Frameless Hardware Company LLC
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SUBJ: FHC ASPIRE STOREFRONT
ENGINEERING REPORT AND WIND LOAD CHARTS

The FHC Aspire storefront system uses aluminum extrusions and glass panels to create entryways and storefronts. The Aspire system may be used in exterior building openings. The purpose of this report is to provide wind load tables that may used for the design of the system when used in typical conditions.

Table of Contents:

All wind loads in this report are service level/ Allowable Stress Design values:
$W_{asd} = 0.6W_{ult}$

The system will meet all applicable requirements of the 2015, 2018 and 2021 International Building Codes and International Residential Codes, 2016 and 2019 California Building and Residential Codes, Florida Building Code and other state codes adopting these versions of the IBC and IRC. Aluminum components are designed per 2020 Aluminum Design Manual unless noted otherwise herein. Glass is designed according to GANA guidelines, ASTM E1300 and *Engineering Structural Glass* published by NSCEA.

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TYPICAL CONDITIONS

The Aspire system is typically used in exterior storefront applications. The mullions utilize a thermal break and capture 1” insulating glass units to create an insulating wall. Doors will lock at the head or the floor and will not directly bear against the jambs.

Typical door and wall heights: 7’0”, 7’ 6”, 8’ 0”, 8’ 6”, 9’ 0” and 9’ 6”.
Typical door and sidelite width: 3’ 0”, 3’ 6”, 4’ 0”.

Doors may be used in a double or single door configuration.

Where the system is used at an exterior wall, the system shall be designed for wind load. The allowable wind load on the system depends on the geometry and anchorage method. This report includes standard anchorage to concrete details that will develop the jamb and mullion allowable wind load pressure tables. The standard 10” rail anchorage method is limited to uncracked concrete. Due to the high redundancy of the connection, it is reasonable to assume uncracked concrete in many situations since a crack in the slab or curb would only affect one or two anchors.

Typical anchorage hardware:
4” rails to 3,000psi cracked concrete: 1/4”x4” Tapcon or KH-EZ at 12” O.C.
10” tails to 3,000psi uncracked concrete: 1/4”x4” KH-EZ at 9” O.C.
Fin anchorage to concrete: (4) 3/8”x4” KH-EZ

ALLOWABLE WIND LOADS

Check the appropriate allowable wind load tables for the project's conditions. Design tables are summarized below. They derived throughout the body of this report. All wind loads in this report are service level/ Allowable Stress Design values:

$W_{asd} = 0.6W_{ult}$

Design checklist:

1) If intermediate mullions are used check Table 1. No need to also check table 3.
2) If no intermediate mullions are used, check Table 3.
3) If a transom and double door are used **and a door stop or locking device attaches to the header**, check tables 5 - 7 as appropriate for the opening width. Otherwise the header does not control design.
4) Calculate the allowable pressure based on the strength of the fin per Equation 1.
5) Use typical anchors to concrete as noted on page 2 of this report (will not control design) or hire a design professional to design custom anchorage for the project specific conditions.

Table 1) Allowable wind load on intermediate mullions (psf)

	Mullion Spacing (in)		
Span[1] (in)	**36**	**42**	**48**
84	45.0	38.6	33.7
90	36.0	30.9	27.0
96	29.3	25.1	22.0
102	24.2	20.7	18.1
108	20.1	17.3	15.1
114	17.0	14.5	12.7

1: Span is measured from bottom of sill to top of header or from bottom of sill to fin connection.

Table 3) Allowable wind load on jambs:

	Mullion Spacing (in)		
Span[1] (in)	**36**	**42**	**48**
84	45.0	45.0	45.0
90	45.0	41.3	36.1
96	39.2	33.6	29.4
102	32.3	27.7	24.2
108	26.9	23.1	20.2
114	22.7	19.5	17.0

1: Span is measured from bottom of sill to top of header or from bottom of sill to fin connection.

Table 5) Allowable wind load on 72” wide door opening (psf)

	Transom Height (in)						
Door Height (in)	**12**	**18**	**24**	**30**	**36**	**42**	**48**
96	36.0	34.7	33.6	32.5	31.4	30.5	29.6
102	34.0	32.9	31.8	30.9	29.9	29.1	
108	32.3	31.2	30.3	29.4	28.6		
114	30.7	29.7	28.9	28.1			

Table 6) Allowable wind load on 84” wide door opening (psf)

	Transom Height (in)						
Door Height (in)	**12**	**18**	**24**	**30**	**36**	**42**	**48**
96	22.7	21.9	21.1	20.4	19.8	19.2	18.6
102	21.4	20.7	20.1	19.4	18.8	18.3	
108	20.3	19.7	19.1	18.5	18.0		
114	19.3	18.7	18.2	17.7			

Table 7) Allowable wind load on 96” wide door opening (psf)

	Transom Height (in)						
Door Height (in)	**12**	**18**	**24**	**30**	**36**	**42**	**48**
96	15.2	14.7	14.2	13.7	13.3	12.9	12.5
102	14.4	13.9	13.4	13.0	12.6	12.3	
108	13.6	13.2	12.8	12.4	12.0		
114	12.9	12.5	12.2	11.8			

To calculate allowable pressure based on fin strength:

$P_a = R_a/(H*W_d/(576)+14HW_s/6912)$ - *Equation 1*

Where P_a = allowable pressure in psf base

R_a = allowable fin reaction per Table 9

H = Wall height in inches

W_d = Total door opening width in inches

W_s = Sidelite width in inches

GLASS

All glass is fully tempered with a mean modulus of rupture of 24,000psi. The Aspire system utilizes 1” IGUs composed of two panes of 1/4” tempered glass. Window glass is designed according to ASTM E1300.

For two panes for tempered glass the glass type factor (GTF) is 3.6 per E1300 Table 2.
For two equal thickness panes the load share factor (LS) is 2.0 per E1300 Table 5.

Check NFL for the largest available lite (48”x144”).

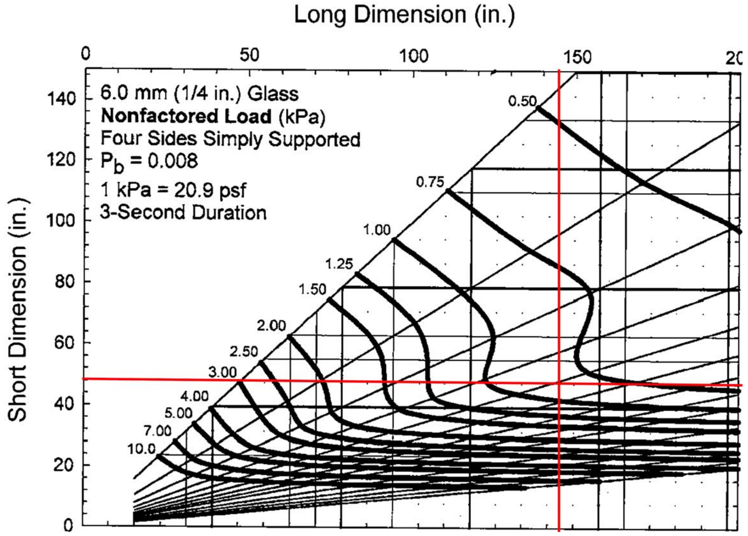
NFL = 20.9psf*0.87 = 18.2psf
LR = 3.6*2.0*18.2psf = 131psf

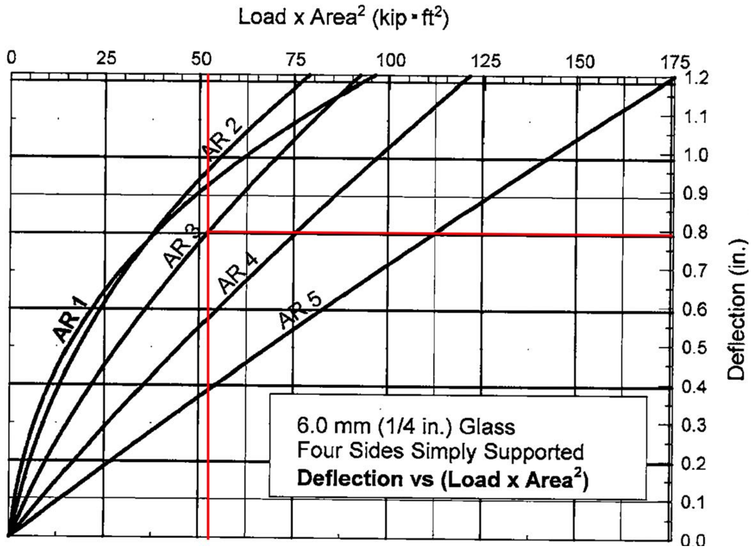
Assume a conservative 45.1psf wind load which is the maximum allowed by the system for a geometry that could result in a 48”x144” sidelite panel.

PA^2 = 45.1psf/(2*1000)*(4’*12’)^2 = 52.0kip-ft^2
AR = 12’/4’ = 3.0

Δ= 0.79”
L/Δ= 48”/0.79” = 60.8 > 60 OK

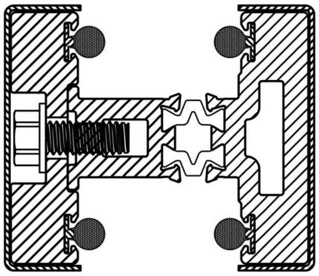
The above analysis combines the weakest panel (4’x12’) with the strongest mullion configuration (12’ mullion with fin support 8’ above ground) and the glass passed all design checks. Therefore, it can be assumed 1” tempered IGUs will develop the allowable wind loads given in this report’s allowable wind load tables.





Note on glass type:
laminated 1/4” glass may be used in place of the 1/4” tempered glass for an allowable wind load of 33 psf maximum.

INTERMEDIATE MULLION



Composite Section Properties:
Calculations will be presented further in this report analyzing the composite action between the face plate and remainder of the mullion.
$A = 1.95in^2$
$I = 1.21in^4$
$S = 1.06in^3$
$M_a = 1.06in^3 * 15.2ksi = 16{,}100$”# For 6063-T6 aluminum

Deflection criteria:
L/175 for spans under 13’ 6”.
These mullions will not be used at spans greater than 9’ 6”.

For calculating allowable wind load:
Moment load criteria, $P_a = 16{,}100$”#*8/(TW/144*L^2)
Deflection criteria, $P_a = 384*10.1*10^6psi*1.21in^4*/(175*5*TW/144*L^3)$
Note that the actual span is 5.875” shorter than the wall height due to where the mullions connect to the sill and head.

Allowable pressures that pass both criteria are listed in the table below.

Table 1) Allowable wind load on intermediate mullions (psf)

	Mullion Spacing (in)		
Span (in)	**36**	**42**	**48**
84	45.0	38.6	33.7
90	36.0	30.9	27.0
96	29.3	25.1	22.0
102	24.2	20.7	18.1
108	20.1	17.3	15.1
114	17.0	14.5	12.7

For calculations performed further in this report it may be desirable to know the maximum shear expected to be carried by each mullion. Estimate maximum shears are calculated by multiplying the maximum wind load from Table 1 by the half the span times the tributary width.

$V_{max} = P_a*span/(2*12)*TW/12$

Table 2) Maximum intermediate mullion shear loads (lbs)

	Mullion Spacing (in)		
Span (in)	**36**	**42**	**48**
84	472.4	472.4	472.4
90	405.4	405.4	405.4
96	351.7	351.7	351.7
102	308.0	308.0	308.0
108	271.9	271.9	271.9
114	241.8	241.8	241.8

Note that the estimated end shear loads are the same regardless of the tributary width and that the shortest mullions result in the highest allowable shear loads.

Max shear load for design = 472# per mullion

Shear clip uses an aluminum angle with at least one 1/4” machine screw per end. Intermediate mullions will use a clip to each side.
Assume 75ksi minimum strength
$V_a = 2*0.6*0.0318in^2*75ksi/3 = 954\# > 472\#$ OK
For bearing and tear out strength assume F_u=75ksi 304 stainless steel
Bearing on aluminum, $V_a = 2*2.4*0.25”*0.094”*75ksi/3 = 2{,}820\# > 472\#$ OK
$V_a = 2*1.2*(0.365”-0.125”)*0.094”*75ksi/3 = 1{,}350\# > 472\#$ OK

JAMB MULLION

Composite Section Properties:
Calculations will be presented further in this report analyzing the composite action between the face plate and remainder of the mullion.
$A = 1.39in^2$
$I = 0.809in^4$
$S = 0.698in^3$
$M_a = 0.698in^3 * 15.2ksi = 10{,}600"\#$ For 6063-T6 aluminum

Deflection criteria:
L/180 for spans under 13’ 6”.
These mullions will not be used at spans greater than 9’ 6”.

For calculating allowable wind load:
Moment load criteria, $P_a = 10{,}600"\# * 8/(TW/144 * L^2)$
Deflection criteria, $P_a = 384 * 10.1 * 10^6 psi * 0.809in^4 * /(175 * 5 * TW/144 * L^3)$
Note for jambs the tributary width is half the spacing of the mullions. Doors do not directly load door jambs. The equations above are based on actual tributary width but for ease of the design, the table below presents wind loads with respect to span and mullion spacing.
Allowable pressures that pass both criteria are listed in the table below.

Table 3) Allowable wind load on jambs:

	Mullion Spacing (in)		
Span (in)	**36**	**42**	**48**
84	45.0	45.0	45.0
90	45.0	41.3	36.1
96	39.2	33.6	29.4
102	32.3	27.7	24.2
108	26.9	23.1	20.2
114	22.7	19.5	17.0

Allowable wind loads shown in Table 3 are greater than those shown in Table 1. Therefore, when intermediate mullions are used, Table 3 need not be checked.

Table 4) Maximum jamb mullion shear loads (lbs) based on allowable wind load

	Mullion Spacing (in)		
Span (in)	**36**	**42**	**48**
84	236.3	275.6	315.0
90	253.1	271.0	271.0
96	235.1	235.1	235.1
102	205.9	205.9	205.9
108	181.8	181.8	181.8
114	161.7	161.7	161.7

Max shear load for design = 316# per jamb mullion

Bearing on aluminum, V_a = 2.4*0.25”*0.094”*75ksi/3 = 1,410# > 316# OK
V_a = 1.2*(0.365”-0.125”)*0.094”*75ksi/3 = 677# > 316# OK

FACE PLATE FASTENER

1/4” grade 5 machine bolt in tapped hole
Screw strength, V_a = 0.6*0.0318in^2*120ksi/2 = 1,140# per ASTM F879.
T_a = 0.0318in^2*120ksi/2 = 1,910# per ASTM F879.
Aluminum failure modes per ADM 2020 Chapter J.
Aluminum bearing, V_a = 2*0.25”*0.164”*30ksi/1.95 = 1,260#
Pullout, T_a= [1.2*0.25”*25ksi*(0.25-0.2”)+1.16*0.539in^2*30ksi*(0.2”-0.125)]/3 = 594#
Pullover, T_a = (0.5”-0.312”)*30ksi*0.164”/3 = 308#

Max tension for 12” O.C. spacing = 36.2psf*3’*1’ = 109# < 308# OK (Worst case tension is a short mullion under its maximum allowable wind load. Note mullions with different tributary widths will calculate the same face plate screw loading using this method.)

The face plate fastener also is used to create composite action between the face plate and the rest of the mullion.

For intermediate mullions:
V_{max} = 472# (triangular load)
For shear transfer calculation, use average shear. This is OK because of the ductility of the mechanically fastened connection.
V_{ave} = 472#/2 = 236#
v = VQ/I
Q = 0.667in^2*0.895” = 0.597in^3
I = 1.21in^4
v = 236#*0.597in^3/1.21in^4 = 116pli
Max fastener spacing for full composite action = 1,140#/116pli = 9.83”=> 9”
Reduction in wind load to go to 12” O.C. = 9.83/12 = 0.819

For jamb mullions:
V_{max} = 318# (triangular load)
For shear transfer calculation, use average shear. This is OK because of the ductility of the mechanically fastened connection.
V_{ave} = 318#/2 = 159#
v = VQ/I
Q = 0.326in^2*0.922” = 0.301in^3
I = 0.809in^4
v = 159#*0.301in^3/0.809in^4 = 47.3pli
Max fastener spacing for full composite action = 812#/47.3pli => 12”

HEADER

$A = 2.445in^2$
$I = 1.743in^4$
$S = 1.525in^3$
$M_a = 1.525in^3 * 15.2ksi = 23{,}200”\#$ For 6063-T6 aluminum

The head may control the allowable wind load on the wall when double doors are used. When single doors are used, the header will never control the allowable wind load on the wall.

Load to center loaded header from door = $P_a * W/12 * H_d/(12*2)$
Load from transom, $w = P_a * H_t/(144*2)$
$M_a = 23{,}200”\# = P_a * W/12 * H_d/(12*2) * W/4 + P_a * H_t/(144*2) * W^2/8$
$P_a = 23{,}200”\#/(W^2 * H_d/1152 + H_t * W^2/2304)$

$\Delta_a = W/175 = P_a * W/12 * H_d/(12*2) * W^3/(48 * 10.1 * 10^6 * 1.743) + 5 * P_a/12 * H_t/(12*2) * W^4/(384 * 10.1 * 10^6 * 1.743)$

$$P_a = 10.1 * 10^6 * 1.743 * W/175/[W/12 * H_d/(12*2) * W^3/48 + 5 * H_t/(12*12*2) * W^4/384]$$

Table 5) Allowable wind load on 72” wide door opening (psf)

	Transom Height (in)						
Door Height (in)	**12**	**18**	**24**	**30**	**36**	**42**	**48**
96	36.0	34.7	33.6	32.5	31.4	30.5	29.6
102	34.0	32.9	31.8	30.9	29.9	29.1	
108	32.3	31.2	30.3	29.4	28.6		
114	30.7	29.7	28.9	28.1			

Table 6) Allowable wind load on 84” wide door opening (psf)

	Transom Height (in)						
Door Height (in)	**12**	**18**	**24**	**30**	**36**	**42**	**48**
96	22.7	21.9	21.1	20.4	19.8	19.2	18.6
102	21.4	20.7	20.1	19.4	18.8	18.3	
108	20.3	19.7	19.1	18.5	18.0		
114	19.3	18.7	18.2	17.7			

Table 7) Allowable wind load on 96” wide door opening (psf)

	Transom Height (in)						
Door Height (in)	**12**	**18**	**24**	**30**	**36**	**42**	**48**
96	15.2	14.7	14.2	13.7	13.3	12.9	12.5
102	14.4	13.9	13.4	13.0	12.6	12.3	
108	13.6	13.2	12.8	12.4	12.0		
114	12.9	12.5	12.2	11.8			

Note that when floor mounted locking hardware is used, and no door stop or lock is provided at the header, the reaction to the header is significantly reduced since there is no reaction to the center of the header from the door. Table 8 provide allowable wind loads when this condition is met on a 96” long header.

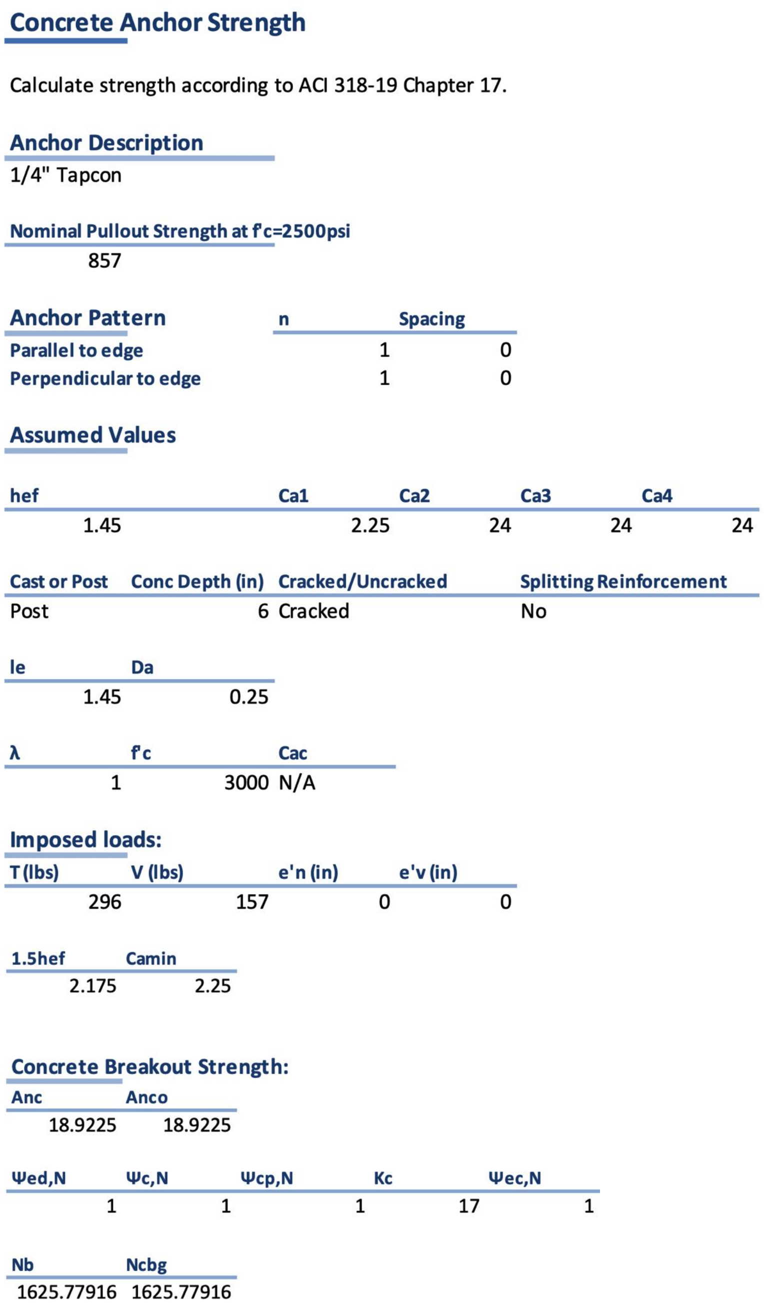
Table 8) Allowable wind load on 96” wide door opening (psf)

	Transom Height (in)						
Door Height (in)	**12**	**18**	**24**	**30**	**36**	**42**	**48**
96	209.6	139.7	104.8	83.8	69.9	59.9	52.4
102	209.6	139.7	104.8	83.8	69.9	59.9	
108	209.6	139.7	104.8	83.8	69.9		
114	209.6	139.7	104.8	83.8			

It can be seen by inspection the allowable pressure based on the header design will not be the controlling allowable pressure when there is no stop or lock connecting the door to the header.

4” SIDELITE RAIL ANCHORAGE

Max loading = 472# mullion reaction at 3’ spacing = 157plf
Tension on fastener = 157plf*1.75”/0.928” = 296plf



Concrete Anchor Strength

Calculate strength according to ACI 318-19 Chapter 17.

Anchor Description

1/4" Tapcon

Nominal Pullout Strength at f'c=2500psi

857

Anchor Pattern	n	Spacing
Parallel to edge	1	0
Perpendicular to edge	1	0

Assumed Values

hef	Ca1	Ca2	Ca3	Ca4
1.45	2.25	24	24	24

Cast or Post	Conc Depth (in)	Cracked/Uncracked	Splitting Reinforcement
Post	6	Cracked	No

le	Da
1.45	0.25

λ	f'c	Cac
1	3000	N/A

Imposed loads:

T (lbs)	V (lbs)	e'n (in)	e'v (in)
296	157	0	0

1.5hef	Camin
2.175	2.25

Concrete Breakout Strength:

Anc	Anco
18.9225	18.9225

Ψed,N	Ψc,N	Ψcp,N	Kc	Ψec,N
1	1	1	17	1

Nb	Ncbg
1625.77916	1625.77916

Side Face Breakout Strength:

Avc	Avco
22.78125	22.78125

Ψed,V	Ψc,V	Ψh,V	Ψec,V
1	1	1	1

Vb	Vcbg
919.576888	919.576888

Pryout Strength:

Kcp
1

Vcpg
1625.77916

Area Calcs:

	Anc		Avc	
Segment:	W	B	H	B
1	2.175	2.175	3.375	3.375
2	0	0		0
3	2.175	2.175		3.375
Total:	**4.35**	**4.35**	**3.375**	**6.75**

To find allowable tension load multiply by φ=0.65 and divide by 1.6 to convert to ASD level loading

All tens
381.386063

To find allowable shear load multiply by φ=0.7 and divide by 1.6 to convert to ASD level loading

All V
402.314889

Interaction:

Check interaction, V/Va+T/Ta<1.2

V/Va+T/Ta= 1.16635804

Anchor Adequacy:

PASS

10” SIDELITE RAIL ANCHORAGE

The design of the 10” rail differs in that the prying action action on the anchors is much greater.

Max loading = 472# mullion reaction at 3’ spacing = 157plf
Tension on fastener = 157plf*7.75”/0.928” = 1,240plf

Due to high tension loading. Use 1/4”x3” KH-EZ. The 10” rail will be limited to uncracked concrete unless a custom anchorage is designed for the specific application. Install anchors at 9” O.C..

Concrete Anchor Strength

Calculate strength according to ACI 318-19 Chapter 17.

Anchor Description

1/4" Tapcon

Nominal Pullout Strength at f'c=2500psi
2350

Anchor Pattern

	n	Spacing
Parallel to edge	1	0
Perpendicular to edge	1	0

Assumed Values

hef		Ca1	Ca2	Ca3	Ca4
1.92		2.25	24	24	24

Cast or Post	Conc Depth (in)	Cracked/Uncracked		Splitting Reinforcement
Post	6	Uncracked		No

le	Da
1.92	0.25

λ	f'c	Cac
1	3000	2.78

Imposed loads:

T (lbs)	V (lbs)	e'n (in)	e'v (in)
930	117.75	0	0

Area Calcs:

	Anc		Avc	
Segment:	W	B	H	B
1	2.25	2.88	3.375	3.375
2	0	0		0
3	2.88	2.88		3.375
Total:	**5.13**	**5.76**	**3.375**	**6.75**

1.5hef	Camin
2.88	2.25

Concrete Breakout Strength:

Anc	Anco
29.5488	33.1776

Ψed,N	Ψc,N	Ψcp,N	Kc	Ψec,N
0.934375	1.4	1	17	1

Nb	Ncbg
2477.20183	2886.06109

Side Face Breakout Strength:

Avc	Avco
22.78125	22.78125

Ψed,V	Ψc,V	Ψh,V	Ψec,V
1	1.4	1	1

Vb	Vcbg
972.690542	1361.76676

Pryout Strength:

Kcp
1

Vcpg
2886.06109

To find allowable tension load multiply by φ=0.65 and divide by 1.6 to convert to ASD level loading

All tens
1045.80776

To find allowable shear load multiply by φ=0.7 and divide by 1.6 to convert to ASD level loading

All V
595.772957

Interaction:

Check interaction, V/Va+T/Ta<1.2

V/Va+T/Ta= 1.08690718

Anchor Adequacy:

PASS

FIN DESIGN

A standard 16” fin for a 4’ transom has been modeled using SCIA Engineer. Using a 1,000# test load, a deflection of 0.10” is measured in the direction of the load. The support stiffness of 1,000#/0.10” = 10 kips/in is used in the wall model to find the reaction to the fin supports.

Stiffness under vertical loading was evaluated using a 100# test load resulting in 0.003” deflection. k_z = 100#/0.003” = 33kips/in.

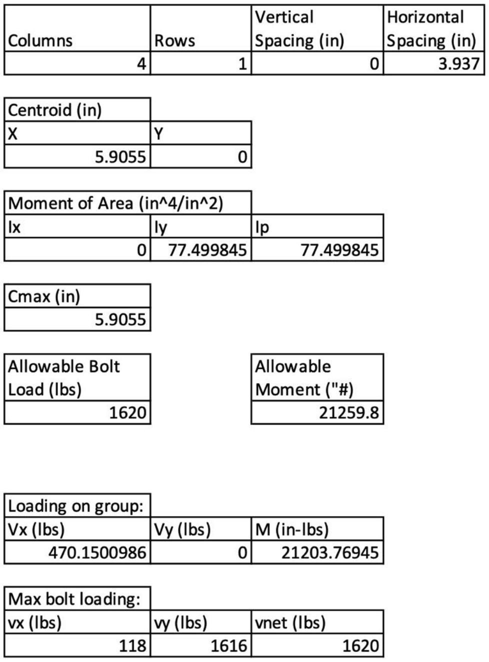
The loading to the fin is based on the the overall wall height and spacing. For fins at headers, the tributary width is the average of the width of door opening and the side lite width.

1/4” countersunk stainless steel bolt with barrel nut in double shear.
Screw strength, V_a = 0.6*2,705#/2*2 = 1,620# per ASTM F879.
T_a = 2,705#/2 = 1,350# per ASTM F879.

Top connection:
3.937” center to center spacing at top connection.
$I_b = 2*(3.937”/2)^2+2*(3.937”*1.5)^2 = 77.5in^4/in^2$
Number of bolts = 4
M = (H_t-2.9”)*V

An iterative process was used to vary V until the bolt reaction was 1,620#, the allowable shear load on the bolt. This will vary for different fin heights. The equation for M with respect to V is shown above.

The spreadsheet shows the results from the 48” transom. The process was repeated for shorter fins in increments of 6”. The table on the following page tabulates the calculated allowable fin reactions.



Columns	Rows	Vertical Spacing (in)	Horizontal Spacing (in)
4	1	0	3.937

Centroid (in)

X	Y
5.9055	0

Moment of Area (in^4/in^2)

Ix	Iy	Ip
0	77.499845	77.499845

Cmax (in)
5.9055

Allowable Bolt Load (lbs)	Allowable Moment ("#)
1620	21259.8

Loading on group:

Vx (lbs)	Vy (lbs)	M (in-lbs)
470.1500986	0	21203.76945

Max bolt loading:

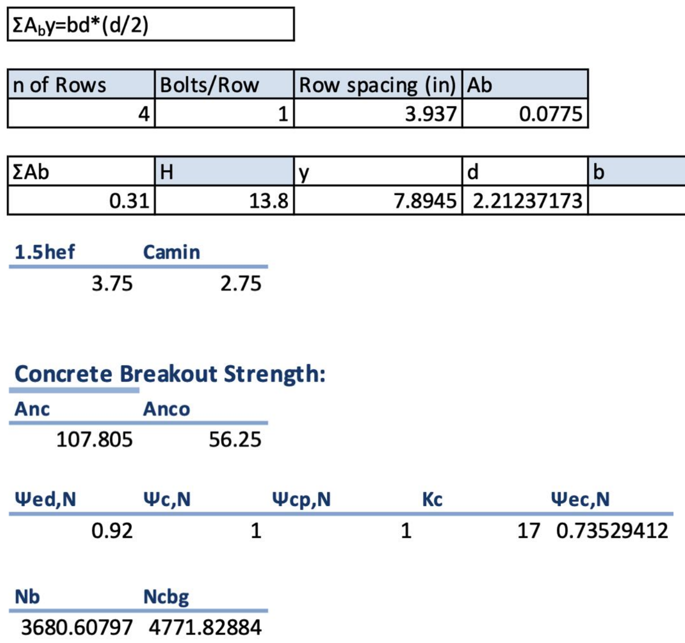
vx (lbs)	vy (lbs)	vnet (lbs)
118	1616	1620

Check lower connection strength:
(2) 1/4” SS screws in tension
Steel strength = 2,705#/3 = 902# each (ASTM F879 annealed condition)
Pullout, L_e = 0.188”
$T_a = [1.2*0.25”*25ksi*(0.25”-0.188”)+1.16*0.539in^2/in *30ksi*(0.188”-0.125”)]/3$
$= 511\#$ each
Recall 1/4” counter sunk screw in barrel nut strength = 1,620# each
The anchorage strength is limited by the machine screws in pullout. The allowable load to the fin shall be limited to 2*511# = 1,020# maximum. This controls the fin strength in the 18” and 12”

Table 9: Allowable fin reactions based on upper bolts

Fin/transom Height (in)	Allowable Fin Reaction (lbs)
48	470
42	542
36	639
30	779
24	996
18	1380
12	2200

Anchorage to concrete:
(4) 3/8”x4” KH-EZ at same spacing as thru bolts.
Check 470# on 48” system and 1,020# on 18” system (strength on the 18” system is limited by the lower connection).



$\Sigma A_b y = bd*(d/2)$

n of Rows	Bolts/Row	Row spacing (in)	Ab
4	1	3.937	0.0775

ΣAb	H	y	d	b
0.31	13.8	7.8945	2.21237173	1

1.5hef	Camin
3.75	2.75

Concrete Breakout Strength:

Anc	Anco
107.805	56.25

Ψed,N	Ψc,N	Ψcp,N	Kc	Ψec,N
0.92	1	1	17	0.73529412

Nb	Ncbg
3680.60797	4771.82884

(t) column shows tension loads for each anchor. Negative values are ignored and indicates the anchor is located within the compression zone.

Eccentricity of anchor load = (970#*3.937”-311#*3.937”)/1,920# = 1.35”
Tension loading must be combined with shear loading for anchor design. Note that the front anchor is the anchor not loaded in tension. Therefore the front edge distance for the design of the tension anchors is the edge distance to the front anchor plus the anchor spacing of 3.937”.
Minimum edge distance to back anchor = 2.75”.

Concrete Anchor Strength

Calculate strength according to ACI 318-19 Chapter 17.

Anchor Description

3/8" KH-EZ

Nominal Pullout Strength at f'c=2500psi

N/A

Anchor Pattern

	n	Spacing
Parallel to edge	1	0
Perpendicular to edge	3	3.937

Assumed Values

hef	Ca1	Ca2	Ca3	Ca4
2.5	6.687	24	24	2.75

Cast or Post	Conc Depth (in)	Cracked/Uncracked	Splitting Reinforcement
Post	6	Cracked	No

le	Da
2.5	0.375

λ	f'c	Cac
1	3000	N/A

Imposed loads:

T (lbs)	V (lbs)	e'n (in)	e'v (in)
1920	470	1.35	0

Side Face Breakout Strength:

Avc	Avco
262.098	201.221861

Ψed,V	Ψc,V	Ψh,V	Ψec,V
1	1	1.292961716	1

Vb	Vcbg
5933.3919	9992.5711

Pryout Strength:

Kcp
2

Vcpg
9543.65768

Area Calcs:

	Anc		Avc	
Segment:	W	B	H	B
1	3.75	3.75	6	21.8415
2	7.874	0		0
3	2.75	3.75		21.8415
Total:	**14.374**	**7.5**	**6**	**43.683**

To find allowable tension load multiply by φ=0.65 and divide by 1.6 to convert to ASD level loading

All tens
1938.55547

To find allowable shear load multiply by φ=0.7 and divide by 1.6 to convert to ASD level loading

All V
4175.35024

Interaction:

Check interaction, V/Va+T/Ta<1.2

V/Va+T/Ta= 1.10299361

Anchor Adequacy:

PASS

The tension anchors have adequate strength under the maximum applied moment loading. For other fin conditions the tension loading will be smaller. For design of the front anchor the shortest fin with the highest shear load will control. Max total shear load = 1,020# (limited by connection at bottom of fin). Shear load concentrated on front anchor = 1,020#/4 = 255#. The anchor design is shown on the following pages.

Concrete Anchor Strength

Calculate strength according to ACI 318-19 Chapter 17.

Anchor Description

3/8" KH-EZ

Nominal Pullout Strength at f'c=2500psi

N/A

Anchor Pattern	n	Spacing
Parallel to edge	1	0
Perpendicular to edge	1	0

Assumed Values

hef		Ca1	Ca2	Ca3	Ca4
2.5		2.75	24	24	12

Cast or Post	Conc Depth (in)	Cracked/Uncracked	Splitting Reinforcement
Post	6	Cracked	No

le	Da
2.5	0.375

λ	f'c	Cac
1	3000	N/A

Imposed loads:

T (lbs)	V (lbs)	e'n (in)	e'v (in)
0	255	0	0

1.5hef	Camin
3.75	2.75

Concrete Breakout Strength:

Anc	Anco
48.75	56.25

Ψed,N	Ψc,N	Ψcp,N	Kc	Ψec,N
0.92	1	1	17	1

Nb	Ncbg
3680.60797	2934.67142

Side Face Breakout Strength:

Avc	Avco
34.03125	34.03125

Ψed,V	Ψc,V	Ψh,V	Ψec,V
1	1	1	1

Vb	Vcbg
1564.7865	1564.7865

Pryout Strength:

Kcp
2

Vcpg
5869.34284

Area Calcs:

	Anc		Avc	
Segment:	W	B	H	B
1	2.75	3.75	4.125	4.125
2	0	0		0
3	3.75	3.75		4.125
Total:	**6.5**	**7.5**	**4.125**	**8.25**

To find allowable tension load multiply by φ=0.65 and divide by 1.6 to convert to ASD level loading

All tens
1192.21026

To find allowable shear load multiply by φ=0.7 and divide by 1.6 to convert to ASD level loading

All V
684.594093

Interaction:

Check interaction, V/Va+T/Ta<1.2

V/Va+T/Ta= 0.37248349

Anchor Adequacy:

PASS

2.75" edge distance passes anchor design.

The above calculations have demonstrated the (4) 3/8"x4" KH-EZ anchorage to concrete will develop the same strength as the fin connections.

The fin is modeled under the prior loads to determine if the glass stresses control over the bolt loading. Allowable glass tension stress = 10,600psi. Fin stress diagrams are provided at the end of this section.

First check 48” fin with 470# reaction:

Max tension stress = 7,970psi < 10,600psi OK

For 12” fin with 2,200# reaction:

Max tension stress = 1,940psi < 10,600psi OK

The glass will be most likely to control over the bolt at the heights which causes bolt loading towards the top edge. Since the glass fin did not control at the 48” maximum height, it can be assumed the glass strength will not control over the bolt strength at any of the cantilevers.

Using the SCIA model for the 12’ tall wall with 4’ sidelites, a reaction from the sidelite can be calculated. Reaction from sidelite under 40psf wind load = 559#.
Reaction from sidelite = 559#/40psf = 14.0 lbs/psf.
The modeled condition is a 9’ 6” door on 12’ wall. This produces the maximum reaction since it uses the maximum wall height with the shortest fin. Modeling has confirmed that as the fin gets shorter, a couple moment forms between the fin and the head which increases the reaction to the fin. Therefore, the value 14.0 lbs/psf may be scaled for other sidelite sizes and the resulting load will be conservative.

The below equations can be used to conservatively estimate the wind load reaction with respect to wall height, door width and sidelite width.
Reaction from door and transom, $R_d = H*W_d/(144*4)*P$
Reaction from sidelite, R_s = 14.0 lbs/psf*P*($H*W_s$)/(144”*48")
Total fin reaction, = $R = R_d+R_s$

To calculate allowable pressure based on fin strength:
$P_a = R_a/(H*W_d/(576)+14HW_s/6912)$
Where P_a = allowable pressure in psf
R_a = allowable fin reaction per Table 9
H = Wall height in inches
W_d = Total door opening width in inches
W_s = Sidelite width in inches

For example:
10’ wall with 2’ transom. Double 36” doors and 48” side lite.
P_a = 996#/(120”*72”/576+14*120”*48”/6912) = 37.4 psf (Allowable mullion wind load per Table 1 = 22.0 psf) Fin does not control design.

Worst case fin:
12’ wall with 4’ transom. Double 48” doors and 36” sidelite. This will result in the lowest fin strength relative to mullion strength.
P_a = 470#/(144”*96”/576+14*144”*36”/6912) = 28.9 psf (Allowable mullion wind load per Table 1 = 29.3psf) Fin barely controls design.

The very worst case fin setup relative to the mullion results in nearly identical strength. The fin will generally not control the allowable wind load but should be checked when mullions are used at lower spacing than the door width.

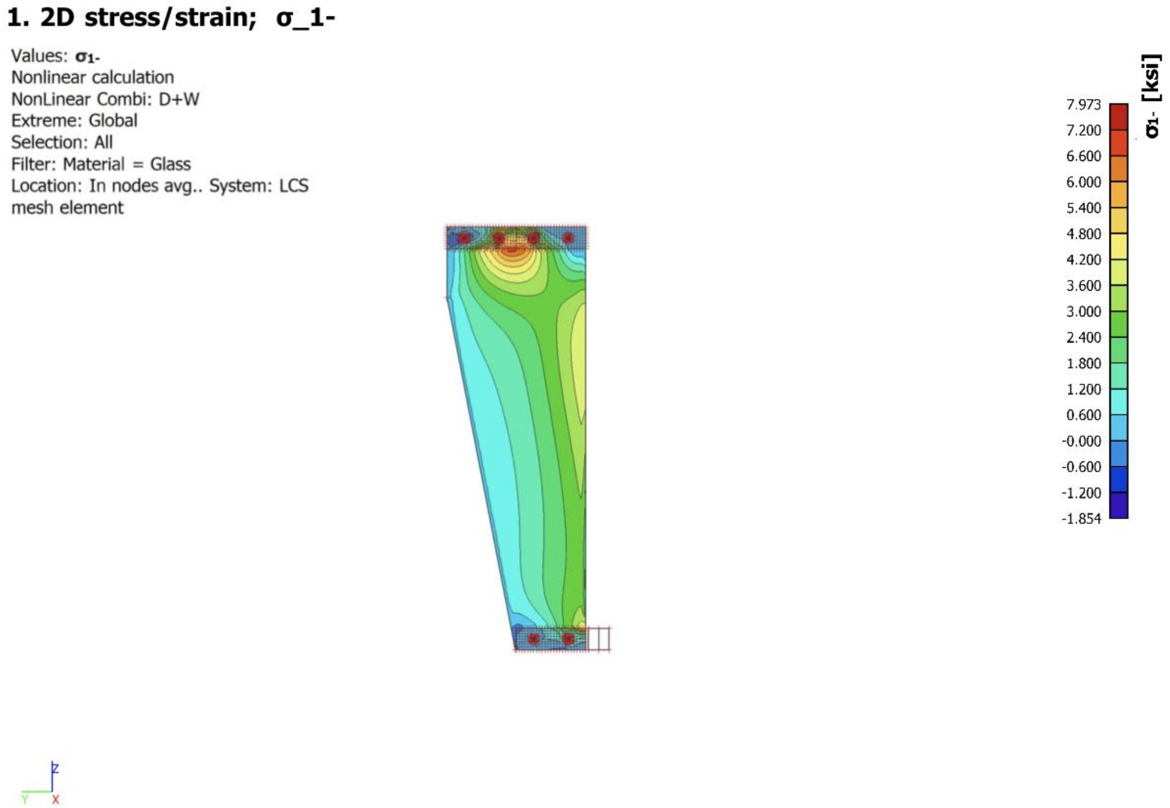
In summary, the allowable pressure on the entrance and sidelites will most likely not be controlled by the strength of the fin connections and will not be controlled by the strength of the fin glass. In some conditions the fin connection strength will control so the allowable pressure should be checked using the table and equation F1 below.

Table 10: Allowable reaction to fins:

Fin/transom Height (in)	Allowable Fin Reaction (lbs)
48	470
42	542
36	639
30	779
24	996
18	1020
12	1020

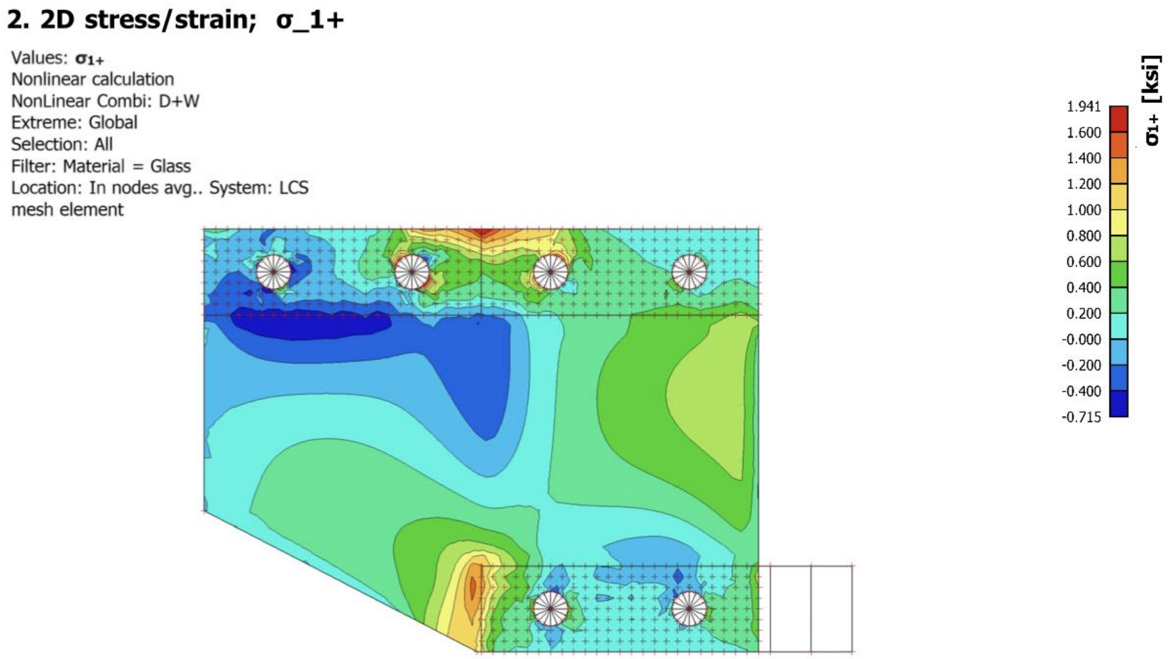
To calculate allowable pressure based on fin strength:
$P_a = R_a/(H*W_d/(576)+14HW_s/6912)$ - *Equation F1*
Where P_a = allowable pressure in psf base
R_a = allowable fin reaction per Table 9
H = Wall height in inches
W_d = Total door opening width in inches
W_s = Sidelite width in inches

Fin model glass stress under maximum allowable wind loads:



1. 2D stress/strain; σ_1-

Values: σ_{1-}
Nonlinear calculation
NonLinear Combi: D+W
Extreme: Global
Selection: All
Filter: Material = Glass
Location: In nodes avg.. System: LCS
mesh element



2. 2D stress/strain; σ_1+

Values: σ_{1+}
Nonlinear calculation
NonLinear Combi: D+W
Extreme: Global
Selection: All
Filter: Material = Glass
Location: In nodes avg.. System: LCS
mesh element

WALL MODEL

Typical wall configurations are modeled using SCIA engineer to compare to the design tables derived in this report. The models indicate the design assumptions of the report are conservative. Model and results are summarized on the following pages. Selected result diagrams will follow the result summaries.

A typical 10' tall wall is modeled using SCIA Engineer. The model assumes 4'x10' sidelite panels and a 8'x8' door opening.

Results of 40psf wind load:
Door jamb:

$M = 5{,}120"\# < 10{,}600"\#$ OK
$\Delta = 0.663"$
$\Delta_a = 96"/175 = 0.549" < 0.663"$

Intermediate mullion:

$M = 9{,}780"\# < 16{,}100"\#$ OK
$\Delta = 0.752"$
$\Delta_a = 96"/175 = 0.549" < 0.663"$

Header:

$M = 18{,}900"\# < 23{,}200"\#$ OK
$\Delta = 1.02"$
$\Delta_a = 96"/175 = 0.549" < 1.02"$

Most critical in this case is the header deflection:
$P_a = 0.549"/1.02"*40$ psf = 21.5 psf (Compared to 14.2 psf from Table 7)
Note for intermediate mullions, $P_a = 0.549"/0.663"*40$ psf = 33.1 psf (For 96" span and 48" tributary width P_a = 18.2 psf. The presence of the fin increases the allowable loading on the same span due to the presence of the back span. Therefore, Table 1 may be conservatively used when a fin is present where the span is measure from the ground to the fin height.)

A typical 12' tall wall is modeled using SCIA Engineer. The model assumes 4'x10' sidelite panels and a 8'x8' door opening.

Results of 40 psf wind load - see pages 28-30 for model results.
Door jamb:

M = 5,120"# < 10,600"# OK
$\Delta = 0.622"$
$\Delta_a = 96"/175 = 0.549" < 0.622"$ deflection limit exceeded

Intermediate mullion:

M = 9,780"# < 16,100"# OK
$\Delta = 0.741"$
$\Delta_a = 96"/175 = 0.549" < 0.741"$ deflection limit exceeded

Header:

M = 18,900"# < 23,200"# OK
$\Delta = 1.24"$
$\Delta_a = 96"/175 = 0.549" < 1.24"$ deflection limit exceeded

Allowable wind load must be reduced below the 40 psf because of deflection limits.
Most critical in this case is the header deflection:
$P_a = 0.549"/1.24"*40psf = 17.7psf$ (Compared to 12.5psf from Table 7)
Note for intermediate mullions, $P_a = 0.549"/0.741"*40psf = 29.6psf$ (For 96" span and 48" tributary width $P_a = 18.2psf$. The presence of the fin increases the allowable loading on the same span due to the presence of the back span. Therefore, Table 1 may be conservatively used when a fin is present where the span is measured from the ground to the fin height.)

Intermediate mullions at 4' spacing and 8' height:

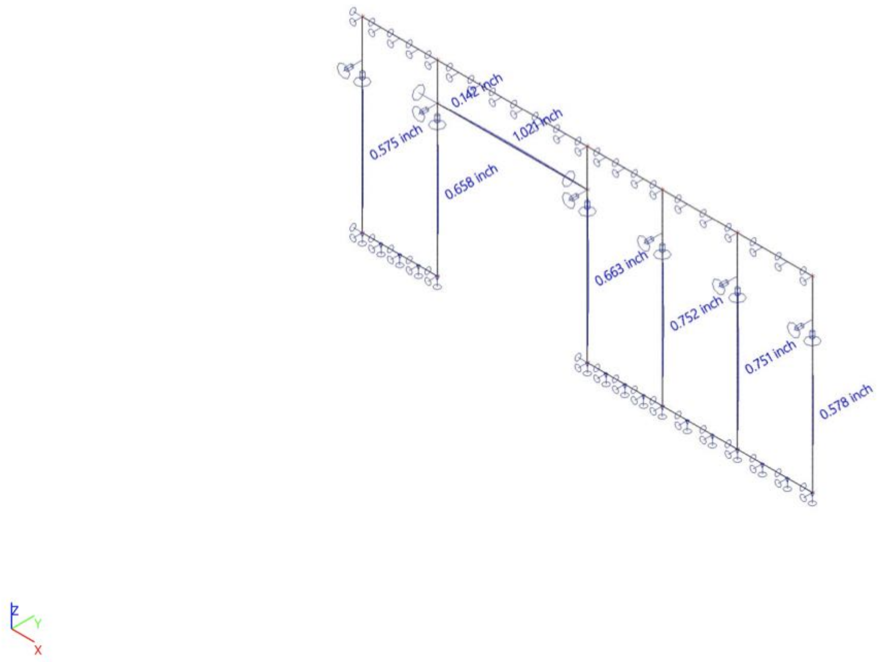
Under 40psf wind load:

M = 9,540"# < 16,100"# OK
$\Delta = 1.05"$
$\Delta_a = 96"/175 = 0.549" < 1.05"$
Allowable wind load = 0.549"/1.05"*40psf = 20.9psf
From Table 1 allowable wind load = 18.2psf < 20.9psf (Table 1 is slightly conservative)

Panels with low aspect ratios will experience non-linear behavior and two way bending that will increase the allowable load slightly above shown in Table 1)

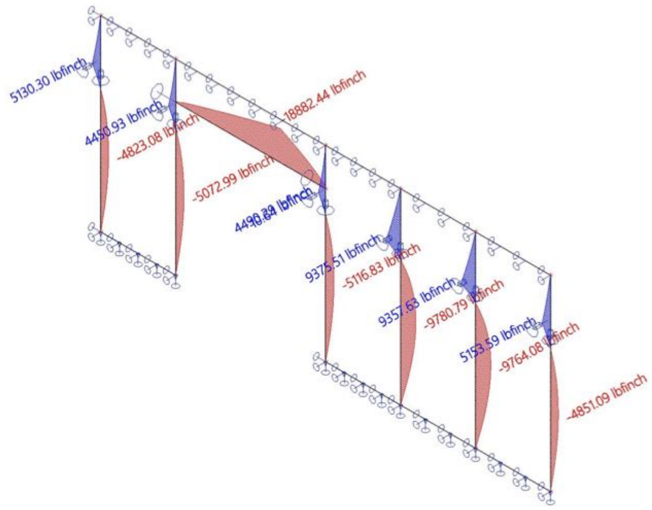
1. 1D deformations; U_total

Values: $\mathbf{U_{total}}$
Linear calculation
Load case: 40psf Wind Load
Coordinate system: Global
Extreme 1D: Member
Selection: All



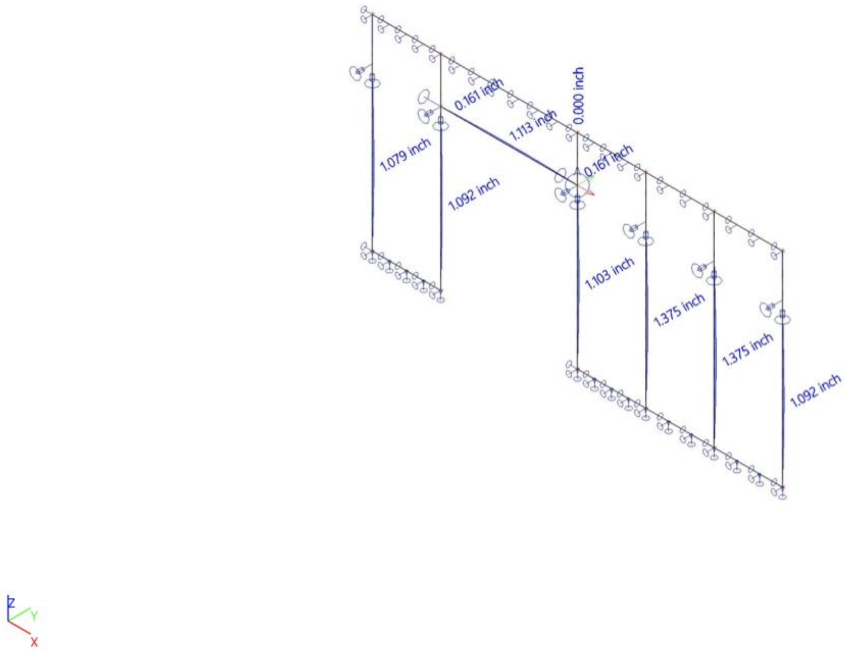
2. 1D internal forces; M_z

Values: $\mathbf{M_z}$
Linear calculation
Load case: 40psf Wind Load
Coordinate system: Principal
Extreme 1D: Member
Selection: All



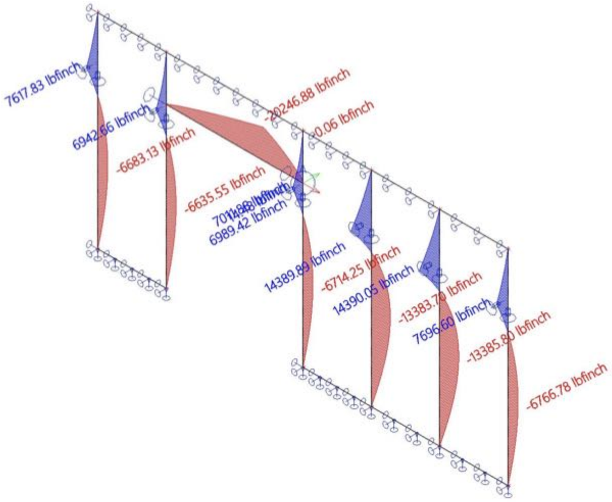
3. 1D deformations; U_total

Values: U_{total}
Linear calculation
Load case: 40psf Wind Load
Coordinate system: Global
Extreme 1D: Member
Selection: All



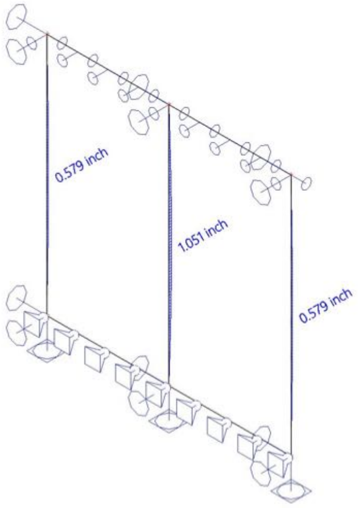
4. 1D internal forces; M_z

Values: M_z
Linear calculation
Load case: 40psf Wind Load
Coordinate system: Principal
Extreme 1D: Member
Selection: All



5. 1D deformations; U_total

Values: **U_{total}**
Linear calculation
Load case: 40psf Wind Load
Coordinate system: Global
Extreme 1D: Member
Selection: All



6. 1D internal forces; M_z

Values: **M_z**
Linear calculation
Load case: 40psf Wind Load
Coordinate system: Principal
Extreme 1D: Member
Selection: All