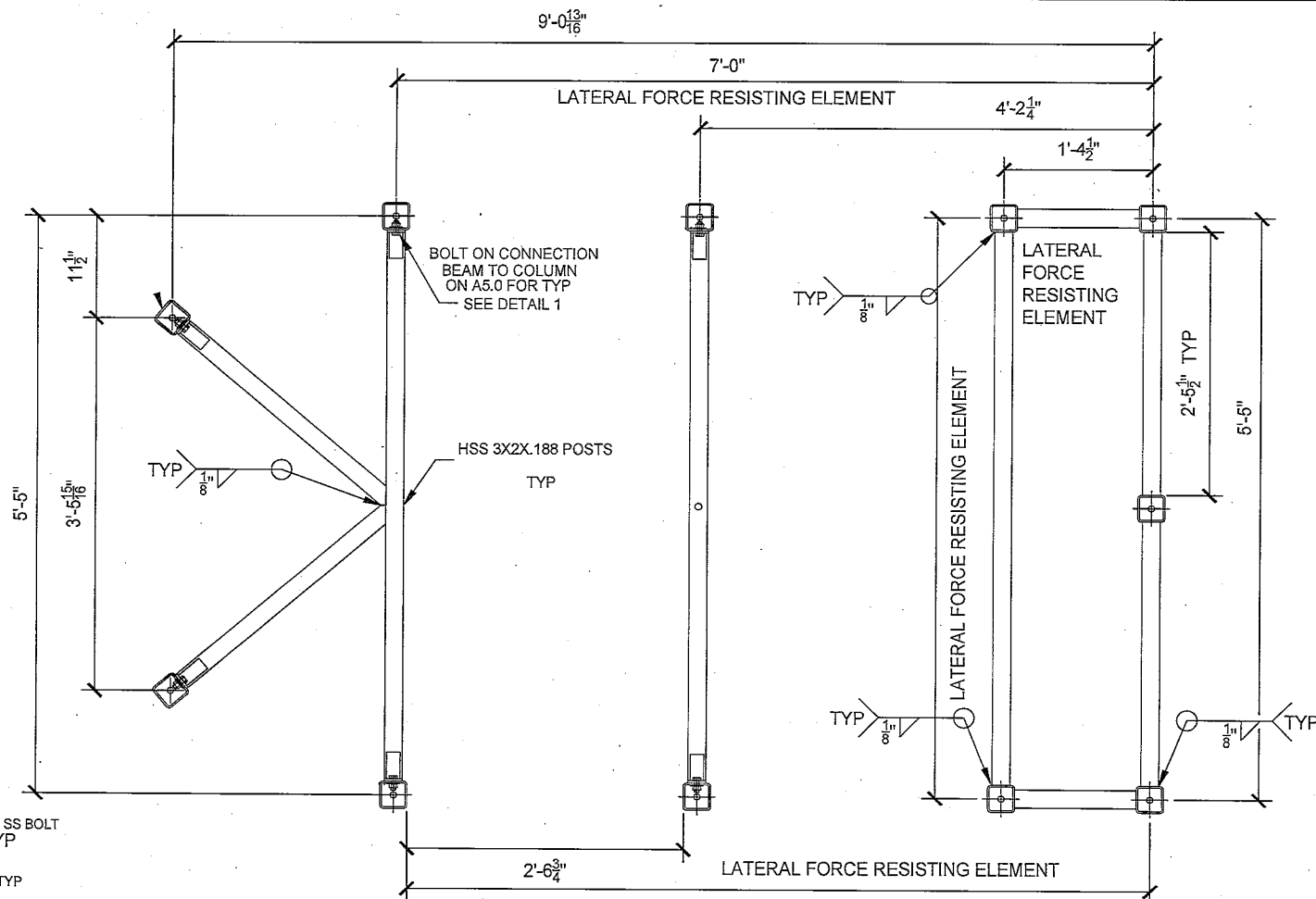
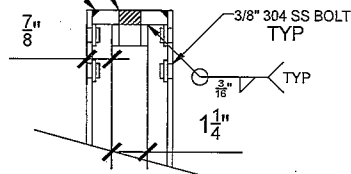


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LEI Proj. # 21230893.000



FOR LIFTING EYE USE
AND BOLTING OF ROOF TO STRUCTURE
TYP

SMAW E309L >



COLUMN AND BEAM LAYOUT

SCALE 1:32

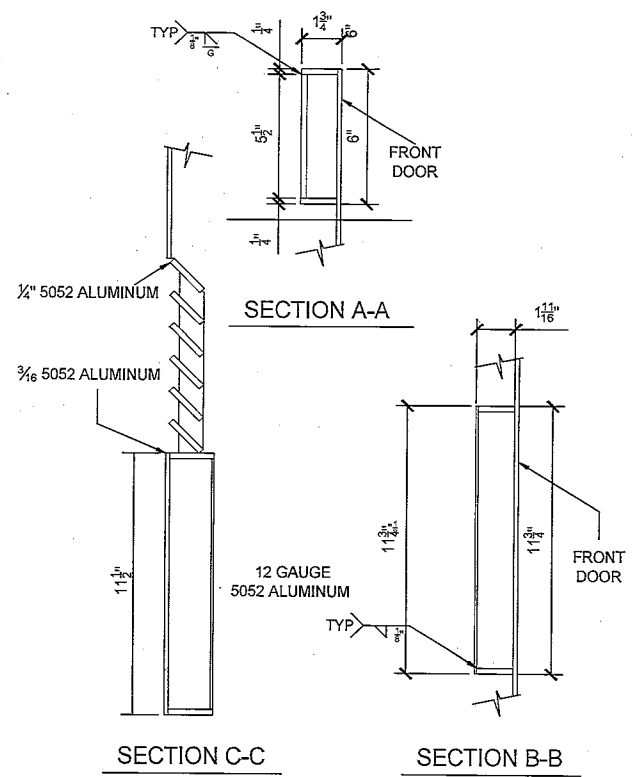
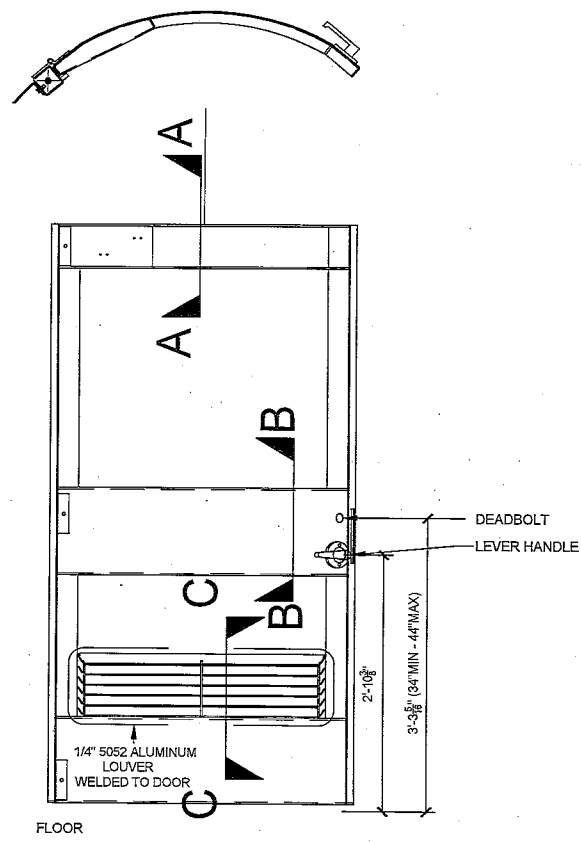
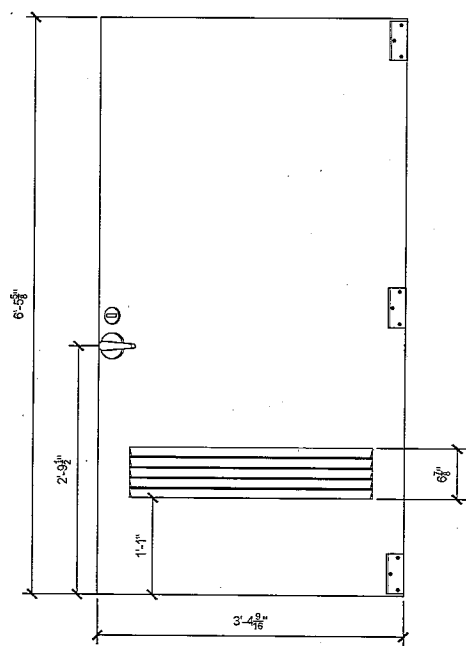
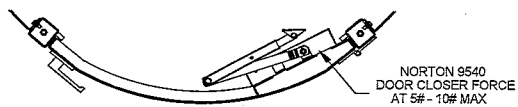
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S1.0

COLUMN LIFTING CAP

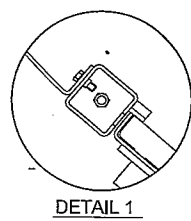
1/2" 304 SS
TAP FOR 3/4-10 Bolt
(11) REQ'D

1.	11/14/23	INITIAL ISSUE	AYK		SOMMERVILLE, MA			
2	12/19/23	REVISED PER REDLINE	AYK		COLUMN & BEAM LAYOUT			
REV	DATE	DESCRIPTION	DRN	APPD	2550 NW 25th Place Portland, OR. 97210 Phone: (503) 226-3968 Fax: (503) 242-2446			
					NAME	DATE	SERIAL NO.	DRW. NO.
					EVAN	08/08/2022		\$1.0



1 LEFT ENTRY DOOR (STANDARD)
PL 3/16" 5052 ALUMINUM
(1) REQ'D

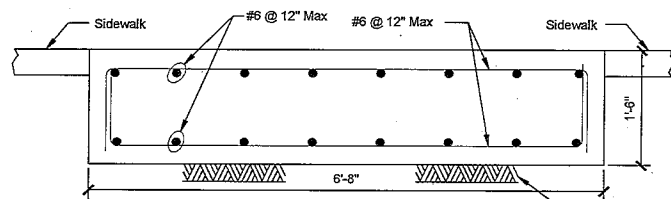
BACKSIDE (INTERIOR)



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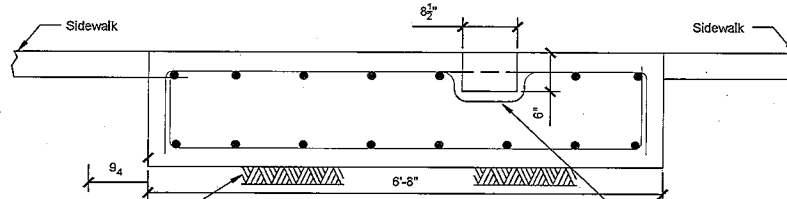
PROFESSIONAL ENGINEER
TREY OR G. WICKIE
STRUCTURAL
No. 52377
REGISTERED
COMMONWEALTH OF MASSACHUSETTS

1.	11/14/23	INITIAL ISSUE	AYK		MADDEN FABRICATION	SOMMERVILLE, MA			
2.	12/19/23	REVISED PER REDLINE	AYK			FRONT DOOR			
REV	DATE	DESCRIPTION	DRN	APPD	2550 NW 25th Place Portland, OR. 97210 Phone: (503) 236-3968 Fax (503) 242-2446	NAME	DATE	SERIAL NO.	DRW. NO.
						EVAN	08/08/2022		S2.0



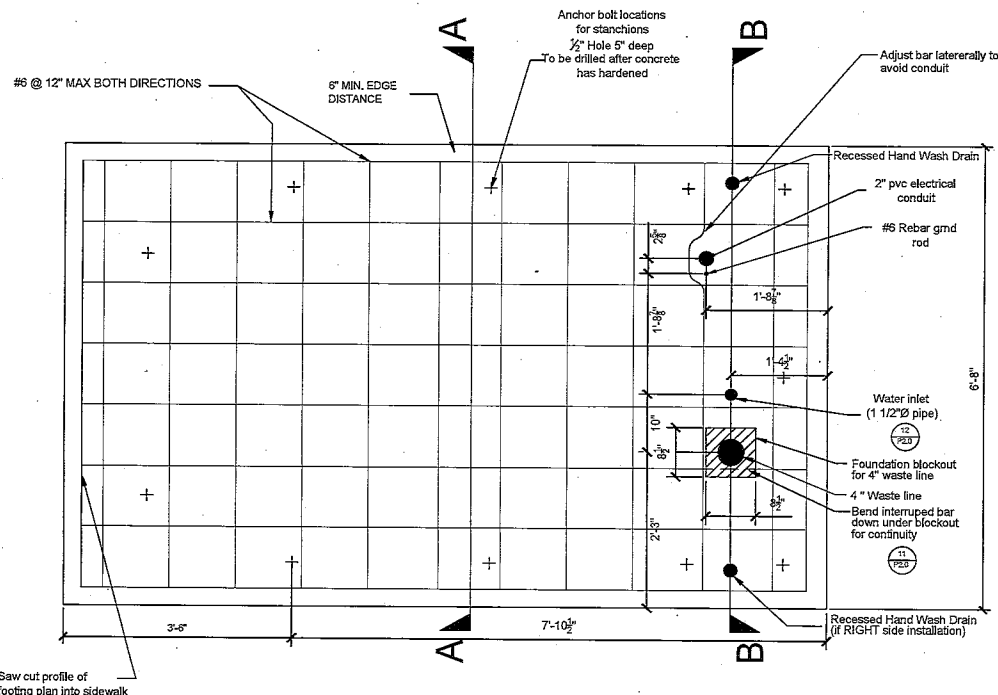
SECTION A-A

EXISTING UNDISTURBED SOIL W/
NET ALLOWABLE BRG PRESSURE =
1500 PSI MIN.



SECTION B-B

Bend interrupted bar
down under blockout
for continuity



GENERAL NOTES:

1. Provide all materials and perform all work according to the current edition of the Specifications of the Authority Having Jurisdiction.
2. All concrete shall be Structural Class 4000. FC = 4000. F'c = 4000psi
3. All reinforcing steel shall conform to ASTM A706 or A615 Grade 60.
4. Place bars 3 inches clear of the nearest face of concrete unless shown otherwise.
5. Concrete footing may be poured integrally with 4' sidewalk, providing a deep cut tool joint is located around perimeter of footing.
6. Stanchions section "A-A" / A1.0 to be fastened to the concrete using 1/2" dia. Titan HD (4" Embed) The Loos columns to be fastened to stanchions.
7. Location of anchor bolts to be located and drilled using template provided before installation by Madden Fabrication.
8. Refer to civil plans for Grade/Slope and location of floor drains.

Larson
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LEI Proj. # 21230893.000



1	11/14/23	INITIAL ISSUE	AYK						
2	12/19/23	REVISED PER REDLINE	AYK						
REV	DATE	DESCRIPTION	DRN	APP'D					

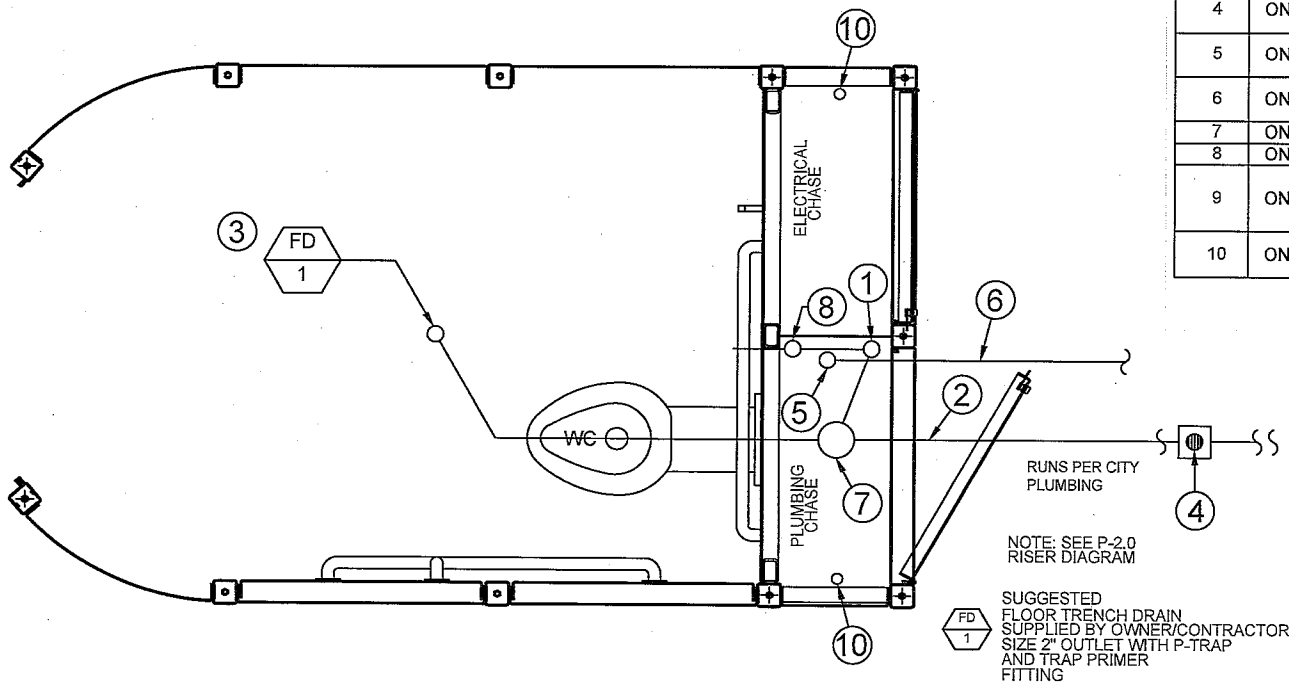
MADDEN
FABRICATION

2550 NW 25th Place Portland, OR. 97210
Phone: (503) 226-3968 Fax (503) 242-2446

SOMMERVILLE, MA

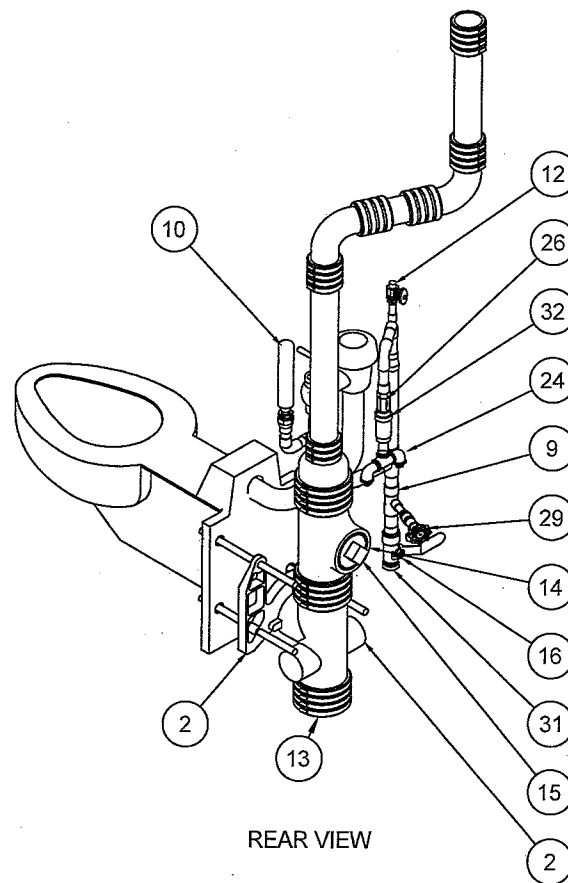
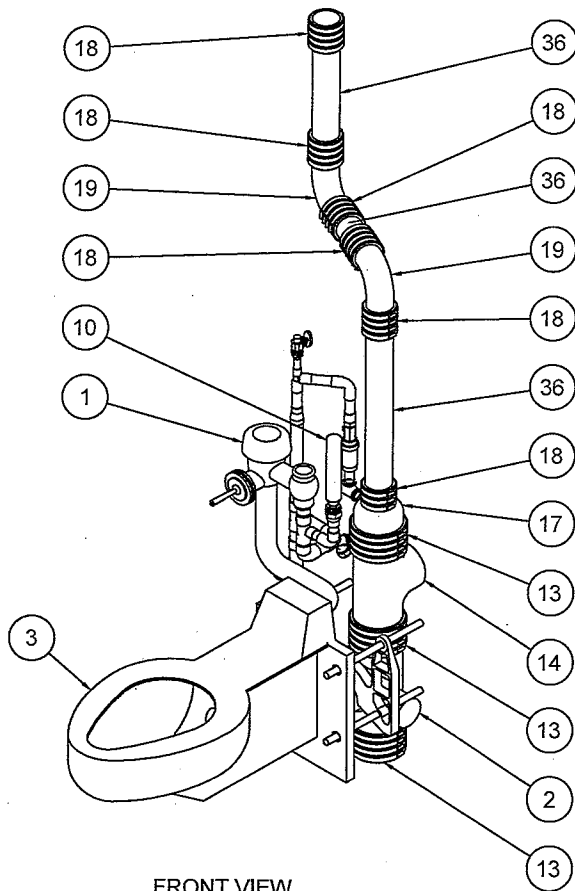
FOUNDATION & ANCHOR BOLT LOCATIONS

NAME	DATE	SERIAL NO.	DRW. NO.
EVAN	08/08/2022		S3.0



BILL OF MATERIAL			
ITEM	QTY	DESCRIPTION	MFG.
1	ONE	2" VENT PIPE TUBE TO ROOF	VARIOUS
2	ONE	LINE TO P.O.C.	VARIOUS
3	ONE	ROUTE CW SUPPLY TO FLOOR DRAIN P-TRAP FROM TRAP PRIMING LOCATED IN PLUMBING CHASE	JAY R. SMITH
4	ONE	4" GRADE CLEAN-OUT (QTY & LOCATION PER LOCAL CODE)	VARIOUS
5	ONE	WATER INLET 1-1/2" C.W. SUPPLY SHOWN ON SHEET 7	VARIOUS
6	ONE	1-1/2" C.W. SUPPLY TO SOV SHOWN ON SHEET 7	VARIOUS
7	ONE	4" SEWER LINE CARRIER SHOW ON P2.0	VARIOUS
8	ONE	TRAP PRIMER VALVE (PER LOCAL CODE)	PRIME-RIGHT
9	ONE	WASH BASIN FLOOR DRAIN P-TRAP FROM TRAP PRIMING LOCATED IN PLUMBING CHASE	VARIOUS
10	ONE	2" SEWER DRAWIN FOR RECESSED HAND WASH	VARIOUS

1.	11/14/23	INITIAL ISSUE	AYK		MADDEN FABRICATION	SOMMERVILLE, MA			
2.	12/19/23	REVISED PER REDLINE	AYK			PLUMBING DETAIL			
REV	DATE	DESCRIPTION	DRN	APP'D	2550 NW 25th Place Portland, OR. 97210 Phone: (503) 226-3968 Fax (503) 242-2446	NAME	DATE	SERIAL NO.	DRW. NO.
						EVAN	08/08/2022		P1.0



BILL OF MATERIAL

ITEM NO.	QTY.	MANUFACTURER NO	DESCRIPTION
1	1	ROYAL 143-1.28	SLOAN - ROYAL 143 FLUSHOMETER, 1.28 GPF, 3-3/4 L, BRASS FINISH
2	1	JR SMITH 0440Y	440 Y W 8" NIPPLE CARRIER - JR SMITH
3	1	2105-W-1-CN-HS	DURA-WARE TOILET W SEAT, 1 1/2" SS 304 CL.
4	2	VIEGA 77022	3/4" ELBOW 90 DEG
5	2	VIEGA 79245	1" MALE ADAPTER
6	2	VIEGA 77027	1" ELBOW 90 DEG
7	1	VIEGA 79315	3/4 FEMALE ADAPTER
8	1	VIEGA 77412	1" TEE
9	1	VIEGA 77432	1" X 1/2" TEE
10	1	PPP SC-750B	WATER HAMMER ARRESTOR 3/4 NPT MALE
11	1	VIEGA 77437	1" X 3/4" TEE
12	1	BRASSCRAFT OR12X-C	1/2" X 3/8" STRAIGHT STOP VALVE
13	3	ANACO 2010	4" NO-HUB RUBBER COUPLING
14	1	AB&I 00190	4" CAST IRON TEE LOW-PRESSURE
15	1	PASCO 1851	4" BRASS PLUG
16	1	UPBA475B-1	1" BALL VALVE
17	1	AB&I 02142	4" X 2" NO-HUB REDUCER
18	6	ANACO 2006	2" NO-HUB RUBBER COUPLING
19	2	AB&I 00190	2" NO-HUB 90 ELBOW CAST IRON
20	2	VIEGA 79215	1/2" MALE ADAPTER
21	1	VIEGA 77417	COPPER TEE 3/4" X 3/4" X 1/2"
22	1	VIEGA 78152	1" X 3/4" REDUCER
23	1	LEGEND 310-103NL	1/2" THREADED BRASS TEE
24	2	LEGEND 310-043 NL	1/2" THREADED BRASS STREET 90 ELBOW
25	1	4568K175 OR EQUIVALENT	1/2" X 3" THREADED BRASS PIPE NIPPLE
26	1	BRASSCRAFT DR583X-R	1/2" SCREWDRIVER STOP
27	1	VIEGA 79310	3/4 X 1/2 FPT" FEMALE FITTING
28	1	PASCO 2152	3/4" PLASTIC CAPS
29	1	LEGEND 107-154NL	1/2" BOILER DRAIN
30	2	LEGEND 310-183 NL	1/2" BRASS PLUG
31	1	LASCO 450-010	1" PVC PLUG
32	1	PPP P2-500	TRAP PRIMER
33	7	5175K126 OR EQUIVALENT	TUBING, TYPE L, 1 TUBE SIZE, COPPER
34	5	5175K125 OR EQUIVALENT	TUBING, TYPE L, 3/4 TUBE SIZE, COPPER
35	2	5175K122 OR EQUIVALENT	TUBING, TYPE K, 1/2 TUBE SIZE, COPPER
36	3	1910N12 OR EQUIVALENT	2" SCH-40 NO-HUB PIPE CAST IRON

1	11/14/23	INITIAL ISSUE	AYK	
2	12/19/23	REVISED PER REDLINE	AYK	
REV	DATE	DESCRIPTION	DRN	APPD

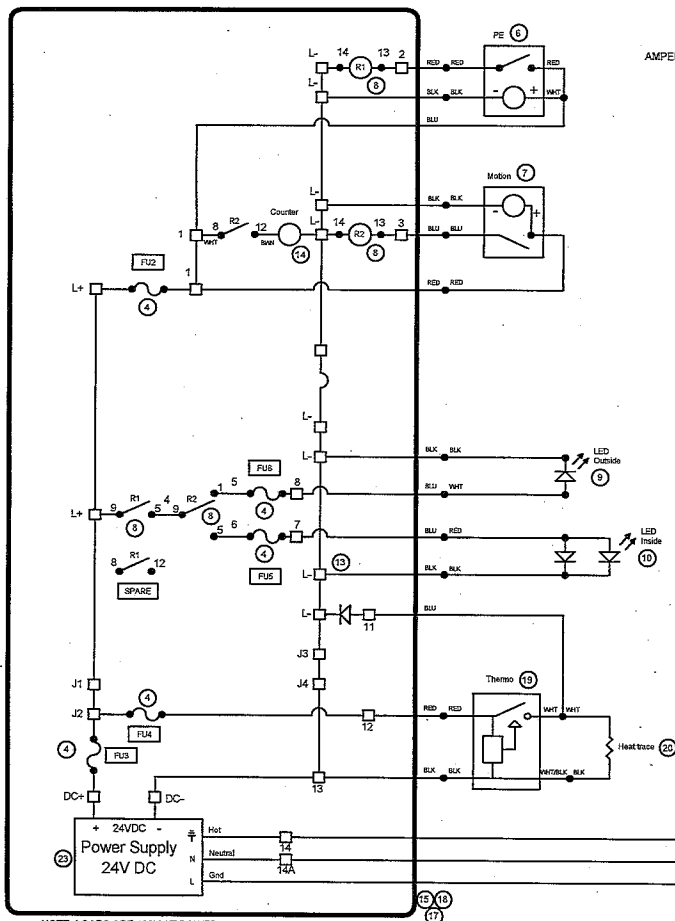
**MADDEN
FABRICATION**

2550 NW 25th Place Portland, OR 97210
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SOMMERVILLE, MA

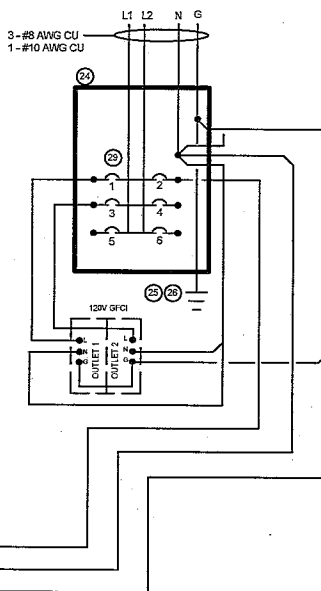
PLUMBING DETAIL

NAME	DATE	SERIAL NO.	DRW. NO.
EVAN	08/08/2022		P2.0



PANEL CAN BE SUPPLIED WITH 240V 1PH OR 208V 1PH
AMPERAGE OF SUPPLY DEPENDS ON CONNECTED OPTIONS/LOADS

240VAC 1PH 40A
ELECTRICAL CONTACTOR TO MAKE
CONNECTION TO SERVICE LOAD CENTER



FUSE/BREAKER DESCRIPTION

FU2	PEMOTON
FU3	DC POWER
FU4	HEAT TRACE
FU5	INSIDE LED LIGHT
FU6	OUTSIDE LED LIGHT
FU7	INSIDE LED LIGHT
FU8	OUTSIDE LED LIGHT
CB8	MAIN 24VDC BREAKER
CB9	PV1 BREAKER
CB10	PV2 BREAKER
CB11	PV3 BREAKER

TERMINAL DETAIL

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BILL OF MATERIAL

ITEM	QTY	DESCRIPTION	MFG.
4	5-7	6A-600V Fuse LPOG-6	Buss
6	ONE	24V Photocell EM-24A2	Watt Stopper
7	ONE	Occupancy Sensor FSP-202	Watt Stopper
8	TWO	24V DC relay 782-2C-24D	Automation Direct
9	ONE	24V LED Light Rope, White 3W/ft x 4ft	Imtra Corp
10	TWO	24V, 5W/LED Blue Acrylic	Current USA
11	ONE	Battery system monitor RM-1	Morningstar
12	ONE	RL-11 Meter Cable	VARIOUS
13	ONE	Ground Bus 44CONN-AL	ITE
14	ONE	1A788 24VDC Electromechanical Counter	Dayton
15	ONE	NEMA1 Enclosure 20"x20" 2020S-1	BENC
16	ONE	6ga Battery Harness cable	ALLBATTERY
17	Lot	14ga Cu wire THHN-14-19STR-CU	VARIOUS
18	Lot	10ga Cu wire Varies	VARIOUS
19	ONE	Solstat 2-10 Solid State Thermostat	ENGENITY
20	ONE	Heat trace - 10 ft, 24V, 3W/ft Kompensator	HEATLINE
21	ONE	15 Amp, 150V DC Breaker	ALTEC CORP
22	ONE	63 Amp, 125V DC Breaker	ALTEC CORP
23	ONE	PULS Dimension CP10.241 Power Supply	PULS
24	ONE	LOAD CENTER, 70A CQ24LYOS 2 BRKR	SQUARE D
25	ONE	Approved UFER ground connection to rebar Sht S2.0	BY OTHERS
26	ONE	4ga Cu wire insulated	VARIOUS
27	LOT	6ga Cu wire insulated	VARIOUS
28	1-3	6ga crimp eyellet and mechanical fastener to PV	VARIOUS
29	ONE	20 Amp Breaker Panel BR48L125SP	EATON
30	ONE	Intermatic FM1D14 Series	VARIOUS
31	ONE	Smart Pac	VARIOUS
32	ONE	Lock Solenoid	VARIOUS
Items not shown on schematic			
A	Lot	Dis Rail 2M - 2 ft	WEIDMULLER
B	EIGHT	Modular Fuse Holder CHCCTDI	BUSS
C	TWO	Relay Pin Base 782-2C-SKT	Automation Direct
D	20	Terminal splice unit	WEIDMULLER
E	TWO	1-5/8" x 1-5/8" x 6" B22SH stud	B-Line or Equal
F	FOUR	3/8" Spring Nut N28	B-Line or Equal
G	FOUR	10-1/2" Shelf Bracket S-205	B-Line or Equal
H	ONE	NEMA1 Panel 20"x20" AW2020	BENC

Jumper Power Configuration		J1 to J2	J3 to J4
Solar Powered only (3 panels)		Yes	Yes
AC Power Only		Yes	Yes
Hybrid AC/Solar Powered (1-2 panels)		No	No

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MADDEN FABRICATION

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SOMMERVILLE, MA

ELECTRICAL SCHEMATIC

NAME	DATE	SERIAL NO.	DRW. NO.
EVAN	08/06/2022		E1.0

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Larson

Sommerville Loo
Sommerville, MA

Structural Calculations

Book 1 of 1
Calculation Release #1

Prepared for
Madden Fabrication
Portland, Oregon

Digitally signed by Trevor G. Wickie
Date: 2023.12.20 11:10:16-06'00'



Larson Engineering, Inc.
Naperville, Illinois
Project Number 21230893.000

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Portland Loo - Seatac **Seatac, WA**

Sheet No.

Design Criteria	101 – 102
Load Determination	201 – 209
Calculations	301 – 339

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Design Criteria



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Design Criteria

Project Information:

Project: Sommerville Loo

Project Location: Sommerville, MA

Project Number: 21230893.000

Load Criteria

1. Structural calculations for Madden Fabrication prototype drawings dated 11/14/2023.
2. Structural Loads per 9th Ed Massachusetts State Building Code

Structural Steel

1. Square and rectangular steel tubes shall meet the requirements of ASTM A-500, Grade B, ($F_y = 46$ ksi, $F_u = 58$ ksi)
2. Steel members are designed per the “Manual of Steel Construction, Allowable Stress Design”, Fifteenth Edition

Stainless Steel

1. Stainless steel alloy designation for plates, all shapes and bars shall be 304 or 316 as shown in drawings and shall meet the requirements of ASTM A-276 ($F_y = 30$ ksi, $F_u = 75$ ksi).
2. Structural stainless steel members are designed per AISC Design Guide 27, “Structural Stainless Steel.”

Concrete

1. Cast-in-place concrete strength is assumed to be $f'_c = 4,000$ psi, normal weight.



Larson

Fasteners, Welds & Anchors

1. Fasteners exposed to weather shall be stainless steel, alloy groups 1, 2, OR 3 (300 Series Only, $F_y = 30$ ksi, $F_u = 75$ ksi).
2. Stainless Steel welding electrode to be minimum E70XX low hydrogen for Grade 50 steel and E308-XX for A304 & A316 stainless steel.
3. All welding shall be by certified welders and shall conform to the latest “Structural Welding Code”, AWS D1.1 and meet AISC minimum requirements for weld size.
4. Threaded concrete bolts shall be Simpson Titen HD anchors having diameter and embedment as called for in the calculations. Install per manufacturer’s recommendations.
5. Substitution requests for alternate products must be approved in writing by the engineer prior to use. Contractor shall provide product/technical information demonstrating that the substituted product is capable of achieving the performance values of the specified product including an icc-es report showing compliance with the relevant building code, seismic use, load resistance, installation category, in-service temperature, installation temperature, etc.

Disclaimers

1. This calculation package is for the final design and installed structural performance of the prefabricated building system. Larson Engineering is not responsible for manufacturing, the installation process, plumbing, electrical, or mechanical design or performance.
2. The following calculation package represents Larson Engineering’s interpretation of the design intent of the shop drawings. Larson Engineering is not responsible for verification of dimensions, material take-offs, installation and coordination with other building trades. If as built conditions differ from the conditions shown in this calculation package, Madden Fabrication must bring these differences to the attention of Larson Engineering so that the as built conditions can be structurally verified.

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Larson

Load Determination



Larson Engineering, Inc.

1488 Bond Street, Suite 100

Naperville, IL 60563

phone: 630-357-0504

JOB TITLE Portland Loo -Somerville

JOB NO. 21230893.000

SHEET NO. _____

CALCULATED BY SRS

DATE 11/29/23

CHECKED BY _____

DATE _____

CS15 Ver 2016.01.10

www.struware.com

STRUCTURAL CALCULATIONS

FOR

Portland Loo -Somerville

Sommerville, MA

Larson Engineering, Inc.

1488 Bond Street, Suite 100

Naperville, IL 60563

phone: 630-357-0504

JOB TITLE Portland Loo -Somerville

JOB NO. 21230893.000

SHEET NO.

CALCULATED BY SRS

DATE 11/29/23

CHECKED BY

DATE

www.struware.com

Code Search**Code:** International Building Code 2015**Occupancy:**

Occupancy Group = B Business

Risk Category & Importance Factors:

Risk Category = I

Wind factor = 1.00

Snow factor = 1.00

Seismic factor = 1.00

Type of Construction:

Fire Rating:

Roof = 0.0 hr

Floor = 0.0 hr

Building Geometry:Roof angle (θ) 0.00 / 12 0.0 deg

Building length (L) 10.6 ft

Least width (B) 6.3 ft

Mean Roof Ht (h) 8.8 ft

Parapet ht above grd 8.8 ft

Minimum parapet ht 0.0 ft

Live Loads:**Roof** 0 to 200 sf: 20 psf

200 to 600 sf: 24 - 0.02Area, but not less than 12 psf

over 600 sf: 12 psf

Floor:

Typical Floor 0 psf

Partitions N/A

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JOB TITLE Portland Loo -Somerville

JOB NO. 21230893.000

SHEET NO.

CALCULATED BY SRS

DATE 11/29/23

CHECKED BY

DATE

Wind Loads :

ASCE 7- 10

Ultimate Wind Speed	127 mph
Nominal Wind Speed	98.4 mph
Risk Category	I
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.849
Kh case 2	0.849
Type of roof	Monoslope

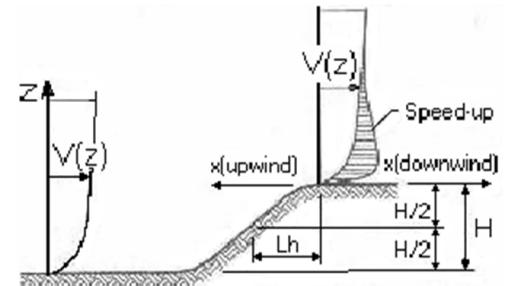
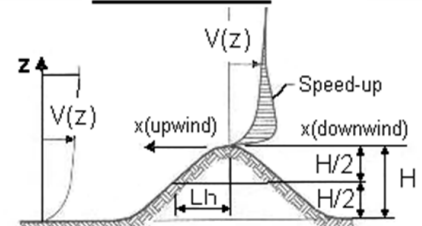
Topographic Factor (Kzt)

Topography	Flat
Hill Height (H)	0.0 ft
Half Hill Length (Lh)	0.0 ft
Actual H/Lh =	0.00
Use H/Lh =	0.00
Modified Lh =	0.0 ft
From top of crest: x =	0.0 ft
Bldg up/down wind?	downwind
H/Lh = 0.00	K ₁ = 0.000
x/Lh = 0.00	K ₂ = 0.000
z/Lh = 0.00	K ₃ = 1.000

At Mean Roof Ht:

$$K_{zt} = (1 + K_1 K_2 K_3)^2 = 1.00$$

H < 15ft; exp C
 $\therefore K_{zt} = 1.0$

**ESCARPMENT****2D RIDGE or 3D AXISYMMETRICAL HILL****Gust Effect Factor**

h =	8.8 ft
B =	6.3 ft
/z (0.6h) =	15.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).

However, if building h/B < 4 then probably rigid structure (rule of thumb).

h/B = 1.39 Therefore, probably rigid structure

G = 0.85 Using rigid structure default**Rigid Structure**

\bar{e} =	0.20
l =	500 ft
Z_{min} =	15 ft
c =	0.20
g_Q, g_v =	3.4
L_z =	427.1 ft
Q =	0.96
I_z =	0.23
G =	0.91 use G = 0.85

Flexible or Dynamically Sensitive Structure

Natural Frequency (η_1) =	0.0 Hz		
Damping ratio (β) =	0		
/b =	0.65		
/α =	0.15		
Vz =	107.2		
N ₁ =	0.00		
R _n =	0.000		
R _n =	28.282	η =	0.000
R _B =	28.282	η =	0.000
R _L =	28.282	η =	0.000
g _R =	0.000		
R =	0.000		
G =	0.000		

h = 8.8 ft

Enclosure Classification

Larson Engineering, Inc.

1488 Bond Street, Suite 100

Naperville, IL 60563

phone: 630-357-0504

JOB TITLE Portland Loo -Somerville

JOB NO. 21230893.000

SHEET NO.

CALCULATED BY SRS

DATE 11/29/23

CHECKED BY

DATE

Test for Enclosed Building: A building that does not qualify as open or partially enclosed.**Test for Open Building:**

All walls are at least 80% open.

$$A_o \geq 0.8A_g$$

Test for Partially Enclosed Building:

	Input
Ao	100000.0 sf
Ag	0.0 sf
Aoi	0.0 sf
Agi	0.0 sf

$$A_o \geq 1.1A_{oi}$$

$$A_o > 4' \text{ or } 0.01A_g$$

$$A_{oi} / A_{gi} \leq 0.20$$

Test
YES
YES
NO

Building is NOT
Partially Enclosed

ERROR: Ag must be greater than Ao

Conditions to qualify as Partially Enclosed Building. Must satisfy all of the following:

$$A_o \geq 1.1A_{oi}$$

$$A_o > \text{smaller of } 4' \text{ or } 0.01 A_g$$

$$A_{oi} / A_{gi} \leq 0.20$$

Where:

Ao = the total area of openings in a wall that receives positive external pressure.

Ag = the gross area of that wall in which Ao is identified.

Aoi = the sum of the areas of openings in the building envelope (walls and roof) not including Ao.

Agi = the sum of the gross surface areas of the building envelope (walls and roof) not including Ag.

Reduction Factor for large volume partially enclosed buildings (Ri) :

If the partially enclosed building contains a single room that is unpartitioned , the internal pressure coefficient may be multiplied by the reduction factor Ri.

Total area of all wall & roof openings (Aog):

0 sf

Unpartitioned internal volume (Vi) :

0 cf

$$R_i = 1.00$$

Altitude adjustment to constant 0.00256 (caution - see code) :Altitude = 0 feet
Constant = 0.00256Average Air Density = 0.0765 lbm/ft³

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DATE 11/29/23

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DATE

Seismic Loads:

IBC 2015

Strength Level Forces

Risk Category : I
 Importance Factor (I) : 1.00
 Site Class : D

Ss (0.2 sec) = 28.00 %g
 S1 (1.0 sec) = 7.00 %g

Fa = 1.576	Sms = 0.441	S _{DS} = 0.294	Design Category = B
Fv = 2.400	Sm1 = 0.168	S _{D1} = 0.112	Design Category = B

Seismic Design Category = D

Number of Stories 1

Structure Type: Light Frame

Horizontal Struct Irregularities: No plan Irregularity

Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: Yes

Building System: **Building Frame Systems**Seismic resisting system: **Light frame (cold-formed steel) walls with wood panels or steel sheets**System Structural Height Limit: **65 ft**

Actual Structural Height (hn) = 8.8 ft

See ASCE7 Section 12.2.5 for exceptions and other system limitation

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) : 7
 Over-Strength Factor (Ωo) : 2
 Deflection Amplification Factor (Cd) : 4.5
 S_{DS} = 0.294
 S_{D1} = 0.112

Seismic Load Effect (E) = $\rho Q_E \pm 0.2 S_{DS} D$ = $\rho Q_E \pm 0.059 D$
 Special Seismic Load Effect (Em) = $\Omega_o Q_E \pm 0.2 S_{DS} D$ = $2.0 Q_E \pm 0.059 D$

ρ = redundancy coefficient
 Q_E = horizontal seismic force
 D = dead load

PERMITTED ANALYTICAL PROCEDURES**Simplified Analysis** - Use Equivalent Lateral Force Analysis**Equivalent Lateral-Force Analysis** - Permitted

Building period coef. (C_T) = 0.020
 Approx fundamental period (Ta) = $C_T h_n^x$ = 0.102 sec x = 0.75 Tmax = CuTa = 0.171
 User calculated fundamental period (T) = 0 sec Use T = 0.102
 Long Period Transition Period (TL) : ASCE7 map = 6
 Seismic response coef. (Cs) : S_{DS}/R = 0.042
 need not exceed Cs = S_{D1}/R = 0.157
 but not less than Cs = $0.044 S_{D1}$ = 0.013
 USE Cs = 0.042
 Design Base Shear V = 0.042W

Model & Seismic Response Analysis - Permitted (see code for procedure)**ALLOWABLE STORY DRIFT**

Structure Type: All other structures

Allowable story drift = 0.020hsx where hsx is the story height below level

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Larson

Calculations

DESIGN CRITERIA

Code: 9th Ed Massachusetts State Building Code

Roof Minimum Load: $LL := 30 \text{ psf}$

Snow Load:

Ground Snow Load: $p_g := 40 \text{ psf}$

Importance Factor: $I_S := 1.00$

Wind Loads:

Ultimate Wind Speed: $V := 127 \text{ mph}$

Exposure Category: Exposure := "C"

Earthquake Load:

$S_S := 0.28$ $S_1 := 0.07$

Occupancy Category: Occupancy := "I"

Seismic Design Category: SDC := "D"

Site Class: SiteClass := "D"

Structural System: Light frame walls sheathed with wood structural panels rated for shear resistance or steel sheets

$R_{eq} := 7.0$

Analysis Procedure: Equivalent Lateral Force Analysis

Importance Factor: $I_e := 1.00$

Redundancy Factor: $\rho := 1.3$

Allowable Soil Bearing Pressure: $Q_{allow} := 1500 \text{ psf}$

Material Reference Design Standards

Steel: AISC Steel Construction Manual - 13th Edition

Concrete: ACI 318-14 Building Code Requirements for Structural Concrete

GRAVITY LOADS

Building Geometry

Greater Building Length:

$$L := 10\text{-ft} + 7.5\text{in}$$

Least Building Width:

$$W := 6\text{-ft} + 4\text{in}$$

Mean Roof Height:

$$H := 8\text{ft} + 8.8125\text{in}$$

Loading

Roof Loads

3/16" Steel Plate:

$$rdl_1 := 11\text{psf}$$

Steel Framing:

$$rdl_2 := 6\text{psf}$$

MEP Components:

$$rdl_3 := 4\text{psf}$$

Miscellaneous:

$$rdl_4 := 2\text{psf}$$

Roof Dead Load:

$$RDL := \sum rdl = 23\text{psf}$$

Roof Live Load:

$$RLL := 20\text{psf}$$

*Minimum Roof Load

Snow Load

Exposure Factor (Table 7-2):

$$C_e := 1.0$$

Thermal Factor (Table 7-3):

$$C_t := 1.0$$

Sloped Factor (Figure 7-2):

$$C_s := 1.0$$

Flat Roof Snow Load (Eq. 7.3-1):

$$RSL := \begin{cases} \max(0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g, I_s \cdot p_g) & \text{if } p_g \leq 20\text{psf} \\ \max(30\text{psf}) & \text{if } p_g > 20\text{psf} \end{cases} = 30\text{psf}$$

$$RSL := RSL = 30\text{psf}$$

Exterior Wall Load

3/16" Steel Plate:

$$wdl_1 := 11\text{psf}$$

Louvers:

$$wdl_2 := 2\text{psf} \quad \text{*in addition to steel plate}$$

Steel Framing:

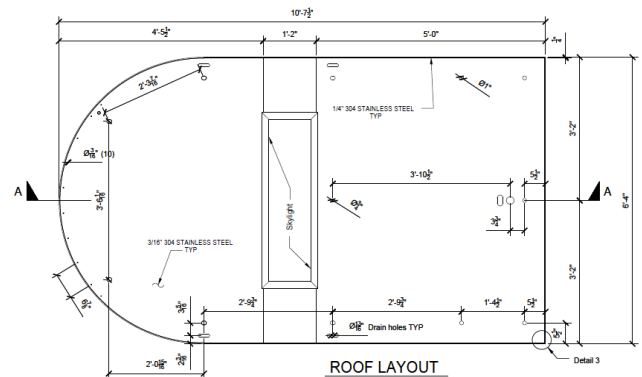
$$wdl_3 := 6\text{psf}$$

MEP Components:

$$wdl_4 := 4\text{psf}$$

Wall Dead Load:

$$WDL := \sum wdl = 23\text{psf}$$



LATERAL LOADS

Wind Loads

End Zone Wall Loading:	$p_{wallend} := 0.6 \cdot 19.5 \text{psf} = 11.7 \text{psf}$
Interior Zone Wall Loading:	$p_{wallint} := 0.6 \cdot 17.6 \text{psf} = 10.56 \text{psf}$
Positive Wall Loading:	$p_{wallpos} := 0.6 \cdot 16 \text{psf} = 9.6 \text{psf}$
End Zone Roof Loading:	$p_{roofend} := 0.6 \cdot 26.8 \text{psf} = 16.08 \text{psf}$
Interior Zone Roof Loading:	$p_{roofint} := 0.6 \cdot 24.1 \text{psf} = 14.46 \text{psf}$
Positive Roof Loading:	$p_{roofpos} := 0.6 \cdot 16 \text{psf} = 9.6 \text{psf}$
End Zone Wall Loading (10ft ²):	$p_{cwallend} := 0.6 \cdot 25 \text{psf} = 15 \text{psf}$
End Zone Roof Loading (16ft ²):	$p_{croofend} := 0.6 \cdot 20.3 \text{psf} = 12.18 \text{psf}$
Positive Roof Loading (16ft ²):	$p_{croofpos} := 0.6 \cdot 16 \text{psf} = 9.6 \text{psf}$

USE C&C LOADING @ 100SF
FOR WALLS & 70SF FOR ROOF

*multiply values by 0.6 to convert to service level loads

Edge Strip Dimemnsion:	$a := 3 \text{ft}$
End Zone Dimemnsion:	$2 \cdot a = 6 \text{ft}$

*Note: Use Components & Cladding wind loading for Main Wind Force Resisting System checks due to small size of structure - C&C loads are higher than MWFRS loads and therefore will be conservative.

Total Wind Load Base Shear

Area of elevation of building:	$A_{max} := L \cdot H = 92.8 \text{ft}^2$
Wind Design Base Shear:	$VW := A_{max} \cdot p_{wallend} = 1.09 \text{kip}$

Wind Load Parallel to Long Walls

Area of elevation of building:	$A_{wall} := W \cdot \left(\frac{H}{2}\right) = 27.66 \text{ft}^2$
Area of end zone of wall:	$A_{wallend} := (2 \cdot a) \cdot \left(\frac{H}{2}\right) = 26.2 \text{ft}^2$
Area of body of wall:	$A_{wallint} := A_{wall} - A_{wallend} = 1.46 \text{ft}^2$
Load to roof from wind load parallel to long walls:	$VWS := A_{wallend} \cdot p_{wallend} + A_{wallint} \cdot p_{wallint} = 0.32 \text{kip}$

Wind Load Parallel to Short Walls

Area of wall of building:	$A_{wall} := L \cdot \left(\frac{H}{2}\right) = 46.4 \text{ft}^2$
Area of end zone of walls:	$A_{wallend} := (2 \cdot a) \cdot \left(\frac{H}{2}\right) = 26.2 \text{ft}^2$
Area of body of wall:	$A_{wallint} := A_{wall} - A_{wallend} = 20.2 \text{ft}^2$
Load to roof from wind load parallel to short walls:	$VWL := A_{wallend} \cdot p_{wallend} + A_{wallint} \cdot p_{wallint} = 0.52 \text{kip}$

Seismic Loads

Seismic Response Coefficient: $C_s := 0.042$ $C_{sASD} := 0.7 \cdot C_s = 0.029$

Area of Roof: $A_{roofE} := L \cdot W = 67 \text{ ft}^2$

Weight of Structure: $Weight := RDL \cdot A_{roofE} + (WDL) \cdot [(2 \cdot L + 2 \cdot W) \cdot H] = 8.36 \cdot \text{kip}$

Seismic Design Base Shear: $V_E := C_{sASD} \cdot Weight \cdot \rho = 0.32 \cdot \text{kip}$

Seismic Loading to Roof: $V_{roof} := C_{sASD} \cdot [RDL \cdot A_{roofE} + (WDL) \cdot [(2 \cdot L + 2 \cdot W) \cdot \frac{H}{2}]] \cdot \rho = 0.19 \cdot \text{kip}$

***Reference Load Determination section for seismic, wind, and snow loading calculations**

Lateral Loading on Structure

	Loads Parallel to Long Wall	Loads Parallel to Short Wall
Seismic Loads	$V_{\text{roof}} = 0.19 \cdot \text{kip}$	$V_{\text{roof}} = 0.19 \cdot \text{kip}$
Wind Loads	$V_{\text{WS}} = 0.32 \cdot \text{kip}$	$V_{\text{WL}} = 0.52 \cdot \text{kip}$

Governing Lateral Design Loads (maximum of wind or seismic loading):

Parallel to Long Walls:	$V_{\text{long}} := \max(V_{\text{WS}}, V_{\text{roof}}) = 0.32 \cdot \text{kip}$
Parallel to Short Walls:	$V_{\text{short}} := \max(V_{\text{WL}}, V_{\text{roof}}) = 0.52 \cdot \text{kip}$

Wind Controls

Wind Controls

GRAVITY DESIGN: STEEL ROOF PLATE

Material Properties - 304 Stainless Steel

Yield Strength: $F_y := 30\text{ksi}$

Modulus of Elasticity: $E := 23800\text{ksi}$

Thickness of Plate: $t := 0.1875\text{in}$

Loading

Maximum Load on Plate:

$$TL_{pos} := \max(RDL + RLL, RDL + RSL, RDL + p_{croofpos}, RDL + 0.75 \cdot p_{croofpos} + 0.75 \cdot RLL, RDL + 0.75 \cdot p_{croofpos} + 0.75 \cdot RSL)$$

$$TL_{pos} = 53\text{psf}$$

$$TL_{neg} := \max(0.6 \cdot RDL + p_{croofend}) = 26\text{psf}$$

$$TL_{max} := \max(TL_{pos}, TL_{neg}) = 53\text{psf}$$

Maximum Load on Plate for Deflection Calculation:

$$TL_{pos\Delta} := \max(RDL + RLL, RDL + RSL, RDL + 0.7 \cdot p_{croofpos}, RDL + 0.75 \cdot 0.7 \cdot p_{croofpos} + 0.75 \cdot RLL)$$

$$TL_{pos\Delta} = 53\text{psf}$$

$$TL_{neg\Delta} := \max(0.6 \cdot RDL + 0.7 \cdot p_{croofend}) = 22\text{psf}$$

$$TL_{max\Delta} := \max(TL_{pos\Delta}, TL_{neg\Delta}) = 53\text{psf}$$

Geometry

Plate Width: $a := 6\text{ft} + 4\text{in}$

Plate Span Length: $b := 30.75\text{in}$

Tabulated Values:

(relations among load, stress and deflection are expressed by dimensionless coefficients shown below)

See Roark's Formulas for Stress & Strain, Sixth Edition - Plate support on 2 sides and pinned

$$\text{coef}_q = \frac{TL_{max} \cdot b^4}{E \cdot t^4} \quad \text{coef}_\sigma = \frac{\sigma \cdot b^2}{E \cdot t^2} \quad \text{coef}_y = \frac{y}{t}$$

$$\text{coef}_q := (0 \ 12.5 \ 25 \ 50 \ 75 \ 100 \ 125 \ 150 \ 175 \ 200 \ 250)^T$$

$$\text{coef}_{\sigma 1} := (0 \ 3.8 \ 5.8 \ 8.7 \ 10.9 \ 12.8 \ 14.3 \ 15.6 \ 17.0 \ 18.2 \ 20.5)^T$$

$$\text{coef}_{y1} := (0 \ 0.430 \ 0.650 \ 0.93 \ 1.13 \ 1.26 \ 1.37 \ 1.47 \ 1.56 \ 1.63 \ 1.77)^T$$

$$\text{coef}_{\sigma 15} := (0 \ 4.48 \ 6.81 \ 9.92 \ 12.25 \ 14.22 \ 16 \ 17.50 \ 18.9 \ 20.3 \ 22.8)^T$$

$$\text{coef}_{y15} := (0 \ 0.625 \ 0.879 \ 1.18 \ 1.37 \ 1.53 \ 1.68 \ 1.77 \ 1.88 \ 1.96 \ 2.12)^T$$

$$\text{coef}_{\sigma 2} := (0 \ 4.87 \ 7.16 \ 10.3 \ 12.6 \ 14.6 \ 16.4 \ 18 \ 19.4 \ 20.9 \ 23.6)^T$$

$$\text{coef}_{y2} := (0 \ 0.696 \ 0.946 \ 1.24 \ 1.44 \ 1.6 \ 1.72 \ 1.84 \ 1.94 \ 2.03 \ 2.2)^T$$

$$\text{coef}_\sigma := \begin{cases} \text{coef}_{\sigma 1} & \text{if } \frac{a}{b} \leq 1.25 \\ \text{coef}_{\sigma 15} & \text{if } 1.25 < \frac{a}{b} \leq 1.75 \\ \text{coef}_{\sigma 2} & \text{otherwise} \end{cases} \quad \text{coef}_y := \begin{cases} \text{coef}_{y1} & \text{if } \frac{a}{b} \leq 1.25 \\ \text{coef}_{y15} & \text{if } 1.25 < \frac{a}{b} \leq 1.75 \\ \text{coef}_{y2} & \text{otherwise} \end{cases}$$

Check Stress

$$\sigma := \text{linterp} \left[(\text{coef}_q), \text{coef}_\sigma, \frac{T_{L\max} \cdot b^4}{E \cdot t^4} \right] \cdot \left(\frac{E \cdot t^2}{b^2} \right) = 3.86 \cdot \text{ksi}$$

$$\sigma := \text{if} \left(\frac{T_{L\max} \cdot b^4}{E \cdot t^4} < 250, "OK", "OUT OF RANGE" \right) = "Ok"$$

$$\frac{(4.2 \text{ ksi}) \cdot 1.67}{F_y} = 0.23 < 1.00 \text{ OK}$$

Check Deflection

$$y_{\text{allow}} := \frac{b}{200} = 0.15 \cdot \text{in}$$

$$y := \text{linterp} \left[(\text{coef}_q), \text{coef}_y, \frac{T_{L\max} \Delta \cdot (b)^4}{E \cdot t^4} \right] \cdot t = 0.12 \cdot \text{in}$$

$$\frac{y}{y_{\text{allow}}} = 0.76 < 1.00 \text{ OK}$$

3/16" stainless steel plate is adequate for roof

GRAVITY DESIGN: STEEL ROOF PLATE CONNECTIONS

Maximum Uplift on Plate: $T_{Lneg} = 25.98 \cdot \text{psf}$

Plate Width: $a = 6.33 \cdot \text{ft}$

Plate Span Length: $b = 2.56 \cdot \text{ft}$

Number of Fasteners: $n := 2$

Maximum Tension on Fastener: $T_{fast} := T_{Lneg} \cdot b \cdot \frac{a}{n} = 210.82 \cdot \text{lbf}$

Allowable Tension 3/8" ϕ 304
Stainless Bolt :

$T_{allow} := 1612 \cdot \text{lbf}$

Minimum Material Thickness
for Allowable Tension:

$t_{min} := 0.2319 \cdot \text{in}$

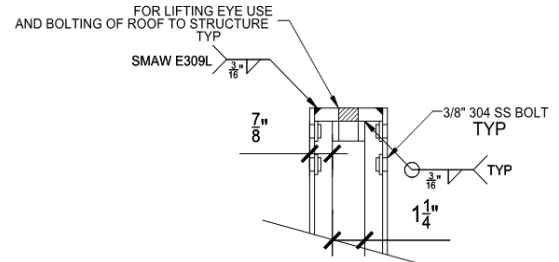
Beam Wall Thickness:

$t_{wall} := 0.174 \cdot \text{in} \quad (\text{HSS}3 \times 2 \times 3/16)$

Allowable Pullout:

$T_{pullout} := \min \left[T_{allow}, T_{allow} \cdot \left(\frac{t_{wall}}{t_{min}} \right) \right] = 1210 \cdot \text{lbf}$

$\frac{T_{fast}}{T_{pullout}} = 0.17 < 1.00 \text{ OK}$



(2) 3/8" diameter bolts are adequate for roof plate to roof beam connection. Hold-down bolts at columns shall also be used. Bolts to be 304 stainless steel, condition "A" ($F_u = 73 \text{ksi}$ minimum).

GRAVITY DESIGN: STEEL ROOF BEAMS

HSS3x2x3/16 Section Properties

$$\begin{aligned} I_x &:= 1.77 \text{ in}^4 & S_x &:= 1.18 \text{ in}^3 & r_x &:= 1.07 \text{ in} & Z_x &:= 1.48 \text{ in}^3 & F_y &:= 30 \text{ ksi} & \Omega_b &:= 1.67 \\ I_y &:= 0.932 \text{ in}^4 & S_y &:= 0.932 \text{ in}^3 & r_y &:= 0.778 \text{ in} & Z_y &:= 1.12 \text{ in}^3 \\ t &:= 0.174 \text{ in} & b &:= 8.49 \text{ in} & h &:= 14.2 \text{ in} & B_f &:= 2 \text{ in} & H_w &:= 3 \text{ in} \end{aligned}$$

Determine Nominal Moment Capacity

$$\text{Slenderness} := \begin{cases} \text{"compact"} & \text{if } \frac{b}{t} < 1.12 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"slender"} & \text{if } \frac{b}{t} > 1.40 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"noncompact"} & \text{if } 1.12 \cdot \sqrt{\frac{E}{F_y}} \leq \frac{b}{t} \leq 1.40 \cdot \sqrt{\frac{E}{F_y}} \end{cases} = \text{"compact"}$$

Yielding

$$M_p := F_y \cdot Z_x = 3.7 \text{ kip-ft}$$

$$M_n := M_p = 3.7 \text{ kip-ft}$$

Flange Local Buckling

$$b_e := \min \left[1.92 \cdot t \cdot \sqrt{\frac{E}{F_y}} \cdot \left(1 - \frac{0.38}{\frac{b}{t}} \cdot \sqrt{\frac{E}{F_y}} \right), b \right] = -2.45 \text{ in}$$

$$I_{eff} := I_x - 2 \cdot \left[(b - b_e) \cdot t \cdot (B_f - t)^2 + \frac{(b - b_e) \cdot t^3}{12} \right] = -2.79 \text{ in}^4$$

$$S_e := \frac{I_{eff}}{\frac{H_w}{2}} = -1.86 \text{ in}^3$$

$$M_{n_flb} := \begin{cases} \min \left[M_p - (M_p - F_y \cdot S_x) \cdot \left(3.57 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E}} - 4.0 \right), M_p \right] & \text{if Slenderness = "noncompact"} \\ F_y \cdot S_e & \text{if Slenderness = "slender"} \\ M_n & \text{if Slenderness = "compact"} \end{cases} = 3.7 \text{ kip-ft}$$

Web Local Buckling

$$M_{n_wlb} := \begin{cases} \min \left[M_p - (M_p - F_y \cdot S_x) \cdot \left(0.305 \cdot \frac{h}{t} \cdot \sqrt{\frac{F_y}{E}} - 0.738 \right), M_p \right] & \text{if Slenderness = "noncompact"} \\ M_n & \text{if Slenderness = "compact"} \end{cases} = 3.7 \text{ kip-ft}$$

$$M_n := \min(M_n, M_{n_flb}, M_{n_wlb}) = 3.7 \text{ kip-ft}$$

Applied Load: $TL_{max} = 53 \cdot \text{psf}$

Maximum Beam Span: $\text{span} := 5\text{ft} + 5\text{in}$

Worst Case Tributary Width: $TW := 2\text{ft} + 10\text{in}$

*Check center beam - worst case loading

Distributed Load to Beam: $w := TL_{max} \cdot TW = 150 \cdot \text{plf}$

Distributed Load to Beam:
(for deflection calculation) $w_{\Delta} := TL_{max\Delta} \cdot TW = 150 \cdot \text{plf}$

Maximum Beam End Reaction: $R_{max} := \frac{w \cdot \text{span}}{2} = 0.41 \cdot \text{kip}$

Check Stress

Moment: $M_{beam} := \frac{w \cdot \text{span}^2}{12} = 0.37 \cdot \text{kip} \cdot \text{ft}$

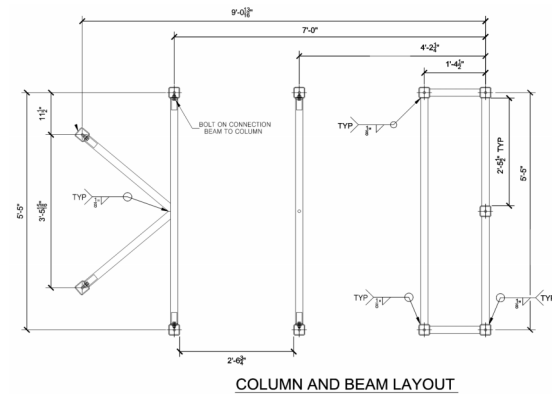
$$\frac{M_{beam} \cdot \Omega_b}{M_n} = 0.17 < 1.00 \text{ OK}$$

Check Deflection

Allowable Deflection: $\Delta_{allow} := \frac{\text{span}}{240} = 0.27 \cdot \text{in}$

Deflection: $\Delta_{beam} := \frac{5 \cdot w_{\Delta} \cdot \text{span}^4}{384 \cdot E \cdot I_x} = 0.07 \cdot \text{in}$

$$\frac{\Delta_{beam}}{\Delta_{allow}} = 0.25 < 1.00 \text{ OK}$$



HSS3x2x3/16" tubes are adequate for roof beams

GRAVITY DESIGN: STEEL ROOF BEAM CONNECTIONS

Maximum Beam End Reaction: $R_{max} = 0.41 \text{ kip}$

Allowable Shear Stress (316SS): $F_v := 12.99 \text{ ksi}$

Allowable tensile Stress (316SS): $F_t := 56.25 \text{ ksi}$

Nominal Bolt Diameter: $d := 0.375 \text{ in}$

Thread Root Area: $A_R := 0.0699 \text{ in}^2$

Tensile Strength of HSS: $F_{uHSS} := 58 \text{ ksi}$

Tensile Strength of Cap Plate: $F_{uCAP} := 58 \text{ ksi}$

$$F_u := \min(F_{uHSS}, F_{uCAP}) = 58 \text{ ksi}$$

HSS Wall Thickness: $t = 0.174 \text{ in}$

Cap Plate Thickness: $t_{cap} := 0.1875 \text{ in}$

$$t := \min(t, t_{cap}) = 0.174 \text{ in}$$

Check Bolted Condition

Maximum Bolt Loads: $V_{max} := \frac{R_{max}}{2} = 203 \text{ lbf}$ $T_{max} := \frac{M_{beam}}{2.5 \text{ in}} = 1.76 \text{ kip}$

Clear Distance to Plate Edge: $l_c := 0.75 \text{ in} - \frac{(d + 0.0625 \text{ in})}{2} = 0.53 \text{ in}$

Safety Factor: $\Omega_{brg} := 2.0$

Bearing Strength: $V_{brg} := \min\left(\frac{1.2 \cdot l_c \cdot t \cdot F_u}{\Omega_{brg}}, \frac{2.4 \cdot d \cdot t \cdot F_u}{\Omega_{brg}}\right) = 3217 \text{ lbf}$

Bolt Strength: $V_{bolt} := F_v \cdot A_R = 908 \text{ lbf}$

$$V_{allow} := \min(V_{brg}, V_{bolt}) = 908 \text{ lbf}$$

$$T_{bolt} := F_t \cdot A_R = 3.93 \text{ kip}$$

$$\left(\frac{T_{max}}{T_{bolt}}\right)^2 + \left(\frac{V_{max}}{V_{allow}}\right)^2 = 0.25 < 1.00 \text{ OK}$$

(2) 3/8" diameter bolts are adequate for roof beam connection to posts.
Bolts to be 316 stainless steel, condition "A" ($F_u = 75 \text{ ksi min}$).

Check Welded Condition

Weld Thickness: $t_w := 0.125 \text{ in}$

Filler Metal Strength: $F_{EXX} := 70 \text{ ksi}$

Safety Factor: $\Omega_w := 2.0$

Weld Width: $b_w := 2 \text{ in}$

Weld Height: $d_w := 3 \text{ in}$

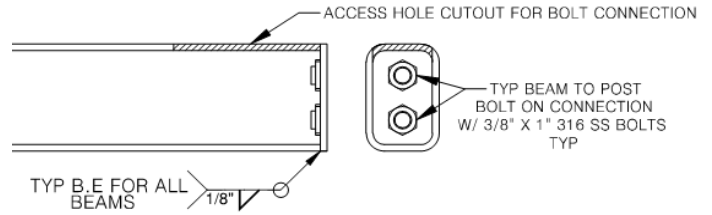
Total Weld Length: $A_w := 2 \cdot b_w + 2 \cdot d_w = 10 \text{ in}$

Actual Weld Loads: $V_{weld} := \sqrt{\left(\frac{R_{max}}{A_w}\right)^2 + \left(\frac{M_{beam}}{9 \text{ in}^2}\right)^2} = 0.49 \frac{\text{kip}}{\text{in}}$

Allowable Weld Strength: $V_{allow} := \frac{0.6 \cdot F_{EXX} \cdot 0.707 \cdot t_w}{\Omega_w} = 1.86 \frac{\text{kip}}{\text{in}}$

$$\frac{V_{weld}}{V_{allow}} = 0.26 < 1.00 \text{ OK}$$

1/8" fillet weld all-around is adequate for roof beam connection to posts.



GRAVITY DESIGN: STEEL POSTS

HSS3x3x3/16 Section Properties

$$\begin{aligned}
 I_x &:= 2.46 \text{ in}^4 & S_x &:= 1.64 \text{ in}^3 & r_x &:= 1.14 \text{ in} & Z_x &:= 1.97 \text{ in}^3 & F_y &:= 30 \text{ ksi} & \Omega_c &:= 1.67 \\
 I_y &:= 2.46 \text{ in}^4 & S_y &:= 1.64 \text{ in}^3 & r_y &:= 1.14 \text{ in} & Z_y &:= 1.97 \text{ in}^3 & A &:= 1.89 \text{ in}^2 & \Omega_b &:= 1.67 \\
 t &:= 0.174 \text{ in} & b &:= 14.2 \cdot t = 2.47 \text{ in} & h &:= 14.2 \cdot t = 2.47 \text{ in} & B_f &:= 3 \text{ in} & H_w &:= 3 \text{ in} \\
 K &:= 1.0 & H &:= 8.73 \text{ ft}
 \end{aligned}$$

Determine Nominal Axial Capacity

$$\text{Slenderness} := \begin{cases} \text{"nonslender"} & \text{if } \frac{b}{t} < 1.40 \cdot \sqrt{\frac{E}{F_y}} = \text{"nonslender"} \\ \text{"slender"} & \text{if } \frac{b}{t} \geq 1.40 \cdot \sqrt{\frac{E}{F_y}} \end{cases}$$

Flexural Buckling

$$F_e := \frac{\pi^2 \cdot E}{\left(\frac{K \cdot L}{r_x}\right)^2} = 18.78 \text{ ksi}$$

$$F_{Cr} := \begin{cases} 0.658 \cdot \frac{F_y}{F_e} \cdot F_y & \text{if } \frac{K \cdot H}{r_x} \leq 4.71 \cdot \sqrt{\frac{E}{F_y}} = 15.37 \text{ ksi} \\ 0.877 \cdot F_e & \text{if } \frac{K \cdot H}{r_x} > 4.71 \cdot \sqrt{\frac{E}{F_y}} \end{cases}$$

$$P_n := \begin{cases} F_{Cr} \cdot A & \text{if Slenderness} = \text{"nonslender"} = 29.05 \text{ kip} \\ \text{"further analysis required"} & \text{if Slenderness} = \text{"slender"} \end{cases}$$

Determine Nominal Moment Capacity

$$\text{Slenderness} := \begin{cases} \text{"compact"} & \text{if } \frac{b}{t} < 1.12 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"slender"} & \text{if } \frac{b}{t} > 1.40 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"noncompact"} & \text{if } 1.12 \cdot \sqrt{\frac{E}{F_y}} \leq \frac{b}{t} \leq 1.40 \cdot \sqrt{\frac{E}{F_y}} \end{cases} = \text{"compact"}$$

Yielding

$$M_p := F_y \cdot Z_x = 4.92 \cdot \text{kip} \cdot \text{ft}$$

$$M_n := M_p = 4.92 \cdot \text{kip} \cdot \text{ft}$$

Flange Local Buckling

$$b_e := \min \left[1.92 \cdot t \cdot \sqrt{\frac{E}{F_y}} \cdot \left(1 - \frac{0.38}{\frac{b}{t}} \cdot \sqrt{\frac{E}{F_y}} \right), b \right] = 2.32 \cdot \text{in}$$

$$I_{\text{eff}} := I_x - 2 \cdot \left[(b - b_e) \cdot t \cdot (B_f - t)^2 + \frac{(b - b_e) \cdot t^3}{12} \right] = 2.03 \cdot \text{in}^4$$

$$S_e := \frac{I_{\text{eff}}}{\frac{H_w}{2}} = 1.36 \cdot \text{in}^3$$

$$M_{n_flb} := \begin{cases} \min \left[M_p - (M_p - F_y \cdot S_x) \cdot \left(3.57 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E}} - 4.0 \right), M_p \right] & \text{if Slenderness} = \text{"noncompact"} = 4.92 \cdot \text{kip} \cdot \text{ft} \\ F_y \cdot S_e & \text{if Slenderness} = \text{"slender"} \\ M_n & \text{if Slenderness} = \text{"compact"} \end{cases}$$

Web Local Buckling

$$M_{n_wlb} := \begin{cases} \min \left[M_p - (M_p - F_y \cdot S_x) \cdot \left(0.305 \cdot \frac{h}{t} \cdot \sqrt{\frac{F_y}{E}} - 0.738 \right), M_p \right] & \text{if Slenderness} = \text{"noncompact"} = 4.92 \cdot \text{kip} \cdot \text{ft} \\ M_n & \text{if Slenderness} = \text{"compact"} \end{cases}$$

$$M_n := \min(M_n, M_{n_flb}, M_{n_wlb}) = 4.92 \cdot \text{kip} \cdot \text{ft}$$

Roof Load: $TL_{max} = 53 \cdot \text{psf}$ *conservatively use TL_{max} - may be negative load

Wall Load: $P_{wallend} = 11.7 \cdot \text{psf}$

Worst Case Tributary Width: $TW := 2\text{ft} + 10\text{in}$

Distributed Load to Column: $w := P_{wallend} \cdot TW = 33 \cdot \text{plf}$

Beam Reaction to Column: $R_{max} = 0.41 \cdot \text{kip}$ *conservatively use TL_{max} - may be negative load

Maximum Column Span: $H = 8.73\text{ft}$

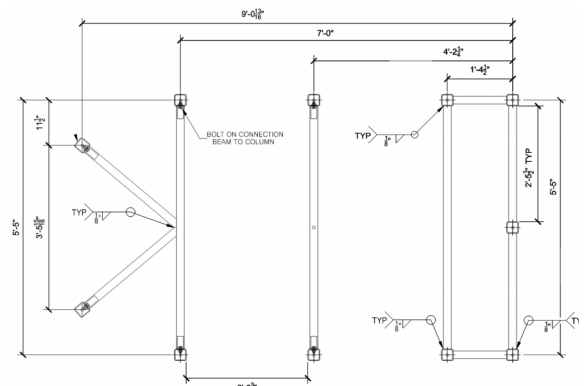
Check Stress

Axial Load: $P_{max} := R_{max} = 0.41 \cdot \text{kip}$

$$\frac{P_{max} \cdot \Omega_c}{P_n} = 0.02 < 1.00 \text{ OK}$$

Moment Due to Wind Load: $M_{dist} := \frac{w \cdot H^2}{8} = 0.32 \cdot \text{kip} \cdot \text{ft}$

$$\frac{M_{dist} \cdot \Omega_b}{M_n} = 0.11 < 1.00 \text{ OK}$$



COLUMN AND BEAM LAYOUT

Combined Stress:

$$\text{UNITY} := \begin{cases} \frac{P_{max} \cdot \Omega_c}{P_n} + \frac{8}{9} \cdot \frac{M_{dist} \cdot \Omega_b}{M_n} & \text{if } \frac{P_{max} \cdot \Omega_c}{P_n} \geq 0.2 \\ \frac{P_{max} \cdot \Omega_c}{2 \cdot P_n} + \frac{M_{dist} \cdot \Omega_b}{M_n} & \text{if } \frac{P_{max} \cdot \Omega_c}{P_n} < 0.2 \end{cases} = 0.12 < 1.00 \text{ OK}$$

HSS3x3x3/16" LSV tubes are adequate for posts

GRAVITY DESIGN: STEEL POST LIFTING CAP

Total Unit Weight: Weight = 8.36-kip

Load to Lifting Plate: $P_{\text{lift}} := \frac{\text{Weight}}{4} = 2.09\text{-kip}$ *conservatively analyze lift from (4) columns only

Check Plate

Plate Thickness: $t_{\text{plate}} := 0.5\text{in}$

Plate Width: $w_{\text{plate}} := 3\text{in} - 2 \cdot (0.174\text{in}) = 2.65\text{-in}$

Plate Yield Strength: $F_{y_{\text{plate}}} := 30\text{ksi}$

Plastic Modulus of Plate: $Z_{\text{plate}} := \frac{t_{\text{plate}}^2 \cdot w_{\text{plate}}}{4} = 0.17\text{-in}^3$

Safety Factor: $\Omega_b := 1.67$

Maximum Moment on Plate: $M_{\text{plate}} := \frac{P_{\text{lift}} \cdot w_{\text{plate}}}{4} = 1.39\text{-kip}\cdot\text{in}$

Allowable Moment on Plate: $M_{\text{allow}} := \frac{F_{y_{\text{plate}}} \cdot Z_{\text{plate}}}{\Omega_b} = 2.98\text{-kip}\cdot\text{in}$

$\frac{M_{\text{plate}}}{M_{\text{allow}}} = 0.47 < 1.00$ OK

Check Weld

Weld Thickness: $t_w := 0.1875\text{in}$

Filler Metal Strength: $F_{EXX} := 70\text{ksi}$

Safety Factor: $\Omega_w := 2.70$

Weld Width: $b_w := w_{\text{plate}} = 2.65\text{-in}$

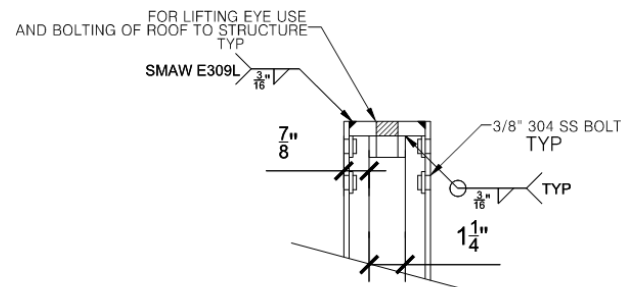
Weld Height: $d_w := w_{\text{plate}} = 2.65\text{-in}$

Total Weld Length: $A_w := 2 \cdot b_w + 2 \cdot d_w = 10.61\text{-in}$

Shear Load on Weld: $V_{\text{weld}} := \frac{P_{\text{lift}}}{A_w} = 0.2 \frac{\text{kip}}{\text{in}}$

Allowable Weld Strength: $V_{\text{allow}} := \frac{0.6 \cdot F_{EXX} \cdot 0.707 \cdot t_w}{\Omega_w} = 2.06 \frac{\text{kip}}{\text{in}}$

$\frac{V_{\text{weld}}}{V_{\text{allow}}} = 0.1 < 1.00$ OK



1
S1.0
COLUMN LIFTING CAP
1/2" 304 SS
TAP FOR 3/4-10 Bolt
(11) REQ'D

1/2" thick stainless steel lifting plate is adequate.
Use 3/16" fillet weld all-around for lifting cap connection to posts.

LATERAL DESIGN: STEEL WALL PANELS (OUT OF PLANE)

Material Properties - 304 Stainless Steel

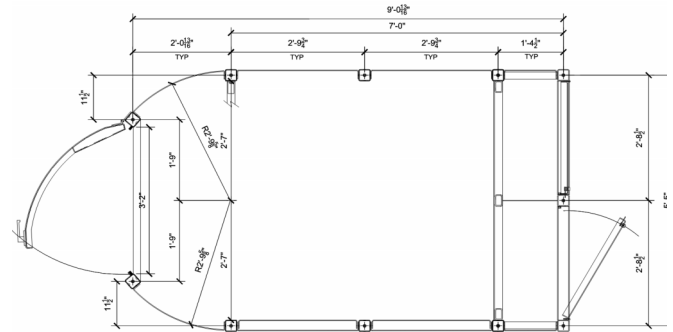
Yield Strength: $F_y := 30\text{ksi}$
Modulus of Elasticity: $E := 23800\text{ksi}$
Thickness of Plate: $t := 0.1875\text{in}$

Loading

Maximum Load on Plate: $p_{cwallend} = 15\text{psf}$

Geometry

Plate Width: $a := H = 8.73\text{-ft}$
Plate Span Length: $b := 36\text{in}$ *conservative



Tabulated Values:
(relations among load, stress and deflection are expressed by dimensionless coefficients shown below)

See Roark's Formulas for Stress & Strain, Sixth Edition - Plate support on 2 sides and pinned

$$\text{coef}_q = \frac{T_{L_{\max}} \cdot b^4}{E \cdot t^4} \quad \text{coef}_\sigma = \frac{\sigma \cdot b^2}{E \cdot t^2} \quad \text{coef}_y = \frac{y}{t}$$

$$\text{coef}_q := (0 \ 12.5 \ 25 \ 50 \ 75 \ 100 \ 125 \ 150 \ 175 \ 200 \ 250)^T$$

$$\text{coef}_{\sigma 1} := (0 \ 3.8 \ 5.8 \ 8.7 \ 10.9 \ 12.8 \ 14.3 \ 15.6 \ 17.0 \ 18.2 \ 20.5)^T$$

$$\text{coef}_{y1} := (0 \ 0.430 \ 0.650 \ 0.93 \ 1.13 \ 1.26 \ 1.37 \ 1.47 \ 1.56 \ 1.63 \ 1.77)^T$$

$$\text{coef}_{\sigma 15} := (0 \ 4.48 \ 6.81 \ 9.92 \ 12.25 \ 14.22 \ 16 \ 17.50 \ 18.9 \ 20.3 \ 22.8)^T$$

$$\text{coef}_{y15} := (0 \ 0.625 \ 0.879 \ 1.18 \ 1.37 \ 1.53 \ 1.68 \ 1.77 \ 1.88 \ 1.96 \ 2.12)^T$$

$$\text{coef}_{\sigma 2} := (0 \ 4.87 \ 7.16 \ 10.3 \ 12.6 \ 14.6 \ 16.4 \ 18 \ 19.4 \ 20.9 \ 23.6)^T$$

$$\text{coef}_{y2} := (0 \ 0.696 \ 0.946 \ 1.24 \ 1.44 \ 1.6 \ 1.72 \ 1.84 \ 1.94 \ 2.03 \ 2.2)^T$$

$$\text{coef}_\sigma := \begin{cases} \text{coef}_{\sigma 1} & \text{if } \frac{a}{b} \leq 1.25 \\ \text{coef}_{\sigma 15} & \text{if } 1.25 < \frac{a}{b} \leq 1.75 \\ \text{coef}_{\sigma 2} & \text{otherwise} \end{cases} \quad \text{coef}_y := \begin{cases} \text{coef}_{y1} & \text{if } \frac{a}{b} \leq 1.25 \\ \text{coef}_{y15} & \text{if } 1.25 < \frac{a}{b} \leq 1.75 \\ \text{coef}_{y2} & \text{otherwise} \end{cases}$$

Check Stress

$$\sigma := \text{linterp} \left[(\text{coef}_q), \text{coef}_\sigma, \frac{P_{\text{cwallend}} \cdot b^4}{E \cdot t^4} \right] \cdot \left(\frac{E \cdot t^2}{b^2} \right) = 1.5 \cdot \text{ksi}$$

$$\sigma := \text{if} \left(\frac{P_{\text{cwallend}} \cdot b^4}{E \cdot t^4} < 250, "Ok", "OUT OF RANGE" \right) = "Ok" \cdot \text{ksi}$$

$$\frac{1.67 \cdot (2.29 \text{ ksi})}{F_y} = 0.13 < 1.00 \text{ OK}$$

Check Deflection

$$y_{\text{allow}} := \frac{b}{200} = 0.18 \cdot \text{in}$$

$$y := \text{linterp} \left[(\text{coef}_q), \text{coef}_y, \frac{0.7 \cdot P_{\text{cwallend}} \cdot (b)^4}{E \cdot t^4} \right] \cdot t = 0.04 \cdot \text{in}$$

$$\frac{y}{y_{\text{allow}}} = 0.24 < 1.00 \text{ OK}$$

3/16" stainless steel plate is adequate for wall panels for out of plane loading

Check Connection

Maximum Wall Panel Reaction: $R_{\text{panel}} := P_{\text{cwallend}} \cdot b = 45 \cdot \text{plf}$

Fastener Spacing: $sp := 24 \cdot \text{in}$

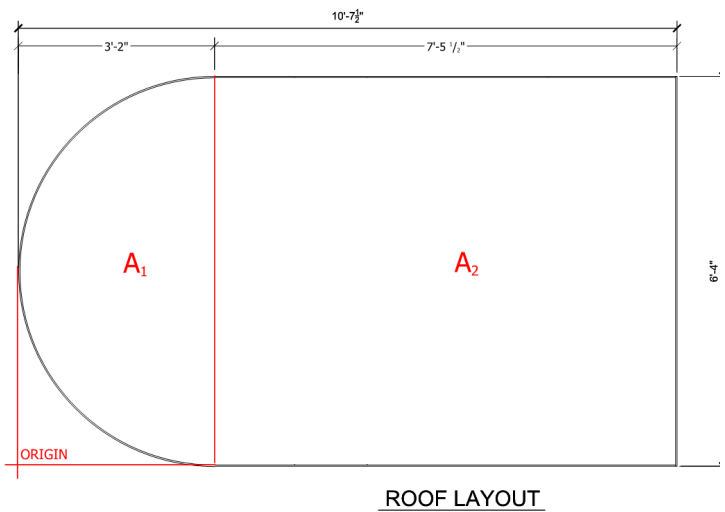
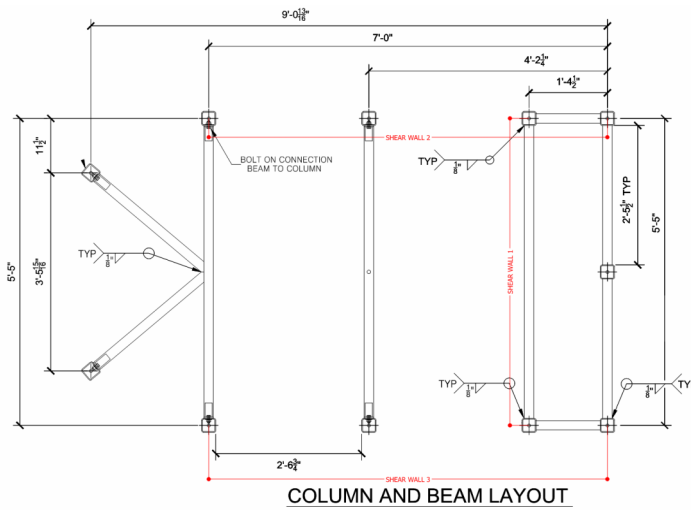
Maximum Screw Shear: $V_{\text{max}} := R_{\text{panel}} \cdot sp = 90 \cdot \text{lbf}$

Fastener Allowable: $V_{\text{allow}} := 912 \cdot \text{lbf}$

$$\frac{V_{\text{max}}}{V_{\text{allow}}} = 0.1 < 1.00 \text{ OK}$$

3/8" diameter fasteners at 24" o.c. maximum are adequate for wall panel connection to posts.
Fasteners to be 316 stainless steel, condition "A" ($F_u = 75 \text{ ksi}$ minimum).

LATERAL DESIGN: LOAD DISTRIBUTION



Geometry

Width: $W = 6.33\text{ft}$

Length: $L = 10.63\text{ft}$

Rectangular Area Length: $L_r := L - \frac{W}{2} = 7.46\text{ft}$

Center of Mass

Area of Front Radius: $A_1 := \frac{\pi \cdot \left(\frac{W}{2}\right)^2}{2} = 15.8\text{ft}^2$

Area of Rectangle: $A_2 := L_r \cdot W = 47.2\text{ft}^2$

Total Area: $A_{\text{tot}} := A_1 + A_2 = 63\text{ft}^2$

Centroid of Front Radius: $\text{Cent}_{A1} := \frac{W}{2} \cdot \left(1 - \frac{4}{3 \cdot \pi}\right) = 1.82\text{ft}$

Centroid of Rectangle: $\text{Cent}_{A2} := \frac{W}{2} + \frac{L_r}{2} = 6.9\text{ft}$

Overall Centroid: $\text{Cent}_x := \frac{A_1 \cdot \text{Cent}_{A1} + A_2 \cdot \text{Cent}_{A2}}{A_{\text{tot}}} = 5.63\text{ft}$

$\text{Cent}_y := \frac{W}{2} = 3.17\text{ft}$

Location of Shear Wall 1: $\text{SW}_{1x} := L - 22\text{in} = 8.79\text{ft}$

Direct Lateral Loads

Parallel to Long Walls: $V_{\text{long}} = 0.32\text{-kip}$

Parallel to Short Walls: $V_{\text{short}} = 0.52\text{-kip}$

Load Distribution

Shear Wall 1: $V_{\text{SW1}} := V_{\text{short}} = 0.52\text{-kip}$

Shear Walls 2 & 3: $V_{\text{SW2}} := \max\left[\frac{V_{\text{long}}}{2}, \frac{V_{\text{short}} \cdot (\text{SW}_{1x} - \text{Cent}_x)}{W}\right] = 0.26\text{-kip}$

LATERAL DESIGN: STEEL POST DESIGN

Geometry

Height of Shear Wall: $H = 8.73\text{ft}$
 Height of Steel Sheet Portion: $H_{\text{steel}} := 6\text{ft}$
 Height of Cantilever: $H_{\text{cant}} := H - H_{\text{steel}} = 2.73\text{ft}$
 Width of Shear Wall 1: $W_{\text{SW1}} := 5.5\text{ft}$
 Width of Shear Walls 2 & 3: $W_{\text{SW2}} := 7\text{ft}$

Check Posts

HSS3x3x3/16 Section Properties

$I_x := 2.46\text{in}^4$ $S_x := 1.64\text{in}^3$ $r_x := 1.14\text{in}$ $Z_x := 1.97\text{in}^3$ $F_y = 30\text{ksi}$ $\Omega_c := 1.67$
 $I_y := 2.46\text{in}^4$ $S_y := 1.64\text{in}^3$ $r_y := 1.14\text{in}$ $Z_y := 1.97\text{in}^3$ $A := 1.89\text{in}^2$ $\Omega_b := 1.67$
 $t := 0.174\text{in}$ $b := 14.2 \cdot t = 2.47\text{in}$ $h := 14.2 \cdot t = 2.47\text{in}$ $B_f := 3\text{in}$ $H_w := 3\text{in}$
 $K := 1.0$ $H = 8.73\text{ft}$

Determine Nominal Axial Capacity

$$\text{Slenderness} := \begin{cases} \text{"nonslender"} & \text{if } \frac{b}{t} < 1.40 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"slender"} & \text{if } \frac{b}{t} \geq 1.40 \cdot \sqrt{\frac{E}{F_y}} \end{cases} = \text{"nonslender"}$$

Flexural Buckling

$$F_e := \frac{\pi^2 \cdot E}{\left(\frac{K \cdot L}{r_x}\right)^2} = 18.78 \cdot \text{ksi}$$

$$F_{cr} := \begin{cases} 0.658 \cdot \frac{F_y}{F_e} \cdot F_y & \text{if } \frac{K \cdot H}{r_x} \leq 4.71 \cdot \sqrt{\frac{E}{F_y}} \\ 0.877 \cdot F_e & \text{if } \frac{K \cdot H}{r_x} > 4.71 \cdot \sqrt{\frac{E}{F_y}} \end{cases} = 15.37 \cdot \text{ksi}$$

$$P_n := \begin{cases} F_{cr} \cdot A & \text{if Slenderness} = \text{"nonslender"} \\ \text{"further analysis required"} & \text{if Slenderness} = \text{"slender"} \end{cases} = 29.05 \cdot \text{kip}$$

Determine Nominal Moment Capacity

$$\text{Slenderness} := \begin{cases} \text{"compact"} & \text{if } \frac{b}{t} < 1.12 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"slender"} & \text{if } \frac{b}{t} > 1.40 \cdot \sqrt{\frac{E}{F_y}} \\ \text{"noncompact"} & \text{if } 1.12 \cdot \sqrt{\frac{E}{F_y}} \leq \frac{b}{t} \leq 1.40 \cdot \sqrt{\frac{E}{F_y}} \end{cases} = \text{"compact"}$$

Yielding

$$M_p := F_y \cdot Z_x = 4.92 \cdot \text{kip} \cdot \text{ft}$$

$$M_n := M_p = 4.92 \cdot \text{kip} \cdot \text{ft}$$

Flange Local Buckling

$$b_e := \min \left[1.92 \cdot t \cdot \sqrt{\frac{E}{F_y}} \cdot \left(1 - \frac{0.38}{\frac{b}{t}} \cdot \sqrt{\frac{E}{F_y}} \right), b \right] = 2.32 \cdot \text{in}$$

$$I_{eff} := I_x - 2 \cdot \left[(b - b_e) \cdot t \cdot (B_f - t)^2 + \frac{(b - b_e) \cdot t^3}{12} \right] = 2.03 \cdot \text{in}^4$$

$$S_e := \frac{I_{eff}}{\frac{H_w}{2}} = 1.36 \cdot \text{in}^3$$

$$M_{n_flb} := \begin{cases} \min \left[M_p - (M_p - F_y \cdot S_x) \cdot \left(3.57 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E}} - 4.0 \right), M_p \right] & \text{if Slenderness = "noncompact"} = 4.92 \cdot \text{kip} \cdot \text{ft} \\ F_y \cdot S_e & \text{if Slenderness = "slender"} \\ M_n & \text{if Slenderness = "compact"} \end{cases}$$

Web Local Buckling

$$M_{n_wlb} := \begin{cases} \min \left[M_p - (M_p - F_y \cdot S_x) \cdot \left(0.305 \cdot \frac{h}{t} \cdot \sqrt{\frac{F_y}{E}} - 0.738 \right), M_p \right] & \text{if Slenderness = "noncompact"} = 4.92 \cdot \text{kip} \cdot \text{ft} \\ M_n & \text{if Slenderness = "compact"} \end{cases}$$

$$M_n := \min(M_n, M_{n_flb}, M_{n_wlb}) = 4.92 \cdot \text{kip} \cdot \text{ft}$$

Check Shear Wall 1

Roof Load: $TL_{max} = 53 \cdot \text{psf}$ *conservatively use TL_{max} - may be negative load
 Beam Reaction to Column: $R_{max} = 0.41 \cdot \text{kip}$ *conservatively use R_{max} - may be negative load
 Shear Load to Shear Wall 1: $V_{SW1} = 0.52 \cdot \text{kip}$
 Width of Shear Wall 1: $W_{SW1} = 5.5 \text{ ft}$
 Reaction due to Shear Couple: $R_{shear} := \frac{V_{SW1} \cdot H}{W_{SW1}} = 0.83 \cdot \text{kip}$

Check Stress

Axial Load: $P_{max} := R_{max} + R_{shear} = 1.23 \cdot \text{kip}$

$$\frac{P_{max} \cdot \Omega_c}{P_n} = 0.07 < 1.00 \text{ OK}$$

Moment Due to Cantilever: $M_{cant} := \frac{V_{SW1} \cdot H_{cant}}{2} = 0.71 \cdot \text{kip} \cdot \text{ft}$

$$\frac{M_{cant} \cdot \Omega_b}{M_n} = 0.24 < 1.00 \text{ OK}$$

Combined Stress:

$$\text{UNITY} := \begin{cases} \frac{P_{max} \cdot \Omega_c}{P_n} + \frac{8}{9} \cdot \frac{M_{cant} \cdot \Omega_b}{M_n} & \text{if } \frac{P_{max} \cdot \Omega_c}{P_n} \geq 0.2 \\ \frac{P_{max} \cdot \Omega_c}{2 \cdot P_n} + \frac{M_{cant} \cdot \Omega_b}{M_n} & \text{if } \frac{P_{max} \cdot \Omega_c}{P_n} < 0.2 \end{cases} = 0.28 < 1.00 \text{ OK}$$

HSS3x3x3/16" tubes are adequate for Lateral force resisting system posts

Check Shear Walls 2 & 3

Roof Load: $TL_{max} = 53 \cdot \text{psf}$ *conservatively use TL_{max} - may be negative load
 Beam Reaction to Column: $R_{max} = 0.41 \cdot \text{kip}$ *conservatively use R_{max} - may be negative load
 Shear Load to Shear Wall 2: $V_{SW2} = 0.26 \cdot \text{kip}$
 Width of Shear Wall 2: $W_{SW2} = 7 \text{ ft}$
 Reaction due to Shear Couple: $R_{shear} := \frac{V_{SW2} \cdot H}{W_{SW2}} = 0.32 \cdot \text{kip}$

Check Stress

Axial Load: $P_{max} := R_{max} + R_{shear} = 0.73 \cdot \text{kip}$

$$\frac{P_{max} \cdot \Omega_c}{P_n} = 0.04 < 1.00 \text{ OK}$$

Moment Due to Cantilever: $M_{cant} := \frac{V_{SW1} \cdot H_{cant}}{2} = 0.71 \cdot \text{kip} \cdot \text{ft}$

$$\frac{M_{cant} \cdot \Omega_b}{M_n} = 0.24 < 1.00 \text{ OK}$$

Combined Stress:

$$\text{UNITY} := \begin{cases} \frac{P_{max} \cdot \Omega_c}{P_n} + \frac{8}{9} \cdot \frac{M_{cant} \cdot \Omega_b}{M_n} & \text{if } \frac{P_{max} \cdot \Omega_c}{P_n} \geq 0.2 \\ \frac{P_{max} \cdot \Omega_c}{2 \cdot P_n} + \frac{M_{cant} \cdot \Omega_b}{M_n} & \text{if } \frac{P_{max} \cdot \Omega_c}{P_n} < 0.2 \end{cases} = 0.26 < 1.00 \text{ OK}$$

HSS3x3x3/16" tubes are adequate for Lateral force resisting system posts

LATERAL DESIGN: STEEL POST ANCHORAGE

Seismic Overstrength Factor: $\Omega_0 := 2.5$

*total base shear over 2 posts (conservative)

Maximum Post Shear: $V_{\text{post}} := \max\left(\frac{V_W}{2}, \frac{\Omega_0 \cdot V_E}{2}\right) = 0.5 \cdot \text{kip}$

Maximum Uplift Force: $P_{\text{uplift}} := \max\left[\frac{\Omega_0 \cdot V_{\text{roof}} \cdot H}{W_{\text{SW1}}}, \frac{\Omega_0 \cdot V_{\text{roof}} \cdot H}{W_{\text{SW2}}}, \frac{(V_{\text{WS}}) \cdot H}{W_{\text{SW1}}} + \frac{\text{Proofend} \cdot L \cdot W}{11}, \frac{(V_{\text{WL}}) \cdot H}{W_{\text{SW2}}} + \frac{\text{Proofend} \cdot L \cdot W}{11}\right] = 0.75 \cdot \text{kip}$

Resisting Force: $P_{\text{resist}} := \frac{\text{Weight}}{11} = 0.76 \cdot \text{kip}$ *total unit weight over 11 posts

Resultant Load: $P_{\text{net}} := |0.6 \cdot P_{\text{resist}} - P_{\text{uplift}}| = 0.3 \cdot \text{kip}$

Check Plate

Plate Thickness: $t_{\text{plate}} := 0.5 \text{ in}$

Plate Width: $w_{\text{plate}} := 3 \text{ in}$

Plate Yield Strength: $F_{y_{\text{plate}}} := 30 \text{ ksi}$

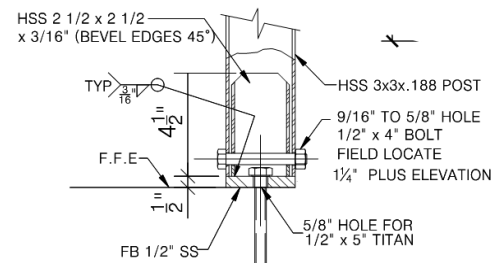
Plastic Modulus of Plate: $Z_{\text{plate}} := \frac{t_{\text{plate}}^2 \cdot w_{\text{plate}}}{4} = 0.19 \cdot \text{in}^3$

Safety Factor: $\Omega_b := 1.67$

Maximum Moment on Plate: $M_{\text{plate}} := \frac{P_{\text{net}} \cdot w_{\text{plate}}}{4} = 0.22 \cdot \text{kip} \cdot \text{in}$

Allowable Moment on Plate: $M_{\text{allow}} := \frac{F_{y_{\text{plate}}} \cdot Z_{\text{plate}}}{\Omega_b} = 3.37 \cdot \text{kip} \cdot \text{in}$

$\frac{M_{\text{plate}}}{M_{\text{allow}}} = 0.07 < 1.00 \text{ OK}$



SECTION A-A
Typ anchoring system
(11) REQ'D

Check Weld

Weld Thickness: $t_w := 0.1875 \text{ in}$

Filler Metal Strength: $F_{\text{EXX}} := 70 \text{ ksi}$

Safety Factor: $\Omega_w := 2.70$

Weld Width: $b_w := w_{\text{plate}} = 3 \cdot \text{in}$

Weld Height: $d_w := w_{\text{plate}} = 3 \cdot \text{in}$

Total Weld Length: $A_w := 2 \cdot b_w + 2 \cdot d_w = 12 \cdot \text{in}$

Vertical Load on Weld: $V_{\text{vert}} := \frac{P_{\text{lift}}}{A_w} = 0.17 \cdot \frac{\text{kip}}{\text{in}}$

Horizontal Load on Weld: $V_{\text{hor}} := \frac{V_{\text{post}}}{A_w} = 0.05 \cdot \frac{\text{kip}}{\text{in}}$

Resultant Load on Weld: $V_{\text{weld}} := \sqrt{V_{\text{vert}}^2 + V_{\text{hor}}^2} = 0.18 \cdot \frac{\text{kip}}{\text{in}}$

Allowable Weld Strength: $V_{\text{allow}} := \frac{0.6 \cdot F_{\text{EXX}} \cdot 0.707 \cdot t_w}{\Omega_w} = 2.06 \cdot \frac{\text{kip}}{\text{in}}$

$\frac{V_{\text{weld}}}{V_{\text{allow}}} = 0.09 < 1.00 \text{ OK}$

Check Bolt

Allowable Shear Stress (316SS): $F_v := 12.99 \text{ ksi}$

Nominal Bolt Diameter: $d := 0.5 \text{ in}$

Thread Root Area: $A_R := 0.1292 \text{ in}^2$

Tensile Strength of HSS: $F_{uHSS} := 58 \text{ ksi}$
 $F_u := F_{uHSS} = 58 \text{ ksi}$

HSS Wall Thickness: $t = 0.174 \text{ in}$

Maximum Bolt Shear: $V_{max} := \frac{P_{net}}{2} = 148 \text{ lbf}$ *two shear planes

Clear Distance to Plate Edge: $l_c := 0.75 \text{ in} - \frac{(d + 0.0625 \text{ in})}{2} = 0.47 \text{ in}$

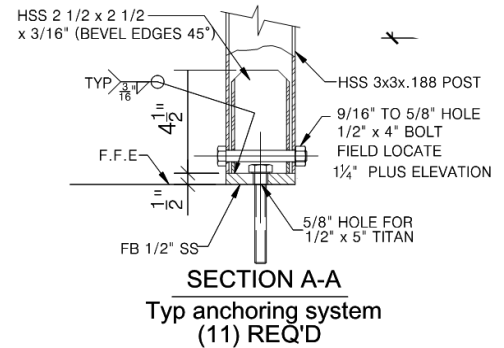
Safety Factor: $\Omega_{brg} := 2.0$

Bearing Strength: $V_{brg} := \min\left(\frac{1.2 \cdot l_c \cdot t \cdot F_u}{\Omega_{brg}}, \frac{2.4 \cdot d \cdot t \cdot F_u}{\Omega_{brg}}\right) = 2838 \text{ lbf}$

Bolt Strength: $V_{bolt} := F_v \cdot A_R = 1678.31 \text{ lbf}$

$V_{allow} := \min(V_{brg}, V_{bolt}) = 1678.31 \text{ lbf}$

$\frac{V_{max}}{V_{allow}} = 0.09 < 1.00 \text{ OK}$



Check Anchorage to Foundation

Base Shear per Post: $V_{post} = 0.54 \text{ kip}$

$V_{anchor} := 1.428 \cdot V_{post} = 0.78 \text{ kip}$ *convert to LRFD load

Net Uplift per Post: $P_{net} = 0.3 \text{ kip}$

$T_{anchor} := 1.6 \cdot P_{net} = 0.47 \text{ kip}$ *convert to LRFD load

1/2" thick stainless steel plate with HSS 2 1/2 x 2 1/2 x 3/16 is adequate.
Use 3/16" fillet weld all-around for HSS connection to plate.
Use 1/2" diameter bolt for post connection to hold downs.
Bolts to be 316 stainless steel, condition "A" ($F_u = 75 \text{ ksi}$ minimum).
Use 1/2" Titen HD (4" embedment) to slab. See Simpson Anchor Designer output for analysis.

LATERAL DESIGN: STEEL PLATE SHEAR WALL

Geometry

Height of Shear Wall: $H = 8.73\text{ft}$
Height of Steel Sheet Portion: $H_{\text{steel}} = 6\text{ft}$
Width of Shear Wall 1: $W_{\text{SW1}} = 5.5\text{ft}$
Width of Shear Walls 2 & 3: $W_{\text{SW2}} = 7\text{ft}$

Load Distribution

Shear Wall 1: $V_{\text{SW1}} = 0.52\text{-kip}$
Shear Walls 2 & 3: $V_{\text{SW2}} = 0.26\text{-kip}$
Angles of Inclination of Load(s):
 $\theta_{\text{SW1}} := \text{atan}\left(\frac{H}{W_{\text{SW1}}}\right) = 57.8\text{-deg}$
 $\theta_{\text{SW2}} := \text{atan}\left(\frac{H_{\text{steel}}}{1.33\text{ft}}\right) = 77.5\text{-deg}$ *narrowest panel, SW2/3
 $\theta_{\text{SW3}} := \text{atan}\left(\frac{H_{\text{steel}}}{2.92\text{ft}}\right) = 64.05\text{-deg}$ *widest panel, SW2/3
Resultant Tension in Tension Field:
$$T_{\text{strap}} := \max\left[\frac{V_{\text{SW1}}}{\cos(\theta_{\text{SW1}})}, \frac{V_{\text{SW2}} \cdot \left(\frac{1.33\text{ft}}{W_{\text{SW2}}}\right)}{\cos(\theta_{\text{SW2}})}, \frac{V_{\text{SW2}} \cdot \left(\frac{2.92\text{ft}}{W_{\text{SW2}}}\right)}{\cos(\theta_{\text{SW3}})}\right] = 0.98\text{-kip}$$

Plate Thickness: $t_{\text{pl}} := 0.1875\text{in}$
Plate Yield Stress: $F_y = 30\text{-ksi}$
Minimum Tension Field Width:
$$w_{\text{strap}} := \frac{T_{\text{strap}}}{\left(\frac{t_{\text{pl}} \cdot F_y}{1.67}\right)} = 0.29\text{-in}$$

Section Modulus of Plate at Bend:
$$S_{\text{plate}} := \frac{t_{\text{pl}}^2 \cdot 11\text{in}}{6} = 0.06\text{-in}^3$$

Allowable Moment at Plate Connection to Post:
$$M_{\text{all_plate}} := \frac{F_y \cdot S_{\text{plate}}}{1.67} = 1.16\text{-kip-in}$$

Weak Axis Moment at Plate Connection to Post:
$$M_{\text{pl_w}} := \max(V_{\text{SW1}}, V_{\text{SW2}}) \cdot 1\text{in} = 0.52\text{-kip-in}$$

$$\frac{M_{\text{pl_w}}}{M_{\text{all_plate}}} = 0.45 < 1.00 \text{ OK}$$

3/16" stainless steel plate is adequate for wall panels for in plane loading

LATERAL DESIGN: STEEL PLATE SHEAR WALL (continued)

Shear Wall 1 Horizontal Load: $V_{SW1} = 0.52 \cdot \text{kip}$

Shear Wall 1 Vertical Shear at Plate to Post: $V_{SW1V} := \frac{V_{SW1} \cdot H_{\text{steel}}}{W_{SW1} - 2 \cdot (3 \text{ in})} = 0.62 \cdot \text{kip}$

Shear Wall 2 & 3 Horizontal Load: $V_{SW2} = 0.26 \cdot \text{kip}$

Shear Wall 2 & 3 Vertical Shear at Plate to Post: $V_{SW2V} := \frac{V_{SW2} \cdot H_{\text{steel}}}{W_{SW2} - 2 \cdot (3 \text{ in})} = 0.24 \cdot \text{kip}$

Check Fasteners to Post ((3) 3/8" Dia. 316 SS Screws)

Allowable Shear Stress (316SS): $F_v := 12.99 \text{ ksi}$

Allowable Tensile Stress (316SS): $F_T := 22.5 \text{ ksi}$

Nominal Bolt Diameter: $d := 0.375 \text{ in}$

Thread Root Area: $A_R := 0.0699 \text{ in}^2$

Tensile Stress Area: $A_S := 0.0775 \text{ in}^2$

Threads Per Inch: $n := 16$

External Thread Stripping Area: $A_{TSE} := 0.0360 \text{ in}^2$

Tensile Strength of HSS: $F_{uHSS} := 58 \text{ ksi}$

Tensile Strength of Plate: $F_{uPL} := 58 \text{ ksi}$

$F_u := \min(F_{uHSS}, F_{uPL}) = 58 \cdot \text{ksi}$

HSS Wall Thickness: $t = 0.174 \cdot \text{in}$

$t := \min(t, t_{pl}) = 0.174 \cdot \text{in}$

Maximum Fastener Shear: $V_{\max} := \frac{\max(V_{SW1V}, V_{SW2V})}{3} = 208 \cdot \text{lbf}$ *conservatively assume 3 fasteners

Clear Distance to Plate Edge: $l_c := 0.75 \text{ in} - \frac{(d + 0.0625 \text{ in})}{2} = 0.53 \cdot \text{in}$ *conservative

Safety Factor: $\Omega_{brg} := 2.0$

Bearing Strength: $V_{brg} := \min\left(\frac{1.2 \cdot l_c \cdot t \cdot F_u}{\Omega_{brg}}, \frac{2.4 \cdot d \cdot t \cdot F_u}{\Omega_{brg}}\right) = 3217 \cdot \text{lbf}$ (Eq. J3-6A & J3-6C AISC)

Fastener Strength: $V_{\text{fast}} := F_v \cdot A_R = 908 \cdot \text{lbf}$

$V_{\text{allow}} := \min(V_{brg}, V_{\text{fast}}) = 908 \cdot \text{lbf}$

Maximum Fastener Tension: $T_{\max} := \frac{\max(V_{SW1}, V_{SW2}) \cdot (1 \text{ in} + 1 \text{ in})}{2.25 \text{ in}} = 462.11 \cdot \text{lbf}$

Pullout Strength: $T_{po} := t \cdot \left(\frac{n}{\text{in}}\right) \cdot A_{TSE} \cdot \frac{(0.75 \cdot F_y)}{\sqrt{3}} = 1301.95 \cdot \text{lbf}$

Fastener Strength: $T_{\text{fast}} := F_T \cdot A_S = 1743.75 \cdot \text{lbf}$

$T_{\text{allow}} := \min(T_{po}, T_{\text{fast}}) = 1301.95 \cdot \text{lbf}$

$\left(\frac{T_{\max}}{T_{\text{allow}}}\right)^2 + \left(\frac{V_{\max}}{V_{\text{allow}}}\right)^2 = 0.18 < 1.00 \text{ OK}$

((3) 3/8" diameter fasteners are adequate for shear wall connection to posts. Fasteners to be 316 stainless steel, condition "A" (F_y = 75ksi minimum).

FOUNDATION DESIGN:

Geometry

Width of Foundation:	$W_{\text{found}} := 6\text{ft} + 8\text{in}$
Length of Foundation:	$L_{\text{found}} := 11\text{ft}$
Depth of Foundation:	$d_{\text{found}} := 18\text{in}$
Density of Concrete:	$\gamma_{\text{conc}} := 150\text{pcf}$
Total Area of Foundation:	$A_{\text{found}} := W_{\text{found}} \cdot L_{\text{found}} = 73.33\text{-ft}^2$
Volume of Foundation:	$V_{\text{found}} := A_{\text{found}} \cdot d_{\text{found}} = 110\text{-ft}^3$
Total Weight of Foundation:	$W_{\text{found}} := V_{\text{found}} \cdot \gamma_{\text{conc}} = 16.5\text{-kip}$
Allowable Soil Bearing:	$Q_{\text{allow}} = 1500\text{-psf}$

Check Bearing

Bearing Pressure on Soil: $w_{\text{brg}} := \frac{TL_{\text{pos}} \cdot L \cdot W + \text{Weight} + W_{\text{found}}}{A_{\text{found}}} = 387.65\text{-psf}$

$\frac{w_{\text{brg}}}{Q_{\text{allow}}} = 0.26 < 1.00 \text{ OK}$

*conservatively combine load on roof plate with structure weight and foundation weight

Check Uplift

Maximum Net Uplift: $w_{\text{uplift}} := 0.6 \cdot \text{Weight} - TL_{\text{neg}} \cdot L \cdot W = 3268.51\text{-lbf}$

Check Overturning

Maximum Shear Wall Net Uplift: $P_{\text{net}} = 0.3\text{-kip}$

$M_{\text{over}} := P_{\text{net}} \cdot \max(L_{\text{found}}, W_{\text{found}}) = 3.25\text{-kip}\cdot\text{ft}$ *apply uplift at edge

$M_{\text{resist}} := .6W_{\text{found}} \cdot \frac{W_{\text{found}}}{2} = 33\text{-kip}\cdot\text{ft}$ *apply resisting load at 1/2 distance of overturning load

$\frac{M_{\text{over}}}{M_{\text{resist}}} = 0.1 < 1.00 \text{ OK}$

1'-6" concrete foundation is adequate to support the structure.

Per IBC 2015 Sec. 1809.5 no frost protection required.



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1. Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General

Design method: ACI 318-14
Units: Imperial units

Anchor Information:

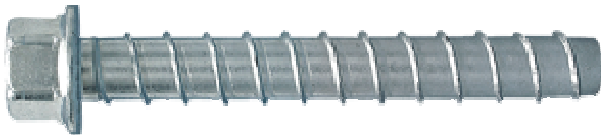
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.500
Nominal Embedment depth (inch): 4.000
Effective Embedment depth, h_{ef} (inch): 2.990
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
 h_{min} (inch): 6.25
 c_{ac} (inch): 4.50
 C_{min} (inch): 1.75
 S_{min} (inch): 3.00

Base Material

Concrete: Normal-weight
Concrete thickness, h (inch): 30.00
State: Cracked
Compressive strength, f'_c (psi): 4000
 $\Psi_{c,v}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Recommended Anchor

Anchor Name: Titen HD® - 1/2"Ø Titen HD, h_{nom} : 4" (102mm)
Code Report: ICC-ES ESR-2713





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Load and Geometry

Load factor source: ACI 318 Section 5.3

Load combination: not set

Seismic design: Yes

Anchors subjected to sustained tension: Not applicable

Ductility section for tension: 17.2.3.4.3 (d) is satisfied

Ductility section for shear: 17.2.3.5.3 (c) is satisfied

Ω_0 factor: not set

Apply entire shear load at front row: No

Anchors only resisting wind and/or seismic loads: No

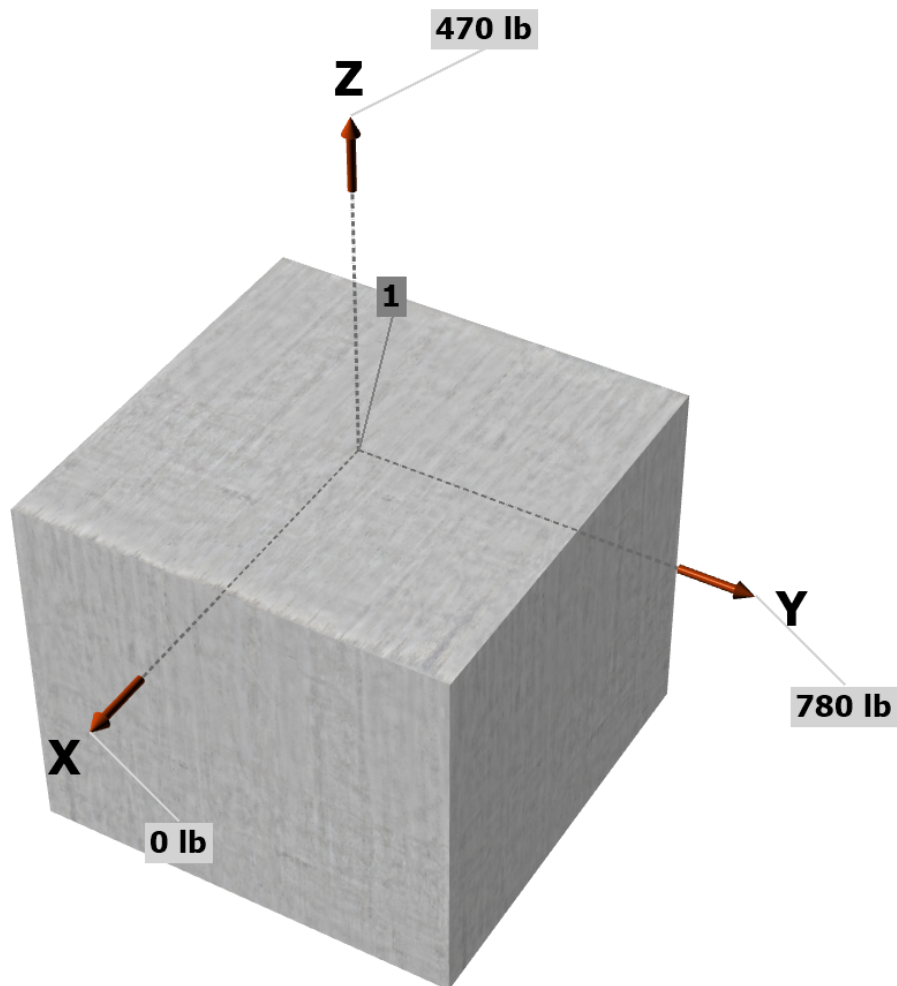
Strength level loads:

N_{ua} [lb]: 470

V_{uax} [lb]: 0

V_{uay} [lb]: 780

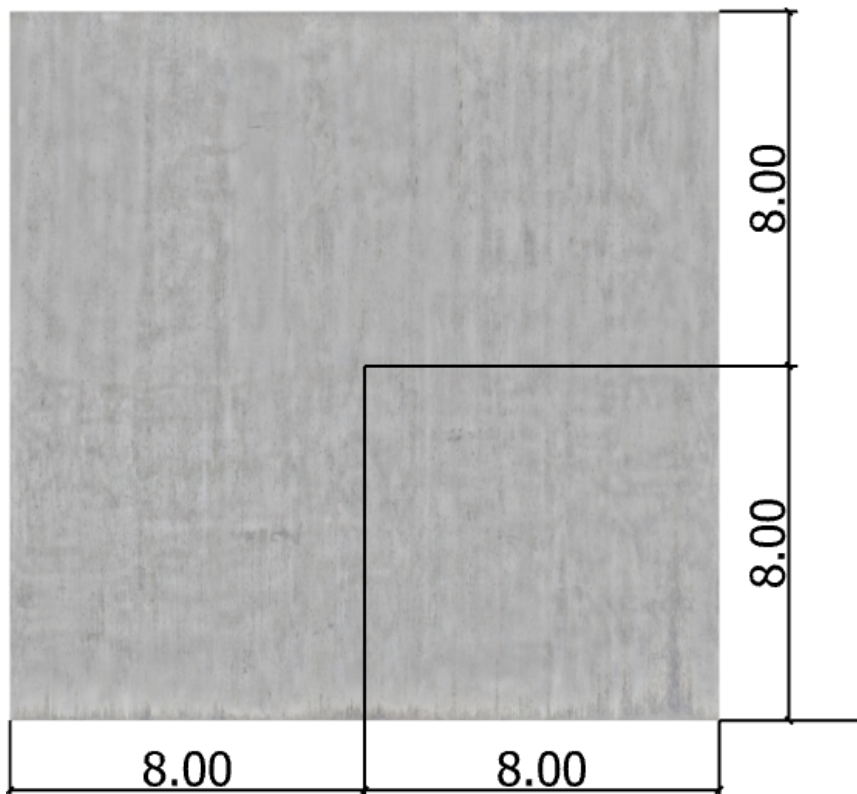
<Figure 1>





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<Figure 2>





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3. Resulting Anchor Forces

Anchor	Tension load, N _{ua} (lb)	Shear load x, V _{uax} (lb)	Shear load y, V _{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2 + (V_{uay})^2}$ (lb)
1	470.0	0.0	780.0	780.0
Sum	470.0	0.0	780.0	780.0

Maximum concrete compression strain (%): 0.00

Maximum concrete compression stress (psi): 0

Resultant tension force (lb): 470

Resultant compression force (lb): 0

Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00

Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00

Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N _{sa} (lb)	ϕ	ϕN_{sa} (lb)
20130	0.65	13085

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

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

k _c	λ_a	f' _c (psi)	h _{ef} (in)	N _b (lb)
17.0	1.00	4000	2.990	5559

$$0.75 \phi N_{cb} = 0.75 \phi (A_{Nc} / A_{Nco}) \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \text{ (Sec. 17.3.1 \& Eq. 17.4.2.1a)}$$

A _{Nc} (in ²)	A _{Nco} (in ²)	c _{a,min} (in)	$\psi_{ed,N}$	$\psi_{c,N}$	$\psi_{cp,N}$	N _b (lb)	ϕ	0.75 ϕN_{cb} (lb)
80.46	80.46	8.00	1.000	1.00	1.000	5559	0.65	2710

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V _{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout} \phi V_{sa}$ (lb)
4790	1.0	0.60	2874

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$$V_{by} = \min[7(l_e / d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f'_c} c_{a1}^{1.5}; 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}] \text{ (Eq. 17.5.2.2a \& Eq. 17.5.2.2b)}$$

l _e (in)	d _a (in)	λ_a	f' _c (psi)	c _{a1} (in)	V _{by} (lb)
2.99	0.500	1.00	4000	8.00	10130

$$\phi V_{cb,y} = \phi (A_{Vc} / A_{Vco}) \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_{by} \text{ (Sec. 17.3.1 \& Eq. 17.5.2.1a)}$$

A _{Vc} (in ²)	A _{Vco} (in ²)	$\psi_{ed,V}$	$\psi_{c,V}$	$\psi_{h,V}$	V _{by} (lb)	ϕ	$\phi V_{cb,y}$ (lb)
192.00	288.00	0.900	1.000	1.000	10130	0.70	4254

Shear parallel to edge in x-direction:

$$V_{by} = \min[7(l_e / d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f'_c} c_{a1}^{1.5}; 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}] \text{ (Eq. 17.5.2.2a \& Eq. 17.5.2.2b)}$$

l _e (in)	d _a (in)	λ_a	f' _c (psi)	c _{a1} (in)	V _{by} (lb)
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Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.strongtie.com



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2.99	0.500	1.00	4000	8.00	10130		
$\phi V_{cbx} = \phi (2)(A_{Vc} / A_{Vco}) \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)							
A_{Vc} (in ²)	A_{Vco} (in ²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
192.00	288.00	1.000	1.000	1.000	10130	0.70	9454

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$$\phi V_{cp} = \phi k_{cp} N_{cb} = \phi k_{cp} (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \quad (\text{Sec. 17.3.1 \& Eq. 17.5.3.1a})$$

k_{cp}	A_{Nc} (in ²)	A_{Nco} (in ²)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	80.46	80.46	1.000	1.000	1.000	5559	0.70	7782

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status	
Steel	470	13085	0.04	Pass	
Concrete breakout	470	2710	0.17	Pass (Governs)	
Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status	
Steel	780	2874	0.27	Pass (Governs)	
T Concrete breakout y+	780	4254	0.18	Pass	
Concrete breakout x-	780	9454	0.08	Pass	
Pryout	780	7782	0.10	Pass	
Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..2	0.00	0.27	27.1%	1.0	Pass

1/2"Ø Titen HD, hnom:4" (102mm) meets the selected design criteria.

12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.
- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.
- Designer must exercise own judgement to determine if this design is suitable.
- Refer to manufacturer's product literature for hole cleaning and installation instructions.