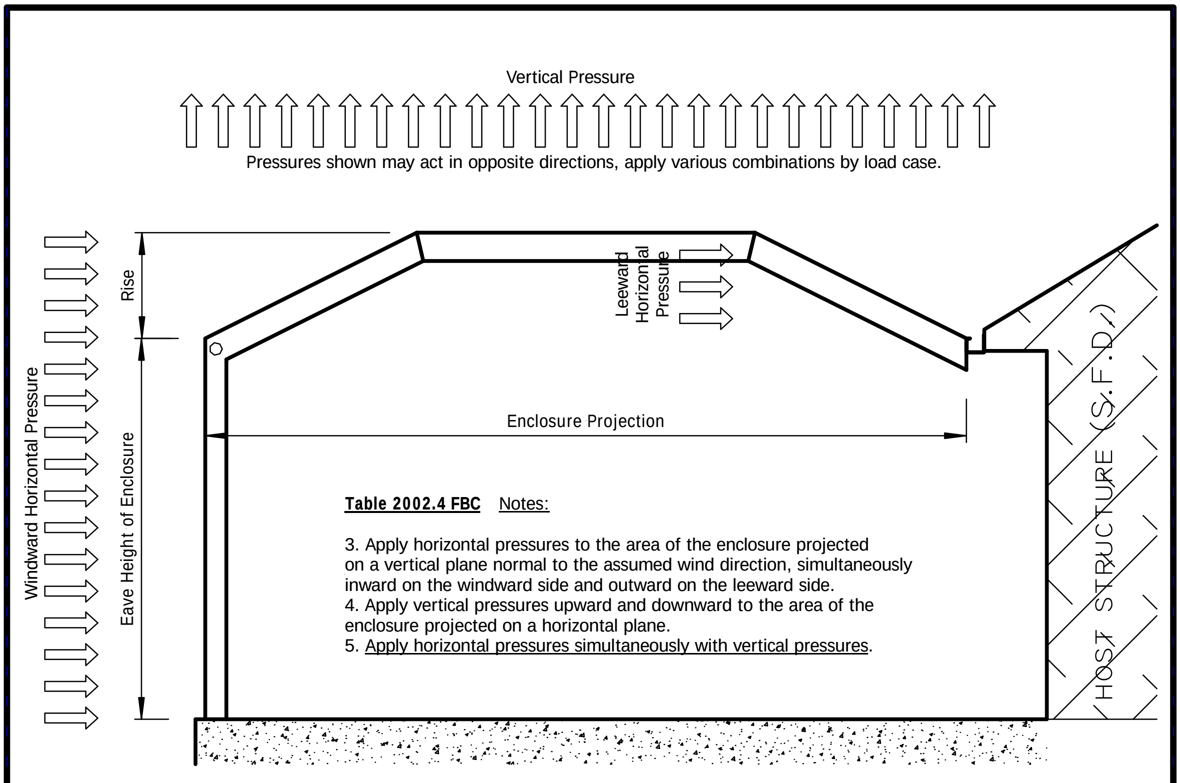
Allowable Span of Mansard Screen Enclosure Self-Mating Beams in accordance with requirements of Table 2002.4 (and the 2005 Aluminum Design Manual) using 6005T5 alloy:

Design pressures from Table 2002.4 for 130 mph Exposure B with a mean roof height not to exceed 30 ft using a 20/20 screen mesh:

Horizontal design pressure (Windward): $p_{w\_wall} := 18 \cdot psf$

Horizontal design pressure (Leeward): $p_{l\_wall} := 14 \cdot psf$

Deflection limit (L/60) Vertical design pressure: $p_{roof} := 5 \cdot psf$



Typical Schematic Section / Mansard Pool Enclosure Post / Beam

Determine the allowable beam span based upon the following project frame geometry:

Wall height (eave height): $h_{eave} := 10 \cdot ft$

Vertical distance, height overall less eave height: $Rise := 3.5 \cdot ft$

Center to center spacing of mansard frames: $Spacing := 7.5 \cdot ft$

Approxmate allowable span based upon frame geometry and allowable major axis bending moments for various industry beams (preliminary span):

Horizontal force applied inward at screen eave:

$$P_{eave} := \frac{h_{eave}}{2} \cdot p_{w\_wall} \cdot Spacing$$

$$P_{eave} = 675\,lbf$$

Induced beam moment (from P):

$$M_P := P_{eave} \cdot Rise$$

$$M_P = 28.35 \cdot kip \cdot in$$

Induced beam moment from windward and leeward mansard legs:

$$M_{Rise} := \frac{Rise^2}{2} \cdot Spacing \cdot \left(p_{w\_wall} + p_{l\_wall}\right)$$

$$M_{Rise} = 17.64 \cdot kip \cdot in$$

Sum of moments induced into beam from horizontal pressures:

$$M_P + M_{Rise} = 45.99 \cdot kip \cdot in$$

By inspection of induced moment from horizontal force at the eave and the allowable major axis bending moments, determine preliminary span for design check and final allowable span:

Major Axis Allowable Bending Moments			
Section	S_x (in^3)	F_b (ksi)	M_a (in kip)
2X4 SMB	1.000	8.7	8.7
2X5 SMB	1.510	9.2	13.9
2X6 SMB	1.944	9.0	17.4
2X7 SMB	2.425	9.2	22.3
2X8 SMB	4.441	15.0	66.7
2X9 SMB	6.123	17.5	107.0
2X10 SMB	8.462	17.4	147.5

By inspection, **SELECT trial member** for span calculation & enter allowable moment:

$$M_a := 107 \cdot kip \cdot in$$

ENTER beam depth of member selected:

$$d := 9 \cdot in$$

Net moment from allowable less moment induced at shoulder:

$$M_{preliminary} := M_a - \left(M_P + M_{Rise}\right)$$

$$M_{preliminary} = 61.01 \cdot kip \cdot in$$

Preliminary allowable span:

$$L_{preliminary} := \sqrt{\frac{8 \cdot M_{preliminary}}{Spacing \cdot p_{roof}}}$$

$$L_{preliminary} = 395.203\,in$$

Profile :=

	0 (Depth)	1 (ID)	2 (t_w)	3 (t_f)	4 (Area)	5 (I_x)	6 (I_y)
0	4	"2X4 SMB"	0.046	0.1	0.782	2	0.47
1	5	"2X5 SMB"	0.05	0.116	0.917	3.644	0.592
2	6	"2X6 SMB"	0.05	0.12	1.022	5.66	0.685
3	7	"2X7 SMB"	0.055	0.12	1.229	8.472	0.849
4	8	"2X8 SMB"	0.072	0.224	1.847	16.577	1.215
5	9	"2X9 SMB"	0.082	0.306	2.357	27.16	1.524
6	10	"2X10 SMB"	0.092	0.374	3.013	42.282	1.926

Self-Mating Beam trial member selected (values from standard sections listed above): $Row := \frac{d}{in} - 4$

Extrustion profile nomenclature: $ID := Profile_{Row,1}$ $ID =$ "2X9 SMB"

Width: $b := 2.00 \cdot in$

Flange Thickness: $t_f := Profile_{Row,3} \cdot in = 0.306\,in$

Web Thickness: $t_w := Profile_{Row,2} \cdot in = 0.082\,in$

X-axis support: $L_d := L_{preliminary}$ is the strong-axis support distance.

Weak-axis Support: $L_b := 84 \cdot in$ is the weak-axis support distance (purlins spacing)

Flange Support: $Spacing_{stitching} := 24 \cdot in$

Self-Mating Beams of alloys 6061T6 OR 6005T5 Aluminum; Development of Properties and Allowable Stresses for the specified profile (i.e. properties by formula and table above from Autocad) in accordance with the 2005 Aluminum Design Manual Specifications:

Channel Flange Width: $b_c := 1.560 \cdot in$

Safety Factor: $n_\mu := 1.95$

Modulus of Elasticity: $E := 10.100 \cdot 10^3 \cdot ksi$

Open-channel Properties (one side of SMB)

Channel Flange Width used for Buckling: $b_f := b_c - (t_w)$ $b_f = 1.478\,in$

Web Height used for Buckling: $h_w := d - (2 \cdot t_f)$ $h_w = 8.388\,in$

Member Flange Width: $b_m := b - 2 \cdot t_w$ $b_m = 1.836\,in$

Total Area: $Area := (b \cdot d) - \left[h_w \cdot (b - 2 \cdot t_w)\right]$

Web Area: $Area_{web} := 2 \cdot t_w \cdot h_w$

Flange Area: $Area_{flange} := 2 \cdot t_f \cdot b$

Check the area equations: $\text{Area} = \text{Area}_{web} + \text{Area}_{flange}$

i.e., $\left|\text{Area}_{web} + \text{Area}_{flange} - \text{Area}\right|$ $\text{Area} = 2.6\,in^2$

Torsional Constant: $$J := \frac{2 \cdot t_f \cdot t_w \cdot (b - t_w)^2 \cdot (d - t_f)^2}{(b \cdot t_w) + (d \cdot t_f) - t_w^{\,2} - t_f^{\,2}}$$

Moments Of Inertia	Section Modulus	Radius of Gyration
$I_x := \text{Profile}_{Row,5} \cdot in^4$	$S_x := \frac{I_x}{\frac{d}{2}}$	$r_x := \sqrt{\frac{I_x}{\text{Area}}}$
$I_y := \text{Profile}_{Row,5} \cdot in^4$	$S_y := \frac{I_y}{\frac{b}{2}}$	$r_y := \sqrt{\frac{I_y}{\text{Area}}}$

Open Channel properties - In weak axis

Channel-width in buckling: $b_{cf} := b_c - t_w$

Channel Height: $d_c := d - \left(\frac{t_f}{2}\right)$

Center of Gravity to Outer Fiber Distance: $$C_1 := \frac{t_f \cdot b_c^{\,2} + (d_c - t_f) \cdot t_w^{\,2}}{2 \cdot \left[t_f \cdot b_c + \left[(d_c - t_f) \cdot t_w\right]\right]}$$ $C_1 = 0.341\,in$

$C_2 := b_c - C_1$ $C_2 = 1.219\,in$

Channel Moment of Inertia: $$I_{yc} := \frac{d_c \cdot C_1^{\,3} - (d_c - t_f) \cdot (C_1 - t_w)^3 + t_f \cdot C_2^{\,3}}{3}$$ $I_{yc} = 0.252\,in^4$

Radius of Gyration: $$r_{yc} := \sqrt{\frac{I_{yc}}{(d_c \cdot t_w) + t_f \cdot (b_c - t_w)}}$$ $r_{yc} = 0.463\,in$

$\text{Area}_{channel} := (d_c \cdot b_c) - (d_c - t_f) \cdot (b_c - t_w)$ $\text{Area}_{channel} = 1.178\,in^2$

Allowable Axial Tension

Specification 3.4.2 Tension: $F_{btf} := 19 \cdot ksi$ (Flat elements in uniform tension)

Specification 3.4.4 (Tension): $F_{btw} := 28 \cdot ksi$ (Flat elements bending in their own plane)

Allowable Axial Compression

Specification 3.4.7 Member Buckling

Assume most severe case: $k \equiv 1$

$$Slenderness7_x := k \cdot \frac{L_d}{r_x} \qquad Slenderness7_y := \frac{k \cdot L_b}{r_y}$$

$$Slenderness7 := \max\left(Slenderness7_x, Slenderness7_y\right) \qquad Slenderness7 = 122.268$$

$$F_{cm} := \begin{cases} (20.2 - 0.126 \cdot Slenderness7) \cdot ksi & \text{if } Slenderness7 < 66 \\ \left(\dfrac{51100}{Slenderness7^2}\right) \cdot ksi & \text{if } Slenderness7 \geq 66 \end{cases} \qquad F_{cm} = 3.418 \cdot ksi$$

Member Buckling between stitch screws (beam half) [ADM 3.4.7]

$$k_{channel} := 0.5$$

$$Slenderness_{channel} := \frac{k_{channel} \cdot Spacing_{stitching}}{r_{yc}}$$

$$F_{ch} := \begin{cases} \left(20.2 - .126 \cdot Slenderness_{channel}\right) \cdot ksi & \text{if } Slenderness_{channel} < 66 \\ \dfrac{51100 \cdot ksi}{Slenderness_{channel}^{\ 2}} & \text{otherwise} \end{cases} \qquad F_{ch} = 16.93 \cdot ksi$$

Specification 3.4.9 Local Web Buckling:

$$Slenderness9_w := \frac{h_w}{t_w}$$

$$F_{cw} := \begin{cases} 21\cdot ksi & \text{if } Slenderness9_w \le 7.6 \\ \frac{491}{Slenderness9_w}\cdot ksi & \text{if } Slenderness9_w \ge 33 \\ (23.1 - 0.247\cdot Slenderness9_w)\cdot ksi & \text{otherwise} \end{cases}$$

$F_{cw} = 4.800\cdot ksi$

Specification 3.4.8.1 Local Buckling, Flange:

$$Slenderness8 := \frac{2\cdot b_f}{t_f}$$

$$F_{cf} := \begin{cases} 21\cdot ksi & \text{if } Slenderness8 \le 2.4 \\ \frac{1970}{Slenderness8^2}\cdot ksi & \text{if } Slenderness8 \ge 12 \\ (23.1 - 0.787\cdot Slenderness8)\cdot ksi & \text{otherwise} \end{cases}$$

$Slenderness8 = 9.66$

$F_{cf} = 15.50\cdot ksi$

Member Elastic Buckling (ADM Eq. 4.7.4-3):

$$F_{ec} := \left(\frac{\pi}{Slenderness7}\right)^2\cdot E$$

$F_{ec} = 6.668\cdot ksi$

Specification 4.7.1 Local Elastic Buckling of the Web:

$$F_{crw} := \frac{\pi^2\cdot E}{(1.6\cdot Slenderness9_w)^2}$$

Local Elastic Buckling, Flange:

$$F_{crf} := \frac{\pi^2\cdot E}{(5.1\cdot Slenderness8)^2}$$

$F_{crf} = 41.1\cdot ksi$

Weighted Average: Web / Flange Local Buckling:

$$F_{ca} := \frac{F_{cw}\cdot Area_{web} + F_{cf}\cdot Area_{flange}}{Area}$$

Specification 4.7.4: Local and Member Buckling Interaction: $F_{cr} := \min(F_{crf}, F_{crw})$

Structural factor of safety: $n_{\mu} = 1.95$

$$F_{rc} := \begin{cases} \dfrac{F_{ec}^{\frac{1}{3}} \cdot F_{cr}^{\frac{2}{3}}}{n_{\mu}} & \text{if } \dfrac{F_{cr}}{n_{\mu}} < F_{ec} \\ F_{cm} & \text{otherwise} \end{cases} \qquad F_{rc} = 2.318 \cdot ksi$$

Maximum allowable Axial compressive stress and Corresponding Axial Load:

$F_c := \min(F_{rc}, F_{ca}, F_{ch}, F_{crf}, F_{ec}, F_{cm})$ $\qquad F_c = 2.318 \cdot ksi$

$F_a := F_c \cdot Area$ $\qquad F_a = 6.026 \cdot kip$

Specification 3.4.2 Allowable Major Axis Bending:

Member Buckling (3.4.14): $Slenderness_{14} := \dfrac{2 \cdot L_b \cdot S_x}{\sqrt{I_y \cdot J}}$ $\qquad Slenderness_{14} = 87.429$

$$F_{bcx} := \begin{cases} 21 \cdot ksi & \text{if } Slenderness_{14} \le 123 \\ \dfrac{23600}{Slenderness_{14}} \cdot ksi & \text{if } Slenderness_{14} \ge 1680 \\ \left[23.9 - \left(0.238 \cdot \sqrt{Slenderness_{14}}\right)\right] \cdot ksi & \text{otherwise} \end{cases} \qquad F_{bcx} = 21.00 \cdot ksi$$

Member Buckling between stitch screws (3.4.11): $Slenderness_{ch} := \dfrac{Spacing_{stitching}}{r_{yc}}$

$$F_{bcxh} := \begin{cases} 21 \cdot ksi & \text{if } Slenderness_{ch} \le 21 \\ \dfrac{87000}{Slenderness_{ch}^{2}} \cdot ksi & \text{if } Slenderness_{ch} \ge 79 \\ \left[23.9 - \left(0.124 \cdot Slenderness_{ch}\right)\right] \cdot ksi & \text{otherwise} \end{cases} \qquad F_{bcxh} = 17.47 \cdot ksi$$

Local Buckling, Web (3.4.18): $Slenderness_{18} := \frac{h_w}{t_w}$

$$F_{bcxw} := \begin{cases} 28 \cdot ksi & \text{if } Slenderness_{18} \le 48 \\ \frac{1520}{Slenderness_{18}} \cdot ksi & \text{if } Slenderness_{18} \ge 75 \\ \left[40.5 - \left(0.270 \cdot Slenderness_{18}\right)\right] \cdot ksi & \text{otherwise} \end{cases}$$

$F_{bcxw} = 14.859 \cdot ksi$

Local Buckling, Flange (3.4.15): $Slenderness_{15} := \frac{2 \cdot b_f}{t_f}$ $Slenderness_{15} = 9.66$

$$F_{bcxf} := \begin{cases} 21 \cdot ksi & \text{if } Slenderness_{15} \le 6.5 \\ \frac{182}{Slenderness_{15}} \cdot ksi & \text{if } Slenderness_{15} \ge 10 \\ \left[27.3 - \left(0.930 \cdot Slenderness_{15}\right)\right] \cdot ksi & \text{otherwise} \end{cases}$$

$F_{bcxf} = 18.316 \cdot ksi$

Allowable Bending Moment per Section 4.7.3 (Weighted Average Bending Strength):

Dimensions:

$C_{cf} := \frac{d}{2} - t_w$ $C_{cf} = 4.418\,in$

$C_{cw} := \frac{d}{2} - \left(2 \cdot t_w\right)$ $C_{cw} = 4.336\,in$

$C_{if} := \frac{d}{2}$ $C_{if} = 4.5\,in$

$C_{tw} := C_{cw}$ $C_{tw} = 4.336\,in$

Flange Group Moment of Intertia: $I_f := 2 \cdot \left[\left(\frac{b \cdot t_f^{\,3}}{12}\right) + b \cdot t_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2\right]$ $I_f = 23.139\,in^4$

Web Group Moment of Intertia: $I_w := 2 \cdot \left[\frac{\left[t_w \cdot \left(d - 2 \cdot t_f\right)^3\right]}{12}\right]$ $I_w = 8.066\,in^4$

$$M_{ac} := \left(\frac{F_{bcxf} \cdot I_f}{C_{cf}}\right) + \left(\frac{F_{bcxw} \cdot I_w}{C_{cw}}\right) \qquad M_{ac} = 123.569 \cdot kip \cdot in$$

$$M_{at} := \left(\frac{F_{btf} \cdot I_f}{C_{if}}\right) + \left(\frac{F_{btw} \cdot I_w}{C_{cw}}\right) \qquad M_{at} = 149.781 \cdot kip \cdot in$$

Major Axis Allowable Bending Stress per Section 4.7.3:

$$F_{bma} := \frac{\min(M_{ac}, M_{at})}{S_x} \qquad F_{bma} = 20.473 \cdot ksi$$

Weighted Average for Web/Flange Local Buckling (Section 3.4):

$$F_{bcxa} := \frac{F_{bcxf} \cdot Area_{flange} + F_{bcxw} \cdot \frac{Area_{web}}{6}}{Area_{flange} + \frac{Area_{web}}{6}} \qquad F_{bcxa} = 17.771 \cdot ksi$$

Maximum Allowable Bending Stress for sections 3.4 & 4.7.3:

$$F_{bx} := \min\left(F_{bcx}, F_{bcxh}, \max\left(F_{bma}, F_{bcxw}, F_{bcxf}\right)\right) \qquad F_{bx} = 17.469 \cdot ksi$$

Major Axis Allowable Bending Moment: $M_x := F_{bx} \cdot S_x = 105.437 \cdot kip \cdot in$

Beam net stress capacity based upon axial stress utilization:

Axial compression in beam due to horizontal windward pressure at screen eave and windward mansard rise: $f_a := p_{w\_wall} \cdot Spacing \cdot \left(h_{eave} + Rise\right)$

$$\frac{f_a}{F_a} = 0.302$$

Net allowable bending moment: $M_n := M_x \cdot \left(1 - \frac{f_a}{F_a}\right) - \left(M_P + M_{Rise}\right)$

$$M_n = 27.557 \cdot kip \cdot in$$

Note: If M_n is negative, trial section N.G., select a deeper beam.

$w := Spacing \cdot p_{roof}$

Allowable span per M_n: $L_M := \sqrt{\frac{8 \cdot M_n}{w}}$

$L_M = 265.605\,in$

Check span limit based upon allowable deflection:

Since deflection limit = L/60, then let: $Limit_\Delta := 60$

$L_\Delta := \sqrt[3]{\frac{384 \cdot E \cdot I_x}{Limit_\Delta \cdot 5 \cdot w}}$

$L_\Delta = 482.544\,in$

Allowable beam span based upon frame conditions specified: $AllowableSpan := min(L_M, L_\Delta)$

$AllowableSpan = 22.134 \cdot ft$