Frameless Hardware Company LLC
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SUBJ: FHC ADVANCE SERIES FRAMELESS GLASS ENTRANCE SYSTEMS
ENGINEERING REPORT AND WIND LOAD CHARTS

The FHC advance series frameless glass entrance systems utilize aluminum extrusions to construct glass entrances. The system is intended for interior and exterior weather exposed applications.

The system will meet all applicable requirements of the 2015 and 2018 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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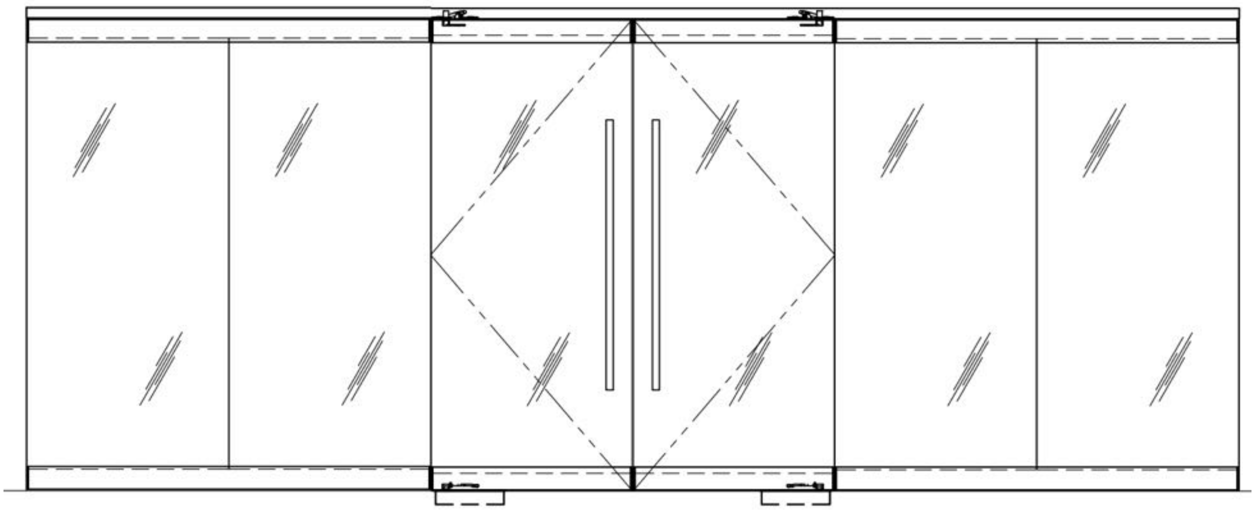
TYPICAL INSTALLATIONS

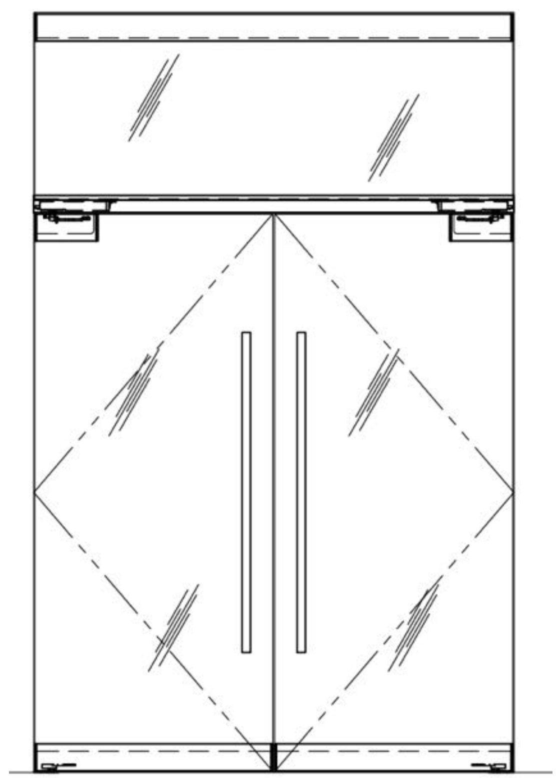
No Header



Door hinges attach directly to the structure and the side lites are unaffected by loading on the door.

Header With No Side Lites

The header supports wind load reactions from the door hinges as well as dead load and wind load reactions from the transom. The ends of the header are attached directly to the structure.



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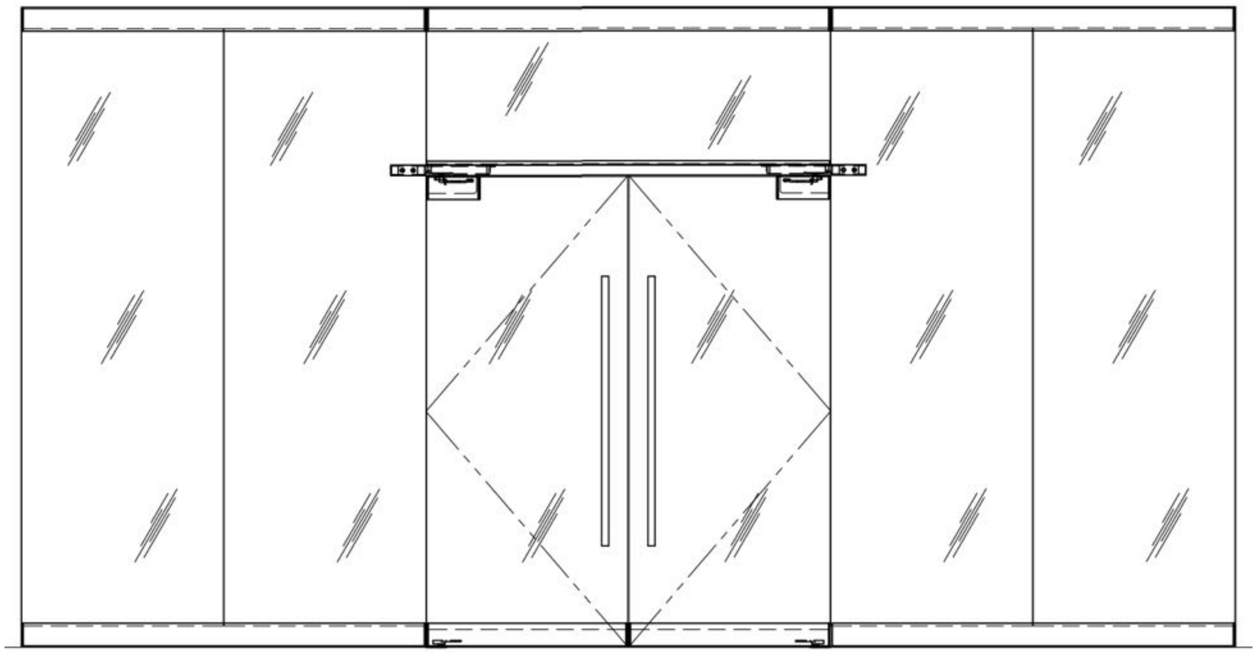
Header With Side Lites



The header supports wind load reactions from the door hinges as well as dead load and wind load reactions from the transom. The ends of the header are attached to the side lites which carry the reactions to the structure. The side lites must be checked for the point load condition.

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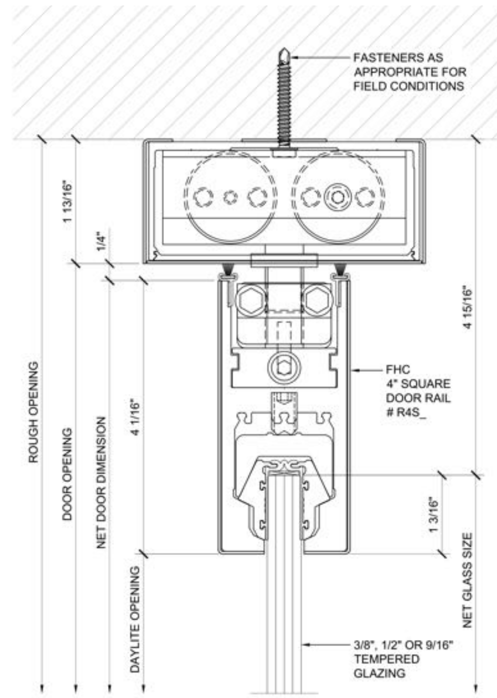
CLAMP AND SPINDLE

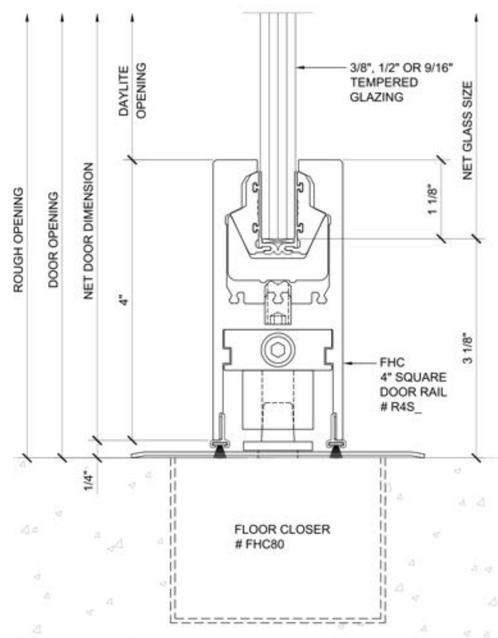


Check spindle strength:
Assume no door stop at latch side of door. The spindle is the only connection for the door at the head and receives half the total wind loading on the door.

Spindle diameter = 1/2”

$V_a = A_v(0.6F_u/\Omega) = 0.224\text{in}^2 * 0.6 * 75\text{ksi}/2 = 5{,}040\#$
(Will not control system strength)

Check connection of glass to rail:
Connection carries some moment due to the eccentricity of the rail.
Glass bite = 1”
Allowable moment in leg =
$M_a = 1” * 0.194”^2/4 * 15.2\text{ksi} = 143”\#/“$
Moment carried by connection
$M = V * 3.6”$
(Where V is the reaction from the glass in pli)
Reaction to leg
$R = V * 3.6”/(1” * 2/3) = V * 5.4$
Max moment in leg
$M_{max} = V * 5.4 * 1.34” < 143”\#/“$
$V < 19.8\text{pli} = 237\text{plf}$



Check anchorage of pivots:
Loading from the doors will be concentrated at the pivots and door stops or locks. For most doors one corner is left unsupported. The three support scenario means that the pivot and door stop can receive up to half the total wind pressure on the door.
Max reaction,
$R = Pwh/2$
(where P is the wind pressure, w is the width of the door and h is the height of the door.)

By inspection the core mounted floor closer will have greater load resistance than the fasteners shown at the head condition. Fasteners for different substrates are described on the following pages.

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DOOR ANCHORAGE TO STEEL

Recall, R = Pwh/2 (where P is the wind pressure, w is the width of the door and h is the height of the door.)

Loading on screws:

V = R

T = R*1.75”/2”

Loading is concentrated at the spindle, lock or door stop. Place (2) 1/4” hex head fasteners at each location.

Recommended fastener 1/4” hex head Tek screw. Tek screw strength is according to ESR 1976 and ADM 2020. Assume 0.06” minimum steel backing.

Tek screw strength per ESR 1976:
Rupture in shear, V_a = 990#
Rupture in tension, T_a = 1,605#
Hole bearing, V_a = 463#
Pullout, T_a = 191#

Tek screw strength into aluminum per ADM 2020:
Hole bearing, V_a = 2*0.25”*0.09”*30ksi/3 = 450#
Pullover, T_a = (0.5”-0.209”)*0.09”*25ksi/3 = 218#

Controlling strength for two screws:
V_a = 2*450# = 900#
T_a = 2*191# = 382#

Allowable reaction:
Tension strength controls since $V_a > 2/1.75*T_a$
R_a = 382#*2”/1.75” = 437#
Allowable wind pressure, P_a = 2*437#/(wh) = 874#/(wh)

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DOOR ANCHORAGE TO CONCRETE

Recall, R = Pwh/2 (where P is the wind pressure, w is the width of the door and h is the height of the door.)

Loading on screws:

V = R

T = R*1.75"/2"

Loading is concentrated at the spindle, lock or door stop. Place (2) 1/4" hex head fasteners at each location.

Recommended fastener (2) 1/4" hex head Tapcon with 2" nominal embedment in concrete. Assumed concrete strength is 2,500# minimum. Assume minimum edge distance is 2" and anchor spacing is 3".

Anchor strength is according to ESR 3699, ACI 318-19 and ADM 2020.

Concrete Anchor Strength

Calculate strength according to ACI 318-19 Chapter 17.

Anchor Description

(2) 1/4" Tapcon

Nominal Pullout Strength at f'c=2500psi

857

Anchor Pattern	n	Spacing
Parallel to edge	2	3
Perpendicular to edge	1	0

Assumed Values

hef	Ca1	Ca2	Ca3	Ca4
1.45	2	6	6	6

Cast or Post	Conc Depth (in)	Cracked/Uncracked	Splitting Reinforcement
Post	4	Cracked	No

le	Da
1.45	0.25

λ	f'c	Cac
1	2500	N/A

Imposed loads:

T (lbs)	V (lbs)	e'n (in)	e'v (in)
306.25	350	0	0

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1.5hef	Camin
2.175	2

Concrete Breakout Strength:

Anc	Anco
30.68625	18.9225

Ψed,N	Ψc,N	Ψcp,N	Kc	Ψec,N
0.97586207	1	1	17	1

Nb	Ncbg
1484.12653	2348.68445

Side Face Breakout Strength:

Avc	Avco
27	18

Ψed,V	Ψc,V	Ψh,V	Ψec,V
1	1	1	1

Vb	Vcbg
703.507353	1055.26103

Pryout Strength:

Kcp
1

Vcpg
2348.68445

Area Calcs:

	Anc			Avc	
Segment:	W	B		H	B
1	2	2.175		3	3
2	0	3			3
3	2.175	2.175			3
Total:	**4.175**	**7.35**		**3**	**9**

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

All tens

696.3125

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

All V

461.6767

Interaction:

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

V/Va+T/Ta= 1.19792316

Anchor Adequacy:

PASS

Using the spreadsheet shown above, it was determined maximum utilization of the anchor is achieved at V = 350# which results in a simultaneous load of T = 350#*1.75"/2" = 306#.

Check aluminum failure modes per ADM 2020:
Strength calculations include both anchors.
Hole bearing, V_a = 2*2*0.25"*0.09"*30ksi/3 = 900# > 350# does not control
Pullover, T_a = 2*(0.5"-0.25")*0.09"*25ksi/3 = 375# > 306# does not control

Allowable reaction, R = 350#
Allowable wind pressure, P = 350#*2/(wh) = 700#/(wh)

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DOOR ANCHORAGE TO CMU

Recall, R = Pwh/2 (where P is the wind pressure, w is the width of the door and h is the height of the door.)
Loading on screws:

V = R
T = R*1.75"/2"

Loading is concentrated at the spindle, lock or door stop. Place (2) 1/4" hex head fasteners at each location.

Recommended fastener (2) Dewalt Ultracon anchors. Assume 1500psi CMU strength, 2" edge distance and 4" spacing.

CMU failure modes are considered according to ESR 3196.
V_a = 2*155# = 310#
T_a = 2*165# = 330#
R/310#+(R*1.75"/2")/330# < 1

R < 170#

Check aluminum failure modes per ADM 2020:
Strength calculations include both anchors.
Hole bearing, V_a = 2*2*0.25"*0.09"*30ksi/3 = 900# > 310# does not control
Pullover, T_a = 2*(0.5"-0.25")*0.09"*25ksi/3 = 375# > 330# does not control

Allowable wind pressure, P = 170#*2/(wh) = 340#/(wh)

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DOOR ANCHORAGE TO WOOD

Recall, R = Pwh/2 (where P is the wind pressure, w is the width of the door and h is the height of the door.)
Loading on screws:

V = R
T = R*1.75”/2”

Loading is concentrated at the spindle, lock or door stop. Place (2) 1/4” hex head fasteners at each location.

Recommended fastener (2) 1/4” hex head wood screws with 1.5” penetration in structurally supported wood.

Wood failure modes are considered according to the 2015 NDS.
Assume G = 0.43.
W’p = 1.6*127pli*1.5” = 305# each

D (in)	Dr(in)	Fyb	Fem	Fes	Lm	Ls	θ
0.242	0.196	70000	3500	60000	1.5	0.09	90

Re	Rt	k1	k2	k3	kθ	Rd 1	Rd 2	Rd 3/4
0.05833333	16.6666667	0.40335641	0.539765	11.901431	1.25	2.46	2.46	2.46

Single Shear

1m	1s	2	3m	3s	4
418.292683	430.243902	173.541634	202.19082	145.115898	194.000662

Z’ = 1.6*145# = 232# each
Also check combined forces:

$\alpha = \tan^{-1}((1.75/2V)/V) = 41.2^\circ$
$Z_\alpha' = 305\#*232\#/(305\#*\cos^2 41.2^\circ+232\#*\sin^2 41.2^\circ) = 259\#$
$z_\alpha = (R^2+(R*1.75/2)^2)^{1/2} = 1.20*R$
For two screws, $R_a = 259\#*2/1.20 = 432\#$

Hole bearing, V_a = 2*0.25”*0.09”*30ksi/3 = 450# > 232# does not control
Pullover, T_a = (0.5”-0.25”)*0.09”*25ksi/3 = 188# < 305# controls

Allowable reaction based on combined shear strength = 2*232# = 464#
Allowable reaction based on combined shear and tension wood failure = 432#
Allowable reaction based on combined tension strength = 2*188#/(1.75”/2”) = 430# (controls)
R_a = 430#

Allowable wind pressure, P = 430#*2/(wh) = 860#/(wh)

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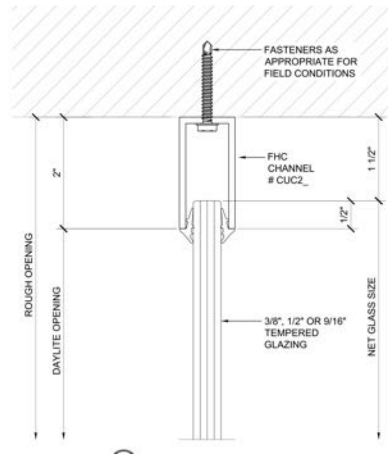
HEAD CHANNEL CONNECTION



Leg thickness = 0.125”
M_a = 12”*0.125”^{2}/4*15.2ksi = 713”#/ft
Allowable reaction = 713”#/ft/(2”-0.125”) = 380plf

Anchorage to steel:
#10 SMS at 12” O.C.
Assume 0.048” thick steel backing.
Per ESR 1976:
V_a = 289#
T_a = 116#
From ADM 2020:
V_a = 2*0.19”*0.125”*30ksi/3 = 475#
T_a = (0.40”-0.159”)*0.125”*30ksi/3 = 301#

uniform reaction = w
w = hP/2 (Where h_t is the height of the transom or sidelite and P is the wind pressure)
V = w*1’
T = w*1’*2”/0.5” = 4w
$T_a < 4V_a$ so tension loading on fastener controls
w_a = 301#/4 = 75.3plf
P_a = 75.3plf*2/h

Anchorage to CMU:
Use 3/16” Dewalt Ultracon
CMU failure modes are considered according to ESR 3196.
V_a = 100#
T_a = 90#
R/100#+4R/90# < 1
R < 18.4plf
P_a = 2*18.4plf/h

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Anchorage to Wood:
#10 wood screw at 12" O.C. with 1.5" penetration in structurally supported wood.
From ADM 2020:
V_a = 2*0.19"*0.125"*30ksi/3 = 475#
T_a = (0.40"-0.19")*0.125"*30ksi/3 = 263#
Wood failure modes are considered according to the 2015 NDS.
Assume G=0.43.
W'p = 1.6*100pli*1.5" = 240# each

D (in)	Dr(in)	Fyb	Fem	Fes	Lm	Ls	θ
0.19	0.152	80000	3500	60000	1.5	0.125	90

Re	Rt	k1	k2	k3	kθ	Rd 1	Rd 2	Rd 3/4
0.05833333	12	0.29910883	0.51373486	8.09196772	1.25	2.2	2.2	2.2

Single Shear

1m	1s	2	3m	3s	4
362.727273	518.181818	154.992756	166.876695	118.833092	139.471876

Z' = 1.6*119# = 190# each
Also check combined forces:

$\alpha = \tan^{-1}((4V)/V) = 76.0^\circ$

$Z_\alpha' = 240\#*190\#/(240\#*\cos^2 76.0^\circ+190\#*\sin^2 76.0^\circ) = 236\#$

$z_\alpha = (R^2+(R*4)^2)^{1/2} = 4.12*R$

For two screws, w_a = 236#/(1'*4.12) = 57.3plf

P_a = 57.3plf*2/h

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Anchorage to concrete:
Use 3/16 (0.19”) Tapcon at 12” O.C.

Concrete Anchor Strength

Calculate strength according to ACI 318-19 Chapter 17.

Anchor Description

3/16" Tapcon

Nominal Pullout Strength at f'c=2500psi

590

Anchor Pattern

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

Assumed Values

hef	Ca1	Ca2	Ca3	Ca4
1.5	2	6	6	6

Cast or Post	Conc Depth (in)	Cracked/Uncracked	Splitting Reinforcement
Post	4	Cracked	No

le	Da
1.5	0.25

λ	f'c	Cac
1	2500	N/A

Imposed loads:

T (lbs)	V (lbs)	e'n (in)	e'v (in)
239.6	59.9	0	0

1.5hef	Camin
2.25	2

Concrete Breakout Strength:

Anc	Anco
19.125	20.25

Ψed,N	Ψc,N	Ψcp,N	Kc	Ψec,N
0.96666667	1	1	17	1

Nb	Ncbg
1561.54971	1425.63705

Side Face Breakout Strength:

Avc	Avco
18	18

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Ψed,V	Ψc,V	Ψh,V	Ψec,V
1	1	1	1

Vb	Vcbg
708.293559	708.293559

Pryout Strength:

Kcp
1

Vcpg
1425.63705

Area Calcs:

	Anc			Avc	
Segment:	W	B		H	B
1	2	2.25		3	3
2	0	0			0
3	2.25	2.25			3
Total:	**4.25**	**4.5**		**3**	**6**

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

All tens
239.6875

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

All V
309.878432

Interaction:

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

V/Va+T/Ta= 1.19293655

Anchor Adequacy:

PASS

From the above spreadsheet, it was determined the maximum allowable loading for concrete failure modes is w = 59.9plf.

P_a = 59.9plf*2/h

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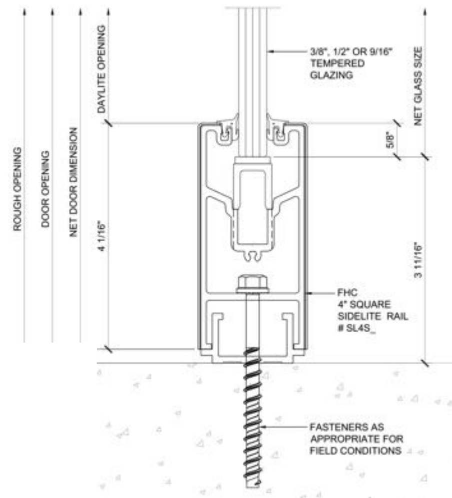

SIDELITE SILL ANCHORAGE
To Concrete

Assume 1/4" Hilti KH-EZ anchors at 12" O.C.. Specify (1.62"+2.5") = 4.12" => 4-1/2" long anchors. This creates at least 2-1/2" nominal embedment and 1.92" effective embedment.

Anchor design is according to ESR 3027 and ACI 318-19.

Concrete Anchor Strength

Calculate strength according to ACI 318-19 Chapter 17.

Anchor Description

1/4" KH-EZ

Nominal Pullout Strength at f'c=2500psi

1165

Anchor Pattern

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

Assumed Values

hef	Ca1	Ca2	Ca3	Ca4
1.92	2	12	12	12

Cast or Post	Conc Depth (in)	Cracked/Uncracked	Splitting Reinforcement
Post	8	Cracked	No

le	Da
1.92	0.25

λ	f'c	Cac
1	3000	N/A

1.5hef	Camin
2.88	2

Concrete Breakout Strength:

Anc	Anco
28.1088	33.1776

$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	Kc	$\Psi_{ec,N}$
0.90833333	1	1	17	1

Nb	Ncbg
2477.20183	1906.3559

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Side Face Breakout Strength:

Avc	Avco
18	18

Ψed,V	Ψc,V	Ψh,V	Ψec,V
1	1	1	1

Vb	Vcbg
815.165722	815.165722

Pryout Strength:

Kcp
1

Vcpg
1906.3559

Area Calcs:

	Anc			Avc		
Segment:	W	B		H	B	
1	2	2.88			3	3
2	0	0				0
3	2.88	2.88				3
Total:	**4.88**	**5.76**			**3**	**6**

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

All tens
518.453633

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

All V
356.635003

$T_a = 518\#$
$V_a = 357\#$

$V = R$
$T = R*3.75''/0.625'' = 6R$
$T_a < 6V_a$ so tension strength controls over shear strength.
$R_a = 518\#/6 = 86.3\#$
Or for combined forces, $R/357\# + 6R/518\# < 1.2$
$R_a = 83.4\#$

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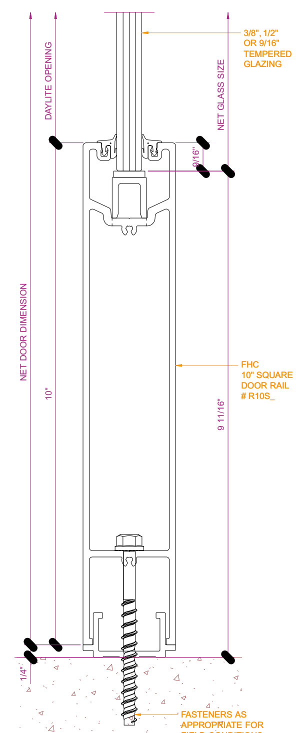
Pullover,
T_a = 0.7*0.125"*(0.56"-0.25")*30ksi/2 = 407#
R_a = 407#/6 = 67.8# (controls)

Aluminum leg bending,
M_a =0.125"^{2}*12"/4*15.2ksi = 713"#/ft
M = 67.8#*1" = 67.8"# << 713"#/ft (Fastener strength controls)
Allowable reaction is 67.8plf.
P_a = 67.8plf/(H/2)

Or check 6" tall rails:
V = R
T = R*5.75"/0.625" = 9.2R
T_a < 9.2V_a so tension strength controls over shear strength.
R_a = 518#/9.2 = 56.3#

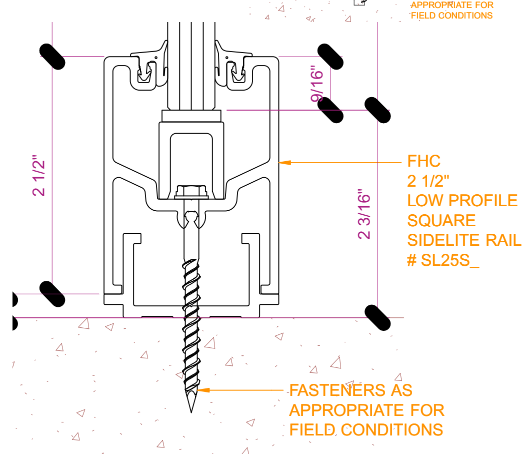


Or for combined forces,
R/357#+9.2R/518# < 1.2
R_a = 58.4#

Pullover,
T_a = 0.7*0.125"*(0.56"-0.25")*30ksi/2 = 407#
R_a = 407#/9.2 = 44.2# (controls)

Aluminum leg bending,
M_a =0.125"^{2}*12"/4*15.2ksi = 713"#/ft
M = 44.2#*4.75"/2 = 105"# << 713"#/ft (Fastener strength controls)
Allowable reaction is 44.2plf.
P_a = 44.2plf/(H/2)

10" tall rails:
V = R
T = R*9.75"/0.625" = 15.6R
Tension only, R_a = 407#/15.6 = 26.1#
Combined = R/357#+15.6R/518# < 1.2
R_a = 36.5#
P_a = 26.1plf/(H/2)



2-1/2" tall rails:
V = R
T = R*2.25"/0.625" = 3.6R
Tension only, R_a = 407#/3.6 = 113#
Combined = R/357#+3.6R/518# < 1.2
R_a = 123#
P_a = 123plf/(H/2)

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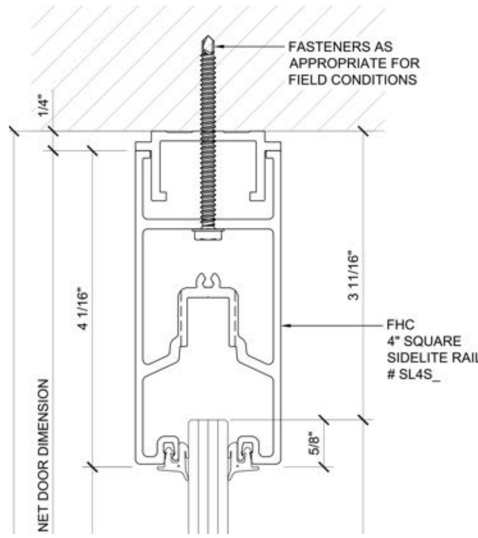
SIDE LITE HEAD ANCHORAGE
Assume anchorage is to wood or cold formed steel.



Anchorage to steel:
1/4" SMS at 12" O.C.
Assume 0.048" thick steel backing.
Per ESR 1976:
V_a = 463# T_a = 153#
From ADM 2020 for aluminum parts:
V_a = 2*0.25"*0.125"*30ksi/3 = 475#
T_a = (0.50"-0.213")*0.125"*30ksi/3 = 359#

V = R
T = R*3.75"/0.625" = 6R
$T_a < 6V_a$ so tension strength controls over shear strength.
R_a = 153#/6 = 25.5#
Aluminum leg bending, M
$_a$ =0.125"^{2}*12"/4*15.2ksi = 713"#/ft
R_a = 713"#/ft/1" = 713plf
Allowable reaction is 25.5plf.
P_a = 25.5plf/(H/2)

Or check 6" tall rails:
V = R
T = R*5.75"/0.625" = 9.2R
$T_a < 9.2V_a$ so tension strength controls over shear strength.
R_a = 153#/9.2 = 16.6#
Aluminum leg bending,
M_a =0.125"^{2}*12"/4*15.2ksi = 713"#/ft
R_a = 713"#/ft/(4.75"/2) = 300plf
Allowable reaction is 16.6plf.
P_a = 16.6plf/(H/2)

10" tall rails:
V = R
T = R*9.75"/0.625" = 15.6R
$T_a < 15.6V_a$ so tension strength controls over shear strength.
R_a = 153#/15.6 = 9.81#
Aluminum leg bending, M_a =0.125"^{2}*12"/4*15.2ksi = 713"#/ft
R_a = 713"#/ft/(8.75"/2) = 163plf
Allowable reaction is 9.81plf.
P_a = 9.81plf/(H/2)

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2-1/2” tall rails:
V = R
T = R*2.25”/0.625” = 3.6R
$T_a < 3.6V_a$ so tension strength controls over shear strength.
R_a = 153#/3.6 = 42.5#
P_a = 42.5plf/(H/2)

Anchorage to Wood:
1/4” wood screw at 12” O.C. with 1.5” penetration in structurally supported wood.
From ADM 2020:
V_a = 2*0.25”*0.125”*30ksi/3 = 625#
T_a = (0.50”-0.213”)*0.125”*30ksi/3 = 359#
Wood failure modes are considered according to the 2015 NDS.
Assume G = 0.43.
W’p = 1.6*127pli*1.5” = 305# each

D (in)	Dr(in)	Fyb	Fem	Fes	Lm	Ls	θ
0.242	0.196	70000	3600	60000	1.5	0.125	90

Re	Rt	k1	k2	k3	kθ	Rd 1	Rd 2	Rd 3/4
0.06	12	0.30680823	0.53879367	9.04926366	1.25	2.46	2.46	2.46

Single Shear

1m	1s	2	3m	3s	4
430.243902	597.560976	183.336624	206.975617	157.499616	196.597841

Z’ = 1.6*157# = 251# each
Also check combined forces:

$\alpha = \tan^{-1}((6V)/V) = 80.5°$

$Z_\alpha' = 305\#*251\#/(305\#*\cos^2 80.5° + 251\#*\sin^2 80.5°) = 303\#$

$z_\alpha = (R^2+(R*6)^2)^{1/2} = 6.08*R$

w_a = 303#/(1’*6.08) = 49.8plf
Or tension only,
w_a = 305#/6 = 50.8plf
P_a = 49.8plf*2/h

Or check 6” side rail:
V = R
T = R*5.75”/0.625” = 9.2R
$\alpha = \tan^{-1}((9.2V)/V) = 83.8°$
$Z_\alpha' = 305\#*251\#/(305\#*\cos^2 83.8° + 251\#*\sin^2 83.8°) = 304\#$
$z_\alpha = (R^2+(R*9.2)^2)^{1/2} = 9.25*R$

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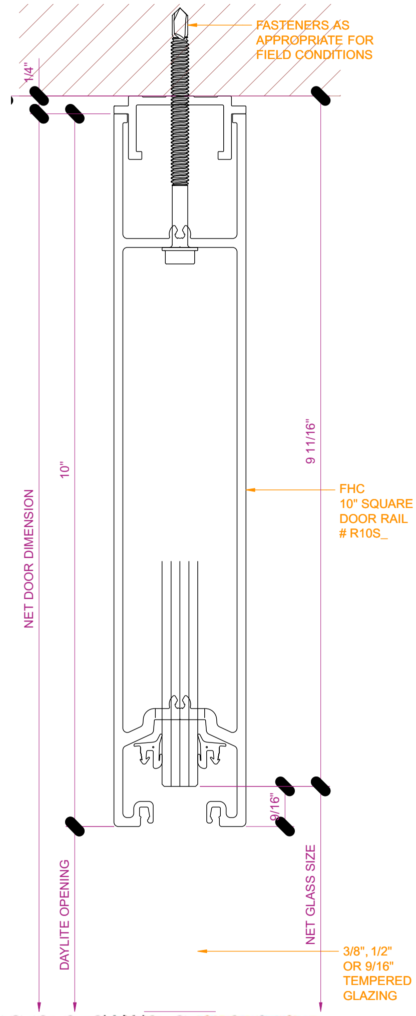
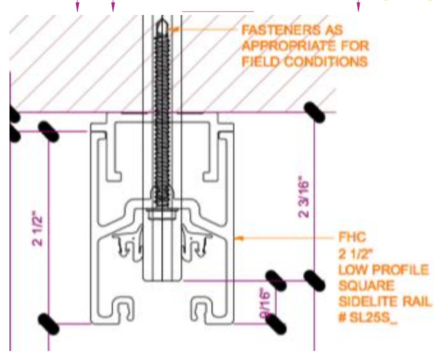
$w_a = 304\#/(1'*9.25) = 32.9plf$
Or tension only, $w_a = 305\#/9.2 = 33.2plf$
$P_a = 32.9plf*2/h$

10" side rail:
$V = R$
$T = R*9.75''/0.625'' = 15.6R$
$\alpha = \tan^{-1}((15.6V)/V) = 86.3°$
$Z_\alpha' = 305\#*251\#/(305\#*\cos^2 86.3°+251\#*\sin^2 86.3°) = 305\#$
$z_\alpha = (R^2+(R*9.2)^2)^{1/2} = 9.25*R$
$w_a = 305\#/(1'*9.25) = 33.0plf$
Or tension only, $w_a = 305\#/15.6 = 19.6plf$
$P_a = 19.6plf*2/h$

2-1/2" side rail:
$V = R$
$T = R*2.25''/0.625'' = 3.6R$
$\alpha = \tan^{-1}((3.6V)/V) = 74.5°$
$Z_\alpha' = 305\#*251\#/(305\#*\cos^2 74.5°+251\#*\sin^2 74.5°) = 300\#$
$z_\alpha = (R^2+(R*3.6)^2)^{1/2} = 3.74*R$
$w_a = 300\#/(1'*3.74) = 80.2plf$
Or tension only, $w_a = 305\#/3.6 = 84.7plf$
$P_a = 80.2plf*2/h$





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ANCHORAGE WIND LOAD TABLES

The fastener strength at 12” O.C. is less than half of the leg bending strength for any of the substrates. Therefore, the allowable wind load can be doubled by using the same fasteners at 6” O.C. These wind load tables do not consider the glass which is considered on the following page. H in the tables is taken as the total wall height including the rails. Allowable wind loads are similar to the glass so the glass should also be checked. When using the 6” tall rail, fasteners at 6” O.C. will typically be desired.

Glazing Channel Allowable Wind Load (PSF)

Anchorage	Spacing (in)	H (in)					
		84	96	102	108	114	120
Wood	12	16.4	14.3	13.5	12.7	12.1	11.5
Wood	6	32.7	28.7	27.0	25.5	24.1	22.9
Steel	12	21.5	18.8	17.7	16.7	15.9	15.1
Steel	6	43.0	37.7	35.4	33.5	31.7	30.1
Concrete	12	17.1	15.0	14.1	13.3	12.6	12.0
Concrete	6	34.2	30.0	28.2	26.6	25.2	24.0

2-1/2" Rail Allowable Wind Load (PSF)

Anchorage	Spacing (in)	H (in)					
		84	96	102	108	114	120
Wood	12	22.9	20.1	18.9	17.8	16.9	16.0
Wood	6	45.8	40.1	37.7	35.6	33.8	32.1
Steel	12	12.1	10.6	10.0	9.4	8.9	8.5
Steel	6	24.3	21.3	20.0	18.9	17.9	17.0
Concrete	12	35.1	30.8	28.9	27.3	25.9	24.6
Concrete	6	70.3	61.5	57.9	54.7	51.8	49.2

4" Rail Allowable Wind Load (PSF)

Anchorage	Spacing (in)	H (in)					
		84	96	102	108	114	120
Wood	12	14.2	12.5	11.7	11.1	10.5	10.0
Wood	6	28.5	24.9	23.4	22.1	21.0	19.9
Steel	12	7.3	6.4	6.0	5.7	5.4	5.1
Steel	6	14.6	12.8	12.0	11.3	10.7	10.2
Concrete	12	19.4	17.0	16.0	15.1	14.3	13.6
Concrete	6	38.7	33.9	31.9	30.1	28.5	27.1

6" Rail Allowable Wind Load (PSF)

Anchorage	Spacing (in)	H (in)					
		84	96	102	108	114	120
Wood	12	9.4	8.2	7.7	7.3	6.9	6.6
Wood	6	18.8	16.5	15.5	14.6	13.9	13.2
Steel	12	4.7	4.2	3.9	3.7	3.5	3.3
Steel	6	9.5	8.3	7.8	7.4	7.0	6.6
Concrete	12	12.6	11.1	10.4	9.8	9.3	8.8
Concrete	6	25.3	22.1	20.8	19.6	18.6	17.7

10" Rail Allowable Wind Load (PSF)

Anchorage	Spacing (in)	H (in)					
		84	96	102	108	114	120
Wood	6	11.2	9.8	9.2	8.7	8.3	7.8
Wood	4	16.8	14.7	13.8	13.1	12.4	11.8
Steel	6	5.6	4.9	4.6	4.4	4.1	3.9
Steel	4	8.4	7.4	6.9	6.5	6.2	5.9
Concrete	6	14.9	13.1	12.3	11.6	11.0	10.4
Concrete	4	22.4	19.6	18.4	17.4	16.5	15.7

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SIDE LITE GLASS

All glass is fully tempered with a mean modulus of rupture of 24,000psi. Typical glass thicknesses are 3/8” monolithic, 1/2” monolithic and 9/16” laminated. The 9/16” laminated glass is assumed to be built up from two plies of 1/4” glass and a 0.06” PVB interlayer.

The effective properties for modeling are calculated according to the appendix of ASTM E1300:

Laminated Glass Effective Thickness				
h1	h2	hv	E	g
0.219	0.219	0.06	10400000	350
hs	hs;1	hs;2	Is	
0.279	0.1395	0.1395	0.00852359	
a	Γ	hef;w	h1;ef;σ	h2;ef;σ
48	0.551441626	0.426185961	0.455648921	0.455648921

Variable	Description
H1 & H2	Glass pane thicknesses
Hv	Interlayer thickness
E	Young's Modulus
g	Shear Modulus
Hs	.5(h1+h2)+hv
Hs;1	hsh1/(h1+h2)
Hs;1	hsh2/(h1+h2)
Is	$h1(hs;2)^2+h2(hs;1)^2$
a	Minimum Pane Width
Γ	$1/(1+9.6(Eishv/(G(ahs)^2))$
hef;w	$\sqrt[3]{((h1)^3+(h2)^3+12\Gamma Is)}$
h1;ef;σ	$\sqrt{((hef;w)^3/(h1+2\Gamma hs;2))}$
h2;ef;σ	$\sqrt{((hef;w)^3/(h2+2\Gamma hs;1))}$

Failure criteria:
$\Delta < L/60$
Stress < 10,600psi

$\Delta = 5P/12*H_g^4/(384*10.4*10^6*t_g^3)$
$M = P/12*H_g^2/8$
$M_a = 10600psi*2*t_g^2 = 21{,}200psi(t_g)^2$
Therefore, P_a is the the lesser of $(384*10.4*10^6*t_g^3)*(H_g/60)/(5/12*H_g^4)$ or $21100t_g^2*12*8/H_g^2$. The preceding equations are used to create the allowable wind load table below. Note that this table does not account for anchorage failure.

Allowable Wind Load (PSF)							
tg,nominal (in)	tg (in)	Hg (in)					
		72	76	96	100	108	112
3/8"	0.355	19.1	16.3	8.1	7.1	5.7	5.1
1/2"	0.469	44.2	37.5	18.6	16.5	13.1	11.7
9/16"	0.426	33.1	28.1	14.0	12.3	9.8	8.8

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Laminated glass is assumed to be two plies of 1/4" tempered glass with a 0.06" PVB interlayer. The effective properties for modeling are calculated according to the appendix of ASTM E1300:

Laminated Glass Effective Thickness

h1	h2	hv	E	g
0.219	0.219	0.06	10400000	350
hs	hs;1	hs;2	Is	
0.279	0.1395	0.1395	0.00852359	
a	Γ	hef;w	h1;ef;σ	h2;ef;σ
48	0.551441626	0.426185961	0.455648921	0.455648921

Variable	Description
H1 & H2	Glass pane thicknesses
Hv	Interlayer thickness
E	Young's Modulus
g	Shear Modulus
Hs	.5(h1+h2)+hv
Hs;1	hsh1/(h1+h2)
Hs;1	hsh2/(h1+h2)
Is	$h1(hs;2)^2+h2(hs;1)^2$
a	Minimum Pane Width
Γ	$1/(1+9.6(Eishv/(G(ahs)^2))$
hef;w	$\sqrt[3]{(h1)^3+(h2)^3+12\Gamma Is)}$
h1;ef;σ	$\sqrt{((hef;w)^3/(h1+2\Gamma hs;2))}$
h2;ef;σ	$\sqrt{((hef;w)^3/(h2+2\Gamma hs;1))}$

Failure criteria:
$\Delta < L/60$
$\Delta_{slip} < 1/4"$
Stress < 10,600psi

Modeling indicates the system will accommodate interior or low exterior wind pressures. Generally, 9/16" laminated or 1/2" monolithic will be desired. 9/16" laminated and 1/2" monolithic pass all design checks for a 10ft wall with 5psf wind pressure. Excessive deflection may occur at higher wall heights. Deflection will be the controlling design concern, stresses do not reach capacity until the glass has deflected far more than desired. Also note that the $\Delta<L/60$ criteria is always met at a lower wind load that the $\Delta_{slip}<1/4"$ requirement. Therefore, it can be accurately assumed that slip is OK when the L/60 deflection criteria is met. Results are shown in table and graph form on the following pages. Result diagrams from SCIA engineer are then provided. The shown results assume 1/2" monolithic glass.

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FEA Testing Results for 8' Door with 2' Transom

Out of Plane Deflection (in)

Pressure (psf)	Glass Thickness (in) 3/8"	9/16"	1/2"
5psf	3.04	2.04	1.294
10psf	5.85	3.95	2.517
25psf	12.7	9.09	5.957

In Plane Slip at Head (in)

Pressure (psf)	Glass Thickness (in) 3/8"	9/16"	1/2"
5psf	0.161	0.061	0.016
10psf	0.662	0.279	0.095
25psf	3.34	1.65	0.663

Max Glass Stress (psi)

ure	Glass Thickness (in) 3/8"	9/16"	1/2"
5psf	5680	4260	3050

FEA Testing Results for 8' Door with 4' Transom

Out of Plane Deflection (in)

Pressure (psf)	Glass Thickness (in) 3/8"	9/16"	1/2"
5psf	7.1	4.77	3.03
10psf	13.2	9.14	5.86
25psf	25.2	19.4	13.4

In Plane Slip at Head (in)

Pressure (psf)	Glass Thickness (in) 3/8"	9/16"	1/2"
5psf	0.808	0.343	0.121
10psf	2.953	1.36	0.522
25psf	11.4	6.52	3

Max Glass Stress (psi)

Pressure (psf)	Glass Thickness (in) 3/8"	9/16"	1/2"
5psf	9650	7410	5370
10psf	18300	14400	10700
25psf	38500	32200	25200

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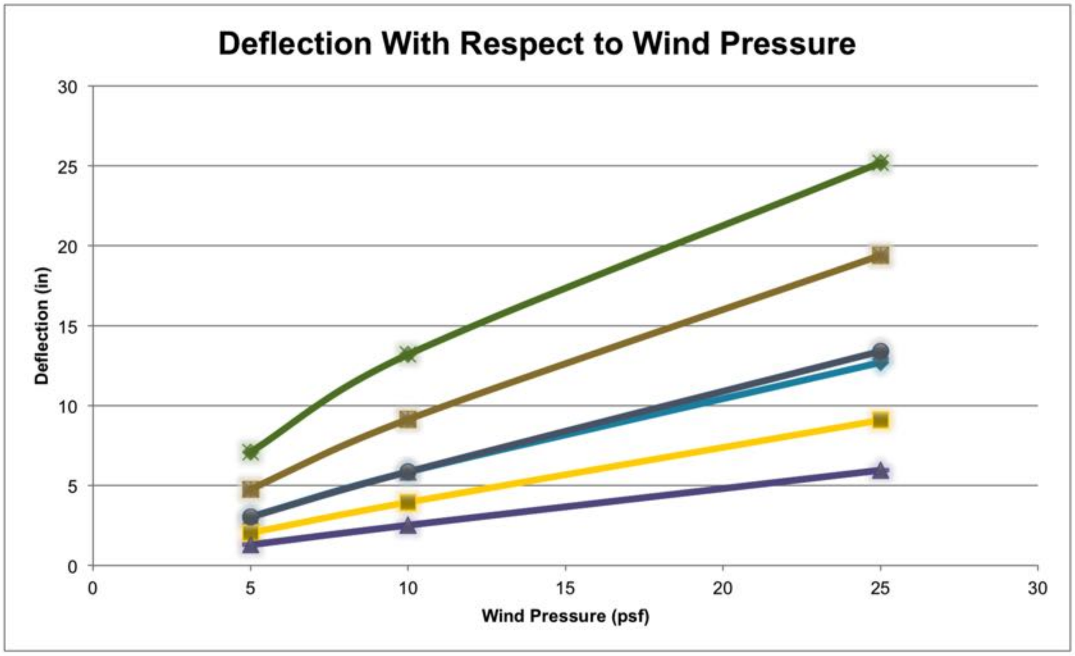

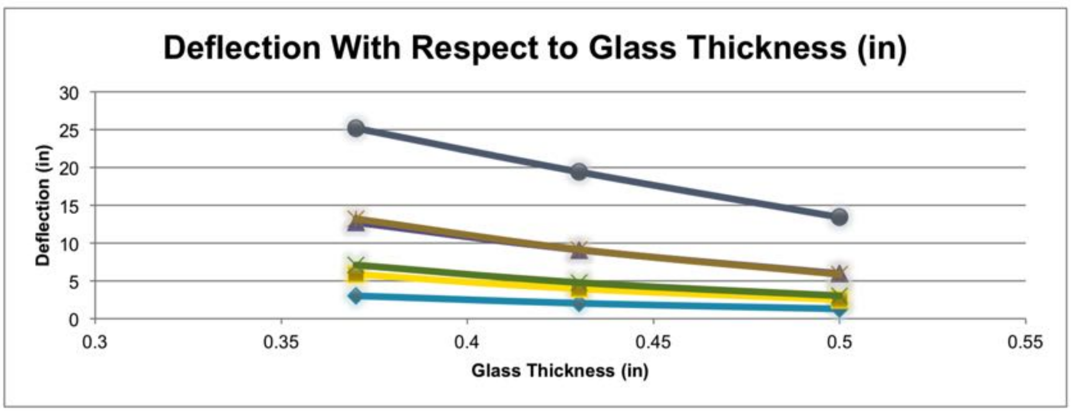

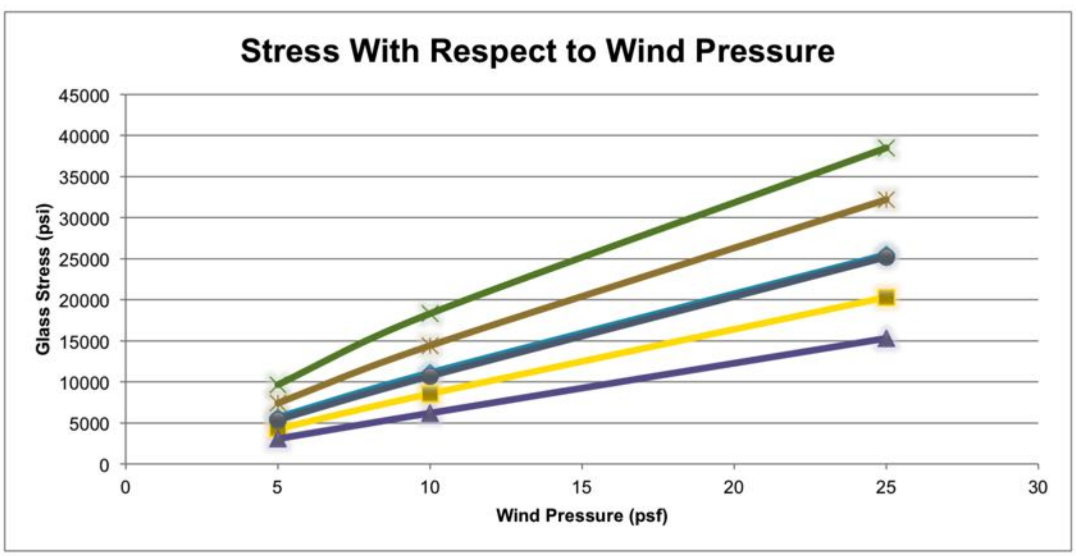

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FLOATING HEADER DETAILS

The floating header bolts to the side lite glass or uses brackets to attach to the wall jamb.

First check the floating header in bending.
Aluminum extrusion properties:
$I_x = 0.2789in^4$
$S_x = 0.2140in^3$
b/t = 4.24"/0.09" = 47.1 > 38
$F_c/\Omega = 484/47.1 = 10.3$ksi for compression element.
However, distance from centroid to extreme compression element, $c_c = 0.447$" while distance to extreme tension element, $c_t = 1.303$".
$F_t/\Omega = 15.2$ksi for tension element
$(F_t/\Omega)/(F_c/\Omega) < c_t/c_c$ Therefore, the strength is controlled by yielding of the tension element.
$M_{a,x} = 0.2140in^3 * 15.2ksi = 3{,}250$"#
$I_y = 2.375in^4$
$S_y = 1.054in^3$
b/t = 1.522"/0.078" = 19.5 < 22.8 local buckling does not control.
$M_{a,y} = 1.054in^3 * 15.2ksi = 16{,}000$"#

Vertical loading:
The header may carry the dead loading from the transom. The transom glass will sit on two bearing blocks located approximately 6" from the ends. Since the dead load stresses will be very low, creep will not be an issue. Therefore, excessive deflection caused by the weight of the glass will be apparent immediately and can be reduced by moving the bearing blocks closer to the ends of the glass.

Check 4' tall transom with 6' wide door using 1/2" glass.
P = 6.5psf*4'*6'/2 = 78.0#

Equal Point Loads Symmetrically Placed

P (lbs)	L (in)	a (in)	I (in^4)	E (ksi)
78	72	6	0.2789	10100

M (in-lbs)	Δ (in)	L/Δ	R (lbs)
468	0.10666231	675.027558	78

M_{max} = 468"# < 3,250"# OK
Δ_{max} = 0.11" < 1/8" OK
Recommended maximum transom height is 48".

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Horizontal loading:
The header will carry wind loading transferred by the transom, door hinges and, if applicable, the top lock/doorstop.

For maximum considered case, check 10’ tall x 6’ wide door with a 4’ tall transom.

Loading from transom:
P = 5psf for purposes of calculations below.
w = 5psf*4’/2 = 10plf

Uniform Load

w (plf)	L (in)	I (in^4)	E (ksi)
10	72	2.375	10100

M (in-lbs)	Δ (in)	L/Δ	R (lbs)
540	0.01215633	5922.83951	30

Loading from hinges:
P = 5psf for purposes of calculations below.
r = 5psf*3’/2*10’/2 = 37.5#

Equal Point Loads Symmetrically Placed

P (lbs)	L (in)	a (in)	I (in^4)	E (ksi)
37.5	72	2.875	2.375	10100

M (in-lbs)	Δ (in)	L/Δ	R (lbs)
107.8125	0.00290626	24774.0851	37.5

Loading from door stop/lock:
P = 5psf for purposes of calculations below.
r = 5psf*3’*10’/2 = 75#

Pont Load At Center

P (lbs)	L (in)	I (in^4)	E (ksi)
75	72	2.375	10100

M (in-lbs)	Δ (in)	L/Δ	R (lbs)
1350	0.02431266	2961.41975	37.5

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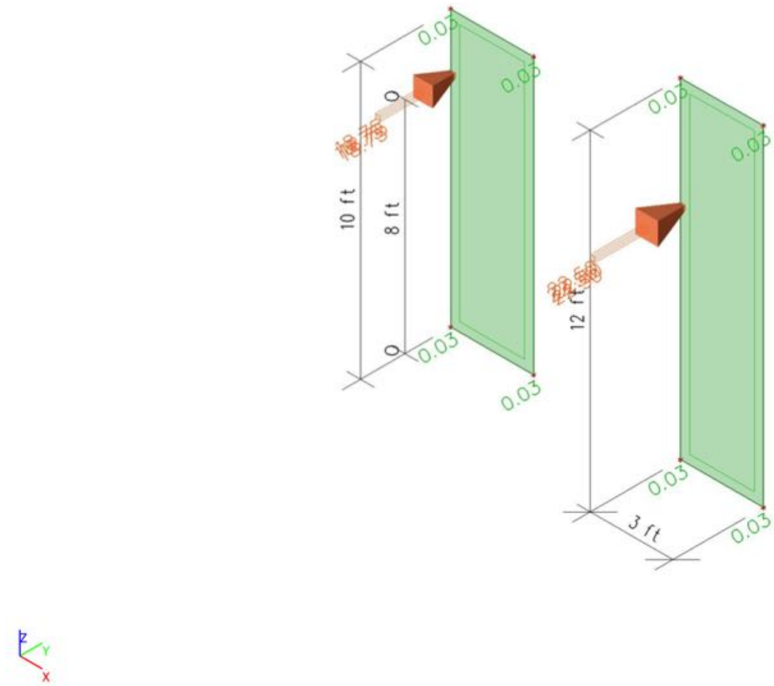
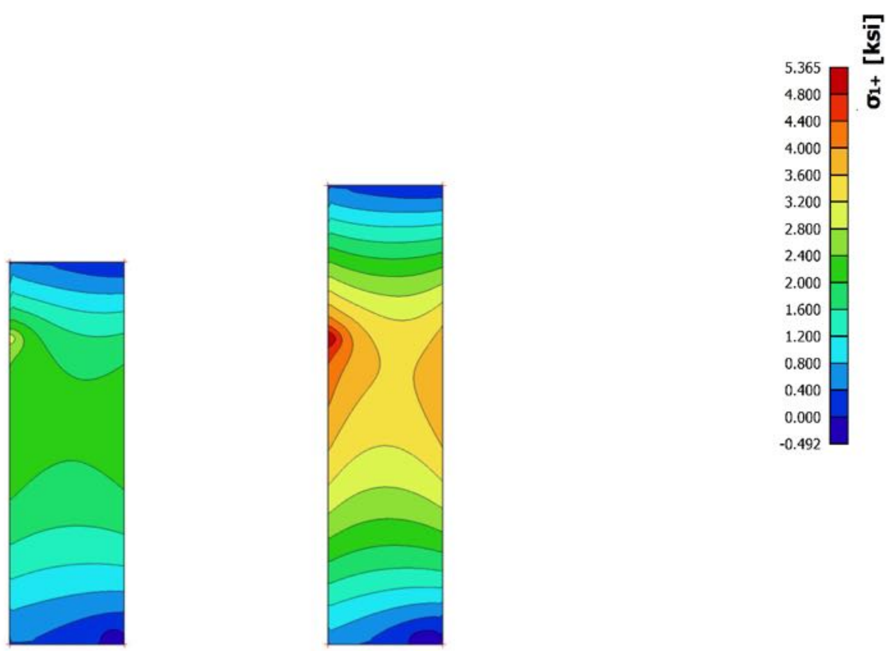
1. Wind Load / Tot. value



2. 2D stress/strain; σ_1+

Values: σ_{1+}
Nonlinear calculation
NonLinear Combi: 5psf Wind Load
Extreme: Global
Selection: All
Location: In nodes avg. on macro.
System: LCS mesh element



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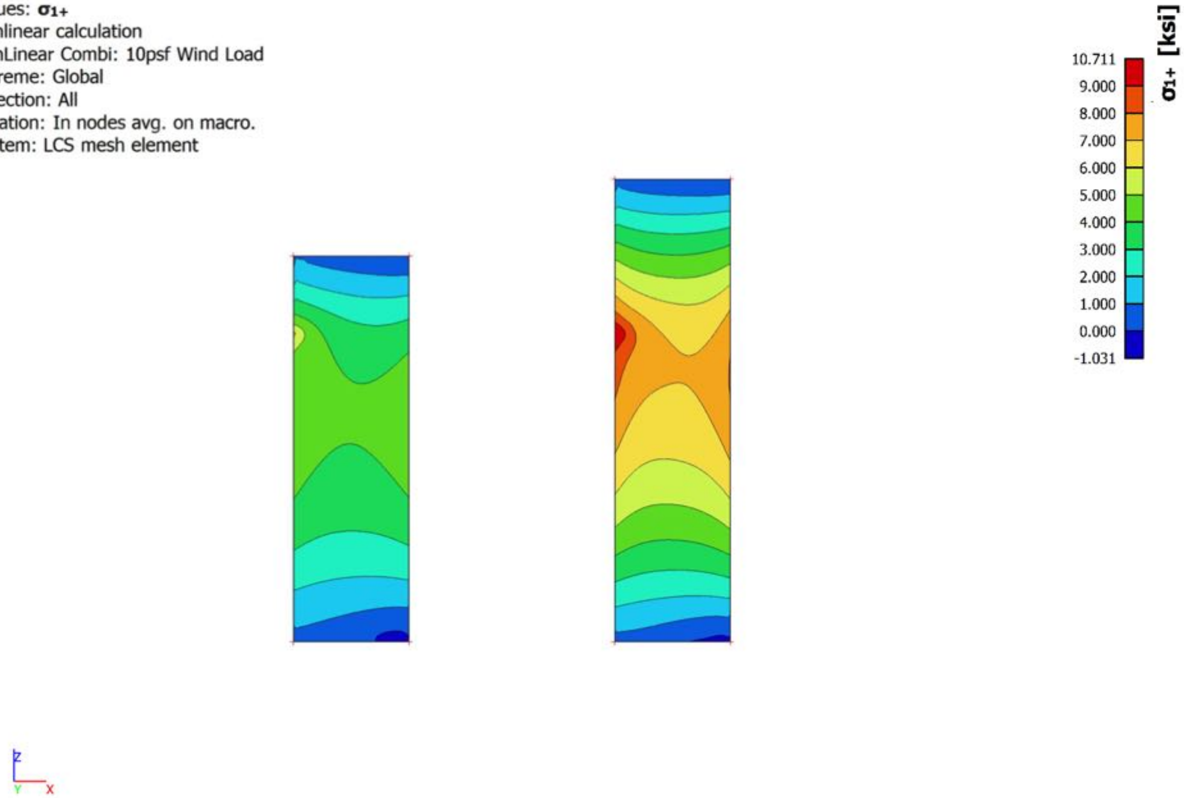
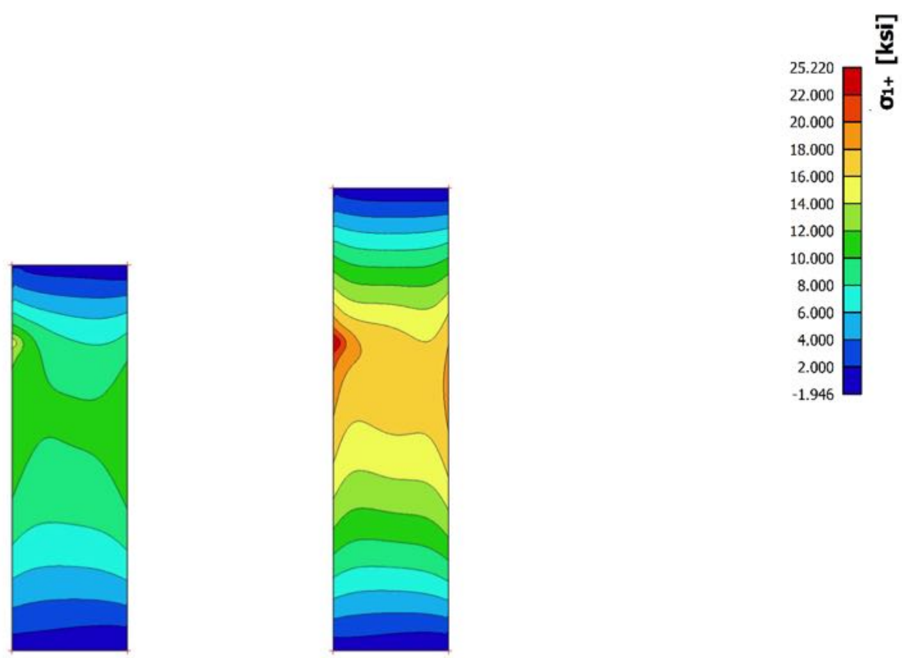
3. 2D stress/strain; σ_1+

Values: σ_{1+}
Nonlinear calculation
NonLinear Combi: 10psf Wind Load
Extreme: Global
Selection: All
Location: In nodes avg. on macro.
System: LCS mesh element



4. 2D stress/strain; σ_1+

Values: σ_{1+}
Nonlinear calculation
NonLinear Combi: 25psf Wind Load
Extreme: Global
Selection: All
Location: In nodes avg. on macro.
System: LCS mesh element



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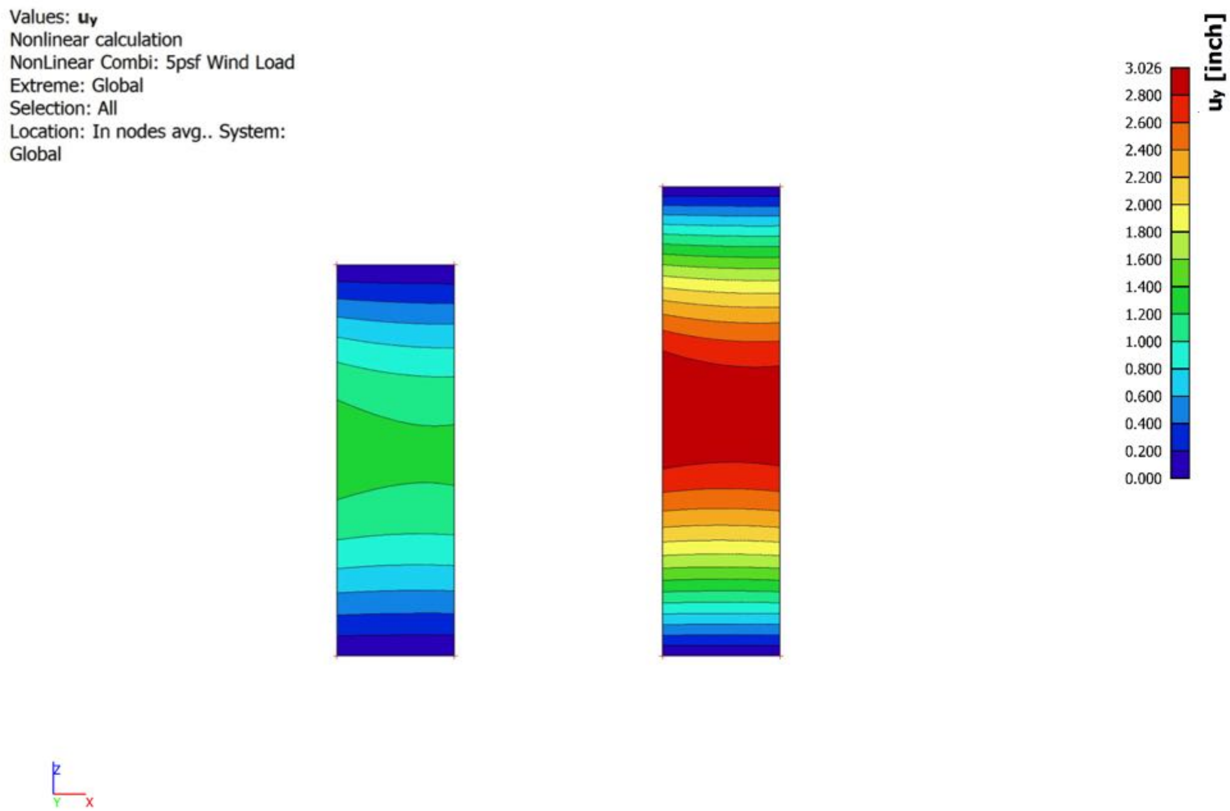
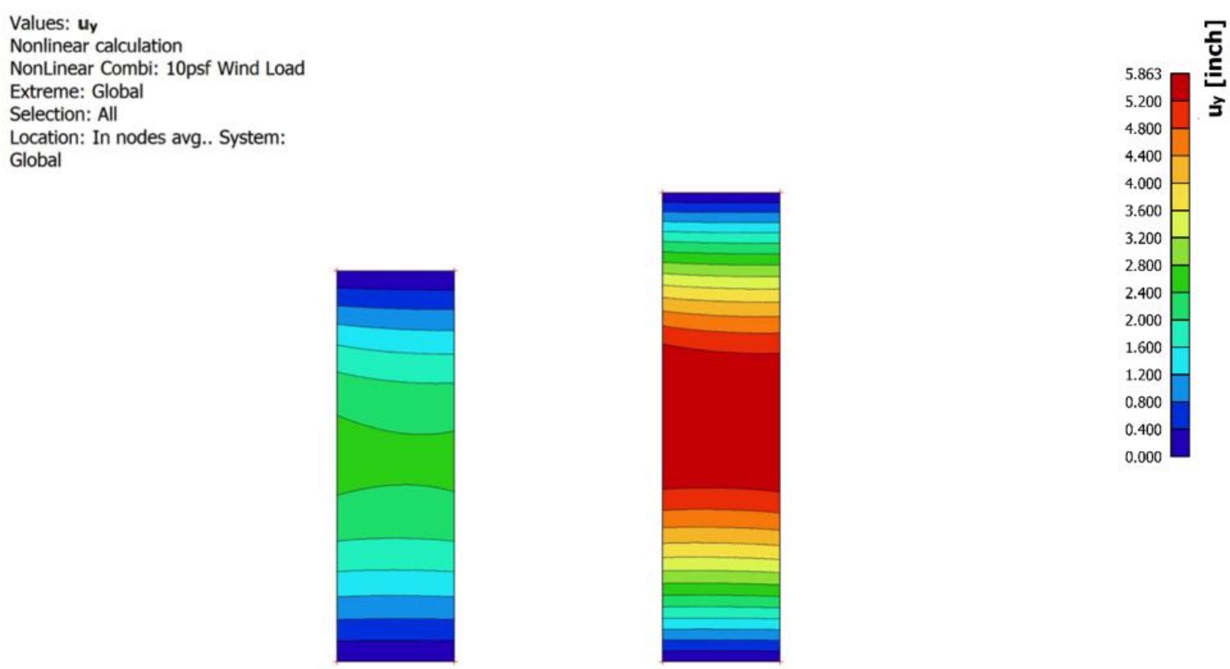
5. 2D displacement; u_y



6. 2D displacement; u_y



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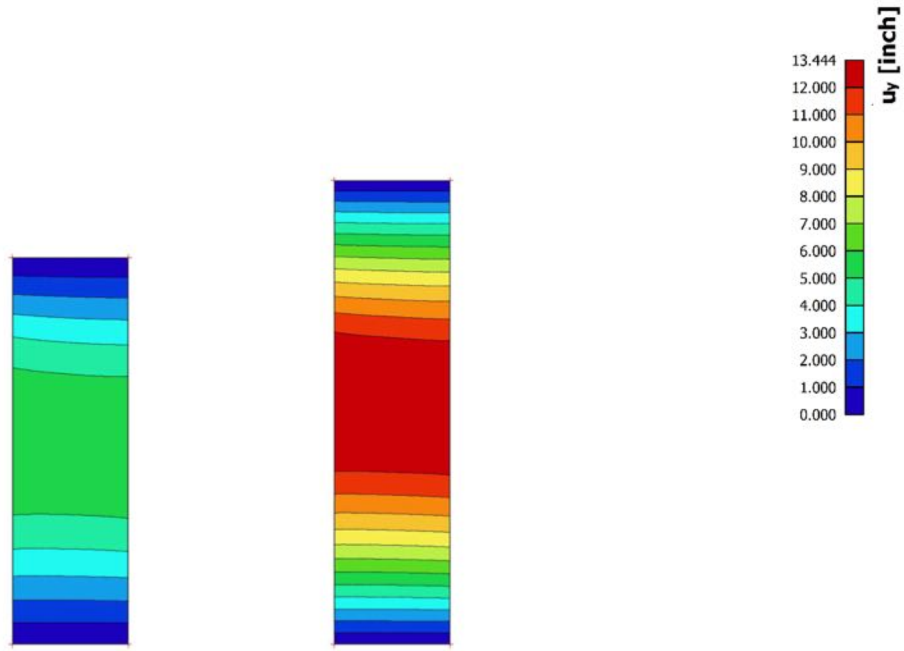
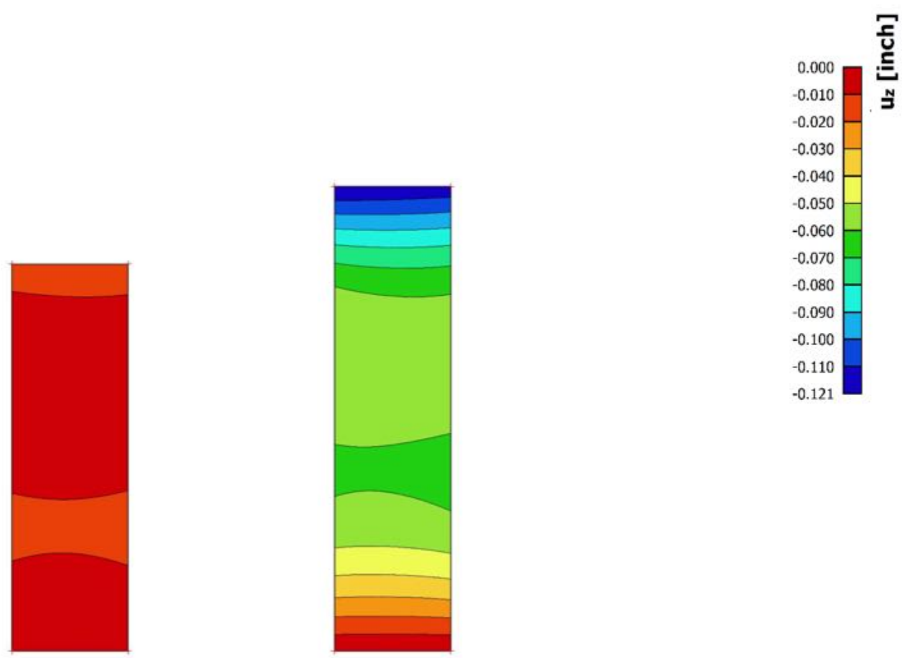
7. 2D displacement; u_y

Values: u_y
Nonlinear calculation
NonLinear Combi: 25psf Wind Load
Extreme: Global
Selection: All
Location: In nodes avg.. System:
Global



8. 2D displacement; u_z

Values: u_z
Nonlinear calculation
NonLinear Combi: 5psf Wind Load
Extreme: Global
Selection: All
Location: In nodes avg.. System:
Global



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Total loading:
M = 540"#+108"#+1,350"# = 2,000"#
M_a = 16,000"#
Allowable wind pressure:
w = 16,000"#/2,000"#*5 psf = 40psf (controls)
Δ = 0.01216"+0.002906"+0.02431" = 0.0394"
Δ_a = 72"/175 = 0.41"
Allowable wind pressure = 0.41"/0.0394"*5psf = 52.0psf (does not control)

The header can be rated for at least 40psf at its maximum recommended geometry. The maximum recommended geometry is 6'x10' door with a 4' tall transom.

Header anchorage to jamb:
A groove is cut in the header for the glass.
Check shear strength at groove:
V_a = 2*1.13"*0.125"*0.6*15.2ksi = 2,580#
#10 fastener:
V_a = 2*2*0.19"*0.125"*30ksi/3 = 950#
Limit maximum horizontal header reaction to 950#
For vertical loads the thru bolt strength is controlled by bearing on the grommet.
Assume E = 2000psi for grommet.
Allow for t/2 deformation.
t = 0.25"
Δ = P*0.25"/(200,000psi*0.25"*t_g) < 0.125"
P < 6,250# for 1/4" glass (Bearing does not control)
Steel strength,
V_a = 2*0.6*75ksi*0.0318in^3/2 = 1,430# OK by inspection, resists dead load from transom only.

Side lite glass modeling:
The side lite must be modeled using a FEA analysis because of the the point load from the floating header. The modeling assumes a 36" wide side lite. Wider side lites will have lower stress and be able to hold the same or higher wind load. Narrower lites must be checked specifically. Two conditions are checked, an 8' door with a 2' transom and an 8' door with a 4' transom. Both of the two conditions are reiterated at each thickness and at 5, 10 and 25psf. The modeling uses a 3rd order non-linear analysis that accounts for large deflections. The results show that for the these typical conditions the stresses and deflections are approximately linear to the uniform pressure. Therefore, linear interpolation can be used between the results provided.

Point load at the header = (12'/2*6'/2)*P = 18P for the 12' wall and (10'/2*6'/2)*P = 15P for the 10' wall.
In the model, point loads are applied as uniform pressures spread over a 2"x2" square at the header height.

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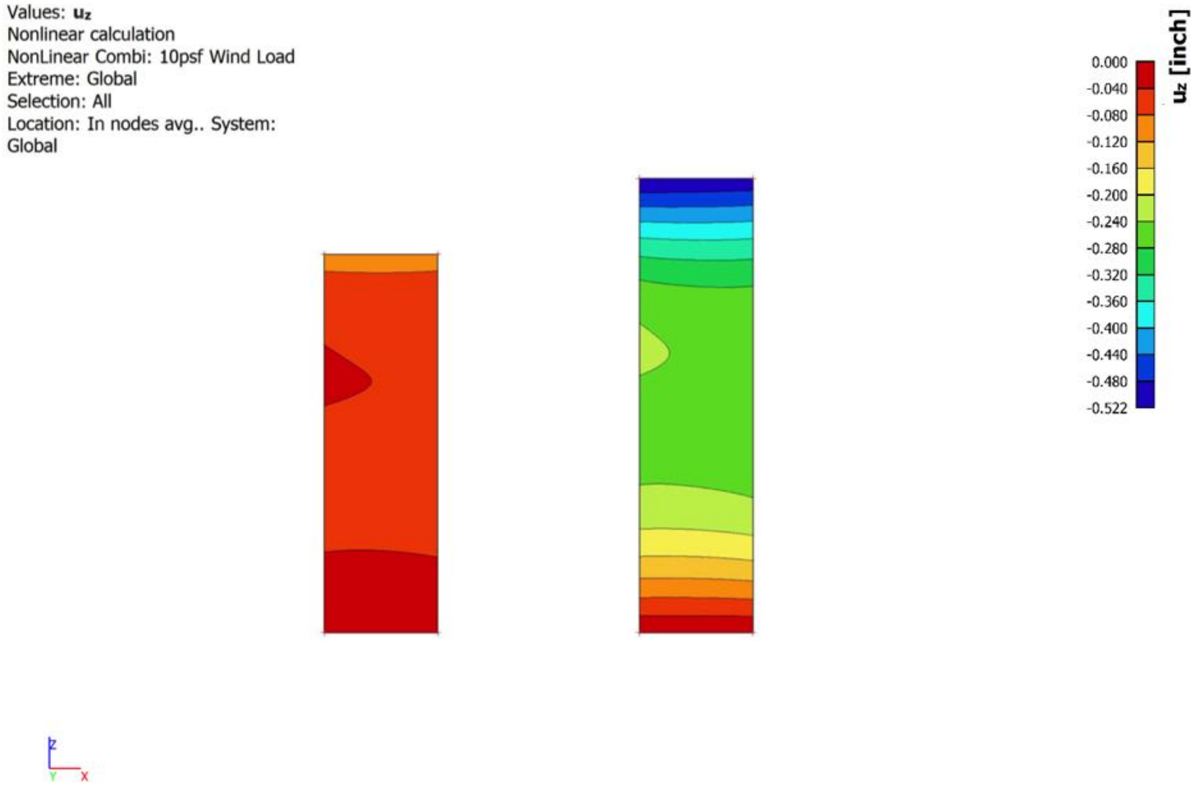
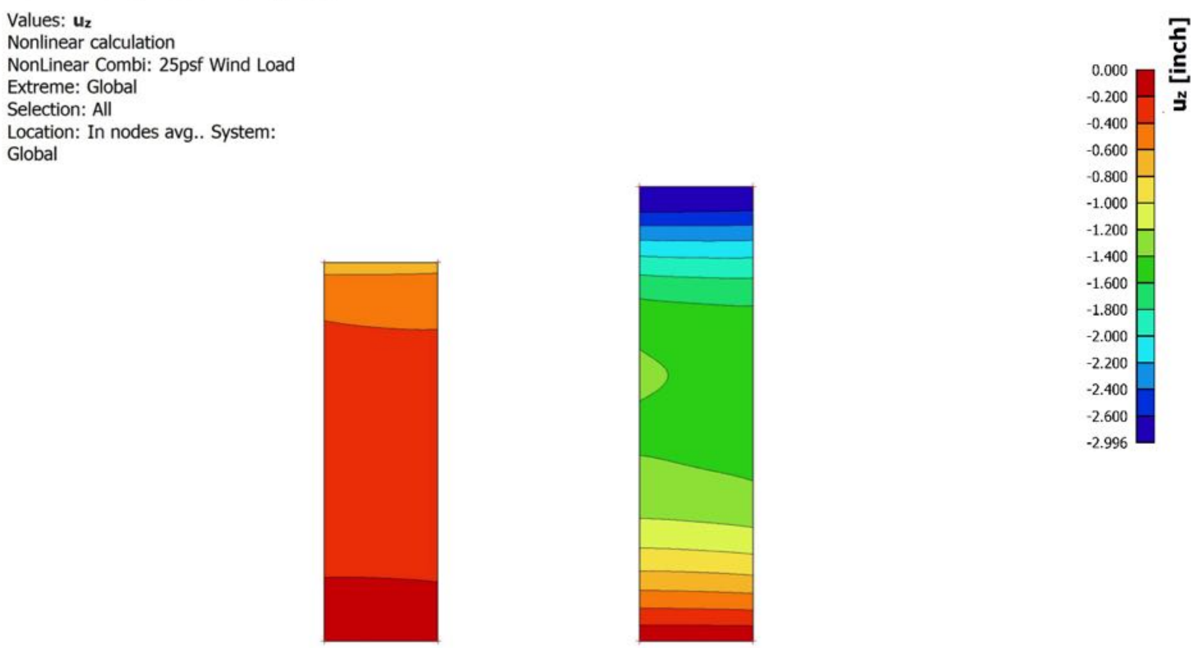
9. 2D displacement; u_z



10. 2D displacement; u_z



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EXP 03/31/2023

EXP 02/28/2023

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION

04/30/2022

Signature Expiration Date of the License

EXP 11/30/2022

Professional Certification. I hereby certify that these documents were prepared or approved by me, and that I am a duly licensed professional engineer under the laws of the State of Maryland,
License No. 52500, Expiration Date: 04/09/2022

EXP 06/30/2022

EXP 03/31/2023

STATE OF MINNESOTA
I hereby certify that this plan, specification. or report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.
Signature: Typed or printed name: Edward C. Robison
Date Lic. No. 58604

EXP 09/30/2021

FIRM #F-12044
EXP 12/31/2021

EXP 12/31/2022

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