A&A ENGINEERING
CIVIL • STRUCTURAL
6036 Renaissance Place
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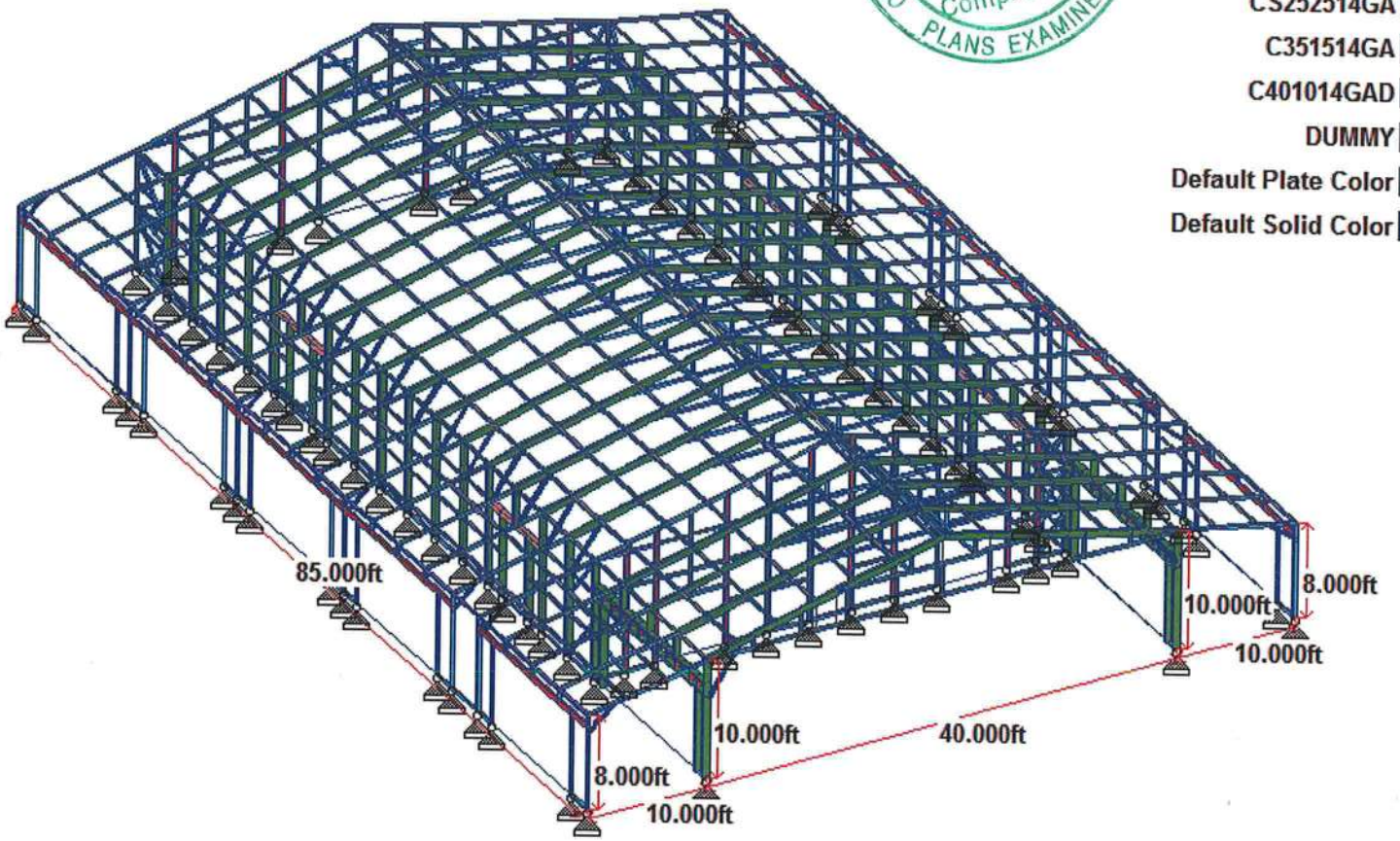
PROJECT: RONNIE MUNSON		PROJECT NO.: 356-22-0558	SHEET NO.: 1/10
CLIENT: REAL STEEL METAL BUILDINGS	LOCATION: FORT WHITE, FL 32038		
CALCULATED BY: A.F.	CHECKED BY: O.A.	APPROVED BY:	SCALE:
DATE: 3/24/2022	DATE: 3/24/2022	DATE:	

Entity Color Legend

- TS252514GA
- TS252514GAD14
- TS252514GADWS
- TS252514GAQ14
- CS252514GA
- C351514GA
- C401014GAD
- DUMMY
- Default Plate Color
- Default Solid Color



SAADY AMIN
LICENSE
No. 86959
STATE OF FLORIDA
PROFESSIONAL ENGINEER

DATE EXPIRES: 02/28/2023
DATE SIGNED: 04/12/2022

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PROJECT: RONNIE MUNSON		PROJECT NO.: 356-22-0558	SHEET NO.: 2/10
CLIENT: REAL STEEL METAL BUILDINGS	LOCATION: FORT WHITE, FL 32038		
CALCULATED BY: A.F.	CHECKED BY: O.A.	APPROVED BY:	SCALE:
DATE: 3/24/2022	DATE: 3/24/2022	DATE:	

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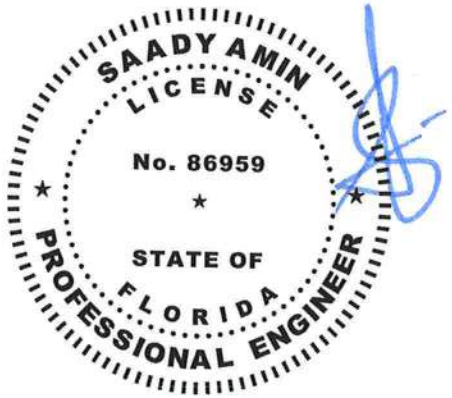

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PROJECT: RONNIE MUNSON		PROJECT NO.: 356-22-0558	SHEET NO.: 3/10
CLIENT: REAL STEEL METAL BUILDINGS	LOCATION: FORT WHITE, FL 32038		
CALCULATED BY: A.F.	CHECKED BY: O.A.	APPROVED BY:	SCALE:
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I. Summary:

1. **Main Building Size:**

Length:	**85** ft
Width:	**40** ft
Eave height:	**10** ft
Roof pitch:	**3** / 12

2. **Left Lean-To Building Size:**

Length:	**85 ft**
Width:	**10 ft**
Eave Heights:	**10 / 8 ft**
Roof Pitch:	**2.4 / 12**

3. **Right Lean-To Building Size:**

Length:	**85 ft**
Width:	**10 ft**
Eave Heights:	**10 / 8 ft**
Roof Pitch:	**2.4 / 12**
Total Building square footage:	**5100** Sq ft

4. **Loads:**

Roof live load:	**20** psf
Ground snow load:	**0** psf
Wind speed:	**120** mph
Exposure:	**C**
Seismic S_s spectral response accel.:	**0.08**
Seismic Design Category:	**B**

II. Code Information:

1. **Code:**

FBC 2020 - 7th Edition (IBC 2018)

2. **Occupancy:**

Occupancy Group:	**R (RESIDENTIAL)**
Construction Type:	**V-B**
Risk Category:	**II**

III. Load Calculations:

3. **Dead Load (D):**

Dead load (super imposed):	D_L = **2.00** psf
Dead load (Self weight):	D_S = **4.51** psf
Total load :	D_T = **6.51** psf

4. **Roof Live Load (L_r):**

Roof live load (specified): L_r = **20 psf**

5. **Snow Load(S):**

Peak ground snow load:	p_g = **0** psf
Exposure factor:	C_e = **1.00**
Thermal factor:	C_t = **1.20**
Importance factor:	I_s = **1.00**
Slope factor:	C_s = **1.0**
Flat roof load:	$p_f = \mathbf{0.7\ C_e\ C_t\ I_s\ p_g}$ ($\geq$ 20 psf)

$\rightarrow p_f = 0.7 \times 1.00 \times 1.20 \times 1.00 \times 0 =$ **0.00** psf

Sloped roof load: $p_s = \mathbf{C_s\ p_f}$

$\rightarrow p_s = 1.0 \times 0.00 =$ **0.00** psf = **S**

****Roof live load controls***

6. **Wind Load (W):**

*Please see Appendix W1-W5 for wind load calculations

Base Shear due to Wind Loads:	**V**
V_W in X dir:	**17.993** Kips
V_W in Z dir:	**15.650** Kips

(Refer Statics Check Results on sheet SO 11 / 30)

7. **Earthquake Load (E):**

Site Class:	**D (Default)**
Seismic Design category:	**B**
Spectral response acceleration at short periods:	S_S = **0.080** g
Site Coeffecient:	F_a = **1.600**
Adjusted spectral response acceleration:	$S_{MS} = F_a S_s$

$\rightarrow S_{MS} = 1.600 \times 0.080 =$ **0.128** % g

Design spectral response acceleration: $S_{DS} = \frac{2}{3} S_{MS}$

$\rightarrow S_{DS} = (2 / 3) \times 0.128 =$ **0.085** % g

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Lateral Force resisting system :
"steel ordinary concentrically braced frames"
Response modification factor: R = **3.25**
(ASCE Table 12.2-1)
Occupancy importance factor: I = **1.00**
Weight of structure: W = **33201.00** lbs
20% of Roof Live Load (20 psf): 0.20 L_r = **0.00** lbs
Effective Seismic weight of structure: $W_s = D + 0.20\ L_r$
W_s = **33201.00** lbs

Seismic response coeffecient: $C_s = \frac{S_{DS}}{R/I}$

→ C_s = 0.085 / (3.25 / 1.00) = **0.026**
Seismic base shear: $V_S = C_s\ W_s$
→ V_S = 0.026 × 33201.00 = **871.74 lbs**

Min. no. of resisting frames in X dir: n_X = **4**
Seismic Force per frame in X dir: $E_X = Vs / n_X$
→ E_X = 871.74 / 4 = **217.93 lbs**

Min. no. of resisting frames in Z dir: n_Z = **4**
Seismic Force per frame in Z dir: $E_Z = Vs / n_Z$
→ E_Z = 871.74 / 4 = **217.93 lbs**

Minimum Base Shear due to Wind Loads:
(Refer Statics Check Results on sheet SO 11 / 30)
V_W in X dir: **17.993** Kips
V_W = 17993.00 lbs > V_S =871.74 lbs

Thus, Wind load controls Base Shear in X direction.

V_W in Z dir: **15.650** Kips
V_W = 15650.00 lbs > V_S =871.74 lbs

Thus, Wind load controls Base Shear in Z direction.

IV. Load Combinations:

a. Strength

1. D + (L_r or S)
2. D + (0.6W or 0.7 E)
3. D + 0.75 (L_r or S) + 0.75 (0.6W)
4. 0.6 D + (0.6W or 0.7 E)

V. Material Properties:

1. **Steel:**

ASTM A572 Grade 50
F_y = **50** ksi
F_u = **65** ksi
E = **29,000** ksi
γ = **490** lbs/ft^3

VI. Section Properties:

Section	A	I_z	I_y	Gauge
TS252514GA	0.802	0.782	0.782	14
TS252514GAD14	1.605	3.596	1.564	14
TS252514GADWS	1.632	11.165	1.995	14
TS252514GAQ14	1.632	7.586	7.586	14
CS252514GA	0.609	0.694	0.409	14
C351514GA	0.32	0.117	0.317	14
C401014GAD	1.188	1.929	4.098	14

VII. Maximum Deflections:

Xmax	$\frac{l_y}{120}$	Ymax	$\frac{l_x}{180}$	Zmax	$\frac{l_y}{120}$
0.27	1.50 OK	0.79	2.67 OK	0.46	1.50 OK

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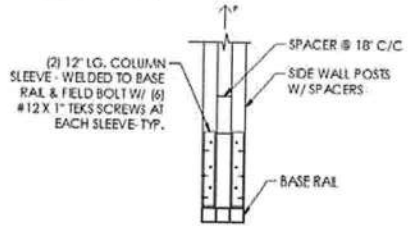
VIII. Connection Calculations:

Note: See ESR2196 for Hilti Kwik-Pro (equivalent to ITW BuildexTEKS screws) specifications

1. Frame Column – Base Connection:



A. Forces:

Uplift force in column: $P = \mathbf{2.79}$ Kips
Max. Shear force $\mathbf{V = 0.93}$ Kips
Combined Uplift + Shear: $F = (P^2 + V^2)^{1/2} = \mathbf{2.94}$ Kips

B. Dimensions & Properties:

a. Column:

Tube side: $s_c = \mathbf{2.5}$ in
Tube thickness: $t_c = \mathbf{0.083}$ in

b. Base rail:

Tube side: $s_b = \mathbf{2.5}$ in
Tube thickness: $t_b = \mathbf{0.083}$ in

c. Connector Sleeve:

Tube side: $s_s = \mathbf{2.25}$ in
Tube thickness: $t_s = \mathbf{0.083}$ in
Length: $l_s = \mathbf{12}$ in

d. Fasteners:

Fastener type: **HILTI Kwik Pro**
Nominal diameter: **#12**
Min. number of fasteners: $n_f = 4$

e. Weld:

Weld size: $t_w = \mathbf{0.125}$ in
Weld tensile strength: $F_{uw} = \mathbf{60}$ ksi

C. Component checks:

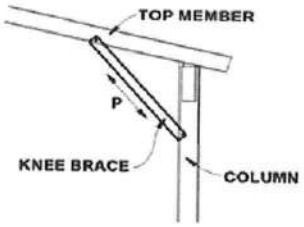
a. Weld:

Total length of weld:
$\rightarrow l_w = 2 \times 2.5 = \mathbf{5}$ in
Nominal shear strength: $P_{nw} = \mathbf{t\, l_w\, F_{uw}}$ (AISI E2.4-3)
$\rightarrow P_{nw} = 0.125 \times 5 \times 60 = \mathbf{37.5}$ Kips
Allowable shear strength: $P_{aw} = \mathbf{P_{nw} / \Omega}$
$\rightarrow P_{aw} = 37.5 / 2.35 = \mathbf{15.96}$ Kips > $\mathbf{P = 2.79}$ Kips **OK**
$\rightarrow P_{aw} = 37.5 / 2.35 = \mathbf{15.96}$ Kips > $\mathbf{V = 0.93}$ Kips **OK**
$\rightarrow P_{aw} = 37.5 / 2.35 = \mathbf{15.96}$ Kips > $\mathbf{F = 2.94}$ Kips **OK**

b. Fasteners:

Allowable shear strength: $V_{af} = \mathbf{0.76}$ Kips (ESR-2196)
Total allowable shear strength: $V_{aft} = n_f\, V_{af}$
$\rightarrow V_{aft} = 4 \times 0.76 = \mathbf{3.04}$ Kips > $\mathbf{P = 2.94}$ Kips **OK**

2. Knee Brace Connection:(Main Bldg)



A. Forces:

Max. force in double knee brace: $\mathbf{2P = 3.835}$ Kips
Max. force in single knee brace: $\mathbf{P = 1.918}$ Kips

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B. Dimensions & Properties:

a. Knee brace:

Channel flanges: s_c = **2.5** in
Channel wall thickness: t_c = **0.083** in

b. Top member:

Tube side: s_b = **2.5** in
Tube thickness: t_b = **0.083** in

c. Fasteners:

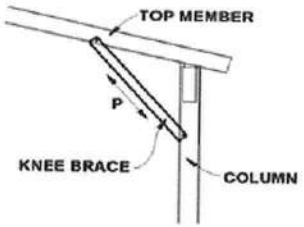
Fastener type: **HILTI Kwik Pro**
Nominal diameter: **#12**
Min. number of fasteners on end: n_f = **3**

C. Component checks:

a. Fasteners in shear:

Allowable shear strength: V_{af} = **0.76** Kips (ESR-2196)
Total allowable shear strength: $V_{aft} = n_f V_{af}$
→ V_{aft} = **3** × 0.76 = **2.28** Kips > **1.918** Kips **OK**

3. Knee Brace Connection:(Lean To)



A. Forces:

Max. force in single knee brace: **P** = **0.704** Kips

B. Dimensions & Properties:

a. Knee brace:

Channel flanges: s_c = **2.5** in
Channel wall thickness: t_c = **0.083** in

b. Top member:

Tube side: s_b = **2.5** in
Tube thickness: t_b = **0.083** in

c. Fasteners:

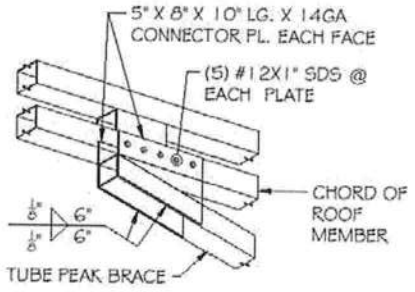
Fastener type: **HILTI Kwik Pro**
Nominal diameter: **#12**
Min. number of fasteners on end: n_f = **2**

C. Component checks:

a. Fasteners in shear:

Allowable shear strength: V_{af} = **0.76** Kips (ESR-2196)
Total allowable shear strength: $V_{aft} = n_f V_{af}$
→ V_{aft} = 2 × 0.76 = **1.52** Kips > **0.704** Kips **OK**

4. Peak Brace Connection:



A. Forces:

Max. force in peak brace: P = **3.251** Kips

B. Dimensions & Properties:

a. Peak brace member:

Tube side: s_c = **2.5** in
Tube thickness: t_c = **0.083** in

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b. Top member:

Tube side: s_b = **2.5** in
Tube thickness: t_b = **0.083** in

c. Plate at top:

Plate thickness: **14GA**
Length: lp_s = **10** in

d. Fasteners:

Fastener type: **HILTI Kwik Pro**
Nominal diameter: **#12**
Number of fasteners at connection: n_{ft} = **5**

C. Component checks:

a. Weld:

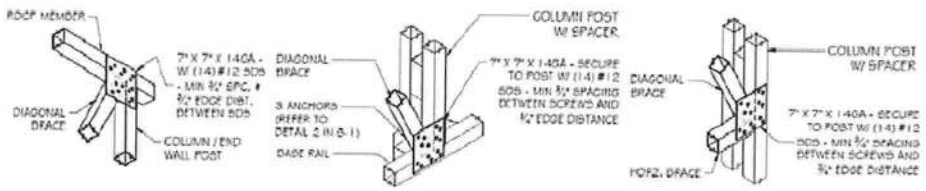
Strength of weld: F_{uw} = **60** ksi
Total length of weld: l_w = **10** in
Nominal shear strength: $P_{nw} = t\, l_w\, F_{uw}$ (AISI E2.4-3)
→ $P_{nw} = 0.083 \times 10 \times 60 =$ **49.8** Kips
Allowable shear strength: $P_{aw} = P_{nw} / \Omega$

→ $P_{aw} = 49.8 / 2.35 =$ **21.19** Kips > **3.251 kips OK**

b. Fasteners:

Allowable shear strength: V_{af} = **0.76** Kips (ESR-2196)
Total allowable shear strength: $V_{aft} = n_f\, V_{af}$
→ $V_{aft} = 5 \times 0.76 =$ **3.8** Kips > **3.251 Kips OK**

5. **Diagonal Bracing Component Checks:**



A. Design Forces:

Max. Axial force in diagonal brace: P = **1.98** Kips

B. Dimensions & Properties:

a. Brace member:

Tube side: s_b = **2.25** in
Tube thickness: t_b = **0.083** in

b. Plate:

Clip thickness: **14GA**
Min. Plate dimension: lp_s = 7 in

c. Fasteners:

Fastener type: **HILTI Kwik Pro**
Nominal diameter: **#12**
Min. number of fasteners at connection: n_{ft} = **4**

C. Component checks:

a. Fasteners :

Allowable shear strength: V_{af} = **0.76** Kips (ESR-2196)
Total allowable shear strength: $V_{aft} = n_f\, V_{af}$
→ $V_{aft} = 4 \times 0.76 =$ **3.04** Kips > **1.98** Kips **OK**

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IX. Foundation Calculations:

1. **Anchor Reactions:**

Total uplift @ Main bldg left Side -wall: $F_{up,1}$ = **17.332** Kips
Total uplift @ End -wall: $F_{up,2}$ = **6.094** Kips
Total uplift @ Main bldg right Side -wall: $F_{up,3}$ = **10.476** Kips
Total uplift @ Lean-to Side -wall: $F_{up,4}$ = **3.385** Kips
Max downwards reaction: F_{dn} = **7.79** Kips
Max. lateral reaction: F_{lt} = **0.93** Kips
Total lateral @ Main bldg left Side -wall: $F_{lt,1}$ = **4.872** Kips
Total lateral @ End -wall: $F_{lt,2}$ = **2.921** Kips
Total lateral @ Main bldg right Side -wall: $F_{lt,3}$ = **2.789** Kips
Total lateral @ Lean-to Side -wall: $F_{lt,4}$ = **0.4** Kip

2. **Anchor Check:**

Anchor type:**POWER-STUD+ SD1 EXPANSION ANCHOR (References: ESR-2818)**
Anchor diameter: φ = **1/2** in
Effective embedment: h_{ef} = **3.25** in
Min. edge distance provided: c = **4** in
Nominal concrete strength: f'_c = **2500** psi

A. Uplift Resistance:

a. Steel strength in tension: φN_{sa} = (0.75) (9080)
→ φN_{sa} = **6.810 Kips**
Concrete break out strength in tension:

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$$

where
$A_{Nco} = 9\, h_{ef}^2 = 9 \times 3.25^2 = 190.13$ in^2
$A_{Nc} = (c + 1.5\, h_{ef})^2 = (4 + 1.5 \times 3.25)^2 = 95.06$ in^2
$\psi_{ed,N} = 0.7 + 0.3\, c / (1.5\, h_{ef}) = 0.7 + 0.3 \times 4 / (1.5 \times 3.25)$
→ $\psi_{ed,N}$ = **0.95**
$\psi_{c,N}$ = **1.00**
$\psi_{cp,N}$ = Min (**1.5 h_{ef} / c_{ac}, 1.0**)
→ $\psi_{cp,N}$ = Min (1.5 × **3.25 / 8** = (0.61, 1.00) = **0.61**
k_c = **17**

$$N_b = k_c \sqrt{f'_c}\, h_{ef}^{1.5} = 17 \times \sqrt{2500} \times 3.25^{1.5} = \mathbf{4980.17}\ \text{lbs}$$

→ φN_{cb} = 0.75 × 95.06 / 190.13 × 0.95 × 1.00 × 0.61 × 4.980
→ φN_{cb} = **2.154** Kips

b. Concrete pull out strength in tension: $\varphi N_{p,cr}$ (Does not control per ESR Table -2)

c. Controlling resistance in tension:
φN_n = min |φN_{sa}, φN_{cb}, $\varphi N_{p,cr}$|
→ φN_n = **2.15** Kips

d. Adjust for ASD:
Controlling combination: 0.6D + 1.6W
Adjustment factor α = 0.6 (0%) + 1.6(100%)
→ α = **1.6**
Total. no. of Anchors at Main bldg left Side -wall: **n = 32**
Allowable tension: $T_{ASD} = n \times \varphi N_n / 1.6$
→ T_{ASD} = **32** × 2.15 / 1.6 = **43.07** Kips > $F_{up,1}$ = **17.332** Kips **OK**

Total no. of Anchors at End -wall: **n = 8**
Allowable tension: $T_{ASD} = n \times \varphi N_n / 1.6$
→ T_{ASD} = **8** × 2.15 / 1.6 = **10.77** Kips > $F_{up,2}$ = **6.094** Kips **OK**

Total. no. of Anchors at Main bldg right Side -wall: **n = 32**
Allowable tension: $T_{ASD} = n \times \varphi N_n / 1.6$
→ T_{ASD} = **32** × 2.15 / 1.6 = **43.07** Kips > $F_{up,3}$ = **10.476** Kips **OK**

Total. no. of Anchors at Lean to Side -wall: **n = 17**
Allowable tension: $T_{ASD} = n \times \varphi N_n / 1.6$
→ T_{ASD} = **17** × 2.15 / 1.6 = **22.88** Kips > $F_{up,4}$ = **3.385** Kips **OK**

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B. Shear Resistance:

a. Steel strength in shear: $\varphi V_{sa} = (0.65)\ (4620)$

→ $\varphi V_{sa} =$ **3.00** Kips

b. Concrete break out strength in shear: $V_{cb} = \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} V_b$

where

$A_{Vco} = 4.5\ c^2 = 4.5 \times 4^2 =$ **144.00** in^2

$A_{Vc} = 1.5\ c\ (c + 1.5\ c) = 3.75 \times 4^2 =$ **60.00** in^2

$\psi_{ed,V} = 0.7 + 0.3\ c / (1.5\ c) = 0.7 + 0.3 \times 4 / (1.5 \times 4)$

→ $\psi_{ed,V} =$ **0.90**

$\psi_{c,V} =$ **1.00**

$l_e =$ **3.25**

$d_o =$ **1/2**

$$V_b = 7\left(\frac{l_e}{d_o}\right)^{0.2} \sqrt{d_0} \sqrt{f_c'}\, c^{1.5}$$

$$\rightarrow V_b = 7 \times \left(\frac{3.25}{0.500}\right)^{0.2} \sqrt{0.500}\sqrt{2500} \times 4.00^{1.5}$$

→ $V_b =$ **2878.89** lbs

→ $\varphi V_{cb} = (0.65)\ 60.00 / 144.00 \times 0.90 \times 1.00 \times 2.88 =$ **1.51** Kips

c. Concrete pry out strength in shear: $V_{cp} = k_{cp} N_{cb}$

→ $\varphi V_{cpr} = 0.65 \times 2.871 =$ **4.02** Kips

d. Controlling resistance in shear:

$\varphi V_n = \min |\varphi V_{sa}, \varphi V_{cb}, \varphi V_{cp}|$

→ $\varphi N_n =$ **1.51** Kips > F_{lt} **OK**

e. Adjust for ASD:

Controlling combination: 1.2D + 1.6Lr

Adjustment factor $\alpha = 0.9\ (30\%) + 1.6(70\%)$

→ $\alpha =$ **1.40**

Min. No. of Anchors at each post: **n = 1**

Allowable tension: $V_{ASD} = n * \varphi N_n / \alpha$

→ $V_{ASD} =$ **1** × 1.51 / 1.40 = **1.08** Kips > $F_{lt} =$ **0.93** Kips **OK**

Total. no. of Anchors at Main bldg left Side -wall: **n = 32**

Allowable tension: $V_{ASD} = n * \varphi N_n / \alpha$

→ $V_{ASD} =$ **32** × 1.51 / 1.40 = **34.51** Kips > $F_{lt,1} =$ **4.872** Kips **OK**

Total no. of Anchors at End -wall: **n = 8**

Allowable tension: $V_{ASD} = n * \varphi N_n / \alpha$

→ $V_{ASD} =$ **8** × 1.51 / 1.40 = **8.63** Kips > $F_{lt,2} =$ **2.921** Kips **OK**

Total. no. of Anchors at Main bldg right Side -wall: **n = 32**

Allowable tension: $V_{ASD} = n * \varphi N_n / \alpha$

→ $V_{ASD} =$ **32** × 1.51 / 1.40 = **34.51** Kips > $F_{lt,3} =$ **2.789** Kips **OK**

Total. no. of Anchors at Lean to Side -wall: **n = 17**

Allowable tension: $V_{ASD} = n * \varphi N_n / \alpha$

→ $V_{ASD} =$ **17** × 1.51 / 1.40 = **18.33** Kips > $F_{lt,4} =$ **0.4** Kip **OK**

A&A ENGINEERING
CIVIL • STRUCTURAL
6036 Renaissance Place
Toledo, Ohio 43623
Tel: 419-292-1983
Fax: 419-292-0955

PROJECT: RONNIE MUNSON		PROJECT NO.: 356-22-0558	SHEET NO.: 10/10
CLIENT: REAL STEEL METAL BUILDINGS	LOCATION: FORT WHITE, FL 32038		
CALCULATED BY: A.F.	CHECKED BY: O.A.	APPROVED BY:	SCALE:
DATE: 3/24/2022	DATE: 3/24/2022	DATE:	

3. Concrete Turn Down Footing:

Width of Footing: b_f = **12.00** in
Depth of Footing: d_f = **18.00** in

A. Bearing Resistance:

Length of bearing on top of concrete: l_b = **27.50** in
Approx. bearing area under post: $A_b = b_f (2d_f + l_b)$
→ $A_b = 12.00 \times (2 \times 18.00 + 27.50) =$ **762.00** in^2
Assumed soil bearing capacity: s_b = **1500** psf
Bearing capacity: $F_b = s_b A_b$
→ $F_b = 762.00 / 12^2 \times 1500 / 1000 =$ **7.94** Kips > $F_{dn} =$ **7.79** Kips **OK**

B. Uplift Resistance:

Unit weight of concrete: γ_c = **150** pcf
Length of concrete slab: l_s = **85.00** ft
Width of concrete slab: b_s = **60.00** ft
Total area of slab: $A_s = l_s b_s$
→ $A_{slb} =$ **5100.00** ft^2
Thickness of slab: t_s = **4.00** in
Volume of concrete in slab: $V_s = A_s t_s$
→ $V_s = 5100.00 (4.00 / 12) =$ **1700.00** ft^3

Area of turn down footing:
→ $A_f =$ **500.00** ft^2

Volume of concrete in footing: $V_f = A_f (d_f - t_s)$
→ $V_f = 500.00 * (18.00 - 4.00) / 12 =$ **583.33** ft^3

Total volume of concrete: $V_t = V_s + V_f$
→ $V_t = 1700.00+583.33 =$ **2283.33** ft^3

Total weight of foundation: $W_t = \gamma_c V_t$
→ $W_t = 150 \times 2283.33 / 1000 =$ **342.50** Kips
Moment arm resisting uplift: $M_n = W_t b_s / 2$
→ $M_t = 342.50 \times 60.00 / 2 =$ **10275.00** Kips-ft
Safety factor: SF = **2.00**
Allowable resisting moment: $M_a = M_n / SF$
→ $M_a = 10275.00 / 2.00 =$ **5137.50** Kips-ft

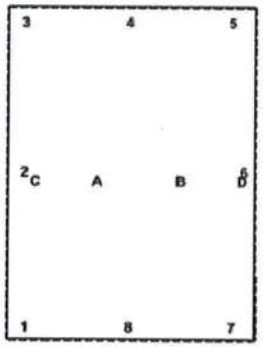
Total uplift in column line at side wall: $F_{up,t} =$ **31.193** Kips
Moment arm causing uplift at side wall:
→ $M_u =$ **900.22** Kips-ft < **5137.50** Kips-ft **OK**

A&A ENGINEERING
CIVIL • STRUCTURAL
SINCE 1996
6036 Renaissance Place
Toledo, Ohio 43623
Tel: (419) 292-1983
Fax: (419) 292-0955

PROJECT:		PROJECT NO.:	SHEET NO.:
CLIENT:	LOCATION:		
CALCULATED BY:	CHECKED BY:	APPROVED BY:	SCALE:
DATE:	DATE:	DATE:	

APPENDICES

Project Number: 356-22-0558
Customer Name: REAL STEEL METAL BUILDINGS
Address: 1382 SW FAULKNER RD, FORT WHITE, FL, 32038
Prepared By: AF



Signature / Stamp

Site Information

Wind Dir.	Exposure
1	C
2	C
3	C
4	C

Basic Wind Speed: 120 mph
Topography: Flat

Optional Factors

This project uses load combinations from ASCE 7.

Structure Information

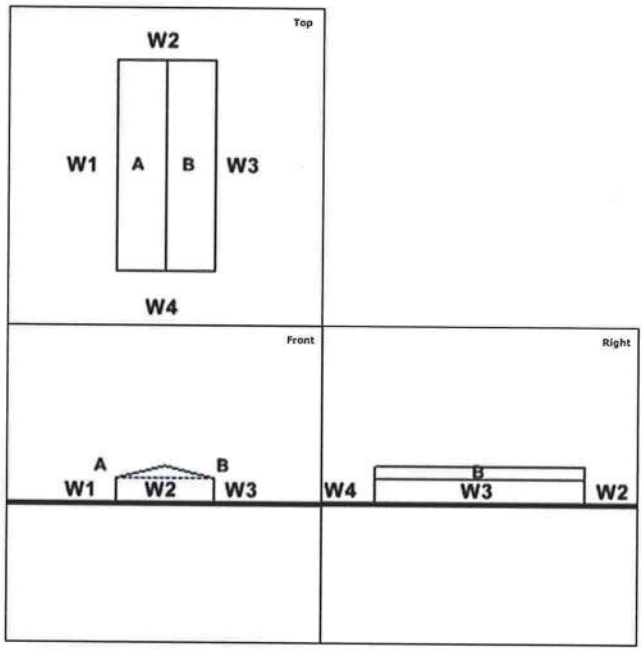
Structure Type: Building
Structure Category: II
Enclosure Classification: Enclosed

Main Section

Wall	Length (ft)	Overhang (ft)
1	85.0	0.00
2	40.0	0.00
3	85.0	0.00
4	40.0	0.00

Eave Height: 10.00 ft
Parapet Height: 0.00 ft
Parapet Enclosure: Solid
Roof Shape: Gabled

Roof	Slope (X:12)
A & B	3.00



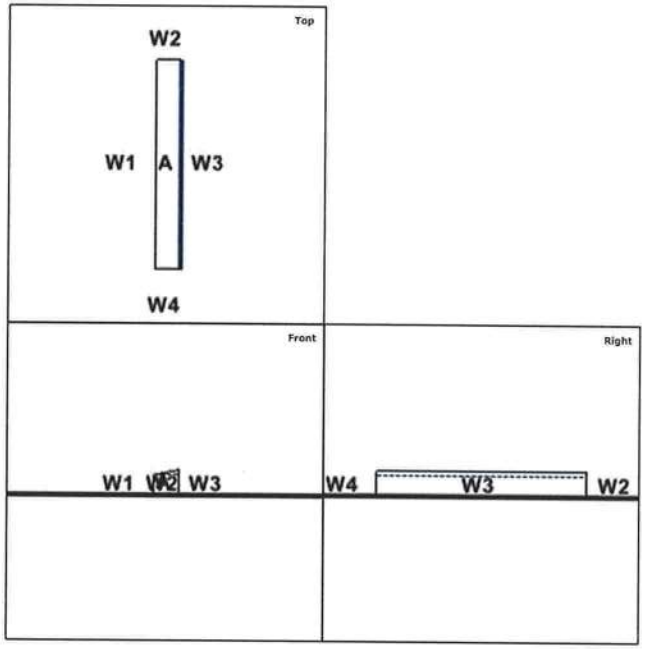
LEFT LEAN

Connected to: Main Section
Connected to Wall: W1
Position on W1: 0.00 ft

Wall	Length (ft)	Overhang (ft)
1	85.0	0.00
2	10.00	0.00
3	85.0	0.00
4	10.00	0.00

Eave Height: 8.00 ft
Parapet Height: 0.00 ft
Parapet Enclosure: Solid
Roof Shape: Monoslope

Roof	Slope (X:12)
A	2.40



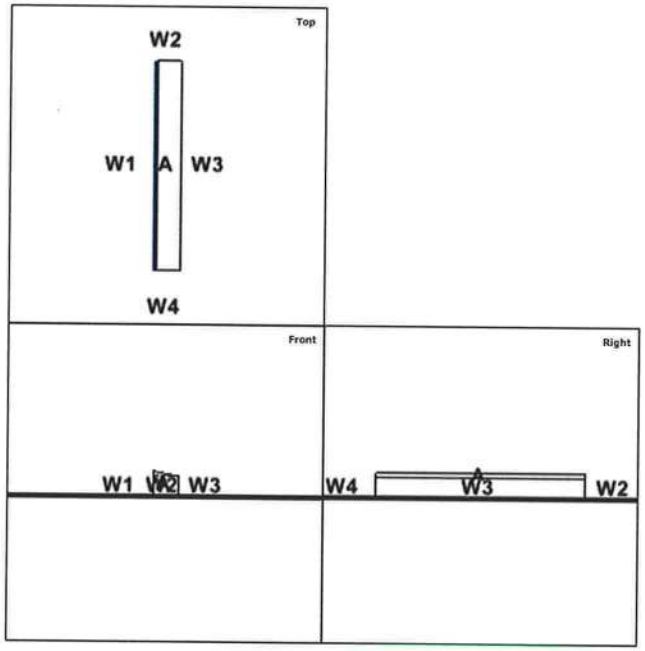
RIGHT LEAN

Connected to:	Main Section
Connected to Wall:	W3
Position on W3:	0.00 ft

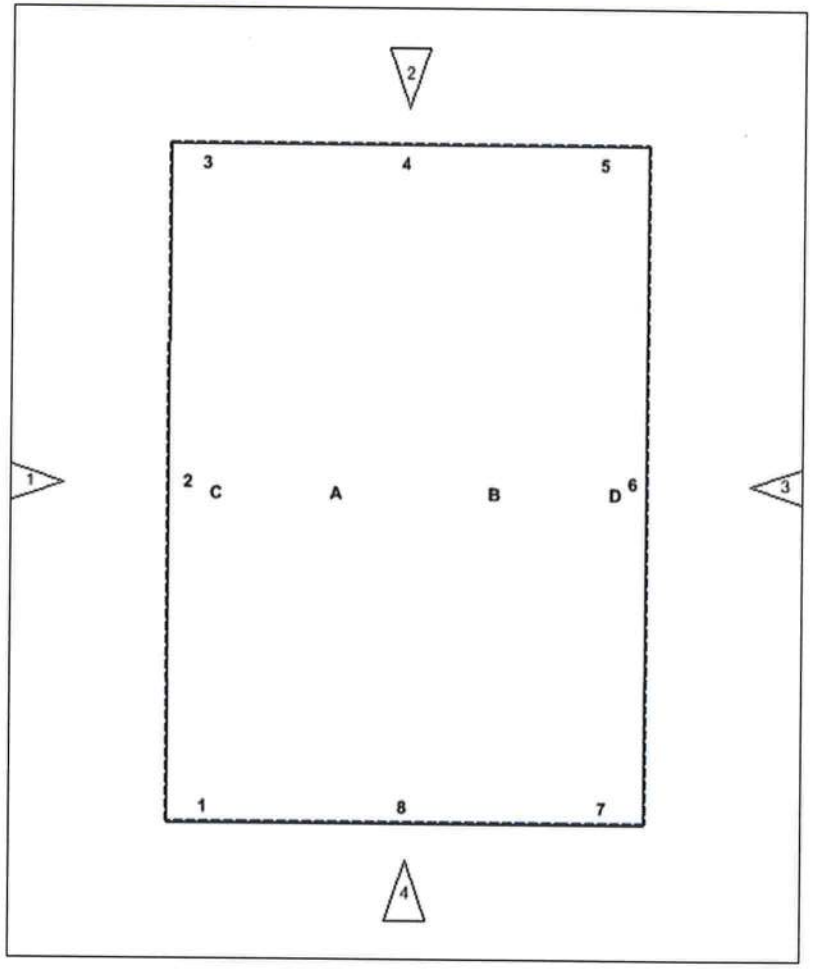
Wall	Length (ft)	Overhang (ft)
1	85.0	0.00
2	10.00	0.00
3	85.0	0.00
4	10.00	0.00

Eave Height:	8.00 ft
Parapet Height:	0.00 ft
Parapet Enclosure:	Solid
Roof Shape:	Monoslope

Roof	Slope (X:12)
A	2.40



Composite Drawing



RONNIE MUNSON

ASCE 7-16 Wind Load Report - Wind Direction 1

March 24, 2022

This data was calculated using the building of all heights method.

#	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
1	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
2	Windward	8.00	26.6	0.870	0.800	0.180	18.5	13.7	23.3
3	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
4	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
5	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
6	Leeward	12.5	26.6	0.870	-0.500	0.180	-11.6	-16.3	-6.78
7	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
8	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
A	Windward Roof	12.5	26.6	0.870	-0.071	0.180	-1.64	-6.43	3.14
		12.5	26.6		-0.589		-13.6	-18.4	-8.82
B	Leeward Roof	12.5	26.6	0.870	-0.471	0.180	-10.9	-15.7	-6.11
C	Windward Roof	12.5	26.6	0.870	-0.180	0.180	-4.16	-8.95	0.623
		12.5	26.6		-1.05		-24.2	-29.0	-19.4
D	Leeward Roof	12.5	26.6	0.870	-0.674	0.180	-15.6	-20.4	-10.8

This is load case 1 in ASCE 7-16 Figure 27.3-8. See Figure 27.3-8 for other cases.

RONNIE MUNSON

ASCE 7-16 Wind Load Report - Wind Direction 2

March 24, 2022

This data was calculated using the building of all heights method.

#	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
1	Leeward	12.5	26.6	0.878	-0.417	0.180	-9.73	-14.5	-4.94
2	Side	12.5	26.6	0.878	-0.700	0.180	-16.3	-21.1	-11.6
3	Windward	9.00	26.6	0.878	0.800	0.180	18.7	13.9	23.5
		10.00	26.6				18.7	13.9	23.5
4	Windward	9.00	26.6	0.878	0.800	0.180	18.7	13.9	23.5
		10.00	26.6				18.7	13.9	23.5
		12.5	26.6				18.7	13.9	23.5
		15.0	26.6				18.7	13.9	23.5
5	Windward	9.00	26.6	0.878	0.800	0.180	18.7	13.9	23.5
		10.00	26.6				18.7	13.9	23.5
6	Side	12.5	26.6	0.878	-0.700	0.180	-16.3	-21.1	-11.6
7	Leeward	12.5	26.6	0.878	-0.417	0.180	-9.73	-14.5	-4.94
8	Leeward	12.5	26.6	0.878	-0.417	0.180	-9.73	-14.5	-4.94
A&B	Roof	0.00 - 6.25*	26.6	0.878	-0.900	0.180	-21.0	-25.8	-16.2
		6.25 - 12.5*	26.6				-21.0	-25.8	-16.2
		12.5 - 25.0*	26.6		-0.500		-11.7	-16.5	-6.89
		25.0 - 85.0*	26.6		-0.300		-7.00	-11.8	-2.22
		0.00 - 85.0*	26.6		-0.180		-4.20	-8.99	0.584
C	Roof	0.00 - 6.25*	26.6	0.878	-0.900	0.180	-21.0	-25.8	-16.2
		6.25 - 12.5*	26.6				-21.0	-25.8	-16.2
		12.5 - 25.0*	26.6		-0.500		-11.7	-16.5	-6.89
		25.0 - 85.0*	26.6		-0.300		-7.00	-11.8	-2.22
		0.00 - 85.0*	26.6		-0.180		-4.20	-8.99	0.584
D	Roof	0.00 - 6.25*	26.6	0.878	-0.900	0.180	-21.0	-25.8	-16.2
		6.25 - 12.5*	26.6				-21.0	-25.8	-16.2
		12.5 - 25.0*	26.6		-0.500		-11.7	-16.5	-6.89
		25.0 - 85.0*	26.6		-0.300		-7.00	-11.8	-2.22
		0.00 - 85.0*	26.6		-0.180		-4.20	-8.99	0.584

This is load case 1 in ASCE 7-16 Figure 27.3-8. See Figure 27.3-8 for other cases.
* Distance from windward edge.

RONNIE MUNSON

ASCE 7-16 Wind Load Report - Wind Direction 3

March 24, 2022

This data was calculated using the building of all heights method.

#	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
1	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
2	Leeward	12.5	26.6	0.870	-0.500	0.180	-11.6	-16.3	-6.78
3	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
4	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
5	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
6	Windward	8.00	26.6	0.870	0.800	0.180	18.5	13.7	23.3
7	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
8	Side	12.5	26.6	0.870	-0.700	0.180	-16.2	-21.0	-11.4
A	Leeward Roof	12.5	26.6	0.870	-0.471	0.180	-10.9	-15.7	-6.11
B	Windward Roof	12.5	26.6	0.870	-0.071	0.180	-1.64	-6.43	3.14
		12.5	26.6		-0.589		-13.6	-18.4	-8.82
C	Leeward Roof	12.5	26.6	0.870	-0.674	0.180	-15.6	-20.4	-10.8
D	Windward Roof	12.5	26.6	0.870	-0.180	0.180	-4.16	-8.95	0.623
		12.5	26.6		-1.05		-24.2	-29.0	-19.4

This is load case 1 in ASCE 7-16 Figure 27.3-8. See Figure 27.3-8 for other cases.

RONNIE MUNSON

ASCE 7-16 Wind Load Report - Wind Direction 4

March 24, 2022

This data was calculated using the building of all heights method.

#	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
1	Windward	9.00	26.6	0.878	0.800	0.180	18.7	13.9	23.5
		10.00	26.6				18.7	13.9	23.5
2	Side	12.5	26.6	0.878	-0.700	0.180	-16.3	-21.1	-11.6
3	Leeward	12.5	26.6	0.878	-0.417	0.180	-9.73	-14.5	-4.94
4	Leeward	12.5	26.6	0.878	-0.417	0.180	-9.73	-14.5	-4.94
5	Leeward	12.5	26.6	0.878	-0.417	0.180	-9.73	-14.5	-4.94
6	Side	12.5	26.6	0.878	-0.700	0.180	-16.3	-21.1	-11.6
7	Windward	9.00	26.6	0.878	0.800	0.180	18.7	13.9	23.5
		10.00	26.6				18.7	13.9	23.5
8	Windward	9.00	26.6	0.878	0.800	0.180	18.7	13.9	23.5
		10.00	26.6				18.7	13.9	23.5
		12.5	26.6				18.7	13.9	23.5
		15.0	26.6				18.7	13.9	23.5
A&B	Roof	0.00 - 6.25*	26.6	0.878	-0.900	0.180	-21.0	-25.8	-16.2
		6.25 - 12.5*	26.6				-21.0	-25.8	-16.2
		12.5 - 25.0*	26.6		-0.500		-11.7	-16.5	-6.89
		25.0 - 85.0*	26.6		-0.300		-7.00	-11.8	-2.22
		0.00 - 85.0*	26.6		-0.180		-4.20	-8.99	0.584
C	Roof	0.00 - 6.25*	26.6	0.878	-0.900	0.180	-21.0	-25.8	-16.2
		6.25 - 12.5*	26.6				-21.0	-25.8	-16.2
		12.5 - 25.0*	26.6		-0.500		-11.7	-16.5	-6.89
		25.0 - 85.0*	26.6		-0.300		-7.00	-11.8	-2.22
		0.00 - 85.0*	26.6		-0.180		-4.20	-8.99	0.584
D	Roof	0.00 - 6.25*	26.6	0.878	-0.900	0.180	-21.0	-25.8	-16.2
		6.25 - 12.5*	26.6				-21.0	-25.8	-16.2
		12.5 - 25.0*	26.6		-0.500		-11.7	-16.5	-6.89
		25.0 - 85.0*	26.6		-0.300		-7.00	-11.8	-2.22
		0.00 - 85.0*	26.6		-0.180		-4.20	-8.99	0.584

This is load case 1 in ASCE 7-16 Figure 27.3-8. See Figure 27.3-8 for other cases.
* Distance from windward edge.

Job Information

	Engineer	Checked	Approved
Name:	AF	OA	
Date:	24-Mar-22	24-Mar-22	

Project ID	
Project Name	

Structure Type	SPACE FRAME

Number of Nodes	1108	Highest Node	1166
Number of Elements	2017	Highest Beam	3533
Number of Plates	613	Highest Plate	4412

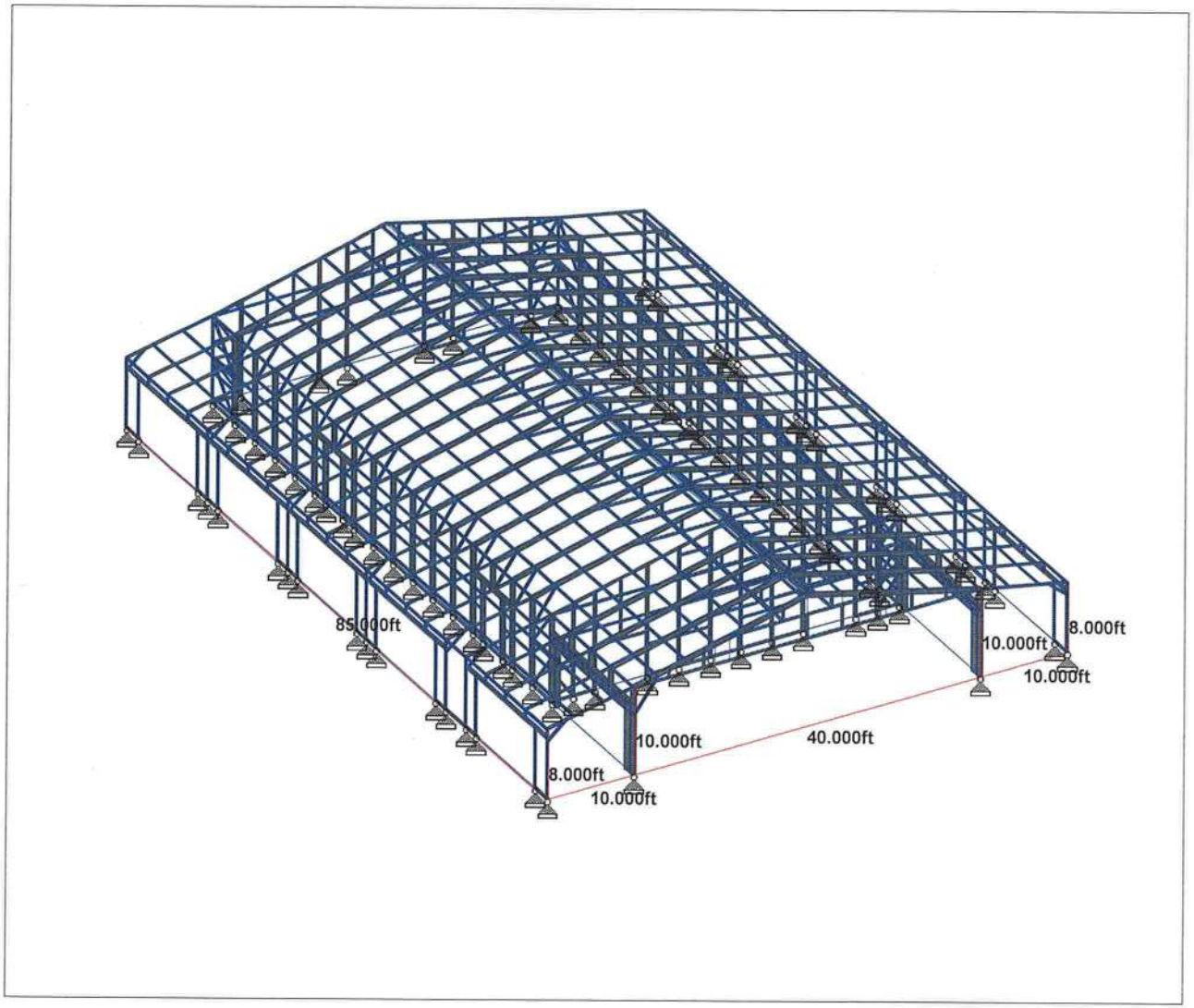
Number of Basic Load Cases	-2
Number of Combination Load Cases	25

Included in this printout are data for:

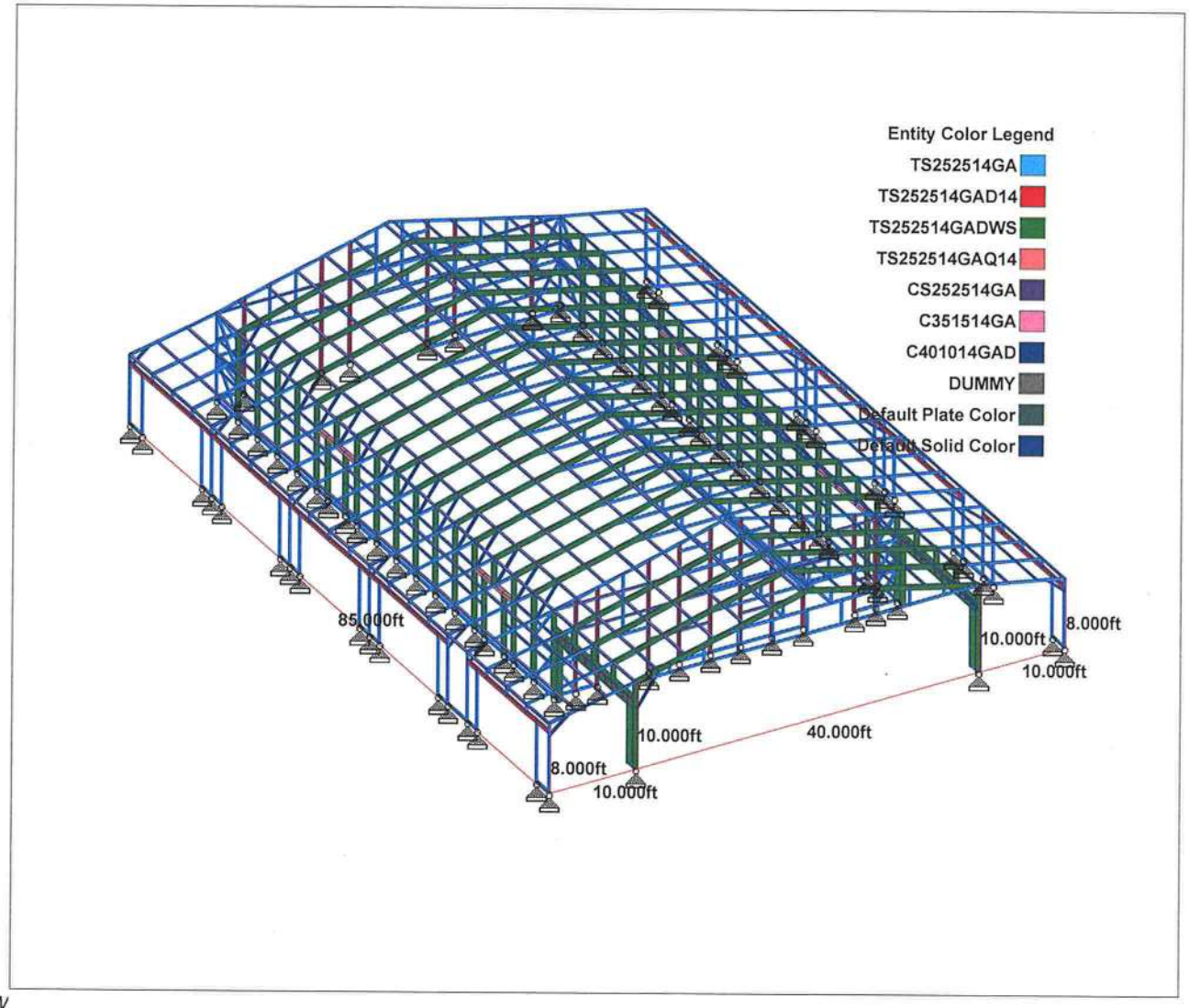
View	MAIN FRAME

Included in this printout are results for load cases:

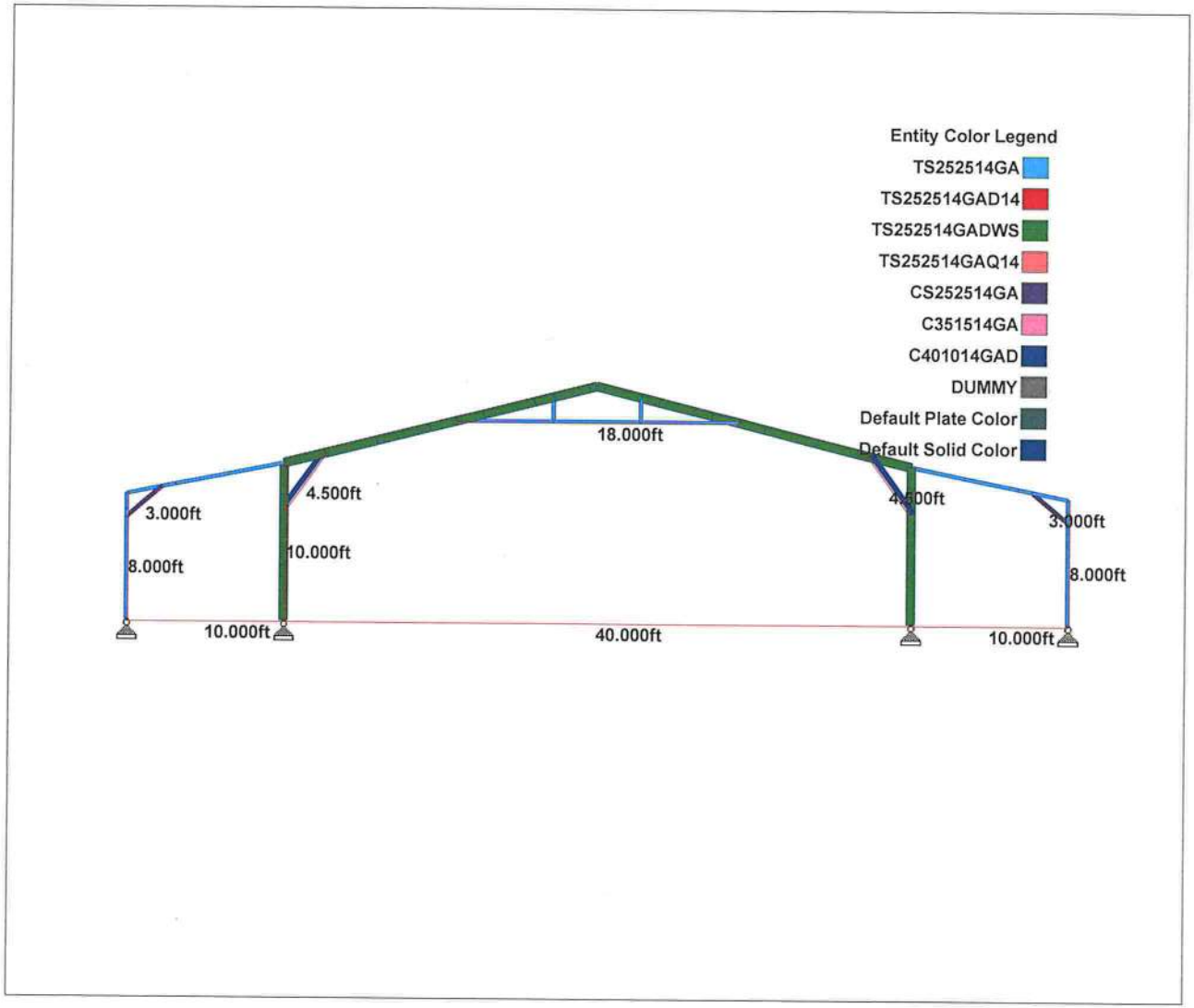
Type	L/C	Name
Combination	11	D+LR
Combination	12	D+0.6WXP
Combination	13	D+0.6WXN
Combination	14	D+0.6WZP
Combination	15	D+0.6WZN
Combination	16	D+0.75LR+0.45WXP
Combination	17	D+0.75LR+0.45WXN
Combination	18	D+0.75LR+0.45WZP
Combination	19	D+0.75LR+0.45WZN



WHOLE STRUCTURE



MAIN FRAME OVERVIEW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TS252514GAD14
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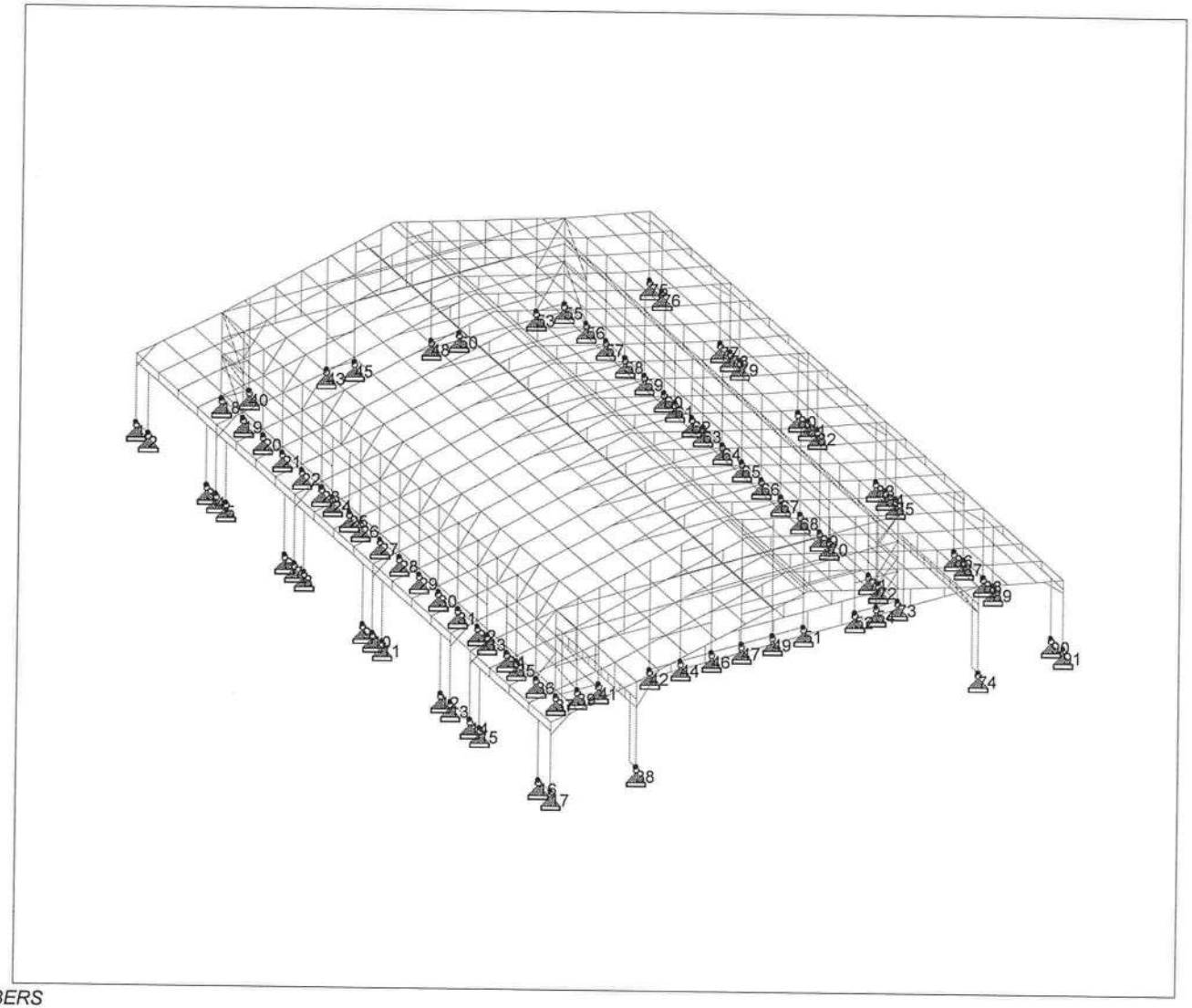
CS252514GA
C351514GA
C401014GAD
DUMMY
Default Plate Color
Default Solid Color
18.000ft
4.500ft
4.500ft
3.000ft
3.000ft
8.000ft
10.000ft
8.000ft
10.000ft
40.000ft
10.000ft

MAIN FRAME



SUPPORTS WITH NUMBERS