21 April 2020

CrystaLite Inc.
3307 Cedar ST
Everett, WA 98201

SUBJ: THOMAS SUNROOM
 26180 LEYMAN LN NE
 KINGSTON, WA 98346

The project proposes constructing a custom built aluminum framed sunroom as shown in the attached details. The calculations demonstrate compliance to the 2015 International Building Code as adopted by the state of Washington.

As shown in the attached calculations the sunroom is designed for the following loading conditions:

Wind Loading: V_{ult} = 110mph Exposure C, K_{zt} = 1.0, I = 1
Snow Loading: p_g = 25psf roof snow load
Seismic Loading: S_{DS} = 0.847

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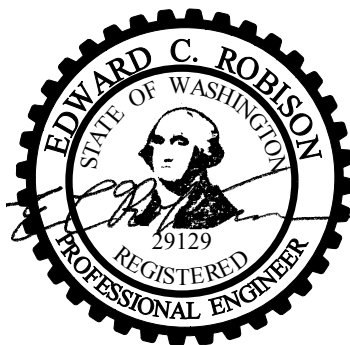
Sealed 21 April 2020

Permit Number: 19-05128

LOADING

Dead Loads:
Glass: 1" insulating glass, 1/4" tempered glass, 1/2" airspace, 1/4" laminated tempered.
D = 6.5psf
Frame: 1.8 plf for rafters.

Wind Loads:
Calculate according to ASCE 7-10 Chapter 30 Figure 30.5-1
110 mph Zone 2 and wind area = 10 S.F.
P_{net30}= 12.5psf or -34.7psf
λ=1.21 (Exposure C and Z=15')
Multiply by 0.6 to convert to ASD level loading
W=13.7psf*1.21*0.6=9.95psf or -37.9psf*1.21*0.6=-27.5psf
Wind zone 5 for walls:
P_{net30}= 21.8psf or -29.1psf
λ=1.21 (Exposure C and Z=15')
Multiply by 0.6 to convert to ASD level loading
W=21.8psf*1.21*0.6=15.8psf or -29.1psf*1.21*0.6=-21.1psf

Seismic Loads:
$F_P =[(0.4*a_P S_{DS} I_P)/R_P][1+2z/h]W_P = [0.4*1.0*0.847*1.0/2.5](1+2) = 0.407W_P$
For glass weight:
E = 0.407*6.5psf = 2.65 psf
Seismic loads are very low compared to wind loading and will not control any part of the design.

Snow Loads:
p = 25psf on roof

Load combinations: ASCE 7-10 2.4.1
D+S = 6.5psf+25psf = 31.5psf
D+0.75S+0.75W = 6.5psf+0.75*25psf+0.75*9.95psf = 32.7psf
0.6D+W = 0.6*6.5psf-27.5psf = -23.6psf

Edward C. Robison, P.E., S.E.
10012 Creviston DR NW   *253-858-0855*
Gig Harbor, WA 98329   *Fax 253-858-0856*

RAFTERS

Rafter Strength

$A = 2.143\ in^2$
$I_{xx} = 4.466\ in^4$
$I_{yy} = 2.910\ in^4$
$S_{xx} = 2.2277\ in^3$ Top
$S_{xx} = 2.2383\ in^3$ Bottom
$S_{yy} = 1.7247\ in^3$
$r_x = 1.4435"$
$r_y = 1.1653"$
$J = 3.19\ in^4$
Allowable stresses:
ADM Table 2-24 6063-T6)
for $b/t \leq 23$ elements don't limit stress
$b \leq 23*0.125 = 2.875" > 2.075"$
All flange elements okay
$F_t = 15.2\ ksi$
$F_{bt} = 15.2\ ksi$

$M_a = 15.2ksi*2.2277in^3 = 33,800"\#$

Rafter loading

Max span = 119"
Max tributary width = (33.6875"+29.6875")/(2*12) = 2.64ft
Max loading, $w = 2.64ft*32.7psf = 86.3plf$
Max moment, $M = 86.3plf/12*119"^2/8 = 12,700"\# < 33,800"\#$ OK
Max deflection, $\Delta = 5*86.3plf/12*119"^4/(384*10.1*10^6*4.466in^4) = 0.416"$
$L/\Delta = 119"/0.416" = 286 > 175$ OK

Rafter loading OK
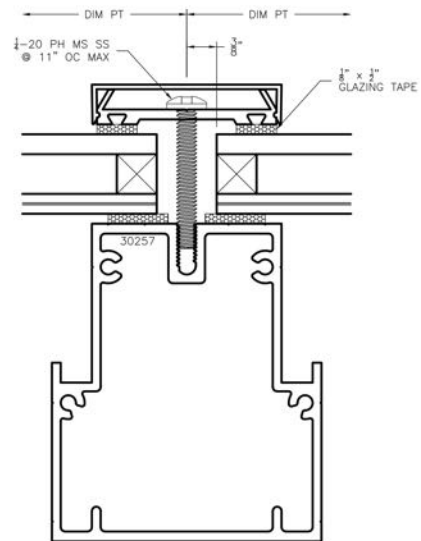
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10012 Creviston DR NW   *253-858-0855*
Gig Harbor, WA 98329   *Fax 253-858-0856*

VERTICAL MULLIONS
Part 4005
$A = 2.2845$ in^2
$I_{xx} = 4.6447$ in^4
$I_{yy} = 1.7284$ in^4
$S_{xx} = $ Varies
$Z_{xx} = 3.071$ in^3
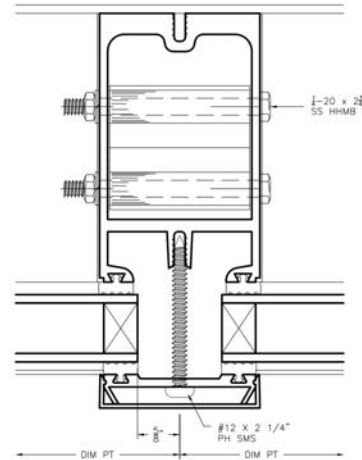$S_{yy} = 1.4552$ in^3
$Z_{yy} = 1.781$ in^3
$r_x = 1.4259$"
$r_y = 0.86981$"
Allowable stresses:
ADM Table 2-24 6063-T6)
for b/t ≤ 23 elements don't limit stress
No elements on this shape are susceptible to local buckling.

Allowable bending moment:
$M_{a,x} = 15.2$ksi$*3.071$in$^3 = 46,679$"#
$M_{a,y} = 15.2$ksi$*1.781$in$^3 = 27,071$"#

Allowable axial loads:
$T_a = 15.2$ ksi$*2.2845$in$^2 = 34.72$ k
$kL/r = 1*108$"$/.86981 = 124$
$F_c/\Omega = 14.2 - .074*124 = 5.02$ksi
$C_a = 5.02$ksi$*2.2845$in$^2 = 11.5$ k

Max compression = 86.3plf/12*119"/2 = 428# << 11.5kips OK

Max tributary width = 3.13'
w= 3.13'*21.1psf = 66.0plf
M = 66.0plf/12*108"^{2}/8 = 8,020"# < 46,679"# OK

Vertical mullions provide stability to the skylight structure.
For calculating overall lateral load, subtract internal pressure from envelope wind pressures.
Internal pressure = 0.6*0.00256*1*0.85*0.85*110^2*0.18 = 2.42psf

Net horizontal wind pressure = (15.8psf-2.42psf)+(21.1psf-2.42psf) = 32.1psf.
Shear on front wall = 32.1psf*9.76'/2*9'/2 = 705#
Other walls transfer shear to the structure through the ledger mount.
Weak axis moment = 705#*71.5"/(2*8) = 3,150"# < 27,070"# OK
Check moment resistance of sill and head connections with (2) 1/4" fasteners:
Bolt rupture = 0.0318in^2*0.6*75ksi/2= 716#
Hole bearing = 2*0.25"*0.125"*30ksi/1.95 = 962#
Moment resistance = 2*716#*2.375" = 3,400"# > 3,150"# OK

Edward C. Robison, P.E., S.E.
10012 Creviston DR NW   *253-858-0855*
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Connection at head:

1/4" screws to shear block:
Allowable shear is controlled by bearing on aluminum.
$R_n/\Omega = 2*0.25"*0.12"*30ksi/3 = 600\#$ each
Allowable shear $= 4*600\# = 2,400\#$
Max shear $= 66.0plf*9'/2 = 297\# < 2,400\#$ OK
Max uplift $= 23.6psf*3.13'*10'/2 = 369\# < 2,400\#$ OK

Check vertical screws to ledger plate plate:
Shear strength will be the same as the horizontal screws to the shear block.
$T_a = 1.2*0.21"*0.12"*30ksi/3 = 302\#$ Wach
Total, $T_a = 2*302\# = 604\# > 369\#$ OK

Sill Connection:
(2) 1/4" bolts in double shear.
$R_n/\Omega = 2*0.25"*(2*0.12")*30ksi/1.95 = 1,850\# > 369\#$ OK

Field fasteners:
Use #10x2" wood screws
$V = 21.1psf*9'/2 = 95.0plf$
$T = 95.0plf*1"/2" = 47.5plf$

$p = 2"-0.69" = 1.31"$
$Z' = 1.6*133\#*1.31"/(10*0.19") = 147\#$
$W'p = 1.6*135pli*1.31" = 283\#$
Check pullover assuming a countersunk head:
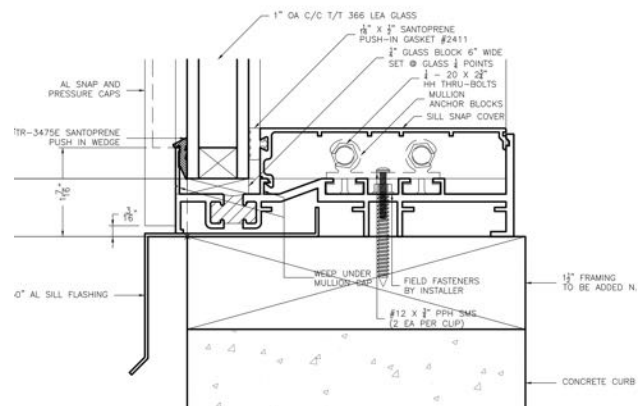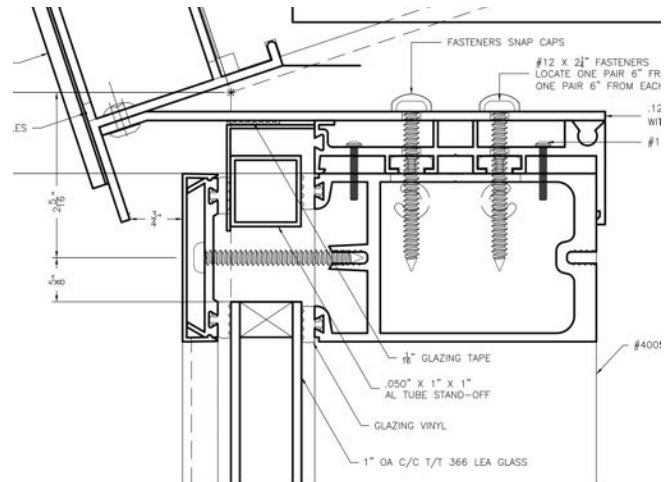$R_n\Omega = (0.27+1.45*0.19"/0.12")*0.19"*0.125"*25ksi/3 = 508\#$

Find max spacing:
Shear strength controls spacing, $S_a = 147\#/95.0plf*12 = 18"$ O.C.

Edward C. Robison, P.E., S.E.
10012 Creviston DR NW   *253-858-0855*
Gig Harbor, WA 98329   *Fax 253-858-0856*

HEADERS

Mullion Name:
PART 4005

Ix (in^4)	Sx (in3)	b/t
1.728	1.455	20.8

Iy (in4)	Sy (in3)	b/t
4.645	3.071	15

Glass WT (psf)	Supported glass height (ft)	Δa (in)	Max Span (in)	E (x10^6 psi)	Bearing Block Load (lbs)
3.5	4	0.125	77	10.1	44.91666667

	Span (in)	Δactual	Error
Max Span for L/4 (in)	106.9	0.12497057	-2.943E-05

	Span (in)	Δactual	Error
Max span for L/8 (in)	125.0	0.12497057	-2.943E-05

	Span (in)	Δactual	Error
Max a for max span (in)	40.2	0.04880696	-0.076193

Mx (in-lbs)	864.6458333

Assuming Maximum Triangular Wind Load:	
P (psf)	21.1
Peak load w (plf)	135.3916667
Total load W (lbs)	434.3815972
My (in-lbs)	5574.563831
Δ	0.327243455
L/Δ	235.2988236

Allowable Moments:	
F/Ω,x	15.2
F/Ω,y	15.2
Ma,x (In-lbs)	22116
Ma,y (in-lbs)	46679.2

Stress Check	
Mx	Pass
My	Pass
Biaxial	Pass

Edward C. Robison, P.E., S.E.
10012 Creviston DR NW   *253-858-0855*
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Permit Number: 19-05128
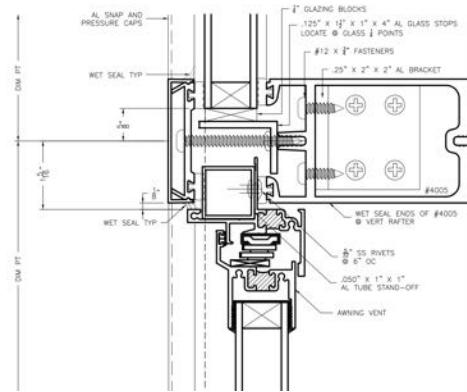
LEDGER MOUNT

Vertical shear = 32.7psf*10'/2 = 164plf
Horizontal shear = 32.1psf*(9.76'/2*9'/2)/18.0'
$\qquad$ = 39.2plf
Tension = 21.1psf*6'/2 = 63.3plf

Check bending in ledge from tension loads:
M = 4.75"*63.3" = 301"#/ft
M_a = 0.125"^{2}*12"/4*15.2ksi = 713"#/ft > 301"#/ft
OK

Check loading on wood screw:
Assume #10x2" wood screws
p = 2"-0.12" = 1.88"
Z' = 1.6*133#*1.88"/(10*0.19") = 211#
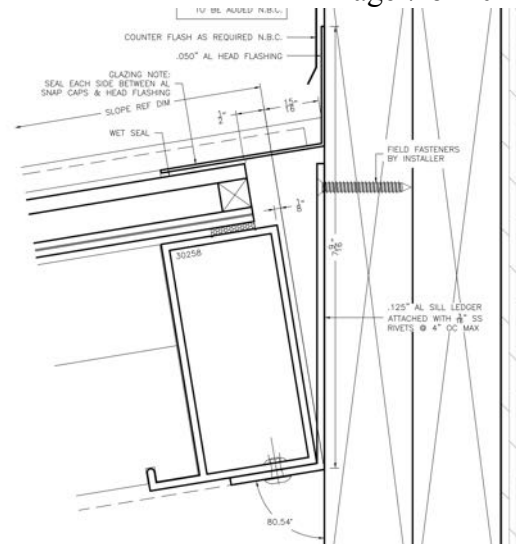W'p = 1.6*135pli*1.88" = 406#
Allowable spacing is controlled by shear loading:
D+.75W+0.75S controls shear loading.
V_{net} = (164^2+(0.75*39.2)^2)$^{1/2}$ = 167plf
Max spacing = 211#/167plf*12 = 15" O.C.

Edward C. Robison, P.E., S.E.
10012 Creviston DR NW   *253-858-0855*
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GLASS STRENGTH
Check glass based on ASTM E 1300-12a
1" Insulating glass at roof panels
1/4" tempered monolithic - 1/2" air space - 1/4" tempered laminated glass

Long duration load = 25psf+6.5psf = 31.5psf
Short duration load = 32.7psf

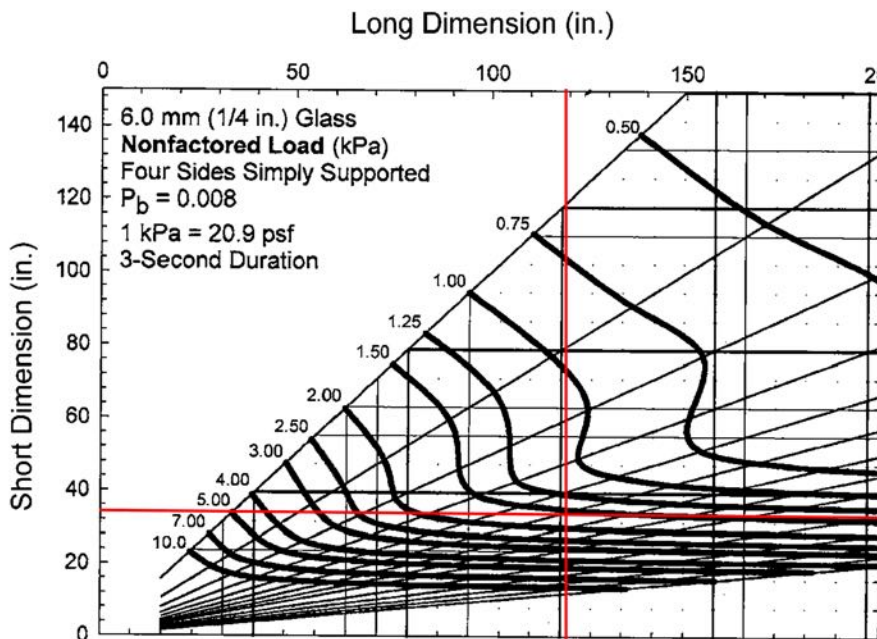Check 32.7psf as a long duration load. If it passes then long duration and short duration loads are OK.

$GTF = 2.85$
$LS_1 = 1.20$ (Monolithic pane)
$LS_2 = 5.96$ (Laminated pane)
The monolithic pane is much stiffer and will control allowable loading.

Dimensions: 34"x119"

$NFL_1 = 1.5*20.9psf = 31.4psf$
$LR_1 = 31.4psf*2.85*1.20 = 107psf > 31.4psf$ OK

Estimate deflection
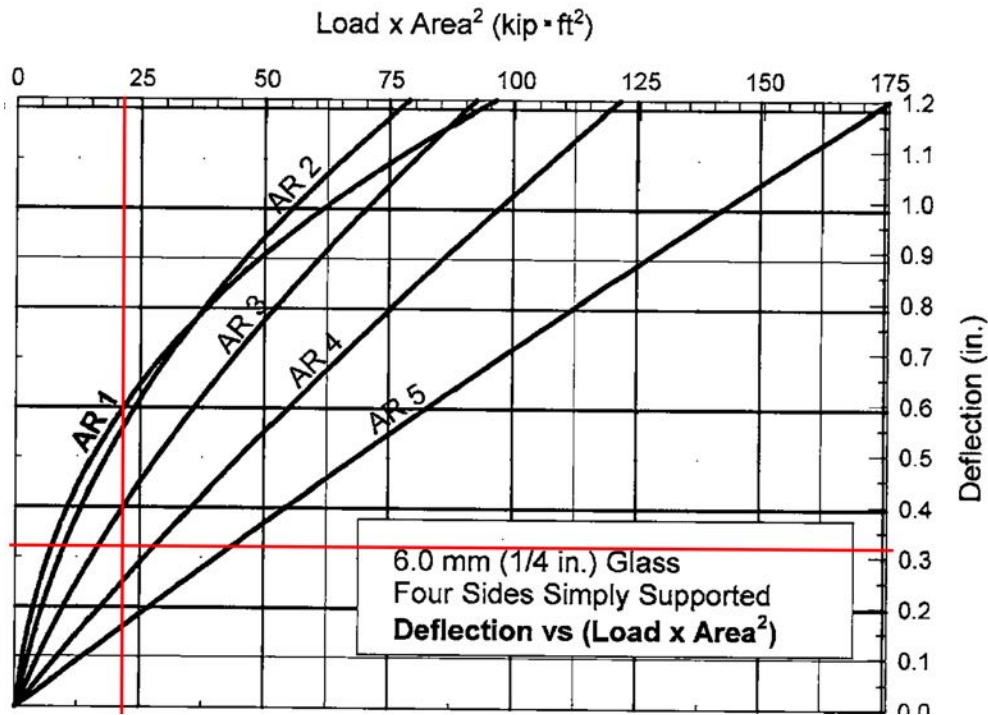$PA^2 = 31.4psf/(1000*1.20)*(34"*119"/144)^2 = 20.7kip\text{-}ft^2$
$AR = 119"/34" = 3.5$

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Load x Area² (kip·ft²)

6.0 mm (1/4 in.) Glass
Four Sides Simply Supported
Deflection vs (Load x Area²)

$\Delta = 0.32" < 3/4"$ OK
$L/\Delta = 34"/0.32" = 106 > 60$ OK

1" Insulating glass at wall panels
1/8" tempered monolithic - 3/4" air space - 1/8" tempered monolithic glass
Wind pressure = 21.1psf

GTF = 3.6
LR = 2
Max dimensions: 35"x108"
(Calculations continued on the following page)

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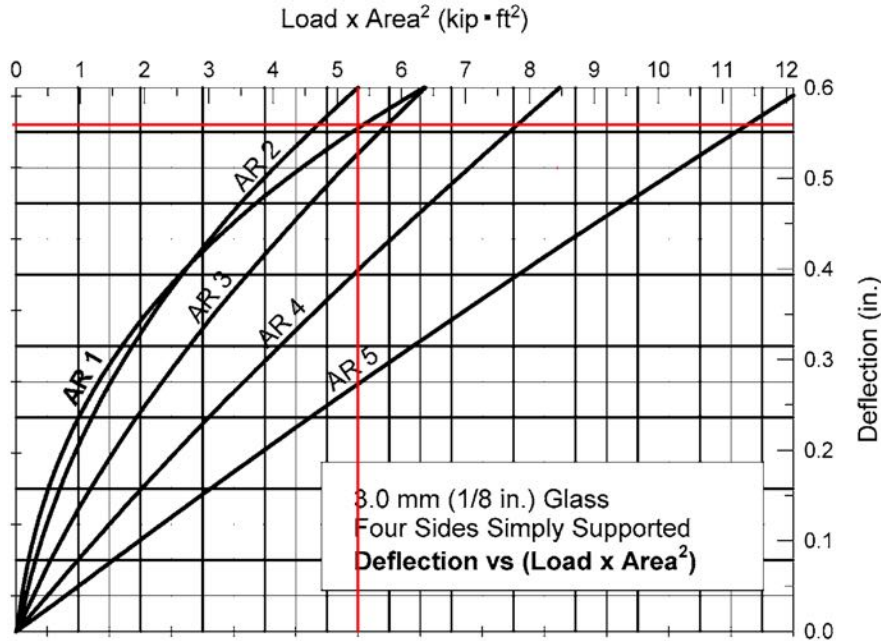*10012 Creviston DR NW*   *253-858-0855*
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Load x Area² (kip·ft²)

3.0 mm (1/8 in.) Glass
Four Sides Simply Supported
Deflection vs (Load x Area²)

NFL = 20.9*0.6 = 12.5psf
LR = 12.5psf*2*3.6 = 90.3psf > 21.1psf OK

Estimate deflection
PA^2 = 21.1psf/(1000*2)*(35"*92"/144)² = 5.28kip-ft²

AR = 92"/35" = 2.6

Δ= 0.56" < 3/4" OK
L/Δ = 35"/0.56" = 63 > 60 OK

3.0 mm (1/8 in.) Glass
Nonfactored Load (kPa)
Four Sides Simply Supported
P_b = 0.008
1 kPa = 20.9 psf
3-Second Duration

Long Dimension (in.)

Short Dimension (in.)

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