Fermilab

Particle Physics Division

Mechanical Department Engineering Note

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Title: Lifting Fixture for COUPP's Veto Top Panel

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Key Words: COUPP, lifting fixture, veto

Abstract/Summary: A double wall scintillator enclosure is needed for the COUPP bubble chamber experiment. In addition to putting four walls of counters surrounding the bubble chamber apparatus, a top panel of counter is needed to complete the coverage. A lifting fixture is thus needed to lift the fully loaded top panel with the aid of a mobile gantry for its final placement.

Applicable Codes: AISC Manual of Steel Construction Allowable Stress Design – & ASME B30.20, BELOW THE HOOK LIFTING DEVICES

Lifting Fixture for COUPP's Veto Top Panel Analysis

PPD/MD Eng Note 107

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May 29, 2006

(I) Introduction:

A double wall scintillator enclosure is needed for the COUPP bubble chamber experiment. In addition to putting four walls of counters surrounding the bubble chamber apparatus, a top panel of counter is needed to complete the coverage. The layout of the fully loaded top panel is shown in Figure 1, it has two layers of counters with 2-inch thick high density polyethylene (HDPE) in between. The first layer of counters will be of KTeV counters and is under the dead-weight loads from the HDPE panels in addition to the second layer of counters recycled from D0.. This fully loaded top panel is then lift up with the aid of a mobile gantry for its final position. Since there is a weak glued join within the KTeV counter, a stiff panel was designed and three or four lifting spots along each outer frame were provided so that excessive deflection is avoided. Taking advantage that one side of the top panel is not covered by the PhotoMultiplier Tube (PMT), the lifting spots at this side can be moved inwards and four lifting spots are provided from the existing spoke members arrangement for further minimizing deflection.

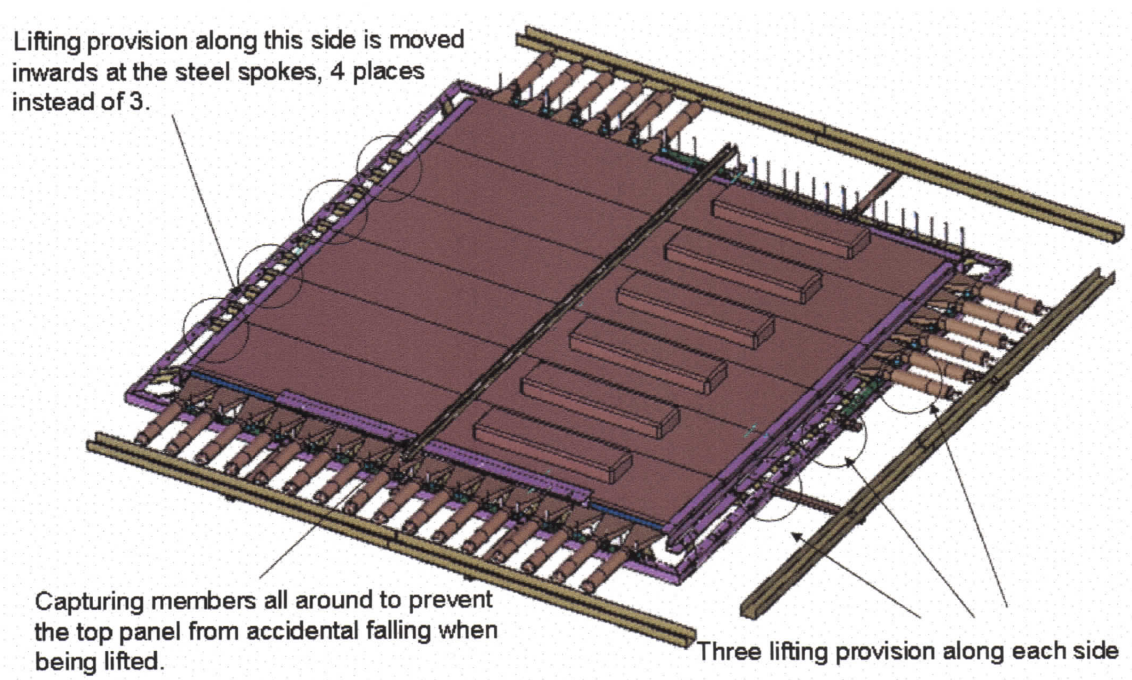

Figure 1. The Fully-Loaded Top Panel

The weight of the fully loaded top panel was originally estimated to be about 1,920 lbs. Due to the clearance limitation under the gantry, two hoists and a lifting fixture were planned to use for lifting up the fully-loaded top panel vertically as shown in Figure 2.

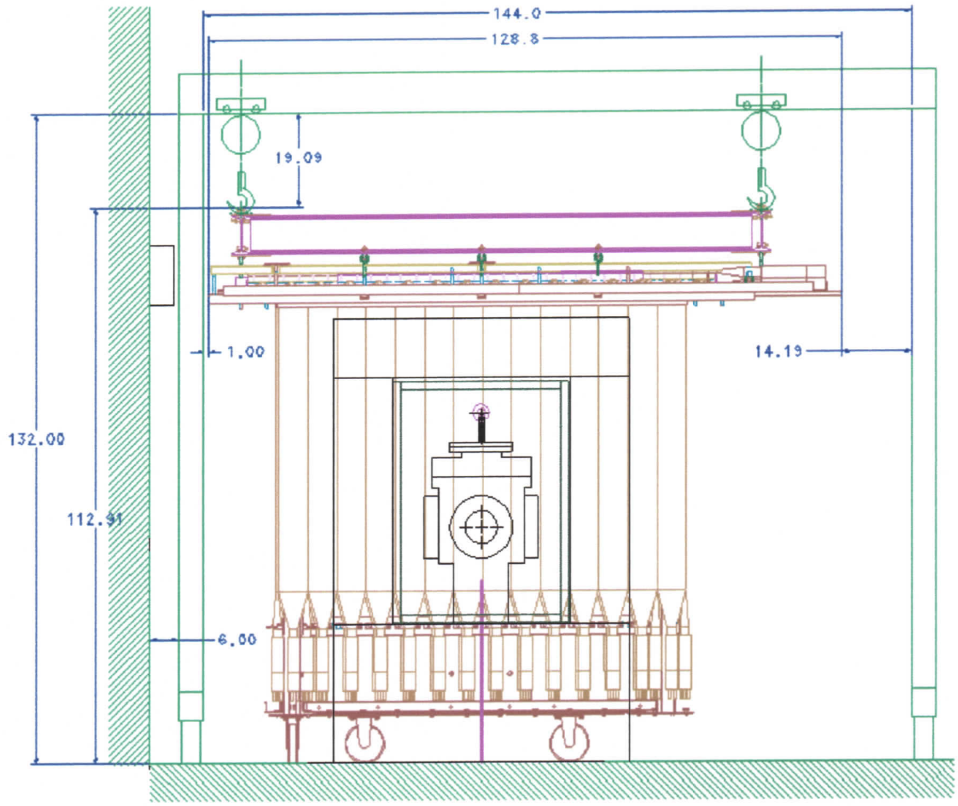

Figure 2. The Lifting Set Up with the Mobile Gantry

The lifting fixture was made of steel I-beam S8X18.4 which was salvaged from the Rail Head. This set of stiff beams would help the deflection issue and hence minimal adjustments during the lifting process might be needed. The design of this lifting fixture was shown in Figure 3. As verified by the FEA in which it showed the bending moments at the corners were small, 3/8"-inch thick aluminum plates were simply used to connect those I-beams together at both sides (bolts and nuts are not shown in this figure). For lifting arrangement, on top, two ½-anchor shackles are provided for hooking up with the hoists, while underneath, a series of shackles and a swivel socket would be used. With this arrangement, all loading would be vertical and all tensions between the lifting fixture and the fully-loaded top panel could be individually adjusted by means of the swivel socket.

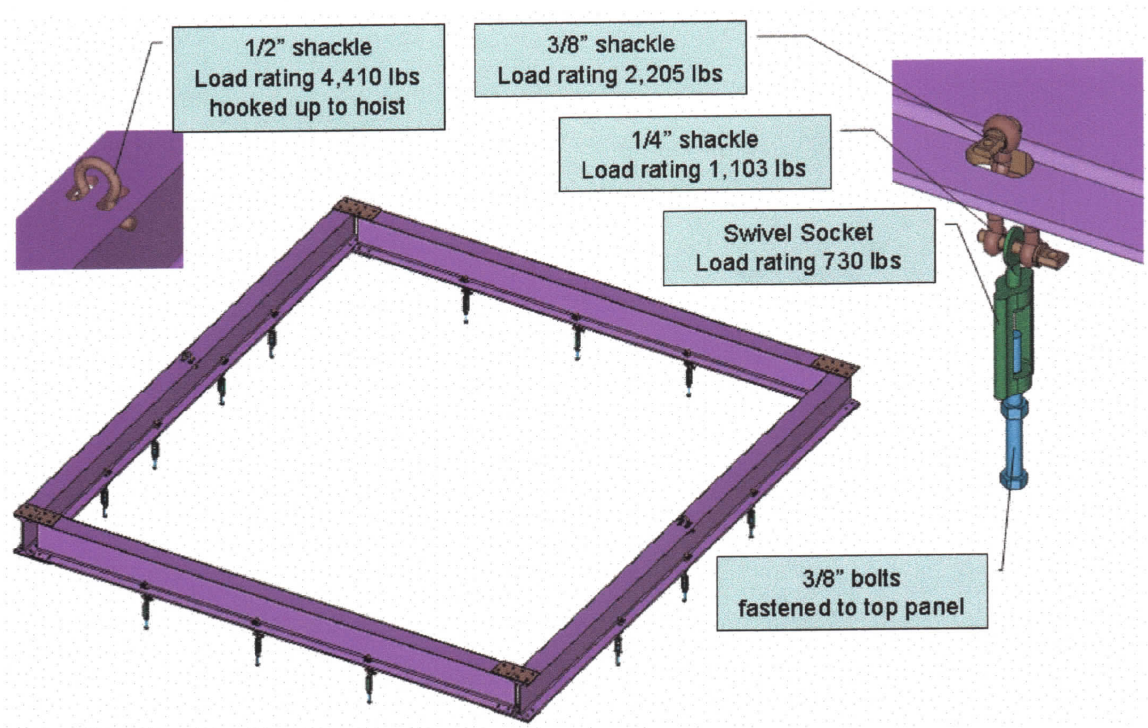

Figure 3. The Lifting Fixture Design

(II) The Engineering Analysis

Some bending moments would exist at corners and it was needed to be understood so that a proper design for the connecting plates could be made. The conventional hand calculation on beam by beam with loads being transferred one by one was first attempted. As a worst case scenario, the end connections of the beam were assumed to be fixed and it would yield a very huge bending moment at the corners. Consequently, this would result a set of very heavy plates for the connections and was impractical. This approach was then abandoned and the Finite Element Analysis (FEA) was used to find out all the loadings at corners.

(III) The Finite Element Analysis (FEA)

Beam elements were used for modeling the lifting fixture. Taking advantage of symmetry, half model was made and the lifting spots for hoists were constrained fixed. Continuous one-piece I-beam was assumed in this model even at the corners . This assumption was valid as long as a rigid connection was successfully warranted. To simplify the analysis, all beams were identical with equal length 112 inches long and each beam equally took 3 loads with each one at 192 lbs. The gravity effect was included and hence the weight of the I-beam was added in this analysis. The boundary conditions of this analysis were as shown in Figure 4.

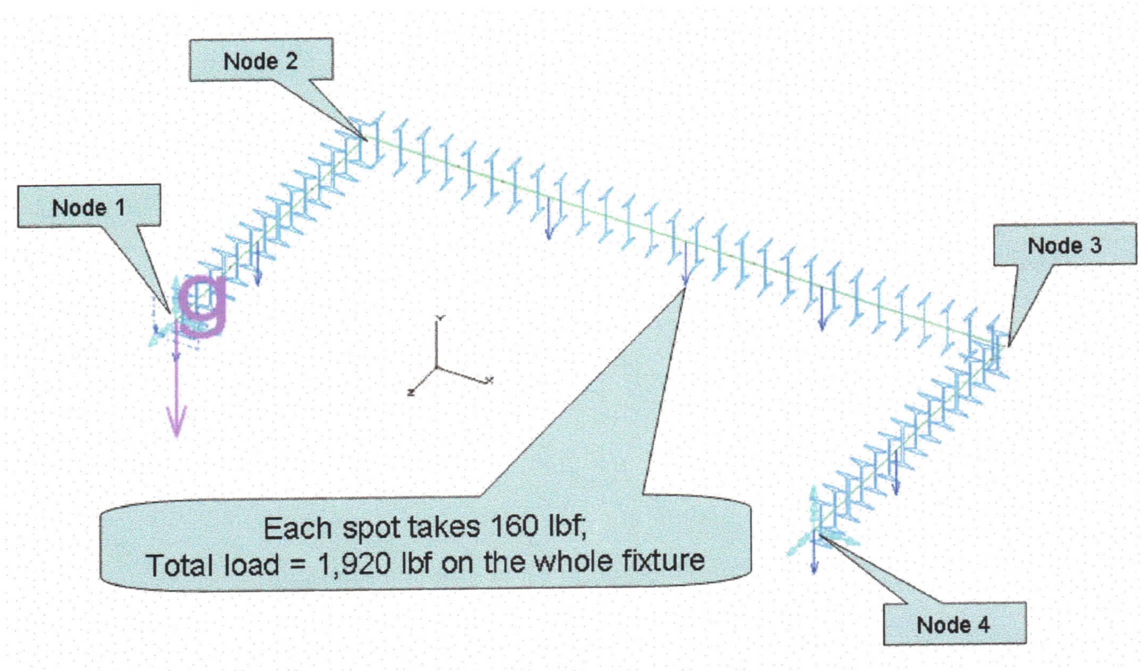

Figure 4. The FEA Model

The displacement result was shown in Figure 5 in which it showed the maximum displacement was only 0.025". Also, the resultant stress was shown in Figure 6. The maximum stress on the beam occurs at the lifting spot and was found to be about 1,730 psi which was far below the allowable stress of A36 steel 12,000 psi.

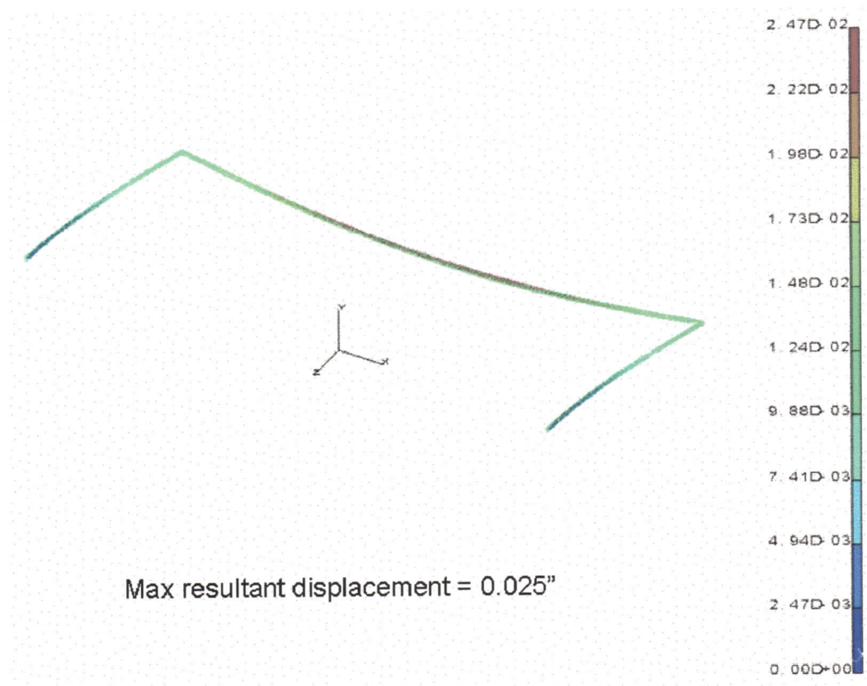

Figure 5. The FEA Model

Figure 6. The Resultant Stress Result

(IV) The Connection Analysis

The resultant element force and the rotational moment results were shown in Figures 7 and 8. At corner, it was found that the element force was about 325 lbf and the moment was about 18.5 lbf-in which was very small.

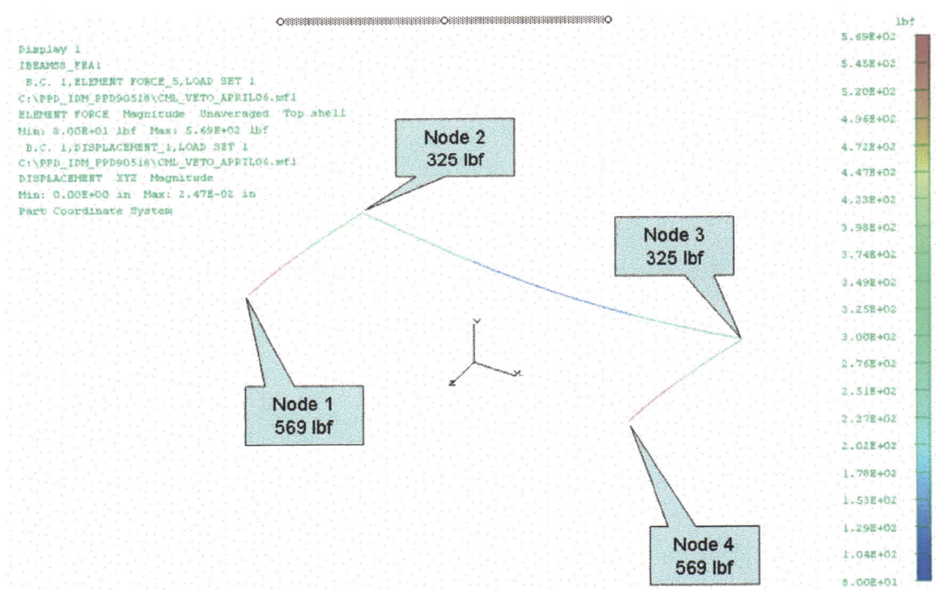

Figure 7. The Resultant Element Force Result within the I-Beam

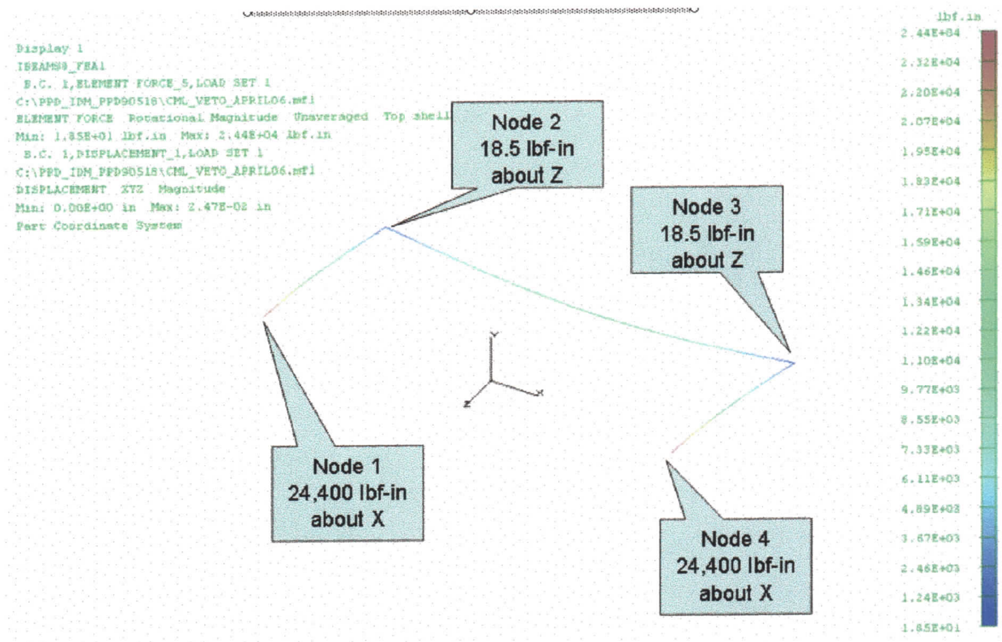

Figure 8. The Rotational Moment Result within the I-Beam

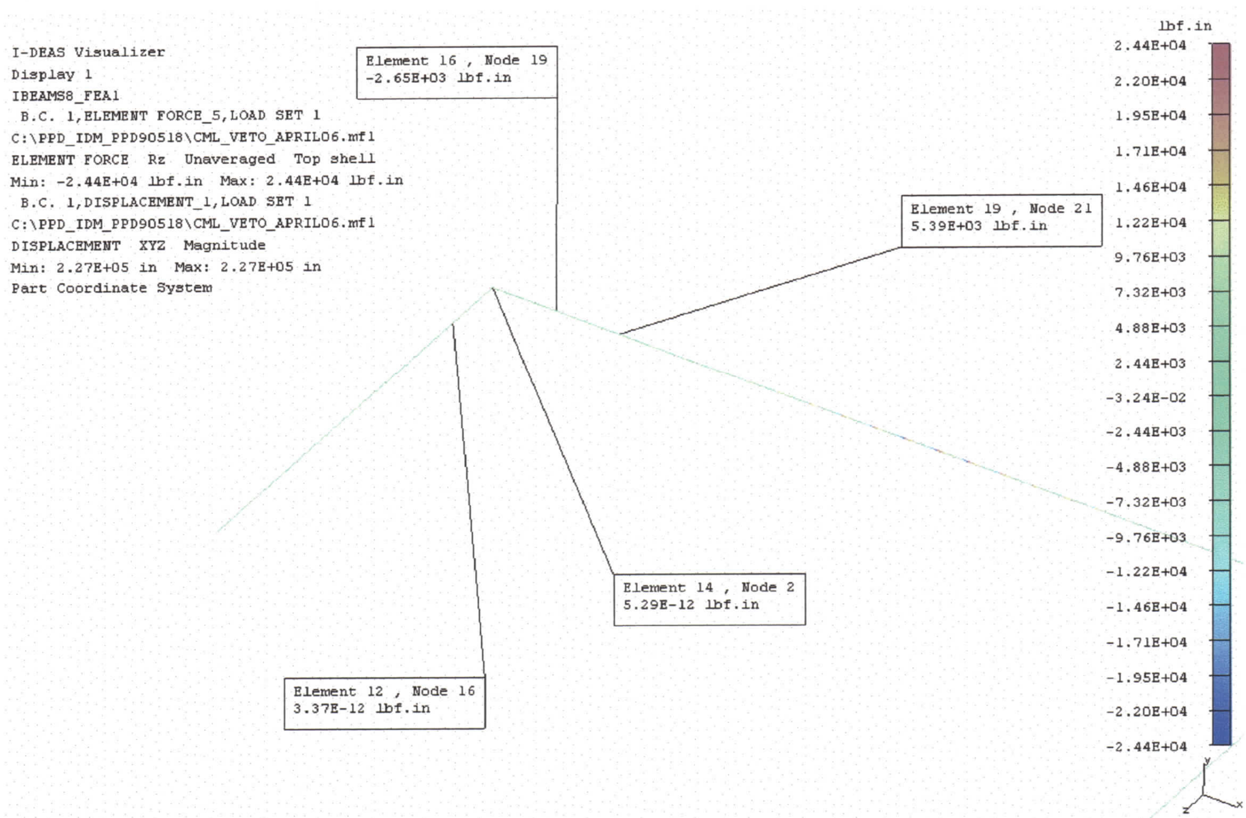

Figure 9. A Typical Detailed Results near Corner

(A) Connecting Plate Analysis

These plates were made of aluminum T6 and were designed with dimensions 8"x4"x3/8" thick. To be conservative, the higher loads within this 8"x 4" region at the corner were used for calculations. A typical detailed result plot showing more nodes near the corner was shown in Figure 9, and all the corresponding loads at those nodes were listed in Table 1. On the other hand, the reaction loading at the lifting spots, nodes 1 and 4, were found to be 569 lbf and 24,400 lbf-in. The total loads taken by the two hoists would then be 2,276 lbs, and the weight of each I-beam was found to be 170 lbs.

Node#	Resultant Stress psi	Force, X lbf	Force, Y lbf	Force, Z lbf	Moment X lbf-in	Moment Y lbf-in	Moment Z lbf-in
17	1310	0	331	0	1310	0	18.5
2	18.5	0	325	0	0	0	18.5
18	1270	0	319	0	0	0	1270
19	2530	0	313	0	0	0	2530

Node 17 is at 4" from corner in Z direction
Node 2 is right at corner
Node 18 is at 8" from corner in X direction
Node 19 is at 4" from corner in X direction

Table 1. The Component Loadings at Nodes near Corner

The connecting plates were then subject to the following loads:

Vertical load, F_Y = 325 lbf
Rotational moment about X axis, M_X = 1,310 lbf-in
Rotational moment about Z axis, M_Z = 2,530 lbf-in

Shear stress, Ss_Y $= (F_Y)/A_P$
$= 325/(4*0.375)$
$= 217$ psi

Referring to Figure 10, the turning moment about Z would generate a couple of forces, F_{PX}, acting on these connecting plates.

Force on plate, F_{PX} = (M_Y at node 19)/(couple length)
= (2,530/8)
= 316 lbf

Normal stress, S_{NX} $= (F_{PX})/A_P$
$= 316/(4*0.375)$
$= 211$ psi

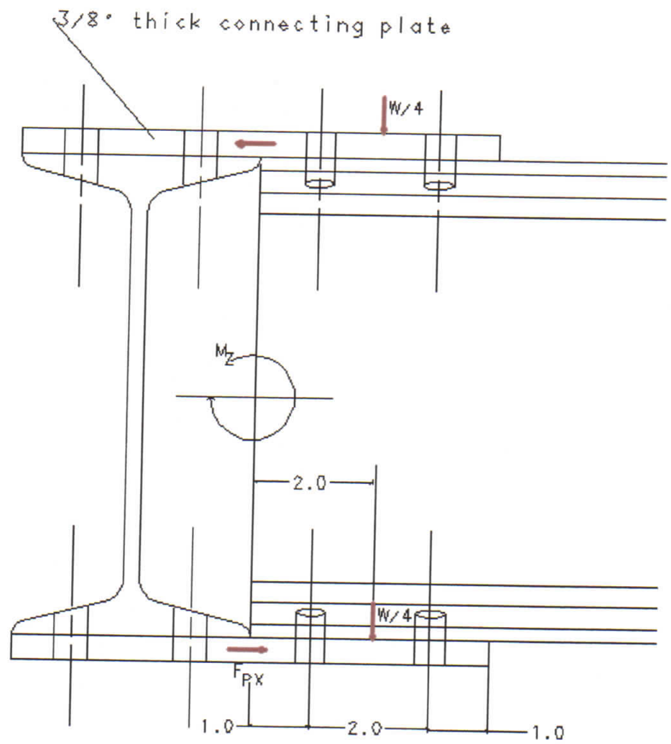

Figure 10. The Loading M_Z on the Connecting Plates

Similarly, the turning moment about X would generate another coupling force on the plates as shown in Figure 11:

Force on plate, F_{PZ} = (M_X at node 17)/(couple length)
= (1,310/8)
= 164 lbf

Normal stress, S_{NZ} = $(F_{PZ})/A_P$
= 164/(4*0.375)
= 109 psi

Total resultant stress, $S_{SR,}$ at plate center = $SQRT(S_{NX}{}^2 + S_{SY}{}^2 + S_{NZ}{}^2)$
= $SQRT(211^2 + 217^2 + 109^2)$
= 322 psi

which was very small. ....OK

In addition, each connecting plate will support one quarter of the beam load, W/4, as shown in Figure 10. A bending moment, $M_{B,}$ was then acting on this plate, and was equal to the product of W/4 and distance L_B where

W = summation of all 3 loadings on beam plus the beam weight
= 3*160 + 170
= 650 lbs

L_B = the distance of the load to the edge of another I-beam
= 2"

Thus, M_B = (650/4)*2
= 325 lbf-in

The corresponding bending stress on plate, S_{BX} = $6* M_B/(b*t^2)$
= $6*325/(4*.375^2)$
= 3,467 psi

Total resultant stress, S_{SR}, on plate surface = $SQRT((S_{BX} + S_{NX})^2 + S_{NZ}^2)$
= $SQRT((3,467+ 211)^2 + 109^2)$
= 3,680 psi

Referring to Aluminum manual, allowable stress T6 for tension in beam, with shapes bent about weak axis is equal to:

$1.3* F_{ty}/n_y = 1.3*35/1.65 = 27.6$ ksi or
$1.42* F_{tu}/(k_t * n_u) = 1.42*42/(1.0*1.95) = 30.6$ ksi

Also, with reference to ASME B30.20, Below the Hook Lifting Devices, the bending stress should be less than 1/3 of F_{ty}, or, 11.7 ksi. Thus, 11.7 ksi is the allowable stress, and which is much larger than the resultant stress....................OK

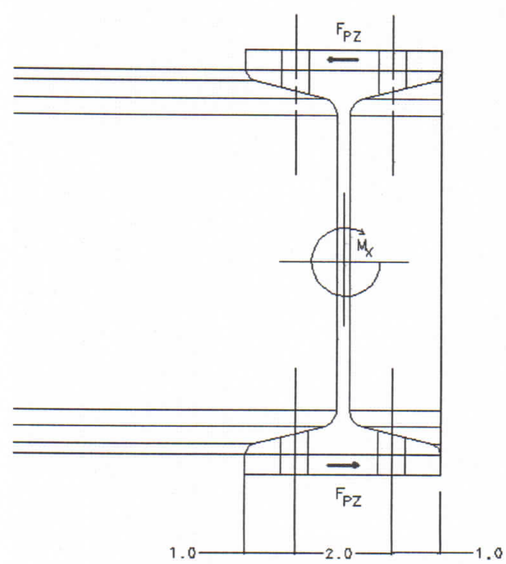

Figure 11. The Moment Loading M_X on the Connecting Plates

(B) The Bolt Group Analysis

Each connecting plate with be fasten by two groups of ½"-13 bolts with one group to one I-beam and another group to another. The element loading F_Y 325 lbf at the connection would then be shared by two groups of such bolts, with 4 units on top and 4 units on bottom side. This load would create a tensile stress on the bolts.

Stress area of bolt, A_s = 0.1419 inches^2

Normal stress, S_{n0} = (F_Y at B)/8 bolts/A_s
= 325/8/0.1419
= 287 psi

The 4-bolt group was also subject to a shear force due to the effect of moments about Z and X.

Cross-sectional area of bolt, A_{bolt} = 0.1963 inches^2

Shear stress, S_{SX} = $(F_{PX})/A_{bolt}$
= 325/4/0.1963
= 414 psi

Shear stress, S_{SZ} = $(F_{PZ})/A_{bolt}$
= 164/4/0.1963
= 209 psi

Total shear stress, S_{Stotal} = $SQRT(S_{SX}{}^2 + S_{SZ}{}^2)$
= $SQRT(414^2 + 209^2)$
= 464 psi

all stresses are smaller than the steel A490 bolt allowable stress 17,800 psi .OK

(V) Further Check

The FEA model was changed slightly at corners. As a conservative analysis, the I-beam was made discontinuous and was ½" apart at corner. This gap was replaced by a couple of connecting plates on top and bottom sides as shown in Figure 12. It was found that all the loading results were basically the same except the resultant displacement was 0.001" more.

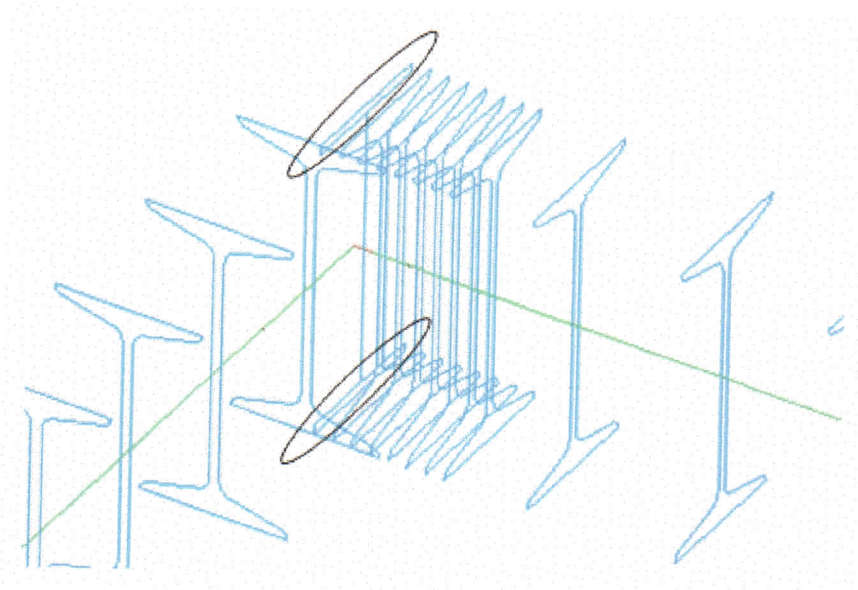

Figure 12. The Modified FEA Model at Corners

(VI) Simulated Loading Test

This following parts and assemblies were weighed:

KTeV counters	= 10.4 lbs x 22 pieces = 230 lbs
D0 counters	= 30 lbs x 6 pieces = 180 lbs
Empty top panel	= 550 lbs.

The weight of the HPDE panels and capturing bars on top of the panel were estimated to be about 640 and 200 lbs correspondingly. Hence the total weight to be lift by the lifting fixture would be the summation of above, and it was 1,800 lbs.

As shown in Figure 13, 90-lbs concrete block were used for the simulated loading test. 24 blocks were used and the total weight was 2,160 lbs which was about 20 % more than it would take. This test was done more than 15 minutes and the lifting frame did not show any sign of yielding.

Figure 13, Simulated Loading Test

(VII) Conclusion

This lifting fixture is safe to lift 1,800 lbs of load.