1.1 REFERENCE MATERIAL FOR DESIGN CALCULATIONS

- □ 2003 International Building Code
- □ American Concrete Institute (ACI) 318-02
- □ Embedment Properties for Headed Studs, TRW Nelson, Design Data Catalog
- □ Steel Construction Manual, AISC, LRFD (1999
- □ ASCE 7-02

1.2 DESIGN CRITERIA USED IN CALCULATIONS

- □ Reinforcing Steel Yield Strength = fy = 60 ksi
- □ Structural Steel is ASTM A 36/A 36M-00
- □ Unconfined Compressive Strength of Concrete = f'c = 5000 psi
- □ Unit weight of Concrete = 110 pcf
- □ Stud Yield Strength = 50 ksi

1.3 INTERNATIONAL BUILDING CODE REQUIREMENTS

The following is a summary of the Code requirements applicable to CellXion precast concrete equipment shelters.

1.3.1 Occupancy Classification

Occupancy may be Group S-2 per sec 311, Group B per sec 304 or Group U per sec 312.

1.3.2 Construction Type

Type V-B per section 602.5 and Table 601.

1.3.3 Building Limitations

Occupancy S-2 or B or U

Relative to the location of the nearest structure or property line:

Walls must be rated one hour if less than 10 feet.	(Table 602)
Maximum size of S-2 building (Table 503) is 13,500 SF, 2 story.	(Table 503)
Maximum size of B building (Table 503) is 9,000 SF, 2 story.	(Table 503)
Maximum size of U building (Table 503) is 5,500 SF, 1 story.	(Table 503)

NOTE: STANDARD SHELTERS MAY BE RATED UP TO 2-HOURS.

REF: Table 720.1(2), Item number 4-1.1, Sand-lightweight concrete 4 inches thick.

IF PROTECTED OPENINGS ARE REQUIRED:

3/4 HOUR RATED OPENINGS ARE REQUIRED IN ONE HOUR ASSEMBLIES.

1.5 HOUR RATED OPENINGS ARE REQUIRED IN TWO HOUR ASSEMBLIES.

Unprotected Openings Allowed	Protected Openings Allowed	Table 704.8
Not permitted up to 5 feet.	Not permitted up to 3 feet.	
10% permitted > 5 feet to 10 feet.	15% permitted > 3 feet to 5 feet.	
15% permitted > 10 feet to 15 feet.	25% permitted > 5 feet to 10 feet.	
25% permitted > 15 feet to 20 feet.	45% permitted > 10 feet to 15 feet.	
45% permitted > 20 feet to 25 feet.	75% permitted > 15 feet to 20 feet.	
70% permitted > 25 feet to 30 feet.	No restriction > 20 feet.	
No restriction > 30 feet.		

1.4 FLOOR LOADS

Floor live load required (Table 1607.1) for light storage is; 125 psf

The summary loading chart in Section 2.0.1 indicates allowable load of:

310 psf 11.667 ft wide **OK**

For some equipment, such as batteries, a concentrated load is realized (2.5 SF in size).

Section 2.3.6 shows that concentrated loads of 1680 lbs can be placed anywhere.

If the concentrated load is next to the wall, 6614 lbs can be used.

1.5 ROOF LOADS Minimum roof live load required (2006 IBC 1607.11.2.1) is:

$L_r = L_o R_1 R_2$ [sec 1607.11.2.1, Eq 16-27]

$L_o =$ 20 psf (worst case) [sec 1607.11.2.1]

$R_1 =$ 1 (worst case for smaller shelters) [sec 1607.11.2.1, Eq 16-28]

F = .167 in per ft slope $R_2 =$ 1 (for F< 4) [sec 1607.11.2.1, Eq 16-31]

$L_r =$ 20 psf

The summary loading chart in Section 2.0.1 indicates allowable loads of:

154 psf 11.67 ft wide shelter **OK**

Snow Loads Section 1608.2 requires use of section 7 of ASCE 7-05

$p_f = 0.7\ C_e\ C_t\ I\ p_g$ [ASCE 7-05, Equation 7-1, Sec 7.3]

$p_f =$ (Min. design live load for roofs from section 2 of these calcs)

= 154 psf 11.67 ft wide shelter

$C_e =$ 1.2 (worst case-ASCE 7-05,Table 7-2, lesser factors may be used as appropriate)

$C_t =$ 1.0 (From ASCE 7-05, Table 7-3, heated structure)

$I =$ 1.0 (Category II, ASCE 7-05 Table 7-4)

Using the design load from section 2 for p_f and solving for p_g:

$p_g = p_f\ /\ (\ 0.7\ C_e\ C_t\ I\ )$

= **(Allowable ground snow load)**

= 184 psf 11.67 ft wide shelter

1.6 WIND LOADS

Sect. 1609.1.1 allows ASCE 7-05, Chapter 6; use sec 6.4, Method 1 - Simplified Procedure:

V = 150 mph [ASCE 7-05, Section 6.5.4 and Figure 6-1]

$I =$ 1.0 [ASCE 7-05,Category II, Table 6-1 >> Table 1-1]

Exposure Classification: C [ASCE 7-05, section 6.5.6.3]

Exposure C multiplier: $\lambda =$ 1.21 [ASCE 7-07, section 6.4.2 & Figure 6-2]

Enclosure Classification: enclosed [ASCE 7-05, section 6.5.9]

Roof angle: 0 to 5 degrees $K_{zt} =$ 1.0 [ASCE 7-05, sec 6.5.7.2]

MWFRS Design Wind Pressures: [From ASCE 7-05, Figure 6-2]

$p_s = \lambda K_{zt} I p_{s30}$ [ASCE 7-05, sec 6.4.2.1, Eq 6-1]

WALLS:	**43.2 psf**	**[zone A]**
	-22.4 psf	[zone B, negligible--> only 1 inch tall]
	28.7 psf	[zone C]
	-13.3 psf	[zone D, negligible--> only 1 inch tall]
	Zone A controls, use it for analysis	
ROOF:	**-51.9 psf**	**[zone E]**
	-29.5 psf	[zone F]
	-36.1 psf	[zone G]
	-22.9 psf	[zone H]
	Zone E controls, use it for analysis	

1.6.1 Check structural connections for carrying wind loads to the foundation.

The worst case for the windward forces are when they are projected onto the long walls. Half of the load is carried to the floor connections and half is carried to the roof connections.

The walls are 9.250 ft tall

The connections which connect the long walls to the end walls are neglected for the purposes of this particular analysis. Analysis with Calculations from section 3

1.6.1.1 Check connections for transfer of windward loads from wall to the floor and roof.

The connections along the top and bottom of the walls are at a standard spacing of 56 inches. This will be the tributary width of wind load for each connection at the floor and roof. The load for this tributary area on the windward wall is then:

P'(w)= P(windward wall) x tributary area

Where tributary area = (9.250 ft / 2) x 4 ft 8 in = 21.583 sq. ft.

= 43.2 psf x 21.583 sq. ft.

P'(w)= 932 lbs

This load is resisted by three main components of the connection at the floor:

5.95	kips	Capacity of P/N 223100 in tension per Clacs Section 3.3.1
22.87	kips	Capacity of the Floor Lifting Insert in shear per Clacs Section 3.7
8.35	kips	Capacity of the weld which connects the plates per Clacs Section 3.8

The capacity of all 3 components exceed the wind load OK

This load is resisted by three main components of the connection at the roof:

3.52	kips	Capacity of P/N 223000 in shear per Clacs Section 3.4.3
5.95	kips	Capacity of P/N 222000 in tension per Clacs Section 3.5.1
8.35	kips	Capacity of the weld which connects the plates per Clacs Section 3.8

The capacity of all 3 components exceed the wind load OK

1.6.1.2 Check connections for transfer of leeward loads from wall to the floor and roof.

The leeward wall has similar construction, but the loads are less and are outward.

P'(l)= P(leeward wall) x tributary area

Where tributary area = (9.250 ft / 2) x 4 ft 8 in = 21.583 sq. ft.

= 43.2 psf x 21.583 sq. ft.

P'(l)= 932 lbs (negative indicating an outward direction)

This load is resisted by three main components of the connection at the floor:

5.95	kips	Capacity of P/N 223100 in tension per Section 3.3.1
22.87	kips	Capacity of the Floor Lifting Insert in shear per Clacs Section 3.7
8.35	kips	Capacity of the weld which connects the plates per Section 3.8

The capacity of all 3 components exceed the wind load OK

This load is resisted by three main components of the connection at the roof:

3.52	kips	Capacity of P/N 223000 in Y-shear per Section 3.4.3
5.95	kips	Capacity of P/N 222000 in tension per Section 3.5.1
8.35	kips	Capacity of the weld which connects the plates per Section 3.8

The capacity of all 3 components exceed the wind load OK

1.6.1.3 Windward and leeward loading transfer to endwalls:

The loads on the top half of the shelter must be transferred to the ground through the connections on the endwalls. There are three connections from the roof to the endwall and three connections from the endwall to the floor. The load on the projected area of the top half of the long side of the shelter is resisted by these connections and is assumed to distribute half of the load to each endwall.

A shelter which is 16.000 feet long has a tributary area of:

Area = (9.250 feet / 2) x (16.000 feet / 2) = 37.000 sq. ft.

P(proj.)= 37.00 sq. ft. x 43.2 psf

= 1,598 lbs.

The roof connection consist of the same three components as were indicated in the sidewalls, except that they are loaded in a different direction. Their capacities are shown below.

7.04 kips Capacity of P/N 223000 in X-shear per Section 3.4.2
22.87 kips Capacity of the Wall Corner Insert per Section 3.6.1
8.35 kips Capacity of the weld which connects the plates per Section 3.8

Since there are three of these connections, the total capacity is: 21.12 kips **OK**

1.6.1.4 Windward and Leeward loading transfer to floor:

The same loads that are transferred to the endwalls from the roof need to be transferred to the floor panel. This is accomplished through the three connections at the base of the endwall.

The floor connections consist of the same three components as were indicated in the sidewalls, except that they are loaded in a different direction. Their capacities are shown below.

14.54 kips Capacity of P/N 223100 in X-shear per Section 3.3.2
22.87 kips Capacity of the Floor Lifting Insert in shear per Clacs Section 3.7
8.35 kips Capacity of the weld which connects the plates per Section 3.8

Since there are three of these connections, the total capacity is: 25.05 kips **OK**

1.6.1.5 Find horizontal forces and overturning moments.

This is used in the tie-down anchor analysis in 1.8 below.

Shelter Dims (feet)			Shelter Weight lbs	Hor.Wind (PxA-hor) lbs	Vert. Wind (PxA-vert.) lbs	Overturn Moment ft-lbs
Width	Length	Height				
11.67	16.00	10.083	34,084	6,969	9,690	91,662

1.6.1.6 Components and Cladding:

$p_{net} = \lambda K_{zt} I p_{net30}$ [ASCE 7-05, sec 6.4.2.2, Eq 6-2]

	POS	NEG		[From ASCE 7-05, Figure 6-3]
ROOF ZONE 1:	15.7	-44.8	psf	(100 sf effective wind area)
ROOF ZONE 2:	18.6	-73.4	psf	(20 sf effective wind area)
ROOF ZONE 3:	20.0	-123.7	psf	(10 sf effective wind area)

Allowable positive load on roof: (From section 2)
154 psf 11.67 ft wide

Allowable negative load on roof: (From section 2, neglecting DL)
-61.0 psf 11.67 ft wide

Allowable negative load on roof: (From section 2, including .6 x DL)
Roof Dead Load: 43.9 psf X .6 = 26.32 psf
-87.4 psf 11.67 ft wide **OK**

	POS	NEG		
WALL ZONE 4:	39.6	-43.4	psf	(200 sf effective wind area)
WALL ZONE 5:	45.9	-59.2	psf	(30 sf effective wind area)

Allowable load on walls: (From section 2)
87 psf 9.25 ft tall **OK**

The larger load at the corners does not produce a significant bending stress, and the shear strength of the roof panel will be more than adequate to resist this uplift load.
In addition, extra connections between the roof and endwalls anchor the roof at these end zones.

1.7 SEISMIC LOADS Section 1613.1, requires ASCE 7-05 for analysis.

Site Class is D [Section 1613.5.2, assumed due to unknown soil properties]
Occupancy Category: II [Table 1604.5]
Seismic Design Category: E [Table 1613.5.6]
Seismic Importance Factor I is: 1.00 [ASCE 7-05, sec 11.5, Table 11.5-1]

$V = C_s W$ [ASCE 7-05, sec 12.8.1, Eq. 12.8-1]
$W = D$ [ASCE 7-05, sec 12.7.2]
$C_s = S_{DS} / (R / I)$ [ASCE 7-05, sec 12.8.1.1, Eq. 12.8-2]
$V = (S_{DS} / (R / I)) D$
$R = 4$ [ASCE 7-05, Table 12.2-1, A.2]
$S_{DS} = 2/3\, S_{MS}$ [Per 1613.5.4, Eq. 16-39]
$S_{MS} = F_a S_S$ [Per 1613.5.3, Eq. 16-37]
$F_a =$ 1.0 [Table 1613.5.3(1)]
$S_S =$ 3.00 [Fig 1613.5(1), meets all US areas]
$S_{MS} =$ 3.00
$S_{DS} =$ 2.00
$V =$ 0.500 D [Use for base shear]

Determine E for use in load combinations on individual panel design.

$E = E_h + E_v$ [ASCE 7-05, sec 12.4.2, Eq. 12.4-1]
$E_h = \rho Q_E$ [ASCE 7-05, sec 12.4.2.1, Eq. 12.4-3]
$E_v = 0.2 S_{DS} D$ [ASCE 7-05, sec 12.4.2.2, Eq. 12.4-4]
$E = \rho Q_E + 0.2 S_{DS} D$
$Q_E = V$ [ASCE 7-05, sec 12.4.2.1] $\rho =$ 1.0 [ASCE 7-05, sec 12.3.4.2]
$E = \rho V + 0.2 S_{DS} D$ = 0.900 D [Use in load comb 4 & 6]
$E_m = E_{mh} - E_v$ [ASCE 7-05, sec 12.4.3, Eq. 12.4-6]
$E_{mh} = \Omega_0 Q_E$ [ASCE 7-05, sec 12.4.3.1 Eq. 12.4-7]
$E_m = \Omega_0 Q_E - 0.2 S_{DS} D$ $\Omega_0 =$ 2.5 [ASCE 7-05, Table 12.2-1, A.2]
$E_m =$ 0.850 D [Use in load comb 7]

$D_{wall} =$ 34.7 psf $D_{roof} =$ 43.9 psf $D_{floor} =$ 42.5 psf (calcs sec 4)

Load combinations: Section 1605.3.1 & 1605.4

Comb 1	D	[Notes 1, 2, 3]
Comb 2	D + L	[Notes 1, 2, 3]
Comb 3	D + L + (Lr or S or R)	[Notes 1, 2, 3]
Comb 4	D + (W or 0.7E) + L + (Lr or S or R)	[Notes 1, 2, 3, 4]
Comb 5	0.6 D + W	[Notes 1, 2, 3]
Comb 6	0.6D + 0.7E	[Notes 1, 2, 3, 4]
Comb 7	0.9D + E_m	See analysis below:

Note 1: Roof and floor panels are designed using 1.4D and 1.7L, exceeds req'd factors.
Note 2: Wall panels are designed using 1.4D and 1.7W, exceeds req'd factors.
Note 3: S, R, and Lr are used as L in panel calculations, see section 2 of these calcs.
Note 4: Wind loads control over Seismic.

Comb 7 check			psf	Min. Design Loads	
Walls:	0.9D + E_m =	1.750 D_{wall} =	61	87 psf	**OK**
Roof:	0.9D + E_m =	1.750 D_{roof} =	77	154 psf	**OK**
Floor:	0.9D + E_m =	1.750 D_{floor} =	74	310 psf	**OK**

1.7.1 Seismic loads from top half of the wall panel are transferred to the roof.

Equipment permanently installed in the building is estimated at 20,000 pounds. For a 16.00 ft long shelter, this is an average of 1250 pounds per linear foot. If this equipment is mounted to the floor and braced at the top, then half the seismic load from the equipment should be added to the top of the walls. Analysis uses sec 3 of these calculations.

The weight of a wall section transferred to the connections at 56" on center is:

= (56/12 ft wide) x (9.250 ft high) x (4 /12 ft thick) x 110 pcf)

W(wall) = 791 lbs

W(equipment) = (56/12 ft width) x(625 plf) = 2917 lbs

W(top of wall) = W(wall) + W(equipment) = 3,708 lbs

For the wall panel, the seismic shear is:

V = 1,854 lbs Seismic shear per connection plate at top of walls

This load is resisted by three main components of the connection at the floor:

5.95 kips Capacity of P/N 223100 in tension per Section 3.3.1

22.87 kips Capacity of the Floor Lifting Insert in shear per Clacs Section 3.7

8.35 kips Capacity of the weld which connects the plates per Section 3.8

The capacity of all 3 components exceed the seismic load OK

This load is resisted by three main components of the connection at the roof:

3.52 kips Capacity of P/N 223000 in shear per Section 3.4.3

5.95 kips Capacity of P/N 222000 in tension per Section 3.5.1

8.35 kips Capacity of the weld which connects the plates per Section 3.8

The capacity of all 3 components exceed the seismic load OK

1.7.2 Seismic loads from roof are transferred to the top of the endwall.

The seismic load at the top connection plates of the endwalls includes the seismic loads from the top quarter of two sidewalls, one half of the roof, and one half of the total equipment. Use a 9.25 ft tall wall x 15.33 ft long, and use a 11.997 ft wide x 16.33 ft long roof.

W(quarter wall)= 35.451 ft^2 x 4 /12 ft x 110 pcf = 1,300 lbs.

W(half roof)= 97.956 ft^2 x 4.25 /12 ft x 110 pcf = 3,816 lbs.

W(equipment) = 8 ft^2 x 625 plf = 5,000 lbs

TOTALW(top of endwall)= 1,300 lbs x 2 + 3,816 lbs + 5,000 lbs = 11,416 lbs.

The seismic load is then: V(top of endwall) = 5,708 lbs.

The roof connection consist of the same three components as were indicated in the sidewalls, except that they are loaded in a different direction. Their capacities are shown below.

7.04 kips Capacity of P/N 223000 in X-shear per Section 3.4.2

22.87 kips Capacity of the Wall Corner Insert per Section 3.6.1

8.35 kips Capacity of the weld which connects the plates per Section 3.8

Since there are three of these connections, the total capacity is:

21.12 kips **This capacity exceeds the seismic load OK**

1.7.3 Seismic loads from endwall are transferred to the floor.

The connections at the bottom of the endwalls have the same seismic load as the connections at the top, except that the seismic load from the endwall itself is added.

The weight of the endwall is: W(endwall)= 11.667 ft x 9.250 ft x 4 /12 ft x 110 pcf = 3,957 lbs

V(endwall)= 1,979 lbs

V(bottom)= V(top of endwall) + V(endwall) = 7,686 lbs

The same loads that are transferred to the endwalls from the roof need to be transferred to the floor panel. This is accomplished through the three connections at the base of the endwall.

The floor connections consist of the same three components as were indicated in the sidewalls, except that they are loaded in a different direction. Their capacities are shown below.

14.54 kips	Capacity of P/N 223100 in X-shear per Section 3.3.2
22.87 kips	Capacity of the Floor Lifting Insert in shear per Clacs Section 3.7
8.35 kips	Capacity of the weld which connects the plates per Section 3.8

Since there are three of these connections, the total capacity is:

25.05 kips **This capacity exceeds the seismic load** **OK**

1.8 Check shelter tie-downs to foundation For tie-down anchor capacity see Sec 3.9 of calcs:

Horizontal:	10472 lbs	Per connection
Vertical:	6615 lbs	Per connection

Horizontal forces due to seismic/wind loads:

Shelter Dims (feet)			Shelter	Contents	Seis.Load	Wind load	Control'g	Tie-down	CHECK	Safety
Width	Length	Height	Weight	Weight	(W x Cs)	1.6.1.5	Load	Capacity		Factor
11.67	16.00	10.083	34,084	11,248	22,666	6,969	SEISMIC	41,887	OK	1.85

Friction against sliding is ignored.

lengths under 24 ft have 4 tie-downs, lengths 24 ft and over have 8 tie-downs

Overturning forces due to seismic/wind loads:

			Seis.load	Overturn	Wind over.	Control'g	Overturn	Tie-down	CHECK	Safety
Shelter Dims (feet)			(W x Cs)	Force	See1.6.1.5	Load	Resist.	Capacity		Factor
Width	Length	Height	lbs.	lbs.	ft-lbs.		ft-lbs.	lbs		1.5 req'd
11.67	16.00	10.083	22,666	114274	91,662	SEISMIC	178946	41,887	OK	3.70

Overturning resistance uses 0.9 x DL of shelter (no contents)

Weight of shelter and contents are the same as in the horizontal force chart above.

2.0 DESIGN CRITERIA

NOTE: These calculations represent the panels of a
11.667 ft wide x 16.000 ft long x 9.250 ft tall shelter.

STRUCTURAL PROPERTY		UNITS	LABEL
Concrete Compressive Strength	5000	psi	f'_c (sand-lightweight)
Reinforcing bar Yield Stress	60000	psi	fy[REBAR]
Concrete Density	110	pcf	DENSITY
Maximum Building Width	11.667	feet	BLDGW
Maximum Building Length	16	feet	BLDGL
Maximum Wall Panel Height	9.25	feet	WALLH
Max. Est. weight of Shelter	34,084	LBS.	BLDGWT
Concrete volume req'd.	10.68	YDS.	CONCYDS
Roof thickness at peak	5	inches	H[ROOF]
Roof thickness at edge	4	inches	
Rebar size used in roof #	4	REBAR	REBARROOF
Rebar lateral spacing: roof	7	inches	ROOFSPACING
Longitudinal rebar spacing-roof:	18	inches	
Steel mesh used in roof:	W4	WIRE	
Steel spacing in roof (12"max.)	4	inches	
Steel mesh used in wall:	W4	WIRE	REBARWALL
Rebar size used in wall #	4	REBAR	REBARWALL2
Steel spacing in wall (12"max.)	4	inches	WALLSPACING
Vertical rebar spacing in wall	36	inches	WALLSPACING2
Horizontal rebar spacing in wall	48	inches	
Wall panel thickness	4	inches	WALLTHICKNESS
Rebar size used in floor #	6	REBAR	REBARFLR
Number of rebar per floor rib	2	each	REBARFLRQTY
Spacing of ribs in floor	19	inches	FLOORSPACING
Floor thickness	5.75	inches	H[FLOOR]
Floor deck thickness	2.75	inches	H[DECK]
Floor rib width	4	inches	B[RIB]
Floor deck steel size	W4	WIRE	
Floor deck steel spacing	4	inches	
Area per roof rebar	**0.200**	sq. in.	A[REBARROOF]
Diameter of roof rebar	**0.500**	inches	DIA[REBARROOF]
Area per roof wire	**0.040**	sq. in.	
Area per wall wire	**0.040**	sq. in.	A[REBARWALL]
Area per extra vert wall rebar	**0.200**	sq. in.	A[REBARWALL2]
Diameter of wall wire	**0.225**	inches	DIA[REBARWALL]
Diameter of wall rebar	**0.500**	inches	
Area of floor rebar	**0.880**	sq. in.	A[REBARFLR]
Diameter of floor rebar	**0.750**	inches	DIA[REBARFLR]
Area of deck rebar/wire	**0.040**	sq. in.	A[REBARDECK]
Diameter of deck rebar/wire	**0.225**	inches	DIA[REBARDECK]
Area of deck steel per foot	**0.120**	sq.in./ft.	A[DECKSTEEL]
Minimum req'd deck steel/foot	**0.059**	sq.in./ft.	A[DECKSTEEL-MIN]

2.0.1 STRUCTURAL LOADING SUMMARY FOR PANELS, AS DESIGNED.

PANEL	ALLOWABLE LOAD				TYPE
roof	**154**	psf	11.667	ft wide	LIVE
floor	**310**	psf	11.667	ft wide	LIVE
wall	**87.3**	psf	9.250	ft tall	WIND

2.0.2 CHECK STEEL RATIOS (ACI 318-05, sect. 21.7.2.3)

					ρ_t	ρ_v	
B_1 =	0.80			ROOF:	0.0114	0.0069	**OK**
	ρ_b	ρ_{max}	ρ_{min}	FLOOR:	0.0100		**OK**
	0.0335	0.0252	0.0033	WALL:	0.0066	0.0062	**OK**

Min reqd. per ACI 318-05, sec 21.7.2.1 0.0025

2.0.3 CHECK DEVELOPMENT LENGTH

		Wall	Roof	Floor
Largest of:	10 db =	2.3 in	5.0 in	7.5 in
		7.5 in	7.5 in	7.5 in
	$1.25\ f_y\ d_b\ /\ (\ 65 \times f'_c{}^{1/2}\ )$	3.7 in	8.2 in	12.2 in

All rebar development lengths are 18 in **OK**

2.1 ROOF PANEL CALCULATIONS

Temperature steel required: Ats
Panels are 4.00 in thick, minimum.
Maximum thickness of roof panel is 5.00 inches at center peak.
Ats= Aconc x 0.0018
= 5.00 in. x 12 in. x 0.0018
= 0.1080 sq. in. per foot of width of roof panel.
Use #4 rebar at 18 inches, longitudinal: Ats(actual)= 0.2533 sq. in. **OK**

2.1.1 Determine shear strength: Vu[ROOF]

b[ROOF] = **12.0** inches
d[ROOFSHEAR]= 3 in. - DIA[REBARROOF] / 2
2.75 inches
Vu[ROOF]= .85 x .85 x 2 x (fc)^.5 x b[ROOF] x d[ROOFSHEAR]
= **3372** lbs.

2.1.2 Determine allowable live load due to shear: w[ROOFSHEARLL]

ROOFSPANSHEAR= bldgw - ((d[ROOFSHEAR + 4) x 2 / 12)
= **10.542** feet 11.67 ft wide shelter
w[ROOFDL]= density x thickness (4.5 in avg) = **41.3** psf (concrete only)
w[ROOFSHEARLL]= (Vu[ROOF] / ROOFSPANSHEAR - 1.4 x w[ROOFDL]) / 1.7
= **154** psf allowable roof live load due to shear strength 11.67 ft wide

2.1.3 Determine allowable live load due to moment: w[ROOFMOMENTLL]

A[ROOFSTEEL]= A[REBARROOF] x (12 inches / ROOFSPACING)
= **0.34** sq. inches per foot of roof panel
d[ROOFMOMENT]= (H[ROOF]) - (1 +DIA[REBARROOF] / 2)
= **3.75** inches
a[ROOF]= (A[ROOFSTEEL] x fy[REBAR]) / (.85 x fc x b[ROOF])
= **0.403** inches (for 8 to 11.5 wide shelters)
Mu[ROOF]= (.9/12) x A[ROOFSTEEL] x fy[REBAR] x (d[ROOFMOMENT] - a[ROOF] / 2)
= **5475** ft-lbs

l[ROOFSPAN]= BLDGW - .5 = **11.17** feet 11.67 ft wide
w[ROOFMOMENTLL]= [(8 x Mu[ROOF] / l[ROOFSPAN]^2) - (1.4 x w[ROOFDL])] / 1.7
= **173** psf allowable roof live load due to bending strength 11.67 ft wide

2.1.4 Determine allowable negative live load due to moment: w[ROOFNEGMOMENTLL]

d[RFNEGMOMENT]= 1 +DIA[REBARROOF] / 2)
= **1.25** inches
a[RFNEG]= (A[ROOFSTEEL] x fy[REBAR]) / (.85 x fc x b[ROOF])
= **0.403** inches
Mu[RFNEG]= (.9/12) x A[ROOFSTEEL] x fy[REBAR] x (d[RFNEGMOMENT] - a[RFNEG] / 2)
= **1617** ft-lbs
l[ROOFSPAN]= BLDGW - .5 = **11.17** feet 11.67 ft wide
w[ROOFNEGMOMLL]= [(8 x Mu[ROOF]) / (l[ROOFSPAN]^2)] / 1.7
= Allowable negative roof live load due to bending strength (neglecting dead load)
= **-61.0** psf 11.67 ft wide

2.1.5 CHECK SHEAR ALLOWED PARALLEL TO PLANE OF ROOF

2.1.5.1 CHECK SHEAR ALLOWED FOR ONE CURTAIN OF REINFORCEMENT

Use 4 inch panel, 4 foot length, for minimum A_{CV}. (ACI 318-05, 21.7.2.2)
$2 A_{CV} \times f'_c{}^{1/2}$ = 27153 lbs [CONTROLS]

2.1.5.2 NOMINAL SHEAR FOR ROOF SECTION (per ACI 318-05, eq. 21-7)

Use 4 inch panel, 4 foot length, for minimum A_{CV}.
$V_n = A_{CV} (\alpha_c \times f'_c{}^{1/2} + \rho_t \times f_y)$ $\rho_t = A_s / A_{CV}$ = 0.0114
A_{CV} = 192 in^2 α_c = 2.0 (for $h_w / l_w > 2$)
= 158173 lbs [DOES NOT CONTROL]

2.1.5.3 NOMINAL SHEAR FOR ROOF DIAPHRAGM (per ACI 318, eq. 21-10)

Use 4 inch panel, 4 foot length, for minimum A_{CV}.
$V_n = A_{CV} (2 \times f'_c{}^{1/2} + \rho_t \times f_y)$
= 158173 lbs [DOES NOT CONTROL]

2.2 WALL PANEL CALCULATIONS

Temperature steel required: Ats
Panel thickness is: 4 inches Ats= Aconc x 0.0018
= 4 in. x 12 in. x 0.0018
= 0.0864 sq. in. per foot of width of wall panel.
(ACI 318-05, 14.3.5; 18" MAX) use 4x4-W4xW4 mesh:
Use #4 rebar at 48 inches, longitudinal: Ats(actual)= 0.1700 sq. in. per foot **OK**

2.2.1 Determine allowable loads perpendicular to plane of wall

2.2.1.1 Determine shear strength perpendicular to plane of wall: (Vu)

b[WALL] = 12 inches
d[WALL] = 2 inches (Distance from outside face of panel to center of rebar)
Vu[WALL]= .85 x .85 x 2 x (fc)^.5 x b[WALL] x d[WALL]
= **2452** lbs.

2.2.1.2 Determine allowable live load due to shear: w[WALLSHEARLL]

WALLSPANSHEAR= WALLH - (d[WALL] x 2 / 12)
= **8.92** feet 9.25 ft tall wall
w[WALLDL]= **36.67** psf (does not add to horizontal force)
NOTE: WALL DEAD LOAD DOES NOT ACT PERPENDICULAR TO PLANE OF PANEL.
w[WALLSHEARLL]= Vu[WALL] / (WALLSPANSHEAR] x 1.7)
= Allowable wall load due to shear strength
= **162** psf 9.25 ft tall wall

2.2.1.3 Determine allowable live load due to WINDWARD moment: w(WALLMOMENTLL)

A[WALLSTEEL]= A[REBARWALL]x(12"/WALLSPACING)+A[REBARWALL2]x12"/WALLSPACING2
= **0.19** sq. inches per foot of wall panel
a[WALL]= (A[WALLSTEEL] x fy[REBAR]) / (.85 x fc x b[WALL])
= **0.220** inches
Mu[WALL]= (.9/12) x A[WALLSTEEL] x fy[REBAR] x (d[WALL] - a[WALL] / 2)
= **1588** ft-lbs
w[WALLMOMENTLL]= [(8 x Mu[WALL] / l[WALLH]^2) - (1.4 x w[WALLDL])] / 1.7
= Allowable wall live load due to bending strength.
= **87.3** psf 9.25 ft tall wall

2.2.1.4 Determine allowable live load due to LEEWARD moment: w(WALLMOMENTLL)

d[LEEWALL] = 2 inches (Distance from inside face of panel to center of rebar)
a[LEEWALL]= (A[WALLSTEEL] x fy[REBAR]) / (.85 x fc x b[WALL])
= **0.220** inches
Mu[LEEWALL]= (.9/12) x A[WALLSTEEL] x fy[REBAR] x (d[WALL] - a[WALL] / 2)
= **1588** ft-lbs
w[LEEWALLMOMENTLL]= [(8 x Mu[WALL] / l[WALLH]^2) - (1.4 x w[WALLDL])] / 1.7
= Allowable wall live load due to bending strength.
= **87.3** psf 9.25 ft tall wall

2.2.2 CHECK SHEAR ALLOWED PARALLEL TO PLANE OF WALL

2.2.2.1 CHECK SHEAR ALLOWED FOR ONE CURTAIN OF REINFORCEMENT

Use 4 inch panel, 4 foot length, for minimum A_{CV}. (ACI 318-05, 21.7.2.2)
$2\ A_{CV} \times f'_c{}^{1/2} =$ 27153 lbs [CONTROLS]

2.2.2.2 NOMINAL SHEAR FOR WALL SECTION (per ACI 318-05, eq. 21-7)

Use 4 inch panel, 4 foot length, for minimum A_{CV}.
$V_n = A_{CV} (\alpha_c \times f'_c{}^{1/2} + \rho_t \times f_y)$ $\rho_t = A_s / A_{CV} =$ 0.0066
$A_{CV} =$ 192 in^2 $\alpha_c =$ 2.0 (for $h_w / l_w > 2$)
= 103716 lbs [DOES NOT CONTROL]

2.2.2.3 NOMINAL SHEAR FOR WALL DIAPHRAGM (per ACI 318-05, eq. 21-10)

Use 4 inch panel, 4 foot length, for minimum A_{CV}.
$V_n = A_{CV} (2 \times f'_c{}^{1/2} + \rho_t \times f_y)$
= 103716 lbs [DOES NOT CONTROL]

2.3 FLOOR PANEL CALCULATIONS

2.3.1 Determine temperature steel required for the deck:

Deck temperature steel required is:

ATS[DECK]= H[DECK] X 12 in. X .0018
= 2.75 in. x 12 in. x 0.0018
= 0.0594 sq. in. per foot of width of floor panel.
A[DECKSTEEL]= **0.120** sq. in per foot of panel. **OK**

2.3.2 Determine floor deck strength:

DECKSPAN= FLOORSPACING - B[RIB]
= **15.0** inches
d[DECK]= H[DECK] -1 (Assumes mesh is 1" clear from bottom of deck)
= **1.75** inches
a[DECK]= (A[DECKSTEEL] x FY[REBAR]) / (.85 x fc x 12 in.)
= **0.1412** inches
Mu[DECK]= 0.9/12 x A[DECKSTEEL] x fy[REBAR] x (d[DECK] - (a[DECK] / 2))
= **907** ft-lbs
w[DECKTOTALMOM]= (Mu[DECK] x 8) / (DECKSPAN x 12 in. per ft.)^2
= **4643** psf
w[DECKDL]= (H[DECK] / 12 in. per ft. x 1 ft.^2 x DENSITY)
= **25.2** psf
w[DECKLLMOM]= (w[DECKTOTAL - 1.4 x w[DECKDL]) / 1.7
= **2711** psf
Vu[DECK]= .85 x .85 x 2 x (fc^.5) x d[DECK] x 12 in.
= **2146** lbs.
w[DECKTOTSHEAR]= 2 x (Vu[DECK] / L
= **3433** psf
w[DECKLLSHEAR]= (w[DECKTOTSHEAR] - 1.4 x w[DECKDL]) / 1.7
= **1999** psf
Allowable live load for the floor deck is: **1999** psf (FLOOR DECK SHEAR CONTROLS)

2.3.3 Determine floor rib strength:

Effective width of flange:	ACI 318-05, 8.10			flange width		
1/4 span:	=			**33.5**	inches	
Effective width of overhang:	ACI 318-05, 8.10					
8 times H[DECK]	=	**22**	inches	**48.0**	inches	
OR 1/2 clear dist.	=	**7.5**	inches	**19.0**	inches	<controls>

bf= **19.0** inches
d[FLOOR]= H[FLOOR] - (.75" + DIA[REBARFLR] / 2)
4.625 inches
a[FLOOR]= (A[REBARFLR] x fy[REBAR]) / (.85 x fc x bf)
= **0.654** inches
Mu[FLOOR]= (.9/12) x A[REBARFLR] x fy[REBAR] x (d[FLOOR] - a[FLOOR] / 2)
= **17020** ft-lbs
FLOORSPANMOM= BLDGW - .5 ft. = **11.17** feet 11.67 ft wide
w[FLOORMOMTOT]= 8 x Mu[FLOOR] / (FLOORSPANMOM)^2
= **1092** plf 11.67 ft wide shelter

w[FLOORDL]= ((H[DECK] x bf / 144) + b[RIB] x (H[FLOOR] - H[DECK]) / 144) x 1 ft.x DENSITY
= **49.1** plf (PER RIB) = 31.0 psf
w[FLOORMOMLL]= [W[FLOORMOMTOT] - (1.4 x w[FLOORDL])] / (1.7 x trib)
= **380** psf 11.67 ft wide shelter

2.3.4 Determine rib shear strength: Vu[FLOOR]

b[RIB] = **4.00** inches
A[RIBSHEAR]= (H[FLOOR] - (.75" + DIA[REBARFLR]/2)) x B[RIB]
18.50 sq. in.
ACI 318-05, 11.3.2.1
Vc[FLOOR]= .85 x (1.9 x (fc)^.5 + (2500 x A[REBARFLR] / (b[RIB] x d[FLOOR]) x 1) x b[RIB] x d[FLOOR]
= **3983** lbs.
But not greater than: .85 x 3.5 x f'c^.5 x b[RIB] x d[FLOOR]
= **3892** lbs.
USE **3892** lbs.
ACI 318-05, 8.11.8 Vc[FLOORALLOW]= 1.1xVc[FLOOR]= **4281** lbs.

2.3.5 Determine allowable live load due to shear: w[FLOORSHEARLL]

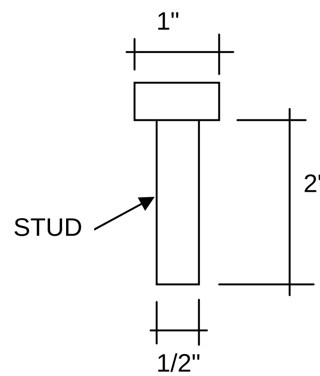
FLOORSPANSHEAR= bldgw - ((d[FLOOR + 8.5) x 2 / 12)
= **9.48** feet 11.67 ft wide shelter
w[FLOORSHEARLL]= (Vc[FLOORALLOW] / (.5xFLOORSPANSHEAR)-1.4 x w[FLOORDL]) / (1.7xFLOORSPACING/12)
= Allowable floor live load due to shear strength
= **310** psf 11.67 ft wide shelter
Allow live load for the 11.67 ft wide floor rib is **310** psf (FLOOR RIB SHEAR CONTROLS)
Gross allowable load = LL + 42 psf DL = 353 psf for a 11.67 ft wide shelter

2.3.6 Determine allowable concentrated load over 2.5 sf.

2.5 square foot area is equivalent to approximately 19 inch x 19 inch, or 1.58 feet x 1.58 feet.
Assume one rib takes the entire concentrated load.
Allowable load based on shear is: 310 psf
For a 11.67 foot wide shelter with an 10.67 foot span the equivalent concentrated load is:
P[shear] = 10.67 ft x 310 psf x 2.00
= **6614 lbs** **Maximum concentrated load (shear).**
Maximum live load for bending on one rib is:
w[FLOORRIBLL]= w[FLOORMOMLL] x BF / 12 = 602 plf
Make uniform load moment equal to concentrated load moment and solve for P.
w[FLOORRIBLL]x (FLOORSPANMOM^2) /8 = P x FLOORSPANMOM / 2
P(moment) = w[FLOORRIBLL] x (FLOORSPANMOM) / 4
= **1680 LBS** **Maximum load in center of floor (bending).**
If the load is next to the wall (as is usually the case with batteries) :
w[FLOORRIBLL]x (FLOORSPANMOM^2) / 8 = P x 1.5
P(moment) = w[FLOORRIBLL] x (FLOORSPANMOM^2) * (2 x 8)
= **6255 LBS** **Maximum load next to wall (bending).**
Shear controls.
Shear controls when load is next to wall.

3.0 INSERT PLATE ANALYSIS

(Analysis per ACI 318-05, Appendix D)



3.1 Material Properties

$f'_c =$ 5000 psi (sand-lightweight)

$f_{uta} =$ 61 ksi

$A_{se} =$ 0.196 in^2

$A_{brg} =$ 0.589 in^2

$h_{ef} =$ 2 in

$d_o =$ 0.5 in

3.2 Stud Analysis

3.2.1 Per D.5.3.4, Pullout strength in tension shall not exceed:

$N_p = 8\ A_{brg}\ f'_c$ = 23,562 lbs/stud

(due to crushing strength of concrete at the head of the stud.

3.2.2 Basic tension breakout strength of stud shall not exceed:

$N_b = k_c\ .85\ (f'_c)^{1/2}\ h_{ef}^{1.5}$ $k_c =$ 24 (for cast-in anchors)

= 4080 lbs/stud [Eq D-7] Sec D.5.2.2

3.2.3 Check ductile strength of stud.

$N_{sa} = A_{se} f_{uta}$ = 11.98 kips/stud

$\phi =$ 0.75 [See D.4.4 a) i)]

$\phi N_{sa} =$ 8.98 kips/stud

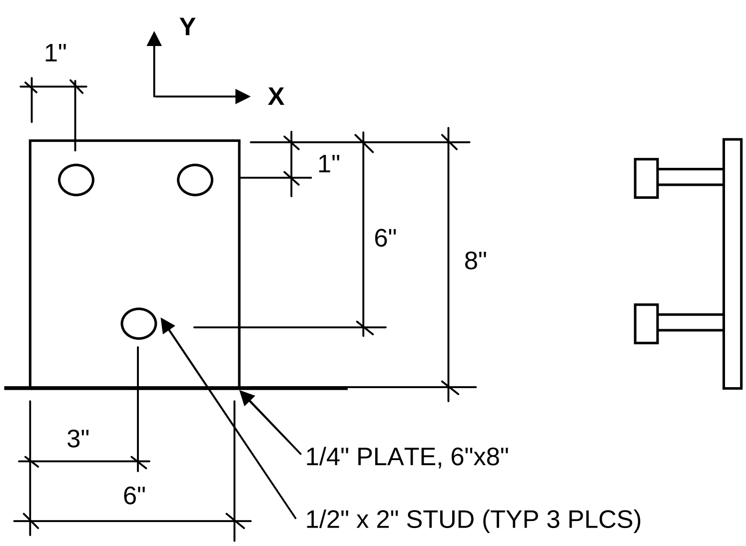
3.2.3 Check shear strength of stud.

$V_{sa} = A_{se} f_{uta}$ = 11.98 kips/stud

$\phi =$ 0.65 [See D.4.4 a) ii)]

$\phi N_{sa} =$ 7.79 kips/stud

3.3 INSERT PLATE "P/N 223100" ANALYSIS



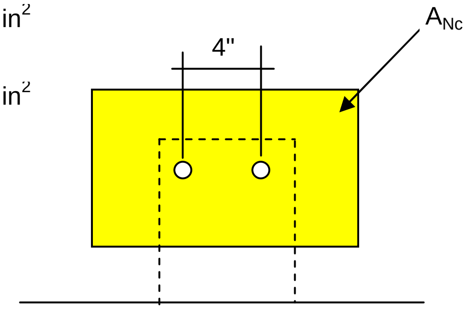
3.3.1 Tension Capacity of "P/N 223100" plate:

$N_{cbg} = (A_{nc}/A_{nco})\psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ [Eq D-5] Sec D.5.2.1

$A_{Nco} = 9h_{ef}^2$ = 36 in^2

Find A_{Nc} for just the two upper studs.

$A_{Nc} = A_{Nco} + 4(3)(h_{ef})$ = 60 in^2



$\psi_{ec,N}$ = 1.0 assume no eccentricity

$\psi_{ed,N}$ = 1.0 (c_a min > 1.5 h_{ef} for 2 studs)

$\psi_{c,N}$ = 1.25 (for cast-in anchors)

$\psi_{cp,N}$ = 1.0 (for cast-in anchors)

N_{cbg} = 8500 lbs ϕ = 0.70 [Use condition B, D.4.4]

ϕN_{cbg} = 5950 lbs
TENSION CAPACITY OF "P/N 223100" PLATE

3.3.2 Shear Capacity of "P/N 223100" plate in the X-direction:

This shear force is parallel to the edge of the panel.

$V_{cbg} = 2(A_{vc}/A_{vco})\psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ V_b$ [Eq D-22] Sec D.6.2.1 (b)

where: $V_b = 7(l_e/d_o)^{0.2}\ (d_o)^{1/2}\ .85(f'_c)^{1/2}\ (c_{a1})^{1.5}$

l_e= h_{ef} = 2 inches

d_o = 0.5 inches c_{a1} = 7 inches

V_b = 7270 lbs/stud [Eq D-24] Sec D.6.2.2

$\psi_{ec,V}$ = 1.0 assume no eccentricity $\psi_{ed,V}$ = 1.0

$\psi_{c,V}$ = 1.2 (for #4 bar between anchor and edge)

h_a = 4 inches s_1 = 4 inches

$A_{vco} = 2(1.5\ c_{a1})\ h_a$ = 84 in^2

$A_{vc} = (\ 2\ (1.5\ c_{a1}) + s_1)\ h_a$ = 100 in^2

V_{cbg} = 20772 lbs ϕ = 0.70 [Use condition B, D.4.4]

ϕV_{cbg} = 14540 lbs
SHEAR CAPACITY OF "P/N 223100" PLATE IN X-DIRECTION

3.3.3 Shear Capacity of "P/N 223100" plate in the (negative) Y-direction:

This shear force is perpendicular to the edge of the panel.

NOTE: The lower stud is ignored since it is close to the free edge.

$V_{cbg} = (A_{vc}/A_{vco})\psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ V_b$ [Eq D-22] Sec D.6.2.1 (b)

V_b = 7270 lbs/stud from 3.3.2 above

$\psi_{ec,V}$ = 1.0 assume no eccentricity

$\psi_{ed,V}$ = 1.0 $c_{a2}>1.5c_{a1}$

$\psi_{c,V}$ = 1.2 (for #4 bar between anchor and edge)

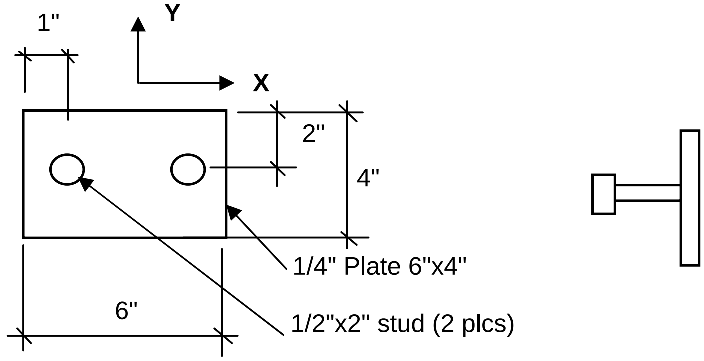
h_a = 4 inches s_1 = 4 inches

A_{vco} = 84 in^2 A_{vc} = 100 in^2 from 3.3.2 above

V_{cbg} = 10386 lbs ϕ = 0.70 [Use condition B, D.4.4]

ϕV_{cbg} = 7270 lbs
SHEAR CAPACITY OF "P/N 223100" PLATE IN Y-DIRECTION

3.4 INSERT PLATE "P/N 223000" ANALYSIS



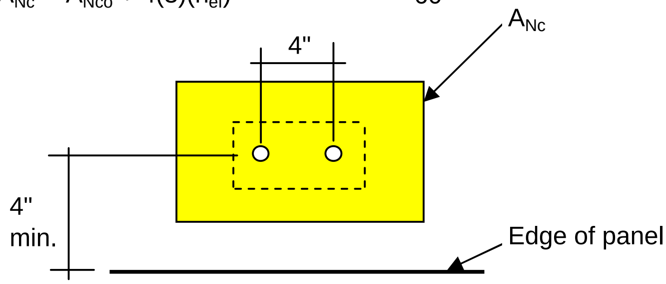
3.4.1 Tension Capacity of "P/N 223000" plate:

$N_{cbg} = (A_{nc}/A_{nco})\psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ [Eq D-5] Sec D.5.2.1 (b)

$A_{Nco} = 9h_{ef}^2$ = 36 in^2

Find A_{Nc} for just the two upper studs.

$A_{Nc} = A_{Nco} + 4(3)(h_{ef})$ = 60 in^2



$\psi_{ec,N}$ = 1.0 assume no eccentricity

$\psi_{ed,N}$ = 1.0 (c_a min > 1.5 h_{ef} for 2 studs considered)

$\psi_{c,N}$ = 1.25 (for cast-in anchors)

$\psi_{cp,N}$ = 1.0 (for cast-in anchors)

N_{cbg} = 8500 lbs

ϕ = 0.70 [Use condition B, D.4.4]

ϕN_{cbg} = 5950 lbs

TENSION CAPACITY OF "P/N 223000" PLATE

3.4.2 Shear Capacity of "P/N 223000" plate in the X-direction:

This shear force is parallel to the edge of the panel.

$V_{cbg} = 2(A_{vc}/A_{vco})\psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ V_b$ [Eq D-22] Sec D.6.2.1 (b)

where: $V_b = 7(l_e/d_o)^{0.2}\ (d_o)^{1/2}\ .85(f'_c)^{1/2}\ (c_{a1})^{1.5}$

l_e= h_{ef} = 2 inches

d_o = 0.5 inches c_{a1} = 4 inches

V_b = 3140 lbs/stud [Eq D-24] Sec D.6.2.2
ψ_{ec},V = 1.0 assume no eccentricity ψ_{ed},V = 1.0
ψ_c,V = 1.2 (for #4 bar between anchor and edge)
h_a = 3.5 inches [at step-joint] s_1 = 4 inches
A_{vco} = $2(1.5\ c_{a1})\ h_a$ = 42 in^2
A_{vc} = $(2\ (1.5\ c_{a1}) + s_1)\ h_a$ = 56 in^2
V_{cbg} = 10049 lbs
ϕ = 0.70 [Use condition B, D.4.4]

ϕV_{cbg} = 7035 lbs
SHEAR CAPACITY OF "P/N 223000" PLATE IN X-DIRECTION

3.4.3 Shear Capacity "P/N 223000" in the neg Y-direction (toward free edge):

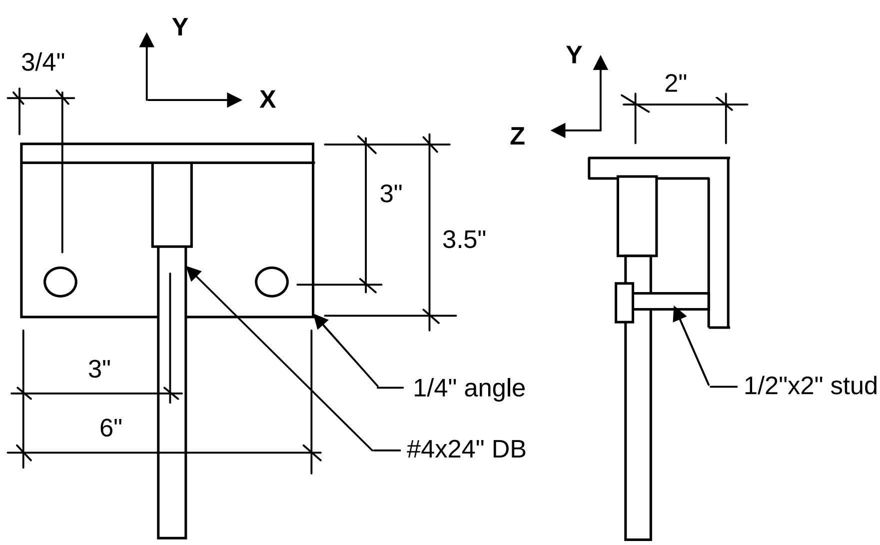
This shear force is perpendicular to the edge of the panel.

$V_{cbg} = (A_{vc}/A_{vco})\psi_{ec},V\ \psi_{ed},V\ \psi_c,V\ V_b$ [Eq D-22] Sec D.6.2.1 (b)

V_b = 3140 lbs/stud from 3.4.2 above
ψ_{ec},V = 1.0 assume no eccentricity
ψ_{ed},V = 1.0 $c_{a2}>1.5c_{a1}$
ψ_c,V = 1.2 (for #4 bar between anchor and edge)
A_{vco} = 42 in^2 A_{vc} = 56 in^2 from 3.4.2 above
V_{cbg} = 5025 lbs ϕ = 0.70 [Use condition B, D.4.4]

ϕV_{cbg} = 3517 lbs
SHEAR CAPACITY OF "P/N 223000" PLATE IN Y-DIRECTION

3.5 INSERT ANGLE "P/N 222000" ANALYSIS



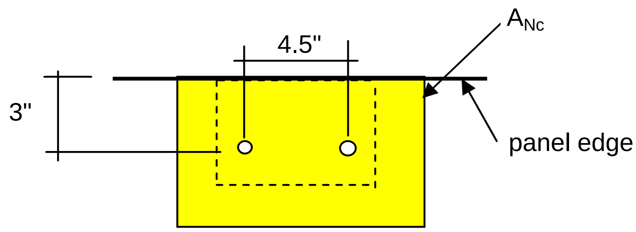
3.5.1 Tension Capacity of "P/N 222000" Insert Angle: (negative Z)

$N_{cbg} = (A_{nc}/A_{nco})\psi_{ec},N\ \psi_{ed},N\ \psi_c,N\ \psi_{cp},N\ N_b$ [Eq D-5] Sec D.5.2.1 (b)

$A_{Nco} = 9h_{ef}^2$ = 36 in^2

Find A_{Nc} for just the two studs.

$A_{Nc} = A_{Nco} + (4.5)(3)(h_{ef}) =$ 60 in^2



$\psi_{ec},N =$ 1.0 assume no eccentricity

$\psi_{ed},N =$ 1.0 (c_a min > 1.5 h_{ef} for 2 studs considered)

$\psi_c,N =$ 1.25 (for cast-in anchors)

$\psi_{cp},N =$ 1.0 (for cast-in anchors)

$N_{cbg} =$ 8500 lbs $\phi =$ 0.70 [Use condition B, D.4.4]

$\phi N_{cbg} =$ 5950 lbs
TENSION CAPACITY OF "P/N 222000" INSERT

3.5.2 Shear Capacity of "P/N 222000" Insert Angle in X direction:

This shear force is parallel to the edge of the panel.

$V_{cbg} = 2(A_{vc}/A_{vco})\psi_{ec},V\ \psi_{ed},V\ \psi_c,V\ V_b$ [Eq D-22] Sec D.6.2.1 (b)

where:

$V_b = 7(l_e/d_o)^{0.2}\ (d_o)^{1/2}\ .85(f'_c)^{1/2}\ (c_{a1})^{1.5}$

$l_e = h_{ef} =$ 2 inches

$d_o =$ 0.5 inches $c_{a1} =$ 3 inches

$V_b =$ 2040 lbs/stud [Eq D-24] Sec D.6.2.2

$\psi_{ec},V =$ 1.0 assume no eccentricity $\psi_{ed},V =$ 1.0

$\psi_c,V =$ 1.2 (for #4 bar between anchor and edge)

$h_a =$ 4 inches [at step-joint]

$s_1 =$ 4.5 inches

$A_{vco} = 2(1.5\ c_{a1})\ h_{a\ =}$ 36 in^2

$A_{vc} = (\ 2\ (1.5\ c_{a1}) + s_1)\ h_a =$ 54 in^2

$V_{cbg} =$ 7343 lbs $\phi =$ 0.70 [Use condition B, D.4.4]

$\phi V_{cbg} =$ 5140 lbs
SHEAR CAPACITY OF "P/N 222000" INSERT, X-DIRECTION

3.5.3 Shear Capacity of "P/N 222000" Insert Angle in Y direction:

This is for uplift forces from the roof panel.

$V_{cbg} = (A_{vc}/A_{vco})\psi_{ec},V\ \psi_{ed},V\ \psi_c,V\ V_b$ [Eq D-22] Sec D.6.2.1 (b)

$V_b =$ 2040 lbs/stud from 3.5.2 above

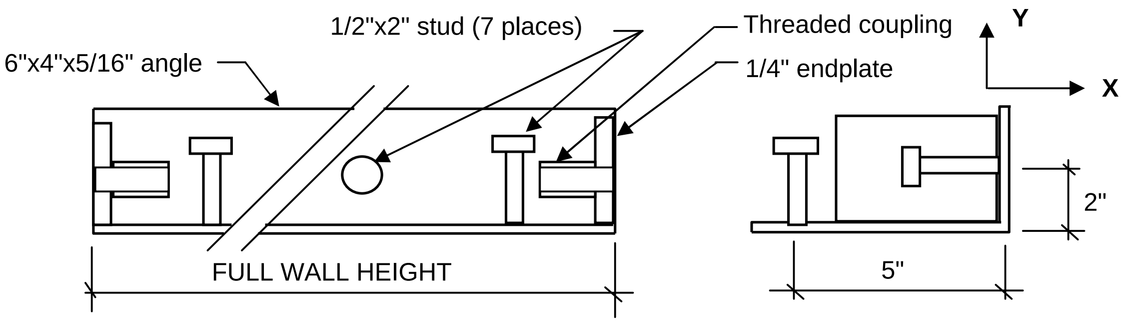
$\psi_{ec},V =$ 1.0 assume no eccentricity

$\psi_{ed},V =$ 1.0 c_{a2}>1.5c_{a1}

$\psi_c,V =$ 1.2 (for #4 bar between anchor and edge)
$A_{vco} =$ 36 in^2 from 3.5.2 above
$A_{vc} =$ 54 in^2 from 3.5.2 above
$V_{cbg} =$ 3672 lbs
$\phi =$ 0.70 [Use condition B, D.4.4]

$\boxed{\phi V_{cbg} = 2570 \text{ lbs}}$
SHEAR CAPACITY OF "P/N 222000" INSERT, Y-DIRECTION

3.6 WALL CORNER INSERT ANALYSIS



This insert is used on the vertical sides of the endwalls. The 4" leg forms the outside edge of the endwalls, and the 6" leg is abutted to the side walls and is used for the welded connection to the side wall, the roof, and the floor.
The primary loads on this insert are those from wind and seismic forces as they are transferred to/from the floor/roof panel by using the endwall as a shearwall against the forces as they are applied to the side walls.
The shearwall forces are applied in the X-direction as applied to the end view on the right side of the picture above. Of the 7 studs (minimum) that are on the insert, three of them would be analyzed for tension and the other four would be in shear. Depending on the direction of shear, (+X or -X direction), the free edge will come into play. This analysis will only consider the free edge allowable loads with the assumption that the insert will exceed that capacity when loaded in the opposite direction.

3.6.1 Capacity of Wall Corner Inserts in X-direction

Check capacity of individual studs on the 6" leg of the angle.
These studs would be in shear toward the free edge.

$V_{cb} = (A_{vc}/A_{vco})\ \psi_{ed},V\ \psi_c,V\ V_b$ [Eq D-21 Sec D.6.2.1 (a0]

where:

$V_b = 7(l_e/d_o)^{0.2}\ (d_o)^{1/2}\ .85(f'_c)^{1/2}\ (c_{a1})^{1.5}$

$l_e =$ $h_{ef} =$ 2 inches
$d_o =$ 0.5 inches $c_{a1} =$ 5 inches
$V_b =$ 4389 lbs/stud [Eq D-24] Sec D.6.2.2
$\psi_{ed},V =$ 1.0
$\psi_c,V =$ 1.2 (for #4 bar between anchor and edge)
$h_a =$ 4 inches [at step-joint] $s_1 =$ 24 inches
$A_{vco} = 4.5\ c_{a1}^2 =$ 112.5 in^2
$A_{vc} = 2(1.5\ c_{a1})\ h_{a} =$ 60 in^2

V_{cb} = 5618 lbs ϕ = 0.70 [Use condition B, D.4.4]

ϕV_{cb} = 3932 lbs
Shear capacity of studs on 6" leg, X direction.

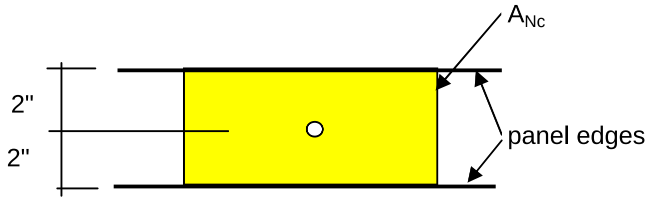
To this, add the tension load from the studs on the 4" leg.

$N_{cb} = (A_{nc}/A_{nco})\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ [Eq D-4] Sec D.5.2.1 (a)

$A_{Nco} = 9h_{ef}^2$ = 36 in^2

Find A_{Nc} c_{a1} = 2 inches h_{ef} = 2 inches

$A_{Nc} = 2(c_{a1}) \times 2(1.5\ h_{ef})$ = 24 in^2



$\psi_{ed,N}$ = 1.0 (c_a min > 1.5 h_{ef} for 2 studs considered)

$\psi_{c,N}$ = 1.25 (for cast-in anchors)

$\psi_{cp,N}$ = 1.0 (for cast-in anchors)

N_{cb} = 3400 lbs ϕ = 0.70 [Use condition B, D.4.4]

ϕN_{cb} = 2380 lbs
Shear capacity of studs on 6" leg, X direction.

These two were analyzed as individual studs since they are spaced 12 inches apart, far enough to act alone, not as a group.
In this direction, there would be a minimum of 4 studs in shear, and three studs in tension. The total allowable load is:

$P_x = 4(\phi V_{cb}) + 3(\phi N_{cb})$ = 22870 lbs
SHEAR CAPACITY OF WALL INSERT, +/- X-direction

3.7 FLOOR LIFTING INSERT ANALYSIS

The floor lifting inserts are made from 5"x5"x5/16" angle with a 5"x5/16" plate welded on the open top, to form a channel, and extend across the entire width of the floor panel at each end of the shelter. The inserts are similar to the wall corner inserts in design as they have no less than 6 studs, 1/2"x4" long, on 12" centers and two studs, 1/2"x2" long. These inserts provide three connection points for the endwall, and the two outer connections also double as side wall connections. The floor panel side inserts are made from a 5"x5"x5/16" angle with one side up and one side out, and extend the entire length of the shelter. They are also similar to the wall corner inserts in design by having a minimum of 6 studs, 1/2"x4" long, on 12" centers and four # 6 x 30" rebar splices. These inserts provide three or more connection points for the sidewall. By inspection these inserts are highly integrated into the floor structure. A failure would require much more than the shear cone failures as provided by the stud design manual. Therefore, the connections will be considered as equivalent to the analysis of the wall corner insert (sec 3.6.1).

3.8 CAPACITY OF WELDS AT CONNECTION PLATES

Welds to be made with SMAW, E70XX electrodes.
All standard connection plates will have a 3/16" weld, 3 inches long.
E70XX welds are good for .928 kips per inch per sixteenth inch of weld.
Weld capacity is then:

Pw = (0.928 k/inch/sixteenth) x (3 inches) x (3 sixteenths)

Pw =	8.352 kips
	CAPACITY OF ALL STANDARD CONNECTION PLATE WELDS

3.9 CAPACITY OF TIE-DOWN CONNECTION PLATES

Three failure modes are noted:

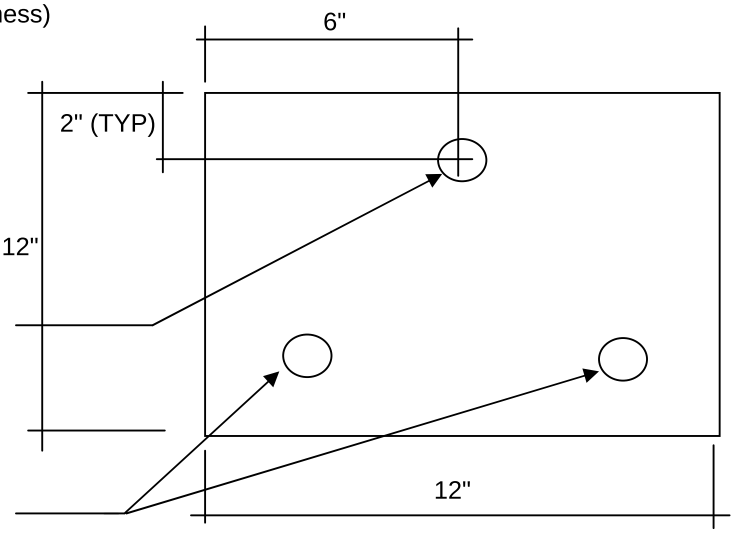
A: Failure of the connection plate.
B: Failure of the bolts connecting the plate to the shelter.
C: Failure of the expansion anchor connecting the plate to the foundation.

t = 0.25 " (plate thickness)

Fy = 36 ksi
Fu = 58 ksi

Bolt DIA = 1.00 "
1.25 "

anchor DIA = 0.75 "
0.875 "



A: Shear through edge of plate at one hole is:

HoleArea(bolt)=	D(top) x t	=	0.3125	in^2
HoleArea(anchor)=	D(bot) x t	=	0.21875	in^2
PL-Area =	t x (2" - (.5 x 1.25"))	=	0.34375	in^2
	cannot exceed t x 4t	=	0.25	in^2 CONTROLS

OK [exceeds 2/3 hole area, AISC, 360-05, D3.2]

Bearing on hole area:	Apl(bolt)=	0.25	in^2		
	Apl(anchor)=	0.1875	in^2		
	Fp(hole) = 1.0 Fu	=		58	ksi
	PL-bearing =	14.50	kips/ bolt hole		
	PL-bearing =	10.88	kips/ anchor hole		
Transient load factor:	1.333				
Capacity of connection plate is:	19.33	kips	(using 1 bolt and 2 anchors)		
	19333	lbs per connection			

B: 1" bolt capacity: Use A307 bolts or better

Fv =	10.0	ksi			
A(bolt) =	0.785	in^2			
Transient load factor:	1.333				
P(bolt) =	10.47	kips / bolt	=	10472	lbs per connection

C: Expansion anchor capacity from Hilti charts:
Reference ICC report #ESR-1385 & Tables 2 & 5

Anchor is Hilti Stainless Steel Kwik Bolt 3, 3/4" x 6.5"

Shear in horizontal direction (due to sliding of shelter):

See Table 5, 3000 psi normal weight concrete, in ICC report.

Embedment depth:	4.75	in OK	
Allowable load:	4225	lbs per anchor	

See Table 2, 3000 psi normal weight concrete, in ICC report.

Edge distance for max load:	9.75	in OK	(in direction of load)
Spacing req'd for full load:	10.75	in	
Min. spacing allowed:	4.75	in (10% reduction per note 4, table 2)	
Actual spacing:	8	in	
Interpolated reduction for spacing:	4.6	%	
Transient load factor:	1.333		
Modified allowable horizontal shear load:	5375	lbs per anchor	
times 2 =	10750	lbs per connection	

Shear in vertical direction (due to uplift of shelter):

Hilti Kwik Bolt 3 requirements

4.75" embedment

4.875" min. edge dist. allowed => use 50% of chart loads (note 6, table 2)

9.75" required for full load strength

6" edge distance => 38.46% Interpolated reduction

Allowable vertical load in 3000 psi concrete:

61.54%	x	4225	lbs	=	2600	lbs per anchor
Reduction for spacing (same as above):					4.6	%
Transient load factor:					1.333	
Modified allowable vertical shear load:					3308	lbs per anchor
times 2 =					6615	lbs per connection

Controlling loads for tie-down connections:

Horizontal (sliding):	10472	lbs
Vertical (uplift):	6615	lbs

4 CONCRETE BUILDING WEIGHT CALCULATOR

	Concrete Density =	110	pcf
	Concrete Required =	10.7	yards
4.1 Shelter Dimensions:		shelter dimensions	
	Width:	11.667	ft
	Length:	16.000	ft
	Height:	9.250	ft,(wall height)
		Weight, lbs	
	Material		
4.2 ROOF	CONCRETE	8281	
	2.25" INSULATION	66	
	7/16" OSB PANELING	248	
	3/8" OSB W/FINISH	211	
	Total Roof Wt.	**8806**	
	Avg. Dead Load, psf	43.9	
4.3 WALLS	CONCRETE	16352	
	1.75" INSULATION	138	
	7/16" OSB PANELING	460	
	3/8" OSB W/FINISH	395	
	Total Wall Wt.	**17345**	
	Avg. Dead Load, psf	34.7	
4.4 FLOOR	CONCRETE	7089	
	L5x5x5/16 PERIMETER BEAM	570	
	STYROFOAM (2 PCF DENSITY)	50	
	TILE, 1/8"	224	
	Total Floor Wt.	**7933**	
	Avg. Dead Load, psf	42.5	

4.5 WEIGHT SUMMARY:

		Building width, ft	Building length, ft	wall height, ft
Total Overall : lbs	**34084**	**11.667**	**16.000**	**9.250**