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Engineering Note
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Department	Date	STAR WBS#
Detector Mechanical Engineering	11/3/98	4.2.10

Project

Solenoidal Tracker at RHIC (STAR)

Subproject

Time Projection Chamber - Installation

Title

Structural Analysis of Support Columns

1.0 Introduction

The support columns are used during installation of the TPC into the magnet. They are comprised of a box beam with two welded on flanges on the ends. The support columns are vertically mounted to the end flanges of the magnet, straddling the magnet's horizontal midplane. The installation beams are bolted to the center of the support columns, and resting on a support bracket mounted off the support column. The support columns must be able to take the weight of the TPC, the installation beams, and the guide trolley. (See Appendix A.)

2.0 The Loads:

Weight of TPC at installation	$W_{TPC} := 18172$
Weight of one installation beam, between columns	$W_{beam} := 2150$
Weight of the guide trolley	$W_{trolley} := 520$

There are two load cases that are going to be considered. (See Appendix B)

Load Case 1: When the TPC is rolled in to its final position, each support column is supporting 1/4 of its weight. The weight of each installation beam is supported by a support column at either end and the guide trolley will be supported by the western pair of support columns.

$$W_1 := \frac{W_{TPC}}{4} + \frac{W_{beam}}{2} + \frac{W_{trolley}}{2} \qquad W_1 = 5.878 \cdot 10^3$$

Load Case 2e: East Columns. As the TPC rolls in half of its weight is brought to bear on the middle of the Installation Beams and the other half is near/over the the east columns. Each east Support Column will take 3/8 of the TPC weight plus half the Installation Beam and half of the trolley weight.

$$W_e := \frac{3 \cdot W_{TPC}}{8} + \frac{W_{beam}}{2} + \frac{W_{trolley}}{2} \qquad W_e = 8.15 \cdot 10^3$$

Load Case 2w: West Columns. As the TPC rolls into the magnet the vertical reaction at the west column increases. At the point where the TPC is centered in the magnet the vertical reaction force is 1/8 of the TPC weight plus 1/2 of the trolley and installation beam weights. In addition the bolts holding the beam to the column in conjunction with the bolts holding the support plate must compensate for the end moment of the beam.

$$W_w := \frac{W_{TPC}}{8} + \frac{W_{beam}}{2} + \frac{W_{trolley}}{2} \qquad W_w = 3.607 \cdot 10^3$$

The length of the beam between the column supports is:

$$L_{beam} := 295.6$$

The moment at the west end of the beam for a beam with one fixed end and one simply supported end is:

$$M := \frac{3}{16} \cdot \left[\frac{W_{TPC}}{8} + \frac{W_{trolley}}{2} \right] \cdot L_{beam} \qquad M = 1.403 \cdot 10^5$$

Note, the beam's weight is not included in the moment calculation because it is secured in place in its deformed state.

3.0 Support Column to Installation Beam Screws:

The Installation Beams are supported directly by four 3/4" steel socket head cap screws at each end. These screw tie the beam directly to the support column. The Installation Beam is also supported by an angle bracket from underneath on the West end and a simple plate on the East end. The angle bracket is bolted onto two bosses on the support column by seven 3/4" steel socket head cap screws. The simple plate is bolted onto a single boss with four 3/4" steel socket head cap screws.

Load Case 1e-East End, Assuming that the four bolts used to tie the installation beam to the support column are not taking any of the vertical load, but only keeping the installation beam from tipping over, then the weight of the different assemblies in Load Case 1 must be fully supported by the support bracket. The support bracket will take the load in direct shear plus an indirect shear caused by the load being applied at a distance offset from the centroid of the support bracket's bolt pattern.

$N_{bolts} := 4 \qquad A_{bolt} := .334 \qquad \sigma_{ult} := 150000$ Grade 8 fastener tensile strength

$$\tau_{bolt} := \frac{W_1}{N_{bolts} \cdot A_{bolt}} \qquad \tau_{bolt} = 4.4 \cdot 10^3$$

The centroid of the bolt pattern is directly in the center of the 4" x 5" bolt pattern. Since this is a bolt pattern that is symmetrical about its centroid, each bolt will take an equal share of the indirect shear. The direction of the shear stress acts perpendicular to the line drawn from the centroid of the bolt pattern to the bolt center. For two of the bolts in the bolt pattern the secondary shear will be additive and for the other two it will be subtractive.

The applied load offset length is: $x_{offset} := 6$

The resultant moment is: $M_{offset} := W_{1} \cdot x_{offset}$ $\quad M_{offset} = 3.527 \cdot 10^4$

The offset moment is resisted by the bolts. The distance between the centroid of the bolt pattern and each bolt is:

$$d_{bolt} := \frac{\sqrt{4^2 + 5^2}}{2} \qquad d_{bolt} = 3.202$$

The reaction force for each bolt is:

$$F_r := \frac{M_{offset}}{N_{bolts} \cdot d_{bolt}} \qquad F_r = 2.754 \cdot 10^3$$

The secondary shear is then:

$$\tau_2 := \frac{F_r}{A_{bolt}} \qquad \tau_2 = 8.245 \cdot 10^3$$

The secondary shear does not act in the same direction as the primary shear so it must be resolved in to x and y components and then recombined.

The angle at which the secondary shear acts is;

$$\alpha := \operatorname{atan}\left[\frac{4}{5}\right]$$

$$\tau_{sum} := \sqrt{\left(\tau_2 \cdot \cos(\alpha)\right)^2 - \left(\tau_2 \cdot \sin(\alpha) + \tau_{bolt}\right)^2} \qquad \tau_{sum} = 1.152 \cdot 10^4$$

Allowable shear is roughly 1/2 tensile

$$SF := \frac{\sigma_{ult} \cdot .5}{\tau_{sum}} \qquad SF = 6.511$$

Load Case 1w-West end.

Load Case 1w-West End, Assuming that the four bolts used to tie the installation beam to the support column are not taking any of the vertical load, but only keeping the installation beam from tipping over, then the weight of the different assemblies in Load Case 1 must be fully supported by the support angle bracket. The support angle bracket will take the load in direct shear plus an indirect shear caused by the load being applied at a distance offset from the centroid of the support angle bracket's bolt pattern.

$N_{bolts} := 7$ $\quad A_{bolt} := .334$ $\quad \sigma_{ult} := 150000$ Grade 8 fastener tensile strength

$$\tau_{bolt} := \frac{W_1}{N_{bolts} \cdot A_{bolt}} \qquad \tau_{bolt} = 2.514 \cdot 10^3$$

In Side View of the assembly, the support angle bracket has 5 3/4" bolts that will resist the moment caused by the load being applied by an offset distance from the centroid of the bolt pattern. The centroid of the bolt pattern is located at the middle bolt. The bolts furthest from the centroid will take a greater share of the indirect shear. The direction of the shear stress acts perpendicular to the line drawn from the centroid of the bolt pattern to the bolt center. In this case the shear will act in a horizontal direction.

The applied load offset length is: $x_{offset} := 2.75$

The resultant moment is: $M_{offset} := W_1 \cdot x_{offset}$ $\quad M_{offset} = 1.616 \cdot 10^4$

The offset moment is resisted by the bolts. The distances between the centroid of the bolt pattern and the furthest bolts and the second furthest bolts are: $d_{bolt1} := 6$ $\quad d_{bolt2} := 3$

$d_{bolt3} := 0$ The middle bolt in the pattern will contribute nothing to resisting the offset moment because it is at the centroid of the bolt pattern.

The reaction force for the furthest bolts is:

$$F_r := \frac{M_{offset} \cdot d_{bolt1}}{2 \cdot d_{bolt1}^2 + 2 \cdot d_{bolt2}^2 + d_{bolt3}^2} \qquad F_r = 1.078 \cdot 10^3$$

The secondary shear is then: $\tau_2 := \dfrac{F_r}{A_{bolt}}$ $\quad \tau_2 = 3.226 \cdot 10^3$

The secondary shear does not act in the same direction as the primary shear so it must be resolved in to x and y components and then recombined.

$$\tau_{sum} := \sqrt{\tau_2^2 + \tau_{bolt}^2} \qquad \tau_{sum} = 4.09 \cdot 10^3$$

Allowable shear is roughly 1/2 tensile

$$SF := \frac{\sigma_{ult} \cdot .5}{\tau_{sum}} \qquad SF = 18.336$$

Load Case 2e-East

For Load Case 2, the East side columns, are supporting a greater load than in Load Case 1, so reevaluating the results from load case 1, the maximum shear in the support column to installation beam bolts can be found to be:

$W_e = 8.15 \cdot 10^3$ $\qquad W_1 = 5.878 \cdot 10^3$ $\qquad N_{bolts} := 4$

$\tau_{bolt} := \dfrac{W_e}{N_{bolts} \cdot A_{bolt}}$ $\qquad \tau_{bolt} = 6.1 \cdot 10^3$

$\tau_2 := \dfrac{F_r}{A_{bolt}} \cdot \dfrac{W_e}{W_1}$ $\qquad \tau_2 = 4.473 \cdot 10^3$

$\tau_{sum} := \sqrt{\left(\tau_2 \cdot \cos(\alpha)\right)^2 - \left(\tau_2 \cdot \sin(\alpha) + \tau_{bolt}\right)^2}$ $\qquad \tau_{sum} = 9.556 \cdot 10^3$

Allowable shear is roughly 1/2 tensile

$SF := \dfrac{\sigma_{ult} \cdot .5}{\tau_{sum}}$ $\qquad SF = 7.849$

Load Case 2w-West

Assuming that the four bolts used to tie the installation beam to the support column are not taking any of the vertical load, but only keeping the installation beam from tipping over, then the weight of the different assemblies in Load Case 2w must be fully supported by the support angle bracket. The support angle bracket will take the load in direct shear plus an indirect shear caused by the load being applied at a distance offset from the centroid of the support angle bracket's bolt pattern. The four installation beam bolts will under go a shear load due to the installation beam bending. First, the support angle bracket bolts will be looked at:

$N_{bolts} := 7$ $\qquad A_{bolt} := .334$ $\qquad \sigma_{ult} := 150000$ $\qquad$ Grade 8 fastener tensile strength

$\tau_{bolt} := \dfrac{W_w}{N_{bolts} \cdot A_{bolt}}$ $\qquad \tau_{bolt} = 1.543 \cdot 10^3$

In Side View of the assembly, the support angle bracket has 5 3/4" bolts that will resist the moment caused by the load being applied by an offset distance from the centroid of the bolt pattern. The centroid of the bolt pattern is located at the middle bolt. The bolts furthest from the centroid will take a greater share of the indirect shear. The direction of the shear stress acts perpendicular to the line drawn from the centroid of the bolt pattern to the bolt center. In this case the shear will act in a horizontal direction.

The applied load offset length is: $x_{offset} := 2.75$

The resultant moment is: $M_{offset} := W_w \cdot x_{offset}$ $\quad M_{offset} = 9.918 \cdot 10^3$

The offset moment is resisted by the bolts. The distances between the centroid of the bolt pattern and the furthest bolts and the second furthest bolts are:

$d_{bolt1} := 6$ $\quad d_{bolt2} := 3$

$d_{bolt3} := 0$

The middle bolt in the pattern will contribute nothing to resisting the offset moment because it is at the centroid of the bolt pattern.

The reaction force for the furthest bolts is:

$$F_r := \frac{M_{offset} \cdot d_{bolt1}}{2 \cdot d_{bolt1}^2 + 2 \cdot d_{bolt2}^2 + d_{bolt3}^2} \qquad F_r = 661.192$$

The secondary shear is then: $\tau_2 := \frac{F_r}{A_{bolt}}$ $\quad \tau_2 = 1.98 \cdot 10^3$

The secondary shear does not act in the same direction as the primary shear so it must be resolved in to x and y components and then recombined.

$$\tau_{sum} := \sqrt{\tau_2^2 + \tau_{bolt}^2} \qquad \tau_{sum} = 2.51 \cdot 10^3$$

Allowable shear is roughly 1/2 tensile

$$SF := \frac{\sigma_{ult} \cdot .5}{\tau_{sum}} \qquad SF = 29.885$$

Installation Beam Bolts. The end of the installation beam will try to rotate more than the slop in the holes for the installation beam-to-support column joint will allow. This means that the joint will act as a fixed support and a beam bending moment will be realized at this joint. The beam bending moment, M, was calculated in section 2, THE LOADS. Since the support angle bracket acts as a pivot, it will not support any of the beam bending moment. (See Appendix C)

The following matrix represents the x and y coordinates for the four bolts:

$$d := \begin{bmatrix} 8.809 & 10 \\ 4.309 & 10 \\ 8.809 & 5.5 \\ 4.309 & 5.5 \end{bmatrix}$$

The distance from the each bolt hole to the pivot point is:

$i := 0..3$

$$D_i := \sqrt{\left(d_{i,0}\right)^2 + \left(d_{i,1}\right)^2}$$

$$Dsqr := \sum \overrightarrow{D^2}$$

D_i
13.327
10.889
10.385
6.987

The resultant force at each bolt is:

$M = 1.403 \cdot 10^5$

$$F_i := \frac{M \cdot D_i}{Dsqr}$$

F_i
$4.129 \cdot 10^3$
$3.374 \cdot 10^3$
$3.218 \cdot 10^3$
$2.165 \cdot 10^3$

The shear stress in bolt #1, which has the highest resultant force is:

$$\tau_{bending} := \frac{F_0}{A_{bolt}} \qquad \tau_{bending} = 1.236 \cdot 10^4$$

Allowable shear is roughly 1/2 tensile

$$SF := \frac{\sigma_{ult} \cdot .5}{\tau_{bending}} \qquad SF = 6.067$$

4.0 Support Column to Magnet Screws:

Eight 3/4" hex head cap screws, are used to join the Support Column to the magnet. The screws will be under shear stress due to direct loading and due to the moment caused by the load being eccentrically applied. Load Case 2e has the greatest vertical load and will be used in the following analysis.

$N_{bolts} := 8 \qquad A_{bolt} := .334$

The primary shear load is:

$$\tau_1 := \frac{W_e}{A_{bolt} \cdot N_{bolts}} \qquad \tau_1 = 3.05 \cdot 10^3$$

To determine the moment arm of the eccentric loading, it is first necessary to find the CG of the bolt group. The horizontal component of the CG can be found by symmetry and is the horizontal mid plane of the magnet. The vertical component will be calculated relative to hole 1. (See Appendix D)

x is the horizontal distance the hole is from the temporary origin located at hole 1

$$xnot := \begin{bmatrix} 0 \\ 4 \\ 9 \\ 15 \\ 0 \\ 4 \\ 9 \\ 15 \end{bmatrix} \cdot in \qquad xbar := \frac{\sum xnot}{N_{bolts}} \qquad xbar = 7 \cdot in$$

The following matrix, L, is the x and y positions of the bolts with respect to the centroid of the bolt pattern.

$$L := \begin{bmatrix} -7 & 32.5 \\ -3 & 42.8 \\ 6 & 52.6 \\ 21 & 61.9 \\ -7 & 32.5 \\ -3 & -42.8 \\ 6 & -52.6 \\ 21 & -61.9 \end{bmatrix}$$

$i := 0..7$

$x_i := L_{(i,0)}$ $\quad y_i := L_{(i,1)}$ $\quad D_i := \sqrt{(x_i)^2 + (y_i)^2}$ $\quad \alpha_i := \mathrm{atan}\left[\frac{x_i}{|y_i|}\right]$

i	x_i	y_i	D_i	α_i
0	-7	32.5	33.245	-0.212
1	-3	42.8	42.905	-0.07
2	6	52.6	52.941	0.114
3	21	61.9	65.365	0.327
4	-7	-32.5	33.245	-0.212
5	-3	-42.8	42.905	-0.07
6	6	-52.6	52.941	0.114
7	21	-61.9	65.365	0.327

$Dsqr := \sum \overrightarrow{D^2}$

The horizontal distance from the applied force to the CG of the bolt pattern is Df

$Df := 8.72$

The horizontal offset between the applied force and the bolt pattern centroid results in a moment, M

$W_e = 8.15 \cdot 10^3$ $\qquad M := W_e \cdot Df$

The applied torque can be related to the force on each screw by the following relation:

$Ft_i := \frac{M \cdot D_i}{Dsqr}$

Ft_i
117.874
152.123
187.707
231.757
117.874
152.123
187.707
231.757

The secondary shear stress will be highest for bolts number 4 and 8

$\tau_2 := \frac{Ft_3}{A_{bolt}}$ $\qquad \tau_2 = 693.884$

The shears can be resolved into x and y components and algebraically summed to give a total shear.

$$\tau := \sqrt{\left(\tau_2 \cdot \cos(\alpha_3)\right)^2 + \left(\tau_2 \cdot \sin(\alpha_3) + \tau_1\right)^2}$$

$$\tau = 3.338 \cdot 10^3$$

For a grade 2 fastener (common low strength) $\sigma_{ult2} := 36000$

$$SF := \frac{\sigma_{ult2}}{\tau} \qquad SF = 10.784$$

5.0 Installation Column Welds

The load path from column to magnet is through the support plate boss and column mounting flanges. These two items are welded to the 8" x 1/2 wall square mechanical tubing. (See Appendix E)

5.1 Mounting Flange Welds

Load Case 2e has the highest vertical load:

$$W_e = 8.15 \cdot 10^3$$

Using the the American Institute of Steel Construction (AISC), guidelines on eccentric loads on weld groups, Table XVIII, where:

l is the height of the rectangular weld group $l := 20$

kl is the width of the rectangular weld group $kl := 8$

al is the distance from the centroid of the weld group to the vertical load P $al := 6$

D is the the width of the weld in 16ths $D := .4 \cdot 16$ $D = 6.4$

C1 is the coefficient for electrode used, for E70XX electrodes: $C1 := 1$

a, and k can be solved for:

$$a := \frac{al}{l} \qquad a = 0.3$$

$$k := \frac{kl}{l} \qquad k = 0.4$$

From Table XVIII

$$C := 1.58$$

The permissible eccentric load, P, in kips is:

$$P := C \cdot C1 \cdot D \cdot l$$

$$P = 202.24$$

Since the vertical welds are skip welds instead of continuous, the permissible eccentric load must be derated.

The theoretical length of weld used is:

$$L_{theory} := 2 \cdot l + 2 \cdot kl$$

The actual length of weld used is:

$$L_{actual} := 2 \cdot (.6 \cdot l) + 2 \cdot kl$$

The percentage of weld area available to resist the shear loading is:

$$\%A := \frac{L_{actual}}{L_{theory}} \qquad \%A = 0.714$$

The permissible eccentric load, P is now;

$$P := P \cdot \%A \qquad P = 144.457$$

There are two identical weld groups, one on each end plate, that resist the shearing load, so the permissible eccentric load for each weld group is twice P.

$$P := 2 \cdot P \qquad P = 288.914 \text{ kips} \qquad W_e = 8.15 \cdot 10^3 \text{ pounds}$$

The actual eccentric load, We, is well below the Permissible eccentric load as allowed by AISC.

5.2 Support Angle Bracket Boss Welds:

The support angle bracket forms a shelf for the west ends of the installation columns to rest on. The support bracket is made of a 4"x7" angle that is bolted to two support bosses on the installation column. One boss is a 6" x 8" boss, and the other is 5" x 14" boss. Both are welded on all sides with a 1/2 fillet, The 5" x 14" boss overhangs the support column so the actual weld pattern is 4" x 14". Load Case one will be used since the vertical load is greater for the West side.

For the 6" x 8" Boss:

Using the American Institute of Steel Construction (AISC), guidelines on eccentric loads on weld groups, Table XVIII, where:

l is the height of the rectangular weld group — $l := 8$

kl is the width of the rectangular weld group — $kl := 6$

al is the distance from the centroid of the weld group to the vertical load P — $al := 6$

D is the the width of the weld in 16ths — $D := .5 \cdot 16$ $\quad D = 8$

C1 is the coefficient for electrode used, for E70XX electrodes: — $C1 := 1$

a, and k can be solved for:

$$a := \frac{al}{l} \qquad a = 0.75$$

$$k := \frac{kl}{l} \qquad k = 0.75$$

From Table XVIII

$$C := 1.27$$

The permissible eccentric load, P, in kips is:

$$P := C \cdot C1 \cdot D \cdot l \qquad P = 81.28$$

For the 5" x 14" Boss:

Using the American Institute of Steel Construction (AISC), guidelines on eccentric loads on weld groups, Table XVIII, where:

l is the height of the rectangular weld group — $l := 14$

kl is the width of the rectangular weld group — $kl := 4$

al is the distance from the centroid of the weld group to the vertical load P — $al := 3.75$

D is the the width of the weld in 16ths — $D := .5 \cdot 16$ $\quad D = 8$

C1 is the coefficient for electrode used for E70XX electrodes: $C1 := 1$

a, and k can be solved for:

$a := \frac{al}{l}$ $a = 0.268$

$k := \frac{kl}{l}$ $k = 0.286$

From Table XVIII

$C := 1.50$

The permissible eccentric load, P, in kips is:

$P := C \cdot C1 \cdot D \cdot l$ $P = 168$

The actual eccentric load applied to the two weld groups is :

$W_1 = 5.878 \cdot 10^3$ pounds

Each weld group by itself can support the applied eccentric load of 6 kips, since the AISC maximum eccentric load for the above bosses was 81 and 168 kips.

5.3 Support Plate Boss Welds:

The support plates form a shelf for the east ends of the installation columns to rest on. Each support plate is bolted to a 6" x 8" boss on the support columns. Load case 2e will be used for the following analysis since it has the greater vertical load for the east side.

Using the American Institute of Steel Construction (AISC), guidelines on eccentric loads on weld groups, Table XVIII, where:

l is the height of the rectangular weld group $l := 8$

kl is the width of the rectangular weld group $kl := 6$

al is the distance from the centroid of the weld group to the vertical load P $al := 6$

D is the the width of the weld in 16ths $D := .5 \cdot 16$ $D = 8$

C1 is the coefficient for electrode used, for E70XX electrodes: $C1 := 1$

a, and k can be solved for:

$a := \frac{al}{l}$ $a = 0.75$

$k := \frac{kl}{l}$ $k = 0.75$

From Table XVIII

$C := 1.27$

The permissible eccentric load, P, in kips is: The Applied load, We, is:

$P := C \cdot C1 \cdot D \cdot l$ $\qquad P = 81.28$ kips $\qquad W_e = 8.15 \cdot 10^3$ pounds

According to AISC, the weld is more than sufficient for this loading condition.

Summary:

3.0 Support Column to Installation Beam, Screws

Load Case 1e	SF = 6.5
Load Case 1w	SF = 18.3
Load Case 2e	SF = 7.8
Load Case 2w	SF = 29.9

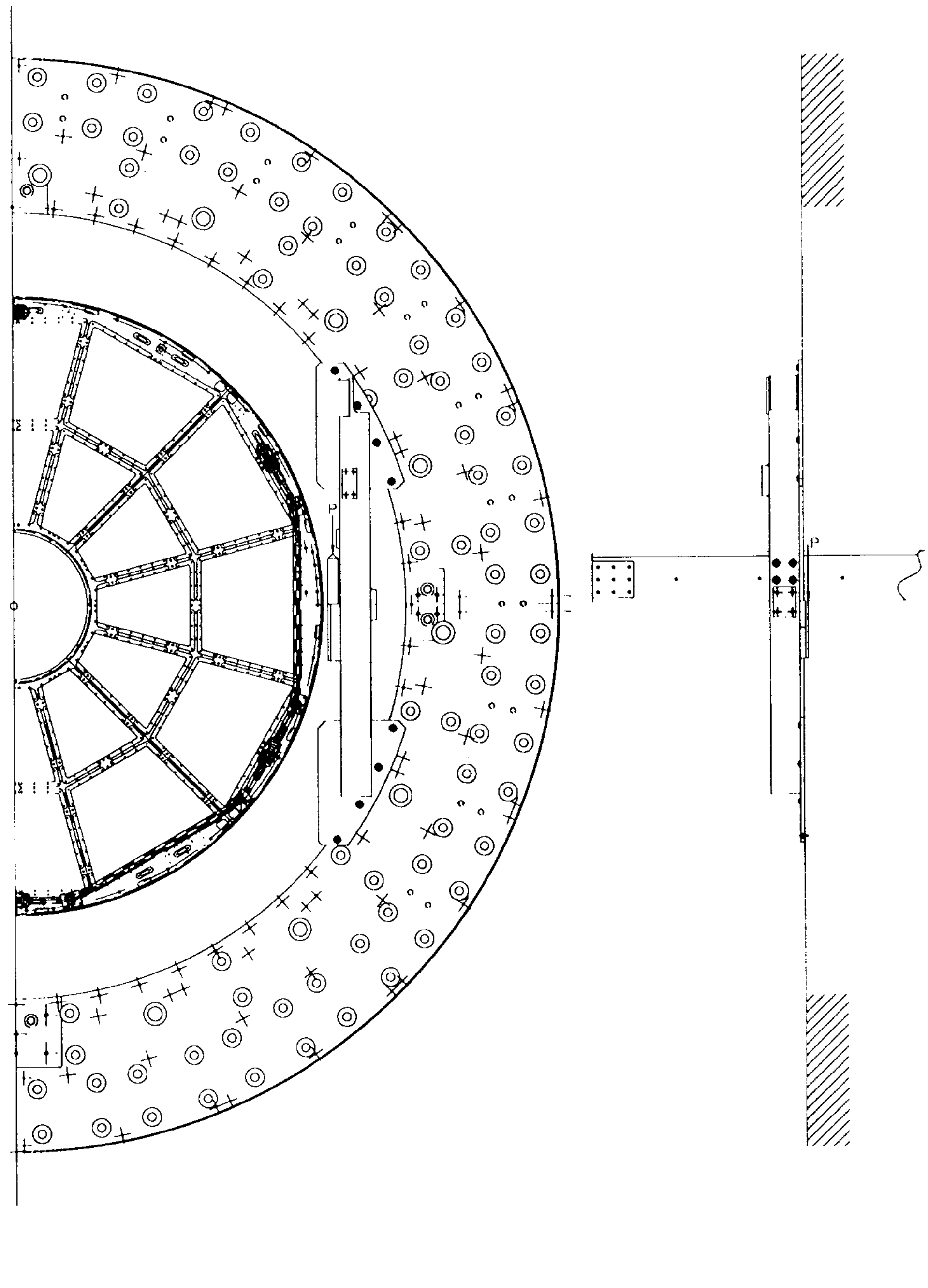
4.0 Support Column to Magnet, Screws

Load Case 2e	SF = 10.8

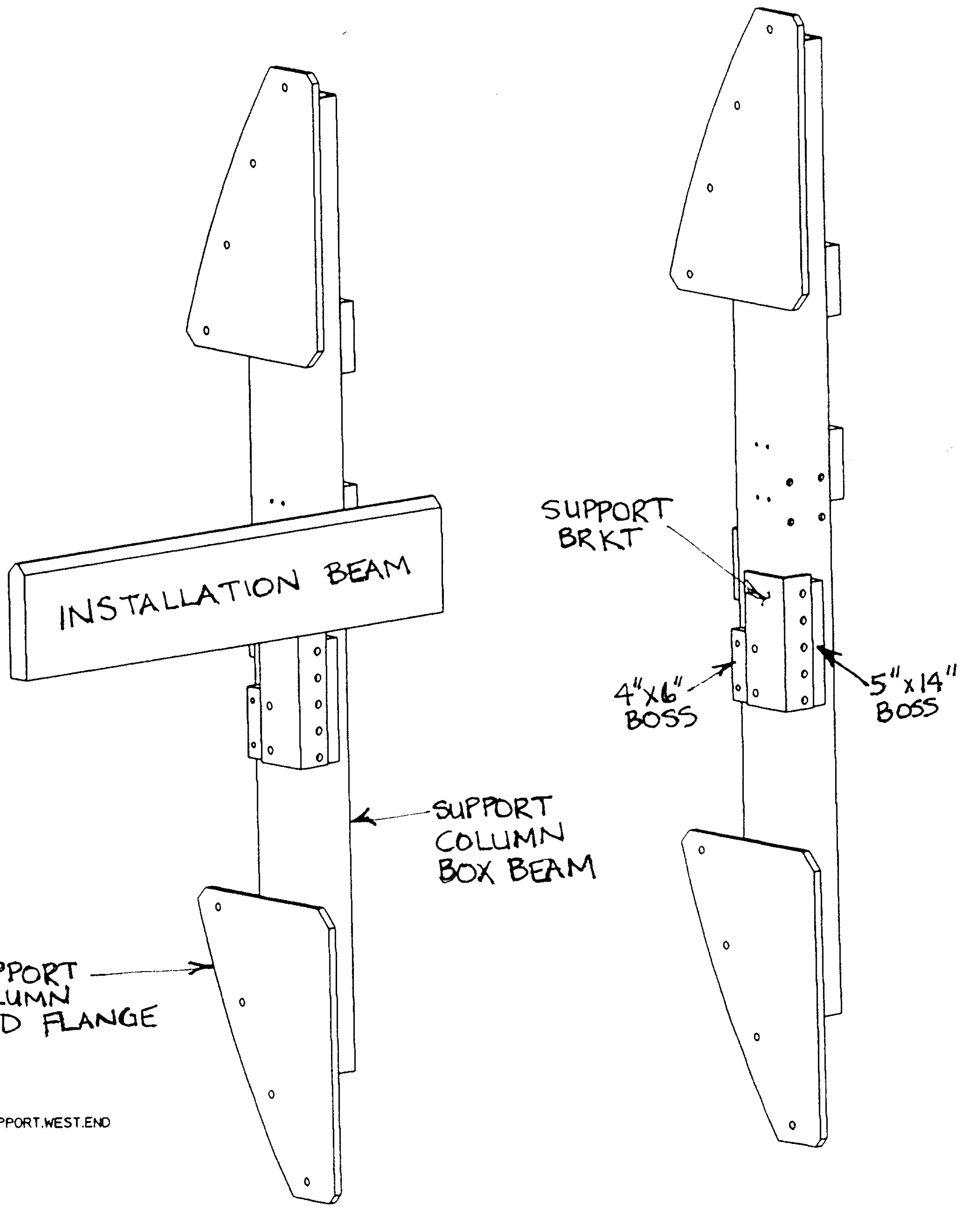
5.0 Installation Column Welds	Allowable	Actual
5.1 Mounting Flange Welds	289 kips	8 kips
5.2 Support Angle Bracket, Boss	81, 168 kips	6 kips
5.3 Support Plate, Boss	81 kips	8 kips

A-1

TPC INSTALLATION – SUPPORT COLUMN



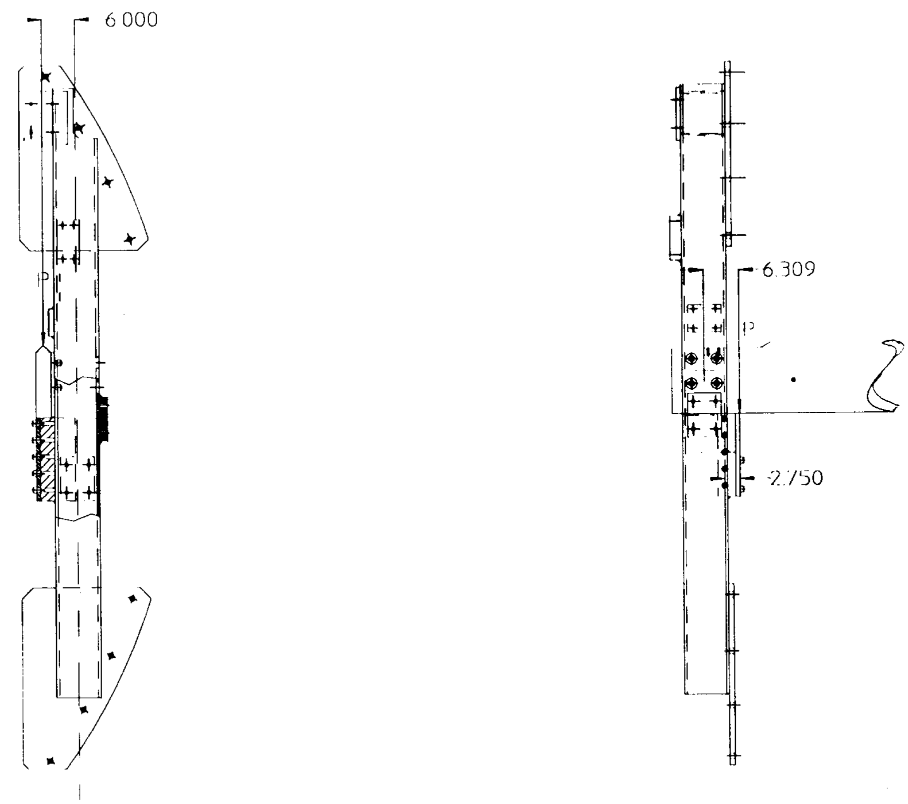
A - 2



FILENAME BEAM.SUPPORT.WEST.END

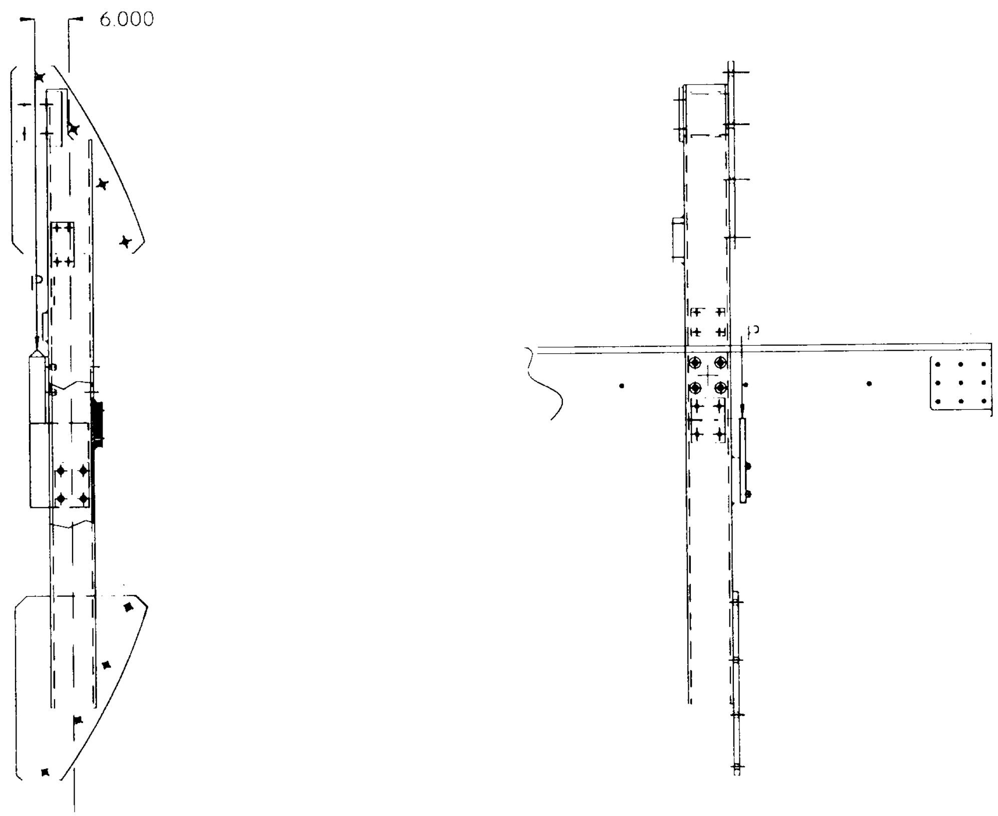
A-3

WEST END ASSEMBLY



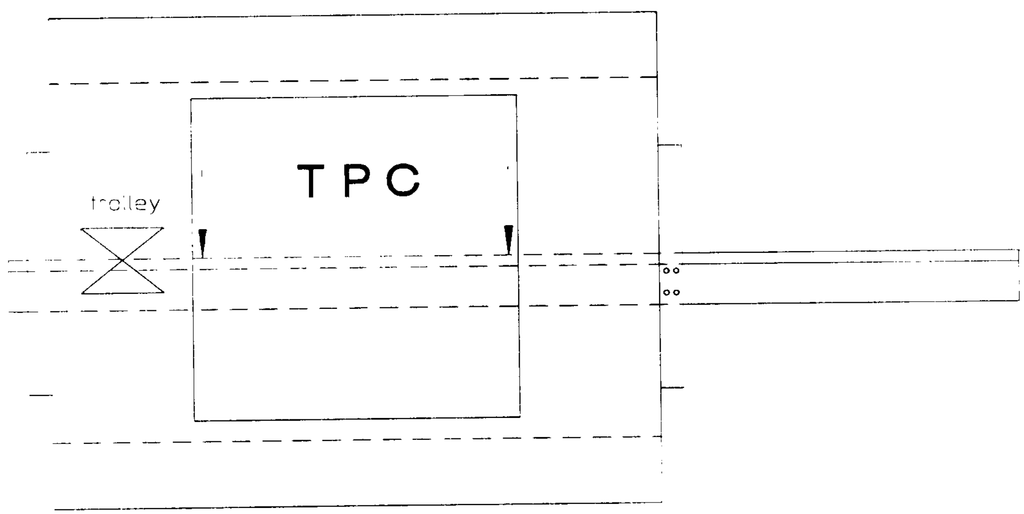
A - 4

EAST END ASSEMBLY

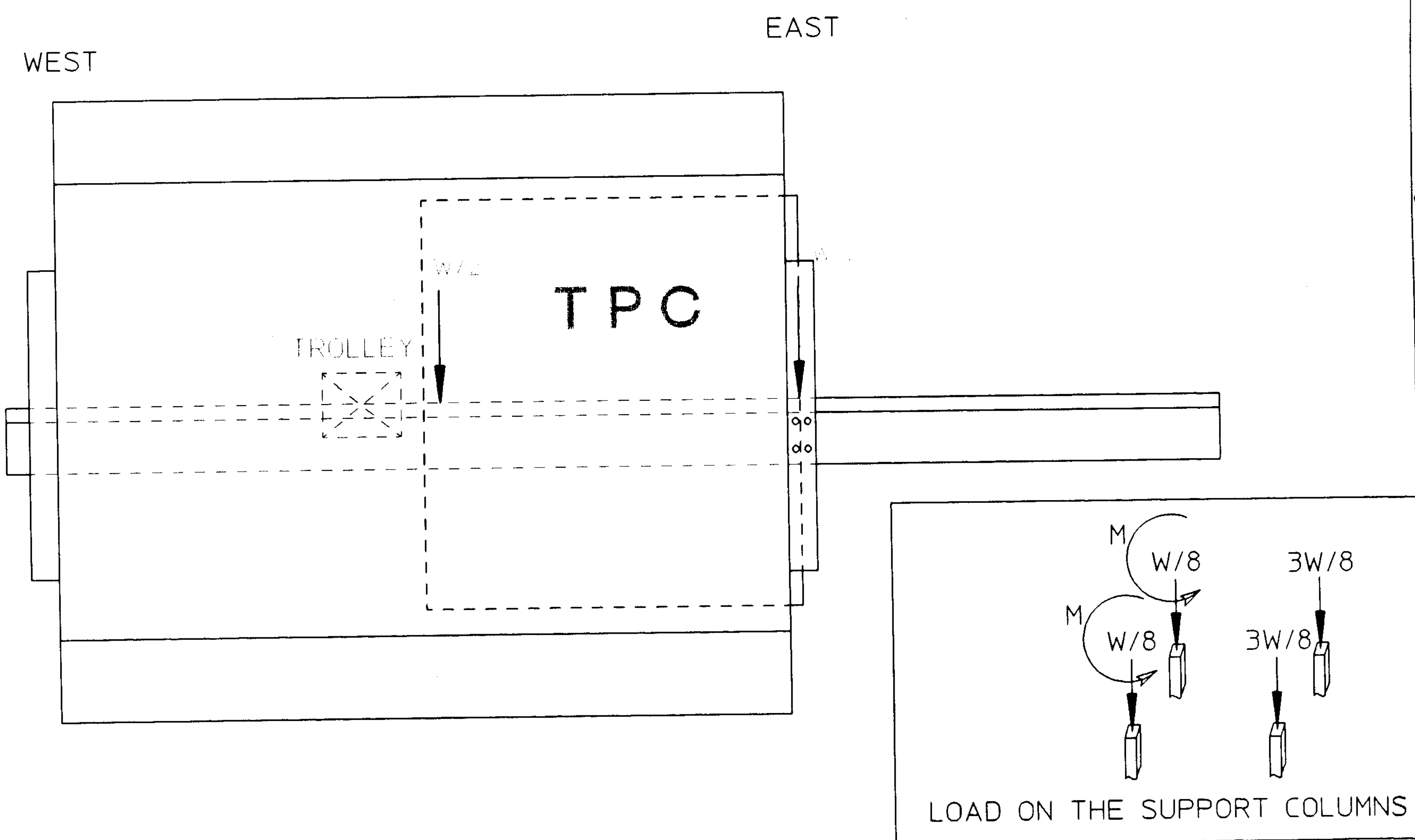


B-1

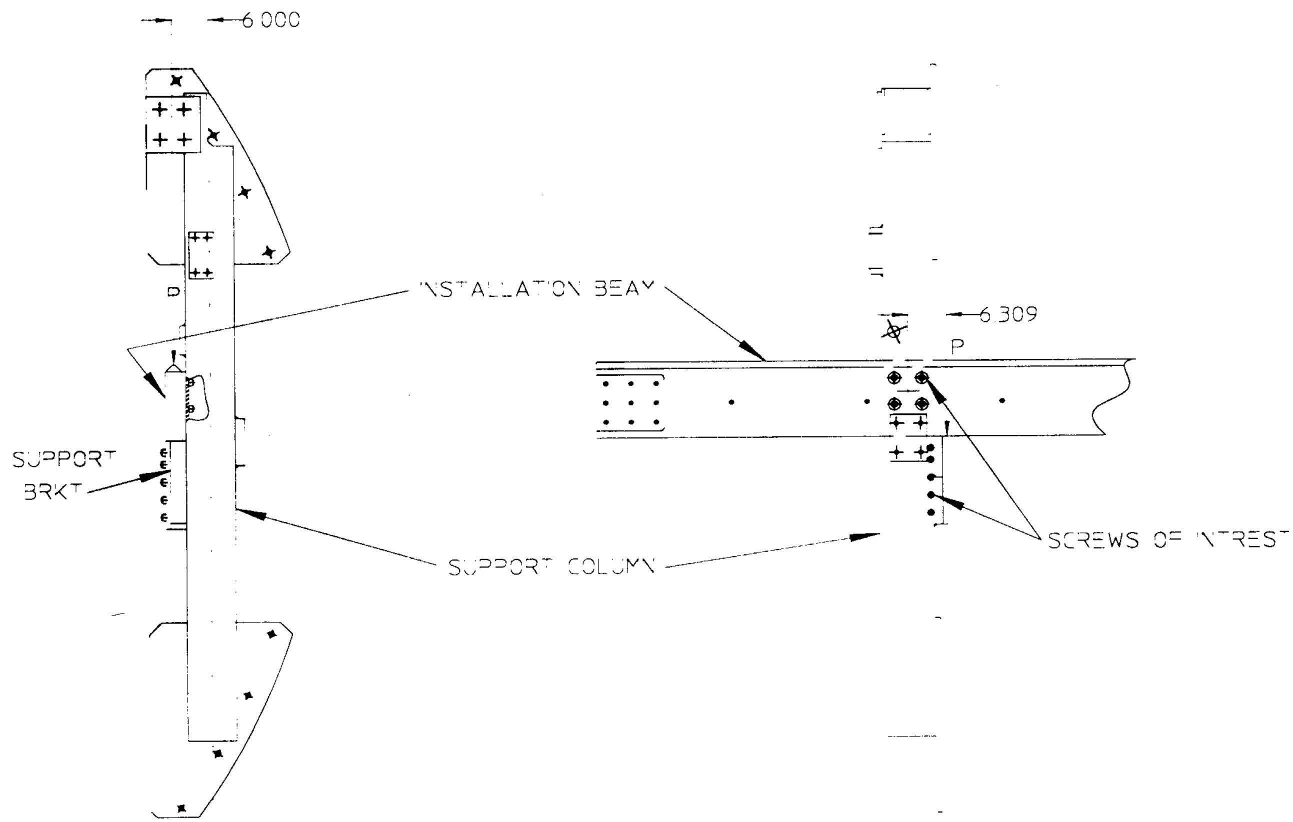
LOAD CASE 1



LOAD CASE 2e-w



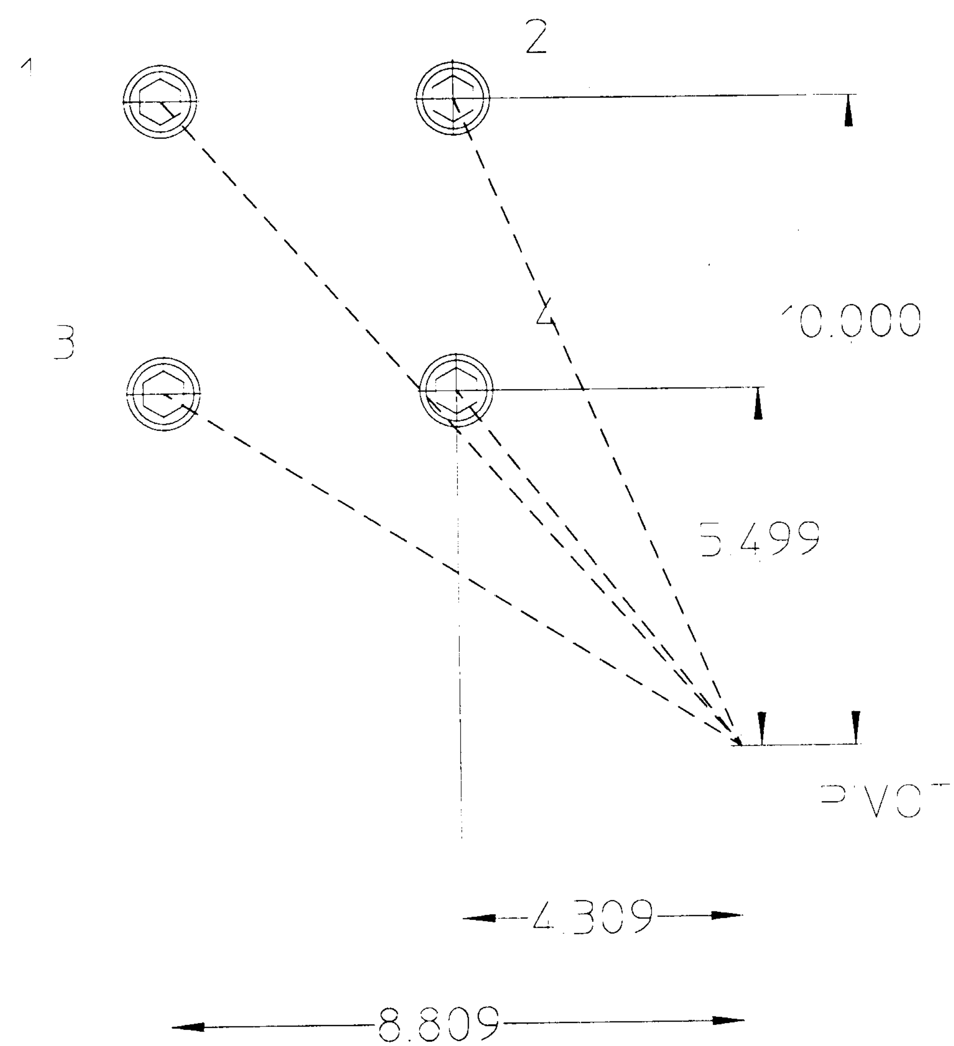
3.0 SUPPORT COLUMN TO BEAM INSTALLATION SCREWS



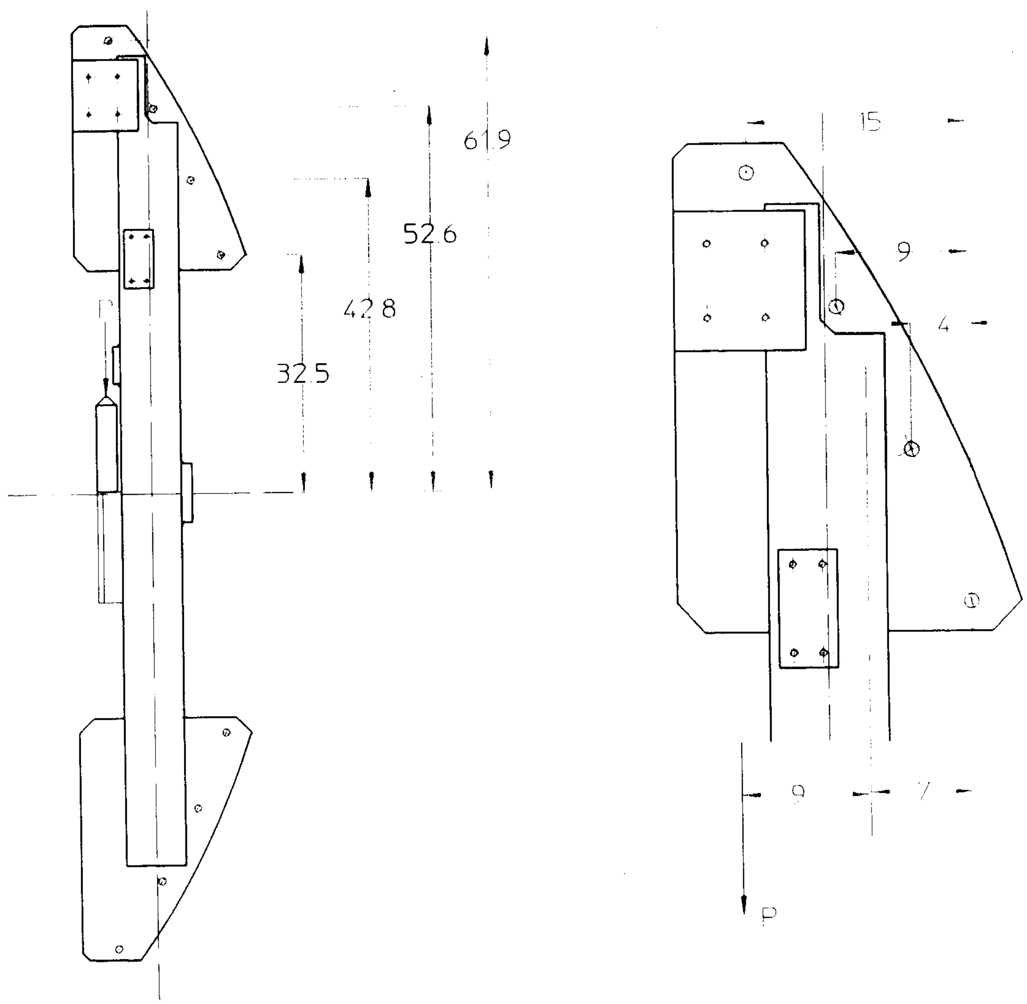
C-1

C-2

INSTALLATION BEAM BOLTS

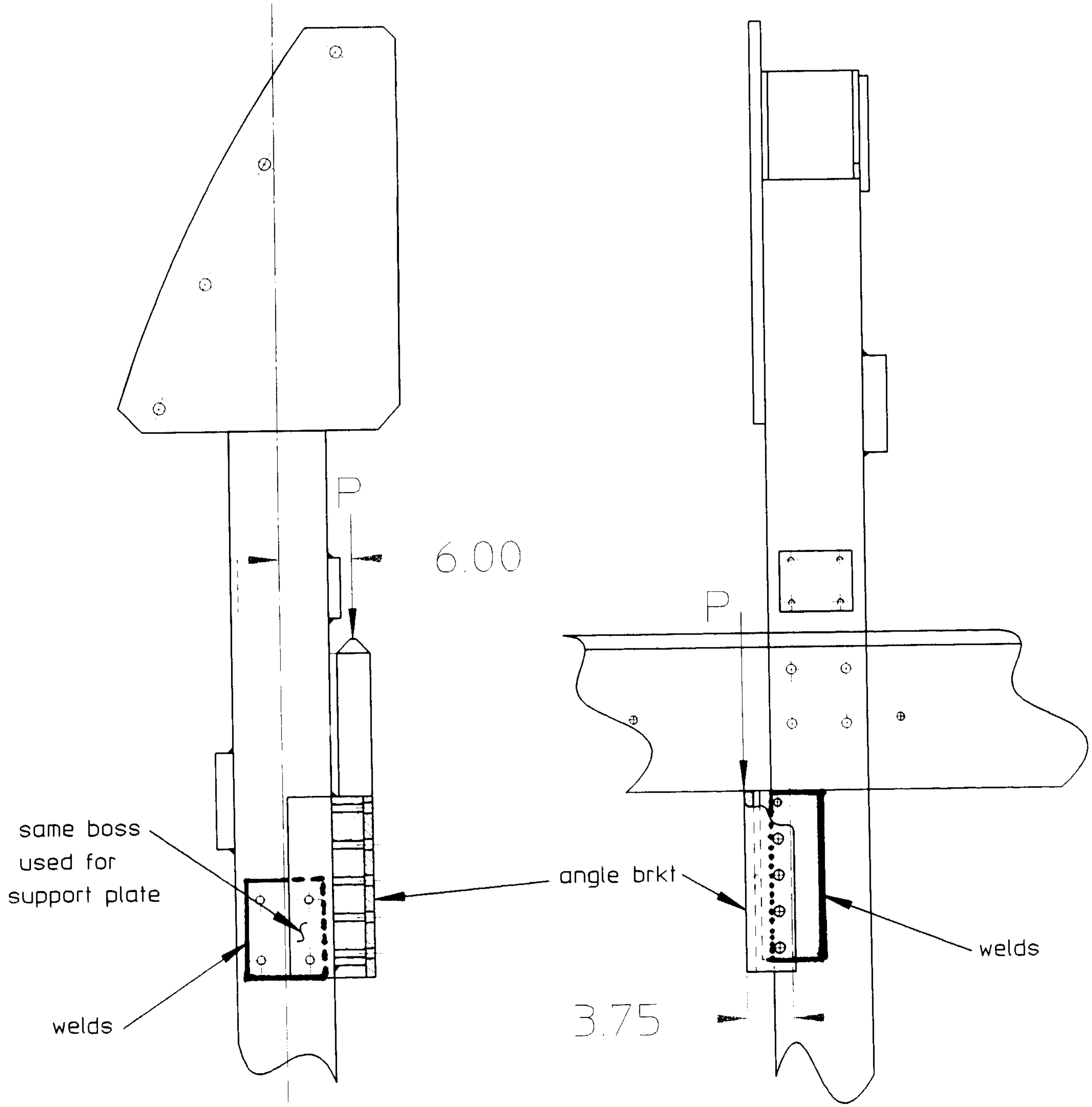


5.0 SUPPORT COLUMN TO MAGNET BOLTS



D

5.0 INSTALLATION COLUMN WELDS



5.0 INSTALLATION COLUMN WELDS MOUNTING FLANGE