APA detector Structural Analysis

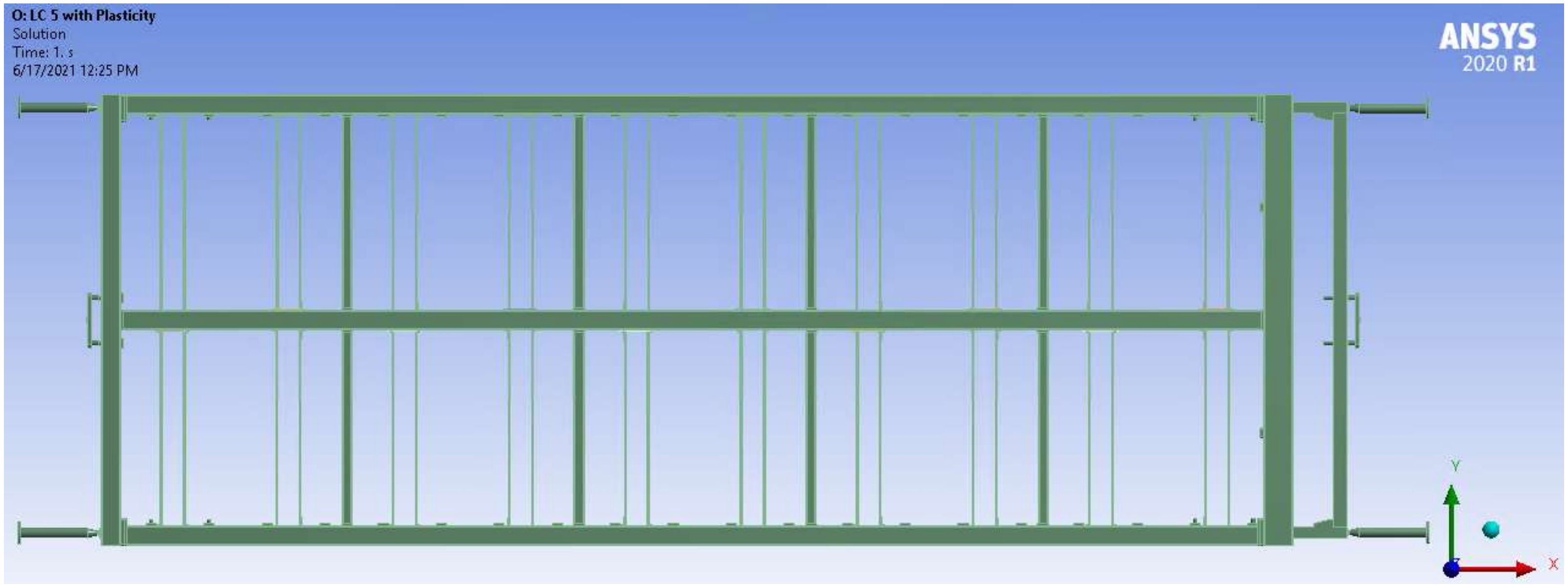
APA group

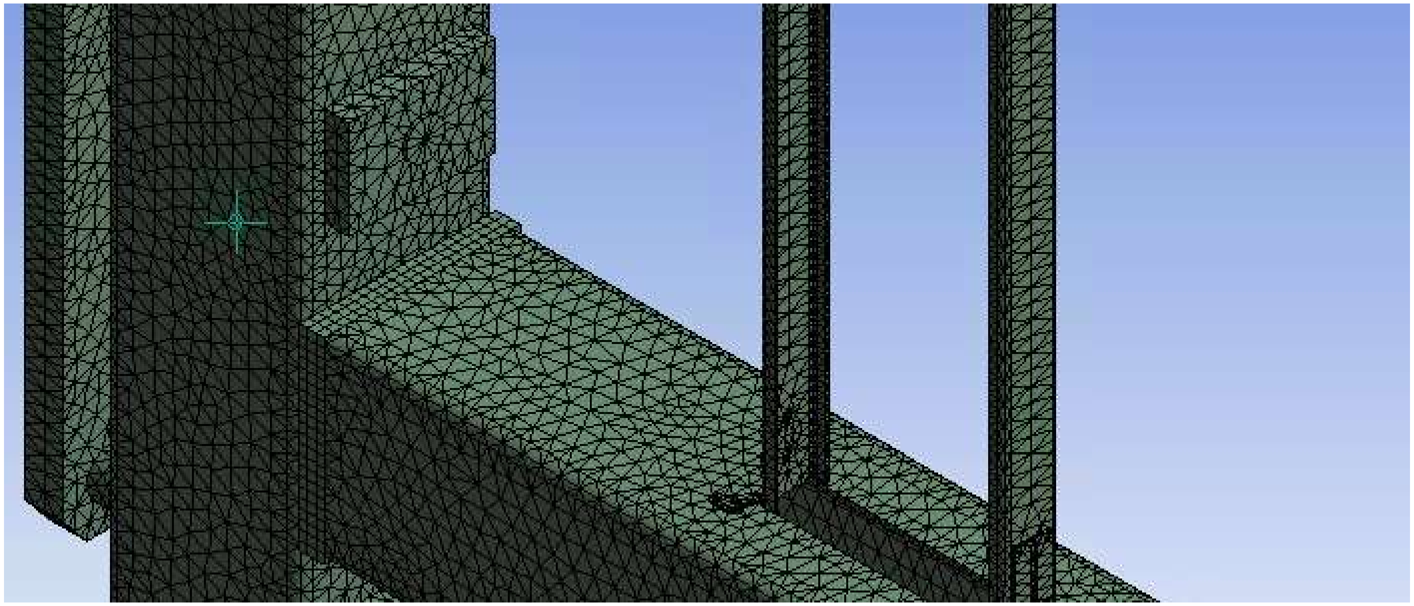
Fermilab

16-JUNE-2021

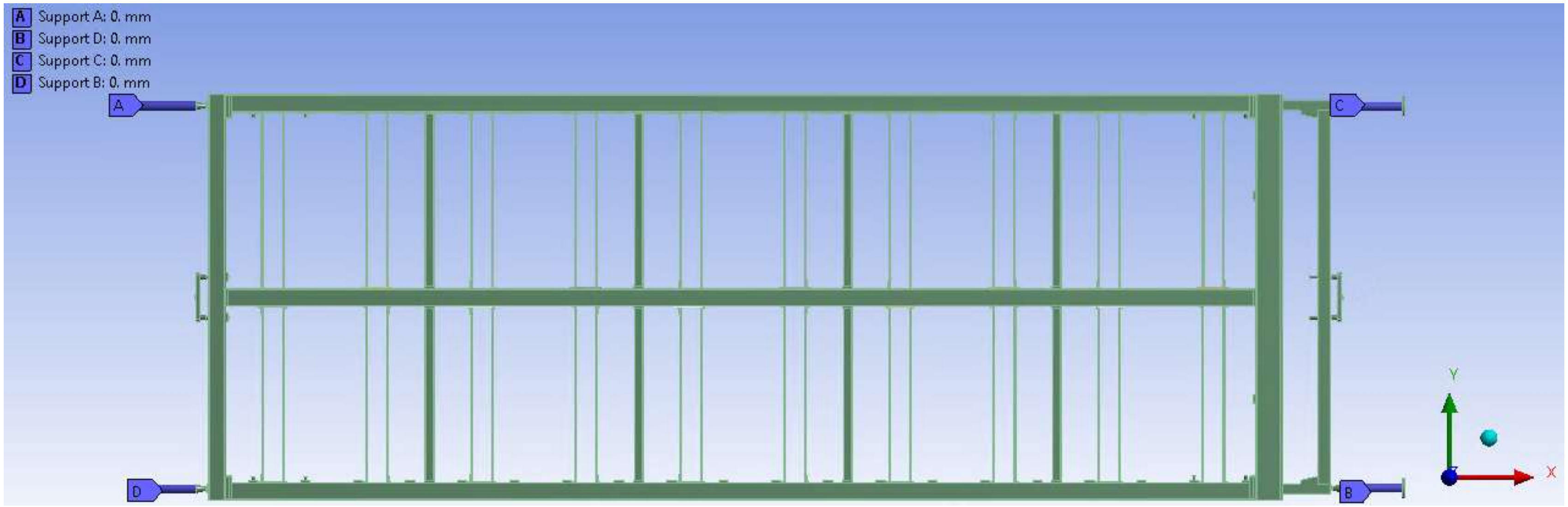
FEA model – Elasto-plastic (bilinear) behavior to evaluate peak stress

Geometry & Mesh of the complete tetrahedral model





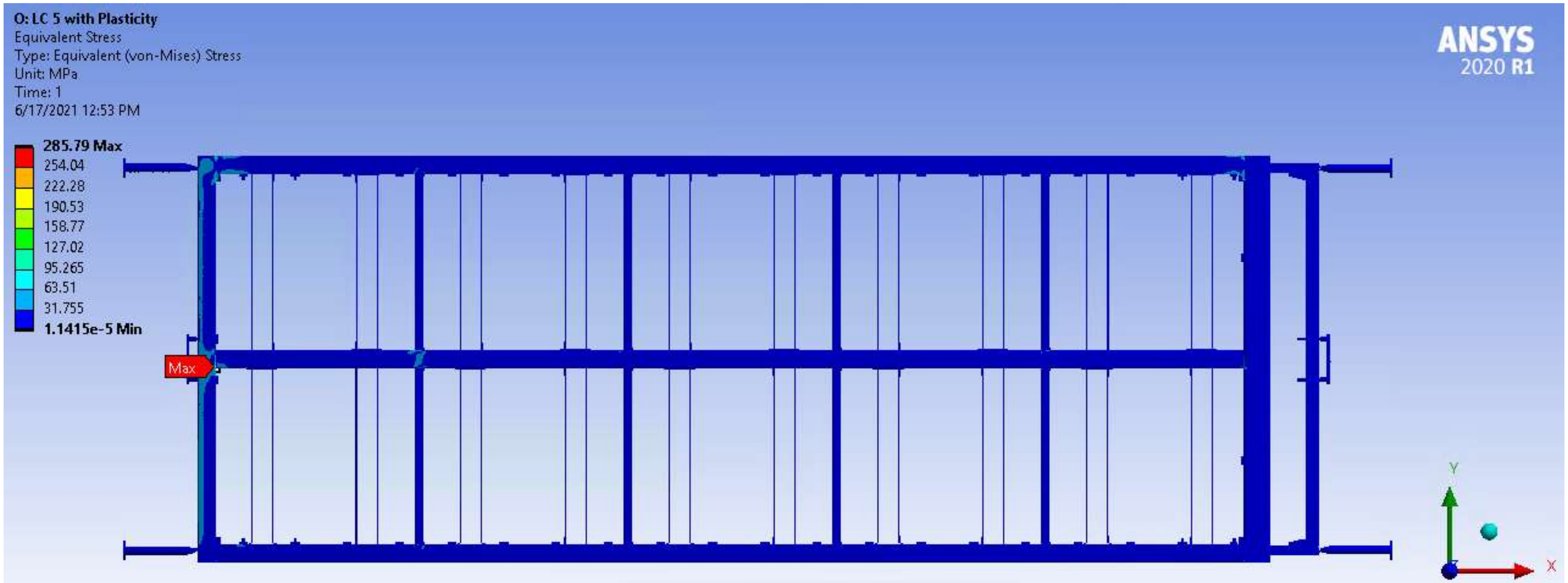
Load and constraints of the complete tetrahedral model



LC 5,6	[3] + [5] + [6]

Component ID	Description	Nominal Mass [kg]	Mass with contingency [kg]	Contingency [%]	COG (location)
[1]	APA frame structure only	315.5	**322**	2	-
[2]	Upper/Lower APA with four wire layers (highest)	464.9	**481.1**	3.5	-
[3]	Upper/Lower Factory APA	492.3	**510**	3.6	
[4]	Integrated APA assembly	631	**654.5**	3.7	-
[5]	APA protection	74	**81.4**	10	-
[6]	Winder support bars	66	**67.3**	2	-

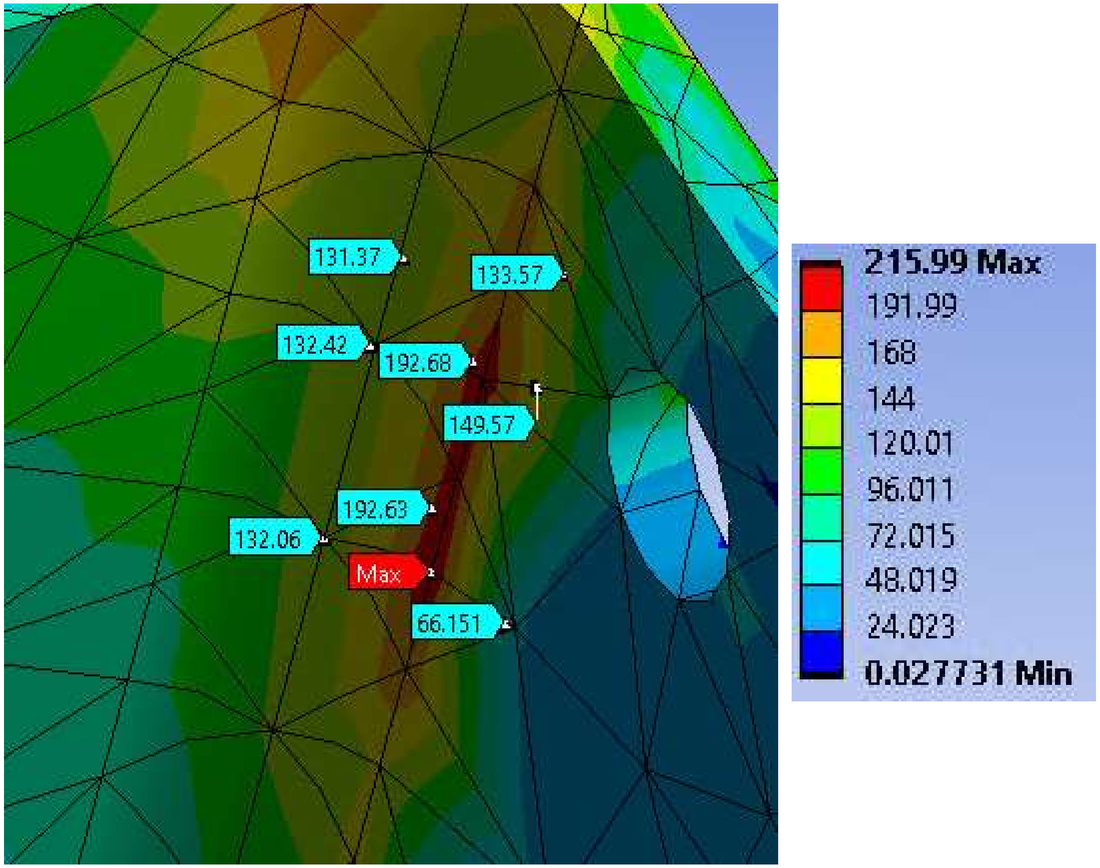
V-Mises Stress plot of the complete tetrahedral model (LC 5)



Elasto-Plastic Analysis

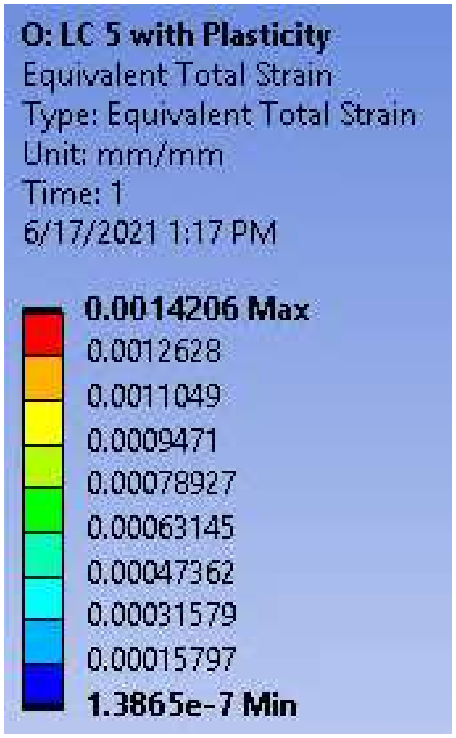
- Bilinear Isotropic Hardening
  - Yield Strength – 189 MPa
  - Youngs Modulus – 200 GPa
  - Tangent Modulus – 2.835 MPa

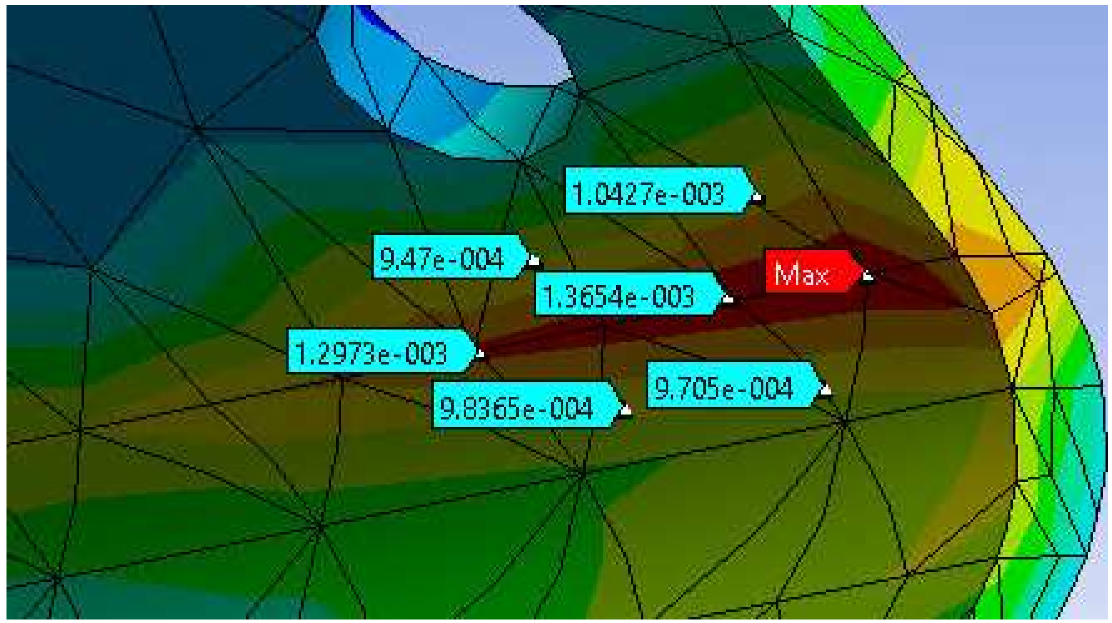
V Mises Stress Plot in area of interest (LC 5)



- Max Stress around 215 MPa
- Element edge length around 5mm

Total Strain of the Sub-model





- Total max strain 1.42e-3

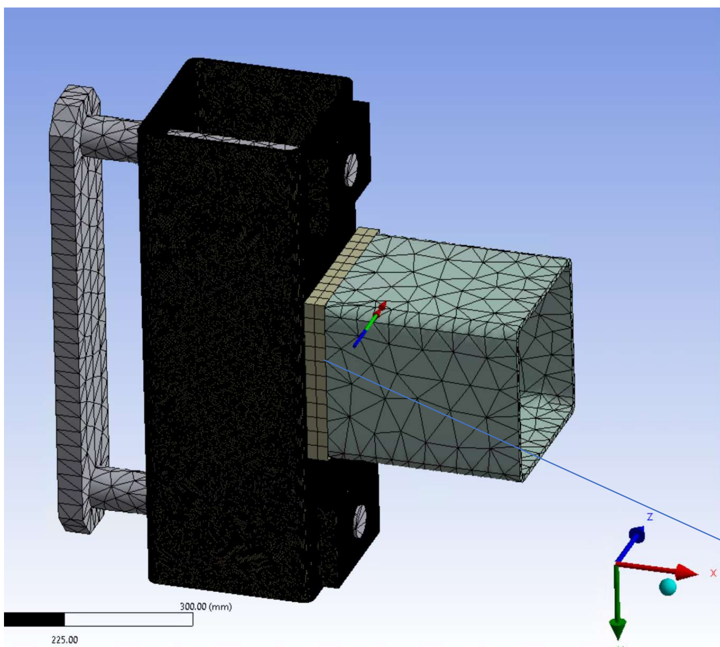
(Proposed) analytical method to evaluate peak stress

Extract reaction forces and moments

Details of "Force Reaction"

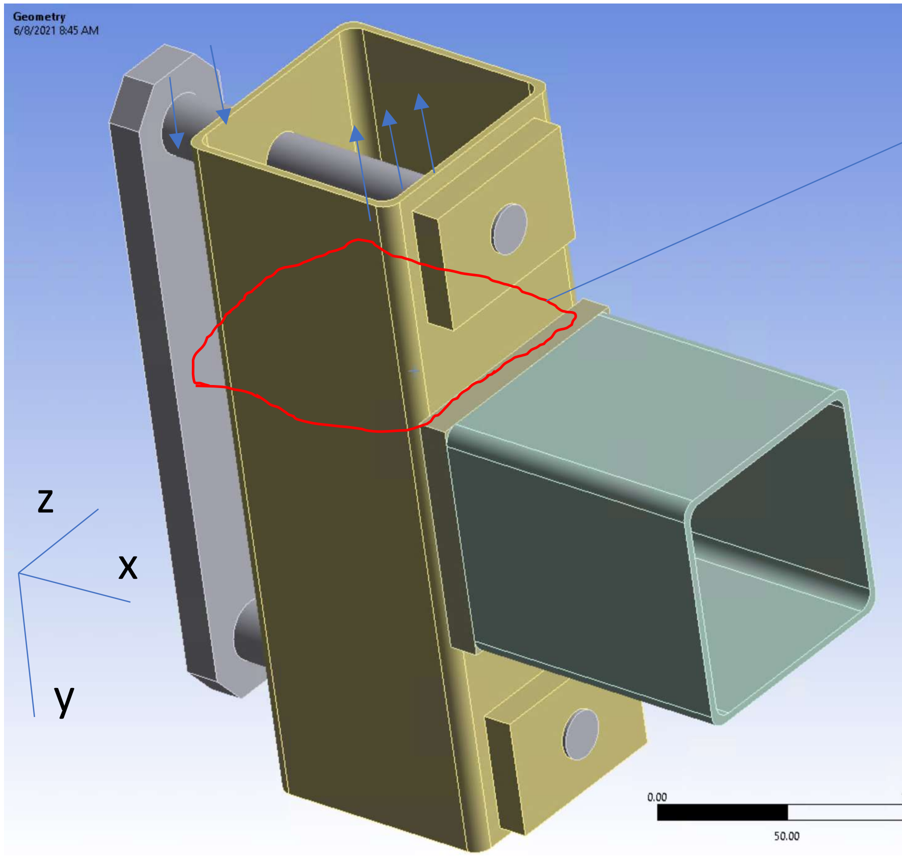
Type	Force Reaction
Location Method	Contact Region
Contact Region	Contact Region
Orientation	Global Coordinate System
Extraction	Contact (Underlying Element)
Suppressed	No
Options	
Result Selection	All
Display Time	End Time
Results	
X Axis	16493 N
Y Axis	1397.8 N
Z Axis	21.375 N
Total	16552 N

Definition	
Type	Moment Reaction
Location Method	Contact Region
Contact Region	Contact Region
Orientation	Global Coordinate System
Summation	Centroid
Extraction	Contact (Underlying Element)
Suppressed	No
Options	
Result Selection	All
Display Time	End Time
Results	
X Axis	581.7 N·mm
Y Axis	-9657.3 N·mm
Z Axis	8.8083e+005 N·mm
Total	8.8088e+005 N·mm



Contact region
(used to extract reaction forces and moments)

Extract reaction forces and moments

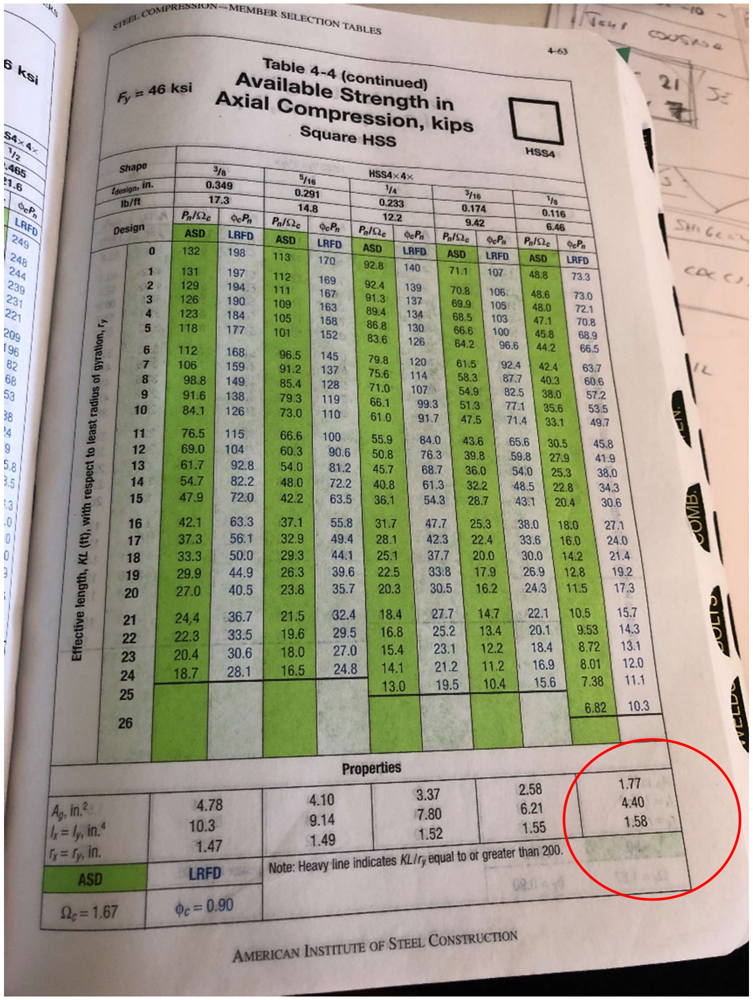


Section located on the foot beam nearby the contact region

Assumption – forces and moments will not have a significant variation between the contact region and the marked section

Fx [kN]	16.5
Fy [kN]	1.4
Fz [kN]	0
Mx [Nxm]	0.6
My [Nxm]	-9.7
Mz [Nxm]	880

Head tube – geometrical properties



STEEL COMPRESSION–MEMBER SELECTION TABLES 4-63

F_y = 46 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS4

Shape	HSS4×4×3/8		HSS4×4×5/16		HSS4×4×1/4		HSS4×4×3/16		HSS4×4×1/8	
t_{design}, in.	0.349		0.291		0.233		0.174		0.116	
lb/ft	17.3		14.8		12.2		9.42		6.46	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
0	132	198	113	170	92.8	140	71.1	107	48.8	73.3
1	131	197	112	169	92.4	139	70.8	106	48.6	73.0
2	129	194	111	167	91.3	137	69.9	105	48.0	72.1
3	126	190	109	163	89.4	134	68.5	103	47.1	70.8
4	123	184	105	158	86.8	130	66.6	100	45.8	68.9
5	118	177	101	152	83.6	126	64.2	96.6	44.2	66.5
6	112	168	96.5	145	79.8	120	61.5	92.4	42.4	63.7
7	106	159	91.2	137	75.6	114	58.3	87.7	40.3	60.6
8	98.8	149	85.4	128	71.0	107	54.9	82.5	38.0	57.2
9	91.6	138	79.3	119	66.1	99.3	51.3	77.1	35.6	53.5
10	84.1	126	73.0	110	61.0	91.7	47.5	71.4	33.1	49.7
11	76.5	115	66.6	100	55.9	84.0	43.6	65.6	30.5	45.8
12	69.0	104	60.3	90.6	50.8	76.3	39.8	59.8	27.9	41.9
13	61.7	92.8	54.0	81.2	45.7	68.7	36.0	54.0	25.3	38.0
14	54.7	82.2	48.0	72.2	40.8	61.3	32.2	48.5	22.8	34.3
15	47.9	72.0	42.2	63.5	36.1	54.3	28.7	43.1	20.4	30.6
16	42.1	63.3	37.1	55.8	31.7	47.7	25.3	38.0	18.0	27.1
17	37.3	56.1	32.9	49.4	28.1	42.3	22.4	33.6	16.0	24.0
18	33.3	50.0	29.3	44.1	25.1	37.7	20.0	30.0	14.2	21.4
19	29.9	44.9	26.3	39.6	22.5	33.8	17.9	26.9	12.8	19.2
20	27.0	40.5	23.8	35.7	20.3	30.5	16.2	24.3	11.5	17.3
21	24.4	36.7	21.5	32.4	18.4	27.7	14.7	22.1	10.5	15.7
22	22.3	33.5	19.6	29.5	16.8	25.2	13.4	20.1	9.53	14.3
23	20.4	30.6	18.0	27.0	15.4	23.1	12.2	18.4	8.72	13.1
24	18.7	28.1	16.5	24.8	14.1	21.2	11.2	16.9	8.01	12.0
25					13.0	19.5	10.4	15.6	7.38	11.1
26									6.82	10.3

Effective length, KL (ft), with respect to least radius of gyration, r_y

Properties					
A_g, in.^{2}	4.78	4.10	3.37	2.58	1.77
$I_x = I_y$, in.^{4}	10.3	9.14	7.80	6.21	4.40
$r_x = r_y$, in.	1.47	1.49	1.52	1.55	1.58

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates KL/r_y equal to or greater than 200.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

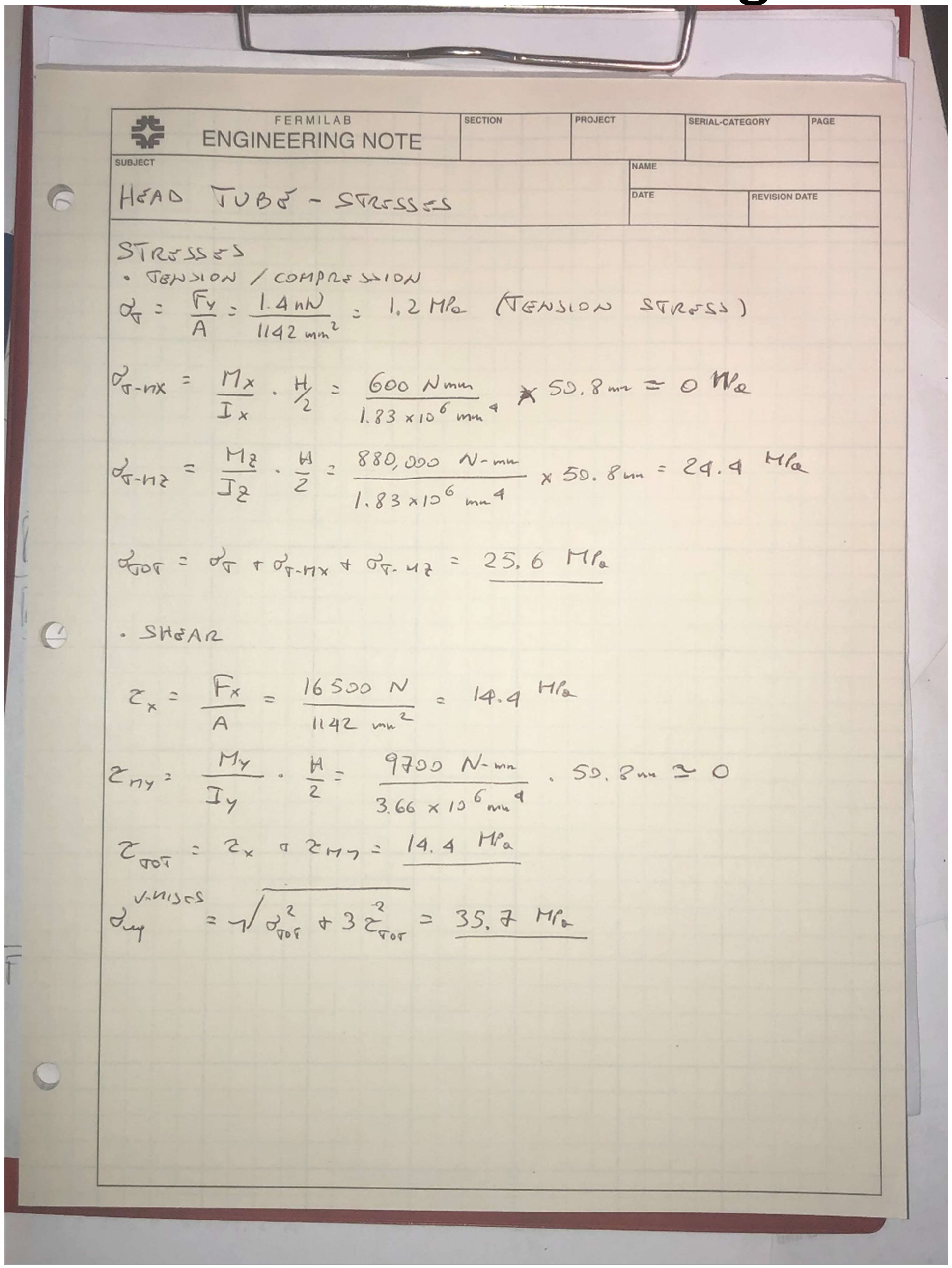
A =1.77in^2 or 1142 sq-mm

Ix or Iz = 4.4 in^4 or 1.83*10^6 mm^4

Iy = Ix+Iz = 3.66x10^6 mm^4

H = 4in or 101.6mm

Head tube – geometrical properties


FERMILAB ENGINEERING NOTE

SECTION | PROJECT | SERIAL-CATEGORY | PAGE

SUBJECT: HEAD TUBE - STRESSES

NAME | DATE | REVISION DATE

STRESSES

• TENSION / COMPRESSION

$\sigma_T = \frac{F_Y}{A} = \frac{1.4 \text{ kN}}{1142 \text{ mm}^2} = 1.2 \text{ MPa}$ (TENSION STRESS)

$\sigma_{T\text{-}MX} = \frac{M_X}{I_X} \cdot \frac{H}{2} = \frac{600 \text{ Nmm}}{1.83 \times 10^6 \text{ mm}^4} \times 50.8 \text{ mm} \simeq 0 \text{ MPa}$

$\sigma_{T\text{-}MZ} = \frac{M_Z}{I_Z} \cdot \frac{H}{2} = \frac{880{,}000 \text{ N-mm}}{1.83 \times 10^6 \text{ mm}^4} \times 50.8 \text{ mm} = 24.4 \text{ MPa}$

$\sigma_{TOT} = \sigma_T + \sigma_{T\text{-}MX} + \sigma_{T\text{-}MZ} = 25.6 \text{ MPa}$

• SHEAR

$\tau_x = \frac{F_X}{A} = \frac{16500 \text{ N}}{1142 \text{ mm}^2} = 14.4 \text{ MPa}$

$\tau_{MY} = \frac{M_Y}{I_Y} \cdot \frac{H}{2} = \frac{9700 \text{ N-mm}}{3.66 \times 10^6 \text{ mm}^4} \cdot 50.8 \text{ mm} \simeq 0$

$\tau_{TOT} = \tau_x + \tau_{MY} = 14.4 \text{ MPa}$

$\sigma_{eq}^{V\text{-}MISES} = \sqrt{\sigma_{TOT}^2 + 3\tau_{TOT}^2} = 35.7 \text{ MPa}$

V-Mises eqv. stress = 35.7MPa, factor 5.3 smaller than the allowable stress (190 Mpa). – stresses in the head tube are fine.

Buckling Analysis
(Expanded the buckling slide from 2nd June 2021)

Head tube – geometrical properties

Radius of gyration of a 4"x4" long tube in the unbraced direction

Slenderness ratio shows that the Euler formula for critical buckling load is accurate

Factor of 1.5 applied for additional tension from cooling wires

Shows a safety factor of 10 against buckling, even if the load was applied in a configuration that allowed for buckling

$r_g = 1.58\ [in] =\ .0401\ [m] (per\ AISC\ Manual\ of\ Structural\ Steel\ Construction)$

$L_{beam} = 6.0\ [m]\ , \frac{6.0[m]}{.0401[m]} = 150\ slenderness\ ratio, which\ is\ a\ slender\ column$

$I = 1.77\ [in^2] * (1.58\ [in])^2 = 4.42\ [in^4] = 1.84e^{-6}[m^4]$

$I\ for\ all\ beams = 3 * I = 5.52e^{-6}[m^4]$

$P_{cr} = \frac{pi^2 * E * I}{L^2} = 3.14^2 * 207e^{11}\ [Pa] * \frac{5.52e^{-6}[m^4]}{(6.0[m])^2} = 313200\ [N]$

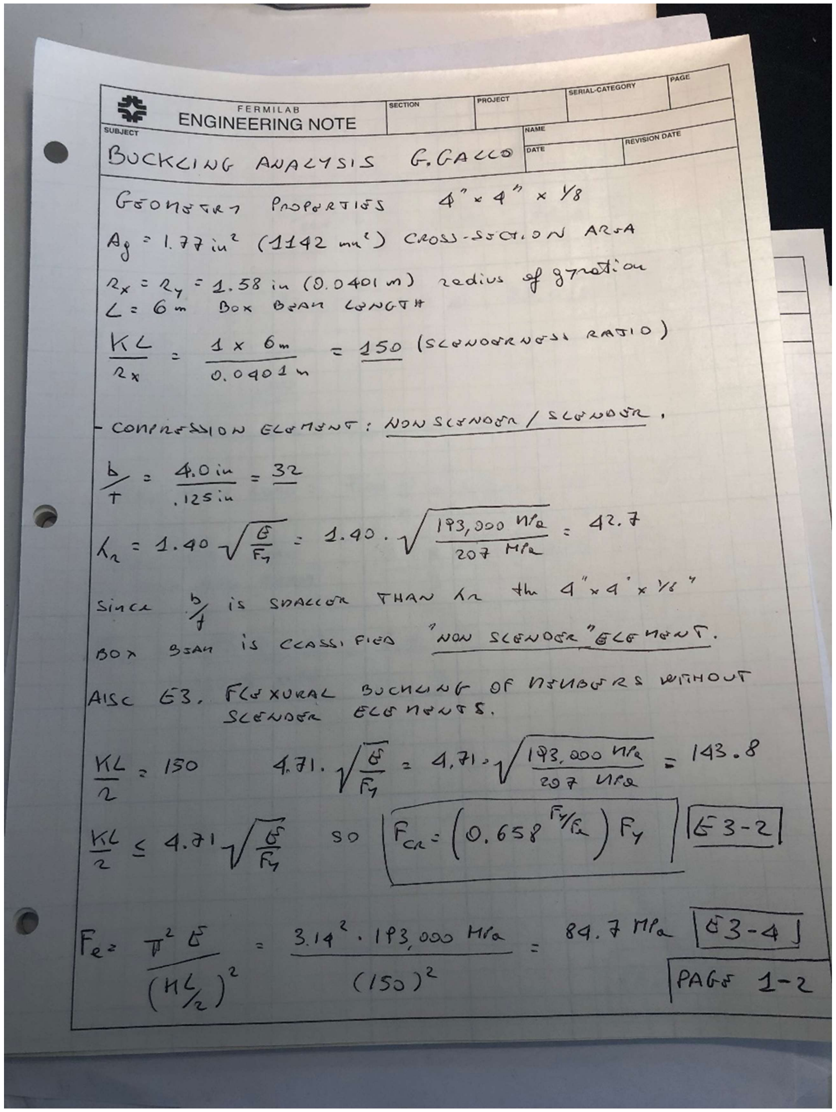
$Wire\ load\ is\ 6.5\ N\ per\ wire, or\ 9100 \frac{N}{m}$

$9100 \left[\frac{N}{m}\right] * 2.3[m] * 1.5 = 31395\ [N]$

$\frac{313200[N]}{31395[N]} =\ 9.98\ Safety\ margin\ against\ buckling$

This buckling consideration has been used for several previous reviews and has approved.
EDMS #2142671 & 2100877

Buckling Analysis – Analytical method based on AISC G.Gallo



FERMILAB ENGINEERING NOTE

SUBJECT: BUCKLING ANALYSIS G.GALLO

GEOMETRY PROPERTIES 4" x 4" x 1/8

$A_g = 1.77\ in^2$ ($1142\ mm^2$) CROSS-SECTION AREA

$r_x = r_y = 1.58\ in$ ($0.0401\ m$) radius of gyration

$L = 6\ m$ BOX BEAM LENGTH

$\frac{KL}{r_x} = \frac{1 \times 6\ m}{0.0401\ m} = 150$ (SLENDERNESS RATIO)

- COMPRESSION ELEMENT: NON SLENDER / SLENDER.

$\frac{b}{t} = \frac{4.0\ in}{.125\ in} = 32$

$\lambda_r = 1.40\sqrt{\frac{E}{F_y}} = 1.40 \cdot \sqrt{\frac{193{,}000\ MPa}{207\ MPa}} = 42.7$

Since b/t is SMALLER THAN λ_r the 4" x 4" x 1/8" BOX BEAM is CLASSIFIED "NON SLENDER" ELEMENT.

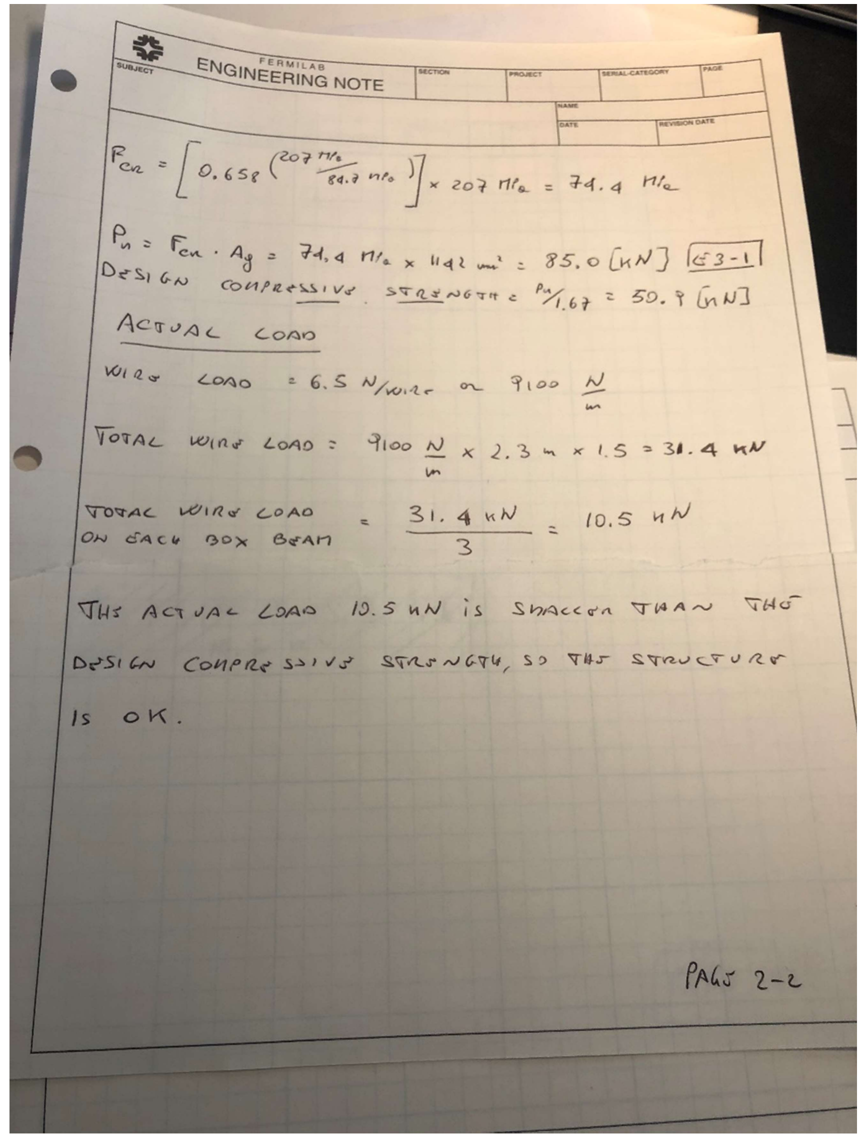
AISC E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS.

$\frac{KL}{r} = 150 \quad 4.71 \cdot \sqrt{\frac{E}{F_y}} = 4.71 \cdot \sqrt{\frac{193{,}000\ MPa}{207\ MPa}} = 143.8$

$\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$ SO $F_{cr} = \left(0.658^{F_y/F_e}\right) F_y$ [E3-2]

$F_e = \frac{\pi^2 E}{\left(KL/r\right)^2} = \frac{3.14^2 \cdot 193{,}000\ MPa}{(150)^2} = 84.7\ MPa$ [E3-4]

PAGE 1-2



FERMILAB ENGINEERING NOTE

$F_{cr} = \left[0.658^{\left(\frac{207\ MPa}{84.7\ MPa}\right)}\right] \times 207\ MPa = 74.4\ MPa$

$P_n = F_{cr} \cdot A_g = 74.4\ MPa \times 1142\ mm^2 = 85.0\ [kN]$ [E3-1]

DESIGN COMPRESSIVE STRENGTH $= P_n/1.67 = 50.9\ [kN]$

ACTUAL LOAD

WIRE LOAD $= 6.5\ N/wire$ or $9100\ \frac{N}{m}$

TOTAL WIRE LOAD $= 9100\ \frac{N}{m} \times 2.3\ m \times 1.5 = 31.4\ kN$

TOTAL WIRE LOAD ON EACH BOX BEAM $= \frac{31.4\ kN}{3} = 10.5\ kN$

THE ACTUAL LOAD 10.5 kN is SMALLER THAN THE DESIGN COMPRESSIVE STRENGTH, SO THE STRUCTURE IS OK.

PAGE 2-2

Buckling Analysis – AISC source
Geometrical properties of the box beam 4in x 4in x 1/8in

4-63

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

$F_y = 46$ ksi

HSS4

Shape	HSS4×4× 3/8		HSS4×4× 5/16		HSS4×4× 1/4		HSS4×4× 3/16		HSS4×4× 1/8	
t_{design}, in.	0.349		0.291		0.233		0.174		0.116	
lb/ft	17.3		14.8		12.2		9.42		6.46	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, KL (ft), with respect to least radius of gyration, r_y: 0	132	198	113	170	92.8	140	71.1	107	48.8	73.3
1	131	197	112	169	92.4	139	70.8	106	48.6	73.0
2	129	194	111	167	91.3	137	69.9	105	48.0	72.1
3	126	190	109	163	89.4	134	68.5	103	47.1	70.8
4	123	184	105	158	86.8	130	66.6	100	45.8	68.9
5	118	177	101	152	83.6	126	64.2	96.6	44.2	66.5
6	112	168	96.5	145	79.8	120	61.5	92.4	42.4	63.7
7	106	159	91.2	137	75.6	114	58.3	87.7	40.3	60.6
8	98.8	149	85.4	128	71.0	107	54.9	82.5	38.0	57.2
9	91.6	138	79.3	119	66.1	99.3	51.3	77.1	35.6	53.5
10	84.1	126	73.0	110	61.0	91.7	47.5	71.4	33.1	49.7
11	76.5	115	66.6	100	55.9	84.0	43.6	65.6	30.5	45.8
12	69.0	104	60.3	90.6	50.8	76.3	39.8	59.8	27.9	41.9
13	61.7	92.8	54.0	81.2	45.7	68.7	36.0	54.0	25.3	38.0
14	54.7	82.2	48.0	72.2	40.8	61.3	32.2	48.5	22.8	34.3
15	47.9	72.0	42.2	63.5	36.1	54.3	28.7	43.1	20.4	30.6
16	42.1	63.3	37.1	55.8	31.7	47.7	25.3	38.0	18.0	27.1
17	37.3	56.1	32.9	49.4	28.1	42.3	22.4	33.6	16.0	24.0
18	33.3	50.0	29.3	44.1	25.1	37.7	20.0	30.0	14.2	21.4
19	29.9	44.9	26.3	39.6	22.5	33.8	17.9	26.9	12.8	19.2
20	27.0	40.5	23.8	35.7	20.3	30.5	16.2	24.3	11.5	17.3
21	24.4	36.7	21.5	32.4	18.4	27.7	14.7	22.1	10.5	15.7
22	22.3	33.5	19.6	29.5	16.8	25.2	13.4	20.1	9.53	14.3
23	20.4	30.6	18.0	27.0	15.4	23.1	12.2	18.4	8.72	13.1
24	18.7	28.1	16.5	24.8	14.1	21.2	11.2	16.9	8.01	12.0
25					13.0	19.5	10.4	15.6	7.38	11.1
26									6.82	10.3

Properties

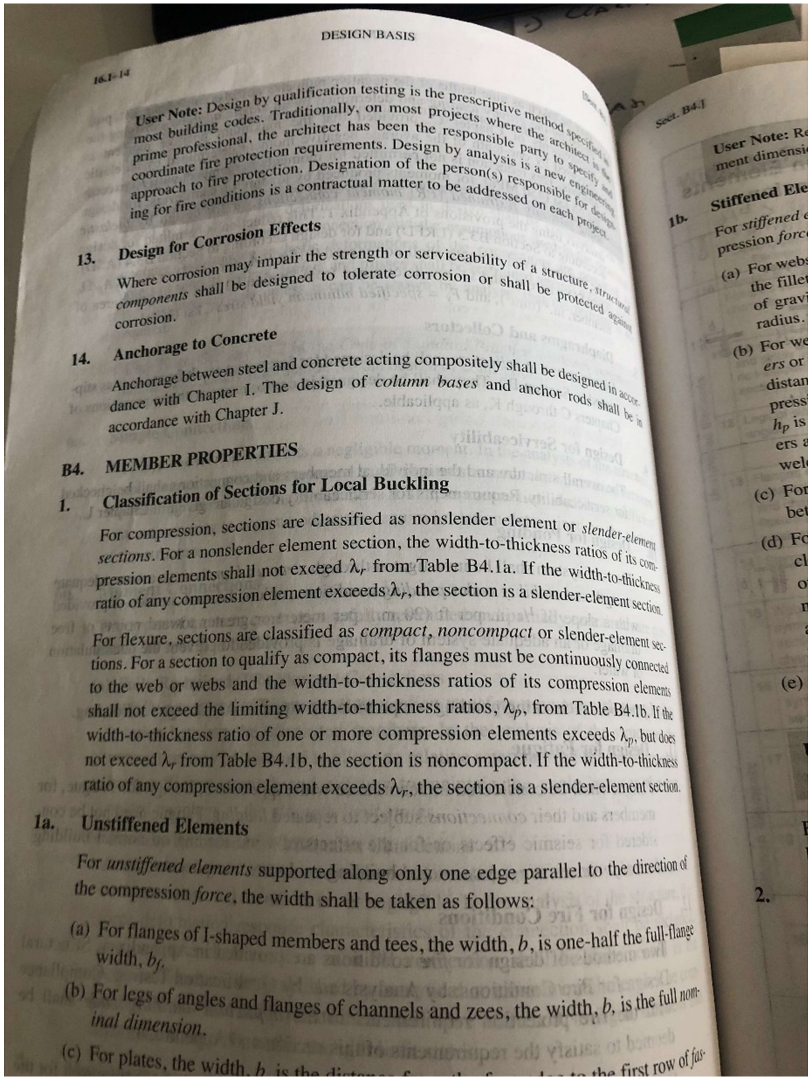
	3/8	5/16	1/4	3/16	1/8
A_g, in.^{2}	4.78	4.10	3.37	2.58	1.77
$I_x = I_y$, in.^{4}	10.3	9.14	7.80	6.21	4.40
$r_x = r_y$, in.	1.47	1.49	1.52	1.55	1.58

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates KL/r_y equal to or greater than 200.

American Institute of Steel Construction

Buckling Analysis – AISC source
Definition of nonslender/slender element



16.1-14 DESIGN BASIS

User Note: Design by qualification testing is the prescriptive method specified in most building codes. Traditionally, on most projects where the architect is the prime professional, the architect has been the responsible party to specify and coordinate fire protection requirements. Design by analysis is a new engineering approach to fire protection. Designation of the person(s) responsible for designing for fire conditions is a contractual matter to be addressed on each project.

13. Design for Corrosion Effects

Where corrosion may impair the strength or serviceability of a structure, *structural components* shall be designed to tolerate corrosion or shall be protected against corrosion.

14. Anchorage to Concrete

Anchorage between steel and concrete acting compositely shall be designed in accordance with Chapter I. The design of *column bases* and anchor rods shall be in accordance with Chapter J.

B4. MEMBER PROPERTIES

1. Classification of Sections for Local Buckling

For compression, sections are classified as nonslender element or *slender-element sections*. For a nonslender element section, the width-to-thickness ratios of its compression elements shall not exceed λ_r from Table B4.1a. If the width-to-thickness ratio of any compression element exceeds λ_r, the section is a slender-element section.

For flexure, sections are classified as *compact*, *noncompact* or slender-element sections. For a section to qualify as compact, its flanges must be continuously connected to the web or webs and the width-to-thickness ratios of its compression elements shall not exceed the limiting width-to-thickness ratios, λ_p, from Table B4.1b. If the width-to-thickness ratio of one or more compression elements exceeds λ_p, but does not exceed λ_r from Table B4.1b, the section is noncompact. If the width-to-thickness ratio of any compression element exceeds λ_r, the section is a slender-element section.

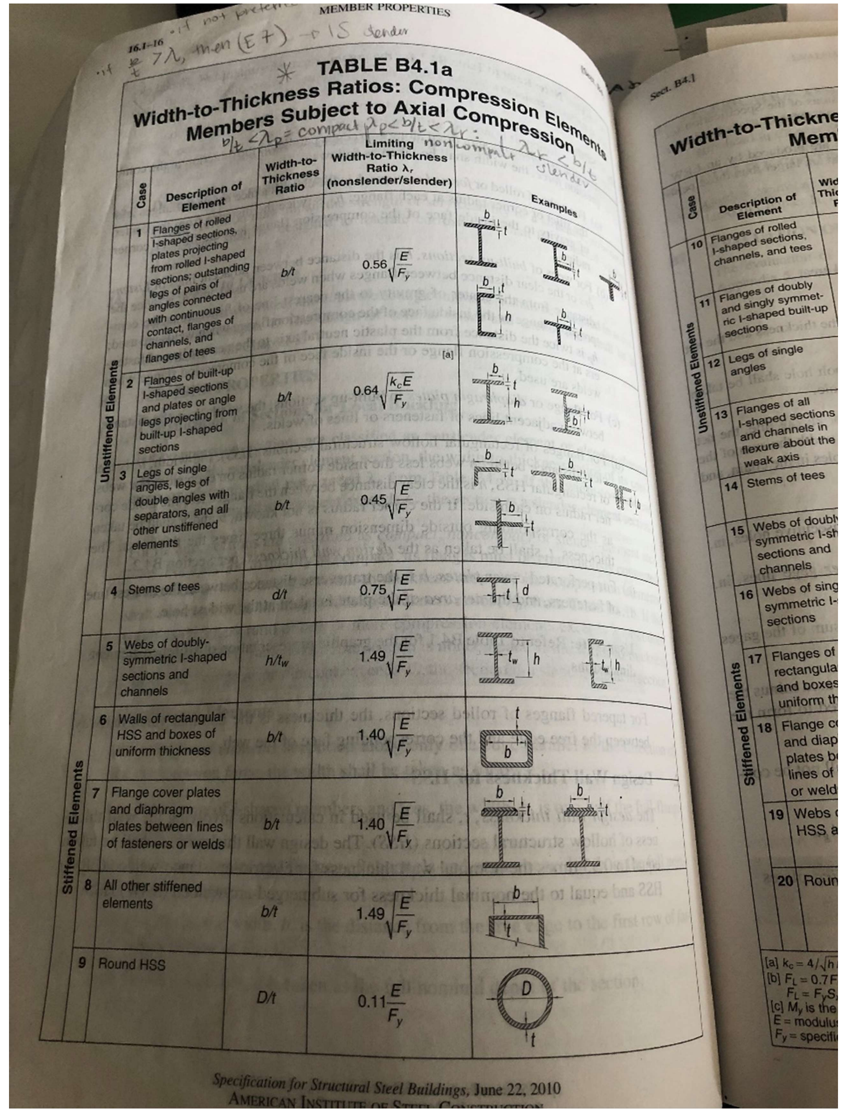
1a. Unstiffened Elements

For *unstiffened elements* supported along only one edge parallel to the direction of the compression *force*, the width shall be taken as follows:

(a) For flanges of I-shaped members and tees, the width, b, is one-half the full-flange width, b_f.

(b) For legs of angles and flanges of channels and zees, the width, b, is the full *nominal dimension*.

(c) For plates, the width, b, is the distance from the free edge to the first row of fas-



16.1-16 MEMBER PROPERTIES

TABLE B4.1a
Width-to-Thickness Ratios: Compression Elements Members Subject to Axial Compression

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_r (nonslender/slender)	Examples
Unstiffened Elements	1	Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees	b/t	$0.56\sqrt{\frac{E}{F_y}}$	
	2	Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	$0.64\sqrt{\frac{k_cE}{F_y}}$ [a]	
	3	Legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	$0.45\sqrt{\frac{E}{F_y}}$	
	4	Stems of tees	d/t	$0.75\sqrt{\frac{E}{F_y}}$	
Stiffened Elements	5	Webs of doubly-symmetric I-shaped sections and channels	h/t_w	$1.49\sqrt{\frac{E}{F_y}}$	
	6	Walls of rectangular HSS and boxes of uniform thickness	b/t	$1.40\sqrt{\frac{E}{F_y}}$	
	7	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.40\sqrt{\frac{E}{F_y}}$	
	8	All other stiffened elements	b/t	$1.49\sqrt{\frac{E}{F_y}}$	
	9	Round HSS	D/t	$0.11\frac{E}{F_y}$	

Specification for Structural Steel Buildings, June 22, 2010
AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Buckling Analysis – AISC source
Nominal compressive strength

CHAPTER E
DESIGN OF MEMBERS FOR COMPRESSION

This chapter addresses members subject to axial compression through the centroidal axis.

The chapter is organized as follows:

E1. General Provisions
E2. Effective Length
E3. Flexural Buckling of Members without Slender Elements
E4. Torsional and Flexural-Torsional Buckling of Members without Slender Elements
E5. Single Angle Compression Members
E6. Built-Up Members
E7. Members with Slender Elements

User Note: For cases not included in this chapter the following sections apply:

- H1 – H2 Members subject to combined axial compression and flexure
- H3 Members subject to axial compression and torsion
- I2 Composite axially loaded members
- J4.4 Compressive strength of connecting elements

E1. GENERAL PROVISIONS

The *design compressive strength*, $\phi_c P_n$, and the *allowable compressive strength*, P_n/Ω_c, are determined as follows.

The *nominal compressive strength*, P_n, shall be the lowest value obtained based on the applicable *limit states* of *flexural buckling*, *torsional buckling*, and *flexural-torsional buckling*.

$$\phi_c = 0.90 \text{ (LRFD)} \qquad \Omega_c = 1.67 \text{ (ASD)}$$

E2. EFFECTIVE LENGTH

The *effective length factor*, K, for calculation of member slenderness, KL/r, shall be determined in accordance with Chapter C or Appendix 7,

where

L = laterally *unbraced length* of the member, in. (mm)
r = radius of gyration, in. (mm)

User Note: For members designed on the basis of compression, the effective slenderness ratio KL/r preferably should not exceed 200.

E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to nonslender element compression members as defined in Section B4.1 for elements in uniform compression.

User Note: When the torsional *unbraced length* is larger than the lateral unbraced length, Section E4 may control the design of wide flange and similarly shaped *columns*.

The *nominal compressive strength*, P_n, shall be determined based on the *limit state* of *flexural buckling*.

$$P_n = F_{cr} A_g \quad \text{(E3-1)}$$

The *critical stress*, F_{cr}, is determined as follows:

(a) When $\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$ (or $\frac{F_y}{F_e} \le 2.25$)

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \quad \text{(E3-2)}$$

(b) When $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ (or $\frac{F_y}{F_e} > 2.25$)

$$F_{cr} = 0.877 F_e \quad \text{(E3-3)}$$

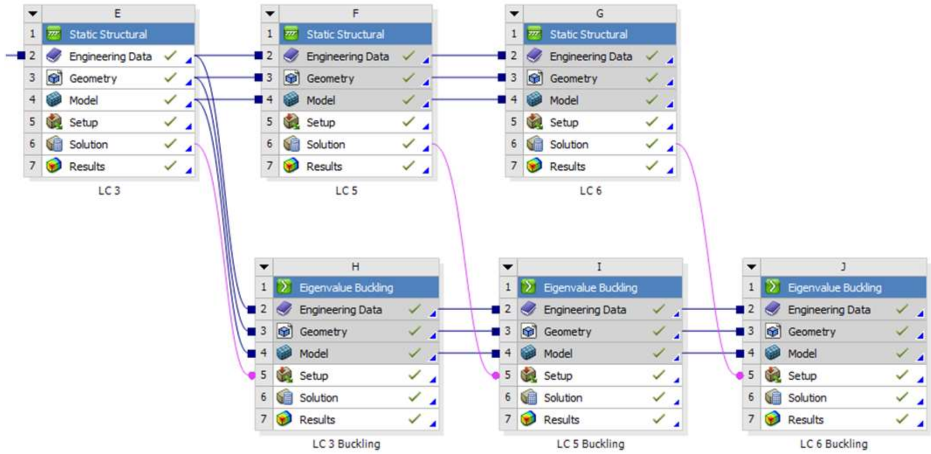
where

F_e = elastic *buckling* stress determined according to Equation E3-4, as specified in Appendix 7, Section 7.2.3(b), or through an elastic buckling analysis, as applicable, ksi (MPa)

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \quad \text{(E3-4)}$$

FEA linear buckling analysis

FEA model – Linear buckling analysis



Eigenvalue buckling – Linear
Large displacement ON