Finite Element Analysis
of a
10’ x 22’ FRP Building
for
TRACOM Corporation

May 29, 2007

A. Background

TRACOM Corporation designs and constructs Fiber Reinforced Plastic Buildings for use as shelters around the world, frequently in very harsh environments. This study examines a proposed 10 foot wide by 22 foot long by 9' - 2" high FRP building with a 6’ x 8’ door centered on the gabled end. The building is fabricated of two layers of 1/8” laminated fiberglass over a 2” Styrofoam (Elfoam T250^{TM}) core, similar to a surfboard. Design forces were 135 mph wind load and a 30 pound per square foot snow load, analyzed using the Finite Element Modeling capabilities of RISA-3D.

B. FEM Model

The building was divided into 12,000+ 3” x 3” quadrilateral plates, with the local “z” axis always pointing away from the building as shown below in Figure 1.

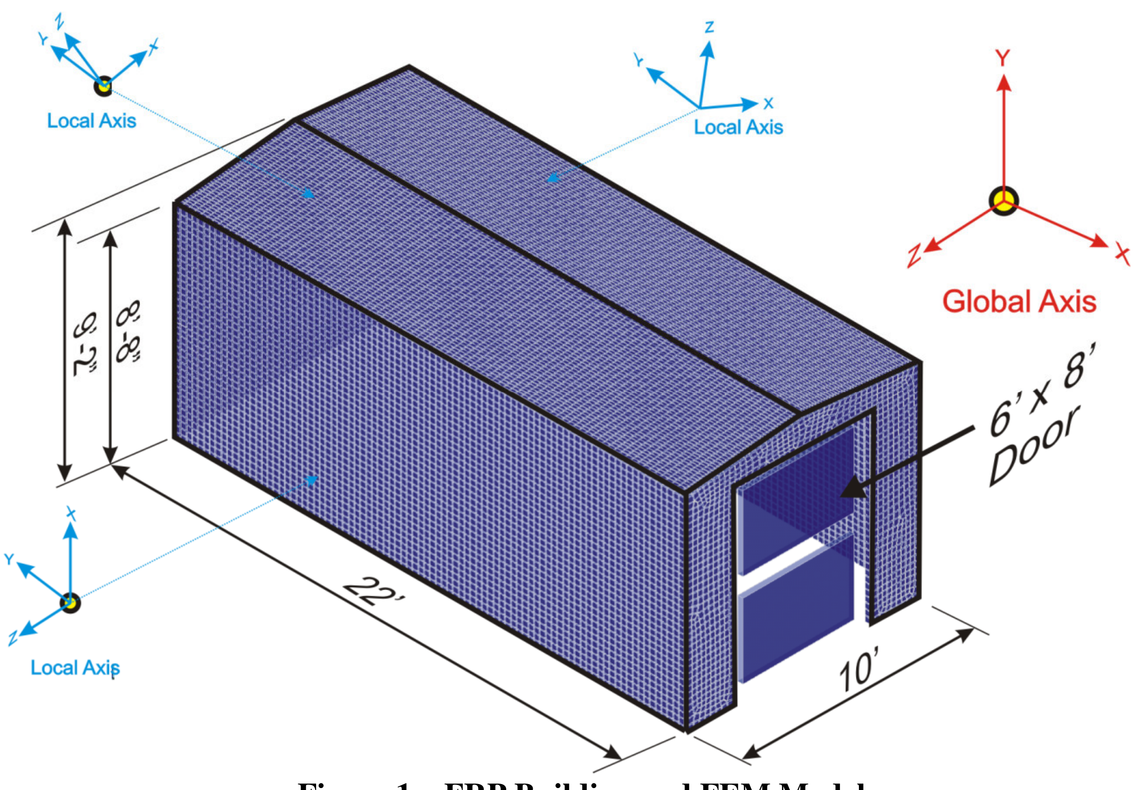


Figure 1 – FRP Building and FEM Model

RISA can only model solid fem plates, and a transformation was made from the 2¼" sandwich panel to a solid fiberglass plate 1.53" thick as shown in Figure 2. Likewise, the door jamb, a laminated 2" x 4", was modeled as a 1.05" thick by 3.15" solid plate.

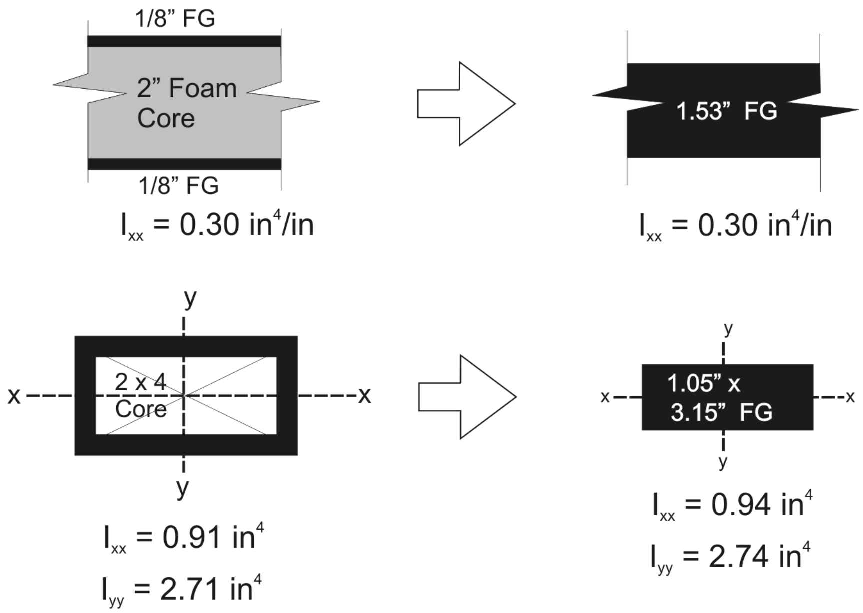


Figure 2 – FRP Panel and FEM Model

Shear deformation is part of the RISA analysis and again a transformation of the Shear Modulus, G, and Poisson's ratio were required. For isotropic materials, values of Poisson's ratio (**μ**) of 0.15 to 0.25 are typically used. But **μ** is highly variable in layered laminates and is dependent on lay-up ply angle and the interior core material. Studies have shown **μ** can range from -60 to 100 *(Larry Peel, 2005)*. For this study, a value of 0.5 was used. Young's modulus was set at 1,900,000 psi.

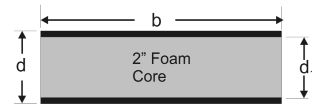
For reaction support, the base of the building was fixed at nodes 24" on center corresponding to the ½" anchor bolts embedded in the concrete slab. Remaining base nodes along the intermediate lip between the bolts were fixed against rotation.

This structural study is a strength analysis. The loads and forces computed from the FEM model were compared with allowable working forces for the fiberglass panel and anchor bolts:

Bending in the fiberglass panel;
Horizontal shear in the 2 inch foam core;
Buckling of the side wall;
Forces on the anchor bolts;
Overturning and sliding.

Ignoring the foam core, the section modulus for two fiberglass rectangles, each 1/8" x 12" and 2" apart is:

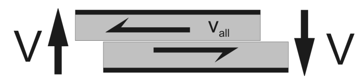
$d := 2.25 \quad d_1 := 2 \quad b := 12 \quad t_{fg} := 0.125 \quad A_{fg} := 2 \cdot t_{fg} \cdot b$



$$S_x := \frac{b \cdot \left(d^3 - d_1^{\,3}\right)}{6 \cdot d} \qquad S_x = 3.01 \ \frac{in^3}{lf} \qquad A_{fg} = 3 \ in^2$$

Testing of TRACOM fiberglass ASTM coupons indicated ultimate bending and tensile stresses of 29.8 ksi. Assigning a 2.5 safety factor, the allowable moment in a one foot wide FG panel is computed as 3.0 ft-kips/foot:

$$F_{ult} := 29800 \qquad SF := 2.5 \qquad f_{all} := \frac{F_{ult}}{SF} \qquad f_{all} = 11920 \ \text{psi}$$



$$M_{all} := f_{all} \cdot S_x \qquad M_{all} = 35926 \ \text{in-pound/ft} \qquad \frac{M_{all}}{12} = 2994 \quad \text{ft-\#/ft}$$

The shear capacity of the FG panel relies on the strength of the foam core. Per laboratory tests, allowable horizontal shear through Elfoam T250 was established at 25 psi. Allowable shear in a one foot fiberglass panel is calculated at 450 pounds/foot:

$$v_{all} := 25 \ \text{psi} \qquad A := d \cdot b \ \ in^2 \qquad A = 27 \ \text{sq. in}$$

$$V_{all} := \frac{2 \cdot v_{all} \cdot A}{3} \qquad V_{all} = 450 \ \frac{lbs}{ft}$$

Allowable buckling load on a 12" wide by 96" high section of the side wall is computed as 1.1 kips and axial loads 22.4 kips:

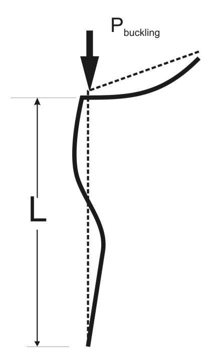
$L := 104$ in $\qquad k := 0.65$ effective column length

$E := 1900000$ psi = modulus of elasticity

$$r := \sqrt{\frac{d^3 - d_1^{\,3}}{12\left(d - d_1\right)}} \qquad r = 1 \qquad \text{r = radius of gyration}$$

$$F_{cr} := \frac{\pi \cdot E}{\left(\frac{k \cdot L}{r}\right)^2} \qquad F_{cr} = 1476 \ \text{psi} \qquad A := 0.25 \cdot 12 \qquad SF := 4$$

$A = 3$ sq. in



$$P_{buckling} := \frac{F_{cr} \cdot A}{SF} \qquad P_{buckling} = 1107 \ \text{pounds}$$

$$f_{all} := \frac{F_{ult}}{SF} \qquad P_{axial} := f_{all} \cdot A_{fg} \qquad P_{axial} = 22350 \ \text{lbs. allowable}$$

C. Loading

Dead load was assigned to the fem plates at 15 psf. A snow load of 40 psf was applied, equivalent to 5 feet of freshly fallen snow. Wind loads of 30 psf (135 mph) were applied. Three load cases, as shown in Figure 3, were modeled:

Case 1 Dead Load + Snow Load
Case 2 Dead Load + Wind Loads onto long face.
Case 3 Dead Load + Wind Loads approaching at 45° angle

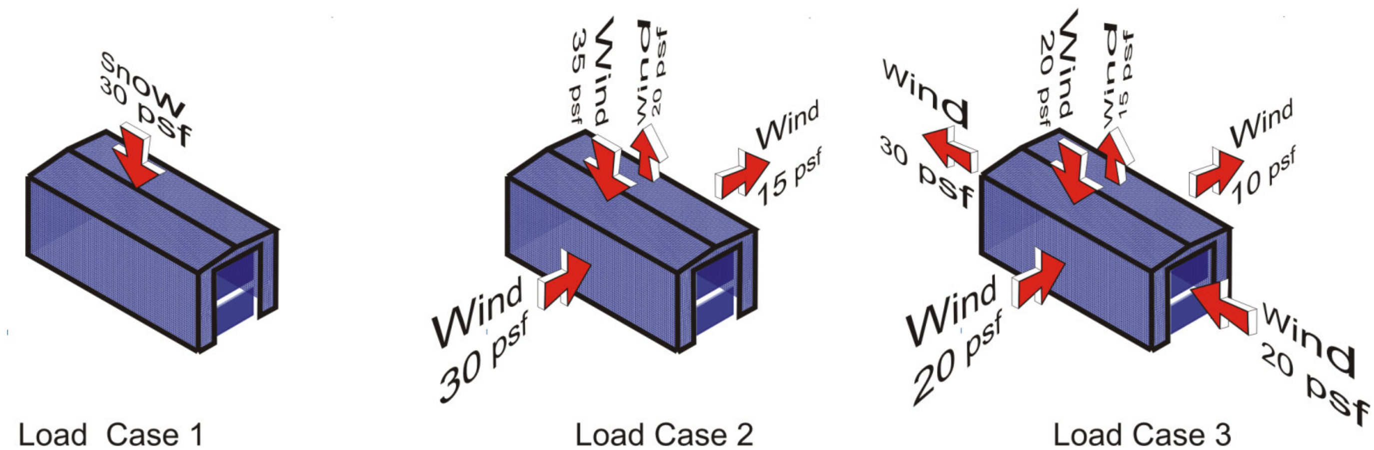


Figure 3 – Loading Cases

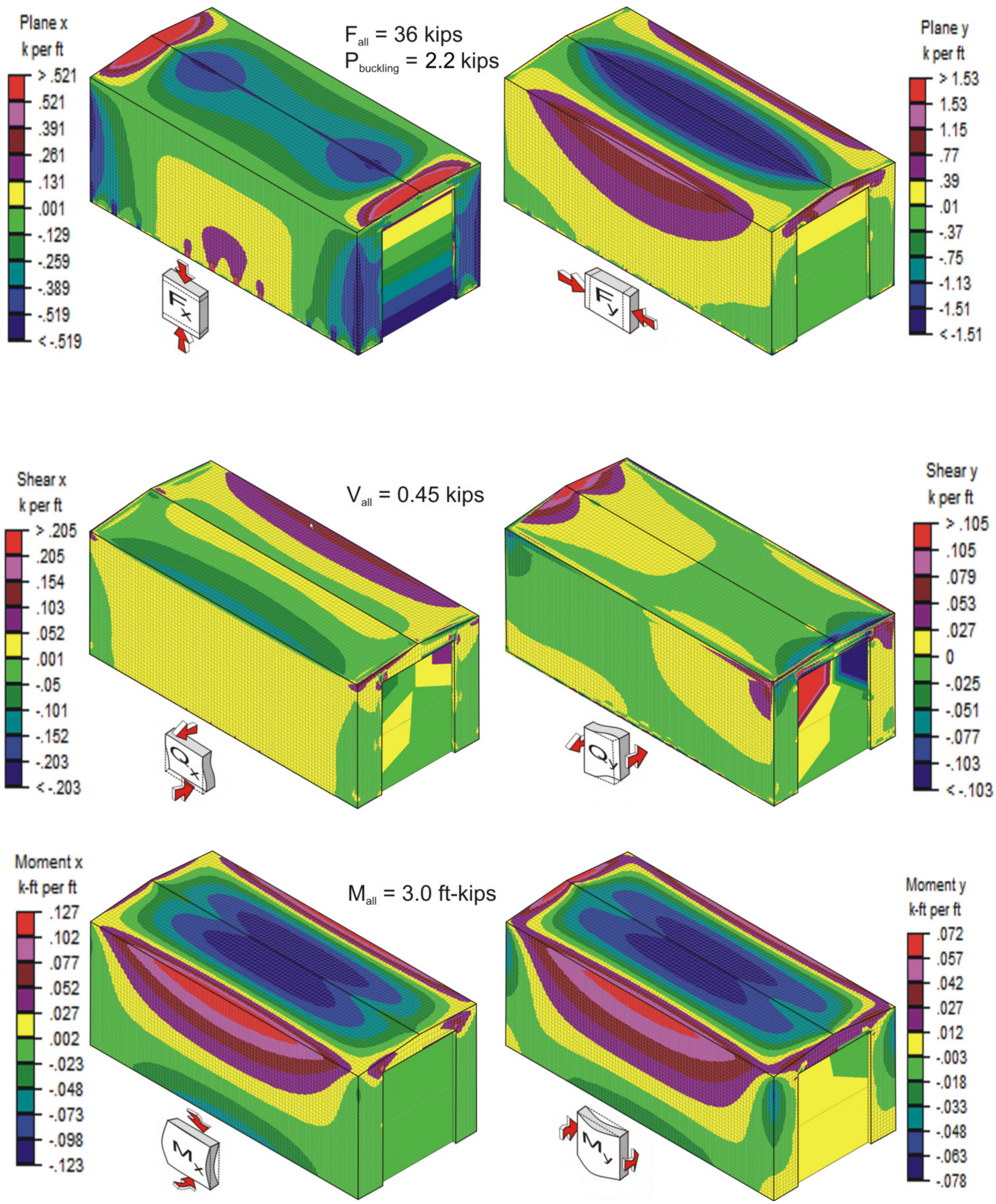
D. Results

The expected forces from this analysis are presented graphically in Figures 4, 5 and 6 as colored force contours in the local coordinate system. Deflection diagrams are shown in Figure 7 in the global coordinate system at a 100:1 exaggeration. A summary table of the maximum forces for each load case is presented in Table 1.

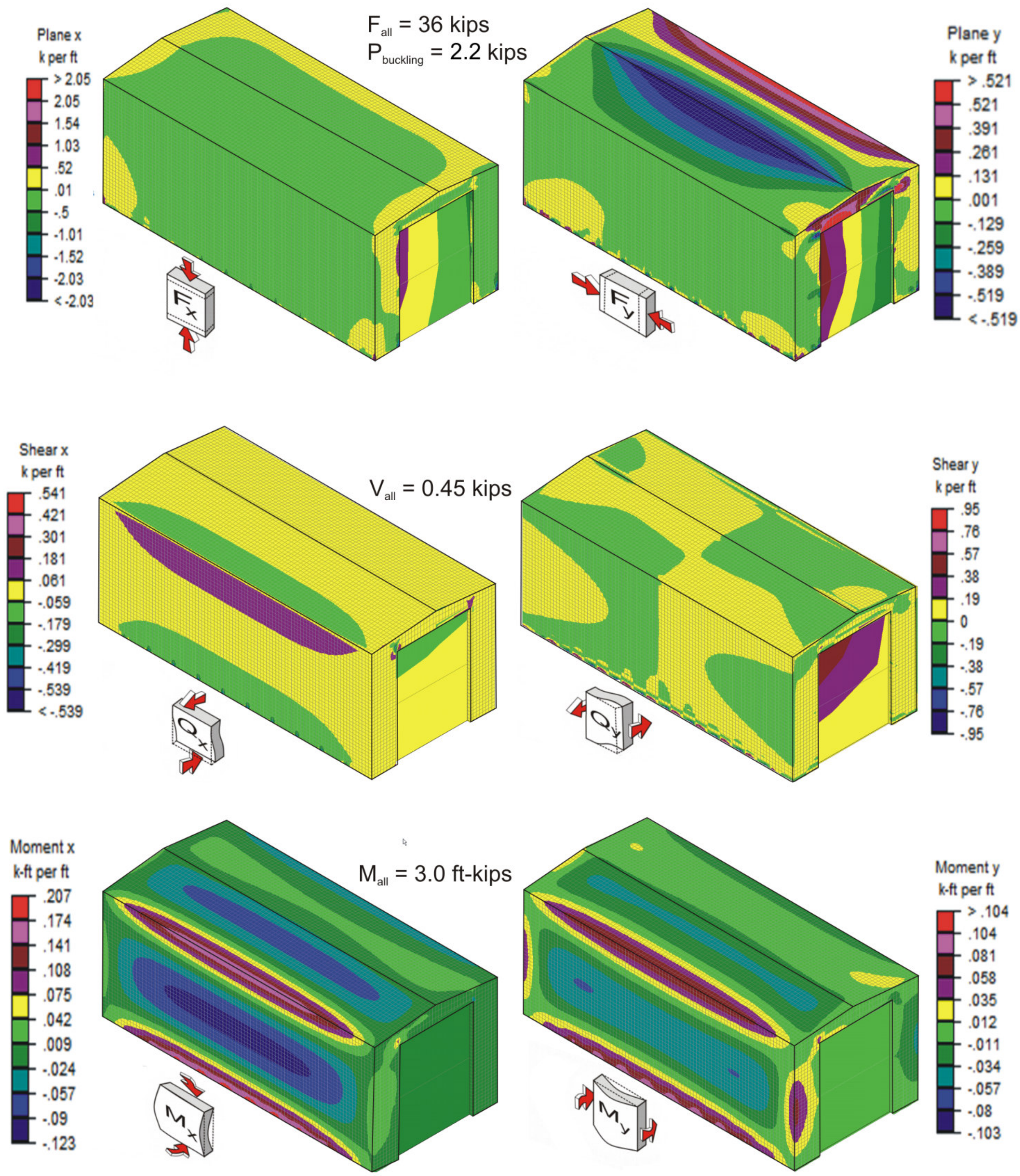
Load Case 1 – Dead Load plus Snow Load Plate forces were minor, less than 30% of the allowable. The deflected shape indicated small movements only, with the center of the roof ridge moving down 0.21". Anchor bolt loads were well below allowable.

Load Case 2 – Dead Load plus Lateral Wind Load Panel moments were acceptable. However along the long wall faces, shear stress reached 75% allowable and the center of this side wall deflected almost 0.10". Bolt forces were the highest of the three load cases, but still acceptable.

Load Case 3 – Dead Load plus Quartering Wind Load The fem forces and anchor bolt loads from this load case were all less than Load Case 2.



<u>**Figure 4 – Forces: Load Case 1**</u>



Force Contours

<u>Figure 5 – Forces: Load Case 2</u>
Force Contours

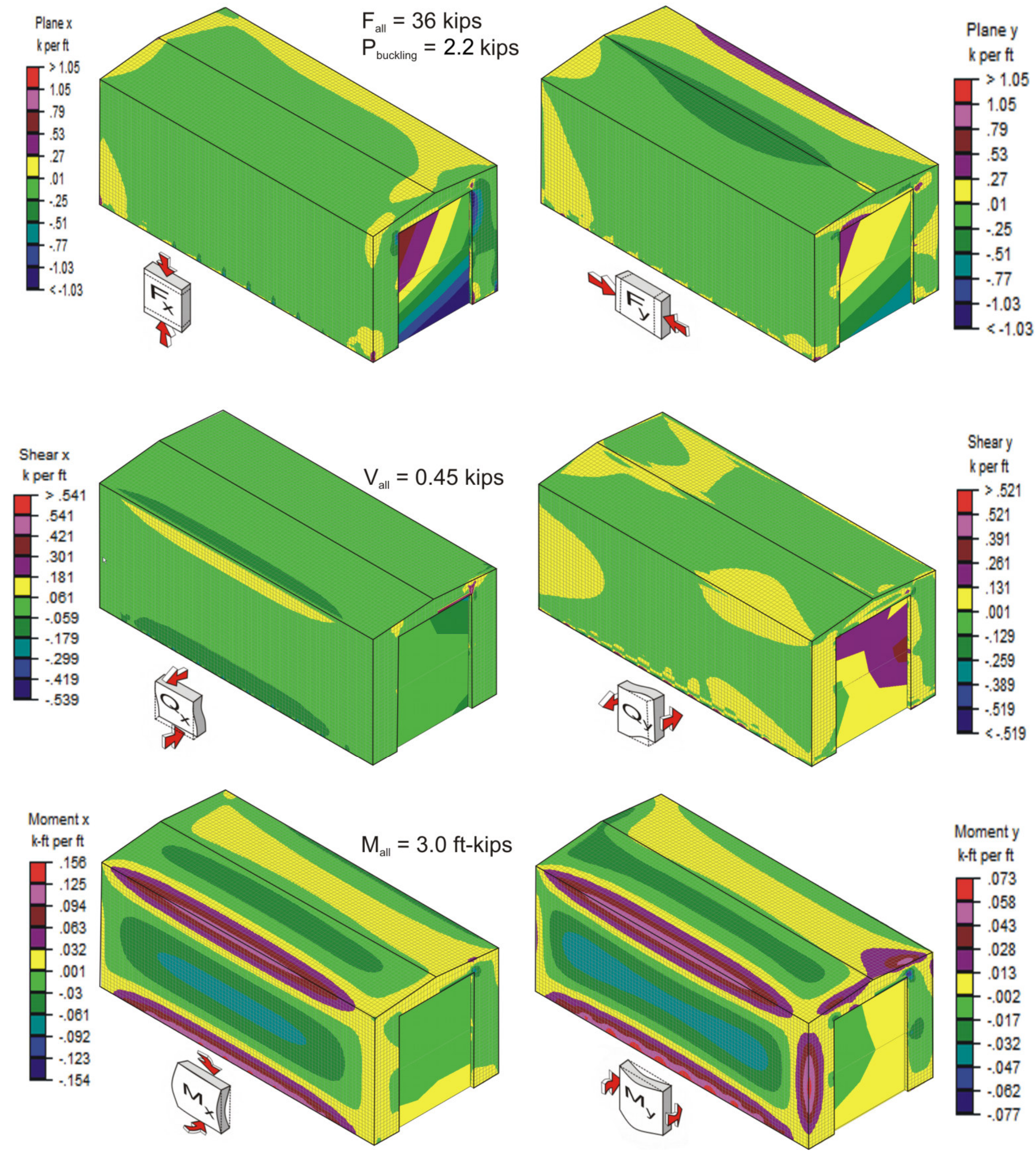


Figure 6 – Forces: Load Case 3

Force Contours

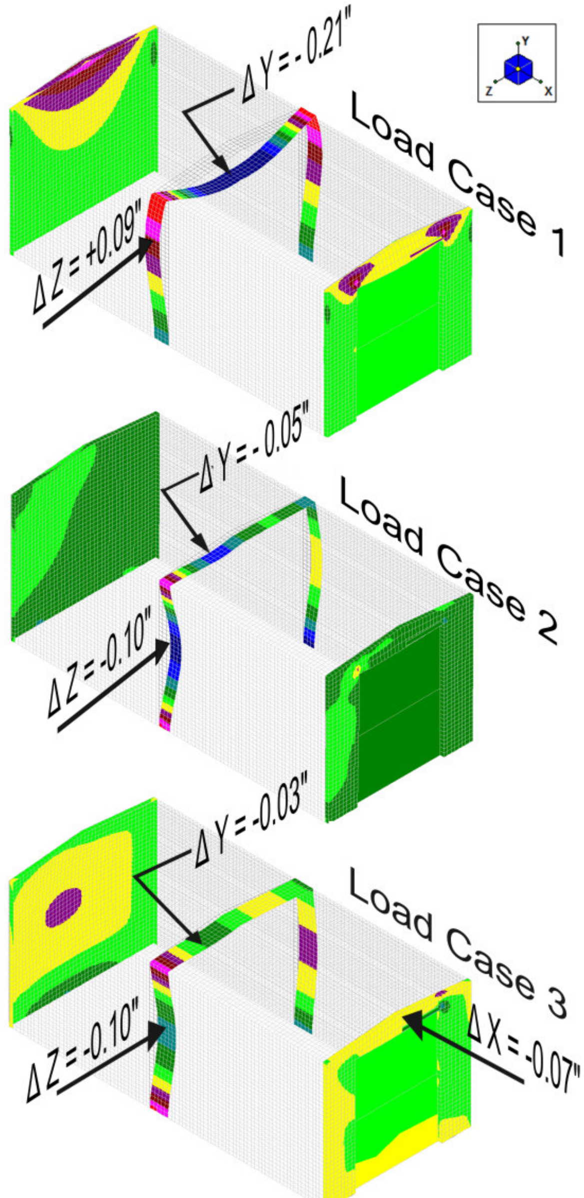
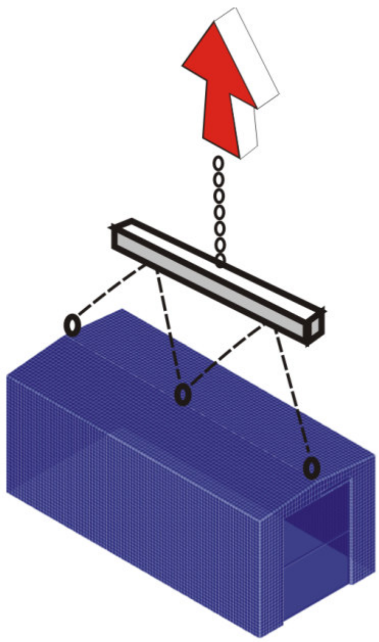


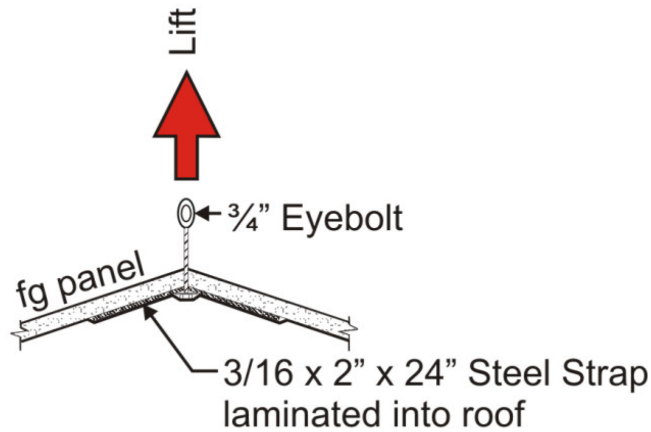
Figure 7 – Deflection Diagrams
Exaggerated 100:1 Mx Force Contours

Table 1 – Summary of FEM Results

	Case 1	Case 2	Case 3	Allowable
Shear, kips	0.18	0.37	0.32	**0.45**
Moment – foot-kips	0.13	0.20	0.13	**2.99**
Axial Load - Kips	1.50	0.51	1.02	**22.4**
Wall Buckling - kips	0.51	0.26	0.13	**1.11**
Bolt Uplift, kips	0.29	1.02	0.75	**1.90**
Bolt Shear, kips	0.29	0.92	0.60	**2.30**

The building is constructed with 3 eye-bolts embedded into the ridge line of the building for lifting and moving the building. Fabricated weight of the FRP building is about 2,500 pounds.





Check load on eyebolt: $\text{Weight} := 2500$ # $T := \frac{\text{Weight}}{3}$ $T_{max} := 2 \cdot T$

$T_{max} = 1666.7$ # $T_{all} = 5{,}200$ # ... OK

Check punching shears in roof:

In the A36 steel strap: Shear area = A_s $D := 1.5$ in $t := \frac{3}{16}$ inches

$A_s := \pi \cdot D \cdot t$ $A_s = 0.88$ in^2 Adding 33% to T for impact, middle bolt:

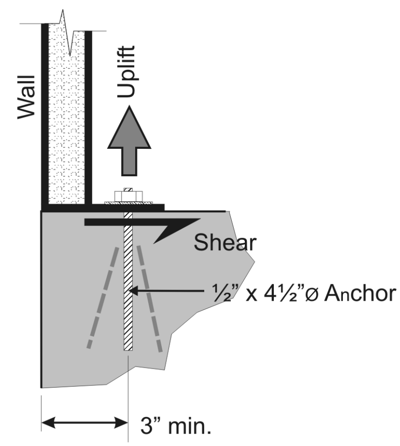
$v_p := \frac{1.33\,T_{max}}{A_s}$ $v_p = 2508.8$ psi $Fv_{all} = 14.4$ ksi ... OK

In fiberglass, check 6" strip either side of roof eyebolt:

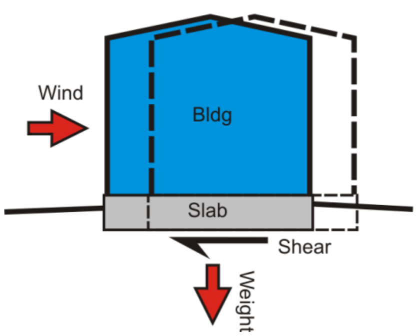
Shear area = A_s $A_s := 2 \cdot 12$ $A_s = 24.00$ sq. in. $SF := 2.5$

$v_p := \frac{1.33\,T_{max}}{A_s}$ $v_p = 92.4$ psi $V_{all} = 2{,}200$ psi ... OK

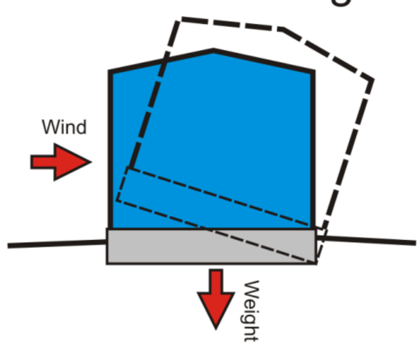
The building is attached to the foundation slab with 32 anchor bolts: 12 bolts along the long sides, two at the door jamb, and four additional bolts along the back wall. The bolts are ½“Ø stud anchors. Peak shear and uplift forces from the FEM model were well below allowable (see Table 1).



Sliding



Oveturning



Check sliding of building: $l := 10$ ft $b := 22$ ft $t_{slab} := 15$ in $Wind := 8666$ # (LC 2)

$$W_{slab} := l \cdot b \cdot \frac{t_{slab}}{12} \cdot 150 \qquad W_{bldg} := 2500 \qquad W := W_{bldg} + W_{slab}$$

$W_{slab} = 41250$ # $W = 43750$ # f = coef. fricrion = 0.3 $f := .3$

$F_{resist} := f \cdot W$ $F_{resist} = 13125$ # $SF := \frac{F_{resist}}{Wind}$ $SF = 1.5$ ...OK

Check overturning about lower right corner:

Heigth of wind = $X_{bar} := 5$ ft $M_{ot} := X_{bar} \cdot Wind$ $M_{ot} = 43330$ ft-#

Center of weight = $X_{wt} := 5$ ft $M_{resist} := X_{wt} \cdot W$ $M_{resist} = 218750$ ft-#

$$SF := \frac{M_{resist}}{M_{ot}} \qquad SF = 5 \quad \text{...OK}$$

For simple structures, the equivalent lateral force equation is used to estimate earthquake forces. The lateral seismic force, V, is calculated as:

$V = ZIKCSW$ (ASCE Equation 7-88)

Where Z = seismic zone coefficient ….. maximum for Zone IV = 1.0
I = occupancy importance factor … for essential facilities = 1.50
K = horizontal shear force factor …. 1 storey bearing wall system = 1.00
C = seismic coefficient = $^1/(15 * T^{0.5}) < 0.12$
S = soil profile coefficient …..sand or weak clays = 1.5
T = fundamental elastic period of vibration
W = total dead load

For a FRP building:

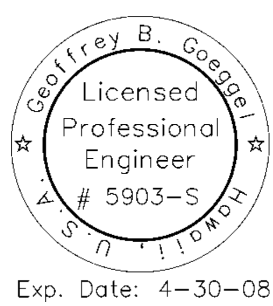
$T = 0.05\ h/D^{0.5} \quad = 0.05 * 9' \times 16'^{0.5} = 0.11$ seconds
$C = 1/[15 * (0.11)^{0.5}] = 0.20 > 0.12$ use $C = 0.12$

The dead load, W, includes the building, attached equipment and snow load:

W = 2,500# + 2,500# + (10' x 22' x 30 psf) = 11,600 pounds
$V = ZIKCSW$
$V = 1.0 \times 1.5 \times 1.0 \times 0.12 \times 1.5 * W$
$= 0.27 \times W = 0.27 \times 11.6$ kips = 3.13 kips < Wind = 8.7 kips

D. Summary

This fiberglass building is structurally sound for the loading conditions of 30 psf snow load and 135 mph winds. A 15" deep slab should provide adequate foundation stability, but this should be confirmed by a structural engineer familiar with the local soils and site topographic condition. The building shell is very stiff, with peak deflections under ½". The metal access doors and frame carry a large wind load and help stabilize the door opening, but were not evaluated in this study.



This report was prepared by:

Geoffrey B. Goeggel, PE
Chief Engineer
Hawaii Consulting Group
http://hicons.net

G.B. Goeggel