# Fermilab

Particle Physics Division 12-01-01

## Mechanical Department Engineering Note

Number: MD-Eng- 538 Date: 24th February 2015

Project: DESI

Project Internal Reference:

Title: DESI HORIZONTAL LIFTING FIXTURE

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Reviewer(s): R J Woods pages 10 & 11

Key Words: DESI, Below-the-hook lifting device

Abstract/Summary: Calculations to show conformance with ASME B30.20 for lifting fixtures used to move the DESI Corrector Barrel

Applicable Codes: FESHM 10110, ASME B30.20, ASME BTH-1-2014.

Givens:

| | |
|---|---|
| DESI Corrector Barrel | 11,000 lbs |
| Number of Lift Points | 2 |
| Design Load per lift pt. | 5,500 lbs |
| Test Weight = Load= | ~11,000 lbs |

# BELOW-THE-HOOK LIFTING DEVICE
## Engineering Note Cover Page

Lifting Device Numbers:

FNAL Site No. ________ (If applicable) Div. Specific No. ________ (If applicable) Asset No. ________ (If applicable)

ASME B30.20 Group: (check one)

[ X ] Chapter 20-1 Structural and Mechanical Lifting Devices
[ ] Chapter 20-2 Vacuum Lifting Devices
[ ] Chapter 20-3 Close Proximity Operated Lifting Magnets
[ ] Chapter 20-4 Remotely Operated Lifting Magnets
[ ] Chapter 20-5 Scrap and Material Handling Grapples

Device Name or Description DESI – HORIZONTAL LIFTING FIXTURE

Device was (check all applicable)

[ ] Purchased from a Commercial Lifting Device Manufacturer. Mfg Name ________
[X] Designed and Built at Fermilab Assy drawing number F10037250
[ ] Designed by Fermilab and Built by a Vendor. Assy drawing number ________
[ ] Provided by a User or other Laboratory
[ ] Other: Describe ________

Engineering Note Prepared by GIUSEPPE GALLO Date 24TH FEB 2015

Engineering Note Reviewed by ________ Date ________

Lifting Device Data:

Capacity 11,000 LBS

Fixture Weight 3,500

ASME BTH -1 Design Category: (See ASME BTH-1 Section 2-2) [ ] Category A [ X ] Category B

ASME BTH -1 Service Class: (See ASME BTH-1 Section 2-3) [ X ] 0 [ ] 1 [ ] 2 [ ] 3 [ ] 4

Duty Cycle ______ 8, 16 or 24 hour rating (applicable to groups III, and IV)

Inspections Frequency BEFORE EACH USE

Service (refer to B30.20 for definitions) [ X ] normal [ ] heavy [ ] severe

Rated Load Test by FNAL (if applicable) Date ________ Load ________

[ ] Check if Load Test was by Vendor and attach the certificate

Satisfactory Load Test Witnessed by: ________

Signature (of Load Test Witness) ________

Notes or Special Information:

# TEST

The Horizontal Lifting Fixture #F10037250 need to be tested according with the procedure listed on FESHM #10110. The dummy load need to be bolted to the support ring through #30 3/4 -10 bolts while the entire stand is placed horizontally as shown on figure.1. The figure.1 shows the Horizontal Lifting Fixture sitting on the 4 support feet bolted to the support ring and the dummy load attached to the ring and the two lifting points installed on the side of the support ring. The two lifting points define the axis of rotation of the entire system. Before any rotation the entire system need to be stabilized in order to minimize the residual moment produced by the residual offset between the center-of-gravity of the total mass and the axis of rotation. Addition dummy weights might needed to be placed to the left or the right side of the support ring and equally distributed on the four legs. Preliminary lifting steps are required to make sure the center-of-gravity of the system lie on the axis of rotation defined by the two side lifting points. The test need to be performed for both vertical and horizontal configurations repeating multiple times the flipping process from the vertical to the horizontal configuration and viceversa.

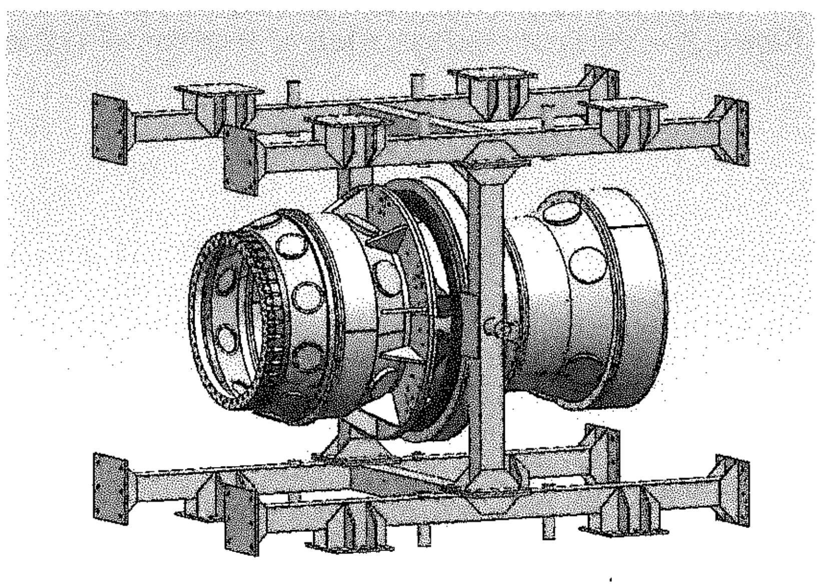


Figure.1 – Horizontal configuration of Horizontal Lifting Fixture during the installation of the dummy load

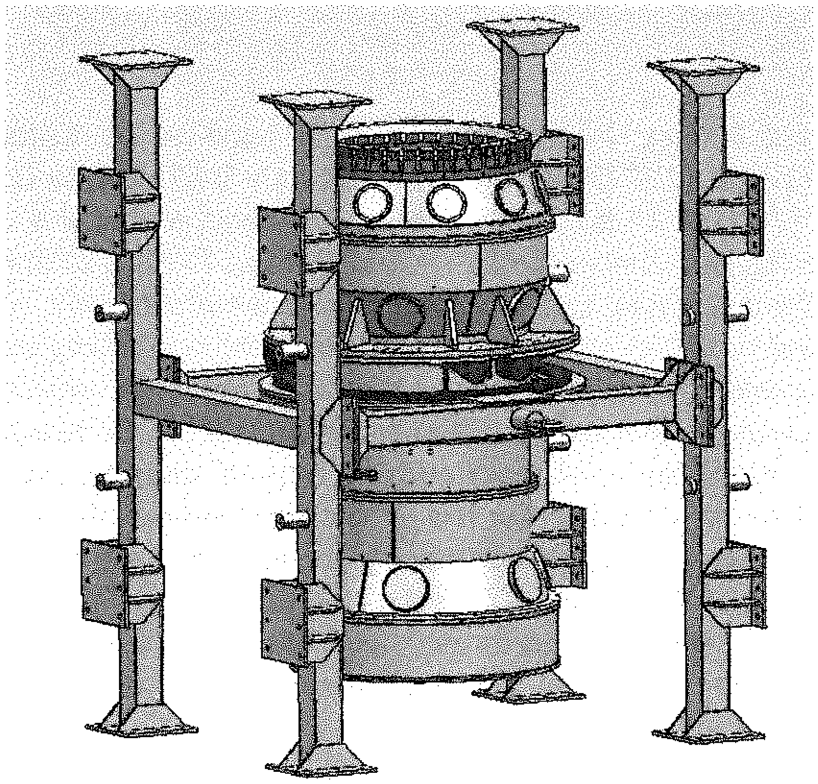


Figure.2 – Vertical configuration of Horizontal Lifting Fixture

# Introduction.

The Horizontal Lifting Fixture will be used to lift the DESI corrector barrel while it is onto the horizontal position, see Appendix-I. The same lifting fixture will be used to flip the barrel from horizontal to vertical position, see Appendix I-II.

Both figures show the location of lifting points. The main structure of the Horizontal Lifting Fixture includes (4) columns "BEAM 3" and (4) cross beam "BEAM 1-2". The columns and the cross beam are connected through bolted/weld joints as shown on the Appendix I-II. To the cross beam a stiffener ring is welded and a 5/8 thick plate connect the "BEAM 1 " to the stiffener ring.

The columns, the cross beams, the bolt/weld joints, the lift points are analyzed according with the procedure listed into FESHM 10110.

## HORIZONTAL CONFIGURATION

The Desi Lifting Fixture will carry 11,000 lbslocated at 3.2 inches from the cross beam "BEAM 1" and the two lift points will be aligned to the CoG of the system which includes the mass of the Barrel, the mass of the dummy load on the Focal Plane flange and the mass of the lifting fixture.

$M\_barrel := 9900 lbf$ mass of the corrector barrel shell

$M\_dummy := 1100 lbf$ mass of the dummy load installed on the FP flange

$M\_fixture := 3500 lbf$ mass of the lifting fixture

$M\_tot := (M\_barrel + M\_dummy + M\_fixture)$ total mass

$M\_tot = 14.5 \times 10^3 lbf$

$R\_x := \frac{M\_tot}{2} \cdot \cos\left(\frac{60}{57.3}\right)$

reaction force calculated on the lift points assuming a lift angle of 60 degree.

$R\_y := \frac{M\_tot}{2} \cdot \sin\left(\frac{60}{57.3}\right)$

$R\_x = 3.6 \times 10^3 lbf$ $R\_y = 6.3 \times 10^3 lbf$

$COG\_horiz\_conf := \frac{M\_dummy \cdot 49.5in}{M\_tot}$ location of COG on the worst case scenario

$\frac{COG\_horiz\_conf}{in} = 3.8$

WELD JOINT - 1

The weld joint 1 is a 1/8" fillet weld.

$l\_weld\_1 := 37.4in$ overall length of the weld joint

$th\_weld\_1 := 0.125in$

$A\_shear\_weld1 := l\_weld\_1 \cdot th\_weld\_1 \cdot (.707)$ area subject to shear stress

$$\frac{A\_shear\_weld1}{in^2} = 3.3$$

$F\_shear\_weld\_1 := Fv\_weld \cdot A\_shear\_weld1$ allowable shear strength

$$F\_shear\_weld\_1 = 38.6 \times 10^3\, lbf$$

$A\_tens\_weld1 := A\_shear\_weld1$

$$F\_tens\_weld\_1 := \frac{Fy\_A36}{Nd} \cdot A\_tens\_weld1$$

$$F\_tens\_weld\_1 = 40.0 \times 10^3\, lbf$$

$$\left(\frac{\frac{R\_x}{A\_shear\_weld1}}{Fv\_weld}\right) + \left(\frac{\frac{R\_y}{A\_tens\_weld1}}{\frac{Fy\_A36}{Nd}}\right) = 251.0 \times 10^{-3}$$ combined stress

Since the combines stress (0.251) is less than 1 the weld joint 1 is ok.

BOLT JOINT - 1

The connection between the column and the cross bar is made with #6 3/4 A307 or grade 2 bolt.

$A\_bolt\_tension := 0.442in^2$ 3/4 bolt cross section

$$f\_bolt\_tension := \frac{R\_y}{6 \cdot A\_bolt\_tension}$$ tension stress

$$f\_bolt\_tension = 2.4 \times 10^3\, psi$$

$$f\_bolt\_shear := \frac{R\_x}{6 \cdot A\_bolt\_tension}$$ shear stress

$$f\_bolt\_shear = 1.4 \times 10^3\, psi$$

$$Ft\_combined\_bolt := \sqrt{(Ft\_bolt)^2 - \left(2.6 \cdot f\_bolt\_shear^2\right)}$$

allowable stress for bolted joints subjected to tension combined with the actual shear stress

$$Ft\_combined\_bolt = 16.7 \times 10^3 psi$$

Since the tension stress calculated on the bolt joint (2.4 ksi) is less than the allowable combined stress (16.7 ksi) the bolt joint is ok.

Assume A307 bolts or grade 2. Then from TABLE I-A, page 4-3 of the AISC 18th edition, the allowable load per bolt is 2900 lbf which is greater than the actual loading 1230 lbf per bolt. According with those results the bolted connections are fine.

BEAM 1

The BEAM 1 is a square tube 6"x6"x.25" with an overall lenght of 77 inches and it support the stiffener ring through a weld joint as shown in Figure.3.

$$f\_beam\_1\_tension := \frac{R\_y}{A\_st}$$

tension stress

$$f\_beam\_1\_tension = 1.1 \times 10^3 psi$$

$$f\_beam\_1\_flexure\_x1 := \frac{R\_y \cdot 3.2in}{I\_st} \cdot \frac{b\_st}{2}$$

flexure stress due to R_x

$$f\_beam\_1\_flexure\_x1 = 2.0 \times 10^3 psi$$

$$\frac{f\_beam\_1\_tension}{Ft\_gross\_area} + \frac{f\_beam\_1\_flexure\_x1}{Fb\_st} = 191.2 \times 10^{-3}$$

Since the sum of the ratio of combined stresses (tension & flexure stress is less then 1, the BEAM 1 is ok.

BEAM 2

The BEAM 2 is a square tube 6"x6"x.25" with an overall lenght of 61.3 inches subject to a compression load R_x= 8,500 lbf where KL is 61.3 inches.

$$f\_compression\_beam\_2 := \frac{R\_x}{A\_st}$$

$$f\_compression\_beam\_2 = 648.6 psi$$

Since the actual compression stress 649psi is less than the allowable stress 14.2ksi the BEAM 2 is ok.

WELD JOINT - 2

The weld joint 2 is a 1/8" fillet weld.

$l\_weld\_2 := 32in$ overall length of the weld joint

$th\_weld\_2 := 0.125in$

$A\_compr\_weld2 := l\_weld\_2 \cdot th\_weld\_2 \cdot (.707)$

$$F\_tens\_weld\_2 := \frac{Fy\_A36}{Nd} \cdot A\_compr\_weld2$$

$$F\_tens\_weld\_2 = 34.2 \times 10^3 \, lbf$$

Since the allowable compression strenght of the weld joint 2 is greater than R_y= 6,300 lbf the weld joint 2 is fine.

WELD JOINT - 3

$l\_weld\_3 := 38.6in$

$th\_weld\_3 := .1875in$ 3/16 max weld thick for tube (AISC P.5-67)

$A\_shear\_weld\_3 := l\_weld\_3 \cdot th\_weld\_3 \cdot (0.707)$

$F\_shear\_weld\_3 := Fv\_weld \cdot A\_shear\_weld\_3$ allowable shear strength

$$F\_shear\_weld\_3 = 59.7 \times 10^3 \, lbf$$

Since the allowable shear strength (59,700 lbf) of the weld joint 3 is greater than the actual shear strenght (14,500 lbf) the weld joint 3 is ok

$M\_weld\_3 := M\_tot \cdot COG\_horiz\_conf$

$$f\_str\_weld\_3 := \frac{M\_weld\_3}{14.6in}$$ actual tensile/compression strenght

$$f\_str\_weld\_3 = 3.7 \times 10^3 \, lbf$$

$A\_tension\_weld\_3 := 12in \cdot th\_weld\_3$

$$Ft\_weld\_3 := \frac{Fy\_A36}{Nd} \cdot A\_tension\_weld\_3$$ allowable tension strength on weld joint 3

$$Ft\_weld\_3 = 27.2 \times 10^3\, lbf$$

Since the allowable tension strength (27,200 lbf) of the weld joint 3 is greater than the actual shear strenght (1,590 lbf) the weld joint 3 is ok

LIFTING ROD

$$d\_lift\_cyl := 3.5in$$ overall diameter of the rod

$$A\_shear\_weld\_4 := \pi \cdot \frac{d\_lift\_cyl^2}{4}$$ shear section

$$f\_rod\_shear := \frac{M\_tot}{2 \cdot A\_shear\_weld\_4}$$ actual shear on the lifting rod

$$f\_rod\_shear = 753.5\, psi$$

$$f\_rod\_tension := \frac{M\_tot}{2} \cdot 6.5in \cdot \frac{d\_lift\_cyl}{2} \cdot \frac{64}{\pi \cdot d\_lift\_cyl^4}$$ actual stress due by bending

$$f\_rod\_tension = 11.2 \times 10^3\, psi$$

$$fcr\_rod := \sqrt{f\_rod\_tension^2 + 3 \cdot f\_rod\_shear^2}$$

$$fcr\_rod = 11.3 \times 10^3\, psi$$

$$Facr\_rod := \frac{Fy\_A36}{Nd}$$

$$Facr\_rod = 12.1 \times 10^3\, psi$$

Since the computed stress (11.3 ksi) is less than the allowable critical stress (12.1) ksi the 3.5" lifting rod is ok.

WELD JOINT - 4

$l\_weld\_4 := \pi 4in + \pi \cdot 3in$

$th\_weld\_4 := .3125in$ 5/16 weld thick for lifting rod

$A\_shear\_weld\_4 := l\_weld\_4 \cdot th\_weld\_4 \cdot (0.707)$

$F\_shear\_weld\_4 := Fv\_weld \cdot A\_shear\_weld\_4$ allowable shear strength

$F\_shear\_weld\_4 = 56.7 \times 10^3 \, lbf$

Assuming during the lifting step the crane rope will have an angle of 20 deg respect to the gravity vector

$$f\_shear\_weld\_4 := \frac{M\_tot}{2} \cdot \sin\left(\frac{20}{57.3}\right)$$ actual shear force on the weld

$f\_shear\_weld\_4 = 2.5 \times 10^3 \, lbf$

Since the allowable shear strength (56,700 lbf) of the weld joint 4 is greater than the actual shear strenght (2,500 lbf) the weld joint 4 is ok

## VERTICAL CONFIGURATION

The Desi Lifting Fixture will carry 11,000 lbslocated at 3.8 inches from the cross beam "BEAM 1" and the two lift points will be located on the top side of the column beam "BEAM 3". During the lifting process the Desi fixture will tilt sitting on one pair of feet and for the structural point of view the worst case scenario will be when the lift point will be aligned with the CoG of the system. This configuration will happen when the lifting fixture will have a tilt of 54 degree respect to the ground. The total mass of the system include the mass of the Barrel, the mass of the dummy load on the Focal Plane flange and the mass of the lifting fixture.

BEAM 3

The BEAM 3 is a square tube 6"x6"x.25" with an overall lenght of 128 inches and it supports the stiffener ring through a weld joint as shown in Appendix II. The lift points will be located 10inches above the CoG of the system

$$f\_beam\_3v\_tension := \frac{R\_v}{A\_st} \cdot \cos\left(\frac{36}{57.3}\right)$$ tension stress

$$f\_beam\_3v\_tension = 908.7\,psi$$

$$f\_beam\_3v\_flexure\_x1 := \frac{R\_v \cdot 13.2in}{I\_st} \cdot \sin\left(\frac{36}{57.3}\right) \cdot \frac{b\_st}{2}$$ flexure stress due to R_x

$$f\_beam\_3v\_flexure\_x1 = 4.8 \times 10^{3}\,psi$$

$$f\_beam\_3v\_flexure\_x2 := \frac{R\_x \cdot 13.2in}{I\_st} \cdot \frac{b\_st}{2}$$ flexure stress due to R_x

$$f\_beam\_3v\_flexure\_x2 = 4.7 \times 10^{3}\,psi$$

$$\frac{f\_beam\_3v\_tension}{Ft\_gross\_area} + \frac{f\_beam\_3v\_flexure\_x1 + f\_beam\_3v\_flexure\_x2}{Fb\_st} = 626.1 \times 10^{-3}$$

Since the sum of the ratio of combined stresses (tension & flexure stress is less then 1, the BEAM 3 is ok.

BOLT JOINT - 1

The connection between the column and the cross bar is made with #6 3/4-10 bolt.

$$f\_bolt\_tension\_h := \frac{R\_y \cdot \cos\left(\frac{36}{57.3}\right)}{6 \cdot A\_bolt\_tension}$$

tension stress

$$f\_bolt\_tension = 2.4 \times 10^{3}\,psi$$

$$f\_bolt\_shear\_h\_x1 := \frac{R\_x}{6 \cdot A\_bolt\_tension}$$

shear stress

$$f\_bolt\_shear\_h\_x1 = 1.4 \times 10^{3}\,psi$$

$$f\_bolt\_shear\_h\_x2 := \frac{R\_y \cdot \cos\left(\frac{36}{57.3}\right)}{6 \cdot A\_bolt\_tension}$$

shear stress

$$f\_bolt\_shear\_h\_x2 = 1.9 \times 10^{3}\,psi$$

$$Ft\_combined\_bolt\_h := \sqrt{Ft\_bolt^{2} - 2.6 \cdot (f\_bolt\_shear\_h\_x1 + f\_bolt\_shear\_h\_x2)^{2}}$$

allowable stress for bolted joints subjected to tension and shear stress

$$Ft\_combined\_bolt\_h = 16.0 \times 10^{3}\,psi$$

Since the actual tension stress calculated on the bolt joint (2.4 ksi) is less than the allowable combined stress (16.0 ksi) the bolt joint is ok.
The total capacity of a two bolt vertical line on each side of the square tube of 3/4 inch bolts has a maximum total capacity of the joint of 26.5 kips which is greater than the actual total load of 4.0 kips

BEAM 1

The BEAM 1 is a square tube 6"x6"x.25" with an overall lenght of 77 inches and it supports the stiffener ring through a weld joint as shown in Figure.x.

$$f\_beam\_1\_tension\_h := \frac{R\_y \cdot \sin\left(\frac{36}{57.3}\right)}{A\_st}$$

tension stress

$$f\_beam\_1\_tension\_h = 660.1\,psi$$

$$f\_beam\_1\_flexure\_h\_x1 := \frac{R\_y \cdot \cos\left(\frac{36}{57.3}\right) \cdot 13.2in}{I\_st} \cdot \frac{b\_st}{2}$$ flexure stress due to R_x

$$f\_beam\_1\_flexure\_h\_x1 = 6.6 \times 10^{3} psi$$

$$\frac{f\_beam\_1\_tension\_h}{Ft\_gross\_area} + \frac{f\_beam\_1\_flexure\_h\_x1}{Fb\_st} = 436.6 \times 10^{-3}$$

Since the sum of the ratio of combined stresses (tension & flexure stress is less then 1, the BEAM 1 is ok.

END PLATE DESIGN

The maximum actual tensile strength is 7,400 lbf which is smaller than the allowable tensile strength 86,400 lbf. Also the maximum actual shear strength 10,300 lbf is smaller than the allowable shear strength 35,200 lbf. So the plate design is ok.

# APPENDIX-III (Allowable stress)

3.2 MEMBER DESIGN - BASED ON ASME BTH-1-2014

Allowable stresses

$Nd := 3$ nominal design factor - category B

Mechanical Properties of ASTM A500-B

$Fy\_A500B := 46ksi$ Tensile Strength, Yield

$Fu\_A500B := 58ksi$ Ultimate Tensile Strength

Mechanical Properties of ASTM A36

$Fy\_A36 := 36.3ksi$ Tensile Strength, Yield

$Fu\_A36 := 58ksi$ Ultimate Tensile Strength

$E := 29000ksi$ Modulus of Elasticity

Dimensions and geometrical properties of square tube - 6"x6"x.25"

$b\_st := 6in$ width of square tube

$th\_st := .25in$ thickness of the square tube

$ry := 2.33in$ minor axis radius of gyration

$I\_st := 30.3in^4$ major axis moment of inertia

$S\_st := \frac{I\_st}{b\_st} \cdot 2$ major axis section modulus

$A\_st := 5.59in^2$ cross-sectional area

$Z\_st := 11.9in^3$ major axis plastic modulus

$J\_st := 45.6in^4$ torsional constant

$Mp\_st := Fy\_A500B \cdot Z\_st$

plastic moment </= 1.5FySx

$$\frac{Mp\_st}{ksi \cdot in^3} = 547.4$$

$Mp\_t := 1.5 \cdot Fy\_A500B \cdot S\_st$

$$\frac{Mp\_t}{ksi \cdot in^3} = 696.9$$

The plastic moment is less than 1.5*Fy*Sx.

(3-2.1) Tension members

$$Ft\_gross\_area := \frac{Fy\_A500B}{Nd}$$

$$Ft\_gross\_area = 15.3 \times 10^{3}\,psi$$

$$Ft\_net\_area := \frac{Fu\_A500B}{1.2 \cdot Nd}$$

$$Ft\_net\_area = 16.1 \times 10^{3}\,psi$$

(3-2.2) Compression Members

$$Cc := \sqrt{\frac{2 \cdot \pi^{2} \cdot E}{Fy\_A500B}}$$

$$Cc = 111.6$$

$KL\_st := 6ft$ (worst case scenario)

$$KL\_st\_ry := \frac{KL\_st}{ry} \qquad KL\_st\_ry = 30.9$$

The KL/r is less than Cc so according with ASME BTH-1-2014, we can use the following equation to calculate the allowable stress.

$$Fa := \frac{\left(1 - \frac{KL\_st\_ry^{2}}{2 \cdot Cc^{2}}\right) \cdot Fy\_A500B}{Nd \cdot \left(1 + \frac{9 \cdot KL\_st\_ry}{40 \cdot Cc} - \frac{3 \cdot KL\_st\_ry^{3}}{40 \cdot Cc^{3}}\right)}$$

allowable stress for compression

$$Fa = 13.9 \times 10^{3}\,psi$$

(3-2.3.1) Allowable stress for Flexural Members - Major Axis Bending of Compact Sections

$$Fb\_st := \frac{1.10 \cdot Fy\_A500B}{Nd}$$

allowable stress for flexural members

$$Fb\_st = 16.9 \times 10^{3}\,psi$$

$$Lp := \frac{0.13 \cdot ry \cdot E}{Mp\_st} \cdot \sqrt{J\_st \cdot A\_st}$$

laterally braced intervals

$$\frac{Lp}{in} = 256.2$$

(3-2.3.6) Shear on Bars, Pins, and Plates

$$Fv := \frac{Fy\_A36}{Nd \cdot \sqrt{3}}$$

allowable shear stress

$$Fv = 7.0 \times 10^{3}\,psi$$

CONNECTION DESIGN
3-3.2 Bolted Connections

$Fu\_A307 := 60.6ksi$ specified minimum tensile strength of the bolt

$$Ft\_bolt := \frac{Fu\_A307}{1.2 \cdot Nd}$$

allowable tensile stress for bolted joints with a minimum of 2 A307 bolts

$Ft\_bolt = 16.8 \times 10^{3} psi$

$$Fv\_bolt := \frac{0.62 \cdot Fu\_A307}{1.2 \cdot Nd}$$

allowable shear stress for bolted joints

$Fv\_bolt = 10.4 \times 10^{3} psi$

WELDED CONNECTIONS
3-3.4 General

$E70\_Fy := 70ksi$

$$Fv\_weld := \frac{0.6 \cdot E70\_Fy}{1.2 \cdot Nd}$$

allowable stress on weld joints

$Fv\_weld = 11.7 \times 10^{3} psi$

PLATES DESIGN
3-3.3.1 Static Strength of the Plates

$t\_plate := .625in$ plate thickness

$b\_eff := 5in$ effective width to each side of the pinhole

$Dh := .875in$ hole diameter

$Dp := .75in$ pin diameter

$$Cr\_plate := 1 - 0.275 \cdot \sqrt{1 - \frac{Dp^2}{Dh^2}}$$

$Cr\_plate = 858.4 \times 10^{-3}$

$$Pt\_plate := Cr\_plate \cdot \frac{Fu\_A36}{1.2 \cdot Nd} \cdot 2 \cdot t\_plate \cdot b\_eff$$

allowable tensile strength

$Pt\_plate = 86.4 \times 10^{3} lbf$

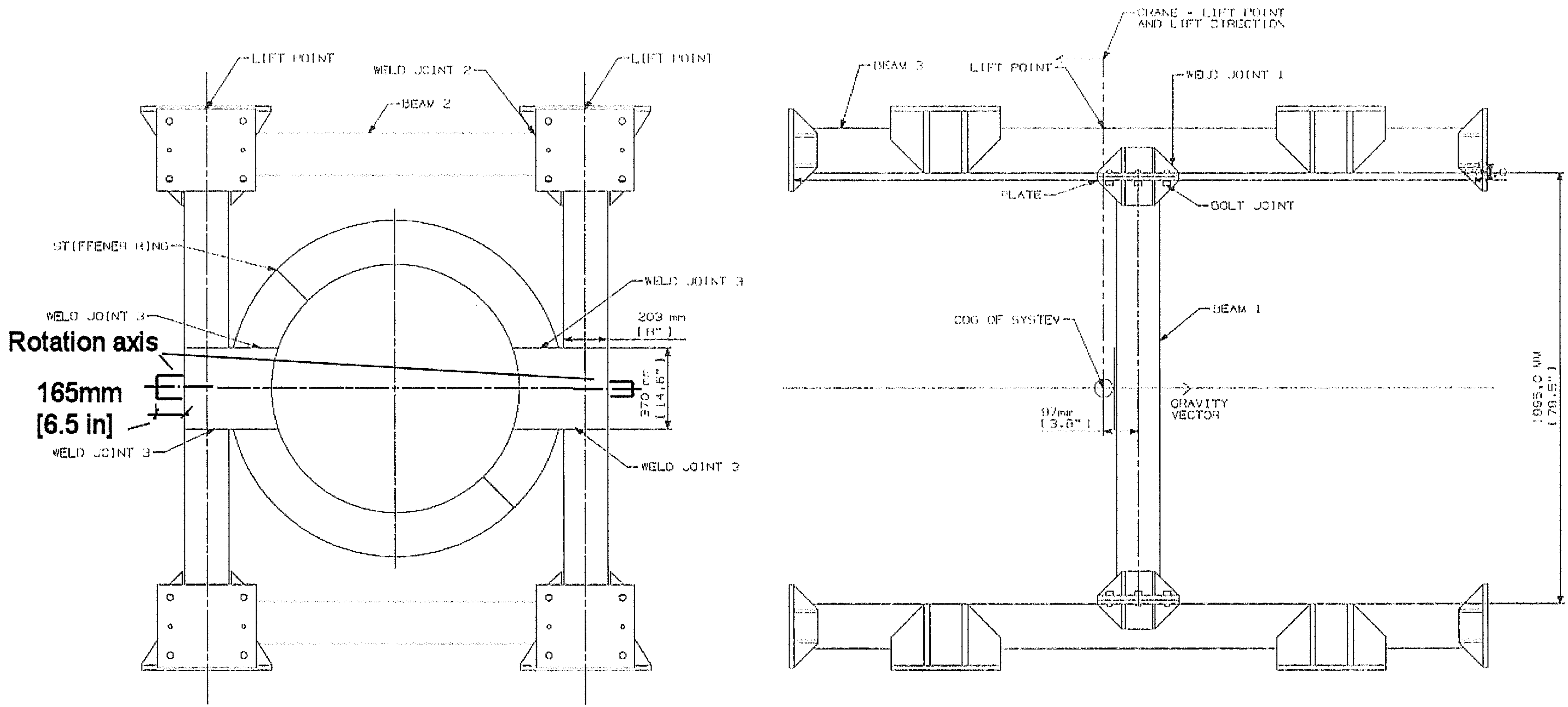
$a\_plate := 1.75in$ distance from the edge of the pinhole to the edge of the plate

$$Av\_plate := 2 \cdot \left[a\_plate + \frac{Dp}{2} \cdot \left(1 - \cos\left(55 \cdot \frac{Dp}{Dh} \cdot \frac{360}{2 \cdot \pi}\right)\right)\right] \cdot t\_plate$$

$$\frac{Av\_plate}{in^2} = 2.3$$

$$Pv\_plate := \frac{0.7 \cdot Fu\_A36}{1.2 \cdot Nd} \cdot Av\_plate$$

$$Pv\_plate = 25.9 \times 10^3 \, lbf$$




DESI LIFTING FIXTURE - VERTICAL CONFIGURATION
G.GALLO