SAMPLE
TC6K Elevator Design Summary

Order No. 37163712-PO-47201

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Elevator System Design Parameters

Nominal Height of Elevator Rail: 205 ft.
Nominal Speed of Elevator Cab: 110 fpm.
Over Speed Emergency Stopping Trip Speed, Elevator Cab: 135 fpm.
Stopping Distance of Elevator Cab: 10 inches per code, designed to 5 inches average.
Gross Design Load, Elevator Cab: 15,500 lbs.
Rated Load, Elevator Cab: 6,000 lbs.
Design Cargo Load, Elevator Cab: 7500. lbs. (125% Rated Load for Passengers)
Empty Elevator Cab Estimated Weight: 8,000. lbs.
Nominal Rail System Width: 48 in.
Horizontal Spacing Between Wheels: 42 in. (approx.)
Rail Nominal Length: 10 ft.
Rail Attachment Points to Structure: Nominally each 13ft-5in. vertically along the rail column, anywhere along 10 ft. rail section. One support span is 14ft-7in.
Rail Support Brackets Bolt into Structural Steel at 13 ft. 5 inch increments.
Structure Support Length: 36 in.
Rail Support - Structure Support Bolting: 2 sets of ¾ in. bolts with 8 in. vertical separation
Vertical Wheel Spacing: 13.9 ft. (approx)
Elevator Cab Size: 6 ft. x 6 ft. x 7.25 ft. (w x d x h) , approx..

Site Specifications

Design Basis: International Building Code 2003
Supplemental Codes:
ASCE Standard ASCE/SEI 7-05
AISC Manual of Steel Construction 9th Ed.
AISC Specification for Structural Joints Using ASTM A325 or A490 Bolts

Elevator Site Location: Tuscaloosa, Alabama.
Minimum Design Metal temperature: 0 degrees F
Maximum Ambient Temperature 100 degrees F (50% relative humidity)
Elevator Site Elevation: 138 ft. above mean sea level (MSL)

Wind:
100 MPH
Exposure C
Category III
Importance Factor: 1.15

Seismic:
Seismic Group II
Seismic Category C
Site Class D
Importance Factor, I_e, 1.25
S_s 25.5%
S_1 10.6%

Snow:
Exposure Category C
Load 5 pounds per square foot
r: 1.0
Importance Factor 1.1

Foundation Loading

As defined in the specification, the elevator cab "Gross Design Load" is taken as 15,500 lbs. This figure reflects a rated load of 6,000 lbs, which by code requirement translates into a "design cargo load" of 7,500 lbs. (125% of rated load). The empty elevator cab weight is estimated to be 8,000 lbs. Thus the "Total Gross Design Load" is 15,500 lbs. (8,000 lbs + 7500 lbs.)

The weight of the rack is calculated as a 210 ft. length of 4.5 in. x 3.5 in. carbon steel bar with a weight density of 0.279 lbs per cubic inch.

In order to calculate the weight of the rail assembly (as well as to assist in structural analysis calculations) a 3-D CAD model of the rail assembly is created using the ALIBRE computer code . The code has the ability to calculate the physical properties, including the mass, of the rail geometry.

The next eight pages contain a summary of results sheet of the elevator component assembly weights, the load imposed on the base foundation, illustrations of the rail component geometry, and respective component weight calculations summaries. The geometry shown may differ slightly from the final design; the differences are not significant enough to alter the calculated weights outside of the window of calculation accuracy.

TC6K 48 Inch Rail Elevator - SAMPLE Refinery - CCR Elevator Weight Summary

Component		Weight lbs.
Lower Rail Assembly		
Rail Pair L7X4X3/8		271.6
4- Horizontal Channels 3X3X1/4		118.6
2- 6X3X1 1/4 Plate		11.6
2- 1 ½ Alignment Clips		0.76
Footing Piece		214
	Total	616.6
General Rail Assembly		
Rail Pair L7X4X3/8		271.6
6- Horizontal Channels 3X3X1/4		142.2
4- 6X3X1 ¼ Plate		23.1
4- 1 ½ Alignment Clips		1.5
	Total	438.4
Mounting Bracket Parts		
Pair L5X5X3/8 for Connection Bracket Support		53.9
Universal Rail Connection Bracket		51.2
	Total	105.1
Beam Support Ass'y (not included in structure total)		181.9
Rack		11014.5
Total Rail Assembly		
Lower Assembly Section		616.6
20 X General Rail Assemblies		8768.4
14 X Mounting Bracket Parts		1471.4
Rack		11014.5
	Total Rail Assembly	21870.9
Total Downward Load on Foundation		
Total Rail Assembly		21870.9
Elevator Cab		15500
Emergency Stop Load		2924.8
Snow		180
Ice		201.6
	Total Downward Load on Foundation	40677.3

TC6K Elevator Rail Assembly
First, Second, & Third Rails, Rear View

TC6K Elevator Rail Assembly
First, Second, & Third Rails, Front View

TC6K Tower Elevator Rail

First Rail Assembly Detail

TC6K Tower Elevator Rail

Second and Upward Rail Assembly Detail

TC6K Tower Elevator
Universal Rail Connection Bracket
Attachment Pieces

TC6K Tower Elevator
Universal Rail Connection Bracket

TC6K Tower Elevator
Building Support Bracket

Loading on the Elevator Rail

The AISC Manual of Steel Construction Allowable Stress Design, Ninth Edition, Section A4 states that, “the nominal loads shall be the minimum design loads stipulated by the applicable code under which the structure is designed or dictated by the conditions involved.” The applicable code in this design case is taken as ASCE/SEI 7-05, Minimum Design loads for Buildings and Other Structures. Section 2.4, Combining Nominal Loads Using Allowable Stress Design, of this code specifies basic combinations of loads on structural members which must be considered in the analysis of the design. Component loads applicable to the design of the elevator include loads due to dead weight, normal and emergency stopping, ice, snow, wind, earthquake, and wind-on-ice. These loads are imparted onto the rail and rack assemblies by the elevator cab assembly.

A pictorial representation of the elevator cab (not an actual drawing of the cab) which illustrates the load applications of the cab on the rails is shown on page 13. This CAD representation is also used as the basis for a finite element model which simulates the load application the cab imparts on the rails under the various combinations of load.

The loads applied to the rails by the cab using the code-specified combinations of conditions are determined using the MathCad simultaneous equation solver and the ALGOR finite element code. Summary of results sheets are presented on pages 14-19. These loads form the basis for the structural analysis of the rail system and the determination of the load the rail imparts on the building structural supports.

Individual case loads are determined initially, then the loads are combined in a manner consistent with the ASCE (ASCE/SEI 7-05, 2.4) code requirements. The integrity of the rail structure and its components is then demonstrated using the allowable stress design rules of the code. The load components are determined as follows:

Combination of Loads ASCE/SEI 7-05, 2.4.1

Dead Loads: ASCE/SEI 7-05, 3.

Live Loads: ASCE/SEI 7-05, 4.

*Wind Loads: ASCE/SEI 7-05, 6.

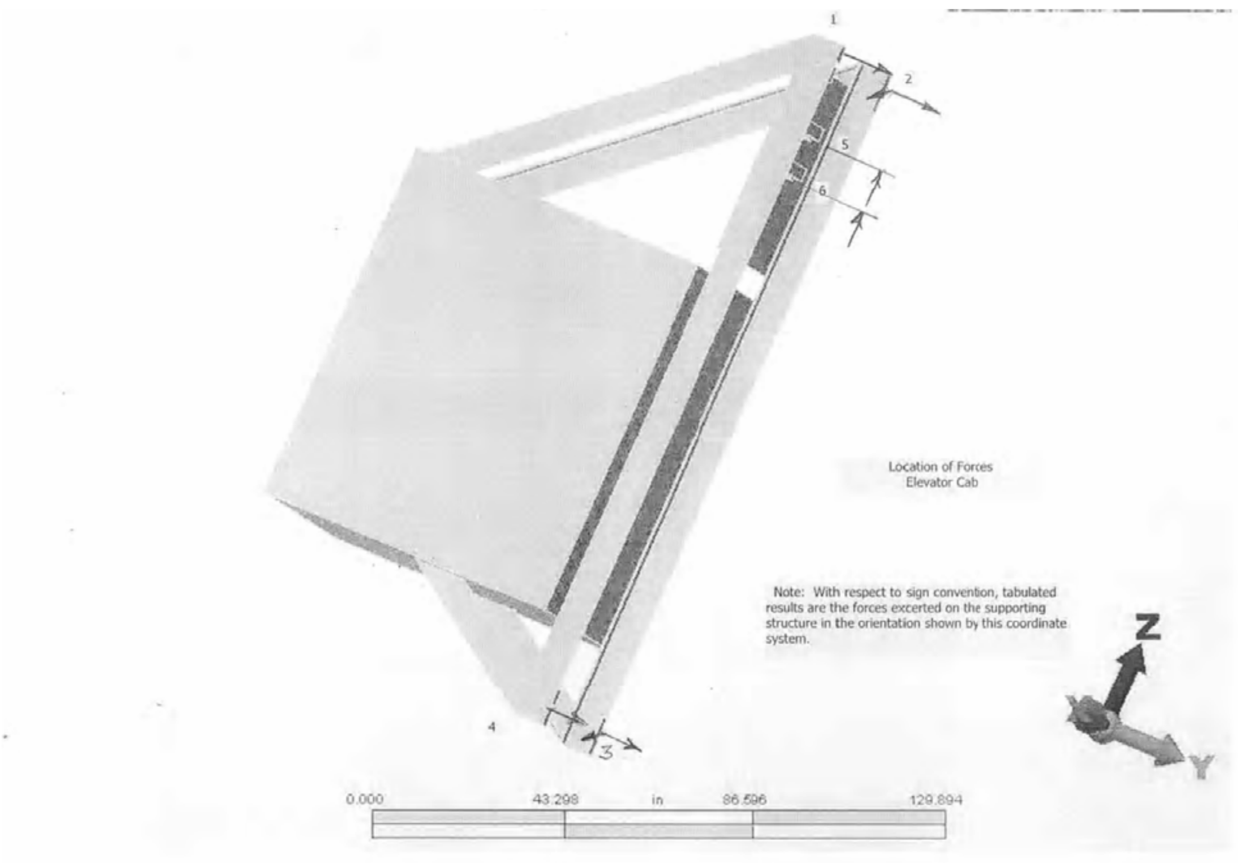
Snow Loads: ASCE/SEI 7-05, 7.

Ice Loads: ASCE/SEI 7-05, 10.

*Seismic Design: ASCE/SEI 7-05 - 11, 12, 13, 15, 16, 17, 18, 20, 21, 23

In seismic design, the designated codes were studied and applied; the final seismic load arrived at is estimated due to lack of supporting building structure data.

* ASCE (ASCE/SEI 7-05, 2.4.1) states that the most unfavorable effects from both wind and earthquake loads shall be considered where appropriate, but they need not be assumed to act simultaneously.



Simulated Elevator Cab

Used in Determining Load on Rails

TC6K Hunt CCR Tower Elevator
Individual Load Cases (IBC 2006 & ASCE 7-05)
Elevator Cab

Symbol	Load Case	Location	Fx lbs.	Fy lbs.	Fz lbs.
D	Dead Weight	1		-1288.7	
		2		-1288.7	
		3		1288.7	
		4		1288.7	
		5			-7285
		6			-8215
L	Braking Load - Smooth	1		-161.5	
		2		-161.5	
		3		161.5	
		4		161.5	
		5			-902.3
		6			-1039.7
L	Braking Load - Emg'y Stop	1		-243.1	
		2		-243.1	
		3		243.1	
		4		243.1	
		5			-1374.7
		6			-1550.1
W	Wind Load - Side (Wind in - X direction)	1		1124	
		2	-1336	-1124	
		3	-2070	-1881	
		4		1881	
		5			
		6			
W	Wind Load - Front (Wind in + Y direction)	1		668.1	
		2		668.1	
		3		1035.3	
		4		1035.3	

Symbol Key

D	Dead Load
Di	Weight of Ice
E	Earthquake Load
F	Load of fluids with well defined pressures and maximum heights
Fa	Flood Load
H	Load due to lateral earth pressure, ground water pressure, or pressure of bulk materials
L	Live Load
Lr	Live Roof Load
R	Rain Load
S	Snow Load
T	Self Straining Force
W	Wind Load
Wi	Wind -on-ice Load

Basic Combinations

1 D + F
2 D + H + F + L + T
3 D + H + F + (Lr or S or R)
4 D + H + F + 0.75(L + T) + 0.75(Lr or S or R)
5 D + H + F + (W or 0.7E)
6 D + H + F + 0.75(W or 0.7E) + 0.75L + 0.75(Lr or S or R)
7 0.6D + W + H
8 0.6D + 0.7E + H

E	Seismic Load	1		-1401.7	
		2	-278.3	-1446.7	
		3	-213.1	1376.2	
		4		1472.2	
		5			
		6			
S	Snow Load	1		-4.4	
		2		-4.4	
		3		4.4	
		4		4.4	
		5			-84
		6			-84
Di	Ice Load	1		-5	
		2		-5	
		3		5	
		4		5	
		5			-93
		6			-105
Wi	Wind-on-ice - side load	1		29.6	
	(wind in - X direction)	2	-35.5	-29.6	
		3	-54	-49.5	
		4		49.5	
		5			
		6			
Wi	Wind-on-ice - front load	1		10.7	
	(wind in+ Y direction)	2		10.7	
		3		34.1	
		4		34.1	
		5			
		6			

TC6K Hunt CCR Tower Elevator Cab — Basic Load Combinations (ASCE 7-05 (2.4))

Case	Symbol	Load Case	Location	Fx lbs.	Fy lbs.	Fz lbs.
1	D+F	Dead Weight	1		-1288.7	
		Fluids	2		-1288.7	
			3		1288.7	
			4		1288.7	
			5			-7285
			6			-8215
2	D + H + F + L + T	Dead Weight	Location			
		Earth Pressure				
		Fluids	1		-1531.8	
		Live Load	2		-1531.8	
		Self-Restraining Force	3		1531.8	
			4		1531.8	
			5			-8659.7
			6			-9765.1
3	D + H + F + (Lr or S or R)	Dead Weight	Location			
		Earth Pressure				
		Fluids	1		-1293.1	
		Live Roof Load, or	2		-1293.1	
		Snow Load, or	3		1293.1	
		Rain Load	4		1293.1	
			5			-7369
			6			-8299
4	D + H + F + 0.75(L + T) + 0.75(Lr or S or R)	Dead Weight	Location			
		Earth Pressure				
		Fluids	1		-1474.3	
		Live Load	2		-1474.3	
		Self-Restraining Force	3		1474.3	
		Live Roof Load, or	4		1474.3	
		Snow Load, or	5			-8379
		Rain Load	6			-9440.6

5a	D + H + F + W	Dead Weight	Location			
		Earth Pressure				
		Fluids	1		-164.7	
		Wind - X dir	2	-1336	-2412.7	
			3	-2070	-592.3	
			4		3169.7	
			5			-7285
			6			-8215
5b	D + H + F + W	Dead Weight	Location			
		Earth Pressure				
		Fluids	1		-620.6	
		Wind - Y dir	2		-620.6	
			3		2324	
			4		2324	
			5			-7285
			6			-8215
5c	D + H + F + 0.7E	Dead Weight	Location			
		Earth Pressure				
		Fluids	1		-2269.9	
		Earthquake	2	-194.8	-2301.4	
			3	-149.2	2252	
			4		2319.2	
			5			-7285
			6			-8215
6a	D + H + F + 0.75(W or 0.7E) + 0.75L + 0.75(Lr or S or R)		Location			
		Dead Weight	1		-631.3	
		Earth Pressure	2	-1002	-2317.3	
		Fluids	3	-1552.5	63.6	
		Wind - X dir	4		2885	
		Live Load	5			-8379.3
		Snow	6			-9440.6

6b	D + H + F + 0.75(W or 0.7E) + 0.75L + 0.75(Lr or S or R)		Location			
		Dead Weight	1		-973.2	
		Earth Pressure	2		-973.2	
		Fluids	3		2250.8	
		Wind - Y dir	4		2250.8	
		Live Load	5			-8379.3
		Snow	6			-9440.6
6c	D + H + F + 0.75(W or 0.7E) + 0.75L + 0.75(Lr or S or R)		Location			
		Dead Weight	1		-2525.6	
		Earth Pressure	2	-208.7	-2559.3	
		Fluids	3	-159.8	2506.4	
		Earthquake	4		2578.4	
		Live Load	5			-8379.3
		Snow	6			-9440.6
7a	0.6D + W + H	Dead Weight	Location			
		Wind - X dir				
		Earth Pressure	1		350.4	
			2	-1336	-1897.6	
			3	-2070	-1107.4	
			4		2654.6	
			5			-4371
			6			-4929
7b	0.6D + W + H	Dead Weight	Location			
		Wind - Y dir				
		Earth Pressure	1		-105.1	
			2		-105.1	
			3		1808.5	
			4		1808.5	
			5			-4371
			6			-4929

8	0.6D +0.7E + H	Dead Weight	Location			
		Earthquake				
		Earth Pressure	1		-1754.4	
			2	-194.8	-1785.9	
			3	-149.2	1736.5	
			4		1803.7	
			5			-4371
			6			-4929

Building Support Structure Loads

As designated in the system specification presented on the first page of this document, the elevator cab loading on the rails is transferred to the building structural supports by two sets of ¾ inch bolts separated 8 inches and aligned vertically.

An examination of the out-of plane (the plane being the plane of the rails) moments and the traverse lateral loads computed using the MathCad simultaneous equation solver for each of the ASCE load cases presented previously gives an indication of the worst case load conditions. A table of results of out-of-plane moments and traverse loads calculated from the data presented in the prior, "TC6K SAMPLE CCR Tower Elevator Cab Basic Load Combinations (ASCE 7-05 (2.4))", tables follows. It is seen that case 6c, or the "dead weight plus earthquake plus live load plus snow " load case imposes the largest out-of-plane moment on the rail structure about the X-axis, case 5a, "dead weight plus wind in x-direction" load case imposes the largest out-of-plane moment on the rail structure about the Z-axis, and cases 5a and 7a, also " dead weight plus wind in x-direction" load case imposes the largest transverse lateral load on the rail structure.

Out-of-Plane Moment
ASCE Load Cases

Case Number	Out-of-Plane Moment About X Axis	Out-of-Plane Moment About Z Axis	Maximum Transverse Load in X Direction
	In. – lbs.		
1	430426	0	0
2	511621	0	0
3	431895	0	0
4	492416	0	0
5a	430425	120200	3406
5b	491748	0	0
5c	763398	1974	344
6a	492441	90148	2554.5
6b	538408	0	0
6c	849170	2114	368.5
7a	258382	120196	3406
7b	319570	0	0
8	591221	1974	344

The case number refers to the case as designated in the table, "TC6K SAMPLE CCR Tower Elevator Cab Basic Load Combinations (ASCE 7-05 (2.4))" on the previous pages. Case 2 represents the normal operating case with emergency stop.

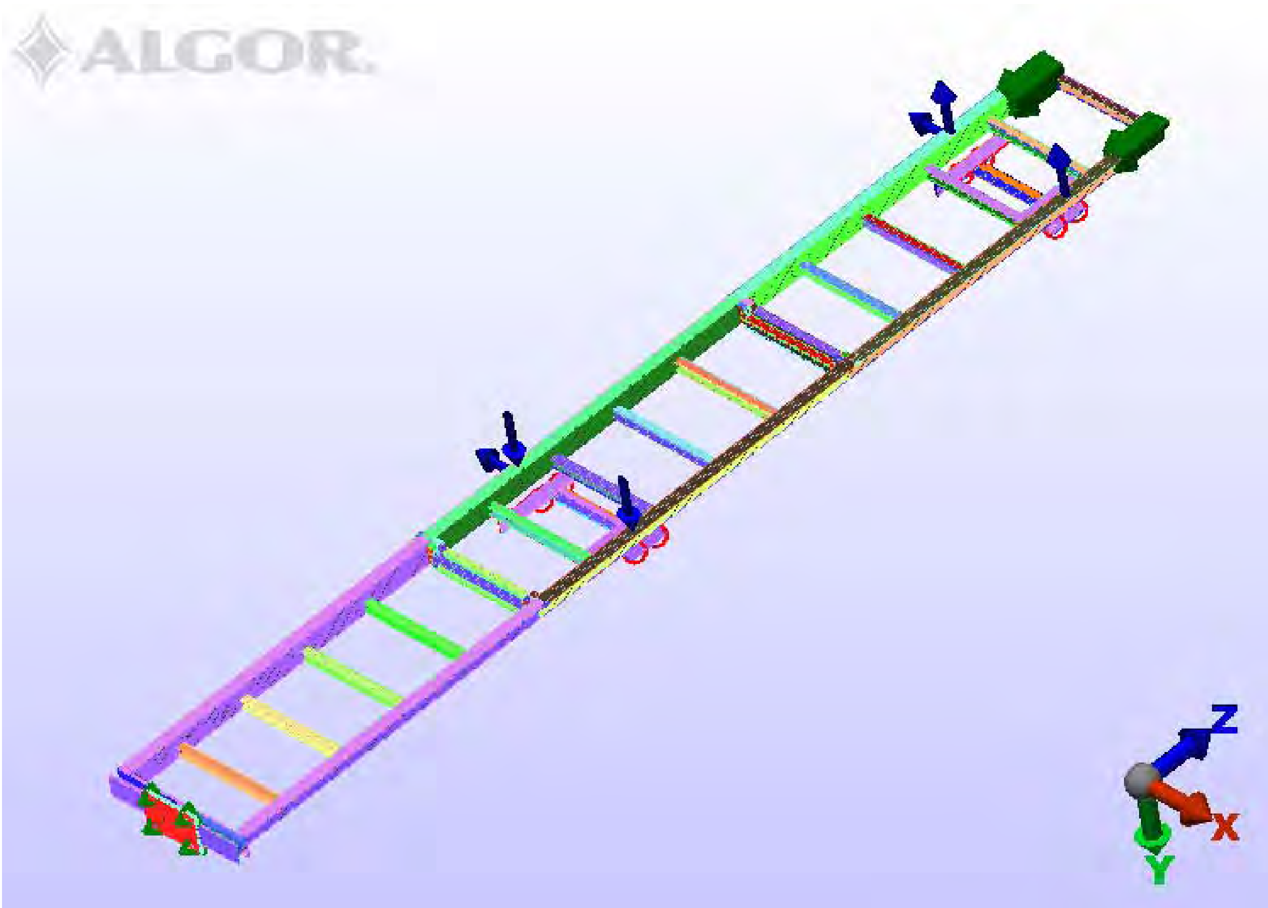
In order to determine the maximum loads imposed upon the building support structure, a finite element model of the rails is created using the ALGOR computer code and the model subjected to the loads applied by the elevator cab. A three rail segment is adequate for analysis purposes. Load case 6c is used here.

An example of the CAD model is found on pages 5 and 6. Illustrations of the corresponding finite element models of the rail follow on pages 21-23.

The finite element model boundary conditions consist of restraining the surfaces of the holes in the footing plate against translation in X, Y, and Z to simulate footing bolting to the foundation and restraining the nodes surrounding the support bracket-to-building-support bolt holes against translation in X and Y to simulate the attachment to the building support beams. The surfaces normal to the "Z" direction at the third rail end are each loaded with a 4909.85 lbs surface force to simulate the weight of the rails above it. The elevator cab loads are applied to the rail at the wheel locations corresponding to the cab lower wheels at 12 inches above ground level, at 161 inches above ground level, and at 155 inches above ground level. The latter two locations are chosen to place the wheel loads directly over the support brackets, a condition of maximum support bracket load.

TC6K Tower Elevator Rail Assembly – Lower Three Rails
Simulated Elevator Cab Loads, Lower Cab Wheels 12 Inches Above Ground Level
Dead Load, Live Load, Earthquake Load, and Snow Load Combination

TC6K Tower Elevator Rail Assembly – Lower Three Rails
Simulated Elevator Cab Loads, Lower Cab Wheels 161 Inches Above Ground Level
Dead Load, Live Load, Earthquake Load, and Snow Load Combination



TC6K Tower Elevator Rail Assembly – Lower Three Rails
Simulated Elevator Cab Loads, Lower Cab Wheels 155 Inches Above Ground Level
Dead Load, Live Load, Earthquake Load, and Snow Load Combination

Elevator Load on Building Supports
(Refer to Force Key Sheet on Preceding Page)

Elevator Lower Wheels Located 12 in. From Ground Level

Location	Fx, lbs.	Fy, lbs.
1	5.5	61.9
2	13.9	36.5
3	61.9	-590.3
4	-96.7	-597.6
5	302.9	-3363.4
6	-522.4	-3427.9
7	100.9	1177.4
8	-87.6	1130.4

Elevator Lower Wheels Located 161 in. From Ground Level

Location	Fx, lbs.	Fy, lbs.
1	-27.1	-987.5
2	-64.1	-1165.2
3	82.3	-1013.9
4	-181.1	-1014.7
5	-43.2	1400.6
6	-43.2	1178.5
7	-543.9	1189.6
8	469.7	1266.5

Elevator Lower Wheels Located 155 in. From Ground Level

Location	Fx, lbs.	Fy, lbs.
1	-1.3	-732.4
2	-38.9	-915.0
3	128.4	-1719.9
4	-295.5	-1724.0
5	161.6	507.7
6	-165.2	351.0
7	-712.9	2070.4
8	557.9	2098.1

Sign Convention of load is as shown in the coordinate axis on the Force Location Key Sheet on page 24.

Rail Stress and Deflection Analysis

Allowable Stresses are defines by AISC A5.1 which states that, ‘Except as provided in Chapter N (Plastic Analysis), all structural members, connections, and connectors shall be proportioned so the stresses due to the working loads do not exceed the allowable stresses specified in Chapters D through K. The allowable stresses specified in these chapters do not apply to peak stresses in regions of connections (see also Sect B9), provided requirements of Chapter K are satisfied. Since the rails are subjected to axial tension and bending stresses, AISC Section H2 is the applicable stress code. The specifics of this code are printed at the end of this analysis.

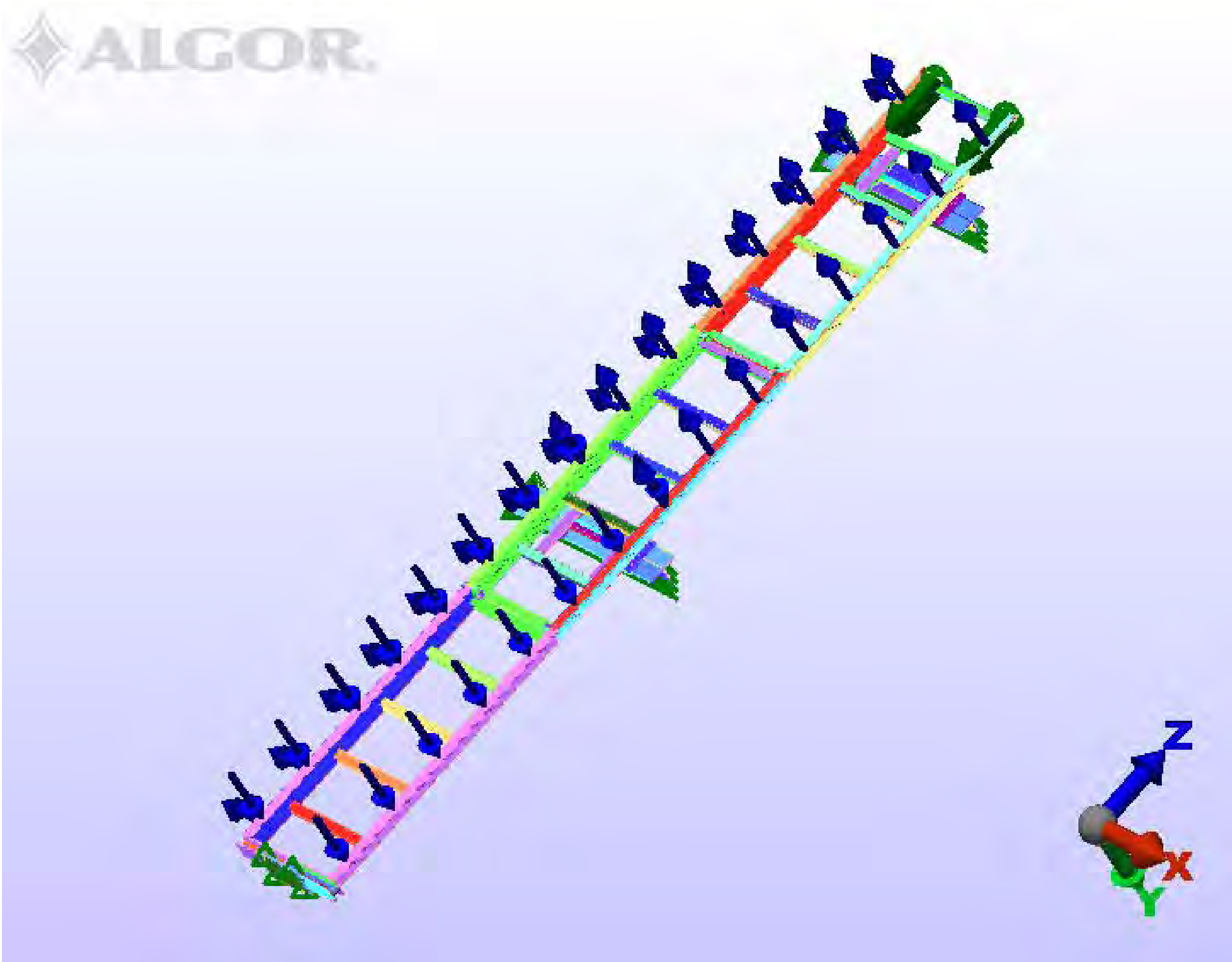
Eight elevator positions relative to the rail splice and building support locations are analyzed in order provide insight as to how the stress levels in various parts of the structure change with location of the cab. These locations put the cab wheels at:

Elevator ”Wheel” Location on Rail for Analysis

Load Case	Lower Wheels, Distance from Ground Level, in.	Upper Wheels, Distance from Ground Level, in.
1	12	179
2	36	203
3	60	227
4	84	251
5	108	275
6	132	299
7	156	323
8	180	347

The figure on page 27 illustrates the locations of all of these loads in one pictorial. In the analysis, for each load case, only four wheel loads normal to the plane of the rail are present, along with two wheel loads parallel to the plane at the same wheel longitudinal locations.(the figure on page 28 is typical) The wheel loads imposed upon the rail by the elevator cab are line forces distributed across the small leg of the “L” angle rails. This eight load case matrix is repeated three times, once each for load cases 6c (Max moment about X-axis), 5a (Max moment about Z-axis), and 7a (Max transverse lateral load) for a total of twenty-four analyses. The vertical load of 4909 lbs on each rail due to the 18 rails located above the three rails in the model is applied to each of the splice plates. The three rail models have a gravitational field impressed, simulating their own weight.

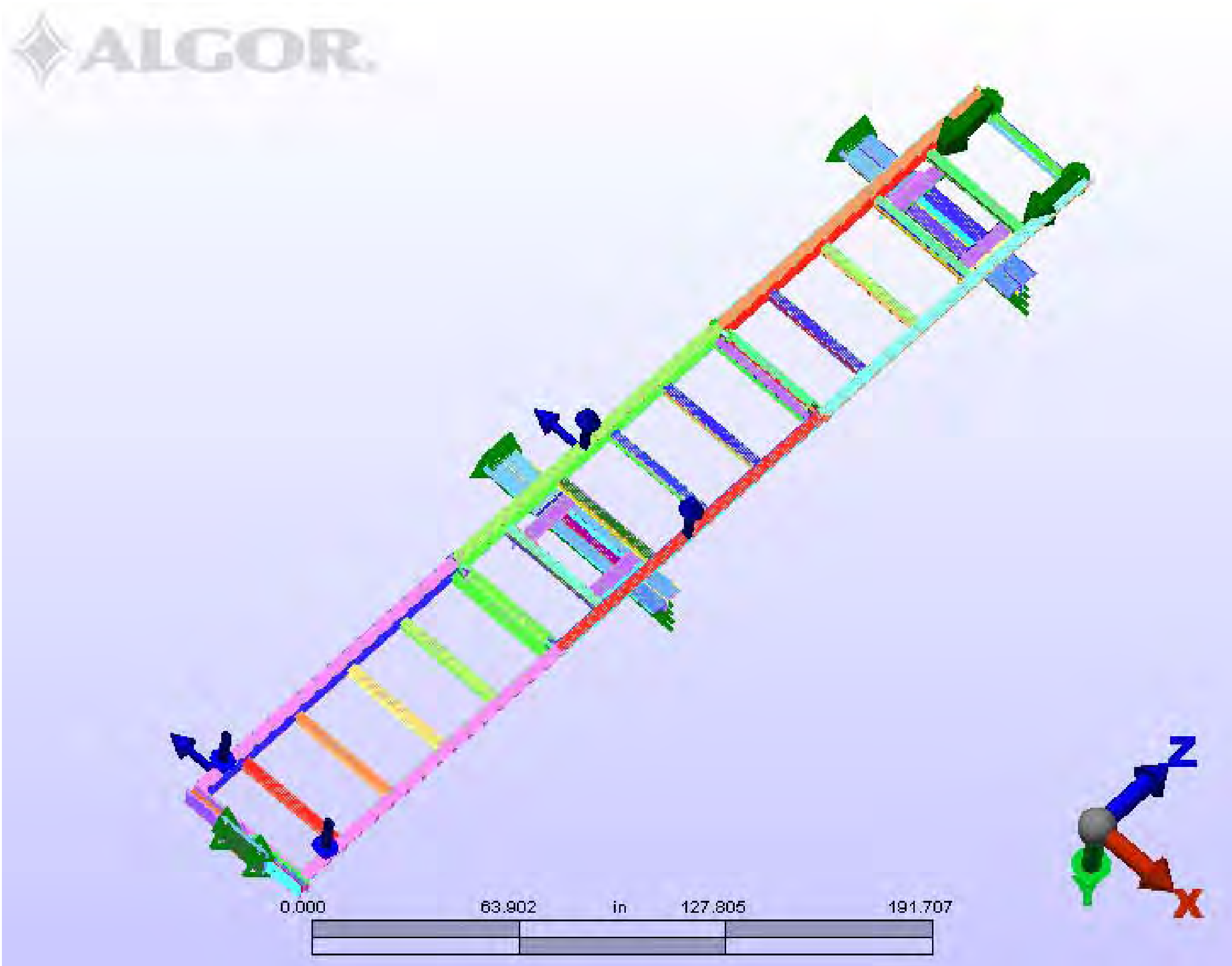
In the finite element model, the inside surface of the bolt holes in the rail footing is fixed to the foundation in translation The support brackets are fixed in translation at building side attachment point. In the rigid support case. the building support brackets have stiffening “springs” with 100,000 lb/in. flexibilities distributed along the thin horizontal surfaces; in the flexible case, these stiffening springs are absent. The two cases are necessary because the actual support stiffness is unknown. The uppermost rail splice plate holes have the hole surfaces restrained in “X” and “Y” translation simulating the bolted joint. The surfaces of adjacent rail sections which “fit” together within the channel of the alignment clips are in “surface” contact, there is a resistance to pressure but no tensile resistance across the surfaces in contact. The surfaces are free to slide without friction.



TC6K Tower Elevator Rail

Multiple Load Locations Superimposed on One Slide

Finite Element Model



TC6K Tower Elevator Rail

Typical Single Load Case Load Application

Finite Element Model – Rigid Building Support

Maximum Stress and Deflection in Rail Assembly
For Different Cab Locations

Load Case – Maximum Moment about X-Axis (Case 6c)	Maximum Von Mises Stress in Rail Assembly, psi	Maximum Deflection in Rail Assembly, in. (magnitude)
1	16496	0.075
2	19916	0.163
3	23244	0.239
4	27695	0.261
5	29629	0.203
6	25558	0.100
7	26191	0.074
8	18219	0.061
Load Case – Maximum Moment about Z-Axis (Case 5a)	Maximum Von Mises Stress in Rail Assembly, psi	Maximum Deflection in Rail Assembly, in. (magnitude)
1	15813	0.085
2	26496	0.281
3	29258	0.431
4	34445	0.405
5	37457	0.303
6	26679	0.149
7	24730	0.070
8	23313	0.100
Load Case – Maximum Transverse Force (Case 7a)	Maximum Von Mises Stress in Rail Assembly, psi	Maximum Deflection in Rail Assembly, in. (magnitude)
1	13938	0.076
2	26566	0.258
3	28529	0.392
4	33737	0.373
5	36388	0.278
6	25410	0.138
7	19597	0.055
8	20636	0.094

Results:

Section H2 of the AISC Manual of Steel Construction states that, “Members subject to both axial tension and bending stresses shall be proportioned at all points along their length to satisfy the following equation:

$$f_a/F_t + f_{bx}/F_{bx} + f_{by}/F_{by} \leq 1.0$$

where f_b is the computed bending tensile stress, f_a is the computed axial tensile stress, F_b is the allowable bending stress, and F_t is the governing allowable tensile stress defined in Sect D1.

Section D1 defines the governing allowable tensile stress as 0.60Fy on the gross area and 0.50Fu on the effective net area. Fy is the material yield stress at room temperature which is 50,000 psi and Fu is the material ultimate stress at room temperature which is 70,000 psi minimum.

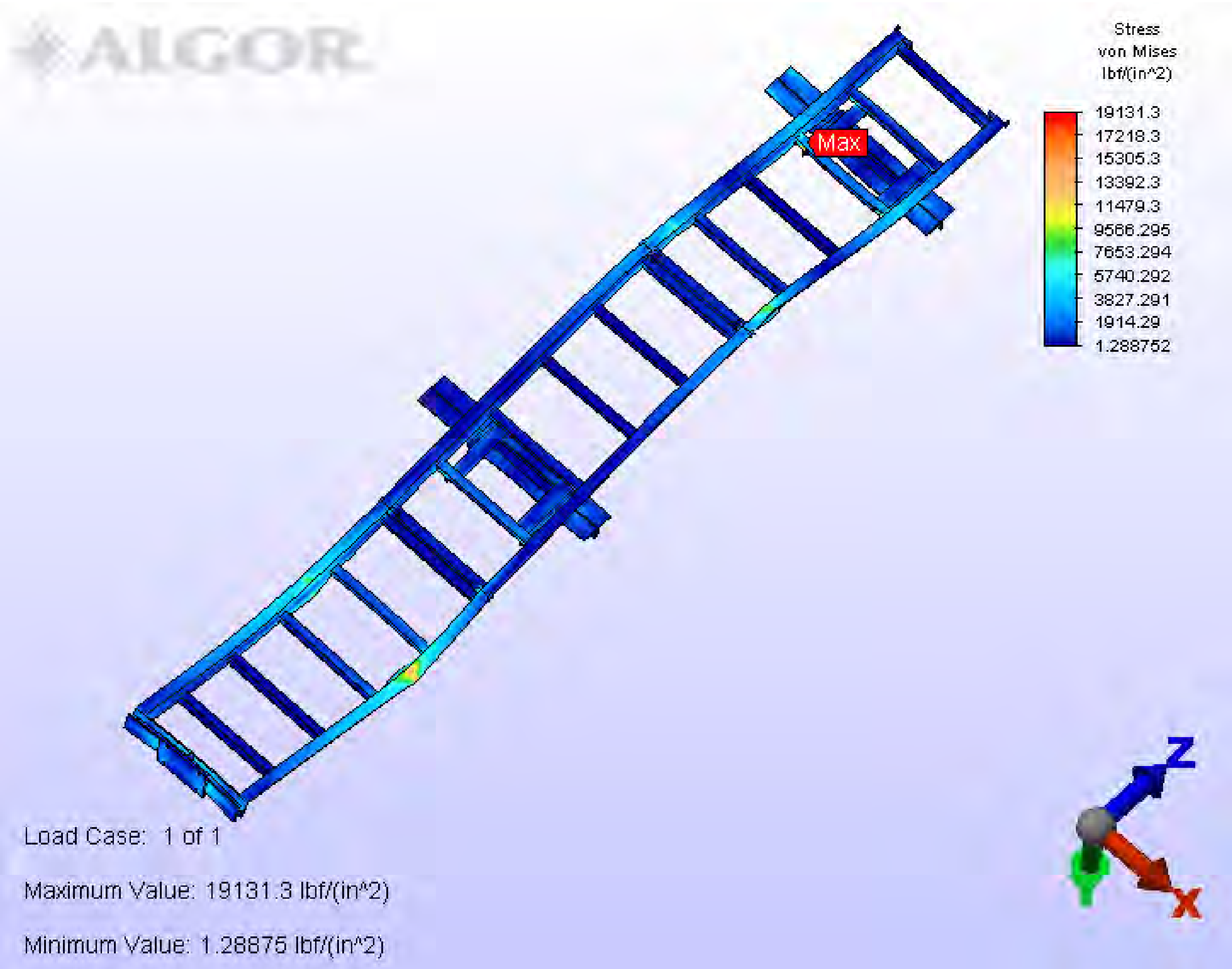
In our analysis we calculate the Von Mises equivalent tensile stress which combines the tensile, direct, and shear stresses into an equivalent tensile stress which is compared to the governing allowable tensile stress, 30,000 psi.

The maximum calculated Von Mises stress in the rails when subjected to the 24 point matrix of loading conditions is under 30.ksi. The four points where the stress exceeds 30 ksi are all peak stresses in regions of connections in which AISC Section A5.1 permits this allowable to be exceeded. The nominal stress at the region of connections is below the target limit of 30 ksi.

There are no deflection limits imposed by the code on this type of structure. The deflections are presented for information purposes only. Theses tabulated deflections are vector magnitude deflections, the total vector sum deflections. They include the deflection of the building supports which are minimal, as the rigid boundary case is used in this analysis.

It is to be noted that the loading used in this analysis is an extreme case designated by code which most probably will never be experienced during the elevator’s lifespan. By contrast, the maximum Von Mises stress and displacement for a normally loaded structure (dead load plus deceleration to a stop) is 20316 psi. and 0.165 in.

Rather than reproduce pictorial results for displacement and stress patterns for each of the 24 load cases, in the interest of brevity, a typical results case for the rail stress, the rail displacements, and the rail peak stress are shown on pages 31-33 to complement the above table.

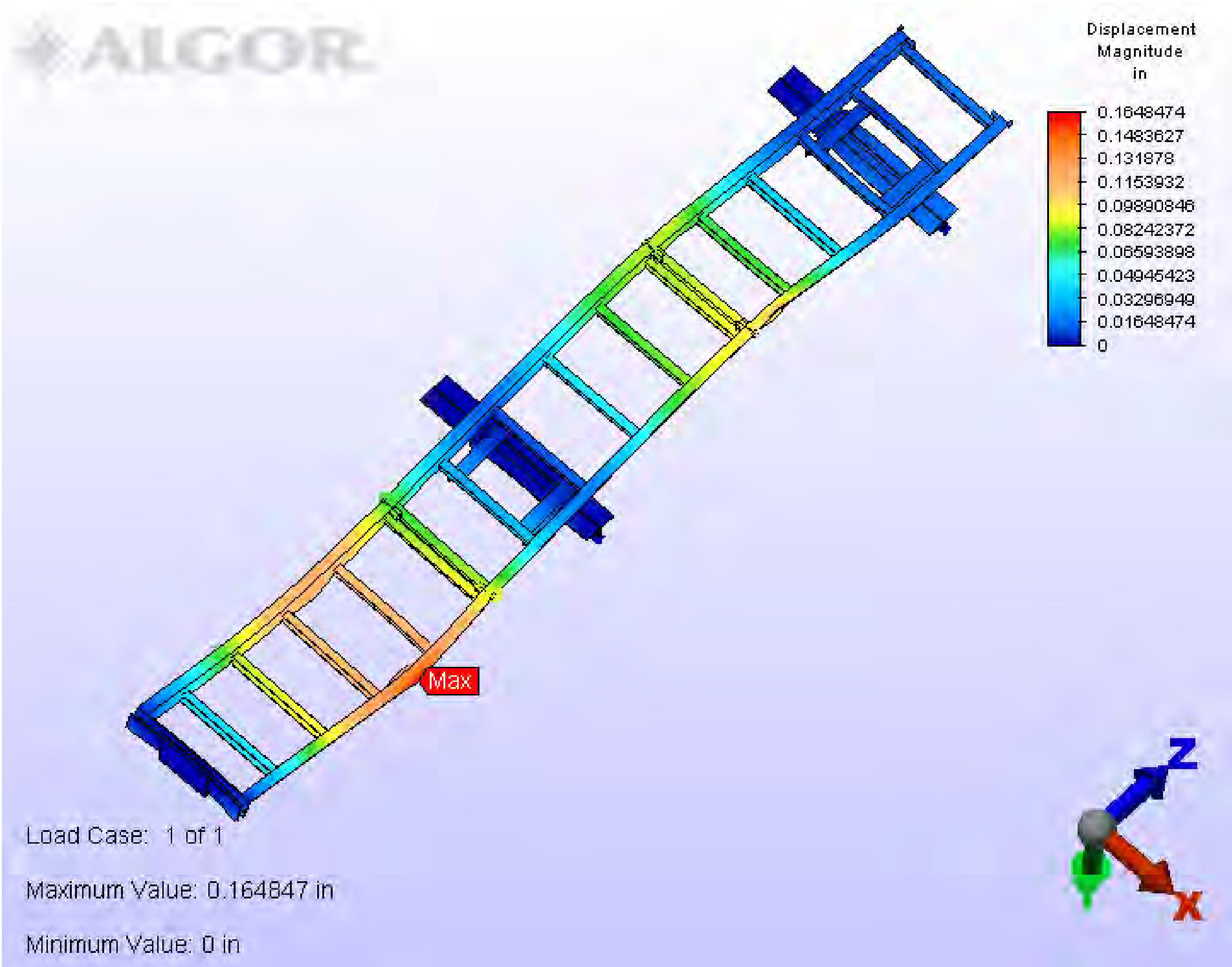


TC6K Tower Elevator Rail Analysis

Finite Element Model – Rigid Building Supports

Von Mises Stress

Typical Pattern of Results

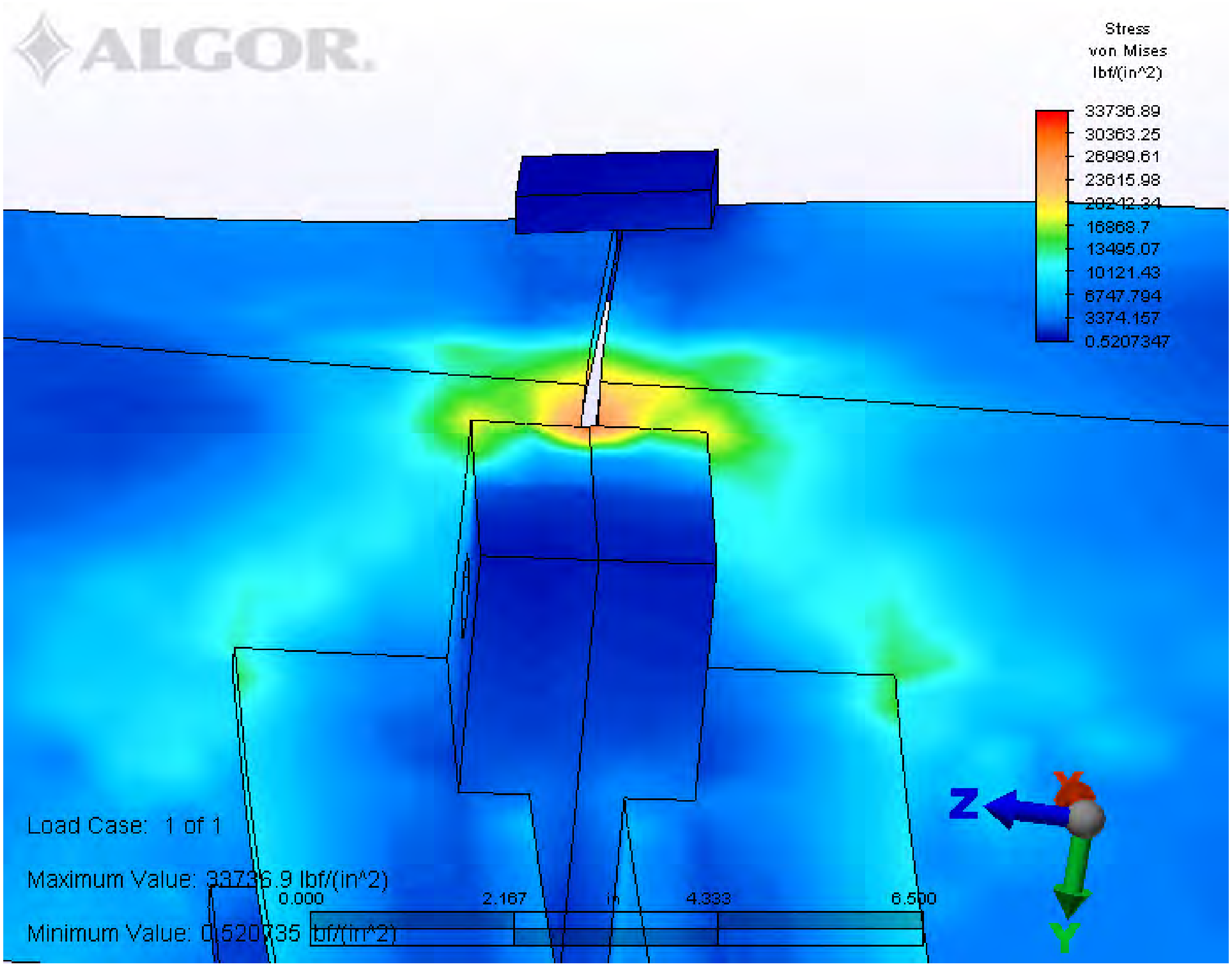


TC6K Tower Elevator Rail Analysis

Finite Element Model – Rigid Building Supports

Displacements

Typical Pattern of Results



TC6K Tower Elevator Rail Analysis

Finite Element Model – Rigid Building Supports

Von Mises Stress

Typical Results at Splice Junction
Showing Localized Peak Stress Pattern

Special 14' – 7" Building Support Span

Late in the design phase of the elevator rail, the customer made a request to extend one span between the building supports from 13' – 5" to 14' – 7". This required the creation and analysis of a special model of the rail consisting of five 10 ft. spans of the rail assembly. This model was created and analyzed in the same manner as the normal rail assembly, described above. The same loadings are utilized as in the normal model, except that the position of the loads due to the cab wheels is changed to apply the wheel loads to the rail over the longer rail span. Page 37 provides a graphic view of the model.

Elevator "Wheel" Location on Rail for Analysis – Special 14' – 7" Support Span

Load Case	Lower Wheels, Distance from Ground Level, in.	Upper Wheels, Distance from Ground Level, in.
1	132	299
2	156	323
3	180	347
4	204	371
5	228	395
6	252	419
7	276	443
8	300	467

In the analysis, for each load case, only four wheel loads normal to the plane of the rail are present, along with two wheel loads parallel to the plane at the same wheel longitudinal locations. The wheel loads imposed upon the rail by the elevator cab are line forces distributed across the small leg of the "L" angle rails. This eight load case matrix is repeated three times, once each for load cases 6c (Max moment about X-axis), 5a (Max moment about Z-axis), and 7a (Max transverse lateral load) for a total of twenty-four analyses. The vertical load of 4406 lbs on each rail due to the 16 rails located above the five rails in the model is applied to each of the splice plates. The five rail models have a gravitational field impressed, simulating their own weight.

In the finite element model, the inside surface of the bolt holes in the rail footing is fixed to the foundation in translation The support brackets are fixed in translation at building side attachment point. In the rigid support case. the building support brackets have stiffening "springs" with 100,000 lb/in. flexibilities distributed along the thin horizontal surfaces; in the flexible case, these stiffening springs are absent. The two cases are necessary because the actual support stiffness is unknown. The uppermost rail splice plate holes have the hole surfaces restrained in "X" and "Y" translation simulating the bolted joint. In order to reduce the analysis time for this model, the surfaces of adjacent rail sections which "fit" together within the channel of the alignment clips are in "bonded" contact, that is there is a resistance to pressure and tensile resistance across the surfaces in contact. The surfaces are not free to slide. This is a slightly different boundary condition that used in the original rail analysis which used "surface" contact at this location. It does not compromise the accuracy of the results.

Maximum Stress and Deflection in Rail Assembly
For Different Cab Locations

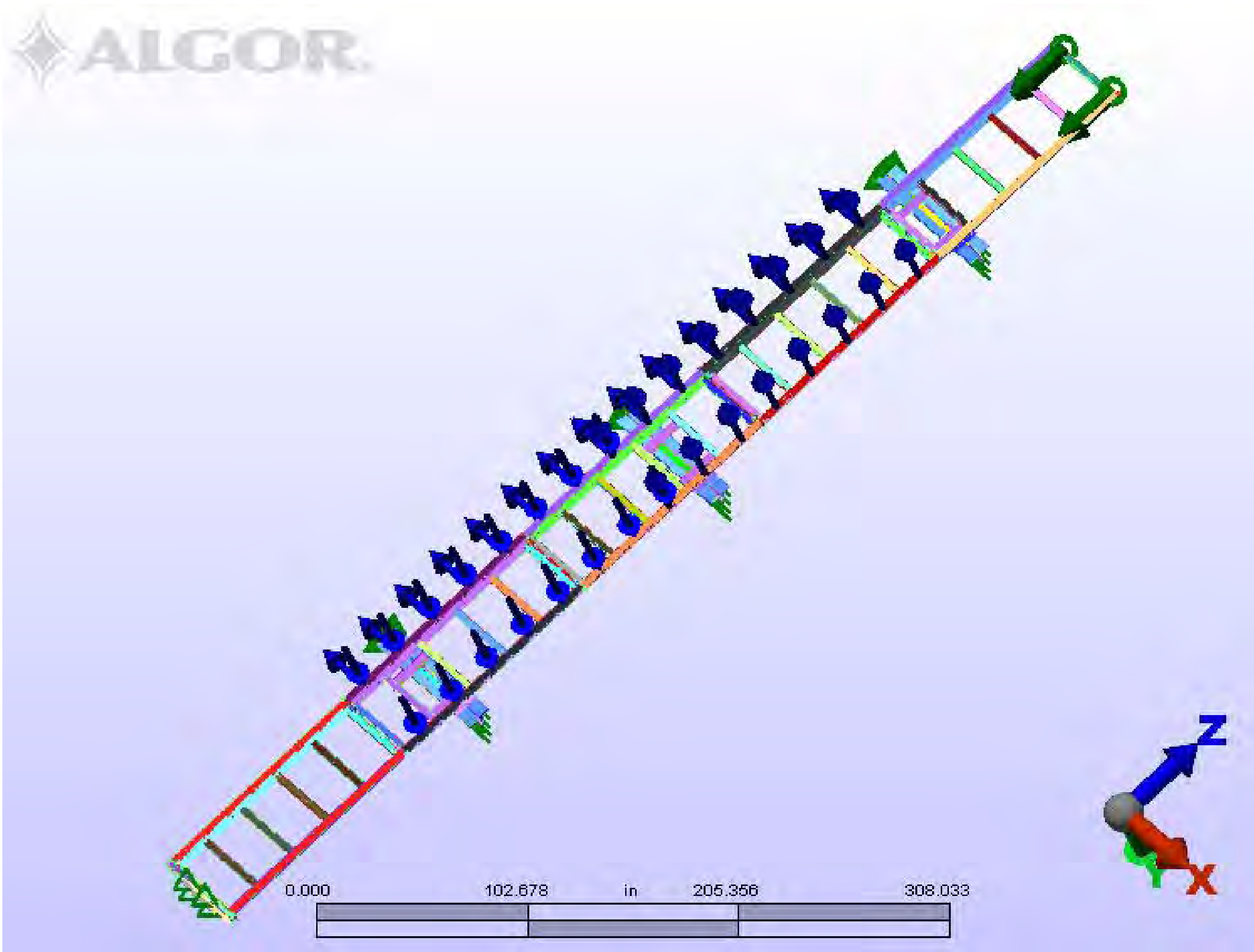
Rail with Special 14' – 7" Support Span

Load Case – Maximum Moment about X-Axis (Case 6c)	Maximum Von Mises Stress in Rail Assembly, psi	Maximum Deflection in Rail Assembly, in. (magnitude)
1	17909*	.060*
2	13435	.039
3	24752	.054
4	35126**	.125
5	58305**	.200
6	59592**	.198
7	30586**	.141
8	17888	.070
Load Case – Maximum Moment about Z-Axis (Case 5a)	Maximum Von Mises Stress in Rail Assembly, psi	Maximum Deflection in Rail Assembly, in. (magnitude)
1	20003*	.121*
2	10100*	.040*
3	26815	.094*
4	41305**	.241*
5	86019**	.346*
6	94324**	.355*
7	49861**	.244*
8	19011*	.091
Load Case – Maximum Transverse Force (Case 7a)	Maximum Von Mises Stress in Rail Assembly, psi	Maximum Deflection in Rail Assembly, in. (magnitude)
1	17286*	.112*
2	8630*	.034*
3	23776	.087
4	35400**	.226
5	75563**	.325
6	83677**	.331*
7	44919**	.228*
8	18456*	.085*

* indicates that the value given is located at a position outside the 14' – 7" span
** indicates that the value given is located at a position outside the 14' – 7" span and is a peak (stress concentration driven) value.

The above table provides a summary of results of calculations performed using the five segment rail with one span extending 14’ – 7”. Note that the results here cannot be compared one-to-one with those of the normal span analysis because the load (i.e. “wheel) positions are at different relative positions along the rail and the original model did not have weld fillets at the splice plates, the location of most of the “peak” stresses. The stress summary presented here applies only to the upper three rail assemblies of the five segment rail under analysis.

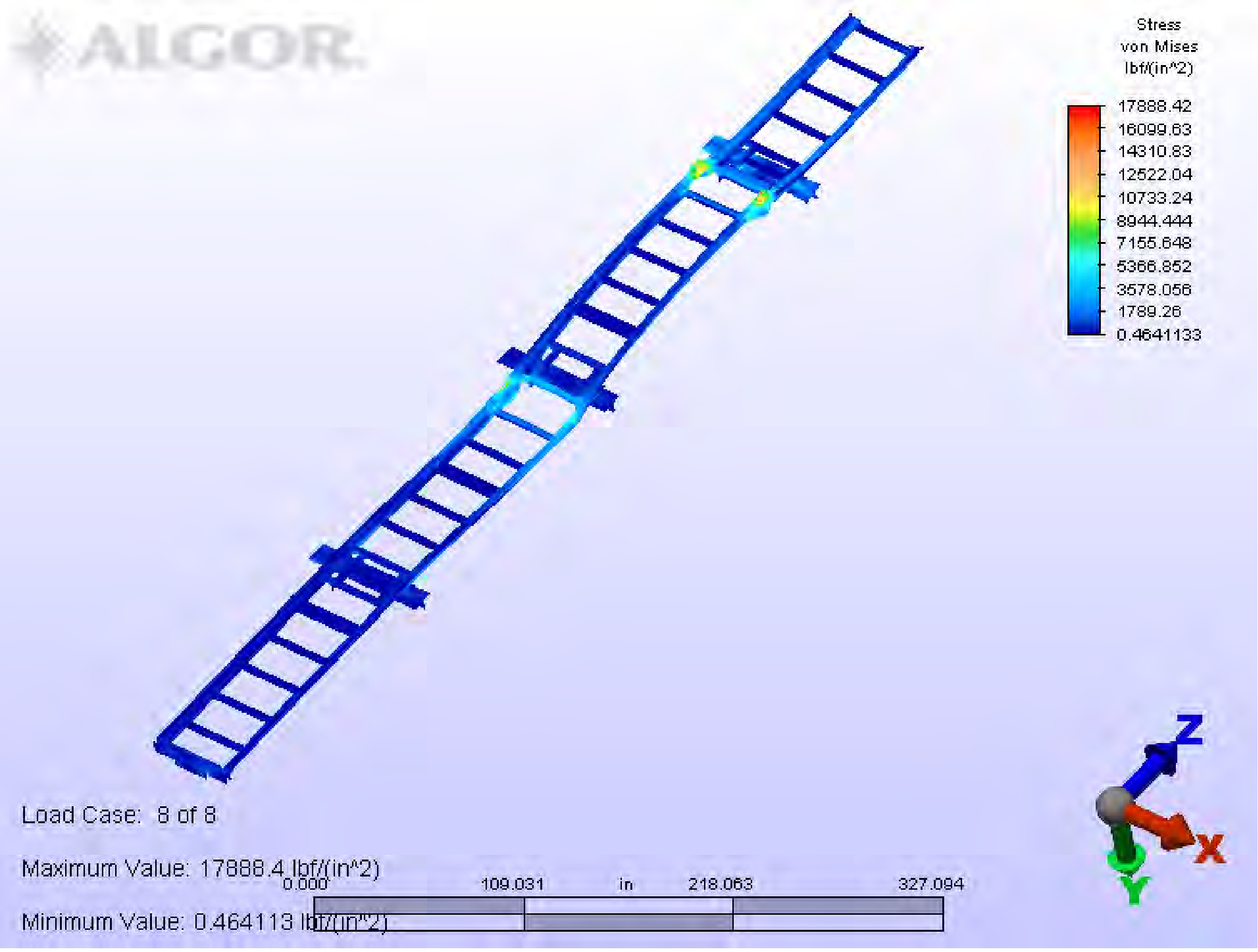
A discussion of the design allowables and their derivation is given in the discussion of the original rail analysis. The extended span meets the allowable stress requirements of the code when the exemption for peak stresses is at connection points is considered. There are no Von Mises stresses over the allowable 30,000 psi stress anywhere in the rail structure for any of the 24 load points analyzed other than the points of peak stress concentration.



TC6K Tower Elevator Rail
"Special" 14' – 7" Span

Multiple Load Locations Superimposed on One Slide

Finite Element Model

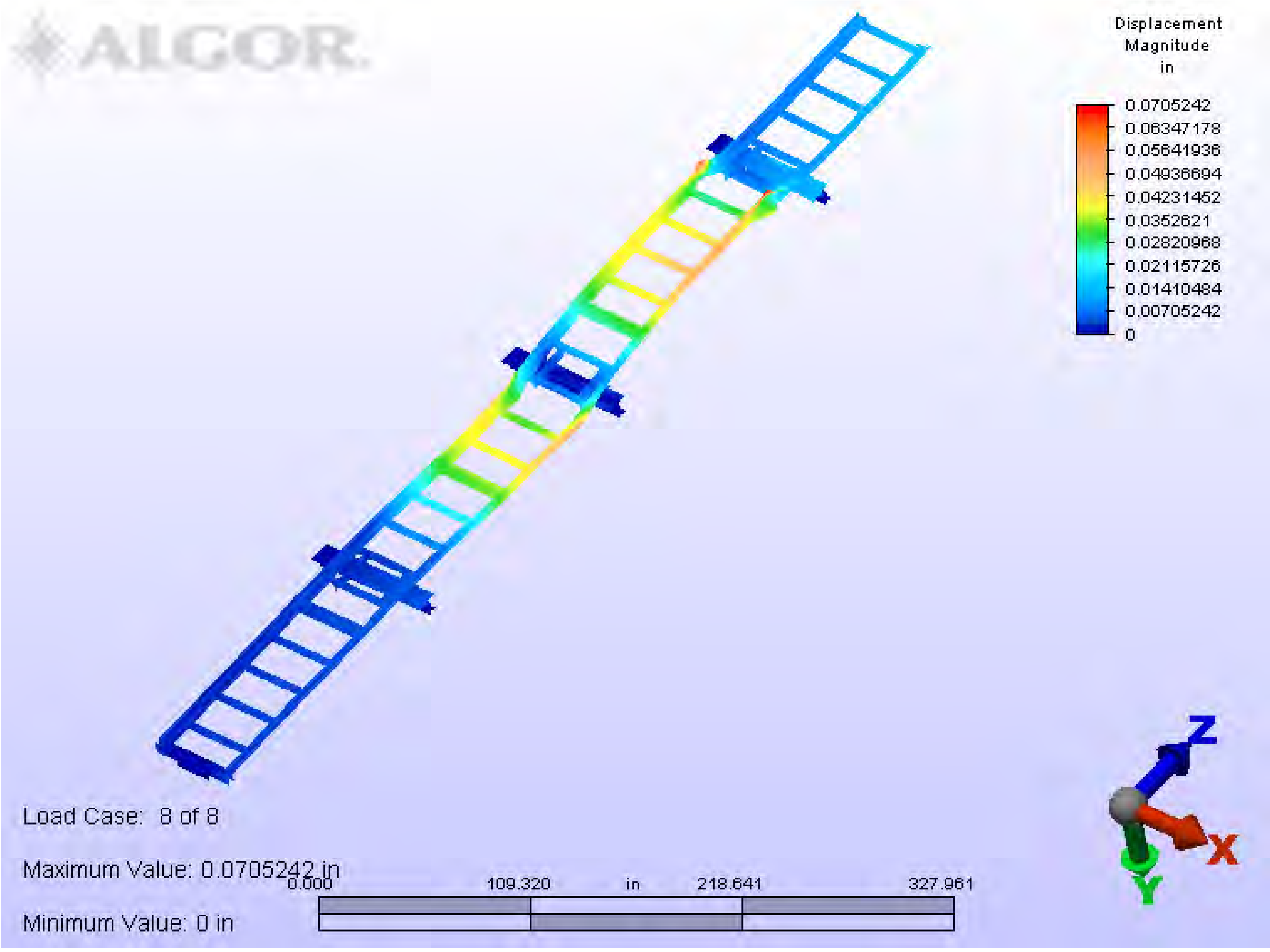


TC6K Tower Elevator Rail Analysis
“Special” 14’ – 7” Span

Finite Element Model – Rigid Building Supports

Von Mises Stress

Typical Pattern of Results



TC6K Tower Elevator Rail Analysis
“Special” 14’ – 7” Span

Finite Element Model – Rigid Building Supports

Displacements

Typical Pattern of Results

Welds at Splice Plate - Rail – Horizontal Brace Junction

Requirements for structural fillet welds, consisting of minimum/maximum weld size and maximum stress level are defined in the AISC Manual of Steel Construction Allowable Stress Design Ninth Edition, Section J2.

The minimum size of fillet welds are given in a table, J2.4, a portion of which is reproduced here for reference. Minimum weld size is dependent upon the thicker of the two parts joined, except that the weld size need not exceed the thickness of the thinner part. (preheat required for this exception). The maximum size of fillet welds that is permitted along the edges of connected parts is (1) for material less than ¼ inch thick, not greater than the thickness of the material; for material ¼ inch or more in thickness, not greater than the thickness of the material minus 1/16 inch, unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness.

The allowable stress on welds is given in Table J2.5 of the same reference manual, a portion of which is also reproduced here.

Table J2.4
Minimum Size of Fillet Welds

Material Thickness of Thicker Part Joined, (in.)	Minimum Fillet Weld Size, (in.)
To ¼ inclusive	1/8
Over ¼ to ½	3/16
Over ½ to ¾	¼
Over ¾	5/16
	These are leg dimensions of single pass fillet welds

Table J2.5
Allowable Stress on Welds

Shear on Effective Area	0.3 X Nominal Tensile Strength of the Weld Material	Weld metal with a strength level equal to or less than “matching” weld metal is permitted
Tension or Compression Parallel to the Axis of the Weld	Same as Base Metal	Same

The fillet weld which joins the splice plate to the longitudinal rails and horizontal braces in the rail assemblies is of three sizes, ¼ in., 1/8 in., and 1/16 in. The ¼ inch fillet is used at the splice plate – rail joint and the horizontal brace – rail joint. The 1/8 inch fillet is used at the horizontal brace – splice plate joint. The 1/16 inch weld, a non-structural weld, is used at the splice plate – rail end face for sealing purposes. The weld is analyzed using the same finite element models as were used to analyze the rail assembly except that the weld fillets are now included in the model and the welded parts are very nearly supported only by the weld. The same boundary conditions and the same 24 point matrix of loads is utilized. The splice plates are assumed “bonded” at the inter-rail junctions and the alignment clips simulate “surface” contact, that is, a pressure between the surfaces is resisted, a tension is not and the surfaces are free to slide without friction.

Load Case – Maximum Moment about X-Axis (Case 6c)	Peak Von Mises Stress in Splice Plate Weld, psi	
1	19311.2	
2	24072.5	
3	42788.6	
4	52411.7	
5	48245.8	
6	41249.3	
7	<17000.0	
8	<19500.0	
Load Case – Maximum Moment about Z-Axis (Case 5a)	Peak Von Mises Stress in Splice Plate Weld, psi	
1	<29500.0	
2	31894.7	
3	45631.4	
4	39265.3	
5	44395.7	
6	32325.3	
7	<18145.5	
8	<23500.	
Load Case – Maximum Transverse Force (Case 7a)	Peak Von Mises Stress in Splice Plate Weld, psi	
1	<26500.	
2	30055.8	
3	44216.4	
4	36712	
5	43051.3	
6	28327.4	
7	<16733.3	
8	20560	

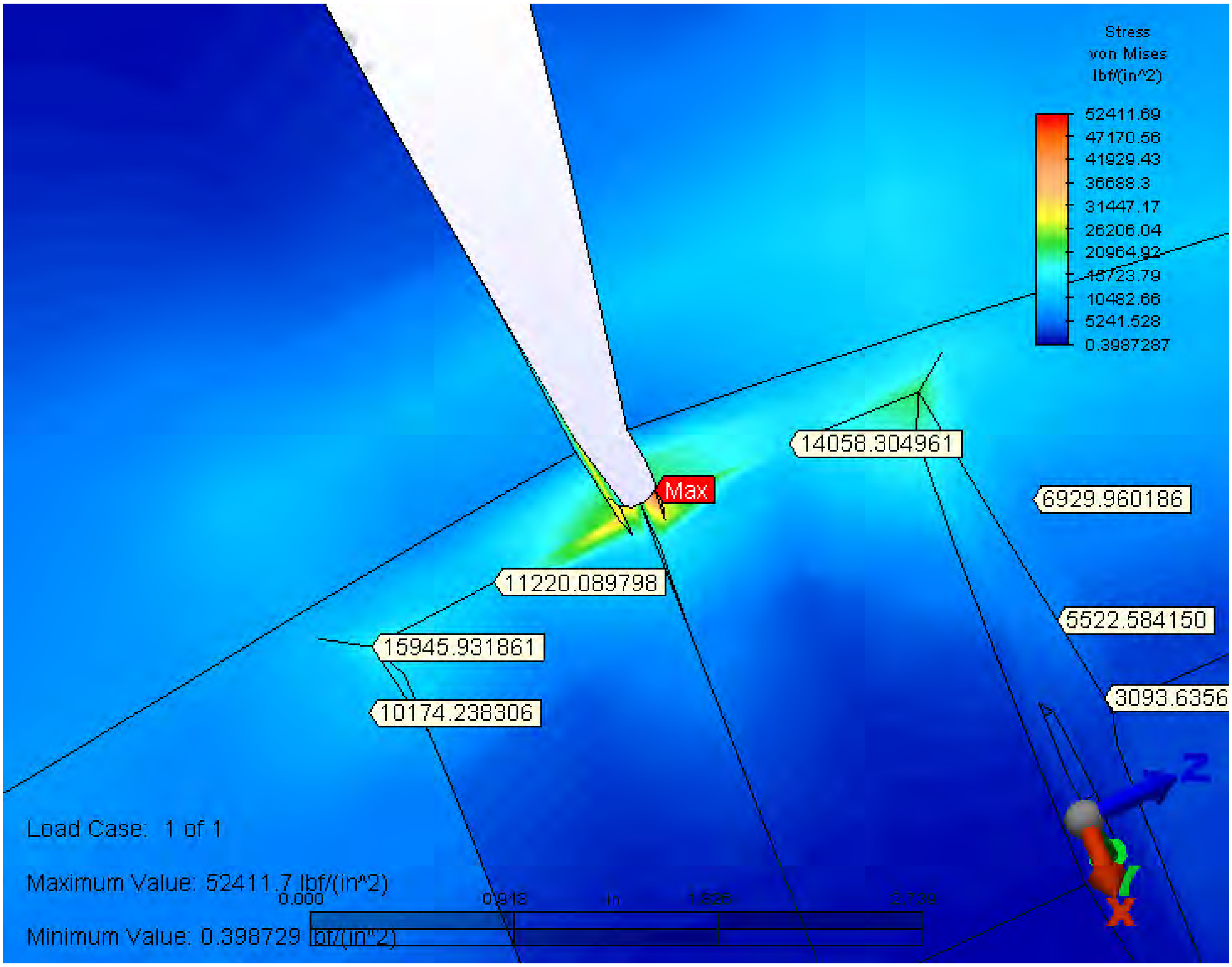
The items following the “<” sign indicate that the maximum Von Mises stress for the rail assembly is not located in the splice plate weld, but rather elsewhere in the structure.

The values of Von Mises stress, or equivalent tensile stress listed in the table are all peak values, localized generally to the corners of the weld. It would be very lengthy to document the stress pattern for each load case at each weld location individually; the approach here is to take the weld at the maximum Von Mises stress location and illustrate, including nominal values of shear stress to demonstrate compliance of the weld with the code stress allowables.

Results:

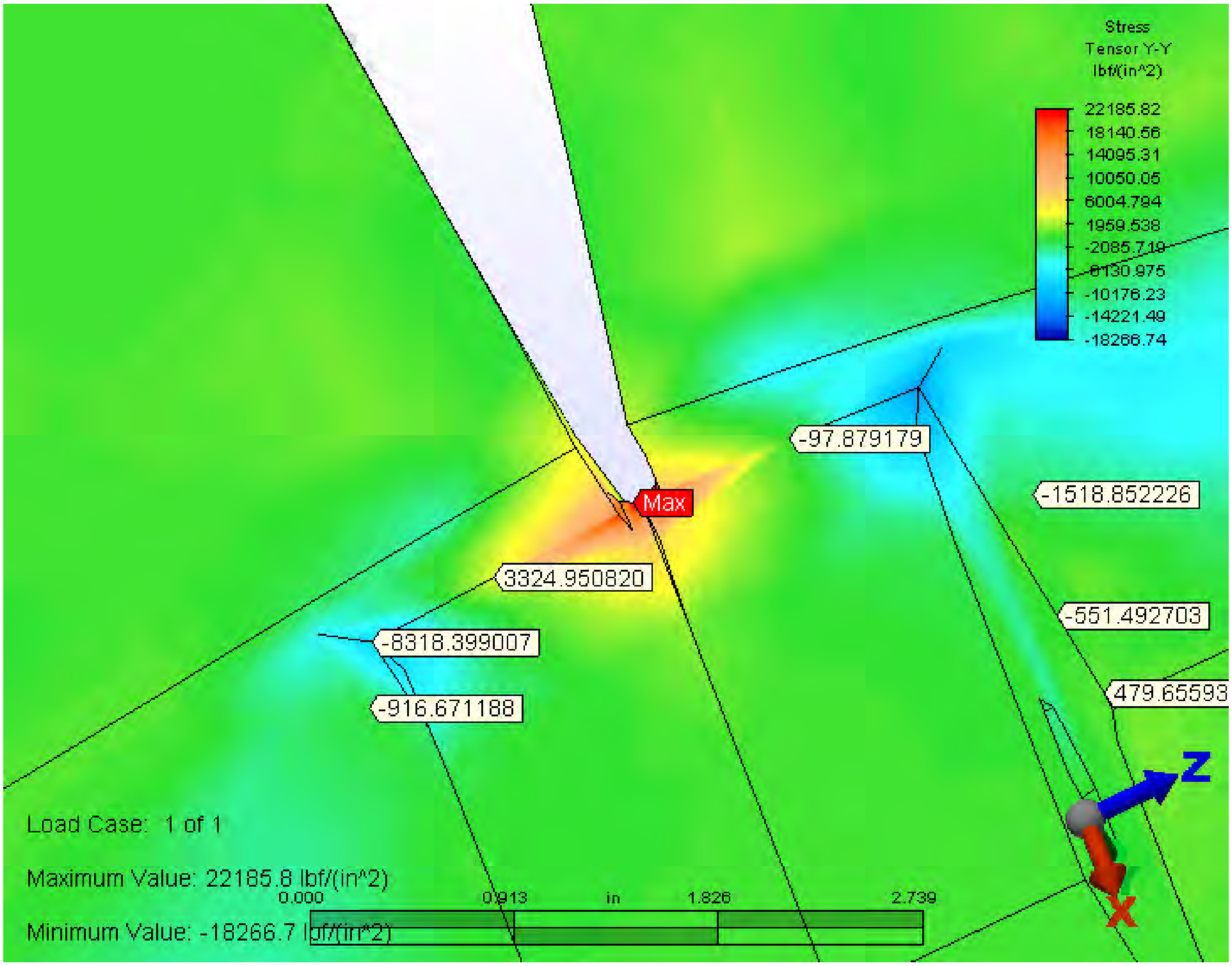
AISC Manual of Steel Construction Allowable Stress Design Ninth Edition, Section J2, Table J2.5 defines the allowable weld stresses. Quantitatively, the allowable stresses are 0.3 X 70,000 psi. = 21,000 psi. in shear and 0.6 X 50,000 psi = 30,000 psi in tension or compression parallel to the axis of the weld. Section A5.1 of the same manual states that these allowable stresses do not apply to peak stresses in regions of connections. It is seen from the figures on pages 43-45 that these design allowable conditions are met in the rail structure.

In comparison, the values of highest Von Mises weld stress and associated shear stress in the splice plate weld for normal operating conditions of maximum dead weight and emergency braking are shown in the figures on pages 46-48. It is seen from these illustrations that the weld stress in the splice plate has a large margin of safety during normal operation.



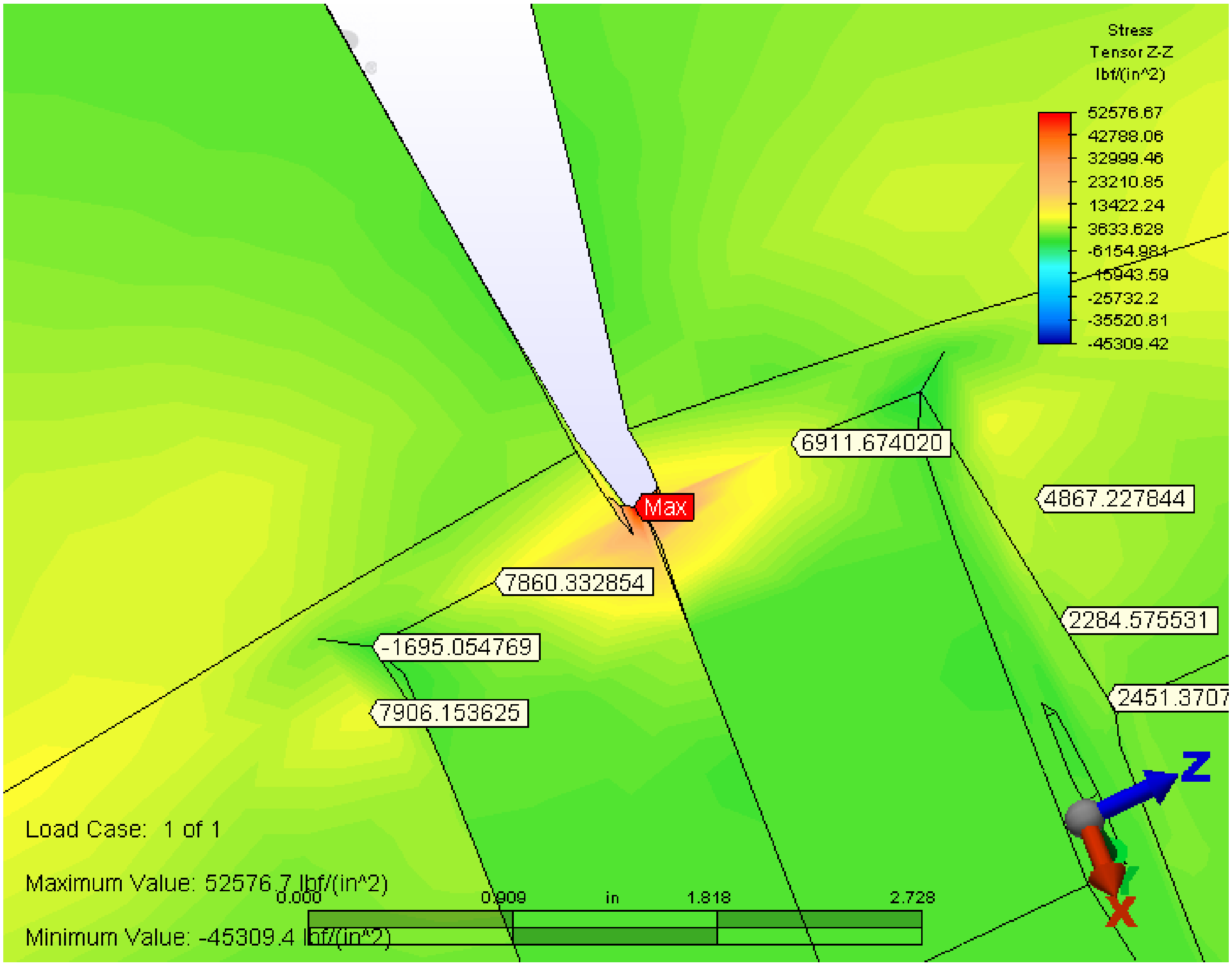
TC6K Tower Elevator Rail
Load Case With Maximum VonMises Stress
in Splice Plate Weld

Finite Element Model Showing Pattern of Stress
At Various Locations in the Weld



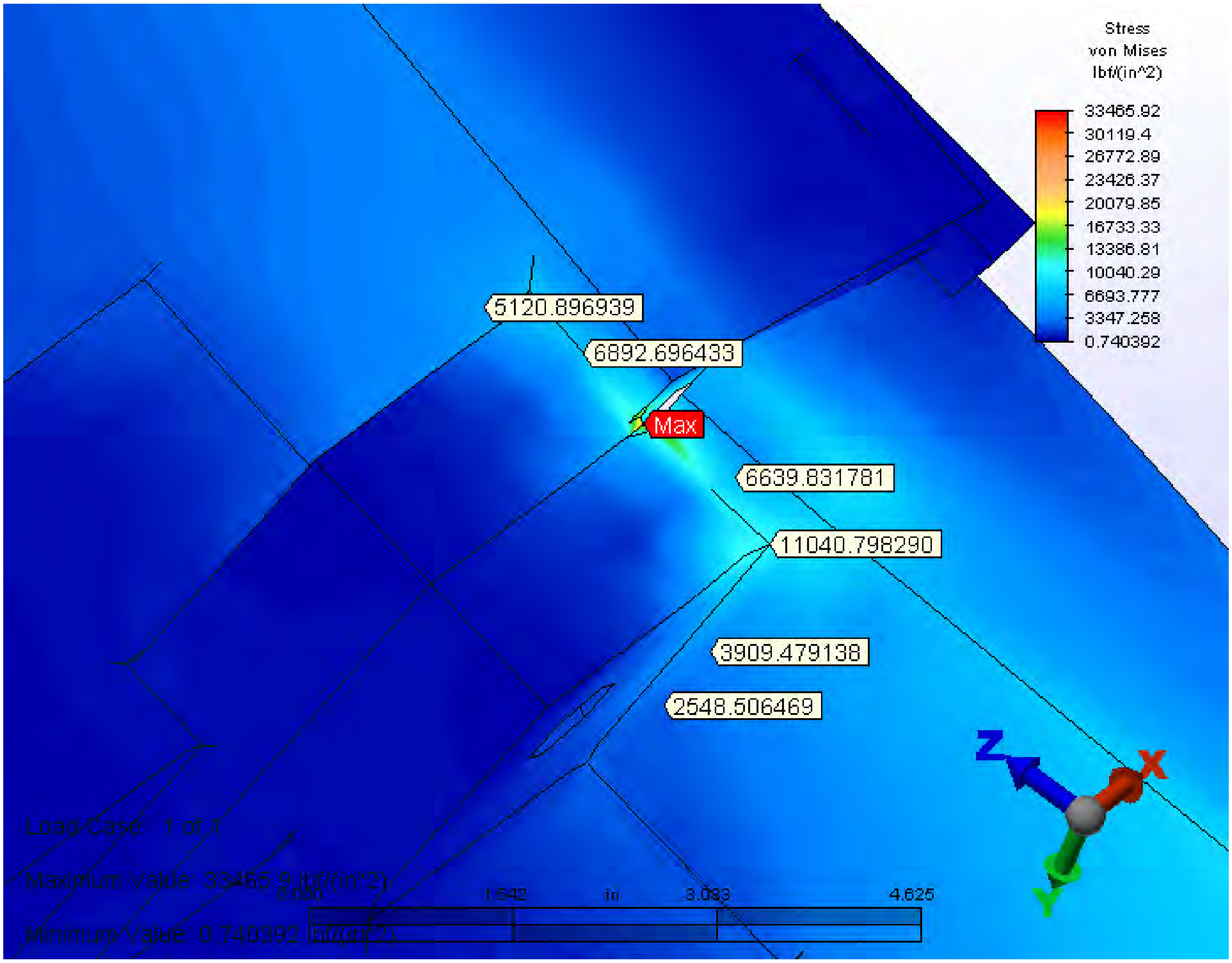
TC6K Tower Elevator Rail
Load Case With Maximum Y - Shear Stress
in Splice Plate Weld

Finite Element Model Showing Pattern of Stress
At Various Locations in the Weld



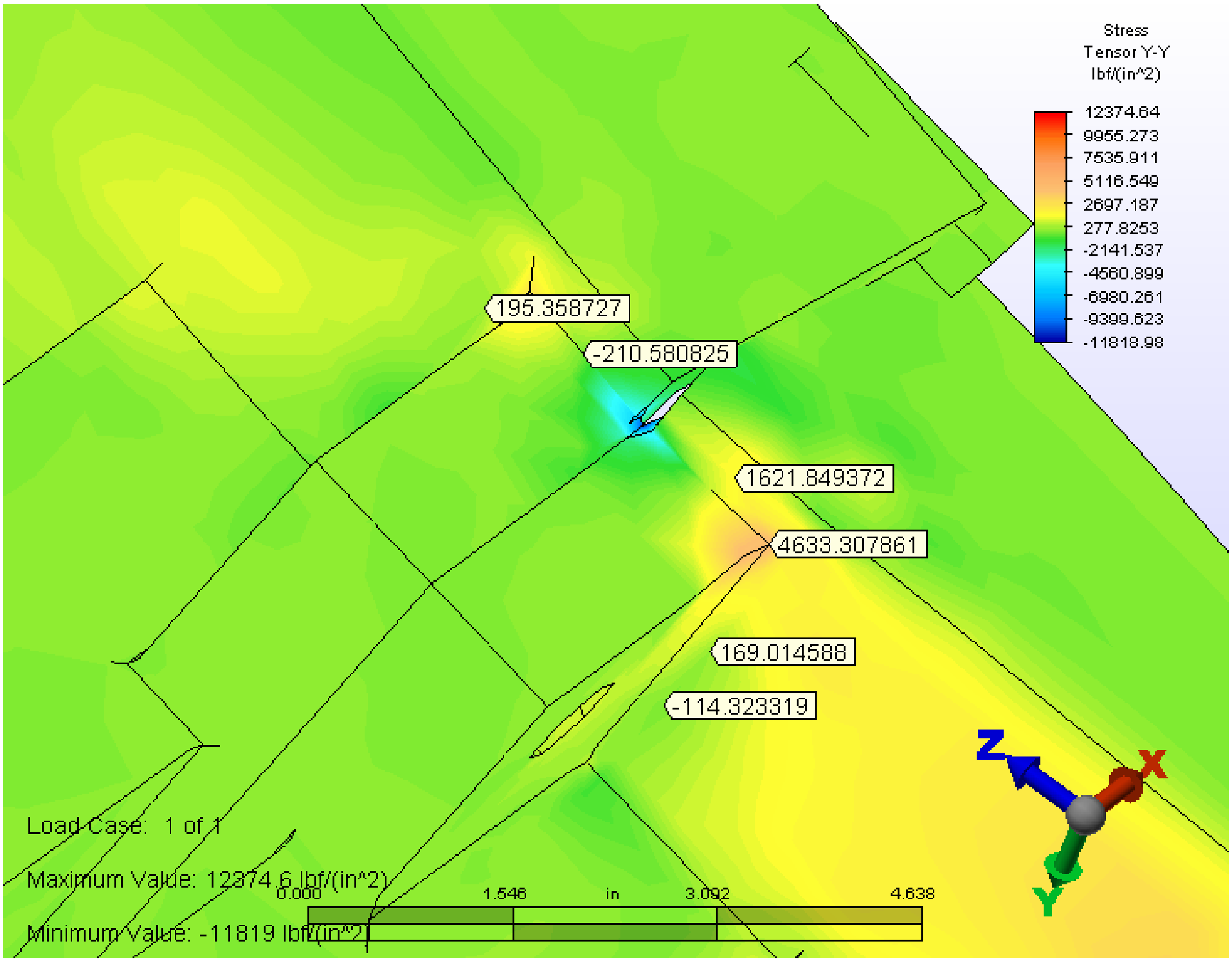
TC6K Tower Elevator Rail
Load Case With Maximum Z - Shear Stress
in Splice Plate Weld

Finite Element Model Showing Pattern of Stress
At Various Locations in the Weld



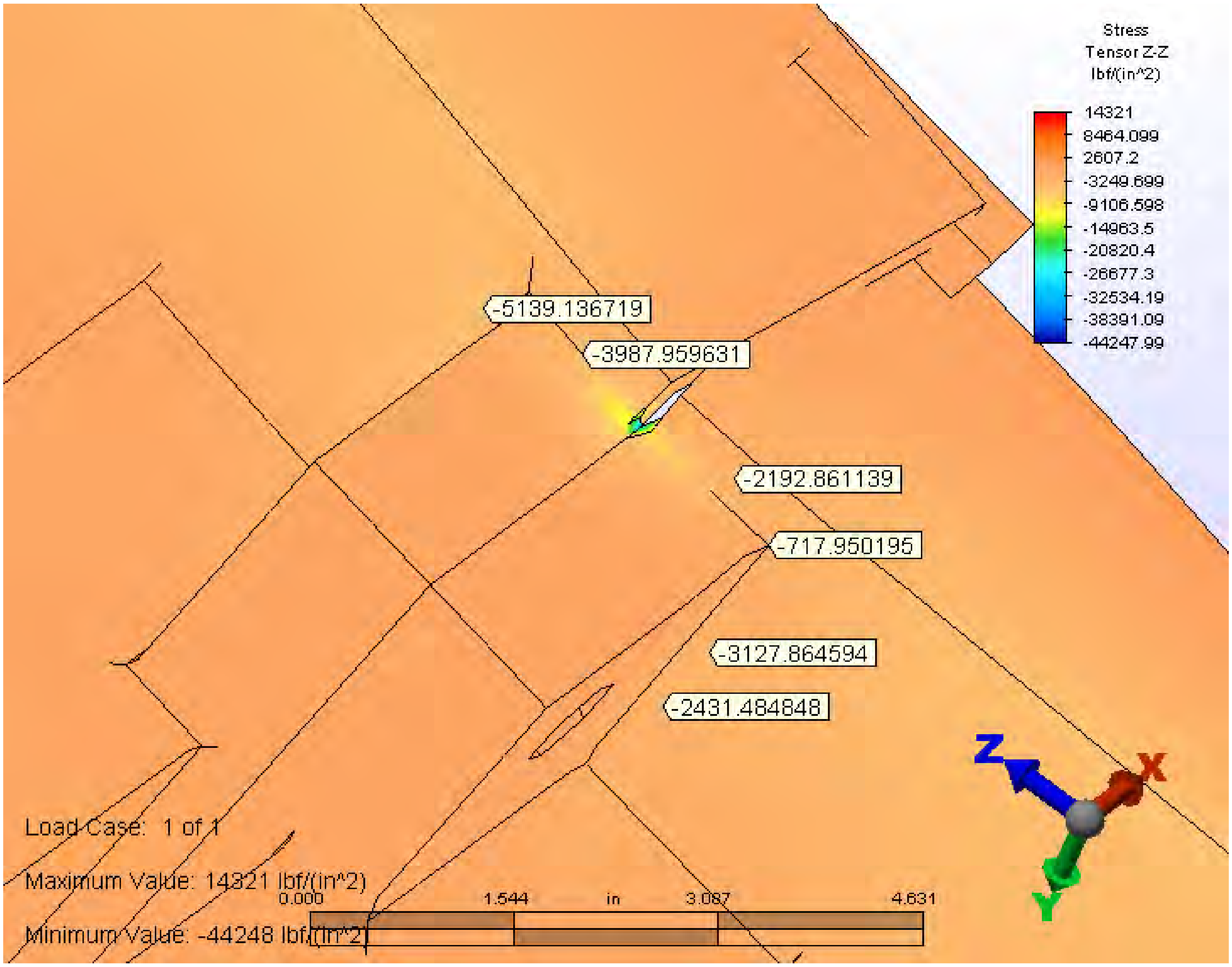
TC6K Tower Elevator Rail
Load Case With Maximum VonMises Stress
in Splice Plate Weld Due to Normal Elevator Operation
Maximum Dead Load Plus Braking

Finite Element Model Showing Pattern of Stress
At Various Locations in the Weld



TC6K Tower Elevator Rail
Load Case With Maximum Y - Shear Stress
in Splice Plate Weld Due to Normal Elevator Operation
Maximum Dead Load Plus Braking

Finite Element Model Showing Pattern of Stress
At Various Locations in the Weld



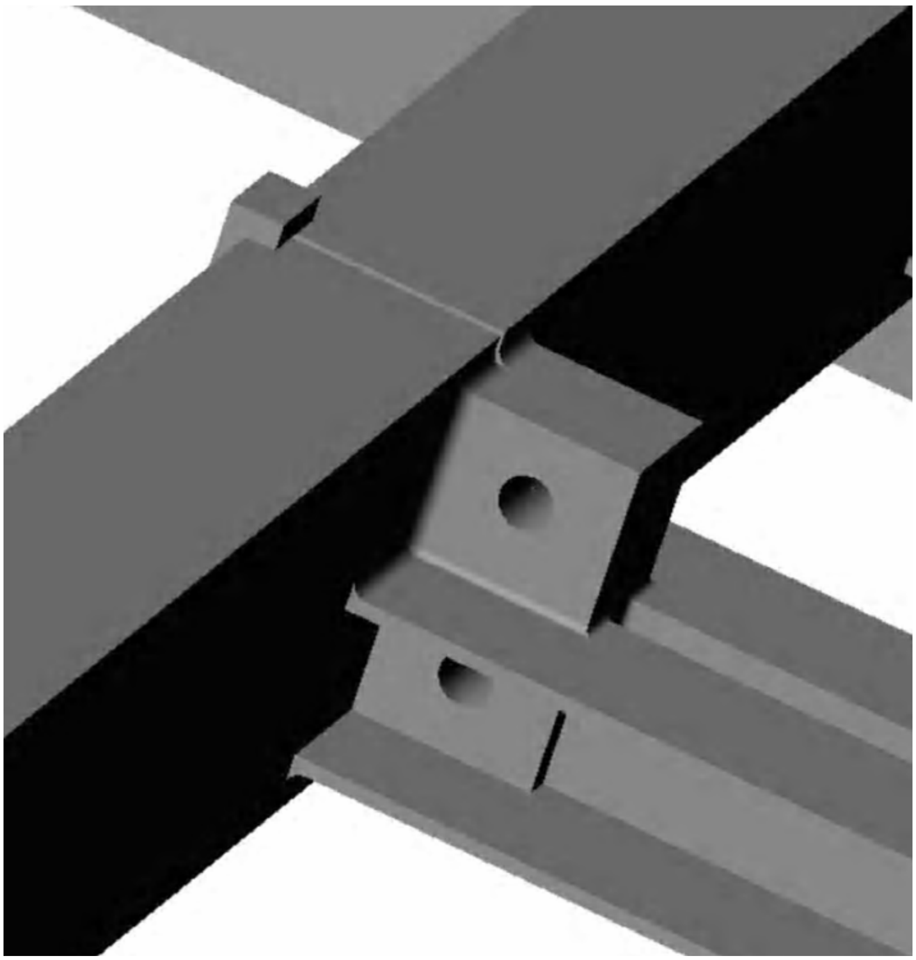
TC6K Tower Elevator Rail
Load Case With Maximum Y - Shear Stress
in Splice Plate Weld Due to Normal Elevator Operation
Maximum Dead Load Plus Braking

Finite Element Model Showing Pattern of Stress
At Various Locations in the Weld

Splice Plate Bolting

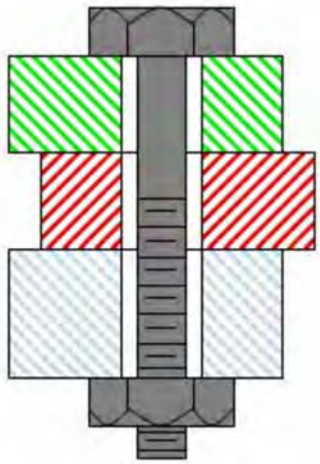
The rail assemblies are held together at their junctions by four seven-eighth inch diameter bolts which pass through 1 1/4 X 3 X 6 inch plates welded to the rail ends.

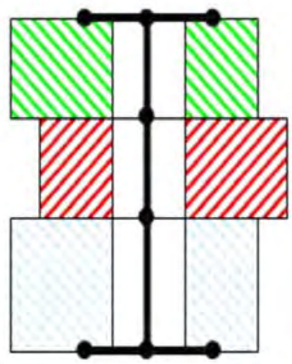
In the finite element model the plates interface in a "surface contact" mode. Loads are transmitted between the adjacent plates in compression while at the same time these surfaces are permitted to separate freely. The bolt preload contributes to the initial deformation (and compressive stress) of the plates. The surfaces are free to slide in a frictionless manner.



The bolts are modeled by beam elements along the centerline of the bolt holes in the splice plates. The beam is bonded at each end to the nodes of the hole so that slippage is assumed not to occur in the analysis. The bolt head and nut are modeled by the use of six spokes at each end which extend from the centerline of the beam model of the bolt beyond the hole to the diameter of the head or nut. The bolts are preloaded to approximately 1000 psi. axial stress in the bolt body. This estimated preload may be changed later in the actual assembly; it is used in the analysis to simulate the "snug-tight" condition as described below.

The bolts in this analysis are preloaded to 750 lbs. in axial load, corresponding to 955 psi. axial stress in the bolts.





Finite Element Representation of Bolted Joint

The same 24 point load matrix and boundary conditions as were used for the general rail analysis and the splice plate weld analysis is used in this model.

The bolts used to join the splice plates are 7/8 inch diameter ASTM A325 or A490 bolts. As such, the governing criteria for acceptability is AISC Allowable Stress Design Specification for Structural Joints Using ASTM A325 or A490 Bolts. Section 4(a) of this document, Table 2 . Applicable data are taken from the table and reproduced here.

Table 2 Allowable Working Stress on Fasteners or Connected Material (psi.)

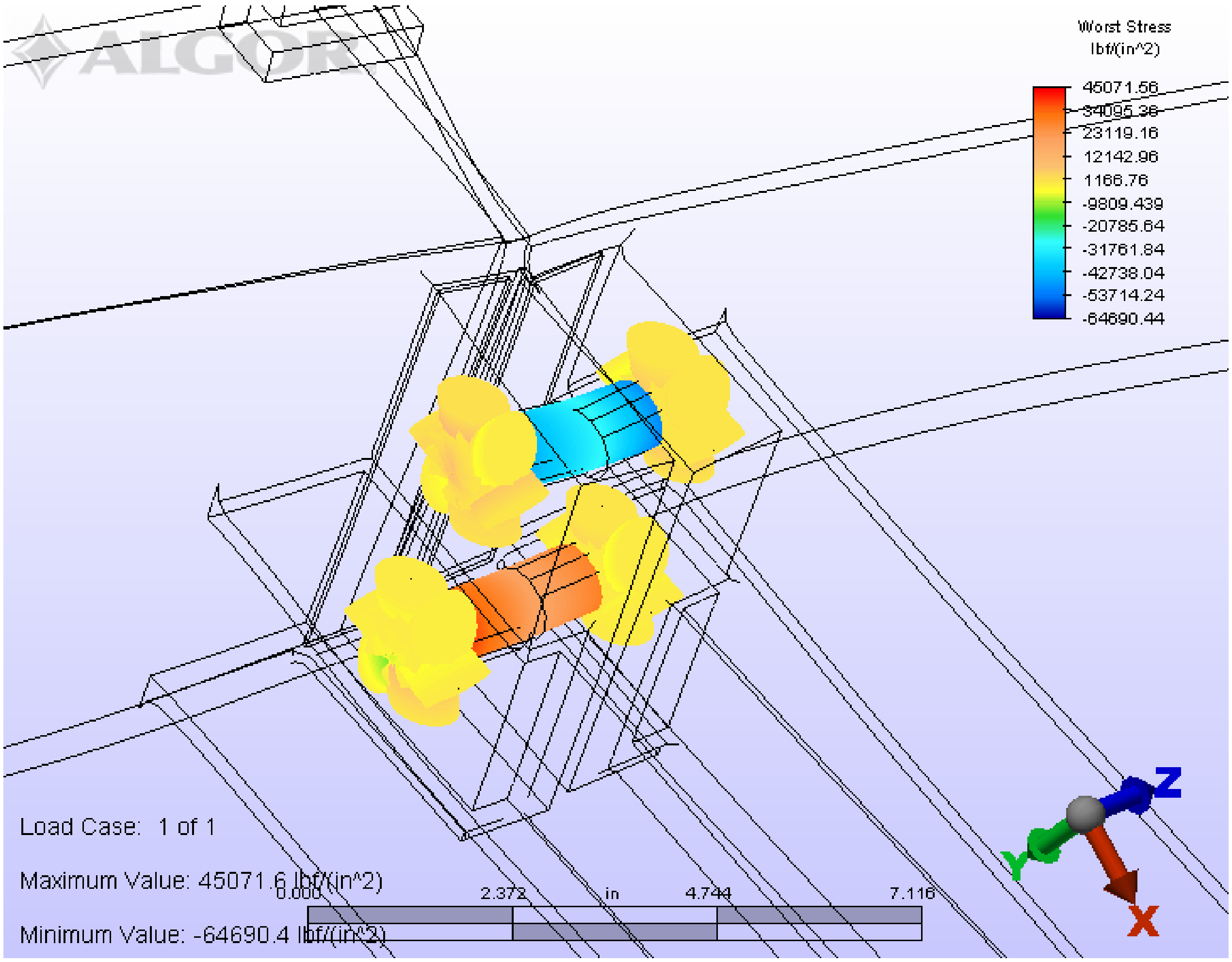
Load Condition	A325	A490
Applied Static Tension	44000	54000
Shear on bolt with threads in shear plane	21000	28000
Shear on bolt without threads in shear plane	30000	40000
Bearing on connected material with single bolt in line of force in a standard or short slotted hole	Fu	Fu
Bearing on connected material with 2 or more bolts in line of force in standard or short slotted holes	1.2Fu	1.2Fu
Bearing on connected material in long slotted holes	Fu	Fu

The allowable working stresses in axial tension in this table are for bolts which are tightened to a minimum fastener tension specified in Table 4 of the AISC document. This table specifies the allowable load for slip-critical connections. Because the rail splice is not considered slip-critical (the joint does not meet the requirements for slip-criticality defined in Section 5(a) of the applicable specification), section 8(c) of the applicable code applies, which states, ”Bolts in connections not within the slip-critical category as defined in Section 5(a) nor subject to tension loads nor required to be fully tensioned bearing-type connections shall be installed in properly aligned holes but need only be tightened to the snug tight condition. A snug tight condition is defined as the tightness that exists when all plies in a joint are in firm contact.”. The code does not specify allowable working stresses for bolts which are not tightened to the minimum fastener tension specified in Table 4, therefore for this analysis, those values given in table 4 are used as guidelines for the design.

The minimum ultimate tensile stress of the splice plate material is 70,000 psi, therefore the allowable stress in bearing is 70,000 psi.

The following table summarizes the maximum bolt stress and accompanying bearing stress in the rail splice plates for the 24 point load matrix presented previously. The bearing stress which the code refers to is the nominal bearing stress. To examine the possibility of hole deformation, i.e. going “oval”, the maximum Von Mises stress in the structure is presented in the table.

It is seen that all bolts meet design requirements and that there is no propensity for plastic deformation in the holes.



TC6K Tower Elevator Rail
Typical Bolting Analysis
Splice Plates

Splice Plate Bolting Stress Summary

Max Mx	Fx	Fy	Sa	Sw	Sb	Tx	Ty	Sbx	Sby
1	379	79	798	11413	<17539	630.301	131.382	346.4351	72.21207
2	667	195	8185	23229	<18887	1109.263	324.2974	609.6892	178.245
3	1302	407	18071	43385	<28834	2165.308	676.8668	1190.128	372.0293
4	1081	415	19904	40907	<31286	1797.771	690.1713	988.117	379.3419
5	940	400	11346	48483	<32007	1563.28	665.2253	859.2322	365.6307
6	747	362	6014	18515	<27537	1242.308	602.0289	682.8154	330.8958
7	262	53	581	6787	<17248	435.7226	88.14236	239.4881	48.44607
8	382	77	668	10919	<18853	635.2902	128.0559	349.1773	70.38391
Max My									
1	361	83	1339	10063	<18393	600.3659	138.0343	329.9817	75.86837
2	639	151	5598	20210	<33843	1062.697	251.1226	584.0951	138.0256
3	1252	267	14659	41937	<49687	2082.155	444.0379	1144.424	244.0585
4	895	453	15335	44062	*<50817	1488.442	753.3677	818.0987	414.0768
5	837	449	9166	52987	*<58421	1391.984	746.7154	765.0823	410.4205
6	625	337	8463	23663	<29392	1039.415	560.4524	571.298	308.0439
7	199	41	541	5754	<15920	330.9496	68.1856	181.9013	37.47715
8	411	69	1677	11609	<22973	683.519	114.7514	375.6856	63.0713
Max Fx									
1	354	82	1219	9977	<16553	588.7244	136.3712	323.5832	74.9543
2	523	150	3330	17554	<32500	869.7821	249.4595	478.0622	137.1115
3	1022	199	10542	35326	<48208	1699.651	330.9496	934.1865	181.9013
4	718	369	10864	37776	<49485	1194.079	613.6704	656.3071	337.2943
5	717	396	7879	45071	*<56233	1192.416	658.5731	655.3931	361.9744
6	556	273	7434	21190	<29228	924.6632	454.0163	508.2267	249.543
7	210	41	690	5921	<14812	349.2433	68.1856	191.9561	37.47715
8	361	68	850	10261	<20619	600.3659	113.0883	329.9817	62.15722

A1=.6013 = Bolt Area
A2 = Bearing Area = 1.094
Fx = Shear Force X
Fy = Shear Force Y
Sa = Axial Stress
Sw = Worst Stress
Sb=Struct. Von Mises Stress
Tx Shear Stress X
Ty Shear Stress Y
Sbx = Bearing Stress X
Sby = Shear Stress Y

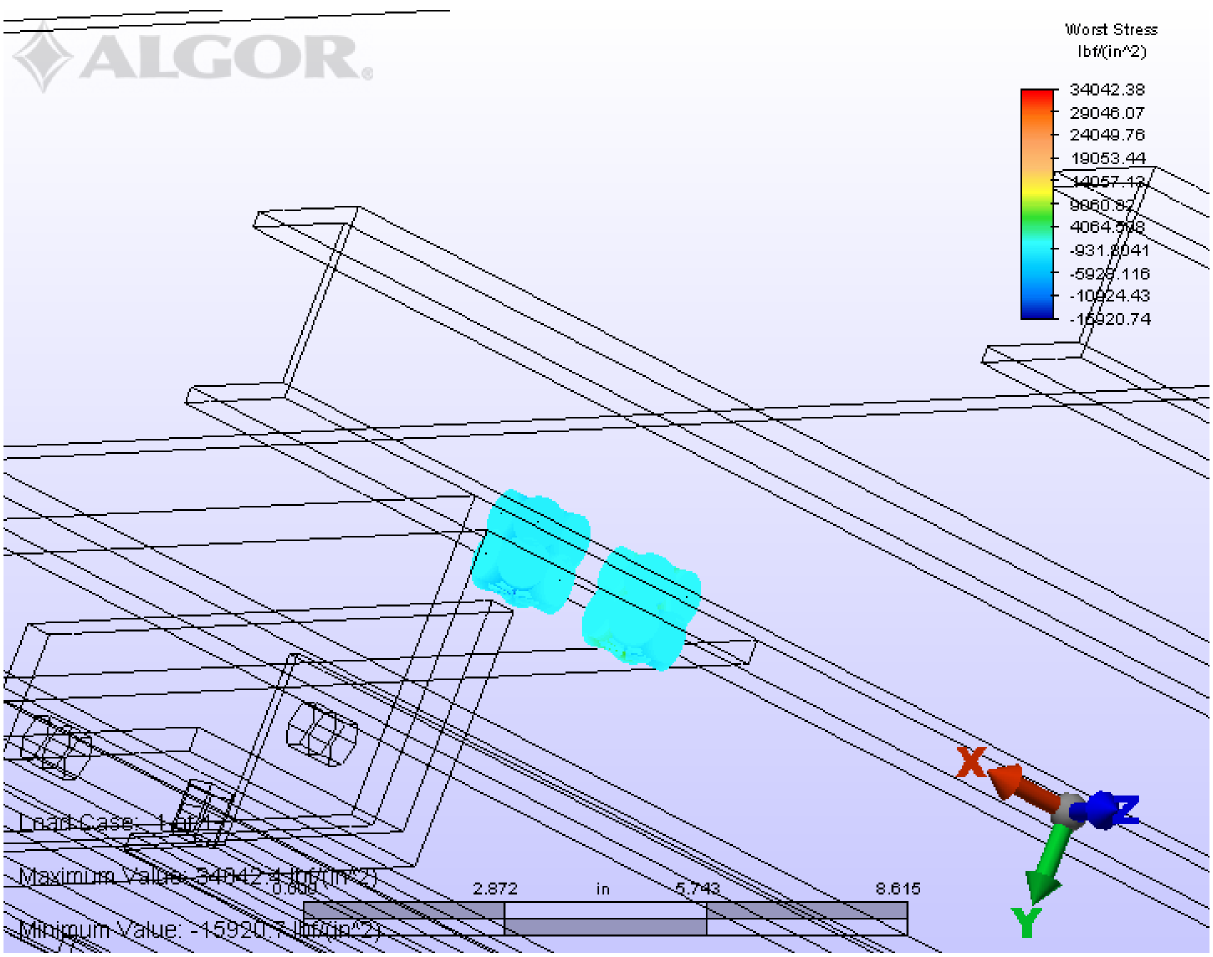
Maximum Shear and Axial may not be on the same bolt

All stresses are in psi units

*Not at Hole

Horizontal Strut Bolting

There are eight ¾ inch diameter bolts connecting each universal bracket to the rail frame. The horizontal strut bolting is analyzed in the same manner as the splice plate bolting. The maximum Von Mises stress in the structure is omitted from the table here because the splice plates are assumed bonded in the analysis to provide more conservative results for the bolting.



TC6K Tower Elevator Rail
Typical Bolting Analysis
Horizontal Strut-Universal Bracket

Horizontal Strut- Universal Bracket Bolting Stress Summary

Max Mx	Fx	Fy	Sa	Sw	Tx	Ty	Sbx	Sby
1	316	2460	9224	52074	715.3	5568.1	1685.3	13120.0
2	354	2720	9392	65299	801.3	6156.6	1888.0	14506.7
3	443	3435	9993	79482	1002.7	7775.0	2362.7	18320.0
4	508	3957	12325	95611	1149.8	8956.5	2709.3	21104.0
5								
6	427	3363	12353	88773	966.5	7612.0	2277.3	17936.0
7	459	2380	7957	51496	1038.9	5387.1	2448.0	12693.3
8	492	2634	2108	33506	1113.6	5962.0	2624.0	14048.0
Max My								
1	291	2298	10522	46749	658.7	5201.4	1552.0	12256.0
2	317	2546	10675	56450	717.5	5762.8	1690.7	13578.7
3	344	2809	8947	61161	778.6	6358.1	1834.7	14981.3
4	392	3239	11045	72818	887.3	7331.4	2090.7	17274.7
5	425	3524	12124	77645	962.0	7976.5	2266.7	18794.7
6	756	3292	11080	66838	1711.2	7451.3	4032.0	17557.3
7	297	2334	7991	47545	672.2	5282.9	1584.0	12448.0
8	343	2562	4377	34042	776.4	5799.0	1829.3	13664.0
Max Fx								
1	281	2218	8440	40384	636.0	5020.4	1498.7	11829.3
2	300	2415	8604	40664	679.0	5466.3	1600.0	12880.0
3	327	2683	7373	50001	740.2	6072.9	1744.0	14309.3
4	369	3058	8937	57980	835.2	6921.7	1968.0	16309.3
5								
6	366	3038	9375	57117	828.4	6876.4	1952.0	16202.7
7	286	2275	6740	40927	647.4	5149.4	1525.3	12133.3
8	314	2338	6075	40845	710.7	5292.0	1674.7	12469.3

A1=.4418 = Bolt Area
A2 = Bearing Area = .1875
Fx = Shear Force X, lbs.
Fy = Shear Force Y, lbs.
Sa = Axial Stress
Sw = Worst Stress
Tx Shear Stress X
Ty Shear Stress Y
Sbx = Bearing Stress X
Sby = Shear Stress Y

All stresses are in psi units

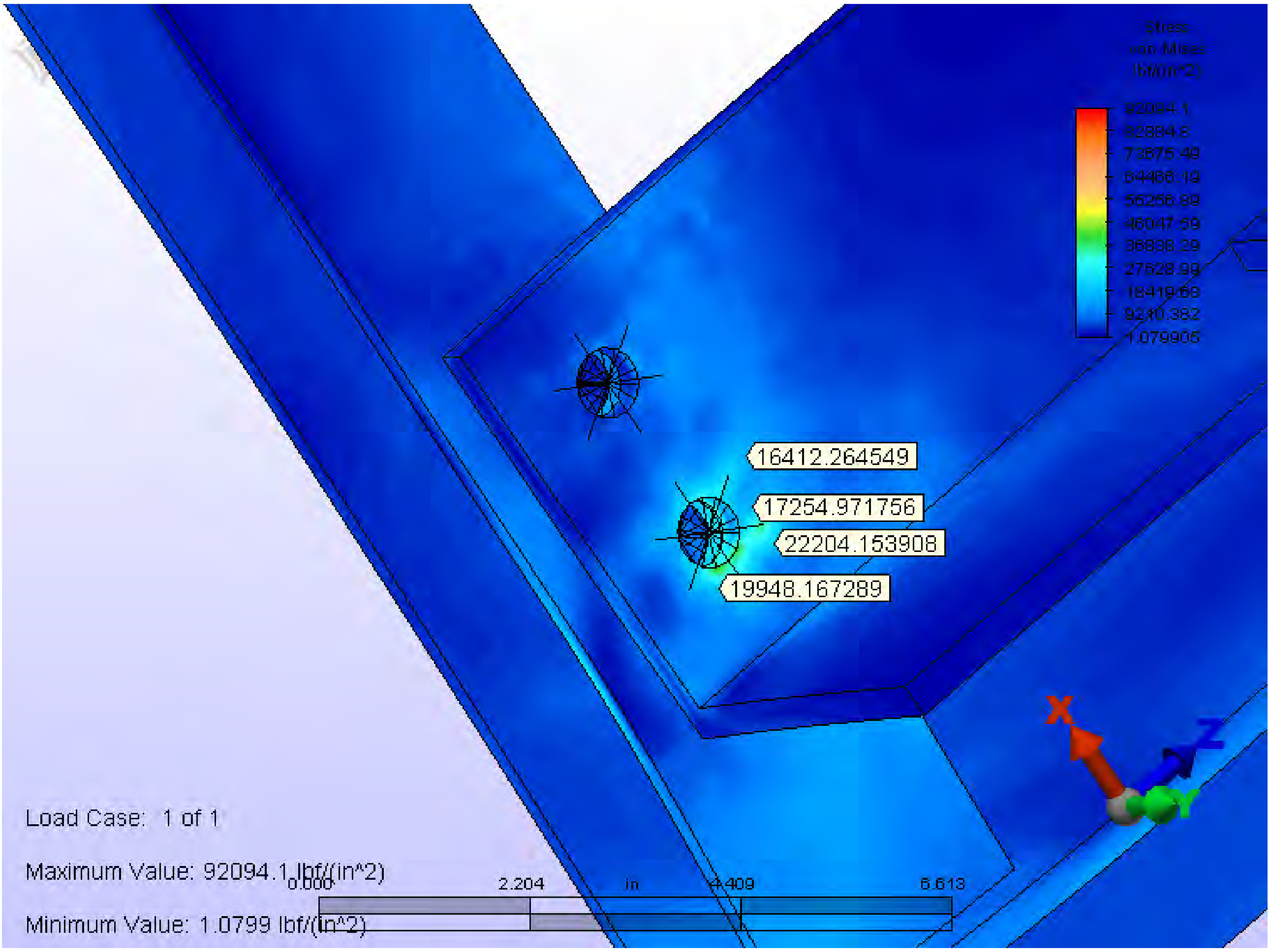
Max Shear and Axial may not be on the same bolt

The allowable stresses for the bolt and bolted material are listed in the "Table 2 Allowable Working Stress on Fasteners or Connected Material (psi) ", above. The allowable working stress in axial tension of the bolts is not exceeded in any of the horizontal strut-universal bracket bolts for any of the 24 load cases analyzed. Similarly, the allowable shear stress in the bolts in not exceeded for any of the 24 load cases utilized. The nominal bearing stress based upon the calculated bolt shear loads in the horizontal struts and the universal bracket is also below maximum allowed for all 24 load cases. While it is not a code criteria, the calculated "worst" stress in the bolts is provided in the table, which is the combined axial plus bending stress. In some cases this value is very high, but it does not exceed the ultimate strength of the bolts, which is 105, 000 or 120,000 psi, depending on size for the A325 bolts and 170,000 psi for the A490 bolts.

Because of this high value of "worst" stress, the bolted material at the location of highest "worst" stress is examined for propensity to tear. As can be seen in the illustration below, the material surrounding the bolt hole in the mating parts does not exhibit stresses near the yield strength of the material (50,000 psi min.).

For comparative purposes, the maximum calculated values of stress in the horizontal strut – universal bracket bolts when the rail assembly is subjected to normal operating loads consisting of dead weight plus emergency stopping are:

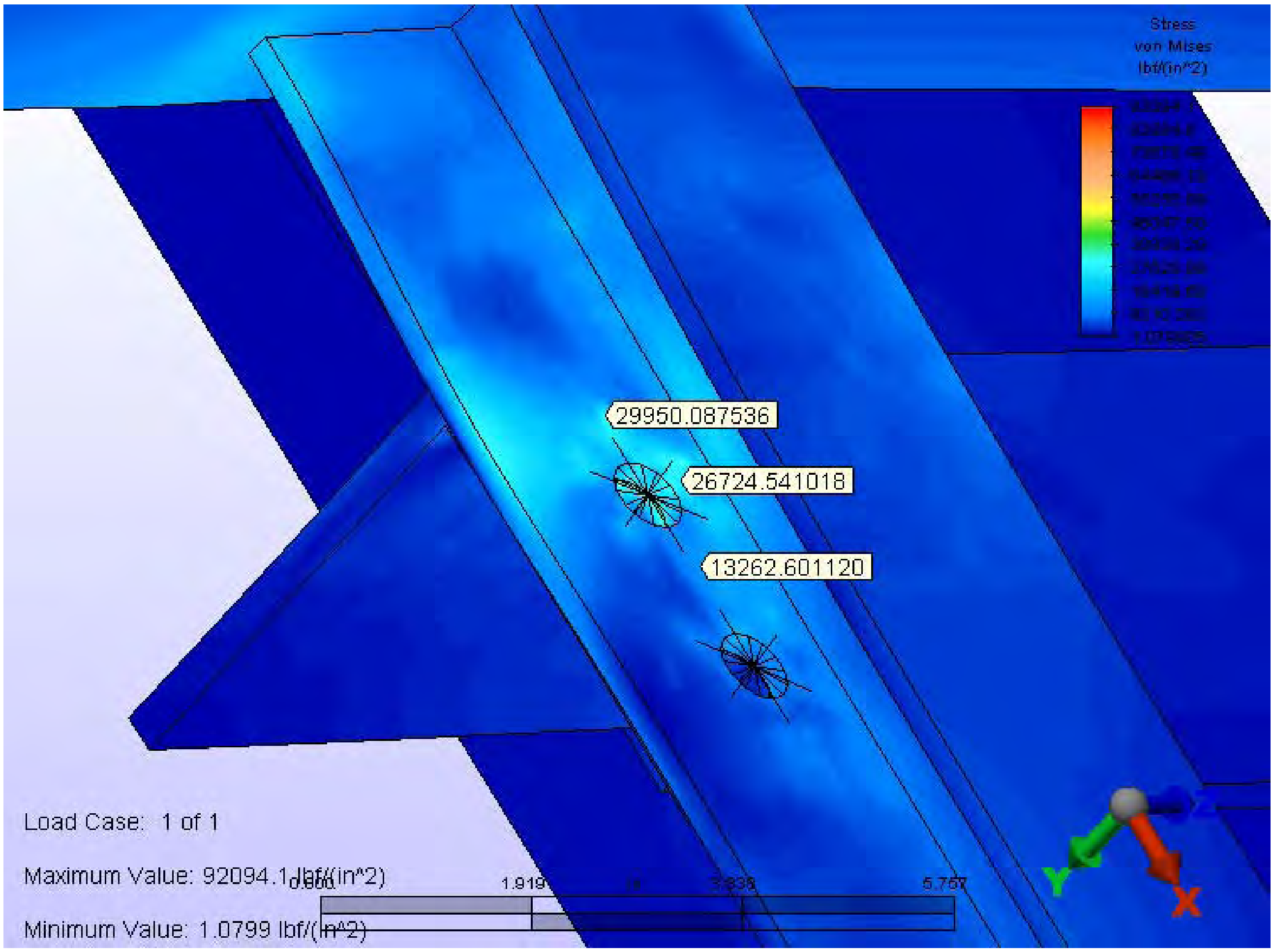
Shear X = 835 psi.
Shear Y = 6435 psi.
Axial = 8345 psi.
"Worst" stress = 66232 psi.
Bearing in X = 1968 psi.
Bearing in Y = 15163 psi.



TC6K Tower Elevator Rail

Examination of Material Surrounding Bolt Hole at Location of "Worst" Stress in Bolt at the Horizontal Strut-Universal Bracket Junction

Von Mises Stress – Bottom View



TC6K Tower Elevator Rail

Examination of Material Surrounding Bolt Hole at Location of "Worst" Stress in Bolt at the Horizontal Strut-Universal Bracket Junction

Von Mises Stress – Top View

Analysis of Wheel Load on Rail

In order to analyze the effect of the wheel load on the rail, a single rail component is subjected to a varying line load (simulating a wheel line of contact) at a mid-rail location and at the free edge of the “L” The model consists of 3-D bricks and is constrained in translation at the footing bolt holes and in “X” and “Y:” translation at the rail splice. The rail material properties are 50,000 psi. yield and 70,000 psi. ultimate, similar to 572 steel. The loads are varied from very low to across the spectrum of load combinations prescribed by the ASCE code described earlier in this document. Illustrations of the model used and typical calculated results are shown on the four pages following.

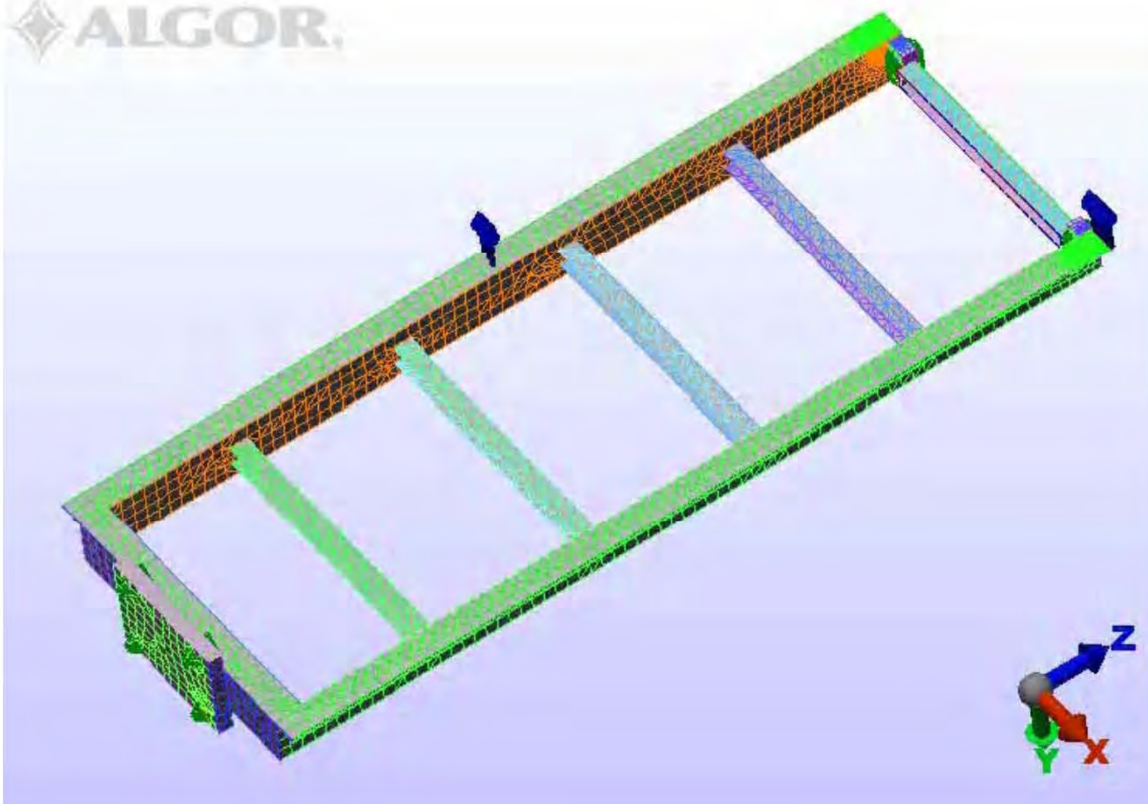
The allowable equivalent Von Mises stress by ASCE code is 0.60 X the yield stress which is 0.60 X 50,000psi = 30,000. psi.

Stress and displacement results for various load cases are given in the following table:

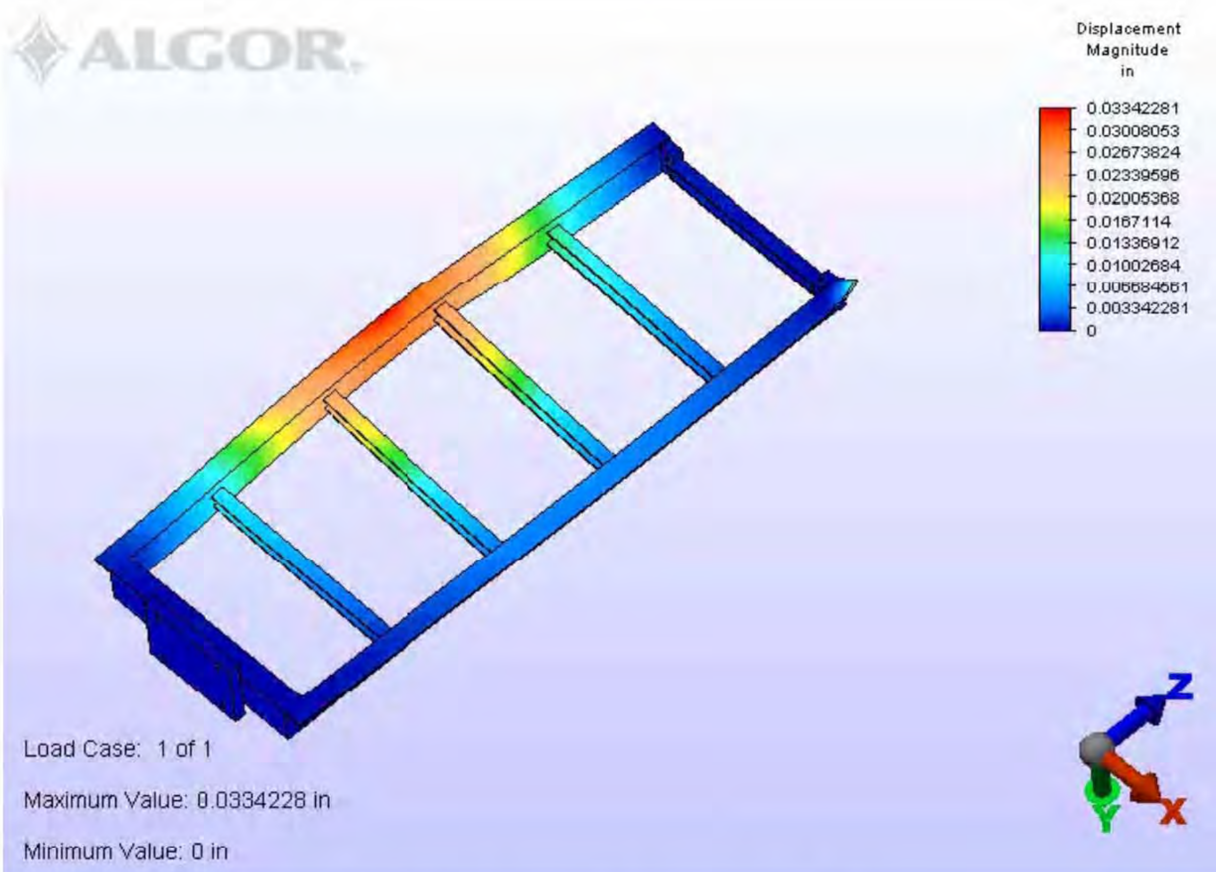
Nominal Rail

Distributed Load, lbs.	Von Mises Stress, psi	Displacement, in.
600	1680	.023
1800	5003	.033
3000	8339	.056
3600	10007	.067

The calculated maximum distributed wheel load for the most adverse single wheel loading (Dead Weight, Cross Wind) is 3170 lbs. The nominal rail does not exceed allowable design stress (30,000psi) with this loading,

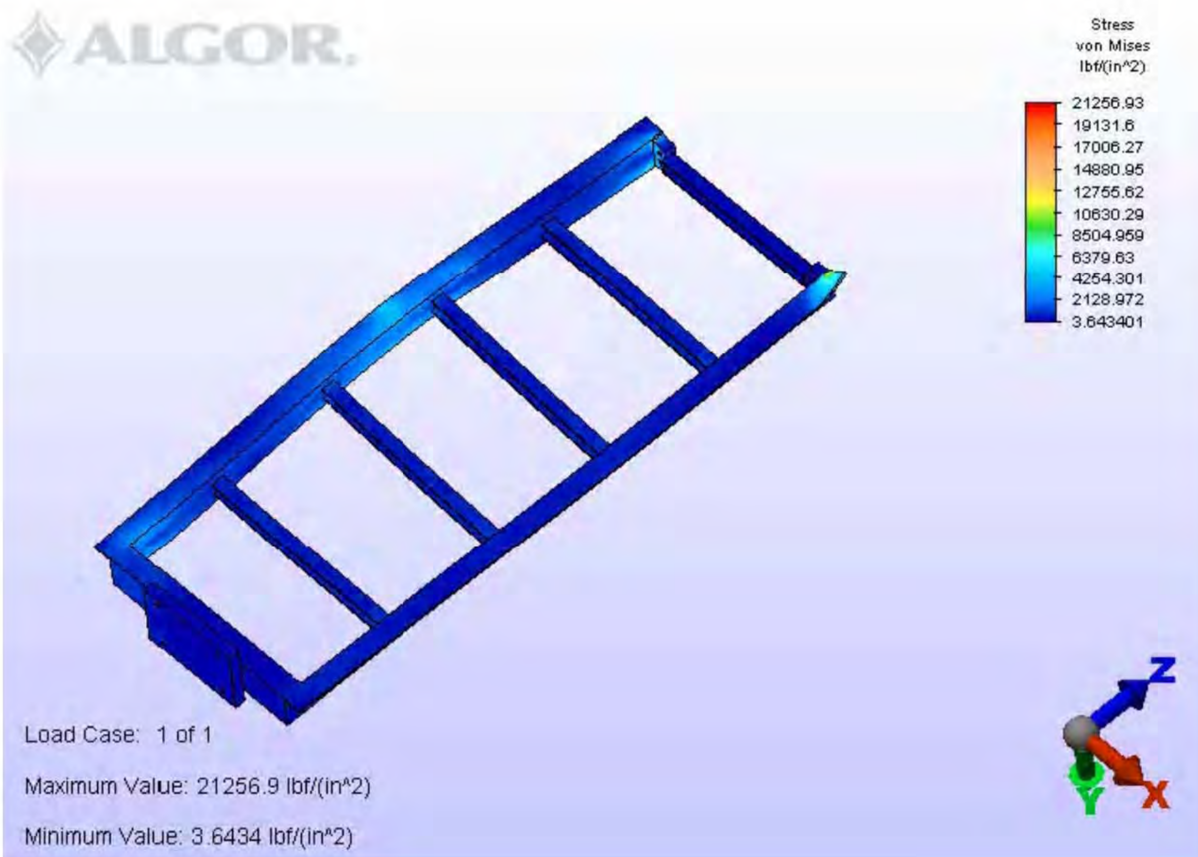


Finite Element Model of TC6K Tower Elevator Rail
Wheel Load on Rail "L" at Mid Rail and at Edge



Finite Element Model of TC6K Tower Elevator Rail
Wheel Load on Rail "L" at Mid Rail and at Edge

Typical Displacement Pattern
(greatly exaggerated)



Finite Element Model of TC6K Tower Elevator Rail
Wheel Load on Rail "L" at Mid Rail and at Edge

Typical Von Mises Stress Pattern

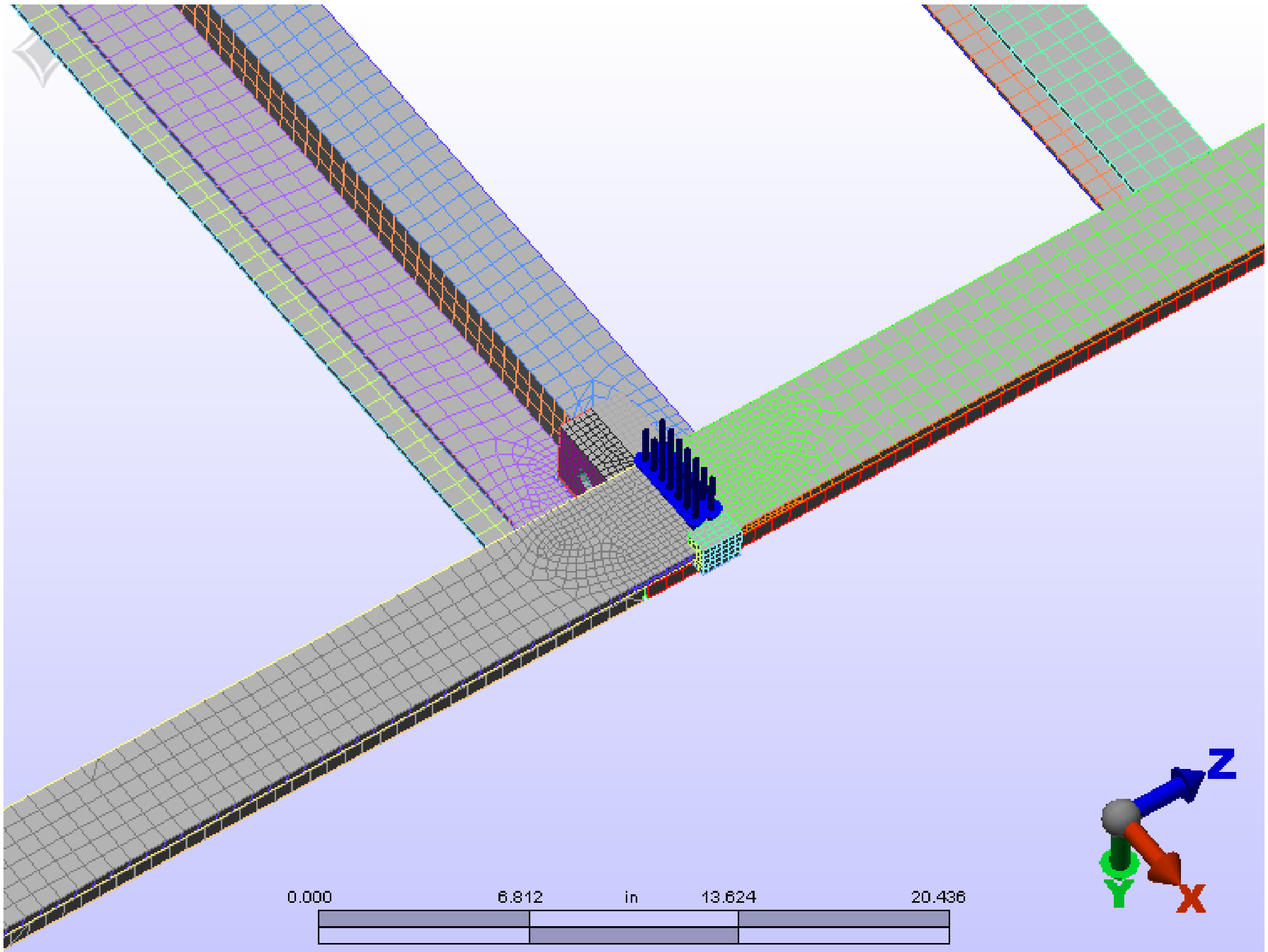
Alignment Clip Analysis

It is necessary to support the free ends of the rail at the rail splice. For this purpose, an alignment clip which is 1 ½ X 1 ¼ X 7/8 inches is welded along one-half of its length to the end of each rail of each pair of mating rails as shown in the figure on the page XX. Application of a distributed load across the rails at the location of the rail splice (an illustration of the finite element model is found on pages 64 and 66) in the "upward" and "downward" directions yielded the following calculated results:

Rail at Splice Junction – Alignment Clip

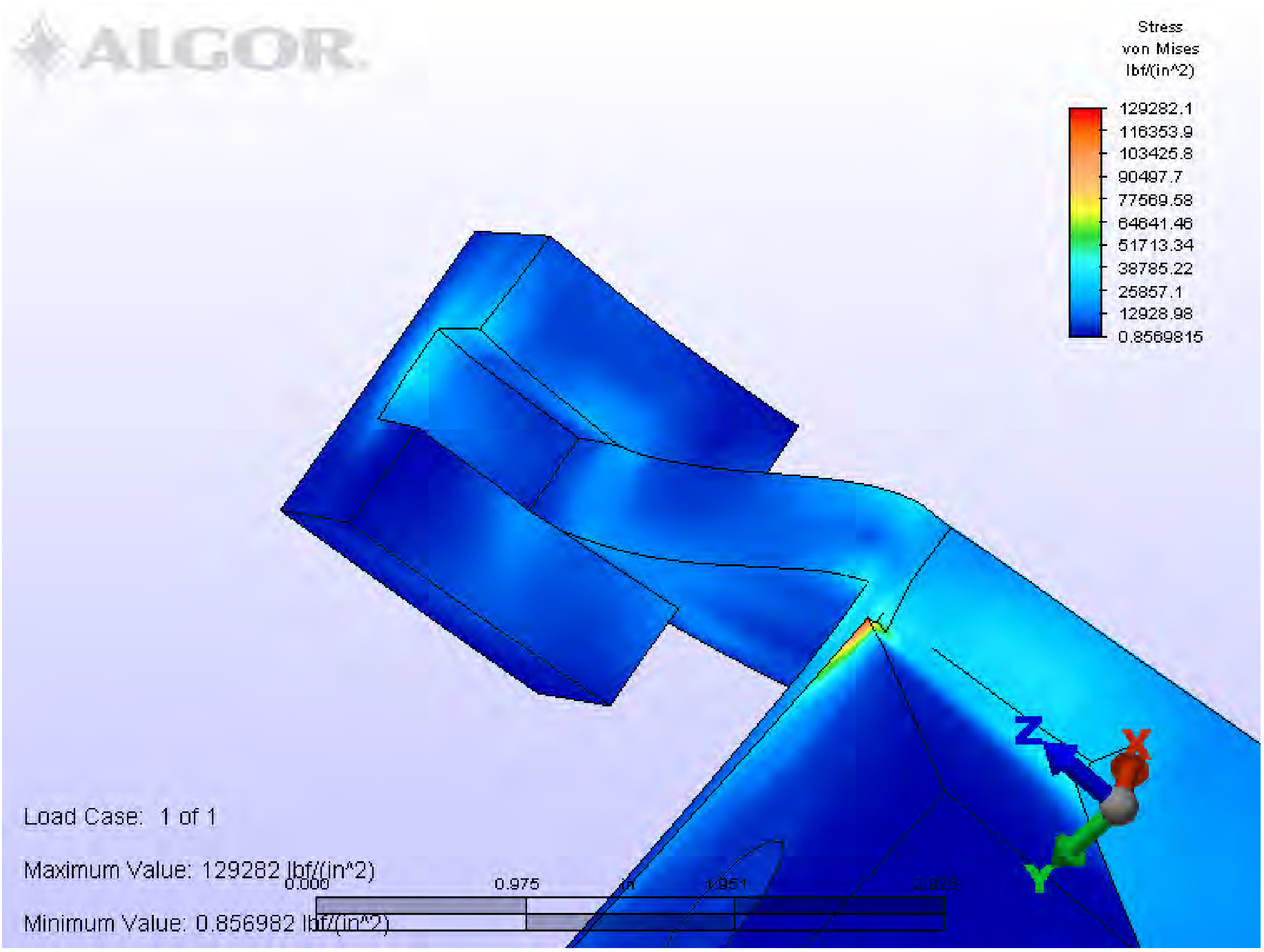
Distributed Load, lbs.	Von Mises Stress, psi "Downward Load"	Von Mises Stress, psi "Upward Load"
500	5450	6178
1000	10900	12357
1500	16352	18537
2000	21803	24716
2500	27255	30896
3000	32705	37075
3500	38157	43256

The maximum Von Mises stress calculated in the alignment clip is localized to the edge of the clip in contact with the rail. The figure on the pages 65 and 67 illustrate this. The calculated Von Mises stress in the rail remains below the yield strength over the range of the loads presented in the table. The arrangement can be is judged acceptable on the basis that the clip will not yield even under an extreme case where the elevator wheel is located exactly at the rail junction during an extreme load case such as an earthquake, the clip is not a "primary structural part" of the rail assembly (i.e. the rail will not fail if the clip does), and the stresses are localized to the edges of the clip – the "nominal" stresses are much lower and below maximum allowable stresses specified by code.



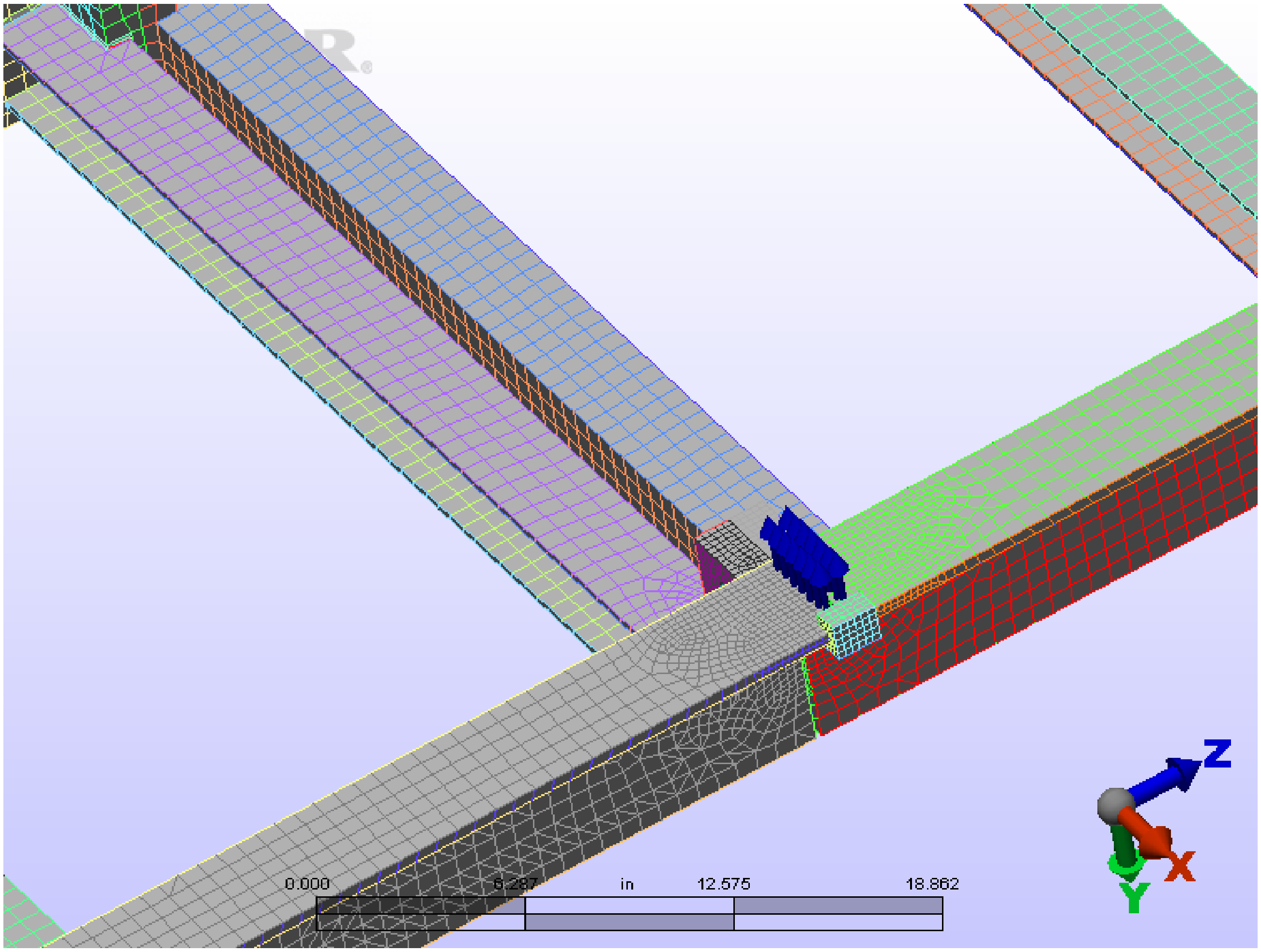
TC6K Tower Elevator Rail

Finite Element Model with Alignment Clip
Showing “Downward” Load Application



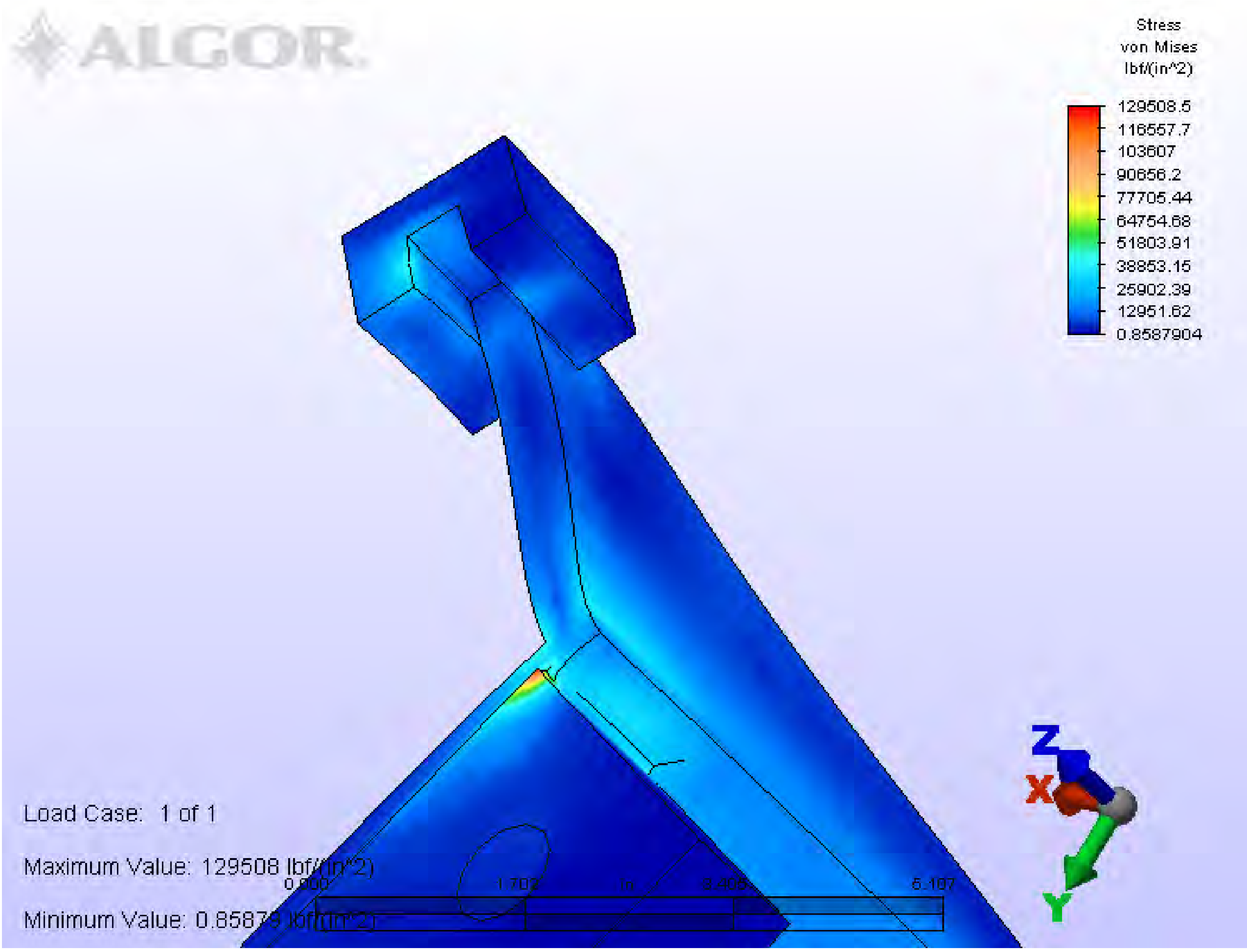
TC6K Tower Elevator Rail

Finite Element Model with Alignment Clip
Showing Typical Stress Pattern
"Downward" Load on Rail Junction



TC6K Tower Elevator Rail

Finite Element Model with Alignment Clip
Showing “Upward” Load Application



TC6K Tower Elevator Rail

Finite Element Model with Alignment Clip
Showing Typical Stress Pattern
"Upward" Load on Rail Junction

Suggested Additional Analysis:

1. The analysis performed during the early stages of the design to determine the load on the rail support brackets (attaching the rail to the building) was performed using only loads due to maximum moment about the X axis (out of the plane of the rail). This should be supplemented with calculations of loads due to maximum moment about the Z axis (out of the plane of the rail) and for maximum transverse load to insure the original estimates of load on the support structure are not exceeded.
2. Late in the design phase, the material thickness of the horizontal struts was changed from 5/16 in. to ¼ in. This resulted in a required downsize of the weld bead attaching the horizontal struts to the rail members from ¼ in to 3/16 in. (unless the weld is especially designated on the drawings to be built out to obtain full throat thickness) in accordance with the code. The analysis assumes that the weld is the same thickness as the material, i.e. ¼ in. While the stresses appear low under all conditions of load, it might be a good idea to verify that the 3/16 in. weld is indeed adequate.