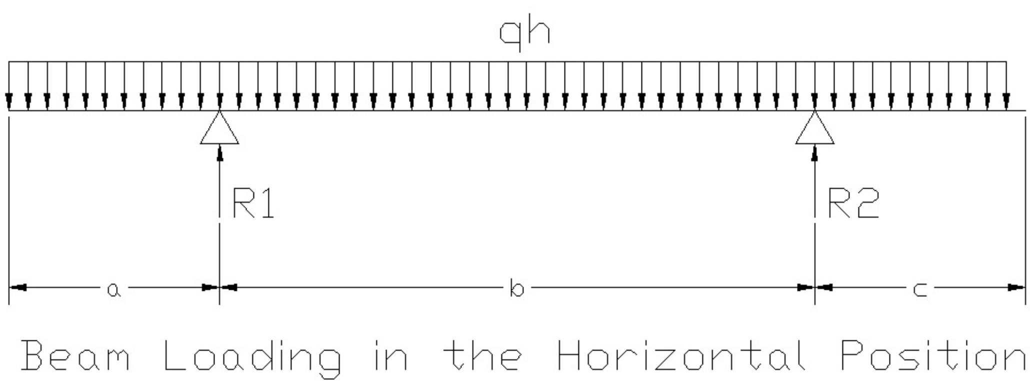
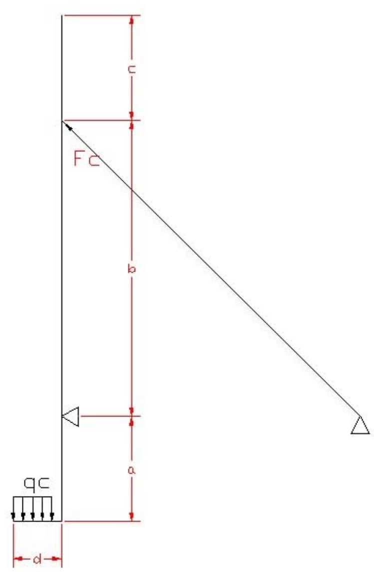
The Pictures below describe the orientation of the block raiser and the dimensions/variables which will beused for the analysis.





1.0 Define Inputs

$tons := 2000 lbf$ $\quad plf := \frac{lbf}{ft}$ $\quad psf := \frac{lbf}{ft^2}$ $\quad pcf := \frac{lbf}{ft^3}$ $\quad kips := 1000 lbf$

$ksi := 1000 psi$

$fy := 36 ksi$ Yield strength of material

$Es := 29000 ksi$ Youngs modulus of steel

$W := 200 tons$ Weight of the Block and block raiser table

$Wt := 50 tons$ Weight of BR Table

$L := 53 ft$ Length and width of the BR table

$a := 11 ft$ Distance from the end of the table to the first support/pivot point

$b := 33 ft$ Distance between supports.

$c := 11 ft$ Distance from support/cylinder to end of table

$te := 66 mm$ Thickness of Extrusions

$Np := 31$ Number of planes in a block

$d := te \cdot Np$ Length of forks

$d = 6.71\ ft$

$Nf := 24$ Number of forks

$Nb := 24$ Number of beams

2.0 Determination of Distributed Loading

$qh := \frac{W}{Nb \cdot L}$ Loading on Table in the horizontal position

$qh = 314.47 \frac{lbf}{ft}$

$qc := \frac{W - Wt}{Nf \cdot d}$ Loading on Forks in the Vertical Position

$qc = 1862.17 \frac{lbf}{ft}$

3.0 Calculation of Critical Moments

Examine the table moments in the horizontal position first.

$Rn := \frac{qh \cdot L}{2}$ Reaction force at support points

$Rn = 8.33\,kips$

$$Mn(x) := \begin{cases} \left(\frac{qh \cdot x^2}{2}\right) & \text{if } a > x \\ \left[Rn \cdot (x - a) - \frac{qh \cdot x^2}{2}\right] & \text{if } a \le x < a + b \\ \left(Rn(x - a) + Rn(x - a - b) - \frac{qh \cdot x^2}{2}\right) & \text{otherwise} \end{cases}$$

$Mn(a) = -19.03\,kips \cdot ft$

$Mn\left(a + \frac{b}{2}\right) = 18.59\,kips \cdot ft$

$x := 0, L .. 1.0in$

Calculate the maximum moment and reaction force on the forks in the vertical position

$Mf := \frac{qc \cdot d^2}{2}$ Maximum moment at end of forks

$Mf = 41.95\,kips \cdot ft$

$Rnf := qc \cdot d$ Reaction force at end of forks

$Rnf = 12.50\,kips$

4.0 Calculate the required sections for beam and forks based on moment.

$Sx1 := \frac{|Mn(a)|}{fy}$

$Sx1 = 6.34\,in^3$

$Sx2 := \frac{Mf}{fy}$

$Sx2 = 13.98\ in^3$ Use a HSS20x8x5/16 at 48.8lbs/ft. and Sx=78.6in^3 and Ix=786in^4

$Ix := 786in^4$

5.0 Calculate the deflections

Table in the horizontal position

$$\Delta end := \frac{qh \cdot a}{24 \cdot Es \cdot Ix} \cdot \left[3 \cdot a^2 \cdot (a + 2 \cdot b) - b^3\right]$$

Deflection at end of beam in the horizontal position

$\Delta end = -0.09\ in$

$$\Delta mid := \frac{qh \cdot b^2}{384 \cdot Es \cdot Ix} \cdot \left(5 \cdot b^2 - 24 \cdot a^2\right)$$

$\Delta mid = 0.17\ in$

Deflection of Forks when Table is in the Vertical Position

$$\Delta f := \frac{qc \cdot d^2 \cdot a \cdot b}{6Es \cdot Ix} + \frac{qc \cdot d^2 \cdot a^2}{6 \cdot Es \cdot Ix}$$

$\Delta f = 0.51\ in$ Horizontal Deflection of fork

$$\Delta fv := \frac{qc \cdot d^4}{8 \cdot Es \cdot Ix} + \frac{qc \cdot d^3 \cdot b}{6Es \cdot Ix} + \frac{qc \cdot d^3 \cdot a}{2 \cdot Es \cdot Ix}$$

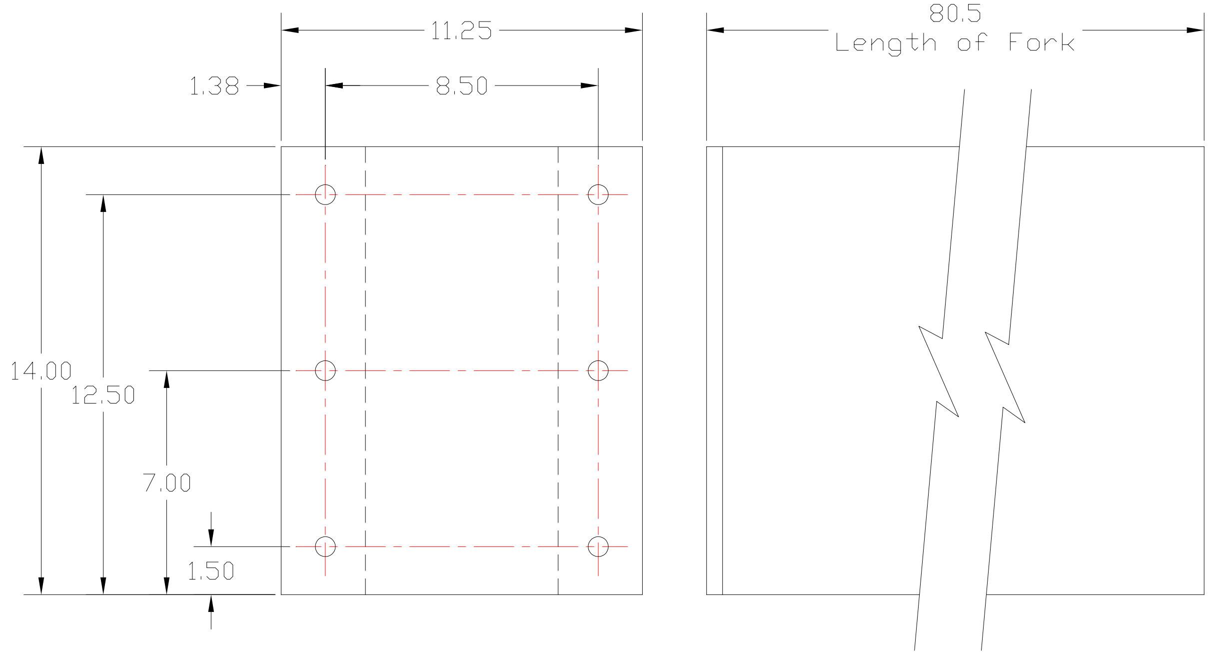
$\Delta fv = 0.51\ in$ Vertical Deflection of Tip of Fork

$$\Delta tv := \frac{qc \cdot d^2 \cdot b^2}{32 \cdot Es \cdot Ix}$$

$\Delta tv = 0.22\ in$ Horizontal Deflection of the table between supports

6.0 Calculations on Fork Design.

The forks will be constructted from rectangular tubes that are 14x6x3/8. The tubes are welded to a 1/2" thick plate which is 11.25" wide and 14" long. This plate has six holes for bolting 5/8" diameter bolts to the block raiser surface. The stresses in the tube, the stresses in the welds of the tube to the plate and the stresses in the bolts have to be calculated. The welds and bolts are subjected to a bending moment as well as a shearing force.



6.1 Stresses in the Rectangular Tube of Fork

$Sxf := 45.3in^3$ Inertia of Tube

$\sigma f := \frac{Mf}{Sxf}$ Maximum Tensile Stress at End of Tube

$\sigma f = 11.11\, ksi$

6.2 Stress in Welds Attaching Tube to 1/2" Plate

$d := 14in$ Length of vertical weld

$b := 6in$ Length of horizontal weld

$Sx := b \cdot d + \frac{d^2}{3}$ Section modulus of weld

$Sx = 149.33\, in^2$

$Mu := 1.4 \cdot Mf$ $Mf = 41.95\, kips \cdot ft$ Factored moment acting on weld

$Rv := \frac{1.4 \cdot Rnf}{2 \cdot d}$ $Rv = 0.63 \frac{kips}{in}$ Factored shear load on weld per unit length of weld (ignore contribution from top/bottom welds)

$Rt := \frac{Mu}{Sx}$ $Rt = 4.72 \frac{kips}{in}$ Factored tensile load on weld per unit length of weld

$Ru := \sqrt{Rv^2 + Rt^2}$ $Ru = 4.76 \frac{kips}{in}$ Factored combined loading on weld per unit length of weld

$\phi w := 0.75$ Strength reduction factor of weld

$Fexx := 70ksi$ Yield strength of weld material

$tw := \frac{Ru}{\phi w \cdot 0.707 \cdot (0.60Fexx)}$ $tw = 0.21\,in$ Required Weld Size

Use tw=0.375" to match the thickness of the tube wall.

For the commonly used grades of teel, only two electrods need to be considered:

E70XX electrodes with steels that have a yield stress of less than 60ksi
E80XX electrodes with steels that have a yield stress of 60ksi or 65ksi

6.3 Analysis of Bolted Connection on Fork

Define Inputs:

$Fu := 50000psi$ Tensile Strength of Plate Material

$Fy := 36ksi$ Yield Strength of Plate Material

$Fub := 120ksi$ Tensile Strength of A325 Bolt Material

$n := 13$ Number of Threads per Inch

$N := 6$ Total Number of Bolts

$db := 0.875in$ Bolt Nominal Diameter

$d := db + \frac{1}{16}in$ Clearance hole in plate for bolt

$m := 1$ Number of Shear Planes

$t := 0.625in$ Thickness of Plate

$Le := 1.75in$ Distance along the line of force from edge of the connected part to the center of hole

$L := 12.25in$ Length of the connection in the direction of shear from the edge of the plate to the last hole

$n3 := 3$ Number of holes in a line in the direction of the shear load

$h := 1.375\,in$ Distance from the edge of the plate to the center of the holes perpendicular to the direction of shear

$\phi s := 0.75$ Strength reduction factor for shear

$\phi f := 0.75$ Strength reduction factor for fracture

$\phi b := 0.75$ Strength reduction factor for bearing

$Tb := 28kips$ Initial Bolt pre-stress used for slip critical connections.

Calculate the factored tensile force in the bolts.
Assume moment is generated about the centroid of the bolts. Calculate the maximum tensile force in top bolt.

$$Pu := \frac{1.4Mf}{2\cdot(10.5in)}$$ Maximum tensile load in bolt

$Pu = 33.56\,kips$

Calculate the shear load per bolt

$$Vu := \frac{1.4\cdot Rnf}{N}$$

$Vu = 2.92\,kips$ Factored shear force per bolt.

Calculation of Areas

$$Ab := \frac{\pi db^2}{4}$$ Gross Cross-Sectional Area of One Bolt

$Ab = 0.60\,in^2$

$$An := 0.785\cdot\left(db - \frac{0.9743in}{n}\right)^2$$ Net Cross-Sectional Area of One Bolt

$An = 0.50\,in^2$

Shear Strength through threads of bolt

$\phi Vn := \phi s\cdot(0.40\cdot Fub)\cdot m\cdot Ab$

$\phi Vn = 21.6\,kips$

Combined Shear and Tension Loading on Bolts

LRFD Table J3.5 lists the design tension stress limit, ϕF'ut, in the presense of the factored shear stress fuv.

Stress variables are F, and f and actual load variables are R. The small f variable represents the stress due to the factored (applied) loads.

$C := 1.3$

C is a constant defined by LRFD to obtain a straight interaction line.

$fuv := \frac{Vu}{Ab}$ $\qquad fuv = 4850.44\ psi$

$\phi F'ut := \phi f \cdot [(0.75 \cdot Fub \cdot C) - (2.5 \cdot fuv)]$

Allowable tensile stress when threads are not in shear plane.

$\phi F'ut = 78.66\ ksi$

$$\phi Rn := \begin{cases} [\phi F'ut \cdot (0.75 \cdot Ab)] & \text{if } \phi F'ut \cdot (0.75Ab) < \phi f \cdot Fub \cdot (0.75 \cdot Ab) \\ [\phi f \cdot Fub \cdot (0.75 \cdot Ab)] & \text{otherwise} \end{cases}$$

$\phi Rn = 35.47\ kips$ Maximum Allowable Tensile Force per Bolt

This is less than Pu (factored tensile load on bolt) so ok.

The value of ϕF'ut has to be less than the maximum value listed in Table J3.5 in LRFD

The tensile is reduced 75% for the root area, 0.8 for the affect of the connection length
Shear strength equals 0.62Fub
(.75*.8*.62=0.40)

Bearing Strength of Plate Material

Bearing Strength of the material is dependent upon the amount of hole deformation that is acceptable and the hole spacing.

$\phi Vn := \phi b \cdot 2.4 \cdot db \cdot t \cdot Fu$

$\phi Vn = 49.2\ kips$

Applies in usual conditions based on deformation limits for the hole. This equation applies for all holse except long slotted holes where the end distance Le is at least 1.5 times the bolt diameter, and the center to center spacing s is at least 3d, and there are two or more bolts in the line of force.

$\phi Vn := \phi b \cdot Le \cdot t \cdot Fu$

Strength limit state for the bolt nearest the edge, according to LRFD formulas J3-1b, J3-2a and J3-2c

$\phi Vn = 41.0\ kips$

Traditionally the center to center spacing of bolts is a minimum of 2 2/3 dimaters and Le is a minimum of 2.67 times the bolt diameter

Minimum Spacing of Bolts

$$s := \frac{Vu}{\phi b \cdot Fu \cdot t} + \frac{db}{2}$$

s is the minimum spacing between bolts and should be more than the calculated value. LRFD recommends 3 bolt diameters and shall not be less than 2 2/3 diameters

$$s = 0.56\,in$$

$$Lmin := \frac{Vu}{\phi b \cdot Fu \cdot t}$$

L is the calculated minimum end distance in the direction of the transmitted force.

$$Lmin = 0.12\,in$$

Calculate Block Shear in Plate

Block shear Failure can occur in two methods: First, by shear yield and tension fracture. Second, shear fracture and tension yield.

$$Agv := t \cdot L$$

Agv is the gross shear area not accounting for holes along the length of the connection. This is in the direction of the tension force.

$$Anv := Agv - [(n3 - .5) \cdot d \cdot t]$$

Anv is the net shear area that subtracts the hole area

$$Anv = 6.19\,in^2$$

$$Agt := h \cdot t$$

Agt is the gross tension area that does not account for the holes. This area is in the direction perpendicular to the direction of tension loading

$$Ant := Agt - \frac{d \cdot t}{2}$$

Anv is the net tension area that subtracts the hole area

$$\phi Ru := \phi f \cdot [(.6 \cdot Fu \cdot Anv) + (Fu \cdot Ant)] \qquad \phi Ru = 160546.87\,lbf$$

Upper Limit AISC

$$\phi R1 := \begin{cases} [\phi f \cdot [(.6 \cdot Fy \cdot Agv) + (Fu \cdot Ant)]] & \text{if } \phi f \cdot [(.6 \cdot Fy \cdot Agv) + (Fu \cdot Ant)] \le \phi Ru \\ \phi Ru & \text{otherwise} \end{cases}$$

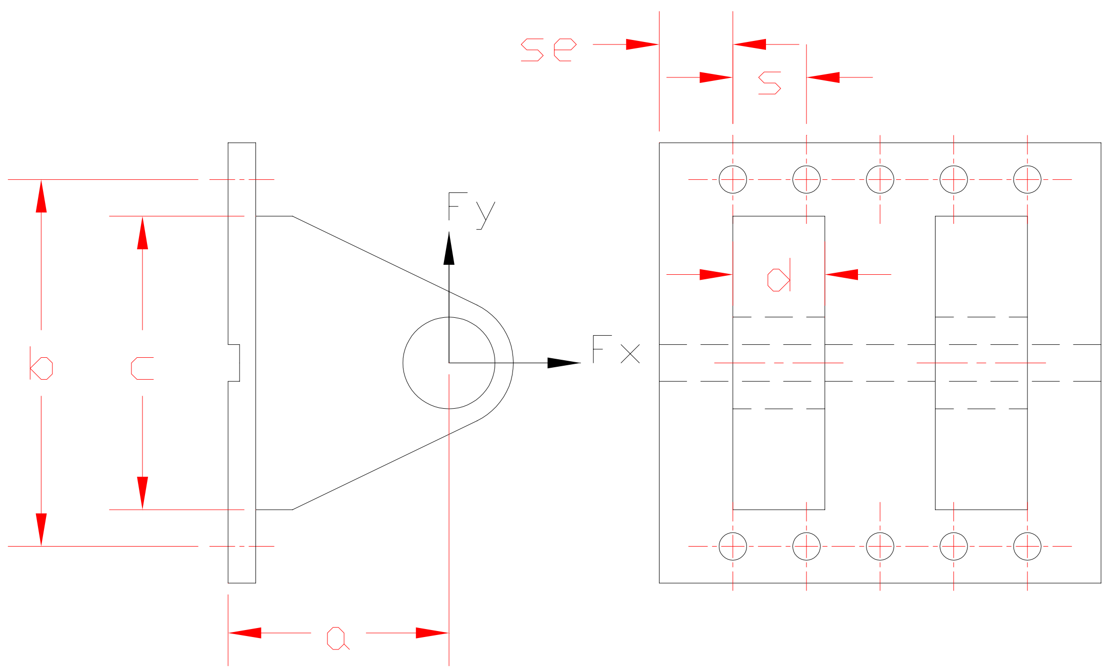
$$\phi R2 := \begin{cases} [\phi f \cdot [(.6 \cdot Fu \cdot Anv) + (Fy \cdot Agt)]] & \text{if } \phi f \cdot [(.6 \cdot Fu \cdot Anv) + (Fy \cdot Agt)] \le \phi Ru \\ \phi Ru & \text{otherwise} \end{cases}$$

$$\phi Rn := \begin{cases} \phi R1 & \text{if } Fu \cdot Ant \ge .6 \cdot Fu \cdot Anv \\ \phi R2 & \text{otherwise} \end{cases}$$

$\phi Rn = 160.55\,\text{kips}$

7.0 Calculation on the Bearing Support Plate

The bearing support plate geometry is shown in the figure below. This plate supports the bearing which allows the BR to rotate. The welds between the gussets housing the bearing and the main plate much be examined, the bolts attaching the Bearing Support Plate to the Table Top and the keyway which carries the shear (Fy) force. The maximum loading on this plate occurs when the BR is in the vertical position because it must support 1/2 of the total weight of the block and table. This plate is designed so that the shearing force, Fy, is carried entirely through a key and that the bolts must only resist the bending moment caused by the application of the force a distance "a" from the bottom of the plate.



7.1 Loads on Bearing Plate

$Fy := 1.4 \cdot 231171.7\,lbf$

Factored shearing force on bearing with BR in Vertical position from Excel spreadsheet calculation of BR forces.

$Fy = 323.64\,kips$

$Fx := 1.4 \cdot 17363\,lbf$

Factored normal force on bearing with BR in Vertical position from Excel Spreadsheet calculation of BR forces

7.2 Calculations on Keyway

$fy := 36\,ksi$

$tk := 0.50in$ — Depth of keyway

$w := 1.0in$ — Width of keyway

$Lk := 24in$ — Length of key

$\sigma b := \frac{Fy}{tk \cdot Lk}$ — $\sigma b = 26.97\,ksi$ — Bearing stress on Key -- less than fy so ok

$\tau k := \frac{Fy}{Lk \cdot w}$ — $\tau k = 13.49\,ksi$ — Shear stress through key -- less than 0.6fy so ok

7.3 Calculation of Welds Between Bearing Gusset and Plate.

$c := 28in$ — Length of vertical weld

$d := 5in$ — Length of horizontal weld

$a := 17in$ — Distance from bearing center to welds

$Sx := c \cdot d + \frac{c^2}{3}$ — Section modulus of weld

$Sx = 401.33\,in^2$

$Mu := \frac{Fy \cdot a}{2}$ — $Mf = 41.95\,kips \cdot ft$ — Factored moment acting on weld

$Rv := \frac{Fy}{4 \cdot c}$ — $Rv = 2.89\,\frac{kips}{in}$ — Factored shear load on weld per unit length of weld (ignore contribution from top/bottom welds)

$Rt := \frac{Mu}{Sx}$ — $Rt = 6.85\,\frac{kips}{in}$ — Factored tensile load on weld per unit length of weld

$Ru := \sqrt{Rv^2 + Rt^2}$ — $Ru = 7.44\,\frac{kips}{in}$ — Factored combined loading on weld per unit length of weld

$\phi w := 0.75$ — Strength reduction factor of weld

$Fexx := 70ksi$ — Yield strength of weld material

$tw := \frac{Ru}{\phi w \cdot 0.707 \cdot (0.60Fexx)}$ — $tw = 0.33\,in$ — Required Weld Size

Use tw=0.5:

7.4 Calculation of Required Bolts

$N := 6$ Number of bolts per row

$db := 0.875in$ Diameter of Bolts

$Fub := 120ksi$ Strength of A325 Bolt Material

$Ab := \frac{\pi db^2}{4}$ Gross Cross-Sectional Area of One Bolt

$Ab = 0.60\,in^2$

$b := 38in$ Distance between rows of bolts

$Ru := \frac{Fy \cdot a}{b \cdot N} + \frac{Fx}{2 \cdot N}$ Tensile Force per bolt

$Ru = 26.16\,kips$

$\phi Rn := \phi f \cdot Fub \cdot (0.75 \cdot Ab)$ Bolt Strength

$\phi Rn = 40.59\,kips$ Since ϕRn is greater than Ru the strength of the bolts is ok.

8.0 Calculations on the Telescoping Tube.

Inputs

$P := 2000psi$ Working Pressure of cylinder

$fy := 50ksi$ Yield stress of cylnder material

$F1 := 75kips$ Extension force

$F2 := 25kips$ Retraction Force

$SFp := 3.0$ Safety factor to resist pressure

$SFb := 5.0$ Safety factor to resist buckling

$Lt := 9ft$ Length of Telescoping Tube Section

8.1 Sizing the smallest section

$Do1 := \sqrt{\frac{4 \cdot F1}{\pi \cdot P}}$ $Do1 = 6.91\,in$ Required Outer Diameter for F1

Calculate the required inner diameter

$Di1 := \sqrt{Do1^2 - \frac{4 \cdot F2}{\pi \cdot P}}$ $\quad Di1 = 5.64\,in$

Calculate required wall thickness:

$t1 := \frac{P \cdot Di1 \cdot SFp}{2 \cdot fy + 2 \cdot P \cdot SFp}$ $\quad t1 = 0.30\,in$ Required wall thickness of inner section

$t1 := 0.5in$

Check buckling of a tube that is Lt long and has a diameter of Di1 and thickness of t1

$I := \pi (Do1 - Di1)^3 \cdot t1$ $\quad I = 3.20\,in^4$ Moment of Inertia of tube

$Pcr := \frac{\pi^2 \cdot Es \cdot I}{Lt^2}$ $\quad Pcr = 78.58\,kips$ Critical buckling Load

8.2 Calculation on wall thicknesses/diameters of sections

$t2 := \frac{P \cdot Do1 \cdot SFp}{2fy}$ $\quad t2 = 0.41\,in$ Required wall thickness to resist internal pressure

$Di2 := 2 \cdot t2 + Do1$ $\quad Di2 = 7.74\,in$ Inner diameter of 2nd section

$Do2 := \sqrt{\frac{4 \cdot F2}{\pi \cdot P} + Di2^2}$ $\quad Do2 = 8.71\,in$ Required outer diameter to provide F2

$t3 := \frac{P \cdot Do2 \cdot SFp}{2 \cdot fy}$ $\quad t3 = 0.52\,in$ Required wall thickness to resist internal pressure

$Di3 := 2 \cdot t3 + Do2$ $\quad Di3 = 9.75\,in$

$Do3 := \sqrt{Di3^2 + \frac{4 \cdot F2}{\pi \cdot P}}$ $\quad Do3 = 10.54\,in$

$t4 := \frac{P \cdot Do3 \cdot SFp}{2 \cdot fy}$ $\quad t4 = 0.63\,in$

$Di4 := 2 \cdot t4 + Do3$ $\quad Di4 = 11.80\,in$

$Do4 := \sqrt{Di4^2 + \frac{4 \cdot F2}{\pi \cdot P}}$ $\quad Do4 = 12.46\,in$