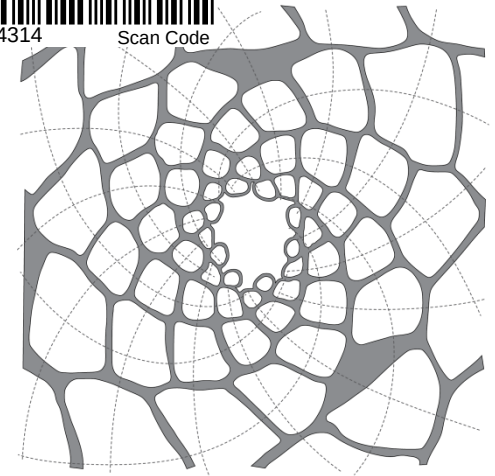




laufs engineering design

LED

Solar Canopy

Candelario and Sutton Canopies

LED facade report – structural report

date: October 21th 2015

architect: M.A.L

client: Situ Studio

detail structural façade engineering: Laufs Engineering Design – LED

Material Properties

According to Aluminum Design Manual 2010

6061 T6

non weld-affected zones:

ALLOY	TEMPER	PRODUCT	THICKNESS in.	F_{tu} ksi	F_{ty} ksi	F_{cy} ksi	F_{su} ksi	E ksi
6061	-T6, T651	Sheet & Plate	0.010 to 4.000	42	35	35	27	10,100
	-T6, T6510, T6511	Extrusions	All	38	35	35	24	10,100
	-T6, T651	Rod & Bar	up thru 8.000	42	35	35	25	10,100
	-T6	Drawn Tube	0.025 to 0.500	42	35	35	27	10,100
	-T6	Pipe	All	38	35	35	24	10,100

weld-affected zones:

ALLOY	TEMPER	PRODUCT	THICKNESS in.	F_{tuw} ksi	F_{tyw} ksi	F_{cyw} ksi	F_{suw} ksi	E ksi
6061	T6, T651, T6510, T6511 ¹	All		24	15	15	15	10,100
6061	T6, T651, T6510, T6511 ²	All	over 0.375	24	11	11	15	10,100

Loading Summary

per New York City Building code and ASCE-7-05

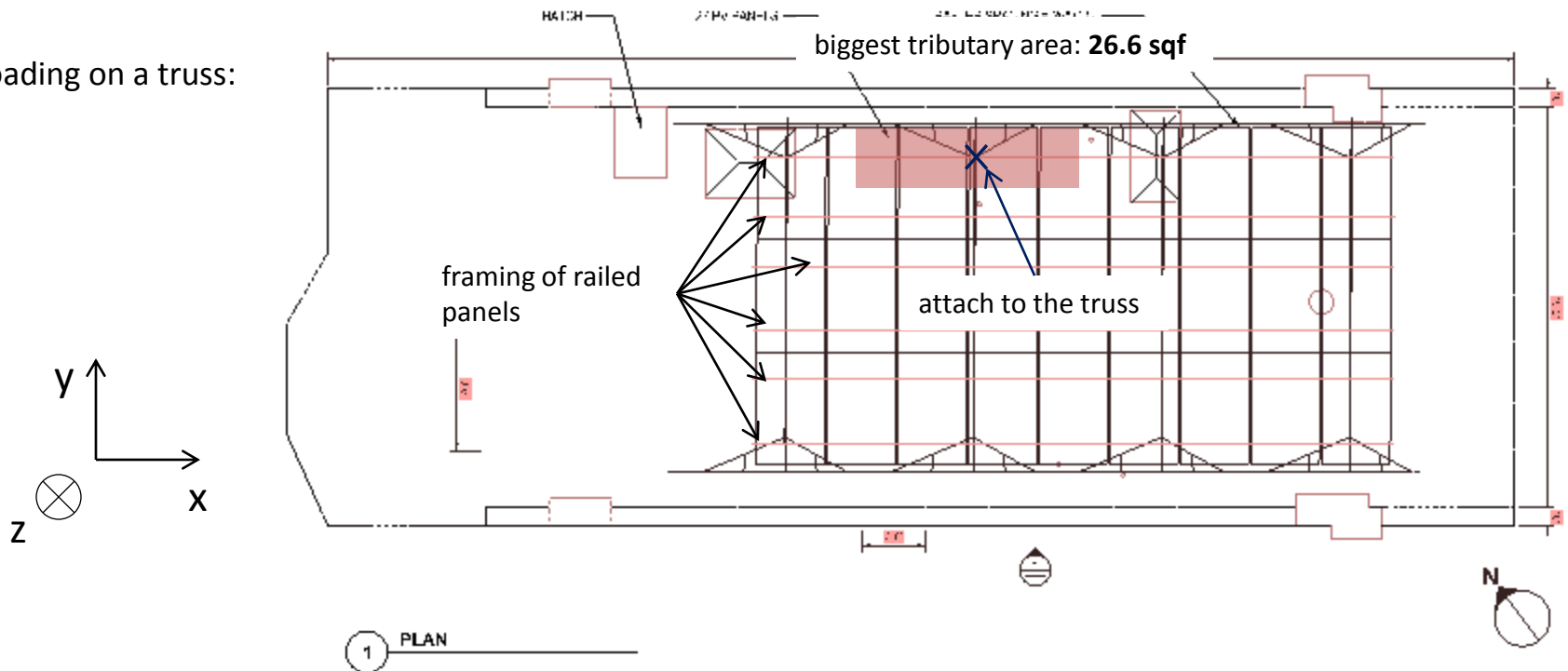
values provided by Tough Trac loading sheet:

DL	according to material densities
SDL	panel&frame weight if 3.5 psf:
Wind	max uplift/downforce: -14.75 psf / +18.44 psf
Snow	snow load applied on panels: 18.9 psf

Sutton: Structural Model

overview

loading on a truss:



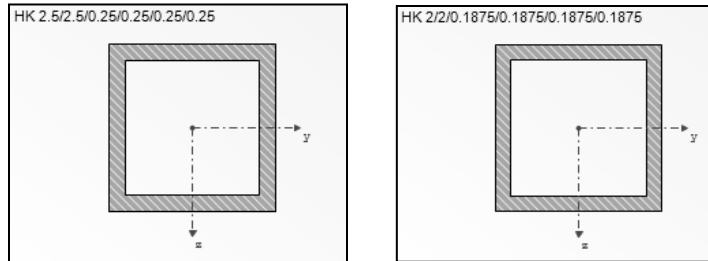
Point load: Surface load*tributary area

	Px [lbf]	Py [lbf]	Pz [lbf]
SDL	0	0	81
WL (uplift)	-35	-35	-392
WL (downforce)	43	43	490
SL	0	0	504

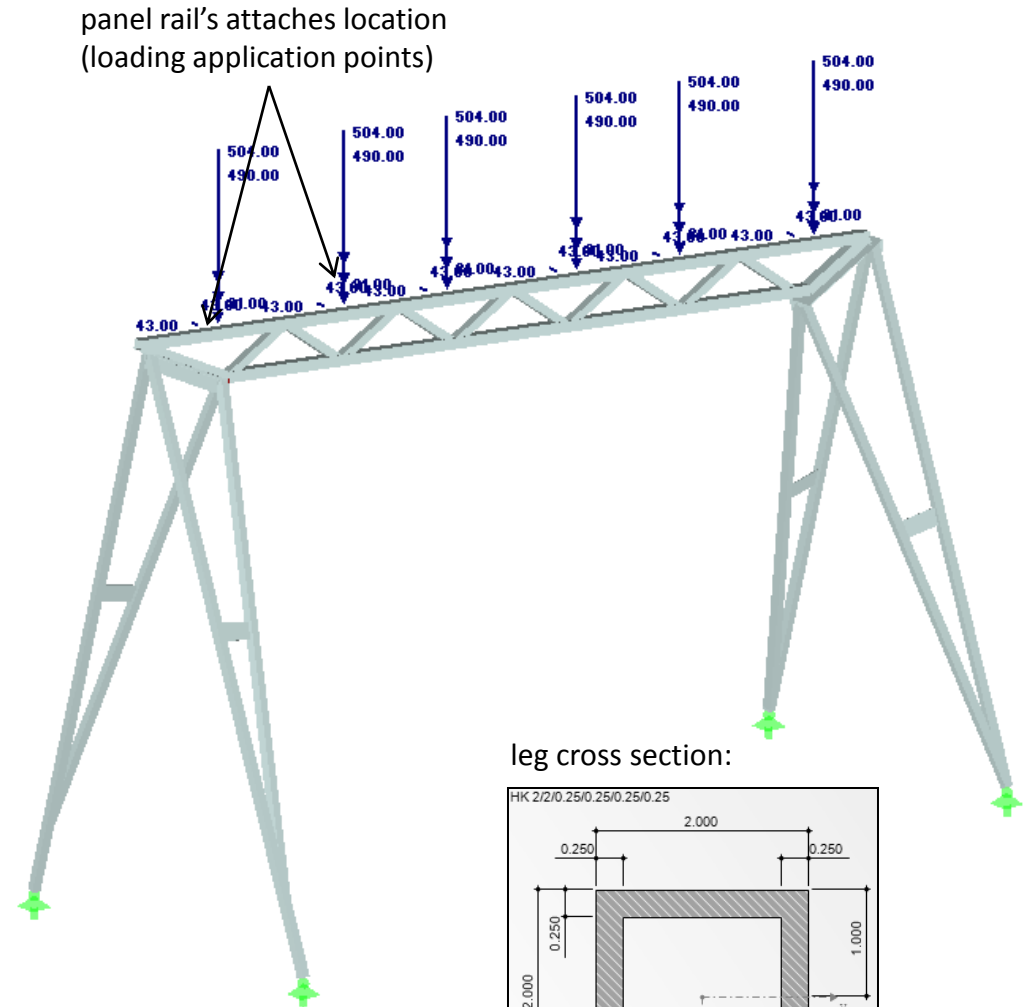
Sutton: Structural Model

overview of the model geometry

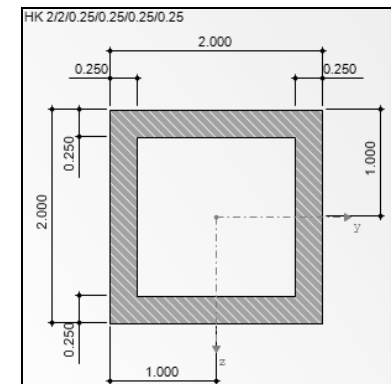
aluminum truss cross section:



connecting member between the truss and the legs:



leg cross section:



Sutton: Structural Model – Deflection Limits - Results

under DL + 0.75*WL + 0.75*S

As specified according to NYC building code (1604.3)

$$L_{\max} = L/60$$

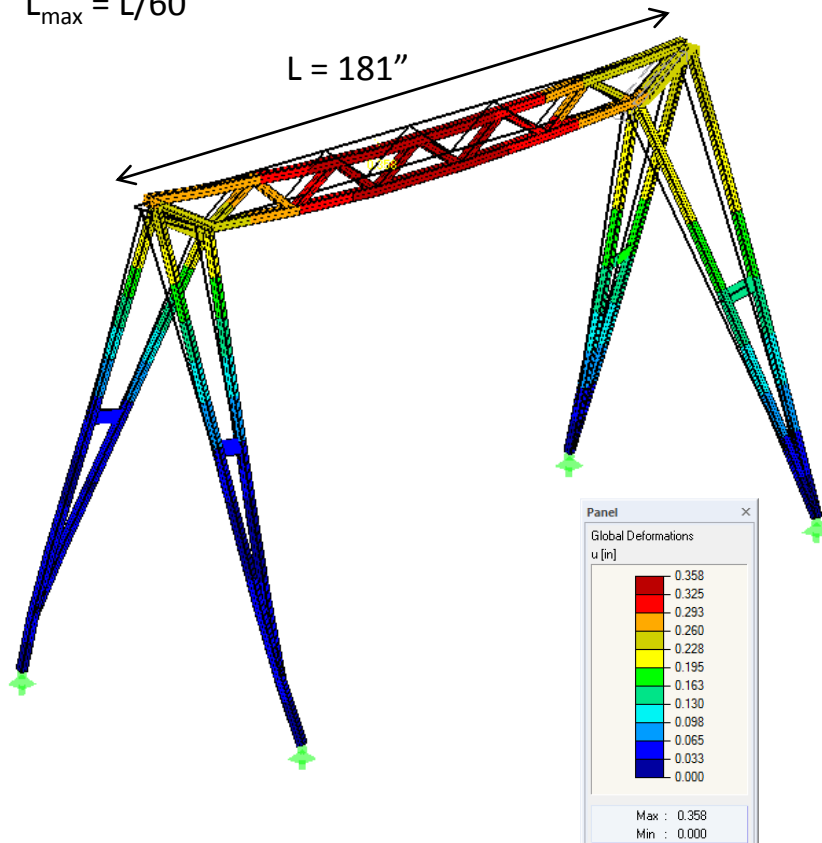


Figure 6.1 Deflection diagram:

maximum displacement = 0.36" = L/ 502 < L/60 - ok

TABLE 1604.3
DEFLECTION LIMITS^{a, b, c, h, i}

CONSTRUCTION	L	S or W _t	D + L _{dg}
Roof members: ^e			
Supporting plaster ceiling	l/360	l/360	l/240
Supporting nonplaster ceiling	l/240	l/240	l/180
Not supporting ceiling	l/180	l/180	l/120
Floor members	l/360	—	l/240
Exterior walls and interior partitions:			
With brittle finishes	—	l/120	—
With flexible finishes	—	l/120	—
Farm buildings	—	—	l/180
Greenhouses	—	—	l/120

For SI: 1 foot = 304.8 mm.

- For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed $l/60$. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed $l/150$. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed $l/90$. For roofs, this exception only applies when the metal sheets have no roof covering.
- Interior partitions not exceeding 6 feet in height and flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.13.
- See Section 2403 for glass supports.
- For wood structural members having a moisture content of less than 16 percent at time of installation and used under dry conditions, the deflection resulting from $L + 0.5D$ is permitted to be substituted for the deflection resulting from $L + D$.
- The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate drainage shall be investigated for ponding. See Section 1611 for rain and ponding requirements and Section 1503.4 for roof drainage requirements.
- The wind load is permitted to be taken as 0.7 times the "component and cladding" loads for the purpose of determining deflection limits herein.
- For steel structural members, the dead load shall be taken as zero.
- For aluminum structural members or aluminum panels used in roofs or walls of sunroom additions or patio covers, not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed $l/60$. For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed $l/120$.
- For cantilever members, l shall be taken as twice the length of the cantilever.

Figure 6.2 Deflection limits

Sutton: Structural Model - Stress Analysis

under general stress analysis

stress analysis carried on all members expected from the connection between the truss and the legs

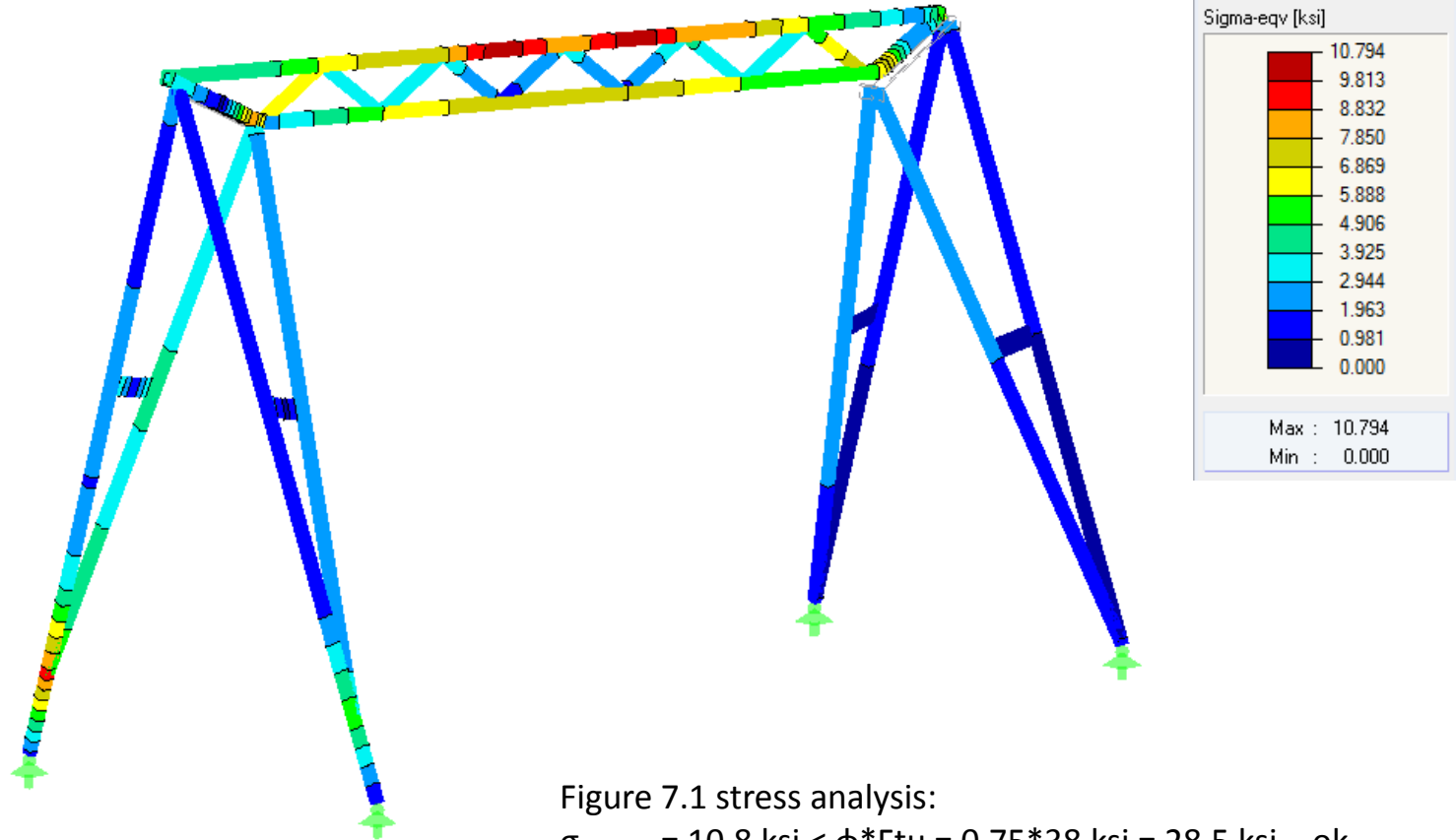


Figure 7.1 stress analysis:

$$\sigma_{eq,max} = 10.8 \text{ ksi} < \phi * F_{tu} = 0.75 * 38 \text{ ksi} = 28.5 \text{ ksi} - \text{ok}$$

Sutton: Structural Model

stability and vibration analysis

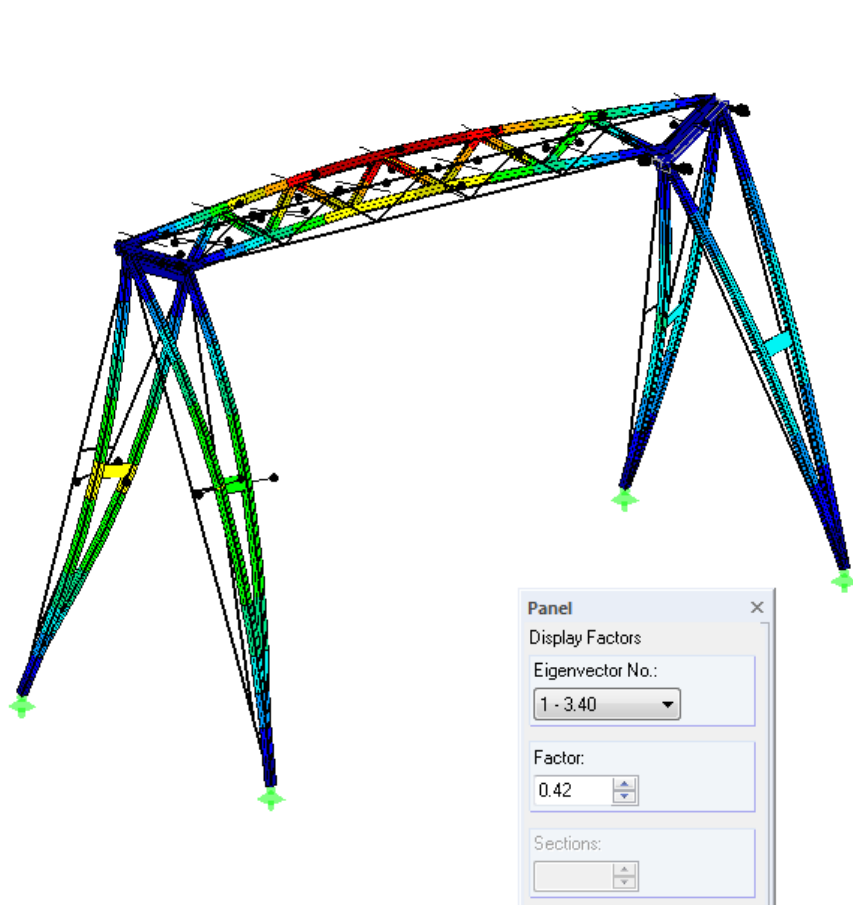


Figure 8.1 stability analysis: under DL + WL + SL
first buckling factor: $3.4 > 1$ - ok

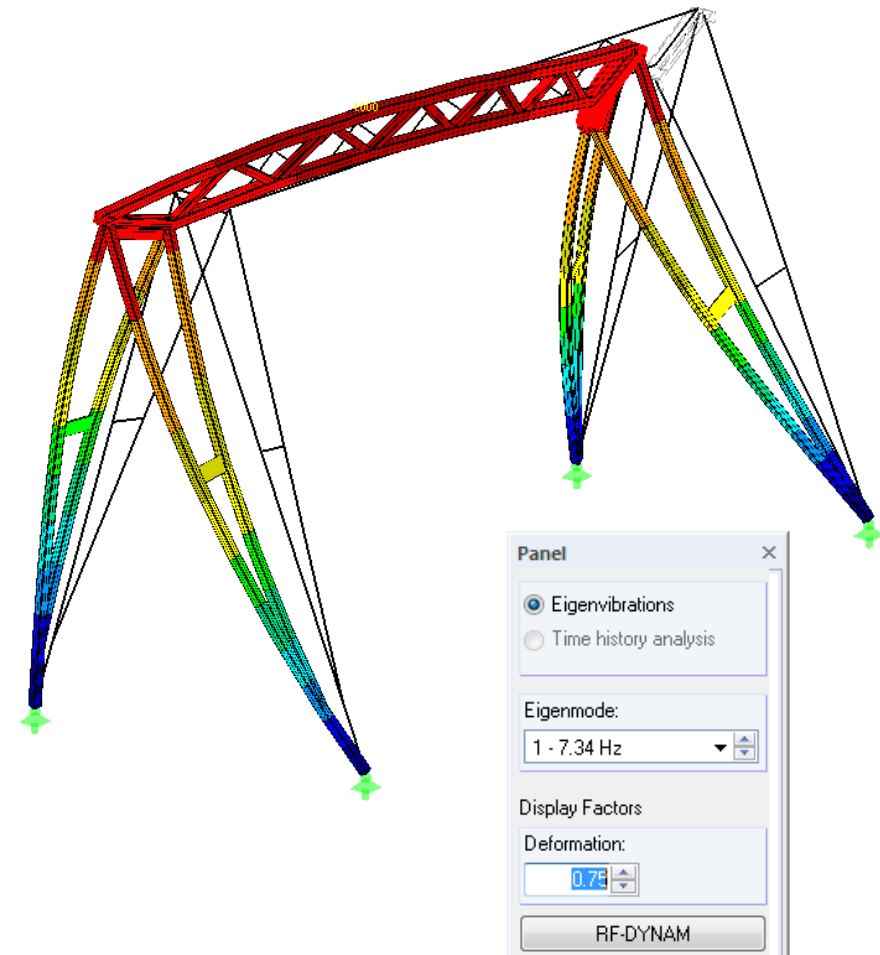


Figure 8.2 dynamic analysis:
first natural frequency: 7.34 Hz > 1.5 - ok
(therefore vibration induced by wind is not likely to happen)

Sutton: Structural Model – reaction forces

factored reaction forces

Reaction forces:

downward wind forces:

Reaction forces of node type:	max Px [kip]	max Py [kip]	max Pz [kip]
1	0.8	0.5	2.3
2	0.9	0.5	1.9

max reaction forces on worst loaded anchor:

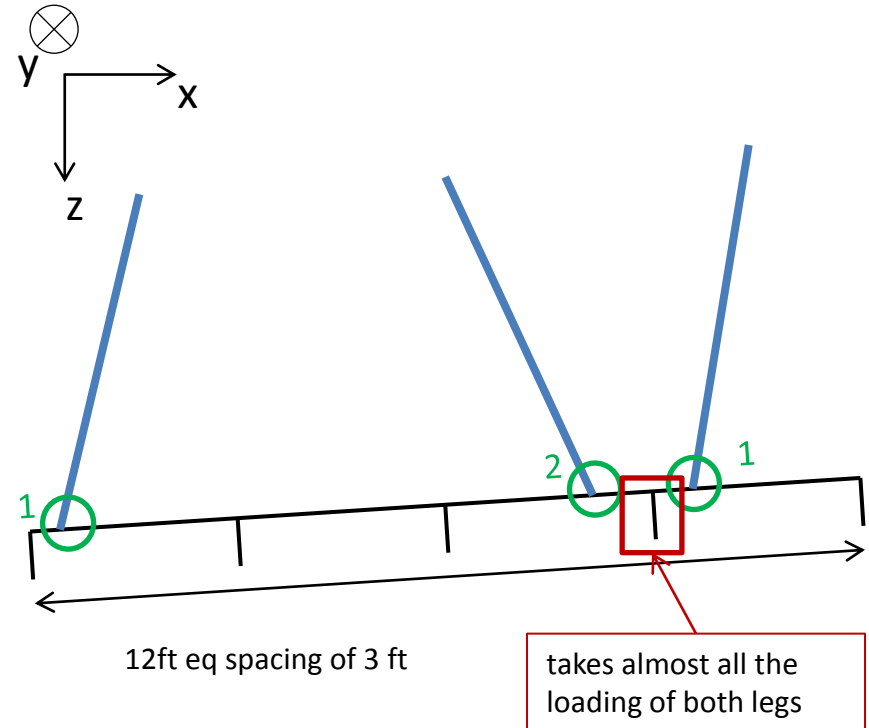
Maximum shear: 2 kip; Maximum Pz: 4.2 kip

uplift wind forces:

Reaction forces of node type:	max Px [kip]	max Py [kip]	max Pz [kip]
1	0.2	0.2	-0.8
2	0.2	0.5	-1

max reaction forces on worst loaded anchor:

Maximum shear: 0.8 kip; Maximum uplift: -1.8 kip



Sutton: Structural Model – Bracket Plates

stress analysis

internal forces in the cross section:

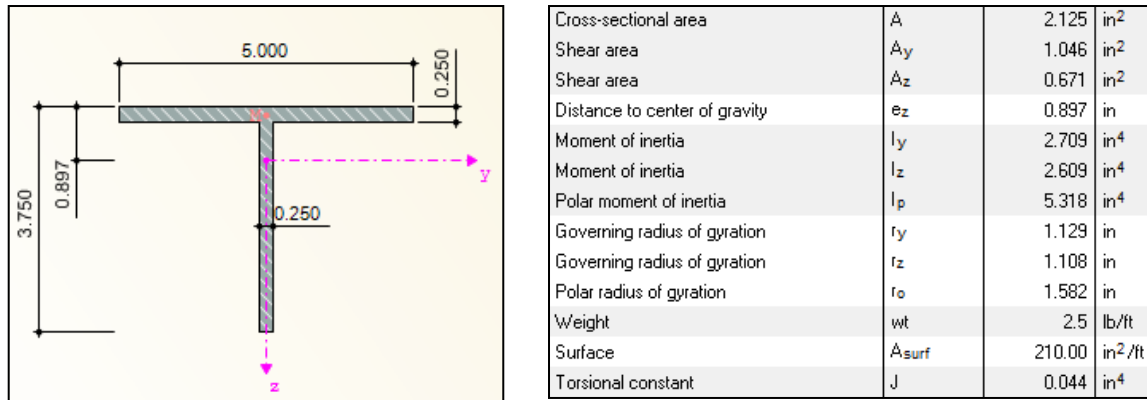


Figure 10.1 cross section geometry and properties

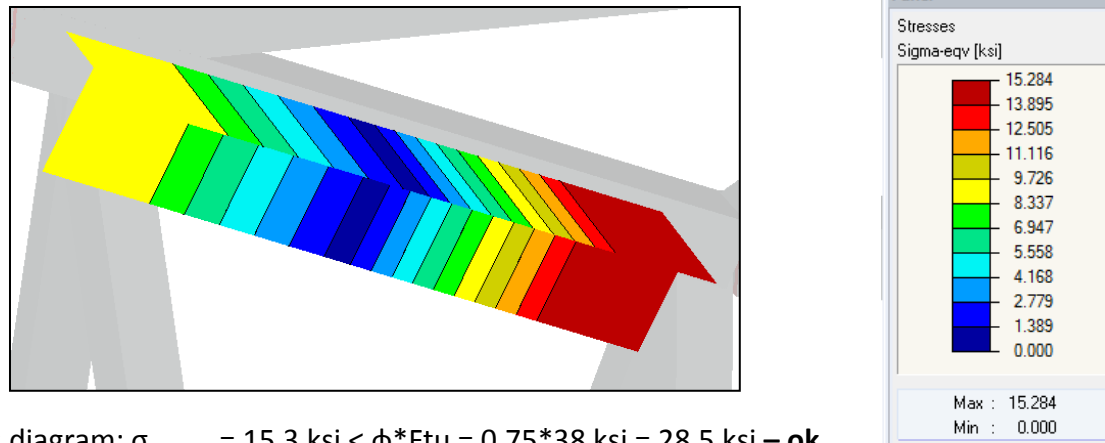


Figure 10.2 stress diagram: $\sigma_{eq,max} = 15.3 \text{ ksi} < \phi * F_{tu} = 0.75 * 38 \text{ ksi} = 28.5 \text{ ksi} - \text{ok}$

Sutton: Structural Model – Bracket Plates

stress analysis: shear check in connecting parts of the bracket plates

internal forces in the cross section:

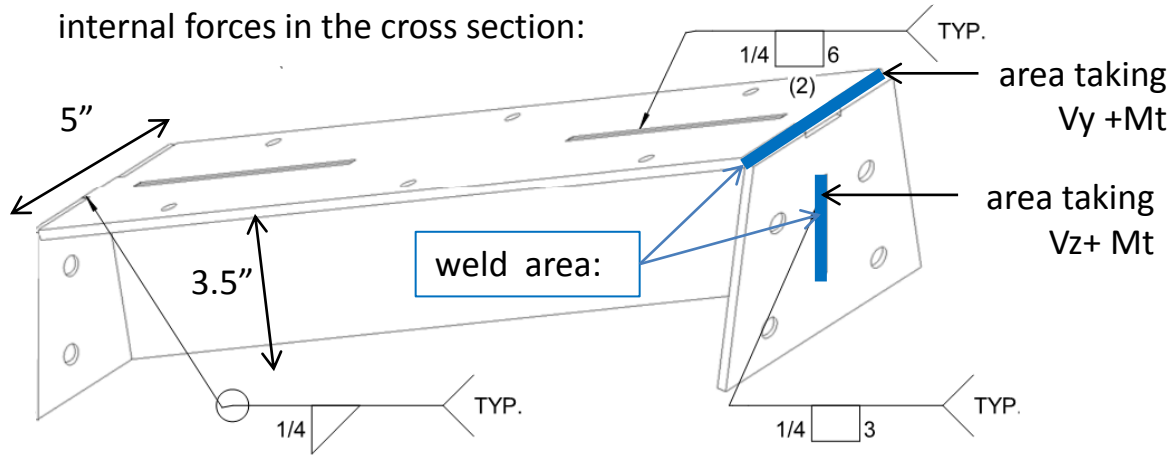


Figure 11.1 overview of the geometry

$max N [kip]$	$max V_y [kip]$	$max V_z [kip]$
1.2	0.2	3.7
$max M_t [kip.in]$	$max M_y [kip.in]$	$max M_z [kip.in]$
0.8	10.5	0.5

Figure 11.2 internal forces in the bracket plate

Shear capacity: aluminum capacity of welded areas: $0.75 \times 15 = 11.25$ ksi (ADM G.1)

shear due to torsion (taken by both weld areas): $v = M_t \cdot t / J = 0.8 \times 0.25 / 0.334 = 4.54$ ksi

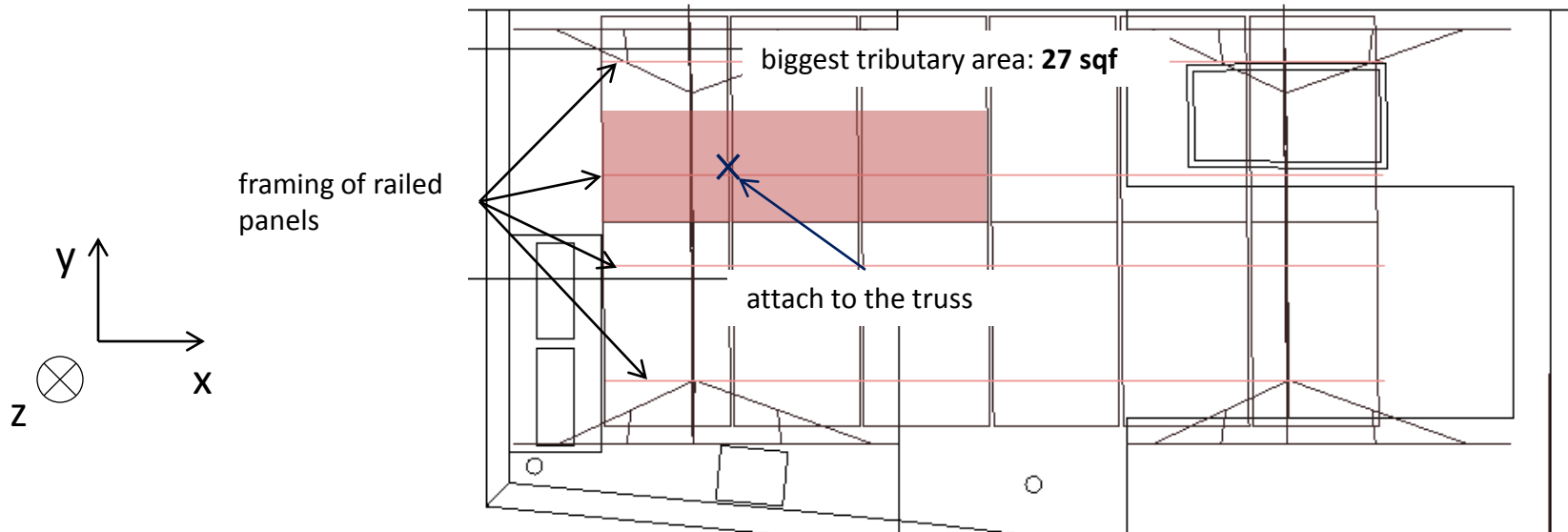
horizontal shear: utilization in shear: $(0.2 / (0.25 \times 5) + 4.54) / 11.25 = \mathbf{0.42} < 1 - \mathbf{ok}$

vertical shear: utilization in shear: $(3.7 / (0.25 \times 3) + 4.54) / 11.25 = \mathbf{0.84} < 1 - \mathbf{ok}$

Candelario: Structural Model

overview

loading on a truss:



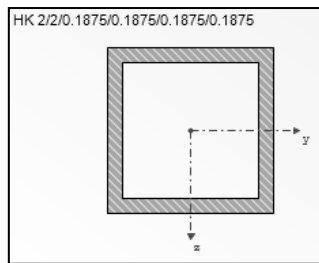
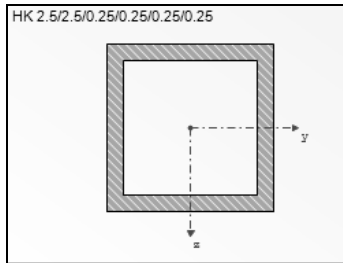
Point load: Surface load*tributary area

	Px [lbf]	Py [lbf]	Pz [lbf]
SDL	0	0	81
WL (uplift)	-35	-35	-400
WL (downforce)	43	44	500
SL	0	0	510

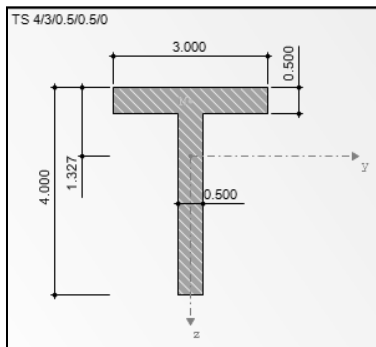
Candelario: Structural Model

overview of the model geometry

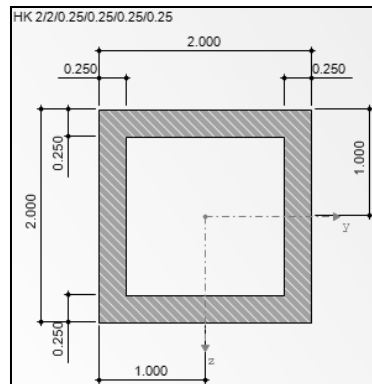
aluminum truss cross section:



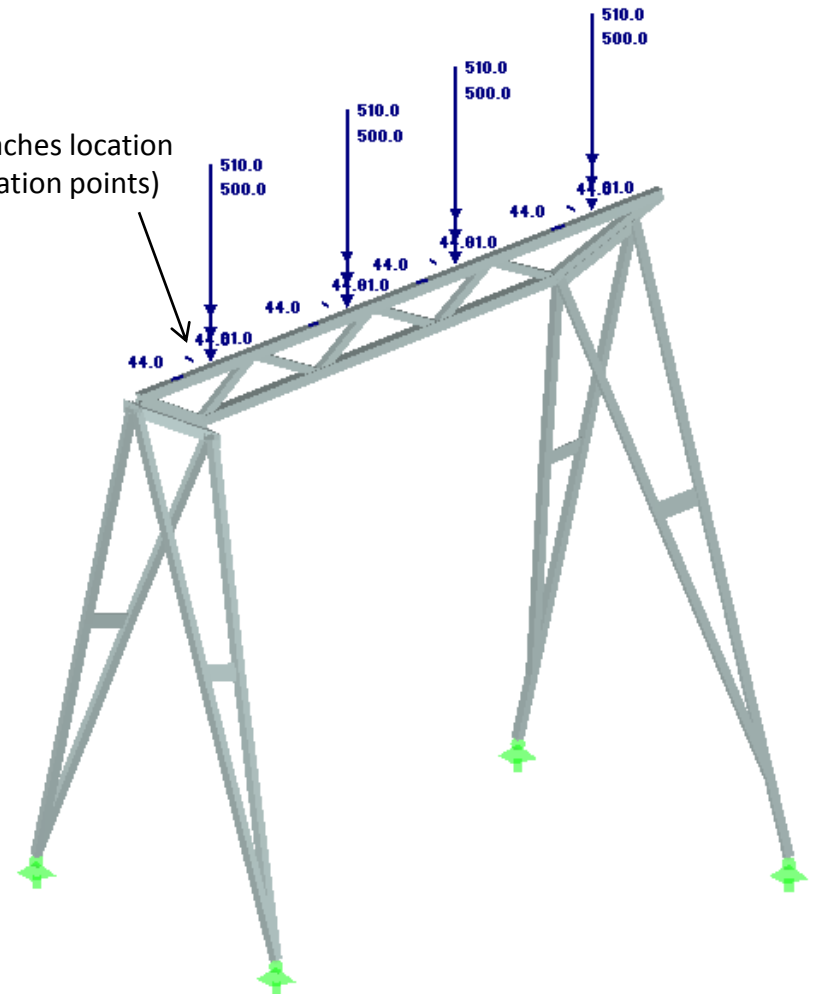
connecting member between the truss and the legs:



leg cross section:



panel rail's attaches location
(loading application points)



Candelario: Structural Model – Deflection Limits - Results

under $DL + 0.75 \cdot WL + 0.75 \cdot S$

As specified according to NYC building code (1604.3): $L_{max} = L/60$

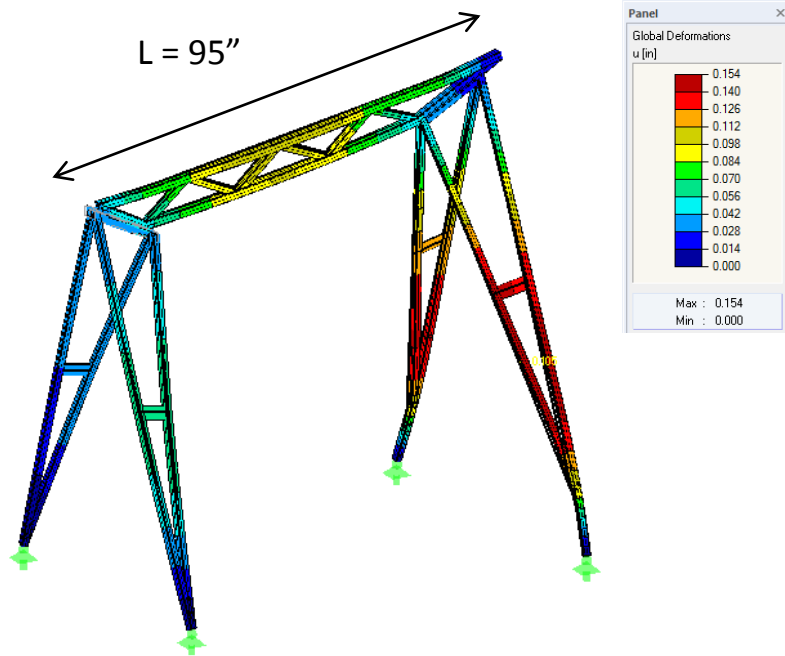


Figure 14.1 Deflection diagram:
maximum displacement = $0.105'' = L/905 < L/60$ - ok

TABLE 1604.3
DEFLECTION LIMITS^{a, b, c, h, i}

CONSTRUCTION	L	S or W _t	D + L _{o.g}
Roof members: ^e			
Supporting plaster ceiling	$L/360$	$L/360$	$L/240$
Supporting nonplaster ceiling	$L/240$	$L/240$	$L/180$
Not supporting ceiling	$L/180$	$L/180$	$L/120$
Floor members	$L/360$	—	$L/240$
Exterior walls and interior partitions:			
With brittle finishes	—	$L/120$	—
With flexible finishes	—	$L/120$	—
Farm buildings	—	—	$L/180$
Greenhouses	—	—	$L/120$

For SI: 1 foot = 304.8 mm.

- For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed $L/60$. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed $L/150$. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed $L/90$. For roofs, this exception only applies when the metal sheets have no roof covering.
- Interior partitions not exceeding 6 feet in height and flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.13.
- See Section 2403 for glass supports.
- For wood structural members having a moisture content of less than 16 percent at time of installation and used under dry conditions, the deflection resulting from $L + 0.5D$ is permitted to be substituted for the deflection resulting from $L + D$.
- The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate drainage shall be investigated for ponding. See Section 1611 for rain and ponding requirements and Section 1503.4 for roof drainage requirements.
- The wind load is permitted to be taken as 0.7 times the "component and cladding" loads for the purpose of determining deflection limits herein.
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- For cantilever members, L shall be taken as twice the length of the cantilever.

Figure 14.2 Deflection limits

Candelario: Structural Model - Stress Analysis

under general stress analysis

stress analysis carried on all members expected from the connection between the truss and the legs

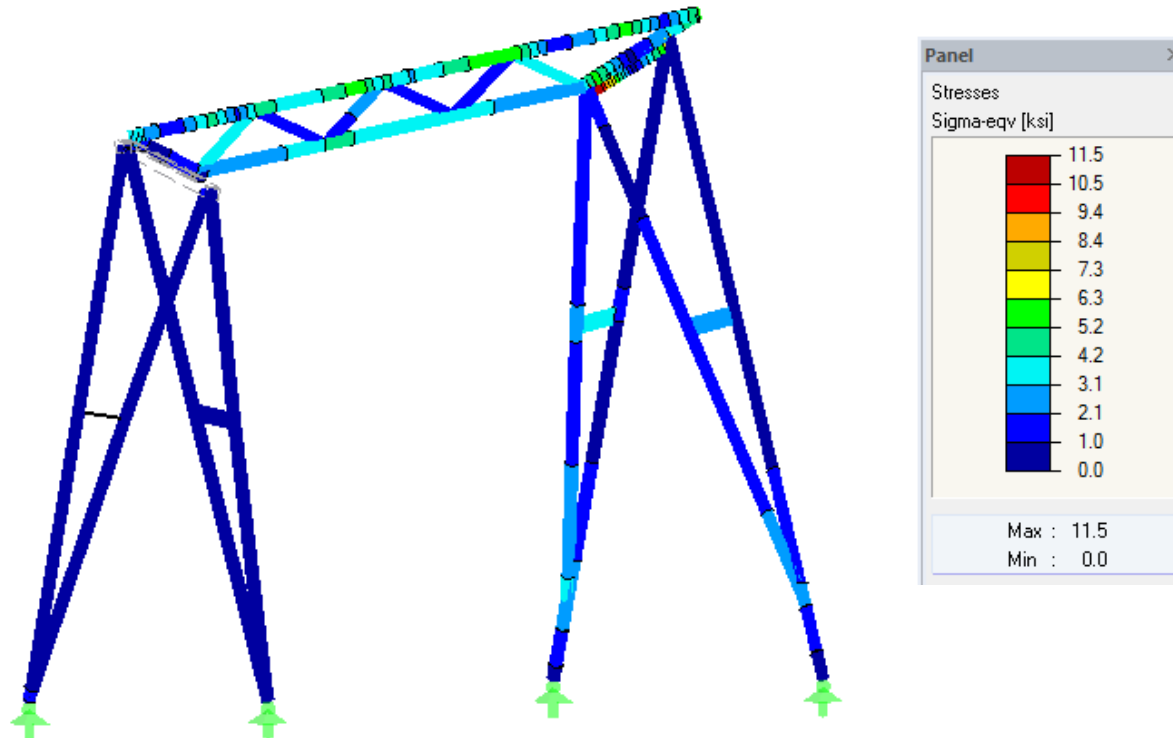


Figure 15.1 stress analysis:

$$\sigma_{eq,max} = 11.5 \text{ ksi} < \phi * F_{tu} = 0.75 * 38 \text{ ksi} = 28.5 \text{ ksi} - \text{ok}$$

Candelario: Structural Model

stability and vibration analysis

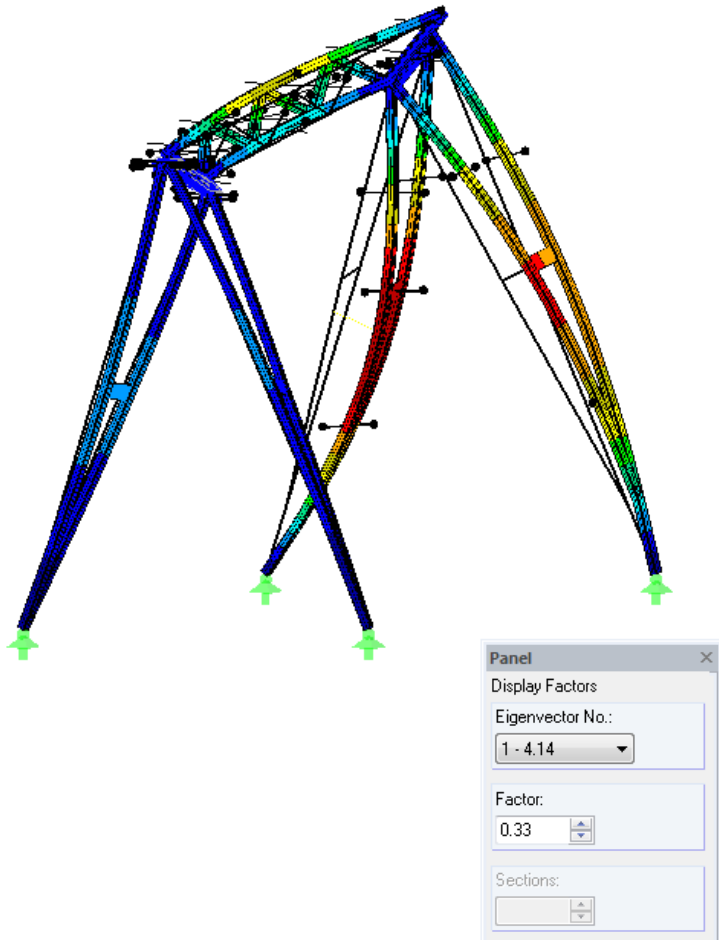


Figure 16.1 stability analysis: under DL + WL + SL
first buckling factor: $4.14 > 1$ - ok

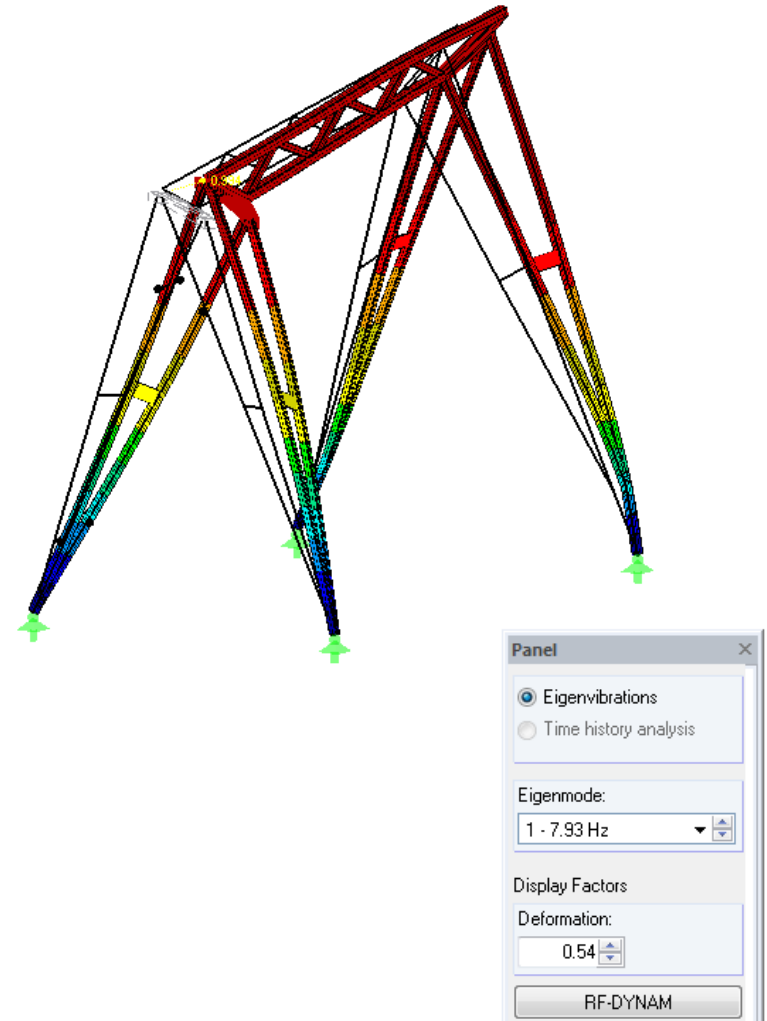


Figure 16.2 dynamic analysis:
first natural frequency: $7.93 \text{ Hz} > 1.5$ - ok
(therefore vibration induced by wind is not likely to happen)

Candelario: Structural Model – reaction forces

factored reaction forces

Reaction forces:

downward wind forces:

Reaction forces of node type:	max Px [kip]	max Py [kip]	max Pz [kip]
1	0.7	0.2	1.7
2	0.6	0.2	1.4

max reaction forces on worst loaded anchor:

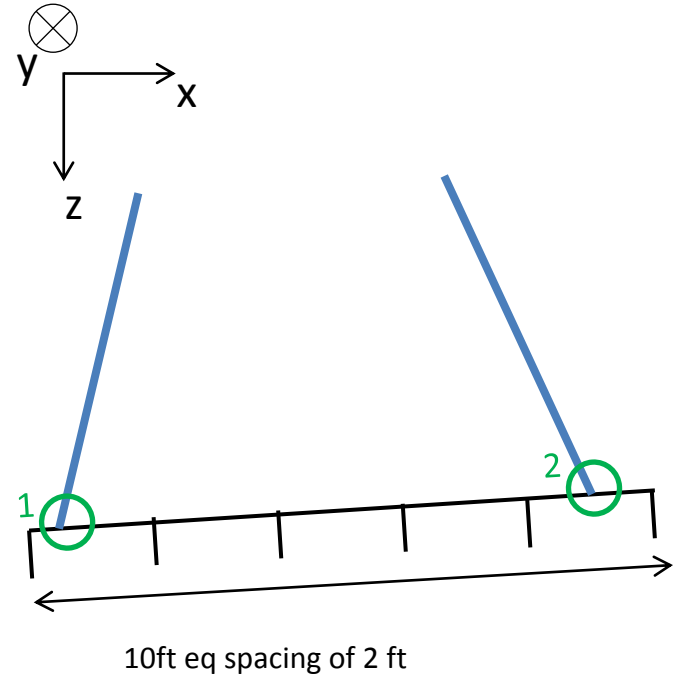
Maximum shear: 0.95 kip; Maximum Pz: 1.7 kip

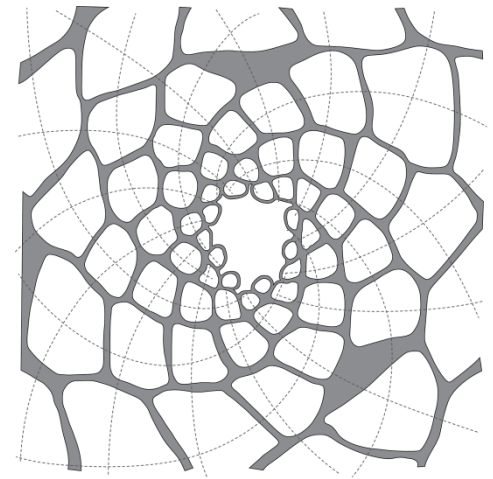
uplift wind forces:

Reaction forces of node type:	max Px [kip]	max Py [kip]	max Pz [kip]
1	0.1	0.2	-0.6
2	0.1	0.4	-0.7

max reaction forces on worst loaded anchor:

Maximum shear: 0.4 kip; Maximum uplift: -0.2 kip





laufs engineering design

LED

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