Special Elite Engineers Webinar Sequence

An Insider's Perspective

Wednesday, July 15th, 2015

3:00 PM – 4:00 PM EST

Our Elite Engineer of the Month: Daniel Baxter, P.E., S.E.

Copyright Materials

Speaker Information

Daniel Baxter, P.E., S.E.

Senior Bridge Engineer | Michael Baker International

EDUCATION

- **MS, Structural Engineering, Washington University in St. Louis**
- **BS, Civil Engineering, Washington University in St. Louis**
- **BA, Carleton College**

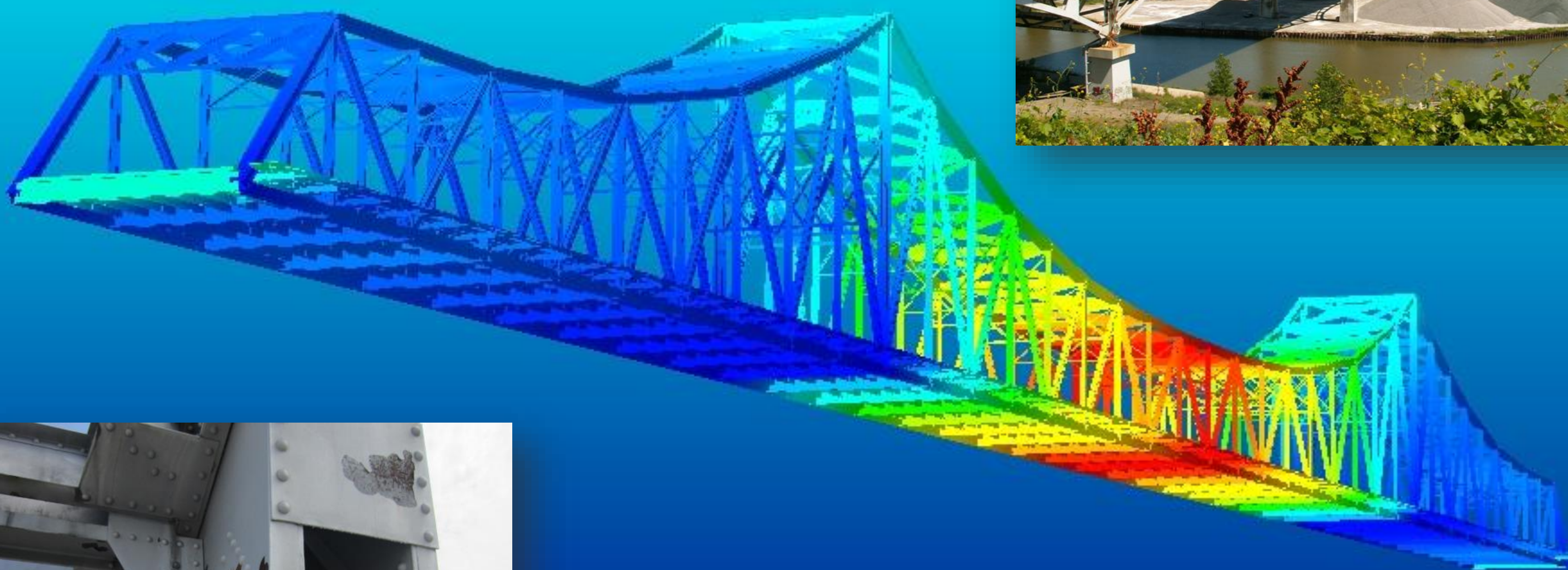
PAST PROJECT

- **Winona Bridge and Cleveland Innerbelt Bridge Truss Rehabilitations**
- **Milton Madison Bridge New Truss Structure**
- **Multiple post-tensioned concrete and steel arch, box, and I girder bridges**

Learning Objectives

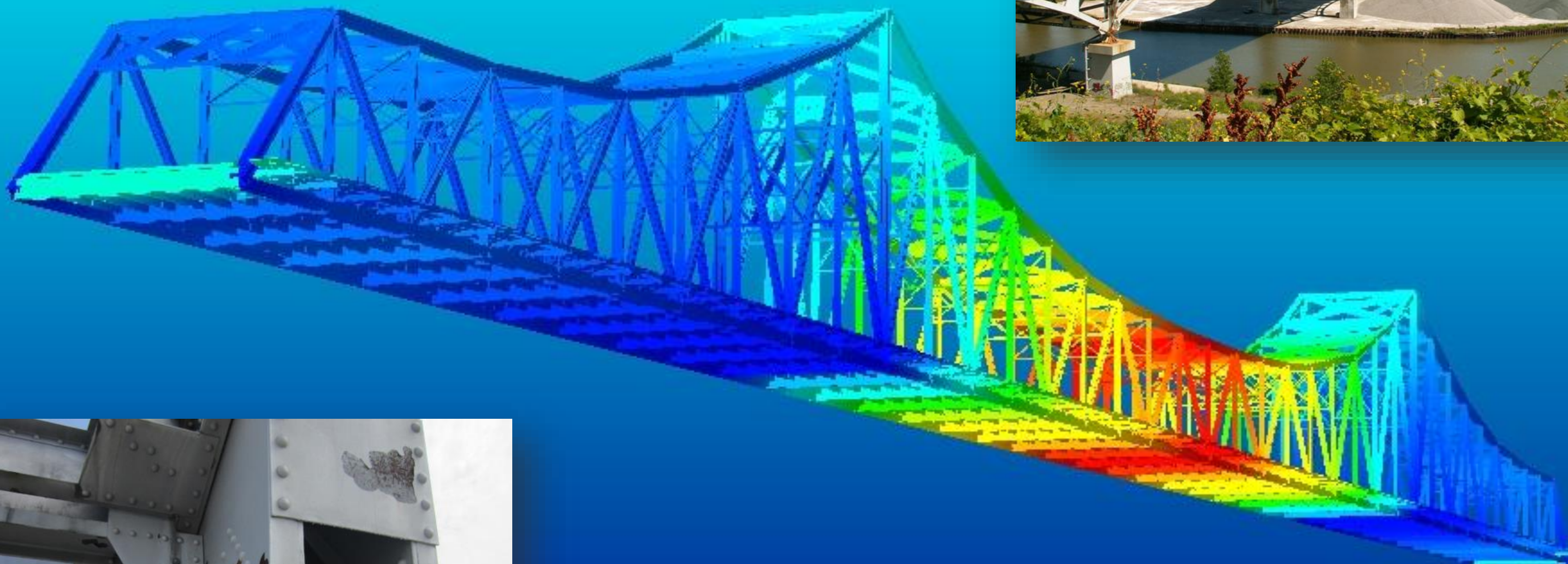
At the end of the this course, participants will be able to understand:

1. Analysis and Modeling methods of Truss Bridges
2. Determining appropriate fixity conditions for truss members
3. When to use different modeling and analysis approaches
4. Application of FEA for specific regions of truss bridges



Analysis and Modeling Approaches for Truss Bridges

7/15/15



Presented by:

Daniel Baxter, P.E., S.E.,

Michael Baker International

Analysis and Modeling

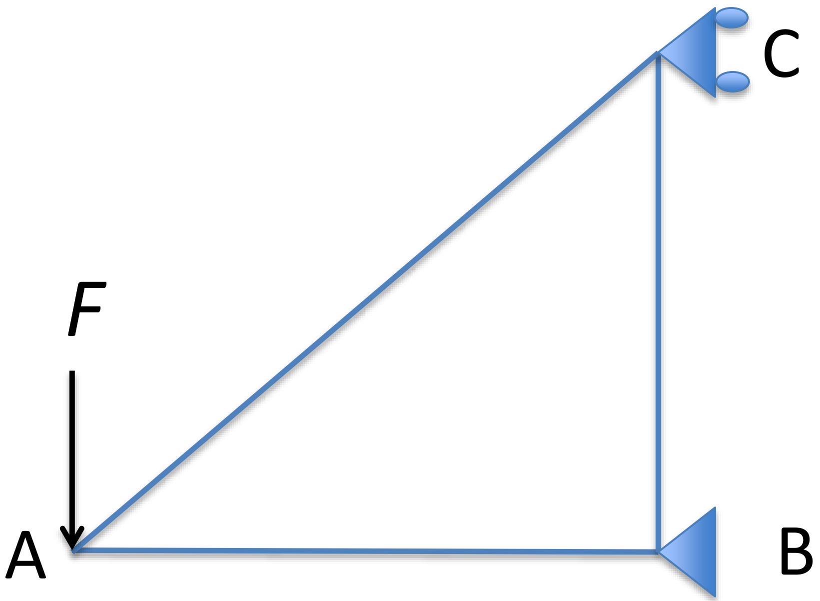
- What is an idealized truss?
- Bending moments in truss bridges
- Should a truss be modeled as pinned or fixed?
- 2D versus 3D modeling
- Case study of detailed 3D modeling

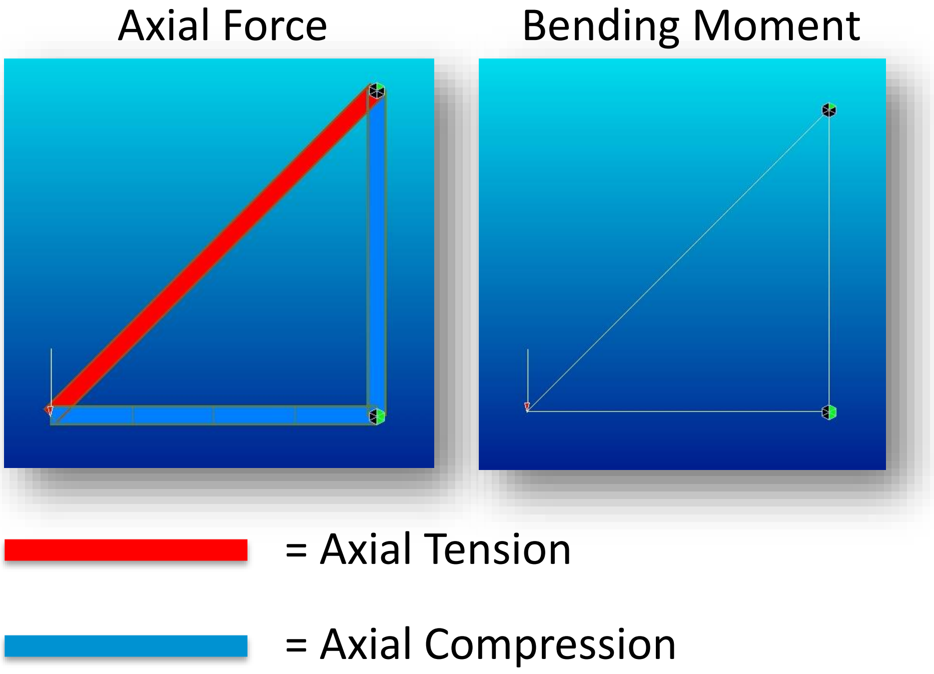
What is an idealized truss?

- *A truss is a structure composed of members joined together at their endpoints. The members are joined together by smooth pins and all loadings are applied at the joints*.
- *Each truss member acts as an axial force member, subject to either axial tension or compression.*

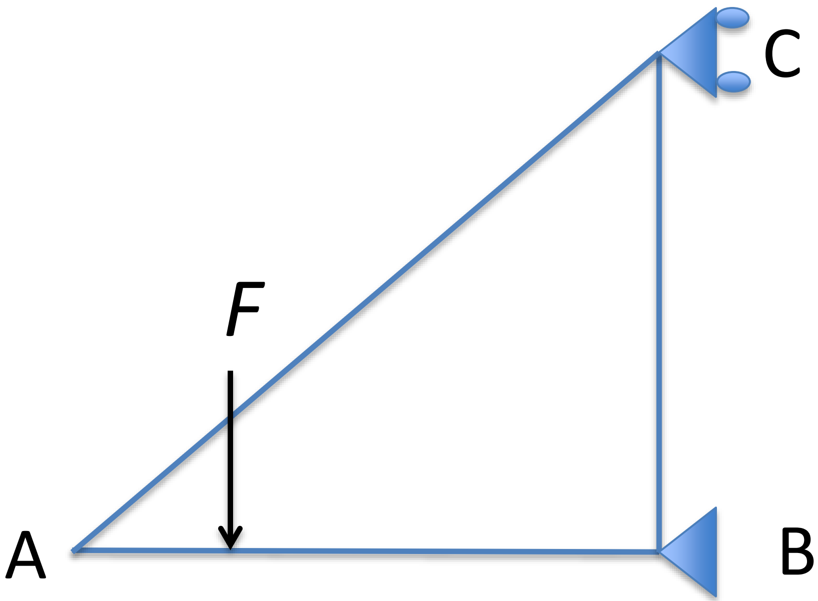
Source: Hibbeler, *Structural Analysis, Fourth Edition*, pp. 74.

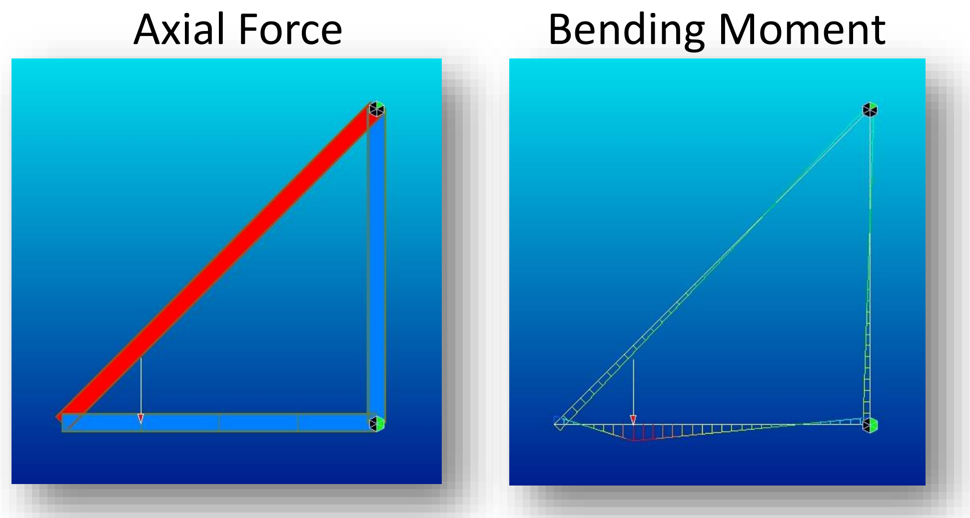
Forces in an Idealized Truss





What happens when load is applied away from a joint?

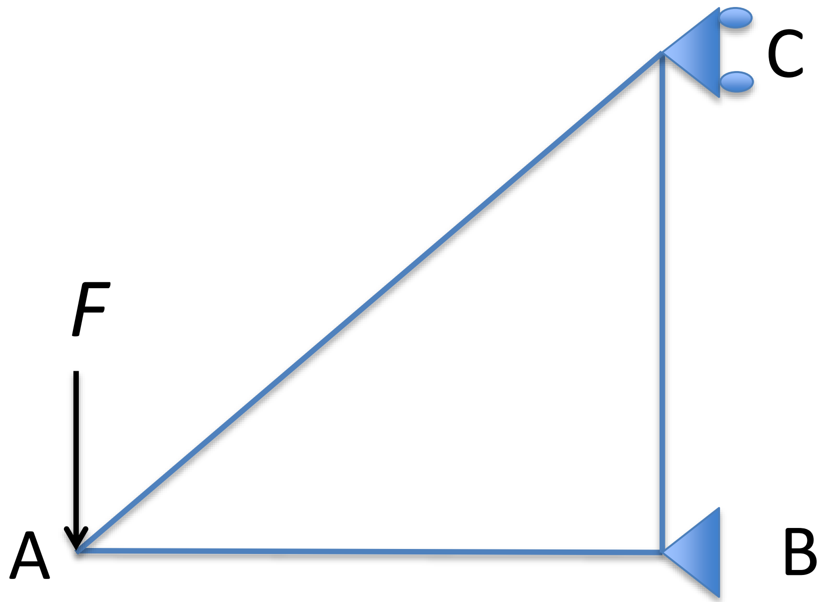


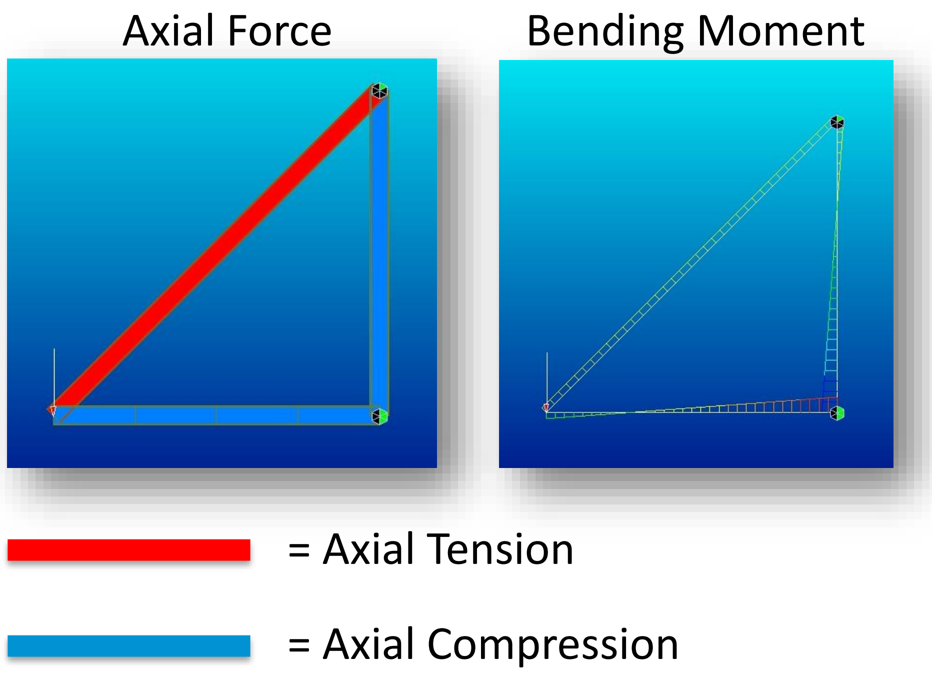


= Axial Tension

= Axial Compression

What happens if the joints are fixed?

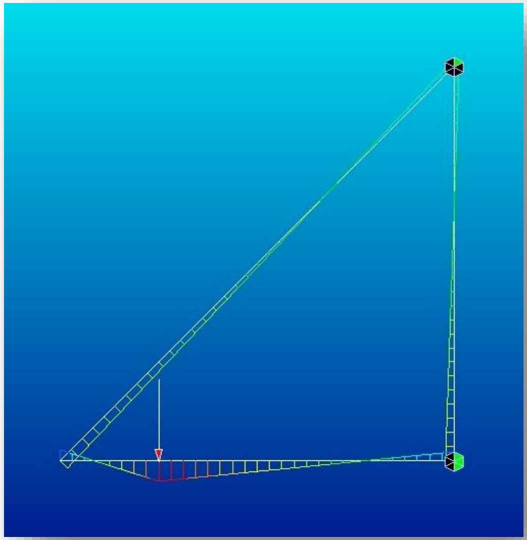




Primary Moments

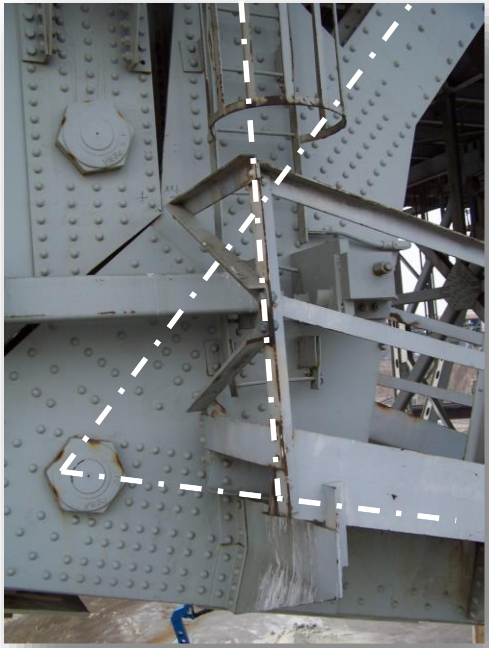
- *Primary moments are bending moments that the truss members must develop to remain in equilibrium while carrying load.*

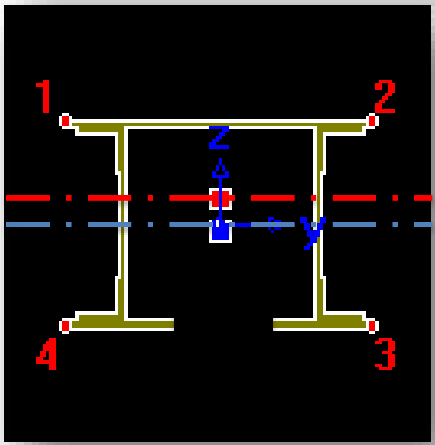
- *Since the truss is loaded away from the work point of a joint, it is said to be eccentrically loaded.*



Sources of Primary Moments

- Worklines that don't meet at a single point
- Centroid does not coincide with the workline
- Loads applied away from panel points
- Member self-weight

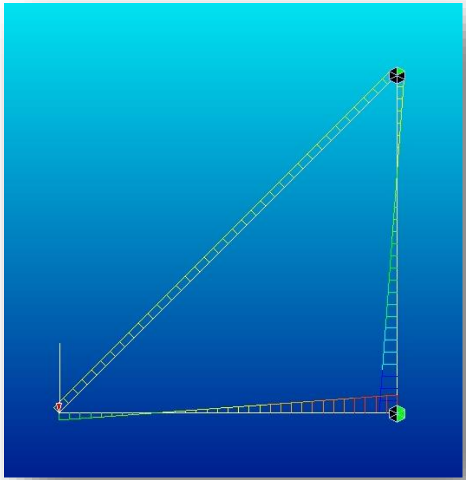




Secondary Moments

- *Bending forces in truss members that are not required to satisfy equilibrium are termed secondary moments, or secondary stresses.*

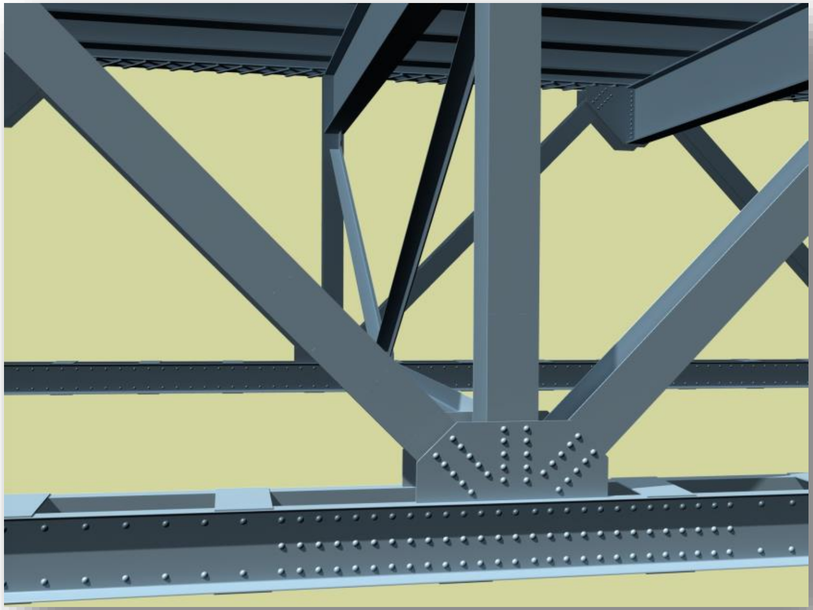
- *These are <u>not</u> the same as second order moments, which are caused by axial forces applied to compression members in their deflected position.*



Sources of Secondary Moments

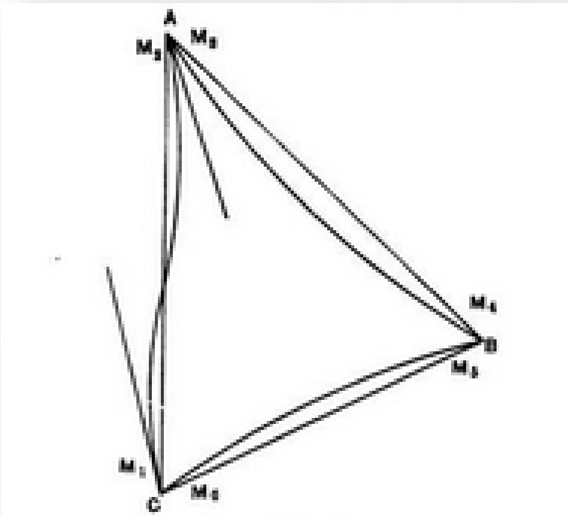
- Rigid connections between members

Pinned



Rigid

Secondary Moments



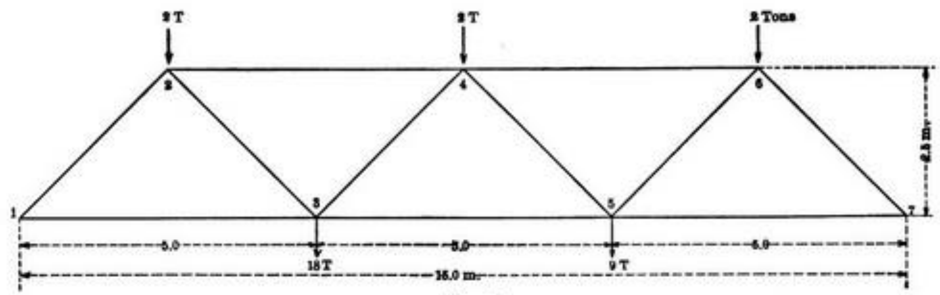


Fig. 45.

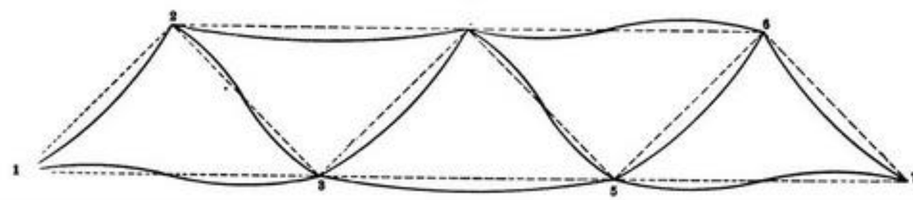


Fig. 45 A.

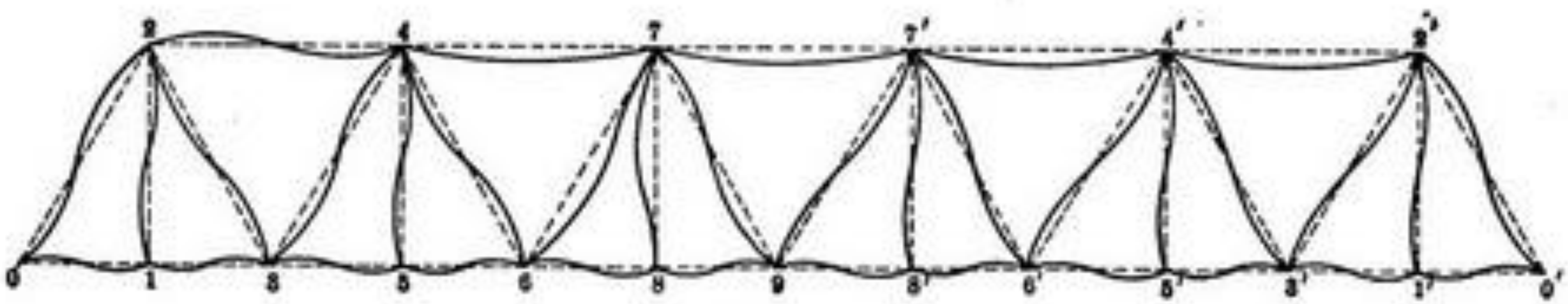


Fig. 48 A.

Should secondary moments be considered for truss analysis?

C.R. Grimm: *Secondary Stresses in Bridge Trusses*, 1908

TABLE 5 AND FIG. 48.

Span 42 m. Wrought Iron.

	Member.	Gross area, sq. cm.	I gross cm.^{4}	l cm.	b cm.	Most dangerous fiber.	e cm.	Stresses. Kg./sq. cm.	Stresses. σ' %	l/e	Sections in cm.
Bottom Chord	0–1	106	...	350	41	Bottom	9.0	+280	54	39	
	0'–1'	106	...	350	41	Bottom	9.0	+160	19	39	
	1–3	106	...	350	41	Top	31.8	+280	143	11	
	1'–3'	106	...	350	41	Bottom	9.0	+160	22	39	
	3–5	138	...	350	42	Top	34.0	+480	83	10.3	
	3'–5'	138	...	350	42	Bottom	7.6	+350	11	46	
	5–6	138	...	350	42	Top	34.0	+480	87	10.3	
	5'–6'	138	...	350	42	Bottom	7.6	+350	11	46	
	6–8	178	...	350	43	Top	35.8	+450	111	9.8	
	6'–8'	178	...	350	43	Bottom	6.8	+400	10	52	
	8–9	178	...	350	43	Top	35.8	+450	45	9.8	
	8'–9'	178	...	350	43	Top	35.8	+400	13	9.8	
Top Chord	2–4	106	14600	700	41	Bottom	31.8	−480	21	22	
	2'–4'	106	14600	700	41	Top	9.0	−300	13	78	
	4–7	162	18100	700	42.2	Top	6.9	−480	8	101	
	4'–7'	162	18100	700	42.2	Top	6.9	−350	9	101	
	7–7'	182	18450	700	43.2	Top	7.1	−400	10	99	
					b = 2e		$e_1 = e_2$			l/b	
Diagonals	2–3	93.6	...	712	35	...	17.5	+460	11	20	
	2'–3'	93.6	...	712	35	...	17.5	+320	17	20	
	4–6	58.8	...	712	26	...	13.0	+340	18	27	
	4'–6'	58.8	...	712	26	...	13.0	+400	15	27	
	7–9	38.8	...	712	17	...	8.5	−260	27	42	
	7'–9	38.8	...	712	17	...	8.5	+530	11	42	
	0–2	114.8	...	712	40	...	20.0	−520	44	18	
	0'–2'	114.8	...	712	40	...	20.0	−300	30	18	
	3–4	87.6	...	712	32	...	16.0	−380	26	22	
	3'–4'	87.6	...	712	32	...	16.0	−310	39	22	
	6–7	52.1	...	712	21	...	10.5	−150	47	34	
	6'–7'	52.1	...	712	21	...	10.5	−420	14	34	
Verticals	1–2	21.1	...	600	15	...	8.0	+590	12	40	
	4–5	21.1	...	600	15	...	8.0	+420	17	40	
	7–8	21.1	...	600	15	...	8.0	+575	5	40	
	7'–8'	21.1	...	600	12	...	8.0	+100	90	40	
	4'–5'	21.1	...	600	15	...	8.0	+100	60	40	
	1'–2'	21.1	...	600	15	...	8.0	+100	65	40	

Secondary stresses are calculated for gross moments of inertia.

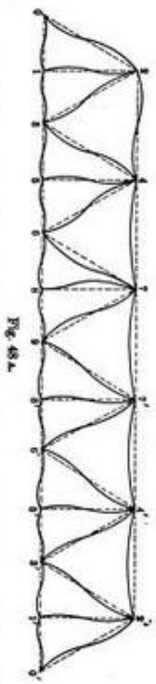


Fig. 48a.

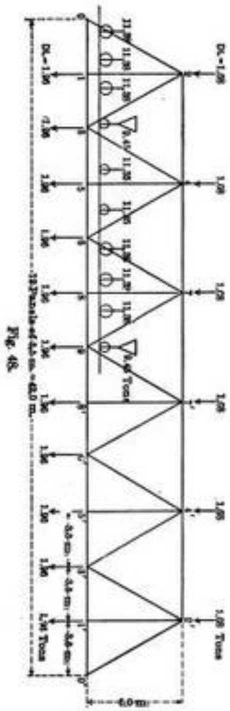


Fig. 48.

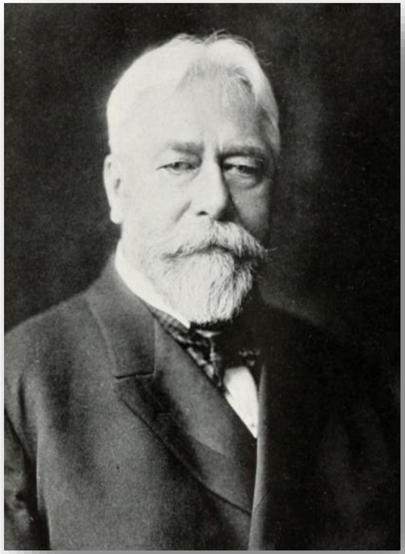
Through Warren Truss with Suspenders of a Railroad Bridge across the river Birs near Mönchenstein in Switzerland. Bridge collapsed in June, 1891.
Calculated by Professor Ritter and taken from "Schweizerische Bauzeitung," 1891.
Secondary stresses determined for dead load and positions of live load as indicated.

C.R. Grimm: *Secondary Stresses in Bridge Trusses*, 1908

- “In common cases there is no necessity for such calculations, yet in particular cases secondary stresses should be investigated;…where [they are] of great magnitude, or where a bridge has to carry much greater loads than those for which it has been designed.”

- “The writer suggests that readers who take a particular interest in this subject…examine trusses and publish their results”.

Sciotoville Bridge, 1916



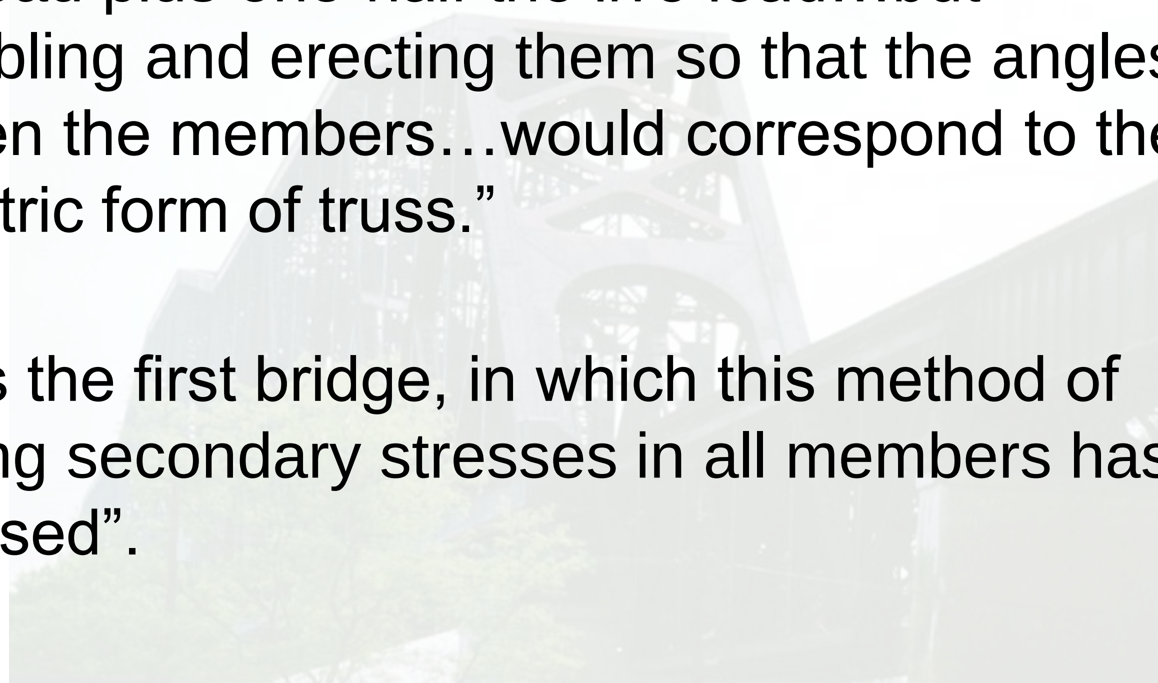
Gustav Lindenthal

David Steinman

Sciotoville Bridge, 1916

- "This was done by cambering the trusses for full dead load plus one-half the live load…but assembling and erecting them so that the angles between the members…would correspond to the geometric form of truss."

- "This is the first bridge, in which this method of reducing secondary stresses in all members has been used".



Source: *The Continuous Truss Bridge Over the Ohio River at Sciotoville, Ohio, of the Chesapeake and Ohio Northern Railway*, by Gustav Lindenthal, ASCE Transactions, 1922.

J. Parcel and E. Murer: *Effect of Secondary Stresses Upon Ultimate Strength*, 1934

TABLE 1.—CHARACTERISTICS OF TEST COLUMNS

Column No.	Test	Area, A, in square inches	Length, L, in inches	Moment of inertia, I, in inches*	Distance, c, from neutral axis to extreme fiber, in inches	Radius of gyration, r, in inches $=\sqrt{\frac{I}{A}}$	Slenderness ratio, $\frac{L}{r}$	Thickness, t, of cover-plates, in inches	Distance, b, between center lines of rivet rows, in inches	Ratio, $\frac{b}{t}$	Thickness, w, of web, in inches	Rivet spacing, p, in inches at center line	Diameter of rivets, D, in inches
1	1	3.91	35.3	23.5	3.04	2.45	35.2	1/8	5 1/8	41	1/8	2	0.17
	2	.....	30.6				33.0						
2	.....	10.70	128.2	108.2	3.75	3.18	40.3	1/4	8	32	0.24	3	3/8
3	.....	9.82	128.2	91.2	3.69	3.13	41.0	3/16	9	48	0.24	2 1/4	3/8
4	End	9.82	.....	.....	.....	....	....	....	....	....	....	.....	....
	Center Line	10.99	128.2	91.2	3.69	3.13	41.0	3/16	9	48*	0.24	2 1/4	3/8

* Reinforced.

2, Column No. 1 was the same as for Test 1, except that it was shorter. The only change in the riveting was in the cover (see Fig. 14(d)) for which a new

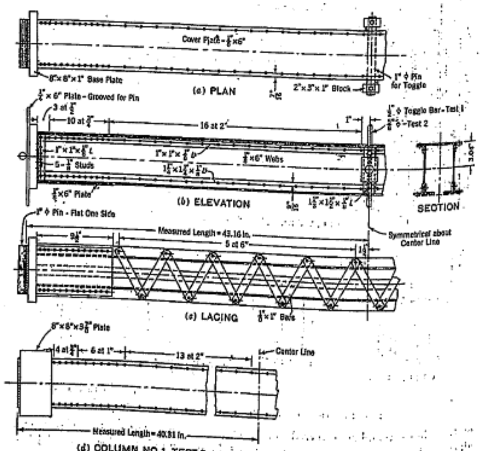


FIG. 14.—DETAILS AND DIMENSIONS, COLUMN NO. 1.

capacity of the testing machine and the difficulty of providing the necessary transverse bending force in the toggle, the model was made one-fifth the area

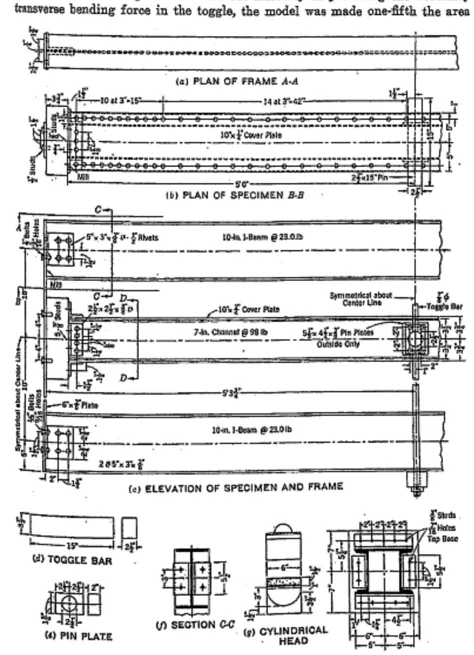
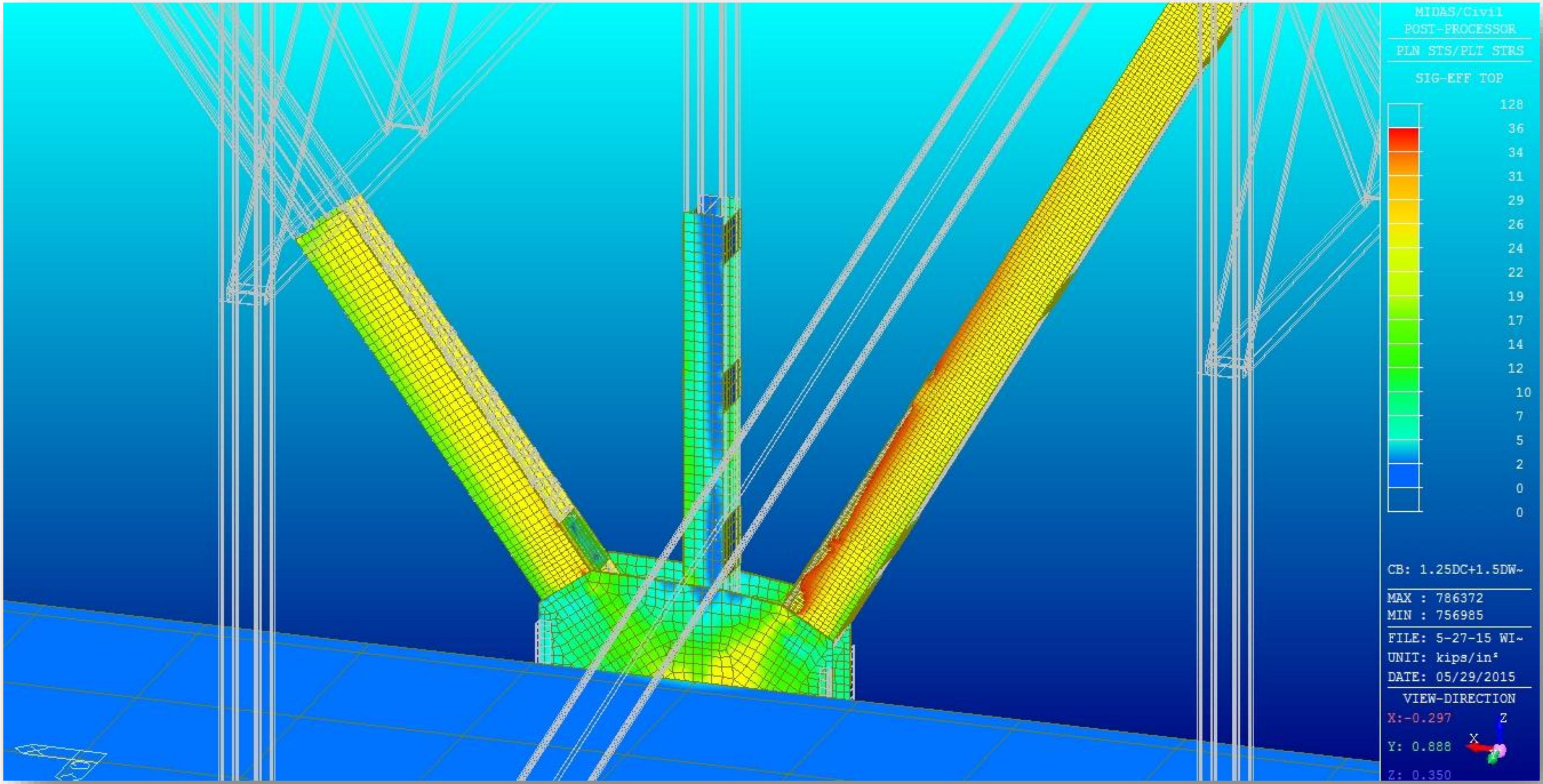


FIG. 13.—DETAILS AND DIMENSIONS OF COLUMN NO. 2.

of the chord member referred to, as closely as commercial material would allow. (The diameter of the rivets was 0.17 in., the rivet holes being drilled. The rivet spacing in the angles is the same as in the cover-plate.) For Test

J. Parcel and E. Murer: *Effect of Secondary Stresses Upon Ultimate Strength*, 1934

- General analysis and laboratory tests
- "For most bridge members…the ultimate strength is practically unaffected, even by high secondary stresses".
- "It is evident that the secondary bending was relieved by the plastic condition on the compressive face…A complete re-adjustment resulting in a nearly uniform distribution over the section was the final state of stress."


MIDAS/Civil
POST-PROCESSOR
PLN STS/PLT STRS
SIG-EFF TOP
128
36
34
31
29
26
24
22
19
17
14
12
10
7
5
2
0
0
CB: 1.25DC+1.5DW~
MAX : 786372
MIN : 756985
FILE: 5-27-15 WI~
UNIT: kips/in²
DATE: 05/29/2015
VIEW-DIRECTION
X:-0.297
Y: 0.888
Z: 0.350

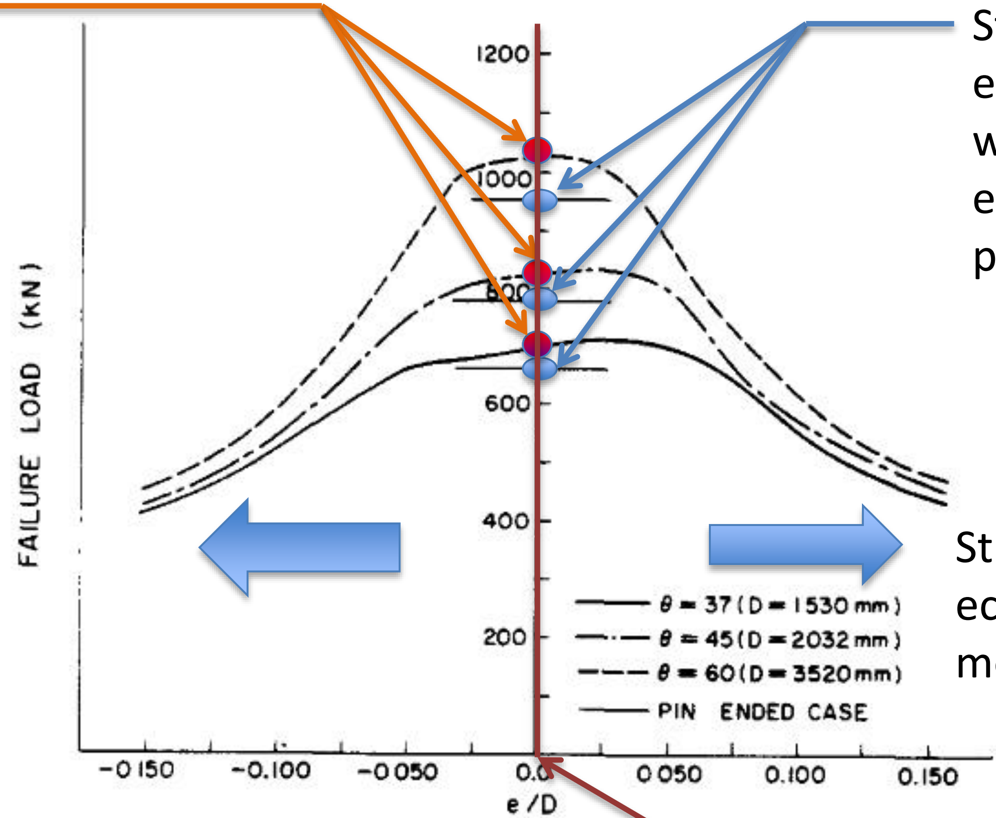
R.M Korol, et al., *On Primary and Secondary Stresses in Triangulated Trusses*, 1986

- General analysis and laboratory tests
- “It appears that Parcel and Murer’s work was taken by the profession as the final word on the subject of secondary stresses in steel trusses”.
- “Continuity at the joints added 5% and 7% to the carrying capacity of two trusses as computed by simple statics”.

R.M Korol, et al., *On Primary and Secondary Stresses in Triangulated Trusses*, 1986

Strength of fixed-ended member with no eccentricity (no primary moment)

Strength of pin-ended member with no eccentricity (no primary moment)



Strength decreases as eccentricity (primary moment) increases

Failure loads with no eccentricity

Current Practice

- "Where loads, other than the self-weight of the members and wind loads there on, are transmitted to the truss at panel points, the truss may be analyzed as a pin-connected assembly"
  -AASHTO LRFD 4.6.2.4

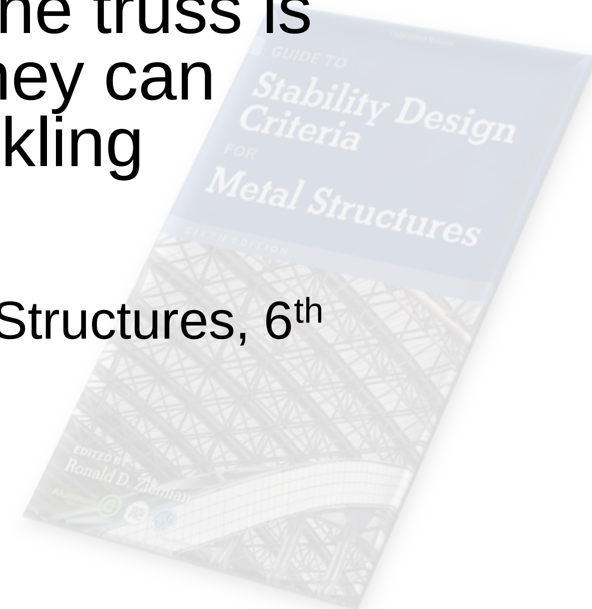
- "Stresses due to the dead load moment of the member shall be considered, as shall those caused by eccentricity of joints or working lines. Secondary stress due to truss distortion or floorbeam deflection need not be considered in any member whose width measured parallel to the plane of distortion is less than one-tenth of its length."
  -AASHTO LRFD 6.14.2.3

Current Practice

- "[Secondary stresses] have little effect on the buckling strength (and tensile strength) of truss members.

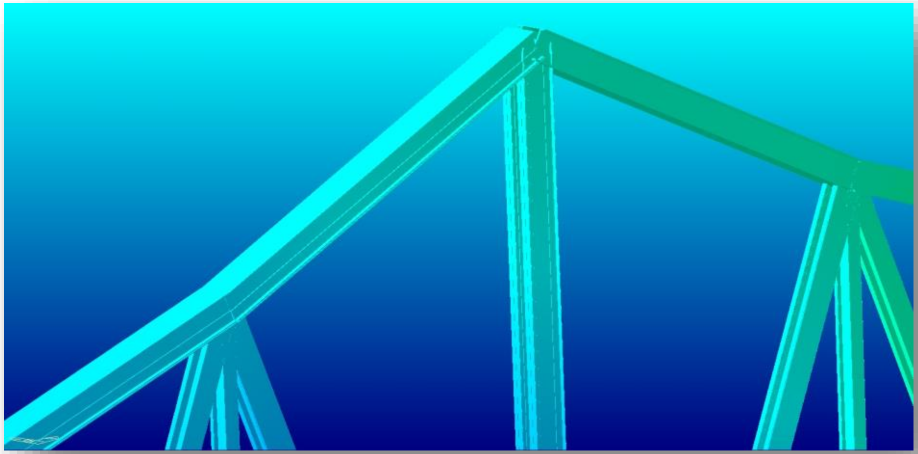
  Because of local yielding of extreme fibers of the members near the joints, the secondary moments gradually dissipate as the truss is loaded to its ultimate strength. They can therefore be neglected in the buckling analysis"

  -Guide to Stability Design Criteria of Metal Structures, 6th Edition, Page 50

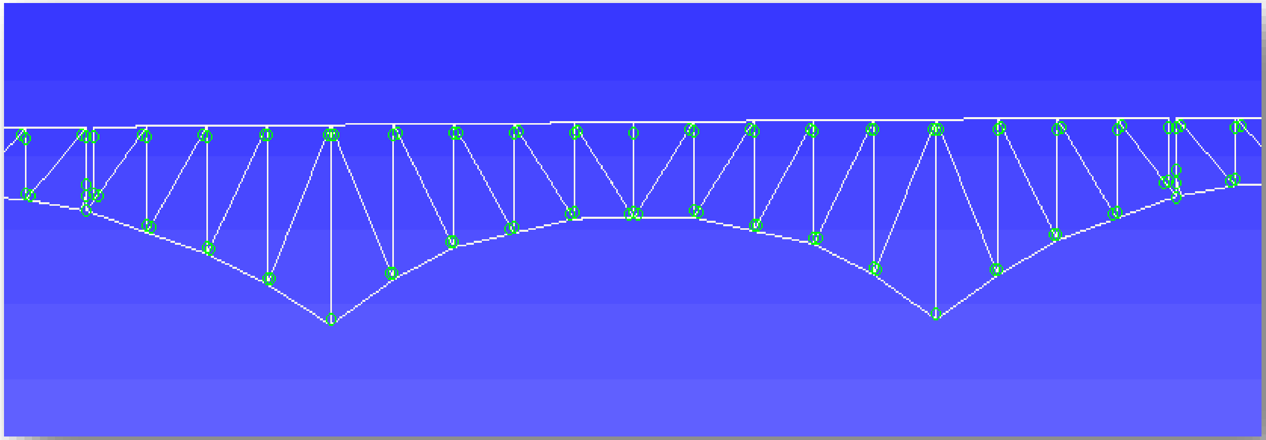


Should trusses be modeled as pinned or fixed?

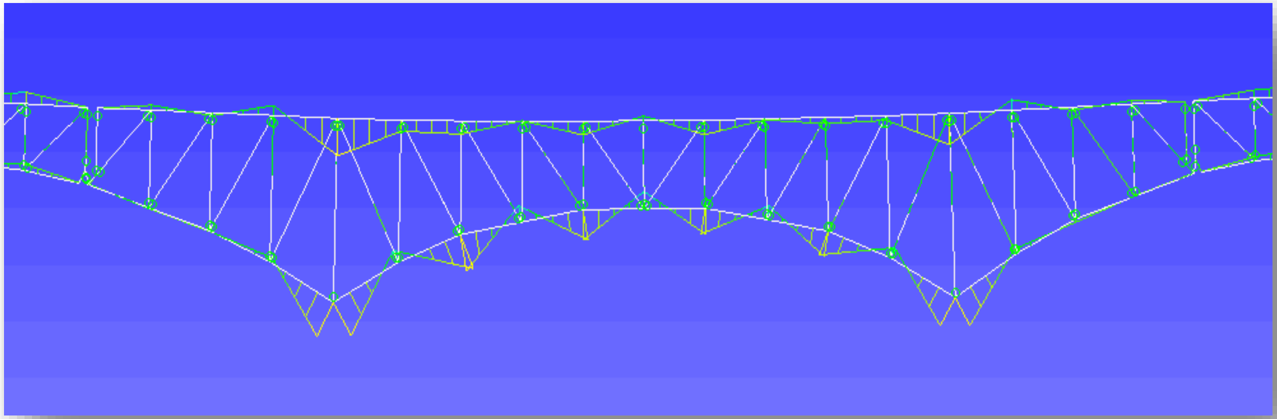
- Pinned = secondary moments neglected
- Fixed = secondary moments included
- Recommend pinned for strength limit state, fixed for fatigue and service limit state
- Include primary moments for all limit states



Member Length Adjustment for Camber

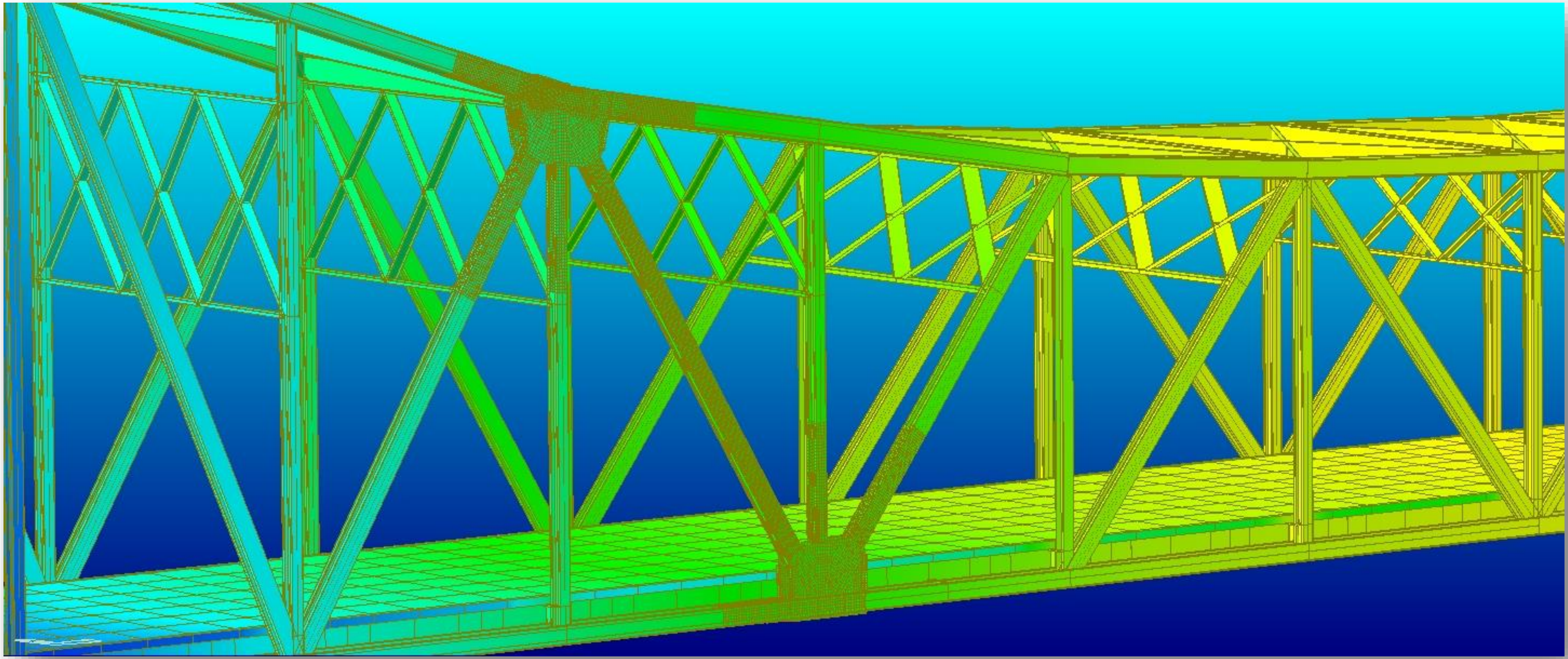


Truss under no load in geometric position prior to camber length adjustment of members



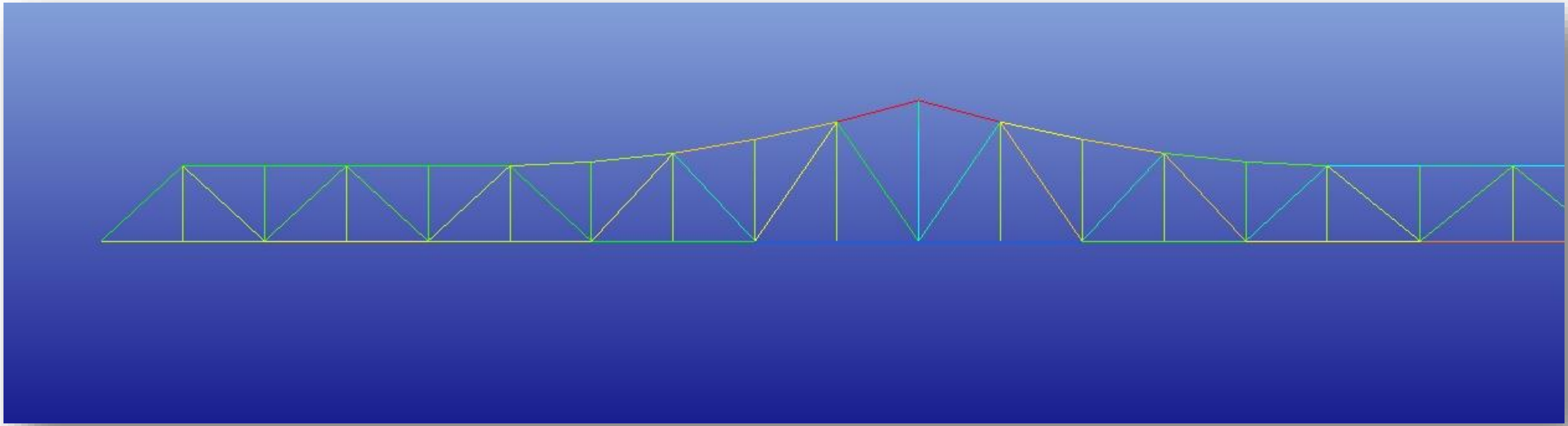
Truss after camber length adjustment of members, showing in-plane moments

How complex should the model be?



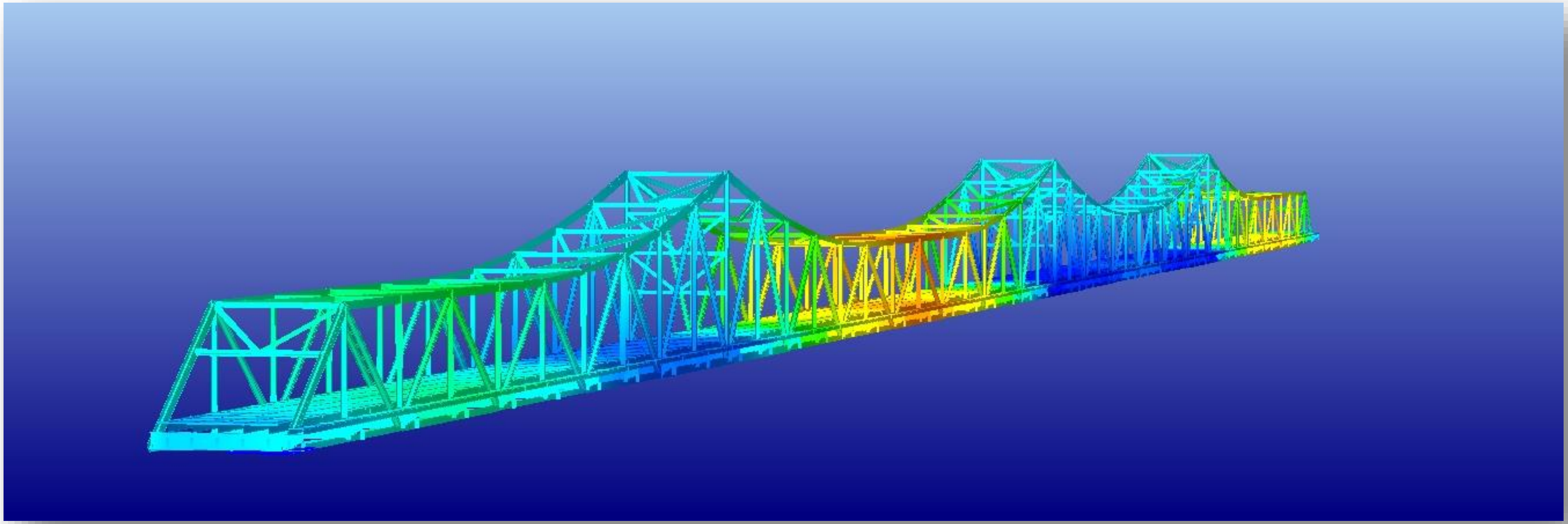
2D Modeling

- Conventional bridges
- Main member dead and live load axial forces
- Use lever rule for live load distribution

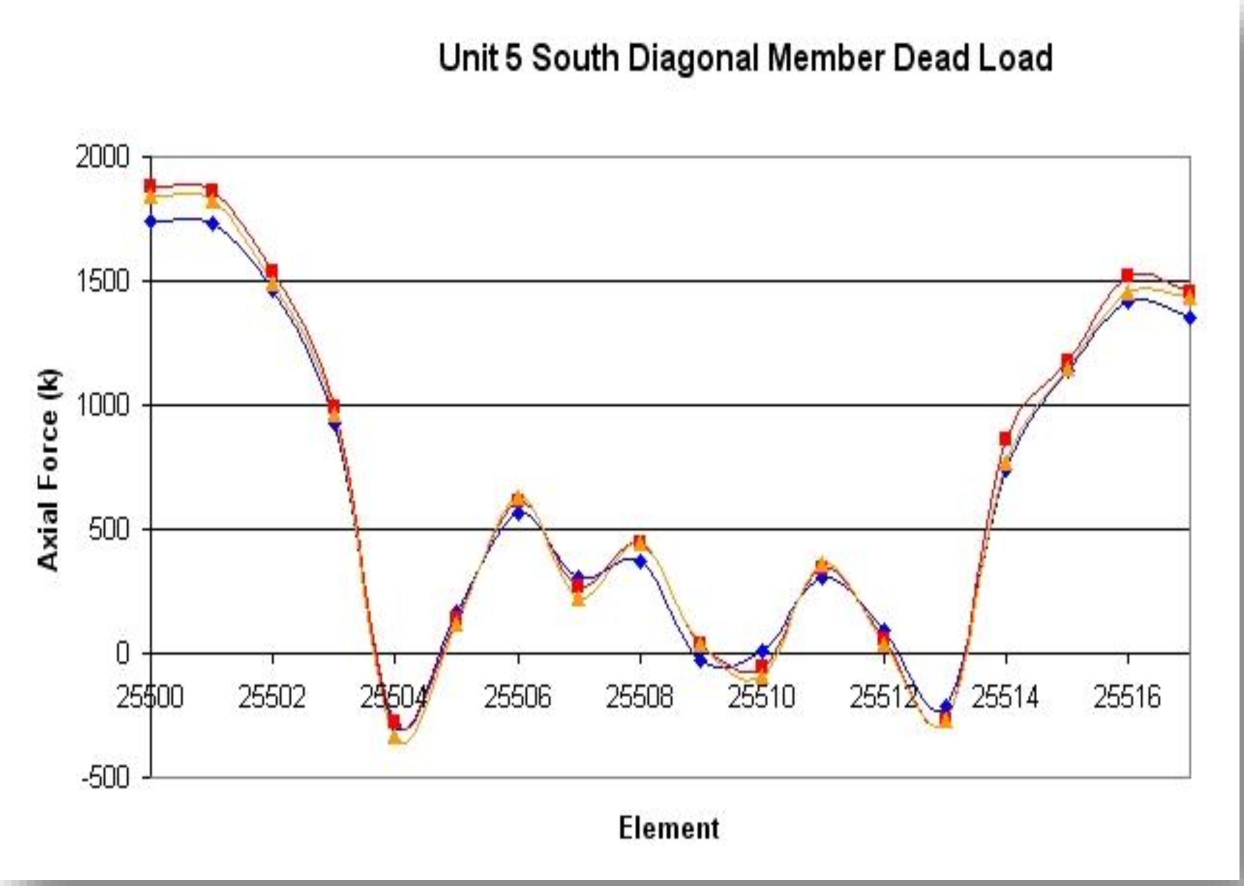


3D Modeling

- Wind analysis
- Floor systems
- Changes to structural configuration of original bridge
- Detailed FEA of specific regions



Axial Force Comparisons

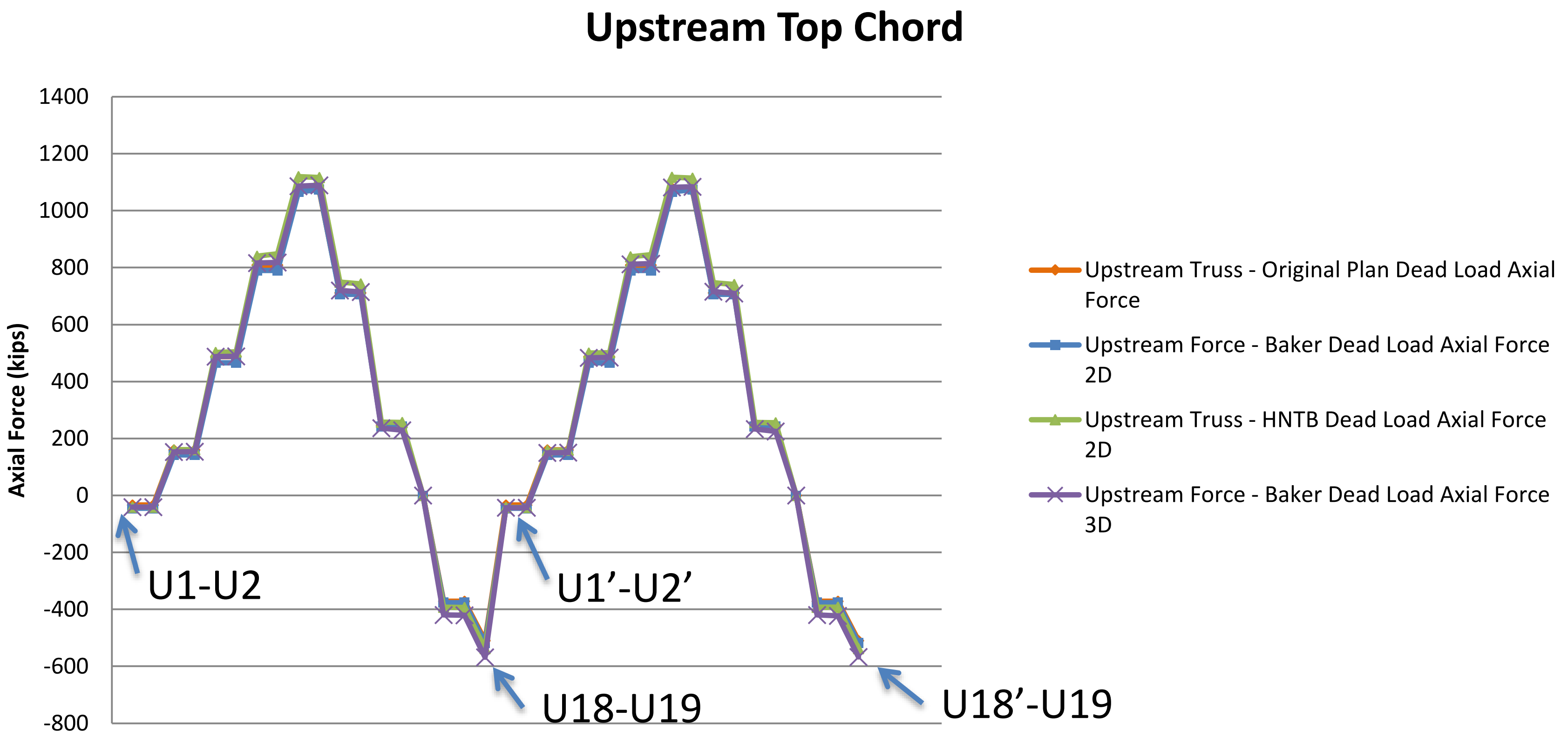


Baker

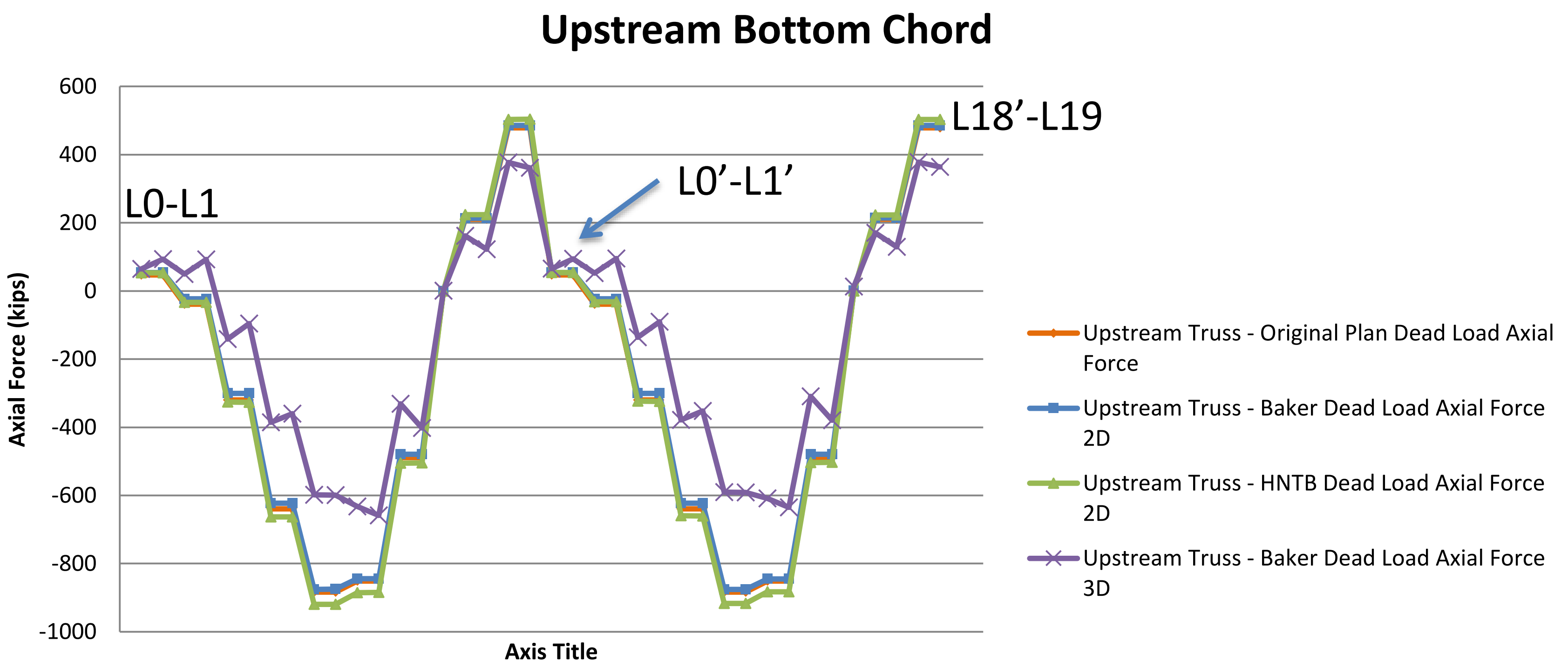
Previous 2D Rating

Original
Plan Forces

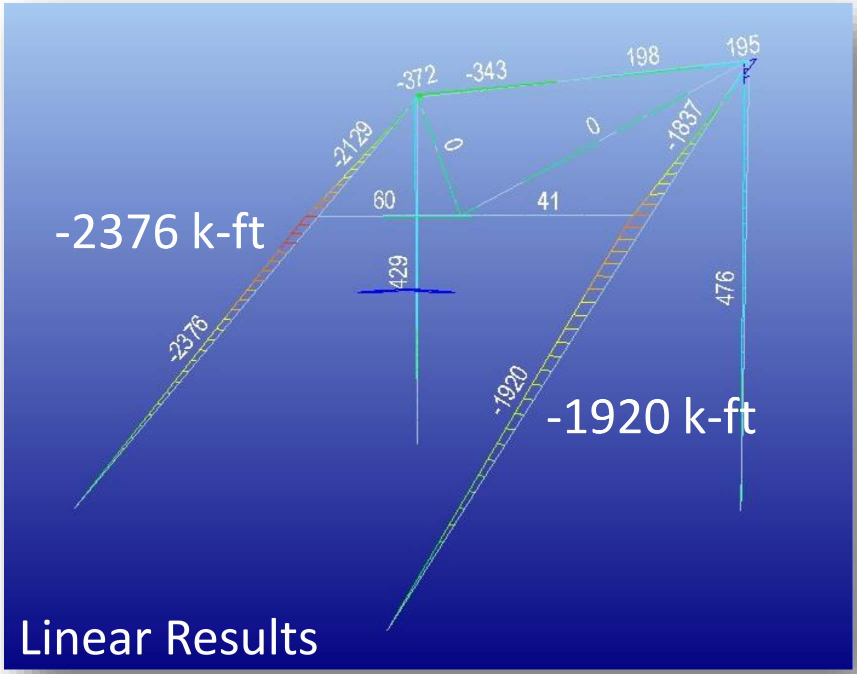
Axial Force Comparisons

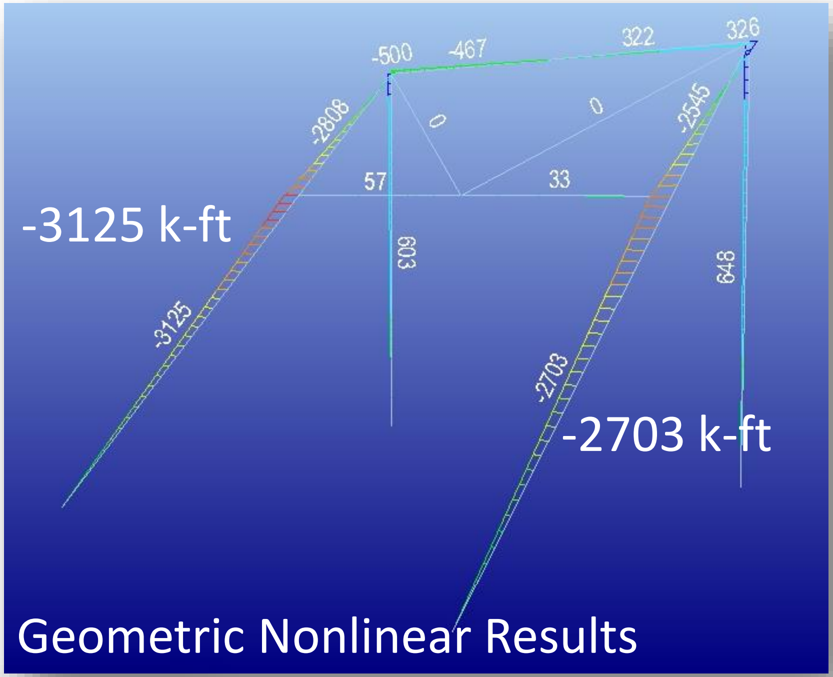


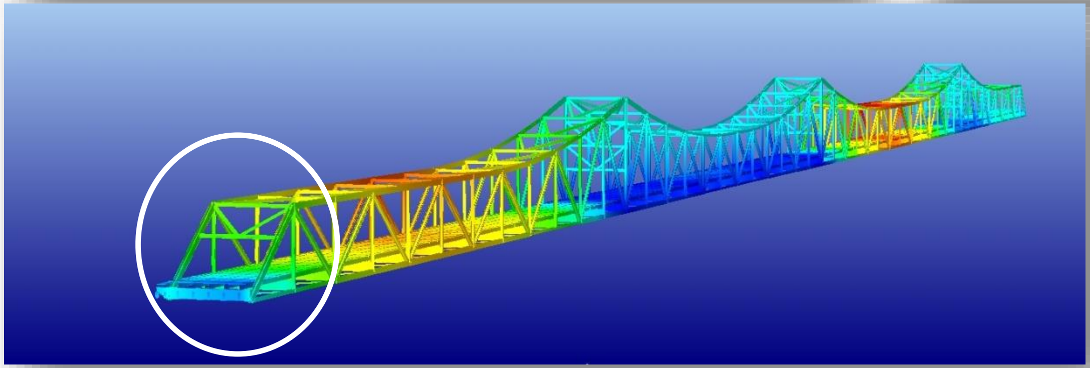
Axial Force Comparisons



Wind Load Moments



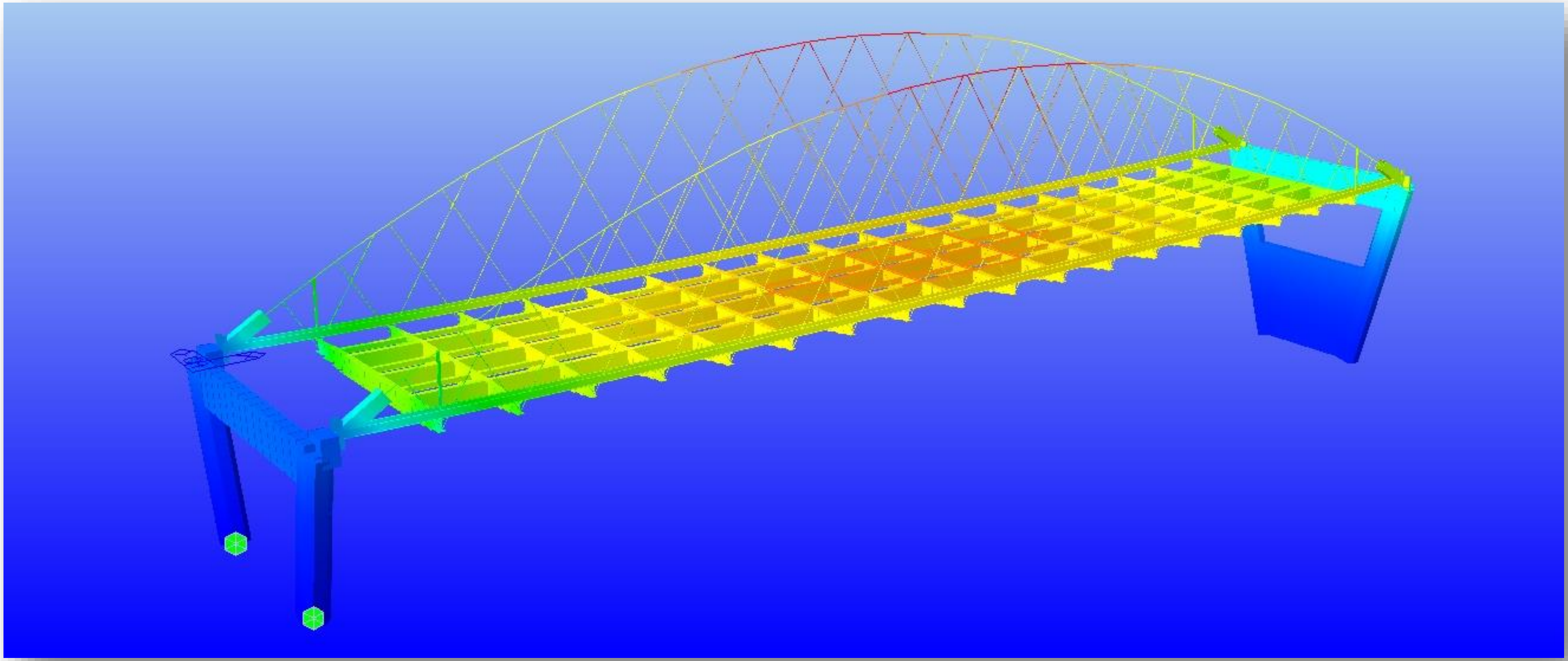




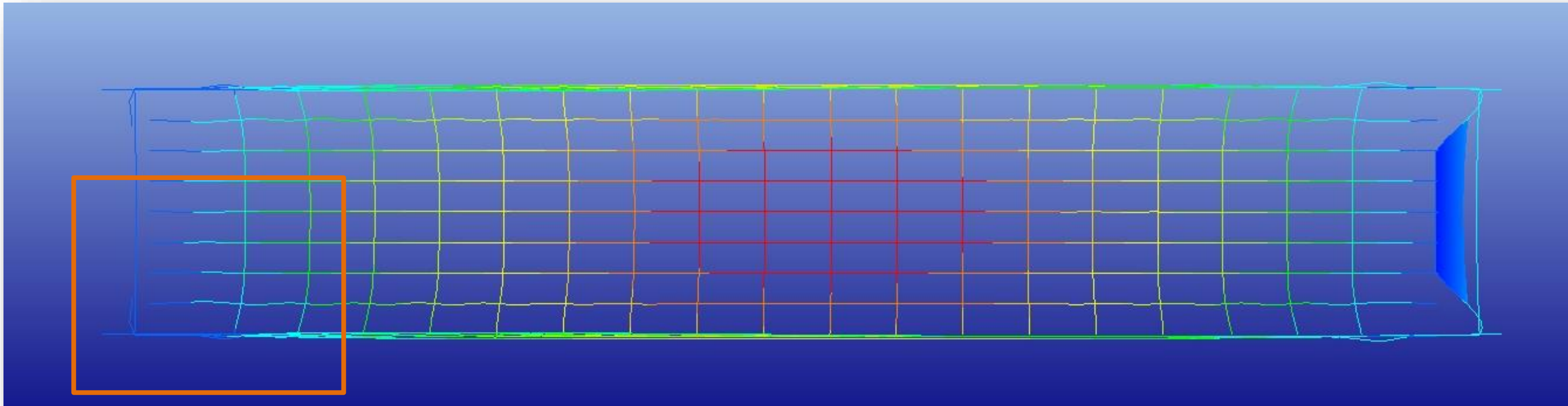
-3125 / -2376 = 1.32

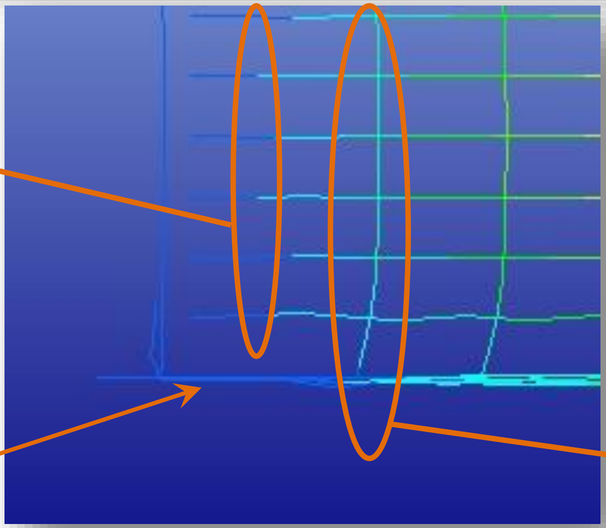
-2703 / -1920 = 1.41

Floor Systems in 3D

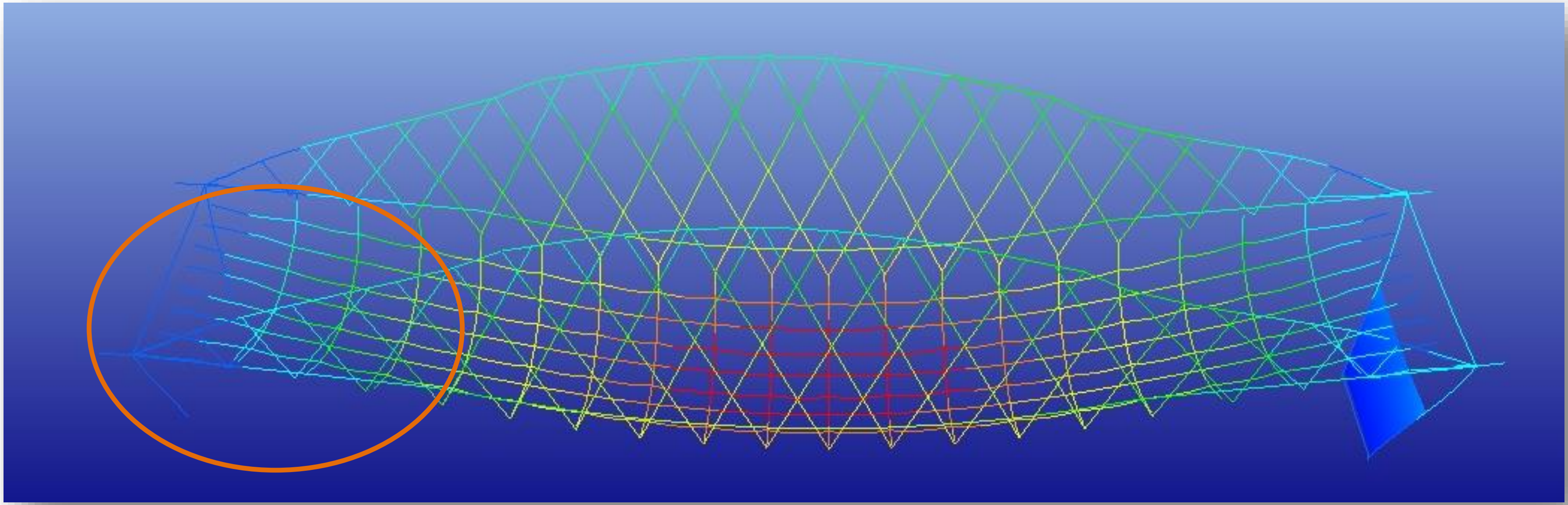


Floor System Deformations



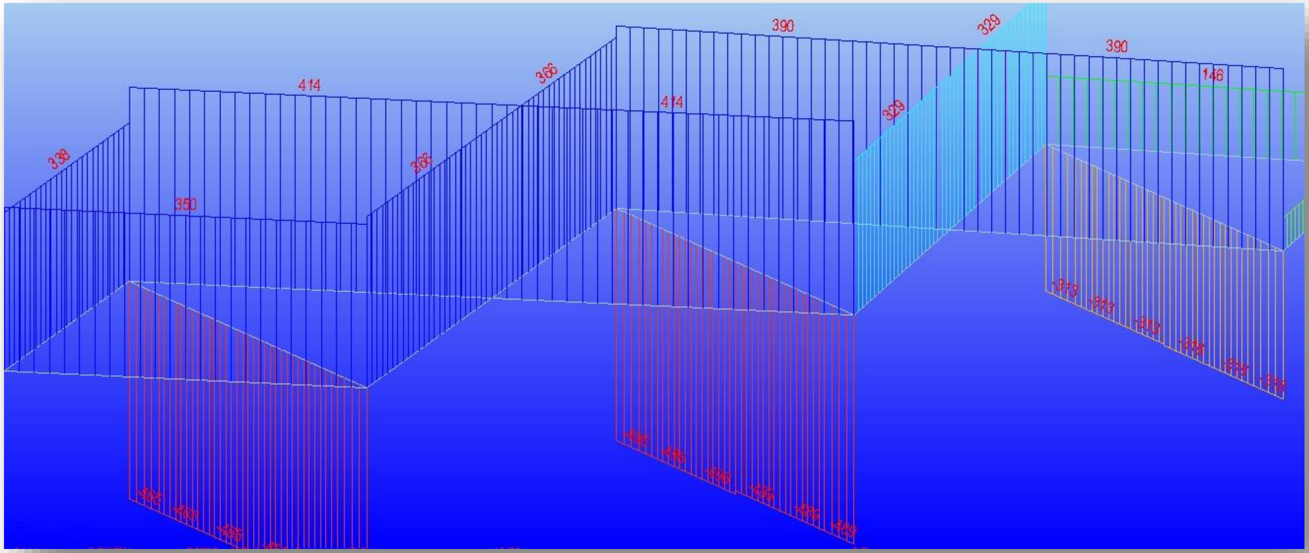


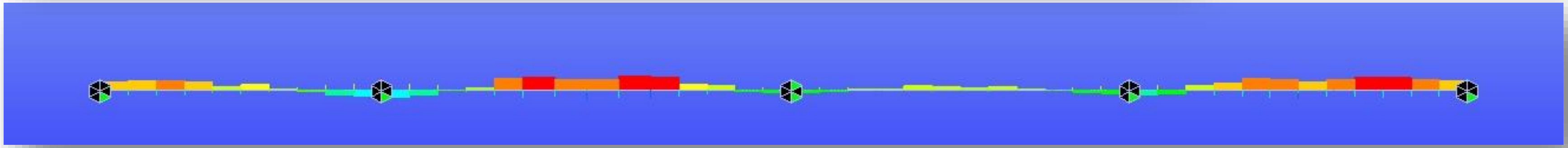
Floor System Deformations



Secondary Members in 3D Models

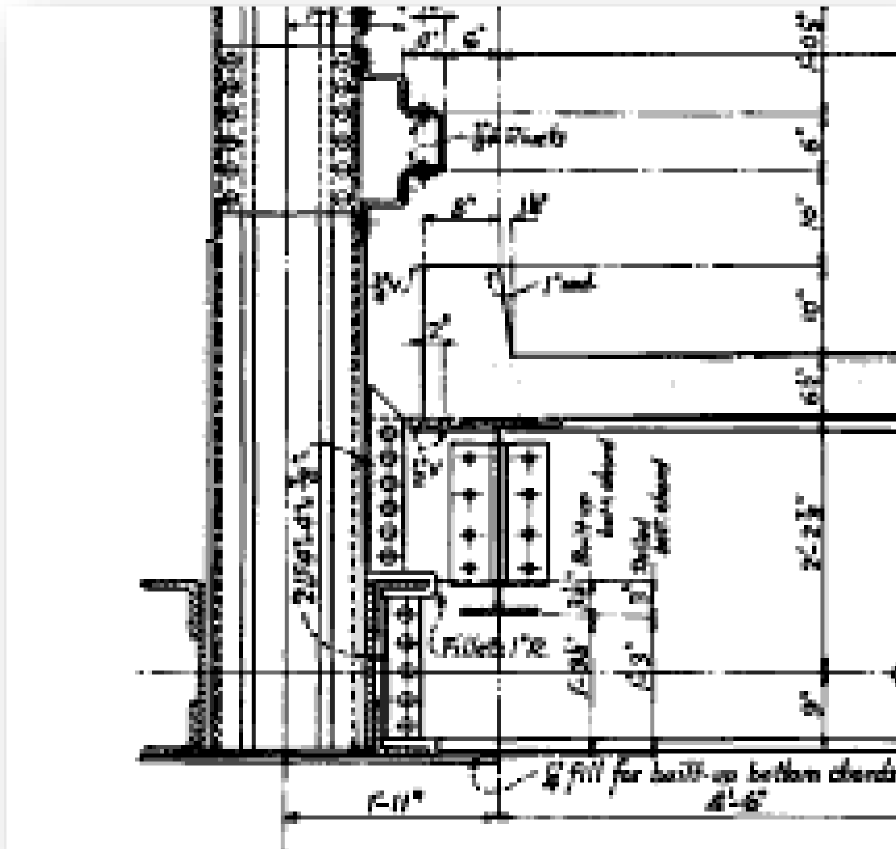
- Laterals and floorbeams supplement top and bottom chords



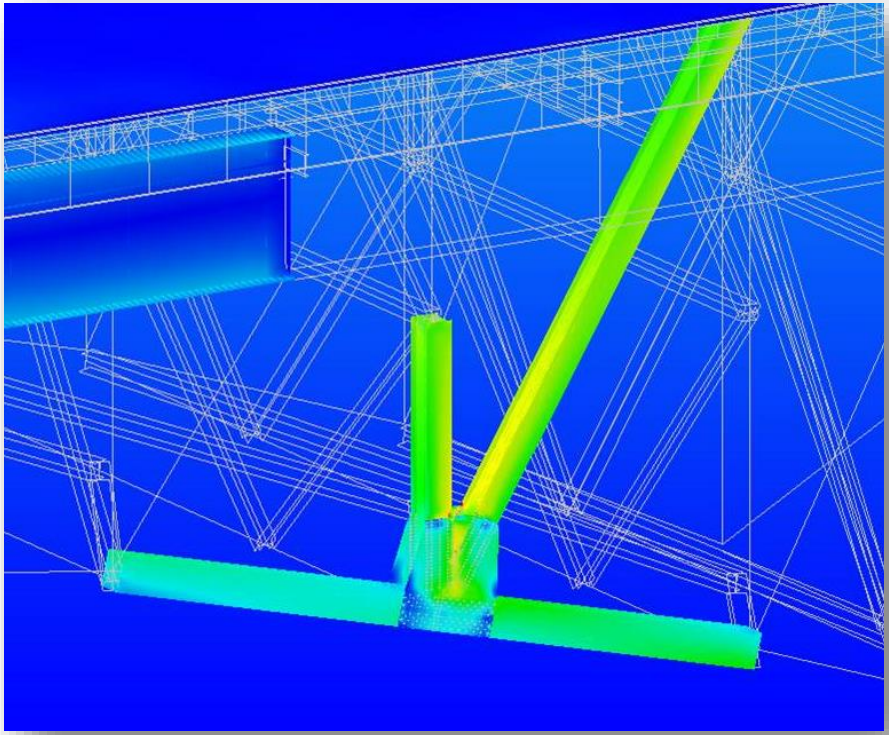


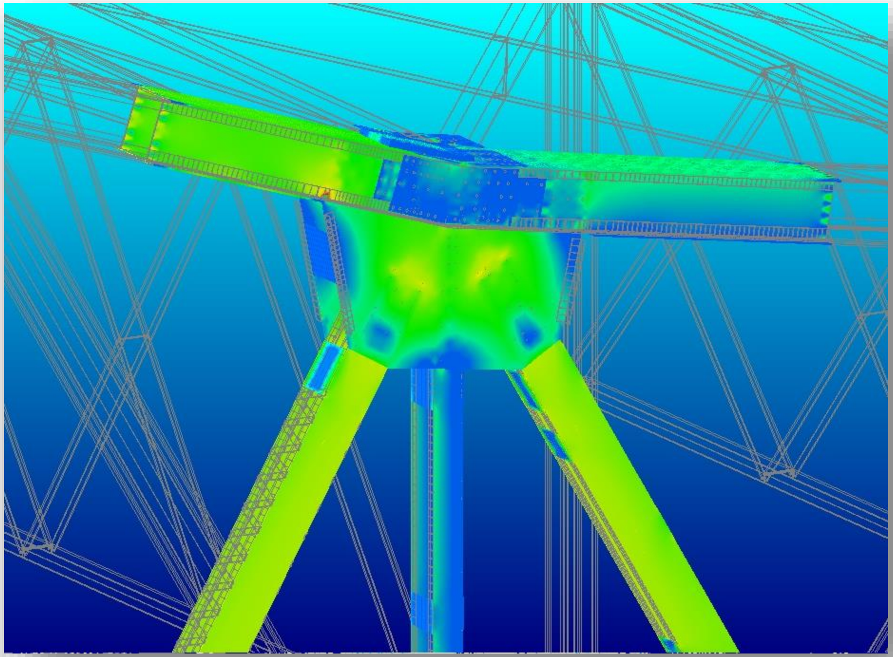
Boundary Conditions in 3D Models

- Consider ability of actual connections to transfer moments



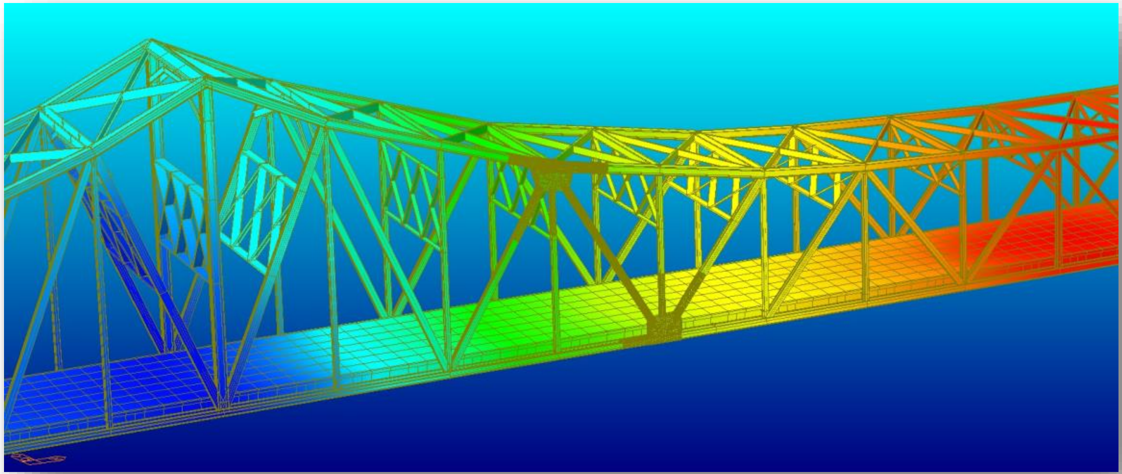
Detailed Modeling of Truss Bridges

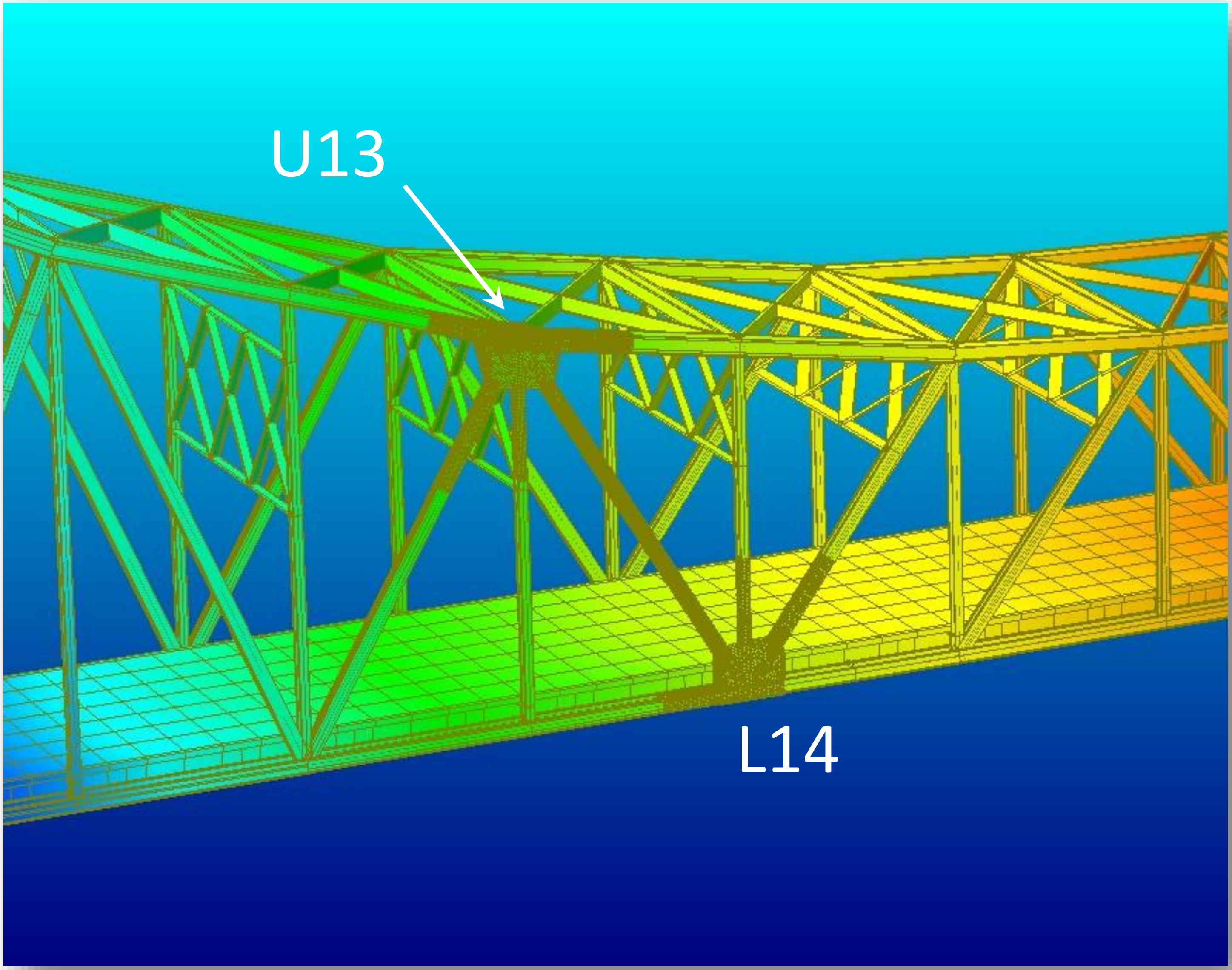




Detailed Modeling Case Study: Winona Bridge

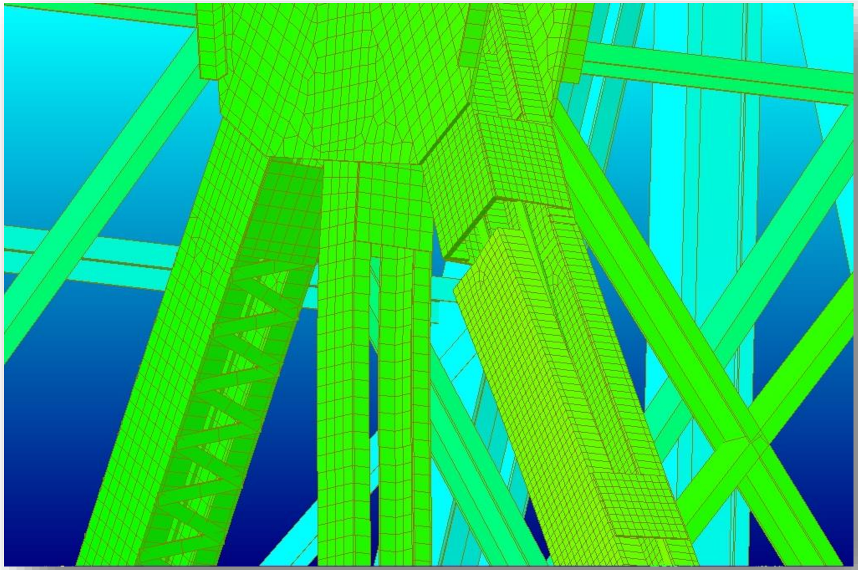
- U13-L14, adjoining connections and members modeled with shell elements
- High-strength bars for U13-L14 modeled with truss elements
- Rivets modeled with beam elements and multilinear links

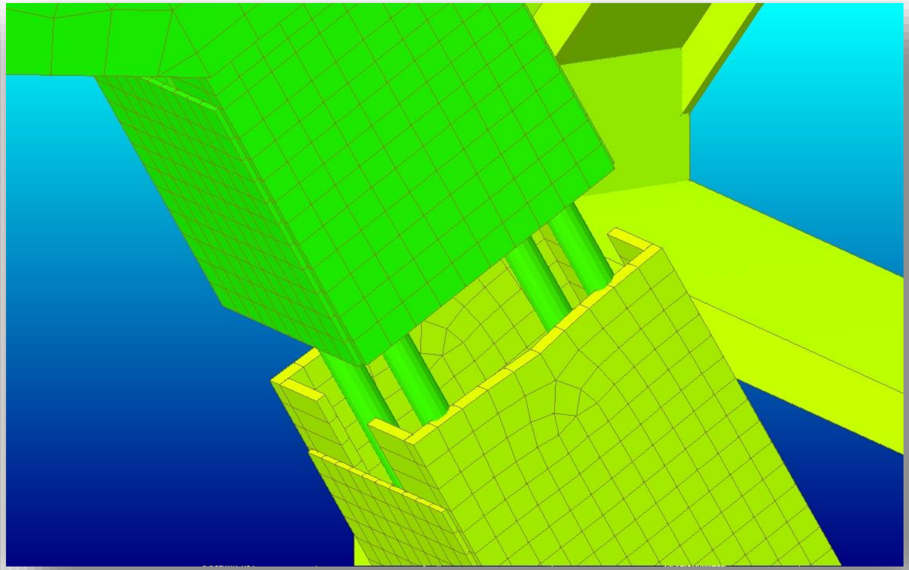


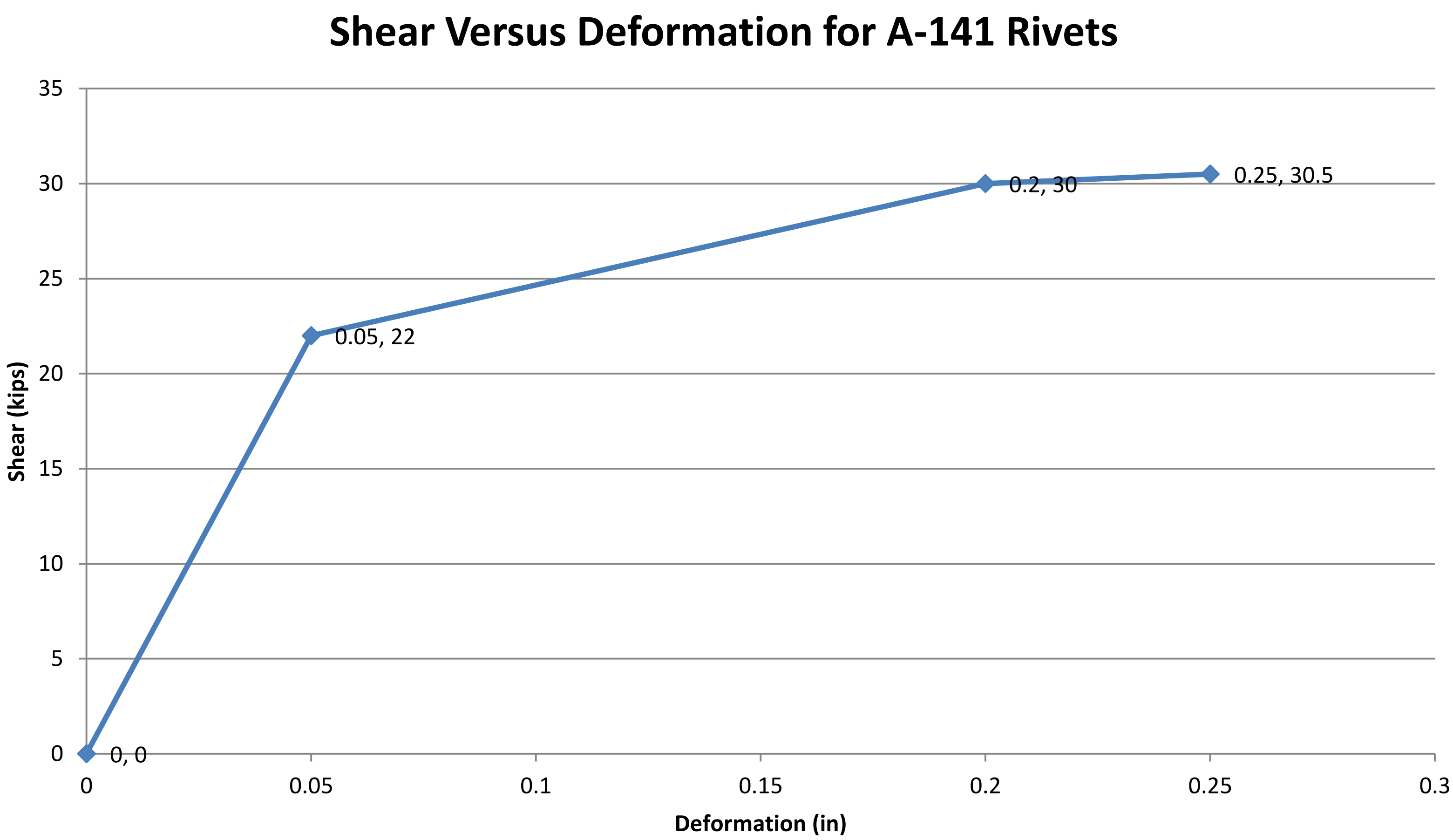

U13
L14

- Questions:
  - What are the demand/capacity ratios in adjacent members after one channel fractures?
  - Will there be large differential displacements between the end of the member and the connection?
  - Will rivets in the gusset plates fail due to large differential displacements between the member and connection?
  - Will the high-strength bars engage to carry load after one channel fractures?

- Three analysis conditions:
  - 1.25DC + 1.5DW + 1.30(90% Double Truck LL + IM) [Extreme III] before fracture
  - Extreme III after one U13-L14 channel fractures
  - Extreme III after both channels fracture





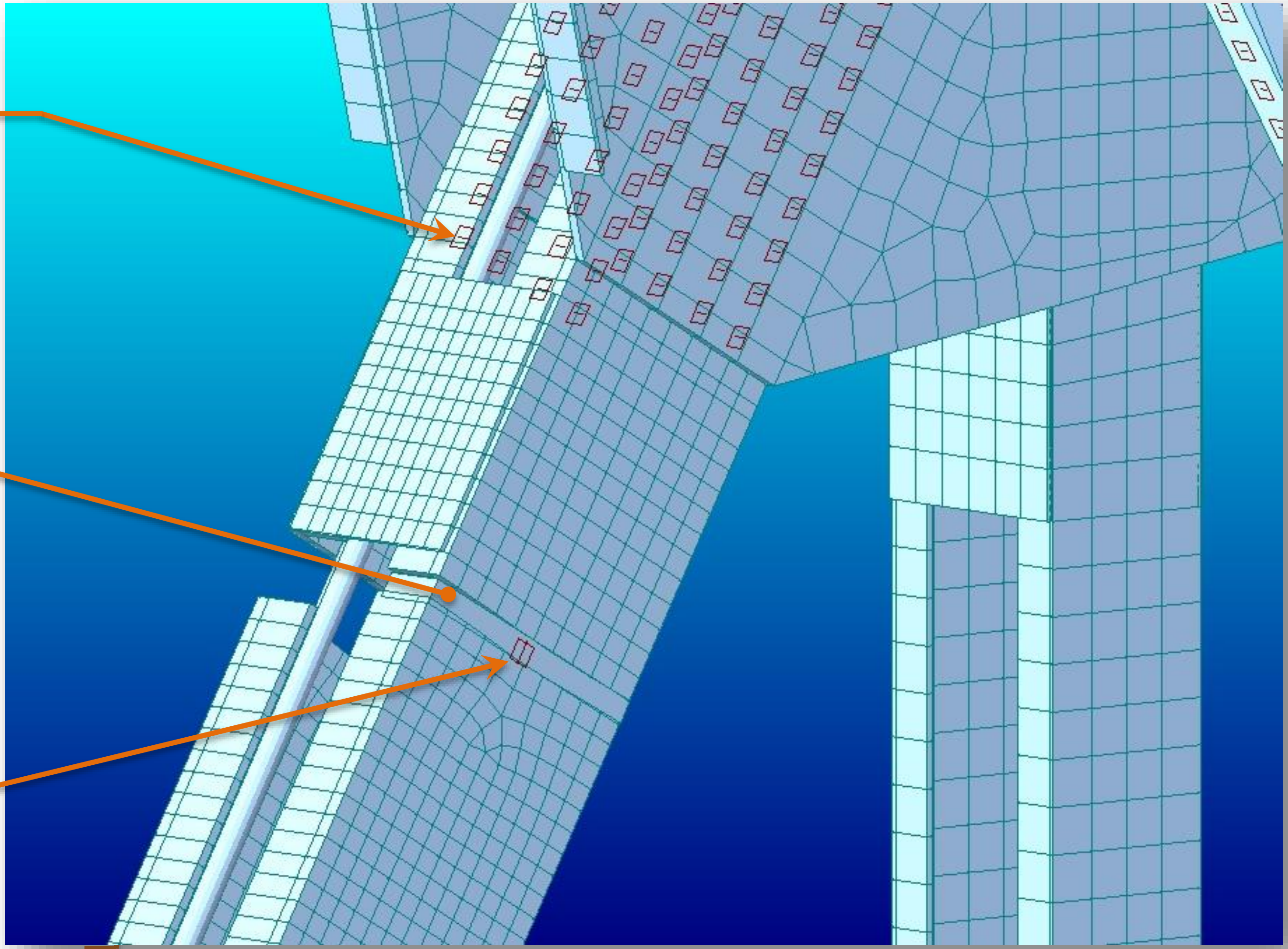


Source: *The Static Strength of Rivets Subjected to Combined Tension and Shear*, Munse, et. al., University of Illinois Bulletin, 1956

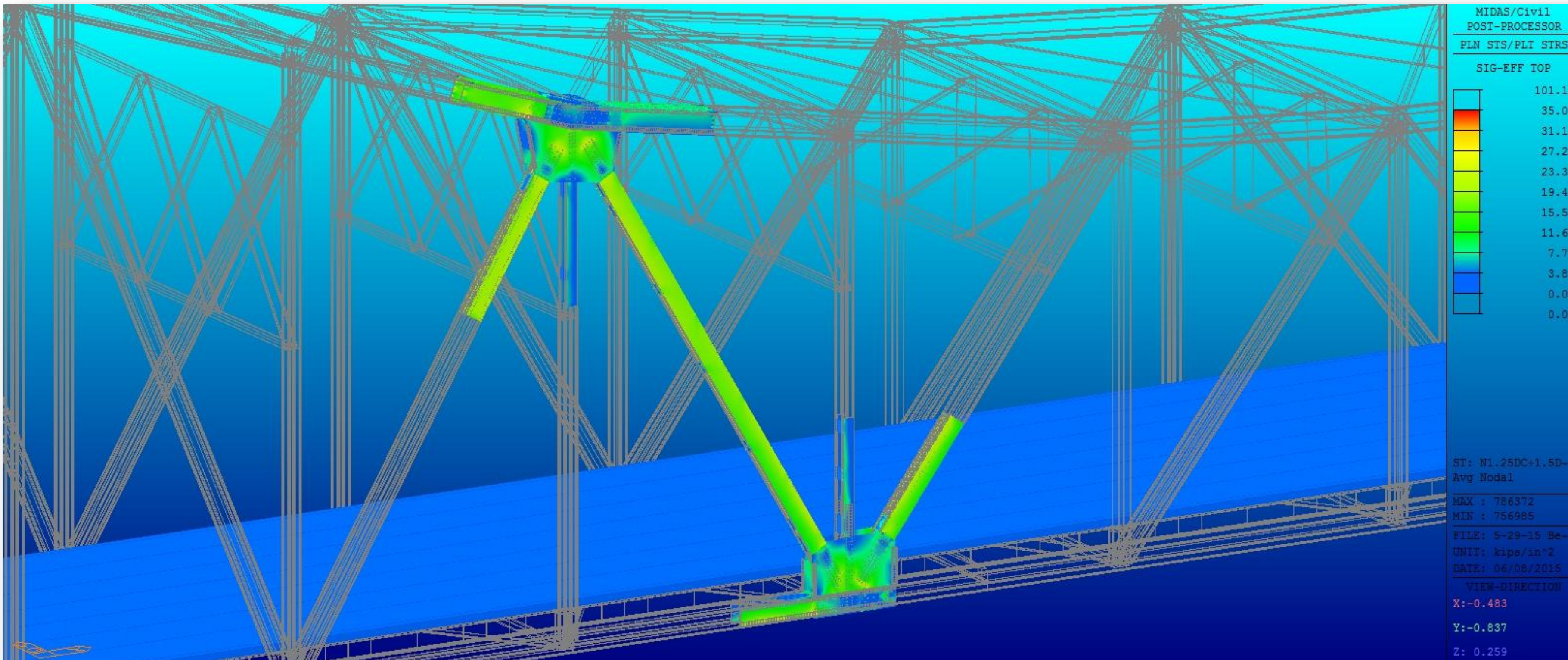
Multilinear elastic link

Element with zero axial stiffness

Multilinear axial stiffness spring



Extreme III Stresses Before Fracture



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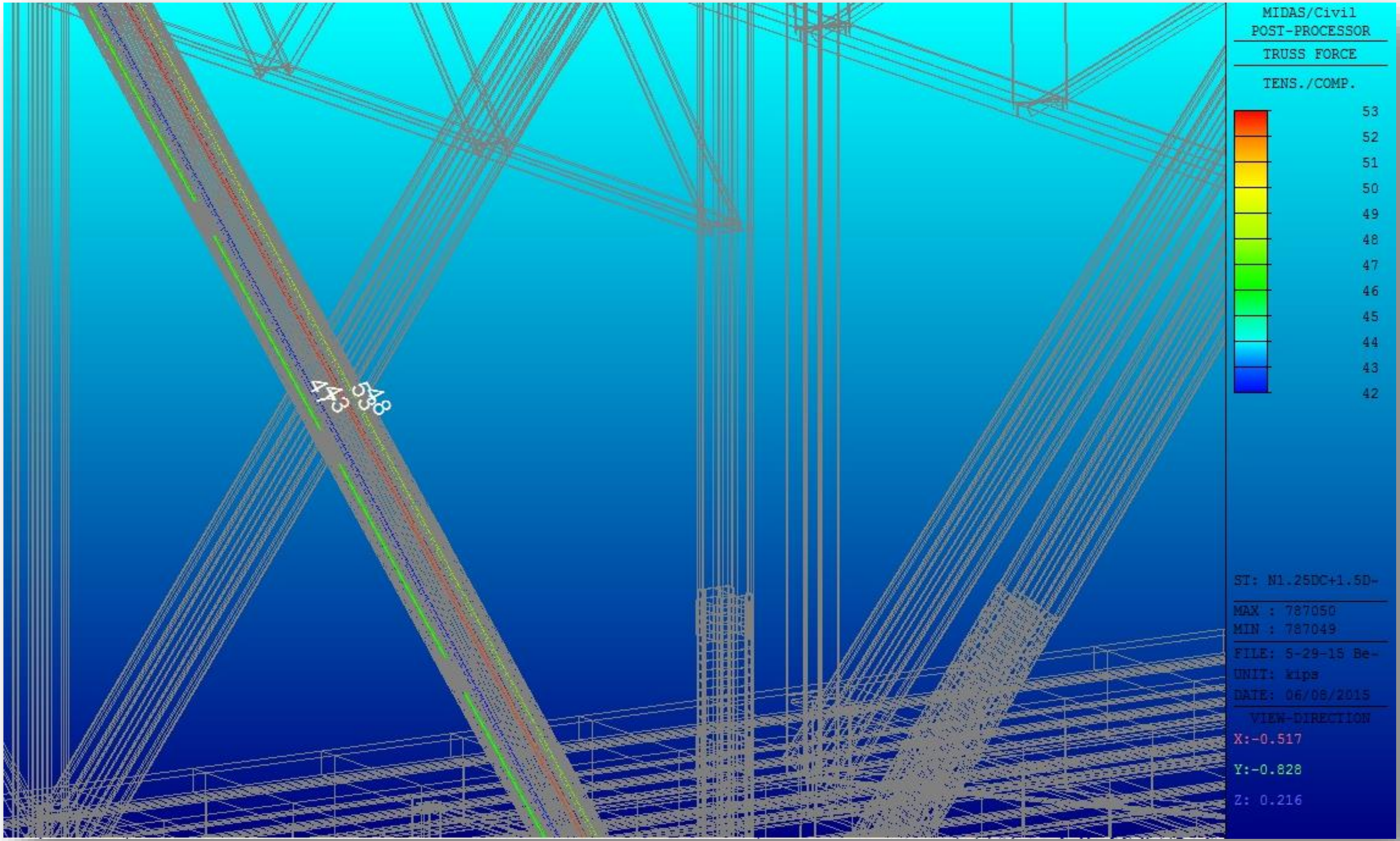
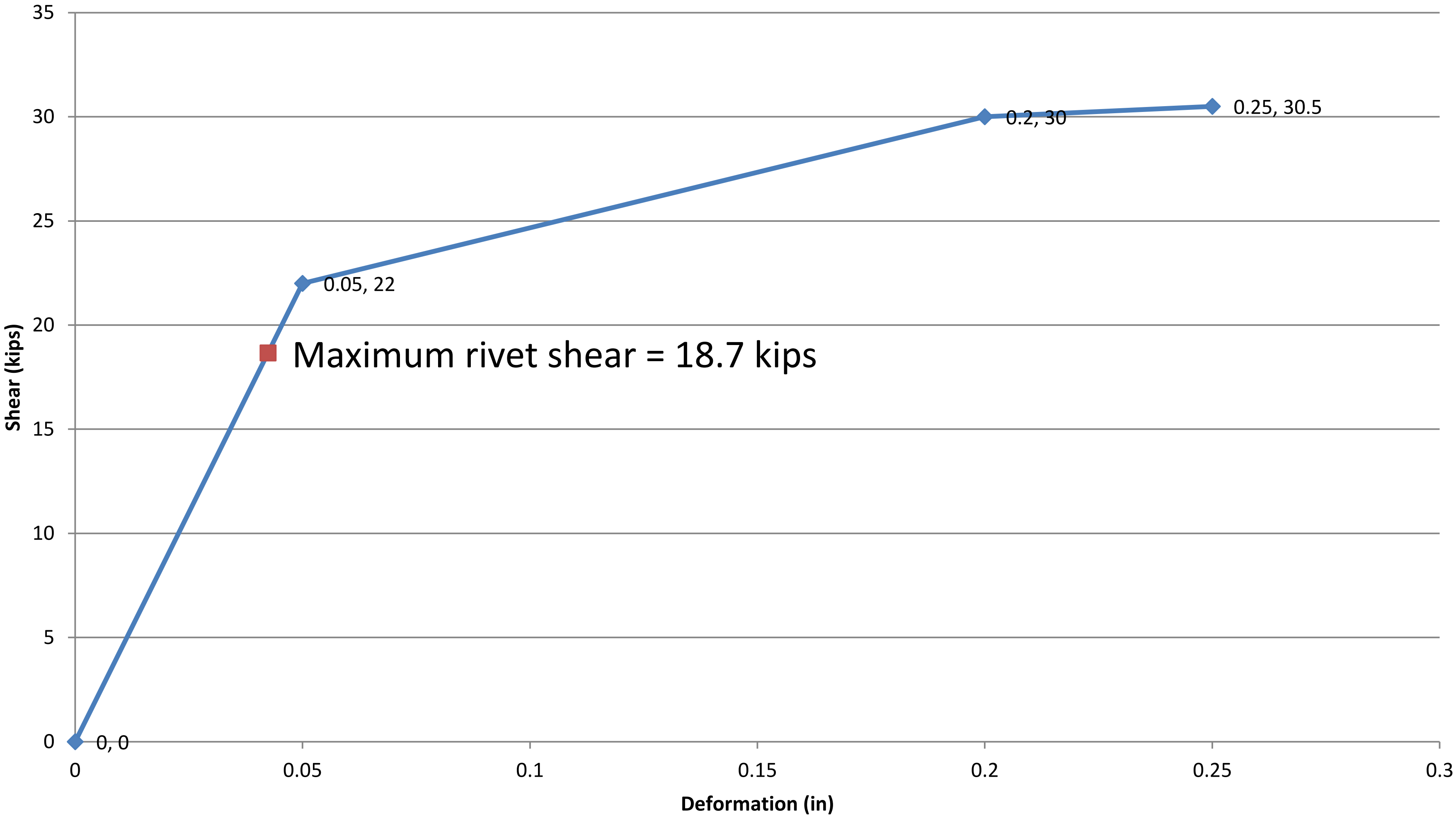
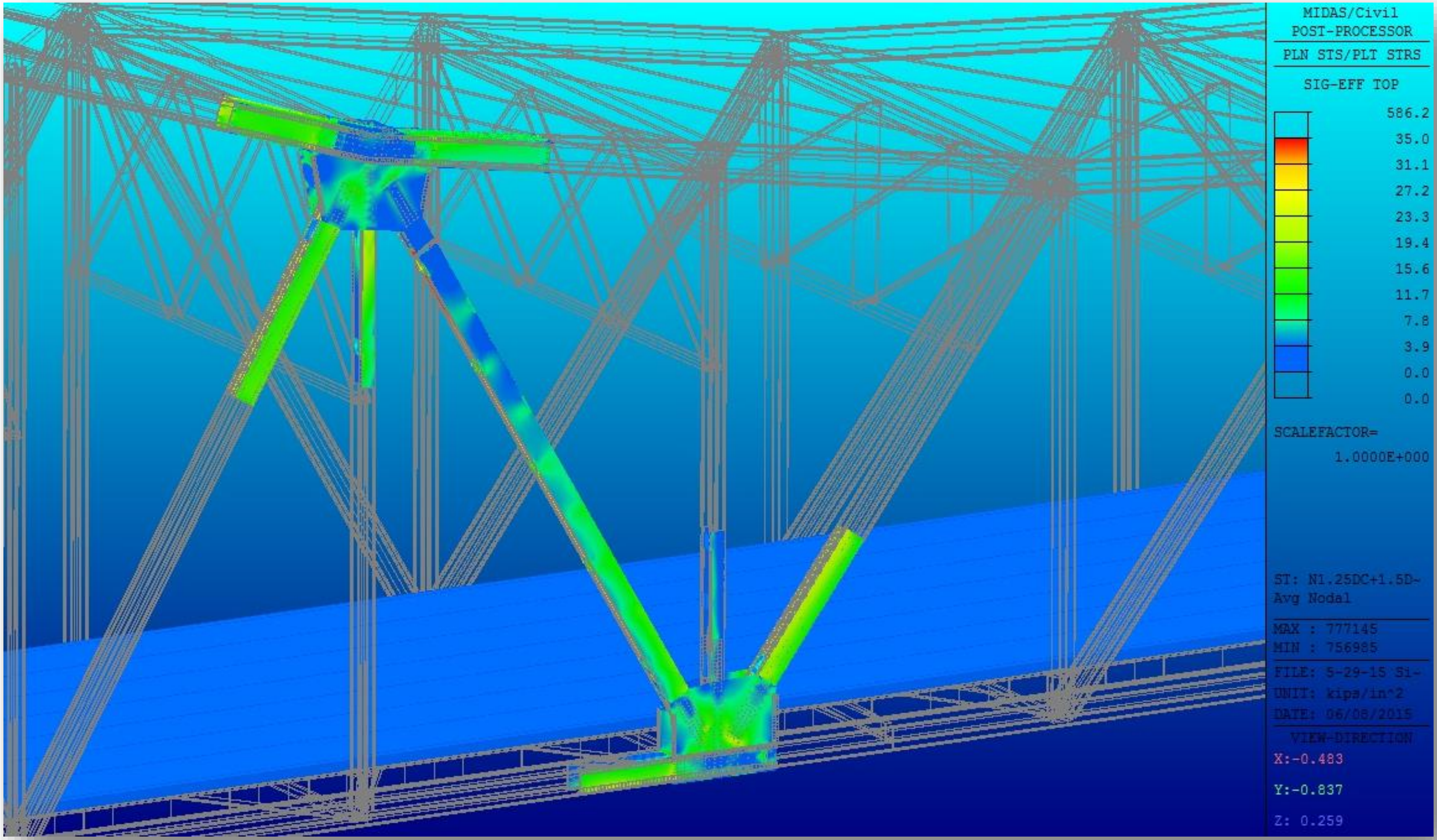
High-Strength Bar Force Before Fracture



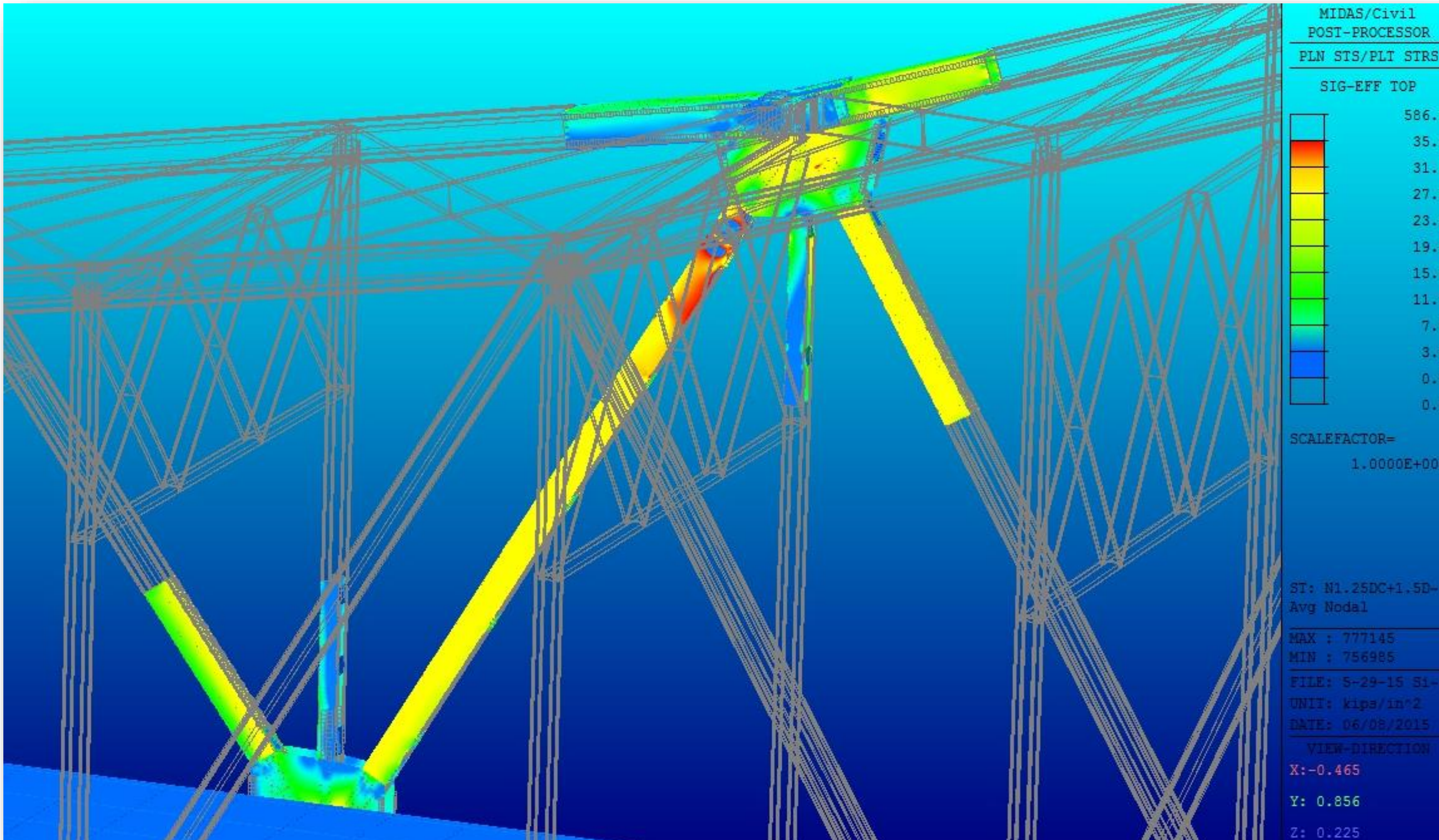
Rivet Shear Versus Deformation Before Fracture



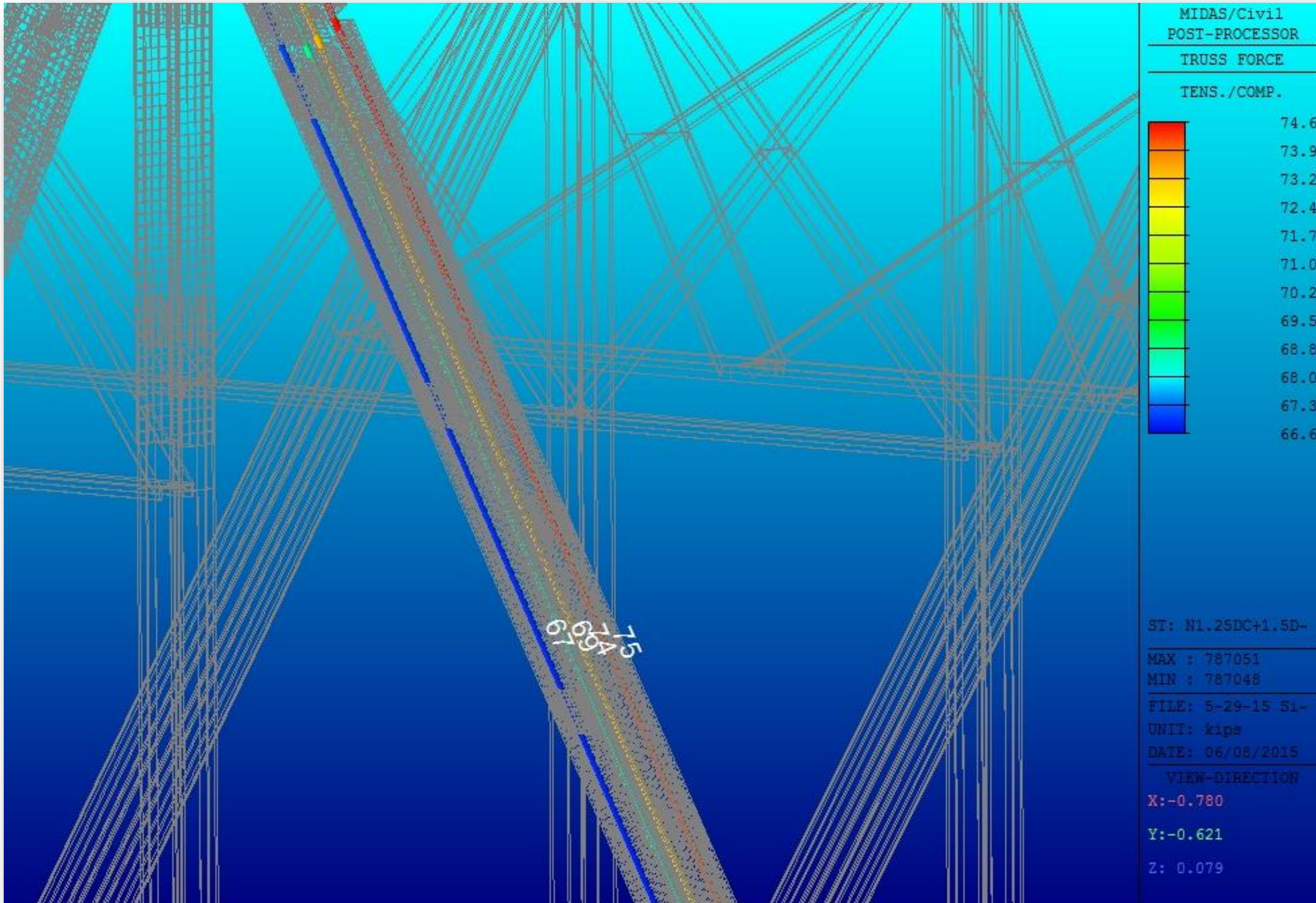
Extreme III Stresses After Fracture



Extreme III Stresses After Fracture



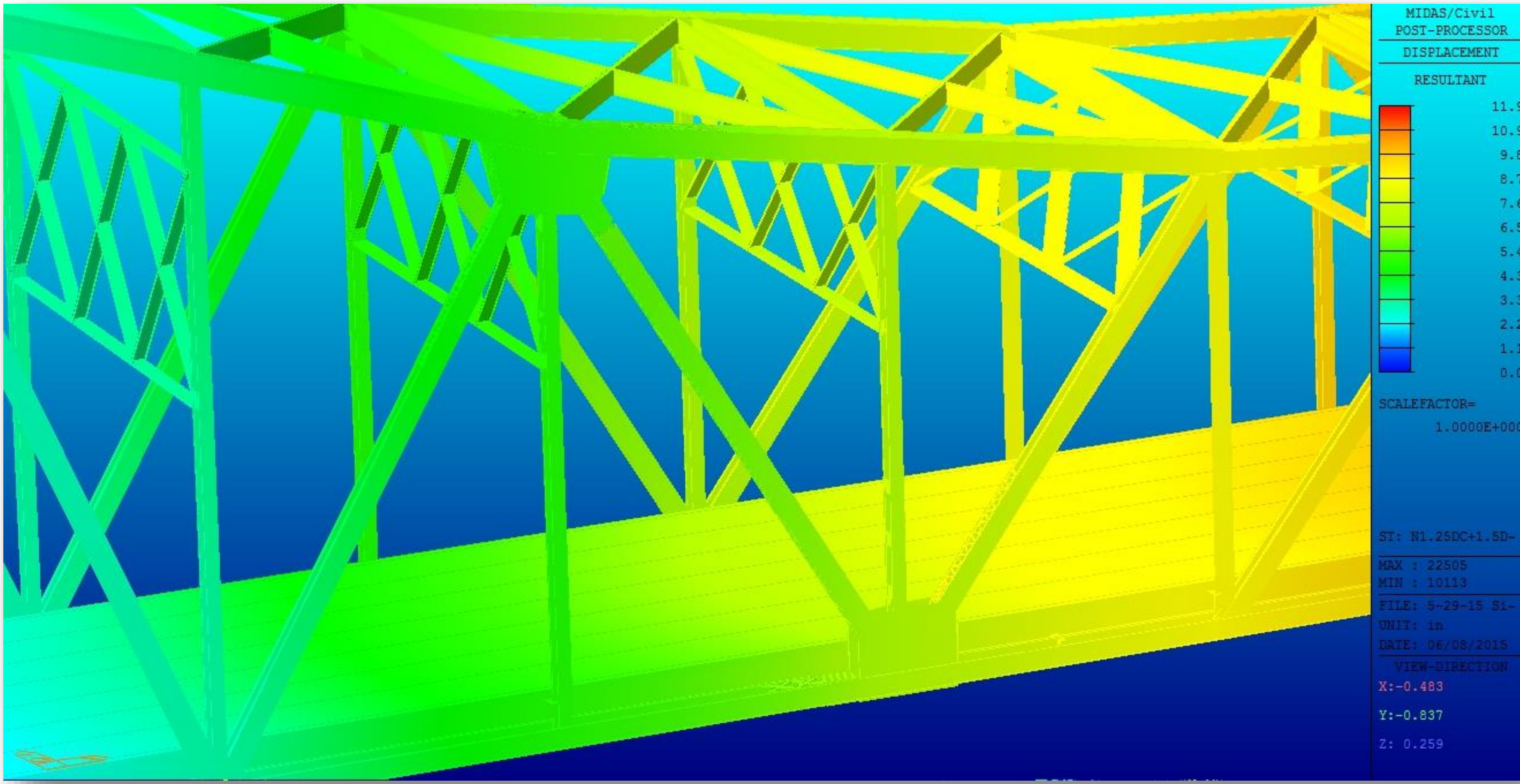
High-Strength Bar Force After Fracture



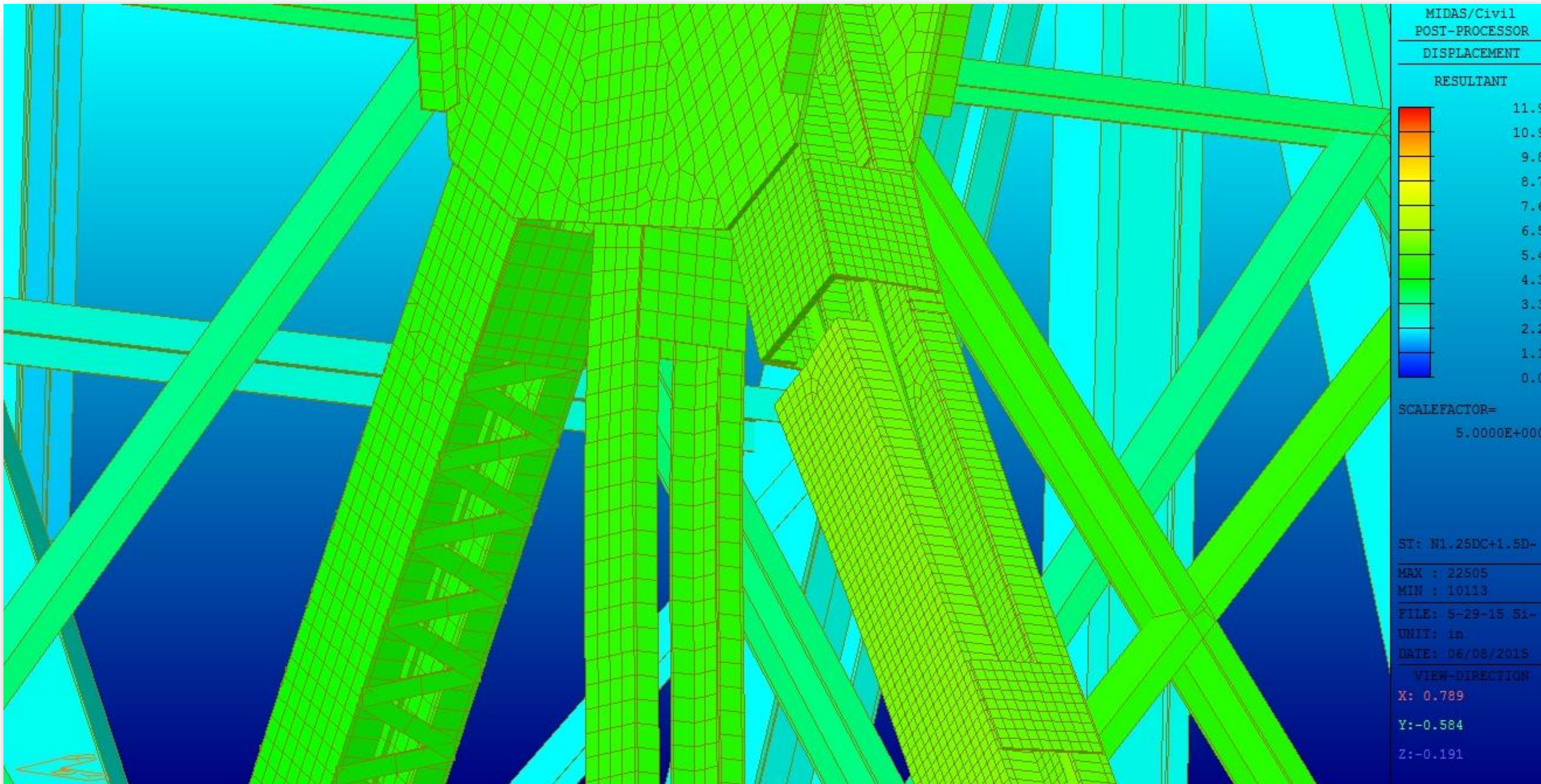
Axial Force Distribution After Fracture

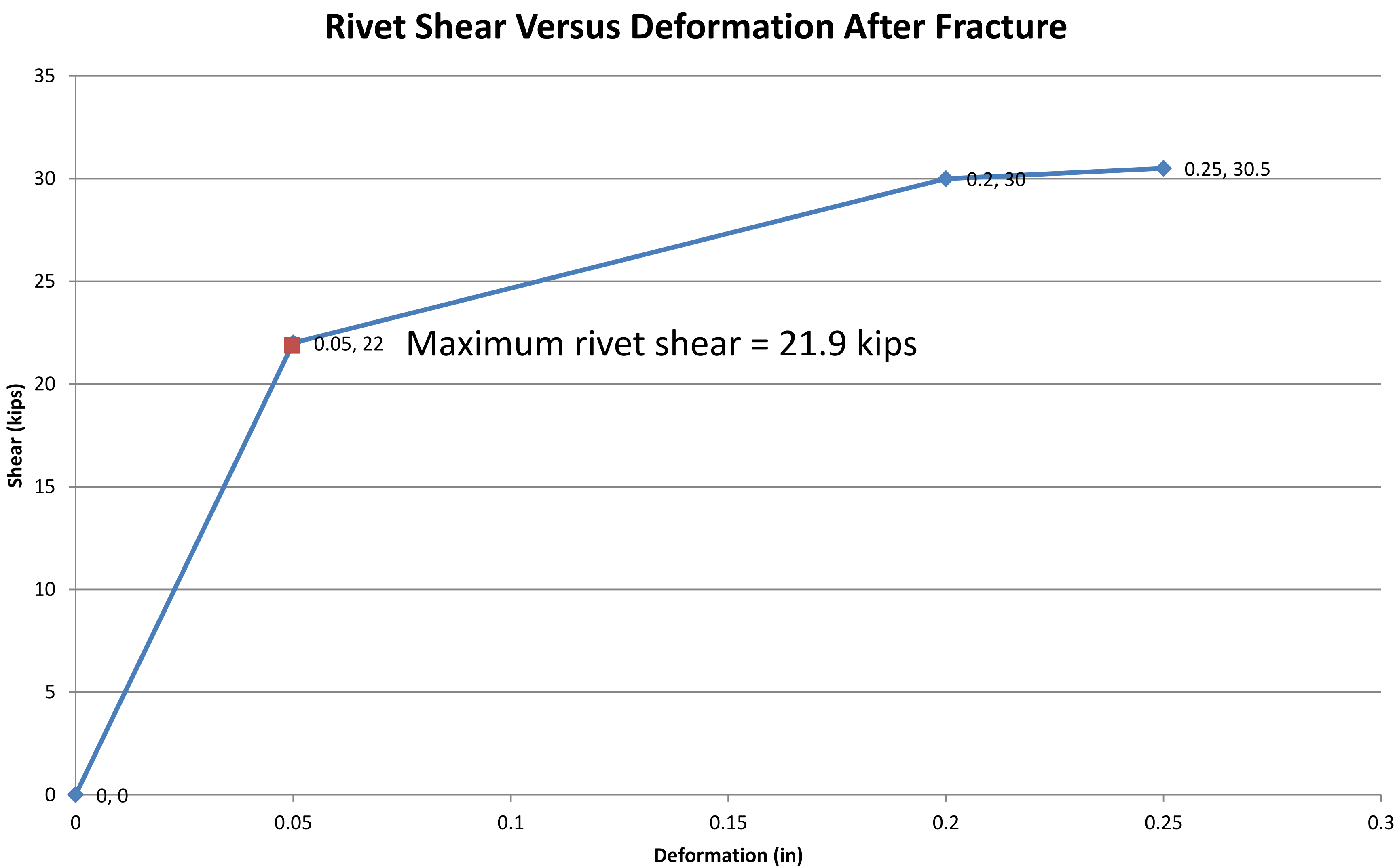
	High Strength Bars	Remaining Channel
Force	285 kips	500 kips
Percent of Total Force	36%	64%
Percent of Total Area	31%	69%

Deformations After Fracture



Deformations After Fracture




Rivet Shear Versus Deformation After Fracture
Shear (kips)
Deformation (in)
0
5
10
15
20
25
30
35
0
0.05
0.1
0.15
0.2
0.25
0.3
0, 0
0.05, 22
0.2, 30
0.25, 30.5
Maximum rivet shear = 21.9 kips

Summary

	Differential Displacement Between U13 and L14
Before Fracture	1.2”
After Fracture	1.3”
High-Strength Bars In Place Only	3.0”

	Force in High-Strength Bars
Before Fracture	191 kips
After Fracture	285 kips
High-Strength Bars In Place Only	967 kips

Note: $0.8A_{bars}f_{pu}$ = (314.4)(4 bars) = 1257.6 kips

Summary

	U13-L14 Longitudinal Strain
Before Fracture	0.0007
After Fracture	0.001
High-Strength Bars In Place Only	0.003

Note: $(0.8f_{pu}) / E_{bar} = 0.004$

Summary

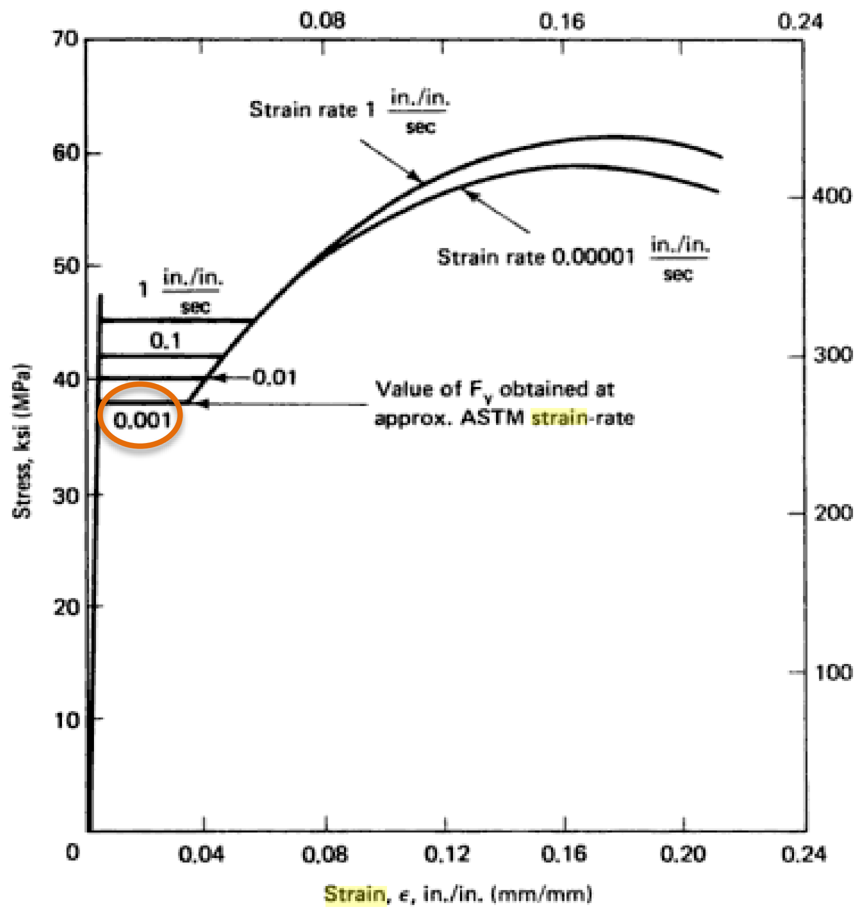
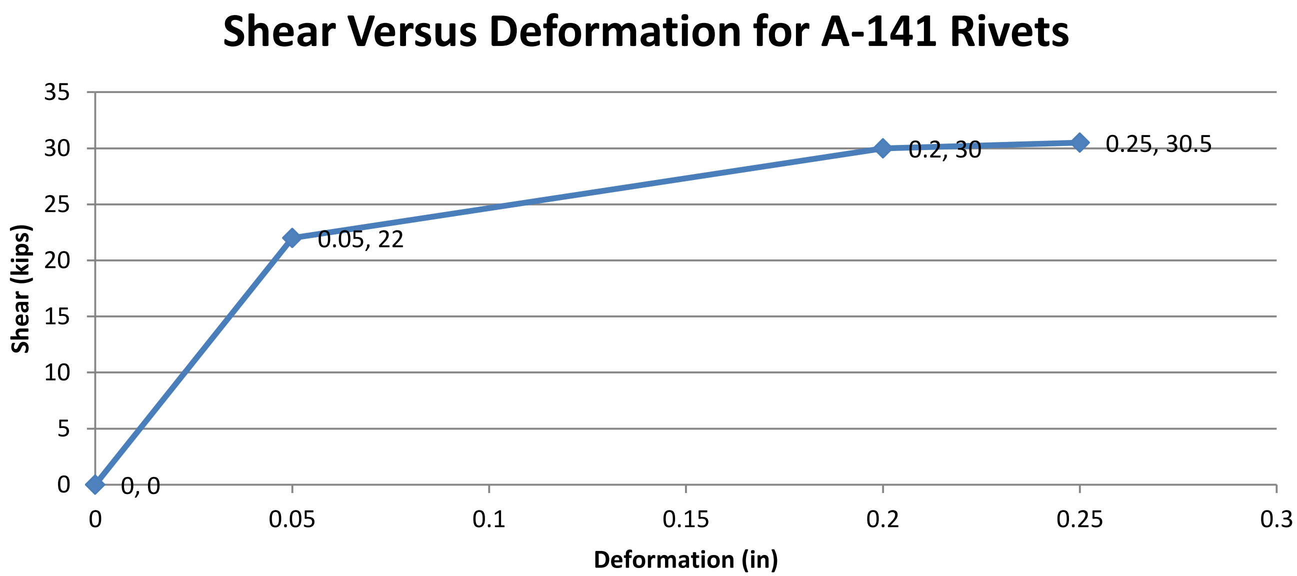


Figure 5.4 Effect of strain rate on stress–strain curve for A7 steel.

Summary

	Maximum Rivet Force	Maximum Rivet Displacement
Before Fracture	18.7 kips	0.042”
After Fracture	21.9 kips	0.050”
High Strength Bars In Place Only	16.5 kips	0.037”

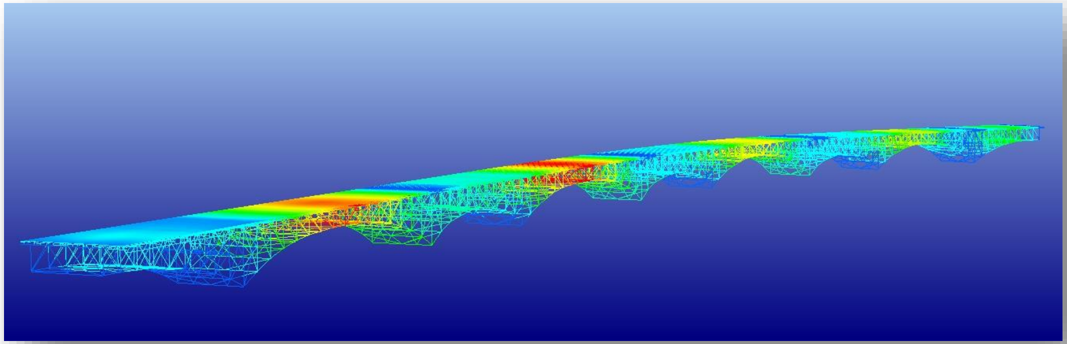


Summary

- Adjacent members have D/C ratios below 1.00 for all three analysis conditions – 5% increase in D/C ratio after fracture for adjacent member
- No signs of connection distress
- High strength bars engage after fracture in proportion to total steel area
- After fracture, strain of remaining channel remains near the beginning of the plastic range
- Strain of remaining channel limited to maximum bar strain of 0.004

Conclusions

- Use pinned ends for the strength limit state
- Consider primary moments
- 2D analysis is sufficient for main members of most truss bridges
- Use 3D when appropriate
- Be consistent with analysis and design in 3D!
- Consider detailed modeling in 3D when needed



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