5022TA

BELOW-THE-HOOK LIFTING DEVICE
Engineering Note Cover Page

Lifting Device Numbers:

FNAL Site No/ ______________ Div. Specific No. 171 ______________ Asset No. ______________

If applicable   If applicable   If applicable

ASME B30.20 Group: (check one)

X] Group I Structural and Mechanical Lifting Devices
[ ] Group II Vacuum Lifting Devices
[ ] Group III Magnets, Close Proximity Operated
[ ] Group IV Magnets, Remote Operated

Device Name or Description 31 PLANE LIFTING FIXTURE

Device was (check all applicable)

[ ] Purchased from a Commercial Lifting Device Manufacturer. Mfg Name ______________
[ ] Designed and Built at Fermilab
[X] Designed by Fermilab & Argonne and Built by a Vendor. Assy drawing number 444201
[ ] Provided by a User or other Laboratory ______________
[ ] Other: Describe ______________

Engineering Note Prepared by GUARINO / VILLEGAS Date 04-05-2010 /04-15-2010

Engineering Note Reviewed by WOODS / FANG Date 10-27-2009/04-28-2010

Lifting Device Data:

Capacity 14,200 LB

Fixture Weight 4,350 LB

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

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

Inspections Frequency VISUAL INSPECTION BEFORE EACH USE

Rated Load Test by FNAL (if applicable) Date 04-26-2010 Load 18,250 LB

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

Satisfactory Load Test Witnessed by: VILLEGAS,2444/GRIFFIN,5108/SHOUN,4959AND OLSZANOWSKI,5036

Signature (of Load Test Witness) E. Villegas

Notes or Special Information:

NOTE : ASME B30.20 GR 1 WAS NOT ADHERE TO FOR BASE WELDMENT (PALLET) WHEN WET.

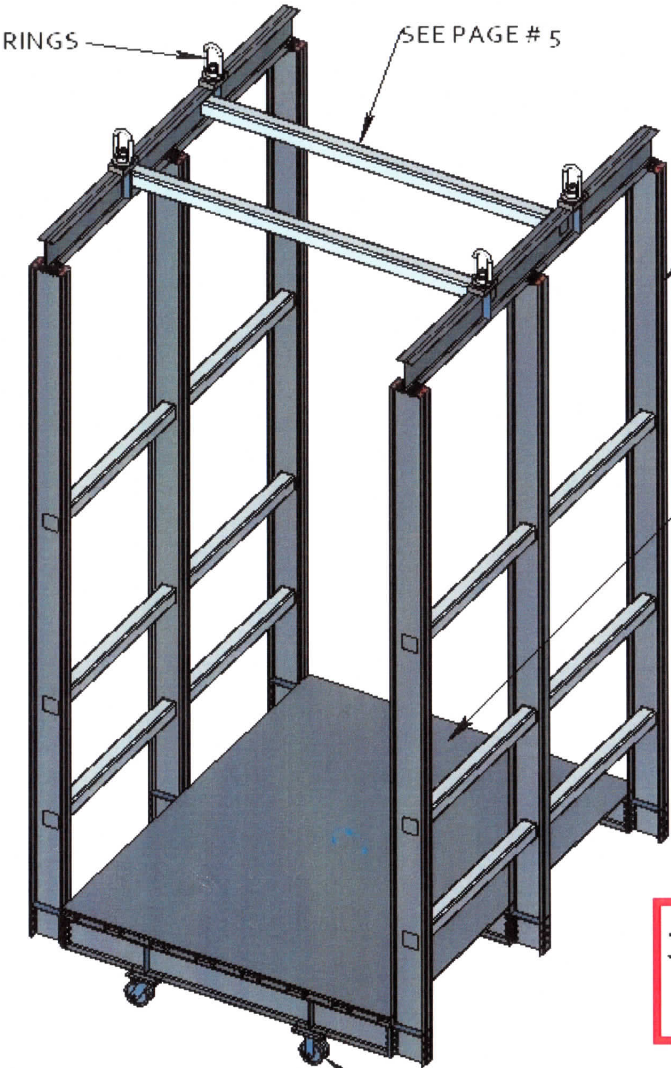
MATERIAL ALL SHAPES : ASTM A992
YIELD STRESS = 50 x 10^3 PSI

MATERIAL TUBING : ASTM A500 GRADE B
YIELD STRESS = 46 x 10^3 PSI

MATERIAL PLATE STEEL = ASTM A36
YIELD STRESS = 36 x 10^3 PSI

ALL BOLTS = GRADE 5
YIELD STRESS = 92 x 10^3 PSI

ALL WELDS USE ELECTRODE E70XX
YIELD STRESS = 60 x 10^3 PSI



DISTRIBUTED FILLED LOAD ON
PALLET = 48,000 LB

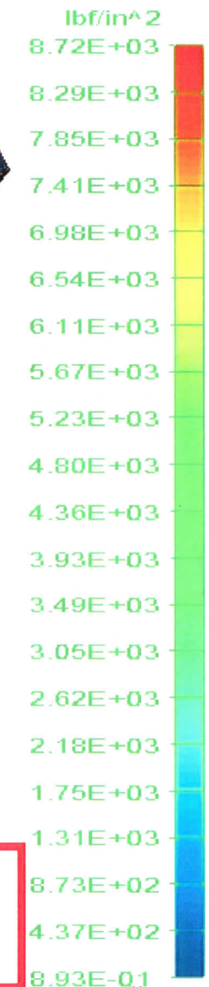
DISTRIBUTED EMPLY LOAD ON
PALLET = 14200 LB

SEE PAGES #2 & 3 FOR FEA OF PALLET

31 PLANE LIFTING FIXTURE
 # 444201

I-DEAS 12 NX Series m2: I-DEAS Visualizer

SUPPORTED FROM (6) BOLTED
CONECTIONS AT ENDS



IPND_LFTG_FIXT_BASE_WLDNT(PALLET)FEM
14200LB_LOAD_EMPTY
STRESS Von Mises Averaged Top shell
Min: 8.93E-01 lbf/in^2 Max: 8.72E+03 lbf/in^2
B.C. 1,DISPLACEMENT_1,LOAD SET 1
DISPLACEMENT XYZ Magnitude
Min: 0.00E+00 in Max: 1.32E-02 in

MATERIAL ALL SHAPES: ASTM A992
ALLOWABLE STRESS = $50 \times 10^3 / 3 = 16{,}667$ PSI

MATERIAL TUBING : ASTM A500 GR B
ALLOWABLE STRESS = $46 \times 10^3 / 3 = 15{,}333$ PSI

MATERIAL PLATE : ASTM A36
ALLOWABLE STRESS : $36 \times 10^3 / 3 = 12{,}000$ PSI

(48) 3/4-10 GRADE 5 BOLTS IN SHEAR SUPPORT
PALLET WEIGHT OF 2539# PLUS 14,000# WEIGHT OF BLOCK.

ALLOWABLE STRESS ON EACH BOLT = $92 \times 10^3 \times .334 / 3 = 10{,}243$ PSI

DESIGN STRESS ON EACH BOLT = $(2539 + 14{,}200) / 48 \times .334 = 116$ PSI

IPND LFTG FIXT BASE WLDNT FEM
444200

PAGE #2

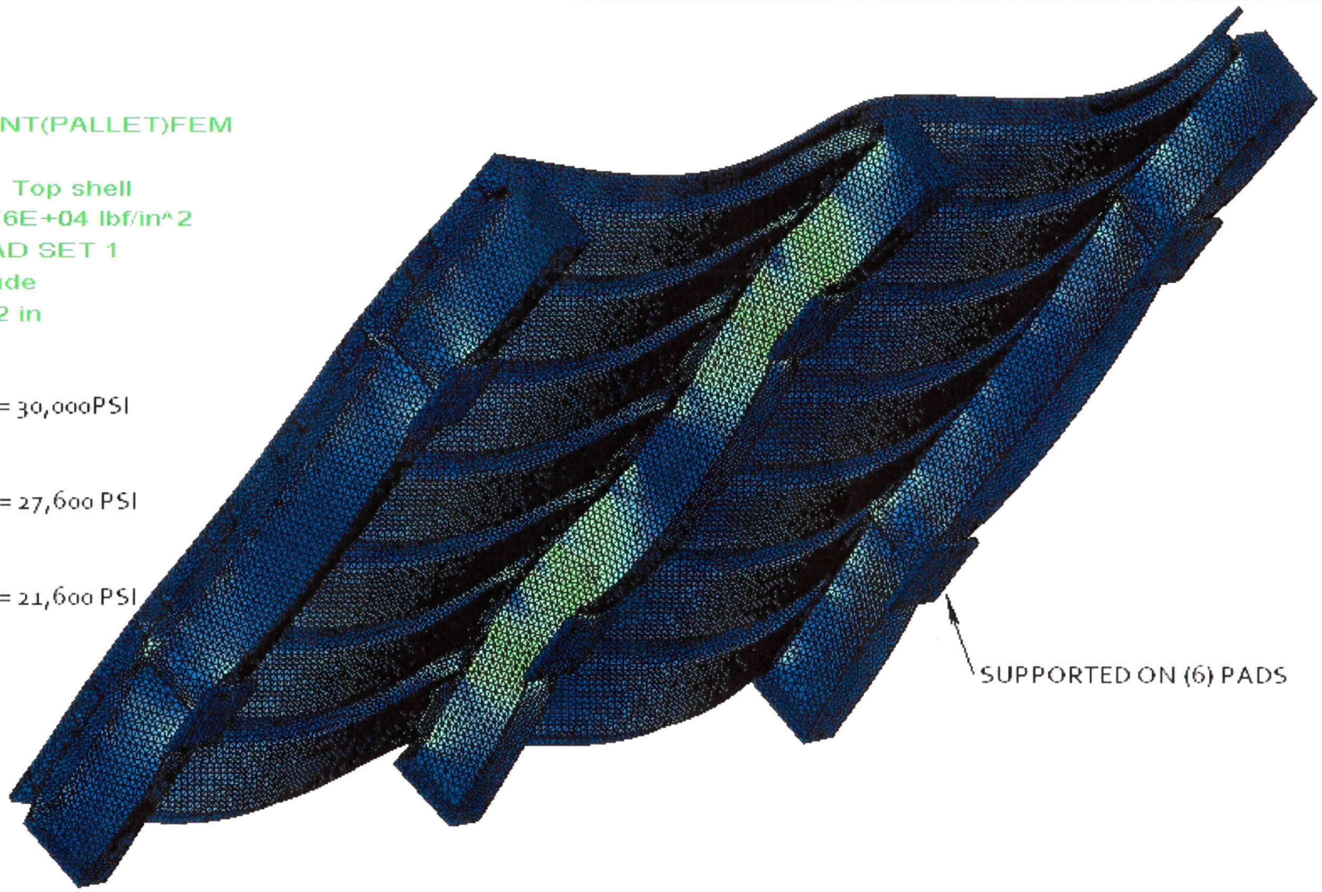
I-DEAS 12 NX Series m2: I-DEAS Visualizer

IPND_LFTG_FIXT_BASE_WLDNT(PALLET)FEM
48000_LB_LOAD_FILLED
STRESS Von Mises Averaged Top shell
Min: 3.96E+00 lbf/in^2 Max: 1.76E+04 lbf/in^2
B.C. 1,DISPLACEMENT_1,LOAD SET 1
DISPLACEMENT XYZ Magnitude
Min: 0.00E+00 in Max: 1.77E-02 in

MATERIAL ALL SHAPES: ASTM A992
ALLOWABLE STRESS = 50 x 10^3 x .6 = 30,000PSI

MATERIAL TUBING: ASTM A500 GR B
ALLOWABLE STRESS = 46 x 10^3 x .6 = 27,600 PSI

MATERIAL PLATE: ASTM A36
ALLOWABLE STRESS = 36 x 10^3 x .6 = 21,600 PSI



lbf/in^2
1.76E+04
1.67E+04
1.59E+04
1.50E+04
1.41E+04
1.32E+04
1.23E+04
1.15E+04
1.06E+04
9.70E+03
8.81E+03
7.93E+03
7.05E+03
6.17E+03
5.29E+03
4.41E+03
3.53E+03
2.65E+03
1.77E+03
8.85E+02
3.96E+00

IPND LFTG FIXT BASE WLDNT (PALLET) FEM
#444200

NOTE: ALLOWABLE STRESSES ABOVE ARE ONLY RELEVENT WHEN PALLET IS IN OPERATON MODE AND FILLED WITH SCINTILATOR OIL

PAGE # 3

I-DEAS 12 NX Series m2: I-DEAS Visualizer

IPND_VERT_SUPT_WLDNT_FEM
8370_LB_LOAD(HALF LOAD)
STRESS Von Mises Averaged Top shell
Min: 4.32E-02 lbf/in^2 Max: 5.17E+03 lbf/in^2
B.C. 1,DISPLACEMENT_1,LOAD SET 1
DISPLACEMENT XYZ Magnitude
Min: 0.00E+00 in Max: 2.53E-02 in

MATERIAL : ALL SHAPES, ASTM A992
ALLOWABLE STRESS = 50 x 10^3 /3 = 16,667 PSI

MATERIAL : TUBING ASTM A500 GR B
ALLOWABLE STRESS = 46 x 10^3 /3 = 15,333 PSI

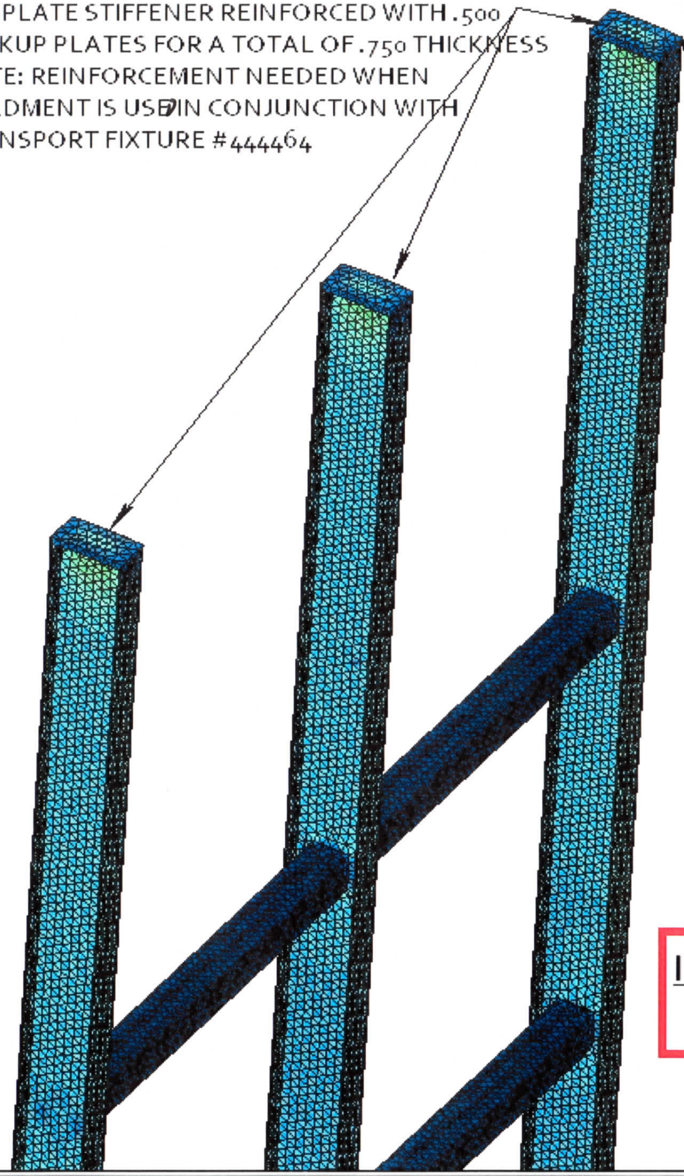
MATERIAL PLATE STIFFENERS : ASTM A36
ALLOWABLE STRESS = 36 x 10^3 /3 = 12,000 PSI

(24) 3/-10 GRADE 5 BOLTS IN SHEAR SUPPORT PALLET WEIGHT OF 2539# PLUS 14,200# WEIGHT OF BLOCK. BOLTS LOCATED ON BOTTOM END NOT SHOWN.

ALLOWABLE TRESS ON EACH BOLT = 92 x 10^3 x .334 /3 = 10,243 PSI

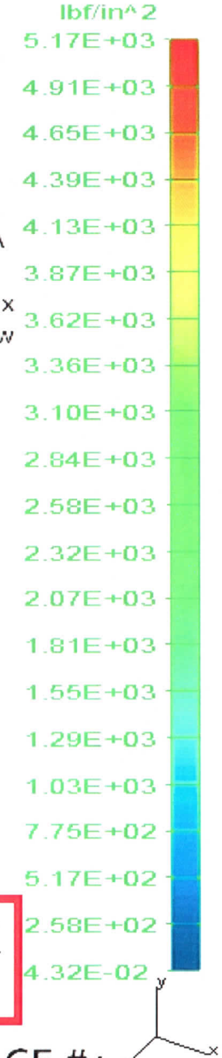
ALLOWABLE STRESS = (2539 + 14,200#) /24 x .334 = 2,088 PSI < 10,243

.250 PLATE STIFFENER REINFORCED WITH .500 BACKUP PLATES FOR A TOTAL OF .750 THICKNESS
NOTE: REINFORCEMENT NEEDED WHEN WELDMENT IS USED IN CONJUNCTION WITH TRANSPORT FIXTURE #444464



ALL FILLET WELD SIZES = .250"
fw = 8370 #/14" x 6 = 100 #/IN

USING E70XX ELECTRODE WITH A YIELD STRESS = 60 x 10^3, THEN THE ALLOWABLE STRESS = sin45 x .250" x 60 x 10^3 /3 = 3,536 #/IN > fw

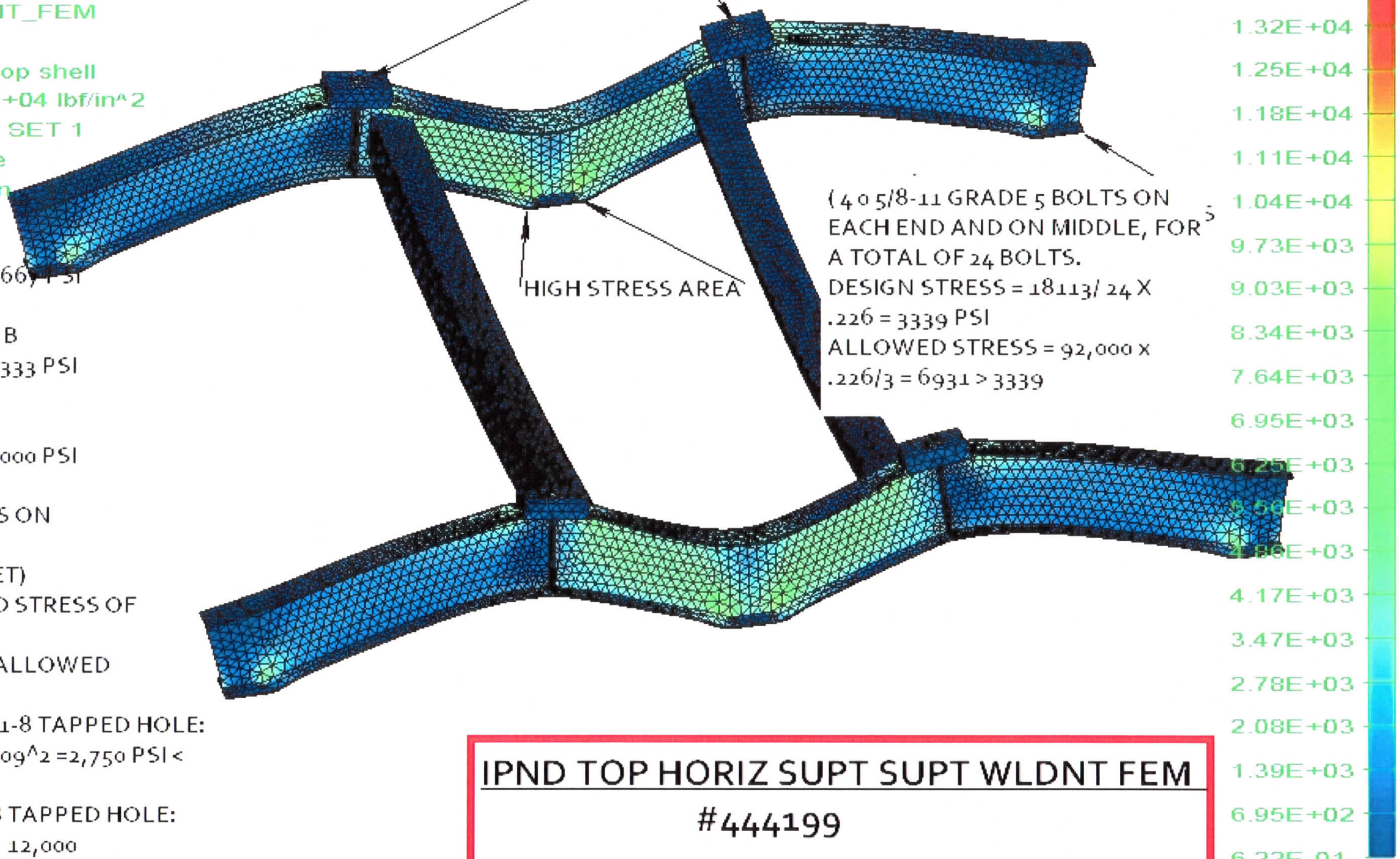


IPND VERT SUPT WLDNT
#444198

PAGE #4

I-DEAS 12 NX Series m2: I-DEAS Visualizer

IPND_TOP_HORIZ_SUPT_WLDNT_FEM
18113LB_LOAD
STRESS Von Mises Averaged Top shell
Min: 6.22E-01 lbf/in^2 Max: 1.39E+04 lbf/in^2
B.C. 1,DISPLACEMENT_1,LOAD SET 1
DISPLACEMENT XYZ Magnitude
Min: 0.00E+00 in Max: 7.08E-03 in



MATERIAL ALL SHAPES: ASTM A992
ALLOWABLE STRESS = 50 X 10^3/3 = 16,667 PSI

MATERIAL TUBING: ASTM A500 GRADE B
ALLOWABLE STRESS = 46 X 10^3/3 = 15,333 PSI

MATERIAL STIFFENERS : ASTM A36
ALLOWABLE STRESS = 36 X 10^3/3 = 12.000 PSI

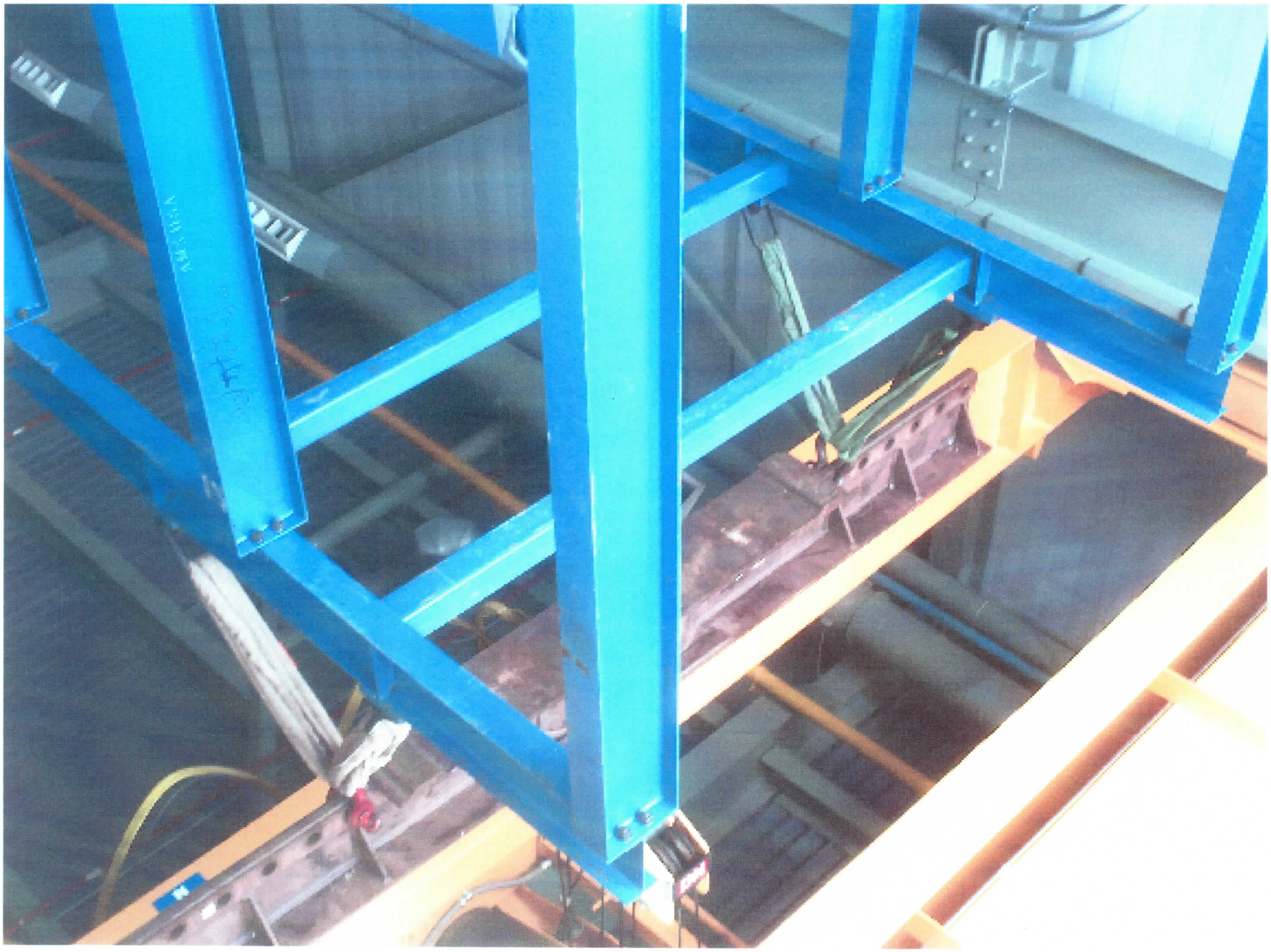
ALL WELDS 1/4"FILLETS EXCEPT WELDS ON
1.500 PLATE WHICH ARE .3125" MIN.
fw = 18,113/4X4"X2 = 566#/IN (5/16 FILLET)
USING E70XX ELECTRODE WITH A YIELD STRESS OF
60X10^3 PSI,
Fw = sin45 x .3125 x 60,000/3 = 4419#/IN ALLOWED

DESIGN THREAD BENDING STRESS ON 1-8 TAPPED HOLE:
fb = 3 x 18113 x .068/4 x 3.14 x12 x .75 x .109^2 =2,750 PSI <
12,000
DESIGN THREAD SHEAR STRESS ON 1/8 TAPPED HOLE:
fv = 18113/4 X 12 X.75 X .109 = 1,469PSI < 12,000

IPND TOP HORIZ SUPT SUPT WLDNT FEM
#444199

FIXTURE WAS LOADED WITH
18,250# (129% OF 14,200#) LIFTED APPROXIMATELY
~~AB~~ 1" TO 1 1/2" OFF THE FLOOR AND HELD
FOR 5 MINUTES


FRAME I
B



BELOW-THE-HOOK LIFTING DEVICE
Engineering Note Cover Page

Lifting Device Numbers:

FNAL Site No/ 171 (If applicable) Div. Specific No. ______ (If applicable) Asset No. ______ (If applicable)

ASME B30.20 Group: (check one)
- [ ] Group I Structural and Mechanical Lifting Devices
- [ ] Group II Vacuum Lifting Devices
- [ ] Group III Magnets, Close Proximity Operated
- [ ] Group IV Magnets, Remote Operated

Device Name or Description ______

Device was (check all applicable)
- [ ] Purchased from a Commercial Lifting Device Manufacturer. Mfg Name ______
- [ ] Designed and Built at Fermilab
- [ ] Designed by Fermilab and Built by a Vendor. Assy drawing number ______
- [ ] Provided by a User or other Laboratory
- [ ] Other: Describe ______

Engineering Note Prepared by ______ Date ______

Engineering Note Reviewed by [signature] Date 10/27/09

Lifting Device Data:

Capacity ______

Fixture Weight ______

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

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

Inspections Frequency ______

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) ______

This manual is subject to change. The current version is maintained on the ESH Section website.

04/09/2010 13:48 FAX 1 630 252 5782 HEP 001

Calculations on IPND Segment Frame and Cart

1.0 Introduction

The IPND cart is a fixture that performs several tasks. First, the fixture provides a surface and framework for the assembly of the IPND in on its side. In the assembly position the IPND and fixture will be transported to FNAL where it will be rotated to the upright position. The fixture is then used to support the IPND blocks when they are filled above ground. Later, the IPND blocks will be emptied and the fixture will be used to lower the IPND into the MINOS service hall underground where the blocks will be filled again.

The ASME-BTH-1-2008 Design of Below the Hook Lifting Devices standard was used to evaluate the fixture when it is lowered into the underground MINOS hall. Since the lift only occurs 1-2 times and the loads are completely predictable the device will be classified as a Design Category A and Service Class 0. The factor of safety, Nd, therefore should be 2 or greater.

AISC LRFD Steel specification was used to evaluate the IPND cart in all other loading conditions.

All welding will be done according AWS 14.1

The reference drawings are:
3929.000-ME-44464 Nova-Detector-Prototype IPND Assembly Table which shows the cart in the assembly configuration

3929.000-ME-444202 IPND Lifting Fixture and CAR which shows the cart as a lifting fixture for lowering into the MINOS hall

3929.000-ME-444253 IPND Lifting Fixture Bookend Assembly which shows the cart in the Detector position

2.0 Define Inputs

$Nd := 2.0$ Safety factor for Design Category A and Service Class 0 lifting Device

$plf := \frac{lbf}{ft}$ $psf := \frac{lbf}{ft^2}$ $kips := 1000lbf$ $ton := 2000lbf$ $ksi := 1000psi$

$Ws := 22.0ton$ Weight of filled IPND Segment of 31 Planes

$We := 7.0ton$ Weight of empty IPND Segment of 31 Planes

$Wc := 2100lbf$ Weight of Bottom Cart

$Lp := 104in$ Length of Base Plate - Length of Rectangular Tube Supports

$Lw := 83in$ Width of Base Plate - Length of I-beam supports

$pe := \frac{1.4 \cdot We}{Lp \cdot Lw}$ $pe = 2.27 \cdot psi$ Distributed pressureload on base plate with empty modules

$ps := \frac{1.4 \cdot Ws}{Lp \cdot Lw}$ $ps = 7.14 \cdot psi$ Distributed pressure load on base plate with filled modules

3.0 Evaluation of Cart Base/Pallet

See Drawing # 3929.000-MD-444200

3.1 Calculations on the Rectangular Tubes -- Lifting Cart

There are seven rectangular tubes that support the IPND block when it is in the detector position. Each tube is supported at two points by an I-beam as shown in MD-444200 when empty and the cart is on the caster wheels. The tubes are supported at 3 point when the block is filled and the 3 I-beams shown in MD-444200 are supposed on blocks at their ends.

3.1.1 Calculation of moments and stresses in each tube Rectangular Tube -- Item #2 shown in MD-444200

3.1.1.1 Calculate the moments in the tube in the filled condition. Assume that the tubes are supported at three points (ends and middle) and that the supporting I-beams are fully supported on the floor.

Calculate the bending moment the bottom rectangular tubes which support the weight across the width of the segment. There are seven tubes. Assume each tube supports Lp/7 width of the plate

For a 8x2x1/4 tube:

$Sx := 2.94in^3$ $\quad Ix := 2.94in^4$ $\quad fy := 36ksi$ $\quad Es := 29000ksi$

$a := 52in$ Distance between the end support and the center support

$s := \frac{Lw}{7}$ $\quad s = 11.86 \cdot in$ Width of Plate Supported by one Rectangular Tube

$qt := ps \cdot s$ $\quad qt = 84.62 \cdot \frac{lbf}{in}$ Distributed loading of filled module along the length of the rectangular tube

$MR2 := -.125 \cdot qt \cdot a^2$

$MR2 = -28.60 \cdot kips \cdot in$ Moment at center support

$Mc := 0.07 \cdot qt \cdot a^2$

$Mc = 16.02 \cdot kips \cdot in$ Moment at mid-span between supports

$R1 := \frac{3}{8} \cdot qt \cdot a$

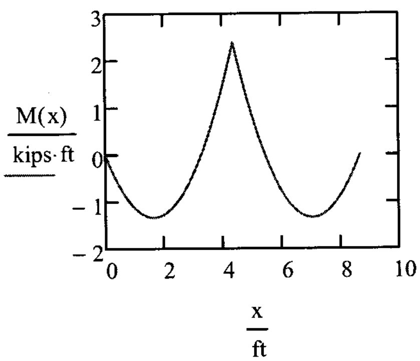
$R1 = 1.65 \cdot kips$ Reaction force at end supports

$R2 := \frac{5}{4} \cdot qt \cdot a$

$R2 = 5.50 \cdot kips$ — Reaction force at center support

$$M(x) := \begin{cases} -R1 \cdot x + \dfrac{qt \cdot x^2}{2} & \text{if } x \le a \\ -R1 \cdot x - R2 \cdot (x - a) + \dfrac{qt \cdot x^2}{2} & \text{if } x > a \end{cases}$$

$x := 0.0in, .1in .. 2 \cdot a$



$M\left(\frac{a}{2}\right) = -1.19 \cdot kips \cdot ft$

$M(a) = 2.38 \cdot kips \cdot ft$

$Mmax := M(a)$

$fy := 46ksi$ — Yield stress of material

$$fce := \frac{|Mmax|}{Sx}$$

$fce = 9.73 \cdot ksi$ — Max Beam Bending Stresses

$SF := \dfrac{fy}{fce}$ $\quad SF = 4.73$ — Safety factor on stresses

$fmax := \dfrac{1.25 \cdot fy}{Nd}$ $\quad fmax = 28.75 \cdot ksi$ — Maximum allowable stress defined by ASME BTH-1

$\dfrac{fmax}{fce} = 2.96$ — Maximum bending stress in beam less than fmax

3.1.1.2 Moments/Stresses in the support tubes in the unfilled condition on the casters. The block is supported only on the outer I-beams with Casters

$a := 104in$ Distance between the end support and the center support

$s := \frac{Lw}{7}$ $s = 11.86 \cdot in$ Width of Plate Supported by one Rectangular Tube

$qe := pe \cdot s$ $qt = 84.62 \cdot \frac{lbf}{in}$ Distributed loading of empty block along the length of the rectangular tube

$R := \frac{qe \cdot a}{2}$

$R = 1400.00 \cdot lbf$ Reaction forces at end of tube on I-beam

$M := \frac{qe \cdot a^2}{8}$

$M = 36.40 \cdot kips \cdot in$ Maximum moment at center of beam

$fc := \frac{M}{Sx}$

$fc = 12.38 \cdot ksi$

$SF := \frac{fy}{fc}$ $SF = 3.72$ Safety factor on beam stresses

$fmax = 28.75 \cdot ksi$ Defined in Section 3.1.1.1 above for maximum allowable stress defined by ASME BTH-1

$\frac{fmax}{fc} = 2.32$ The maximum allowable stress in the beam is less than fmax

3.1.2 Calculation of Rectangular Tube Deflections

HSS 8x2x1/4 rectangular tube with Sy=2.94 in^3 and Iy=2.94in^4

$Es := 29000ksi$ Modulus of steel

$Iy := 2.94in^4$ Moment of inertia of HSS 8x2x1/4 tube

$qe = 323.08 \cdot plf$ Distributed load calculated in section 2.1.2 above

$a := 104in$ Distance between supports

$ymax := \frac{5 \cdot qe \cdot a^4}{384 \cdot Es \cdot Iy}$ Maximum deflection at center of tube between caster supports

$ymax = 0.48 \cdot in$

$\frac{a}{ymax} = 216.21$ Acceptable deflection over the length of support. Actual deflections should be much smaller because of the added stiffness of the bottom plate which is not included in the calculation.

3.2 Calculate Loading and Stresses in the Support I-beam when supported at Casters support points
Use W8x10 Beam

$Ix := 30.8in^4$ $Sx := 7.81in^3$

3.2.1 Evaluate the Moments/Stresses in the I-Beams when supported on the Casters and blocks empty

$qe := \frac{pe \cdot Lp}{2}$

$qe = 1416.87 \cdot plf$ Distributed loading on I-beam

$a := 20.5in$ Distance from Edge to First Support

$b := 41in$ Distance from edge to second support

$c := a$ Distance from 2nd support to end.

$Lw := b + 2 \cdot a$ Length of I-beam

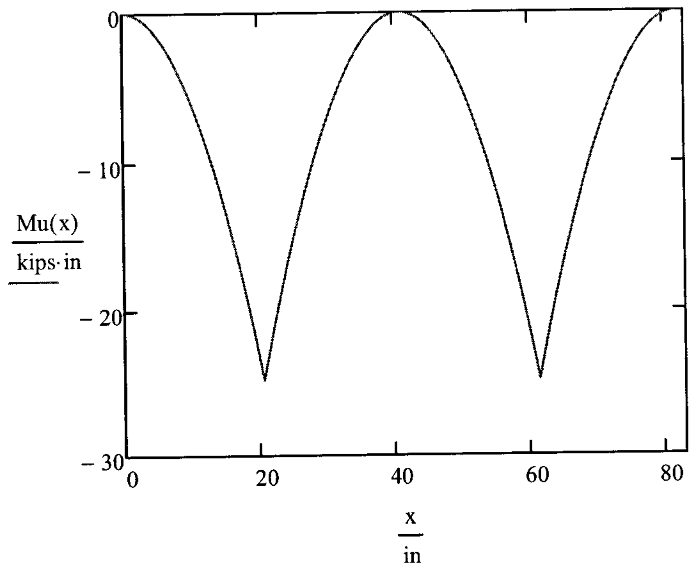
Reaction force on beam from each rectangular tube is equal to R1

$R := \frac{qe \cdot Lw}{2}$

$R = 4.84 \cdot kips$ Force on Caster

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

$x := 0.0in, 0.1in .. Lw$



$x := \frac{2 \cdot a + b}{2}$

Given $\quad x < Lw$

$x > 0ft$

$xmax := Maximize(Mu, x)$ $\qquad xmax = 41.00 \cdot in$

$Mumax := Mu(xmax)$ $\qquad Mumax = 0.00 \cdot kips \cdot ft$

$x := \frac{2 \cdot a + b}{2}$

Given $\quad x < a + b$

$x > a$

$xmin := Minimize(Mu, x)$ $\qquad xmin = 20.50 \cdot in$

$Mumin := Mu(xmin)$ $\qquad Mumin = -24.81 \cdot kips \cdot in$

$fc := \frac{|Mumin|}{Sx}$

$fc = 3.18 \cdot ksi$ $\qquad$ Maximum beam stress

$SF := \frac{fy}{fc}$ $\qquad SF = 14.48$ $\qquad$ Safety factor

$fmax := \frac{1.1 \cdot fy}{Nd}$ $\qquad$ Maximum allowable stress for strong axis bending defined by ASME BTH-1

$fmax = 25.30 \cdot ksi$

$\frac{fmax}{fc} = 7.96$ The maximum stress in the beam is less than fmax

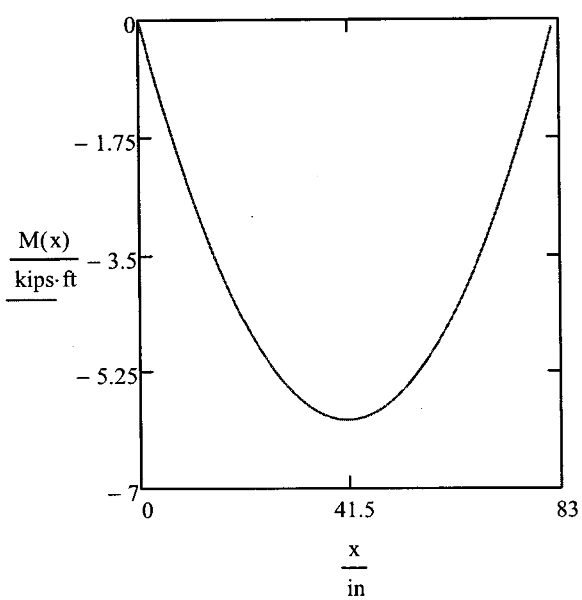
3.2.2 Calculation of Strength of Support I-beams when supported at the ends (lifting case with blocks empty)

$R1 := \frac{We}{4}$ $R1 = 3.50 \cdot kips$

$qe := \frac{\frac{We}{2}}{Lw}$ $qe = 1024.39 \cdot plf$

$M(x) := \frac{qe \cdot x^2}{2} - R1 \cdot x$

$x := 0ft, .1ft .. Lw$



$x := \frac{Lw}{2}$

Given $x < Lw$

$x > 0ft$

$xmax := Maximize(M, x)$ $\qquad xmax = 0.00 \cdot in$

$Mumax := M(xmax)$ $\qquad Mumax = 0.00 \cdot kips \cdot ft$

$x := \frac{Lw}{2}$

Given $\qquad x < Lw$

$x > 0ft$

$xmin := Minimize(M, x)$ $\qquad xmin = 41.00 \cdot in$

$Mumin := M(xmin)$ $\qquad Mumin = -5.98 \cdot kips \cdot ft$

$fc := \frac{|Mumin|}{Sx}$

$fc = 9.19 \cdot ksi$

$SF := \frac{fy}{fc}$ $\qquad SF = 5.01$ $\qquad$ Safety factor

$fmax = 25.30 \cdot ksi$ $\qquad$ Maximum allowable stress defined by ASME BTH-1 in Section 3.2.1 above

$\frac{fmax}{fc} = 2.75$ $\qquad$ The maximum stress in the beam is less than fmax

4.0 Analysis of the Connections between the bottom Pallet and the Lifting Fixture

4.1 Analysis of Bolted Connection from cart to Vertical Support
Connection Subjected only to vertical loading (bolts in shear) during lifting.
See Drawing # MD-444197 and MD-444482

Define Inputs:

$Fu := 50000psi$	Tensile Strength of Plate Material
$Fy := 36ksi$	Yield Strength of Plate Material
$Fub := 120ksi$	Tensile Strength of A325 Bolt Material
$n := 13$	Threads per inch
$N := 8$	Total Number of Bolts
$db := 0.75in$	Bolt Nominal Diameter
$d := db + \frac{1}{16}in$	Clearance hole in plate for bolt

$m := 1$ Number of Shear Planes

$t := 0.5in$ Thickness of Plate

$Le := 1.31in$ Distance along the line of force from edge of the connected part to the center of hole

$L := 6.56in$ Length of the connection in the direction of shear from the edge of the plate to the last hole

$n3 := 4$ Number of holes in a line in the direction of the shear load

$h := 0.875in$ 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.

Calculate the shear load per bolt

$Ru := \frac{We + Wc}{4}$ $Ru = 4.03 \cdot kips$ Vertical Load per Connection

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

$Vu = 0.70 \cdot kips$ Factored shear force per bolt.

Calculation of Areas

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

$Ab = 0.44 \cdot in^2$

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

$An = 0.36 \cdot in^2$

Shear Strength through threads of bolt

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

$\phi Vn = 15.9 \cdot kips$ This is far greater than Vu so ok.

$SF := \dfrac{\phi Vn}{Vu}$ $SF = 22.58$

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.

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

$\phi Vn = 33.7 \cdot kips$

$\dfrac{\phi Vn}{Vu} = 47.91$

Applies in usual conditions based on deformation limits for the hole. This equation applies for all holes 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.

Far greater than Vu so ok

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

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

$\phi Vn = 24.6 \cdot kips$

$SF := \dfrac{\phi Vn}{Vu}$ $SF = 34.87$

Minimum Spacing of Bolts

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

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

$s = 0.41 \cdot in$

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

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

$Lmin = 0.04 \cdot in$

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

$Anv = 1.86 \cdot in^2$

$Agt := h \cdot t$ — 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

$\phi R := \phi f \cdot [(.6 \cdot Fu \cdot Anv) + (Fu \cdot Ant)]$ $\quad \phi R = 50596.88 \cdot lbf$ — Upper Limit AISC

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

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

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

$\phi Rn = 50.6 \cdot kips$ — This is far greater than Vu so ok

$SF := \frac{\phi Rn}{Vu}$ $\quad SF = 71.83$

4.2 Examine Welding of bolt plate to W8x10 bottom support beam

$\phi w := 0.75$	Strength reduction factor of weld
$Lweld := 7.89in$	Length of Weld
$Fexx := 70ksi$	Yield strength of weld material
$Fu := 50ksi$	Ultimate strength of base material
$t := 0.5in$	Thickness of base material
$tw := 0.25in$	Size of fillet weld

$\phi Rn1 := Lweld \cdot tw \cdot \phi w \cdot 0.707 \cdot (0.60 \cdot Fexx)$ Strength of weld based on weld material

$\phi Rn1 = 43.93 \cdot kips$

$\phi Rn2 := Lweld \cdot t \cdot \phi w \cdot (0.60 \cdot Fu)$

$\phi Rn2 = 88.76 \cdot kips$

$$\phi Rn := \begin{vmatrix} \phi Rn1 & \text{if } \phi Rn1 \le \phi Rn2 \\ \phi Rn2 & \text{otherwise} \end{vmatrix}$$

$\phi Rn = 43.93 \cdot kips$

$Ru = 4.03 \cdot kips$ — Vertical Load on Plate (see calculation at beginning of this section). The weld strength is far greater than the applied factored load

4.3 Analysis of Bolted Connection from Vertical Support to Top Horizontal Support Weldment Connection Subjected only to vertical loading (bolts in pure tension) during lifting.
See Drawing 3929.000-MD-444199 and 329.000-MD-444524 for details of hole pattern and Drawing 3929.000-ME-444201 which shows the fixture in the orientation for vertical lifting

4.3.1 Strength of Hole Pattern and bolts

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
$N := 4$	Total Number of Bolts
$db := 0.625in$	Bolt Nominal Diameter
$d := .75in$	Clearance hole in plate for bolt
$m := 1$	Number of Shear Planes
$t := 0.25in$	Thickness of Plate

Calculate the tensile load per bolt

$$Ru := \frac{We + Wc}{6} \frac{1.4}{N}$$

$Ru = 0.94 \cdot kips$ — Tensile load per bolt. There are 6 connections and four bolts (N) per connection.

Calculation of Areas

$$Ab := \frac{\pi\, db^2}{4}$$

Gross Cross-Sectional Area of One Bolt

$Ab = 0.31 \cdot in^2$

Tensile Strength of Fastener

$\phi f := 0.75$

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

$\phi Rn = 20.71 \cdot kips$

Axial Tensile Design strength of one bolt. The tensile area is the root area of the threads and is assumed to be 75% of the gross area.

$\frac{\phi Rn}{Ru} = 22.05$

4.3.2 Examine Welding of bolt plate to W8x10 top horizontal support beam

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

$Lweld := 7.89 in$ Length of Weld

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

$Fu := 50 ksi$ Ultimate strength of base material

$t := 0.25 in$ Thickness of base material

$tw := 0.25 in$ Size of fillet weld

$\phi Rn1 := Lweld \cdot tw \cdot \phi w \cdot 0.707 \cdot (0.60 \cdot Fexx)$ Strength of weld based on weld material

$\phi Rn1 = 43.93 \cdot kips$

$\phi Rn2 := Lweld \cdot t \cdot \phi w \cdot (0.60 \cdot Fu)$

$\phi Rn2 = 44.38 \cdot kips$

$$\phi Rn := \begin{cases} \phi Rn1 & \text{if } \phi Rn1 \ge \phi Rn2 \\ \phi Rn2 & \text{otherwise} \end{cases}$$

$\phi Rn = 44.38 \cdot kips$

$Ru := \frac{1.4 \cdot (We + Wc)}{6}$

Vertical Load on Plate (see calculation at beginning of this section). The weld strength is far greater than the applied factored load

$Ru = 3.76 \cdot kips$

$SF := \frac{\phi Rn}{Ru}$ $SF = 11.81$

5.0 Examine the Vertical Support Beams of the Lifting Fixture

5.1Calculate the moment and determine the size of the vertical main support beam.
The maximum moment in this beam occurs when the segment is in the horizontal position.

Assume that the pivot is at the front so that the beam is simply supported at the ends

$b := 176.25\text{in}$ Length of vertical supports

$qc := \frac{(We + Wc)}{2b}$ Distributed load on the vertical support beams when the segment frame is rotated to the horizontal position for assembly. These beams prevent tipping when the segment is being moved. There are two beams and each takes half of the weight

$qc = 548.09 \cdot \text{plf}$

$R2 := \frac{qc \cdot b}{2}$ $R2 = 4.03 \cdot \text{kips}$

$R1 := qc \cdot b - R2$ $R1 = 4.03 \cdot \text{kips}$

Calculate internal moment

$x := 0\text{ft}, .1\text{ft} .. b$

$Mu(x) := R1 \cdot x - \frac{qc \cdot x^2}{2}$

$x := 5\text{ft}$

Given $x < b$

$x > 0.0\text{ft}$

$xmax := \text{Maximize}(Mu, x)$ $xmax = 7.34\ \text{ft}$

$Mumax := Mu(xmax)$ $Mumax = 14.78 \cdot \text{kips} \cdot \text{ft}$

$x := 5\text{ft}$

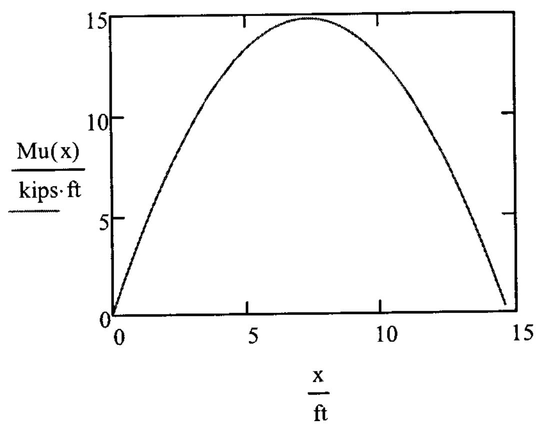
Given $x < b$

$x > 0.0\text{ft}$

$xmin := \text{Minimize}(Mu, x)$ $xmin = 0.00\ \text{ft}$

$Mumin := Mu(xmin)$ $Mumin = 0.00 \cdot \text{kips} \cdot \text{ft}$

$x := 0ft, .1ft .. b$



$Mumax = 14.78 \cdot kips \cdot ft$

$fy := 46ksi$ Yield stress of material

Vertical Supports are W8x15

$Ix := 48in^4$ $Sx := 11.8in^3$ $A := 4.44in^2$ $Zx := 13.8in^3$

$$fc := \frac{|Mumax|}{Sx}$$

$fc = 15.03 \cdot ksi$ Moment of Inertia needed on vertical support beams

$SF := \frac{fy}{fc}$ $SF = 3.06$

Check deflections:

$Es := 29000ksi$

$Ix := 48in^4$

$ymax := \frac{5 \cdot qc \cdot b^3}{384 \cdot Es \cdot Ix}$ $ymax = 0.0023$

6.0 Calculate for Cross Beam for Rotation from Horizontal to Vertical

Drawing #444464 shows the fixture that is used for rotating from the horizontal to the vertical orientation. A rectangular tube, HSS 8x8x1/4 is used to lift the structure from the horizontal to the vertical position. Slings will be wrapped around the tube to insure that the loading always goes through the center of the tube no matter what angle the structure is rotated to. The loading on the tube is the same when in the vertical and horizontal positions.

Calculate the bending moment in the beam assuming a worst case where the entire weight of the block and cart is supported by the tube. Assume that the of the weight is evenly distributed between the three vertical beams attached to the cross beam.

$$R1 := \frac{1.6\cdot(We + Wc)}{4}$$

$R1 = 6.44\cdot kips$ Force from outside vertical beams applied to cross beam.

$$R2 := \frac{1.6\cdot(We + Wc)}{2}$$

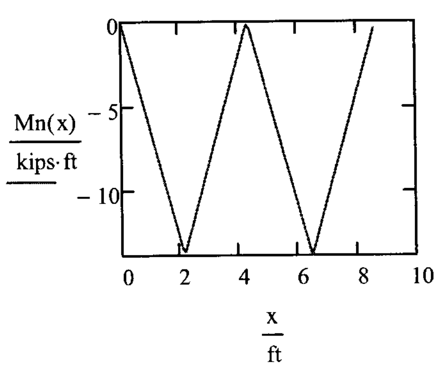
$R2 = 12.88\cdot kips$ Force from middle vertical beam applied to cross beam

$Ra := \frac{2\cdot R1 + R2}{2}$ $Ra = 12.88\cdot kips$ Reaction at the lift points

$s := 26in$ Spacing between beams/supports

$$Mn(x) := \begin{cases} (-R1\cdot x) & \text{if } x < s \\ [-R1\cdot x + Ra\cdot(x - s)] & \text{if } s \le x < 2\cdot s \\ [-R1\cdot x + Ra\cdot(x - s) - R2\cdot(x - 2\cdot s)] & \text{if } 2\cdot s \le x < 3\cdot s \\ [-R1\cdot x + Ra\cdot(x - s) - R2\cdot(x - 2\cdot s) + Ra\cdot(x - 3\cdot s)] & \text{if } 3\cdot s \le x \le 4\cdot s \end{cases}$$

$x := 0.0ft, 0.1ft .. 4\cdot s$



$Mmax := Mn(s)$ $Mmax = -13.95\cdot kips\cdot ft$

For a HSS 8x8x1/4

$Ix := 70.7in^4$ $Sx := 17.7in^3$

$fy := 46ksi$ Yield stress of material

$$fc := \frac{|Mmax|}{Sx}$$

$fc = 9.46\cdot ksi$ Maximum stress in beam

$SF := \frac{fy}{fc}$ $SF = 4.86$

7.0 Calculate the bolt strength of connection between vertical supports and bottom beam during lifting due to shear and bending moment.

Assembly drawing 444635 and 444631. Details of the mounting pattern are shown on 444363

There are eight 3/4" diameter bolts that must take the moment. Assume that the strain in the bolts varies linearly across the height of the plate.

$a := 1.065in$ Distance from center line to first pair of holes

$b := a + 1.75in$ Distance from center line to 2nd pair of holes

$b = 2.81\cdot in$

Calculate moment acting on joint -- moment arm equal to width of block spacer plus half of W8 depth

$Lw = 6.83\,ft$ Moment arm

$$q := \frac{Wc + We}{Lw}$$

$$M := \frac{1}{3}\frac{q\cdot Lw^2}{8}$$

$M = 4.58\cdot kips\cdot ft$

Calculate the maximum tensile load due to bending moment

$$Pu := \frac{M}{2\left(\frac{2\cdot a^2}{b} + 2\cdot b\right)}$$

$Pu = 4.27 \cdot kips$ Maximum tensile loading in bolt

$N := 8$ Total Number of Bolts

$$Vu := \frac{1.6 \cdot (We + Wc)}{2 \cdot N}$$

$Vu = 1.61 \cdot kips$ Shear Force per bolt

Define Inputs for bolt stress calculation:

$Fu := 50000psi$	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
$db := 0.75in$	Bolt Nominal Diameter
$d := db + \frac{1}{16}in$	Clearance hole in plate for bolt
$m := 1$	Number of Shear Planes
$t := 0.25in$	Thickness of Plate
$Le := 1.31in$	Distance along the line of force from edge of the connected part to the center of hole
$L := 3 \cdot (1.75in) + 1.31in$	Length of the connection in the direction of shear from the edge of the plate to the last hole
$n3 := 4$	Number of holes in a line in the direction of the shear load
$h := 0.875in$	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.

Calculation of Areas

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

$Ab = 0.44 \cdot in^2$

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

$An = 0.36 \cdot in^2$

Shear Strength through threads of bolt

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

$\phi Vn = 15.9 \cdot kips$

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.

$fuv := \frac{Vu}{Ab}$ $fuv = 3644.29 \cdot psi$

$\phi F'ut := \phi f \cdot [(0.75 \cdot Fub \cdot C) - (2.5 \cdot fuv)]$ Allowable tensile stress when threads are not in shear plane.

$\phi F'ut = 80.92 \cdot ksi$

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

$\phi Rn = 26.81 \cdot kips$ Maximum Allowable Tensile Force per Bolt

This is greater 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.

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

$\phi Vn = 16.9 \cdot kips$

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.

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

$\phi Vn = 12.3 \cdot kips$

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

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

$s = 0.55 \cdot in$

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

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

$Lmin = 0.17 \cdot in$

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

$Anv = 0.93 \cdot in^2$

$Agt := h \cdot t$ 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

$\phi Ru := \phi f \cdot [(.6 \cdot Fu \cdot Anv) + (Fu \cdot Ant)]$ $\phi Ru = 25.30 \cdot kips$ Upper Limit AISC

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

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

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

$\phi Rn = 25.3 \cdot kips$

Ernest Villegas

From: "ingrid" <ingridf@fnal.gov>
To: <villegas@fnal.gov>; "'Dave Pushka'" <pushka@fnal.gov>; <wands@fnal.gov>
Sent: Wednesday, April 28, 2010 5:40 PM
Attach: Nova Lifting Fixtures Note.doc
Subject: Nova two lifting fixtures

Hi all,

Attached is the results of my investigation of the 31 plane lifting fixture and the IPND block assembly and shipping table lifting fixture.

see Pg #2 & Pg #1

As you will see, three changes are necessary. They are

1. Weld size shall be reduced from 5/16 inch to 3/16 inch due to the AISC code requirements
2. One bolt shall be upgraded from grade 5 to grade 8 due to my new analysis.
3. Two plates shall be added to increase the weld length to compensate for reduction of the weld size. (I am not wrong on this =S)

Bye for now,
Ingrid

April 26, 2010

Stress Analysis of Nova Lifting Fixtures

Ingrid Fang

<u>Introduction and Summary</u>

The original Nova 31 plane lifting fixture and IPND block assembly and shipping table lifting fixture were designed by Victor Guarino from Argonne with the assumption that both fixtures would remain supported by the floor during the installation procedure. It was noted, that in the field, both fixtures were lifted totally off the floor during the rotation process. Ernie Villegas was asked to check the existing fixtures to verify if they meet the requirements of Fermilab ES&H standard 5022 "Below-the-Hook Lifting Devices."

This standard requires that a lifting fixture shall be designed to withstand the forces imposed by its rated load, with a minimum design factor of 3, based on yield strength, for load bearing structural components.

I was asked to performed an independent analysis of both fixtures and verify Ernie's modifications.

The resulting FEA shows that the stresses are well within the lifting fixture requirements. However, the joint forces were examined and these joints were found to be inadequate as originally designed in both fixtures.

For Nova 31 plane lifting fixture, the ¼ inch filet welds between the ¼ inch plate and W8x10 will have to be resized based on the AISC code requirements for weld size. The maximum size is limited to the thickness of the thinnest part to be joined which is 3/16 inch.

For IPND block assembly and shipping table lifting fixture, one ¾ inch diameter grade 5 bolts will have to be upgraded to grade 8. The 5/16 inch filet welds under the lifting point will have to be resized based on the AISC code requirements for weld size. The proper size will have to be 3/16 inch. Another 5/16 inch filet weld at the connection of the top frame will also have to be decreased to 3/16 inch for the same reason. Since the weld size was reduced, two ½ inch plates will have to be added at this joint to increase the weld length.

THESE CHANGES WERE NOT IMPLEMENTED PER MY INSTRUCTIONS
Ernie Villegas
04/29/2010

If these changes are implemented, both fixtures will meet the requirements of Fermilab ES&H standard 5022 "Below-the-Hook Lifting Devices."

Geometry and Loading

Nova 31 plane lifting fixture is as shown in dwg 444201. This lifting fixture will either be picked up at four lifting lugs with the external load of 14200 lbs or rested on the floor with external load of 48000 lbs.

Nova IPND block assembly and shipping table lifting fixture is as shown in dwg 466694. This lifting fixture will be picked up at an angle of 23.037 degrees from the vertical with the external load of 14200 lbs.

Material Properties and Allowable Stresses

This lifting fixture is made of A36 steel plates, A500 steel tubes and A992 wide flange beams. The minimum specified yield of material is 36 ksi for A36, 46 ksi for A500 and 50ksi for A992 respectively. Therefore, the maximum stress (calculated as shear, normal, bending, or a combination thereof, and neglecting stress concentrations) will be limited to 12ksi for A36 ,15.3 ksi for A500 and 16.7 ksi for A992 for normal and bending stress.

The Finite Element Model

The finite element model for Nova 31 Plane lifting fixture is shown in Fig.1. The fixture weighs 4227 lbs and is designed either to be picked up off the floor or remain flat on the floor. The models are referred to in this report as Case1 with 14200 lbs load off the floor and Case2 with 48000 lbs load on the floor respectively. The top tubes next to the lifting lugs are checked for buckling due to the use of the lifting fixture and the sling angle. The actual required welds and bolts are sized with hand calculations covered later in this report.

The finite element model for IPND block assembly and shipping table lifting fixture with 14200 lbs load is shown in Fig. 2. This lifting fixture weighs 5017 lbs and is designed to be lifted off the floor at two pick up points. The model is referred to in this report as Case3. The actual required welds and bolts are also sized with hand calculations covered later in this report.

Results

The deformed shapes of the Nova 31 Plane lifting fixture are shown in Figs.3 and 4.

The stresses in the lifting Fixture are shown in Figs.5 and 6. In the regions away from the concentrations, all stresses are below the 12 ksi limit for A-36 steel,15.3 ksi limit for A500 tube and 16.7 ksi limit for A992 wide flange beam.

The allowable buckling load for 4x4x1/4 x 8ft long tube is 75 kips. The applied force is 4607 lbs {(14200+4227)/4}. Therefore the tube has a safety factor of 16 for buckling.

The deformed shapes of the Nova IPND block assembly and shipping table lifting fixture are shown in Figs.7, 8 and 9.

The stresses in the lifting Fixture are shown in Figs.10, 11, 12 and 13. In the regions away from the concentrations, all stresses are below the 12 ksi limit for A-36 steel,15.3 ksi limit for A500 tube and 16.7 ksi limit for A992 wide flange beam.

Fig. 3 shows the expected Uy deformations in Case1.

Fig. 4 shows the expected Uy deformation in Case2.

Fig. 5 shows the expected stress in Case1.

Fig. 6 shows the expected stress in Case2

Fig. 7 shows the expected Ux deformations in Case3.

Fig. 8 shows the expected Uy deformation in Case3.

Fig. 9 shows the expected Uz deformations in Case3.

Fig. 10 shows the expected stress of the bottom in Case3.

Fig. 11 shows the expected stress of the side1 in Case3.

Fig. 12 shows the expected stress of the top in Case3.

Fig. 13 shows the expected stress of the side2 in Case3.

Welded and Bolted Connections

Welded and bolted connections were checked by extracting nodal forces and moments from all of the finite element models at the top and bottom corners of both fixtures. The welded and bolted connection calculations are given in Appendix I. According to this analysis, the new weld size as specified in this report will be adequate for the imposed loads. Also, the resized bolts will be adequate for the imposed loads.

Conclusion

Both Fixtures will meet the requirements of Fermilab ES&H standard 5022 "Below-the-Hook Lifting Devices" with the modification outlined in this report.

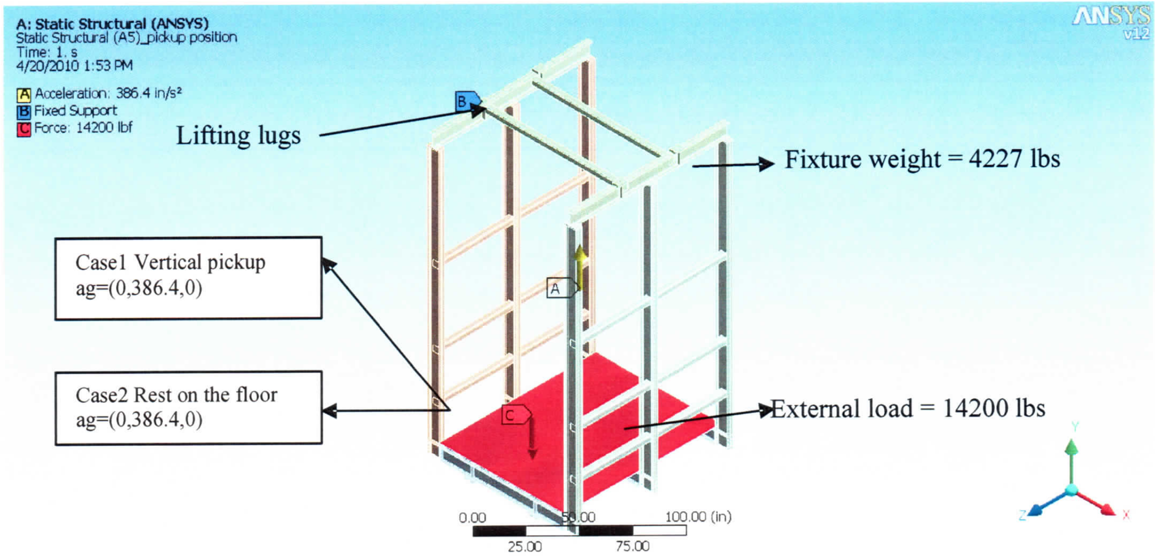


Figure 1. The finite element models for Nova
31 Plane lifting fixture

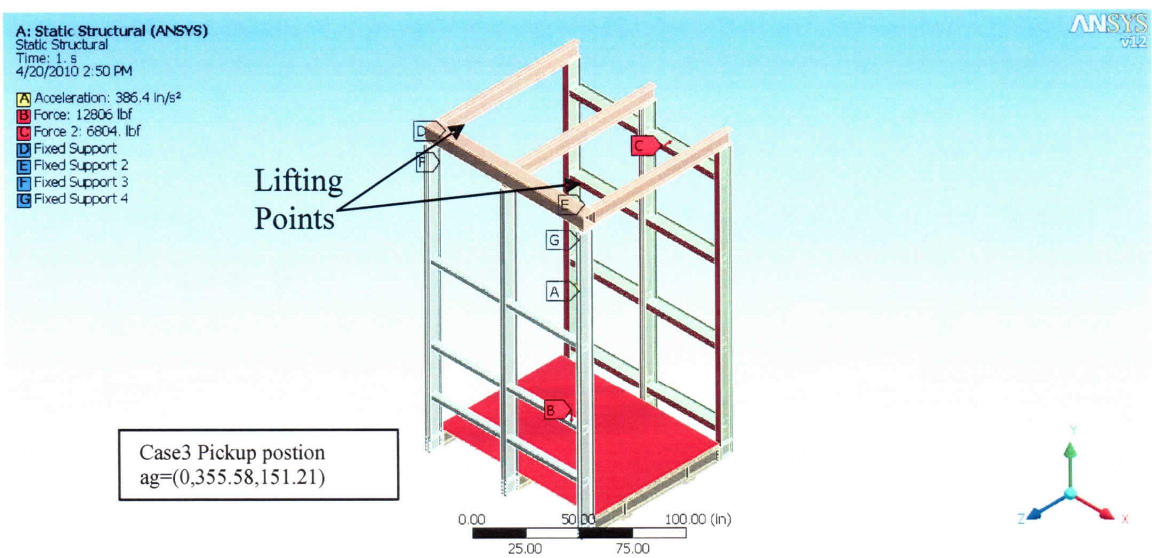


Figure 2. The finite element models for Nova IPND Block
Assembly and Shipping Table lifting fixture

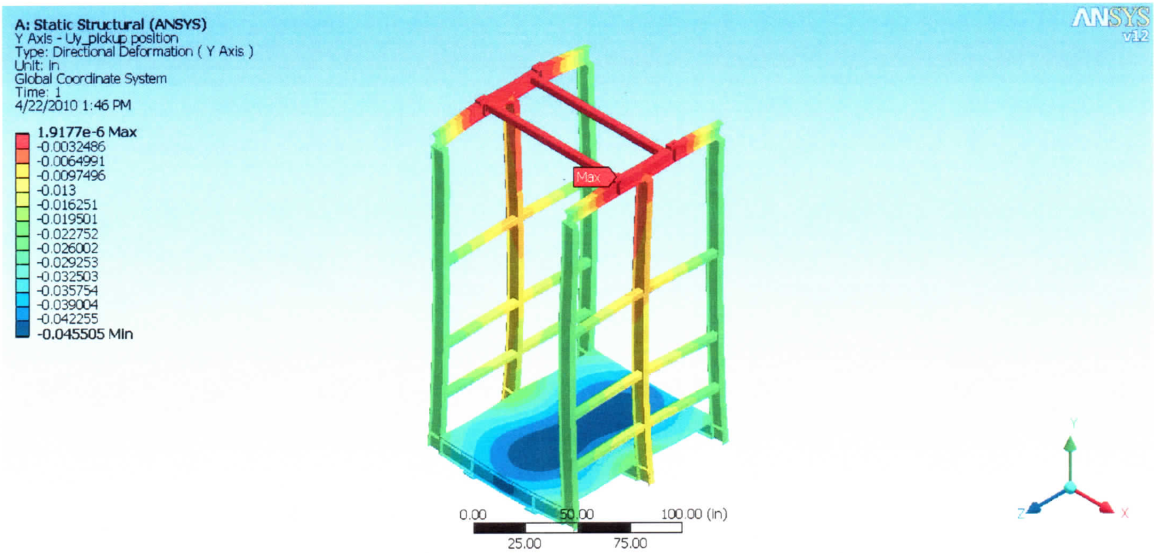


Figure 3. Deformation Uy in Case1

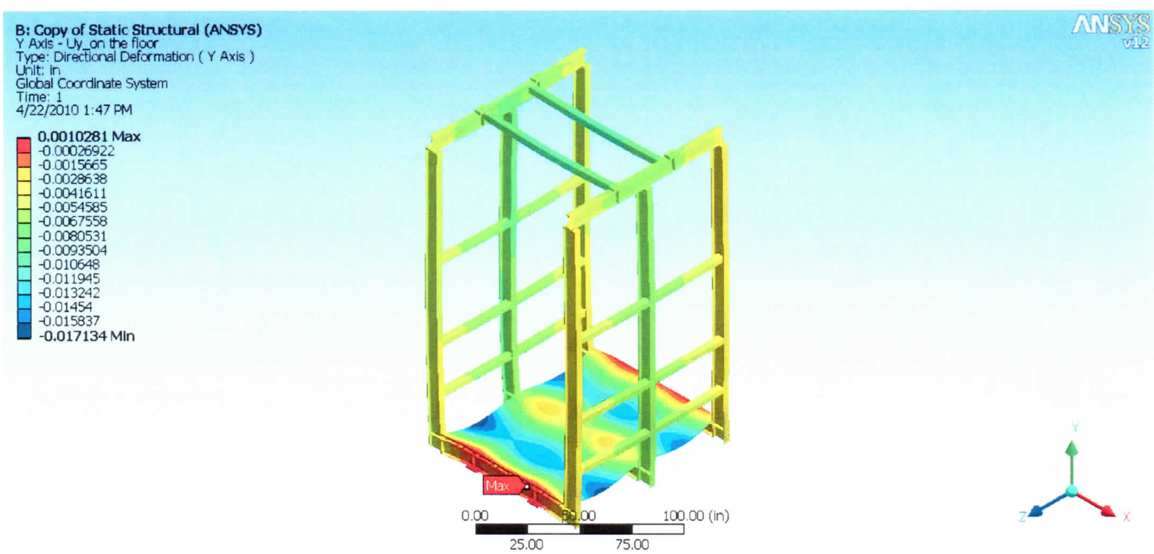


Figure 4. Deformation Uy in Case2

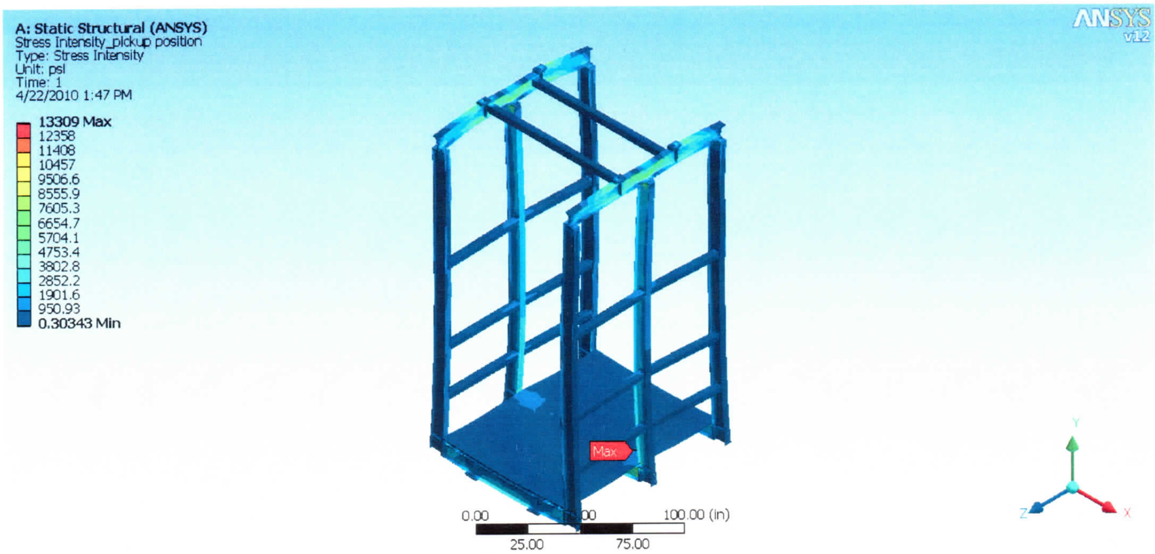


Figure 5. Stress Intensity in Case1

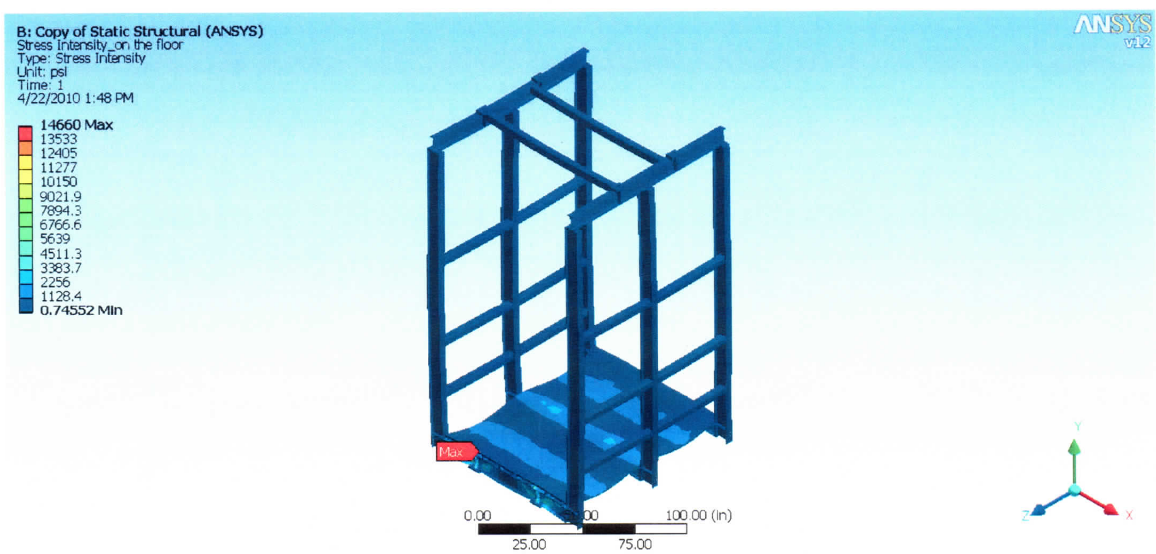


Figure 6. Stress intensity in Case2

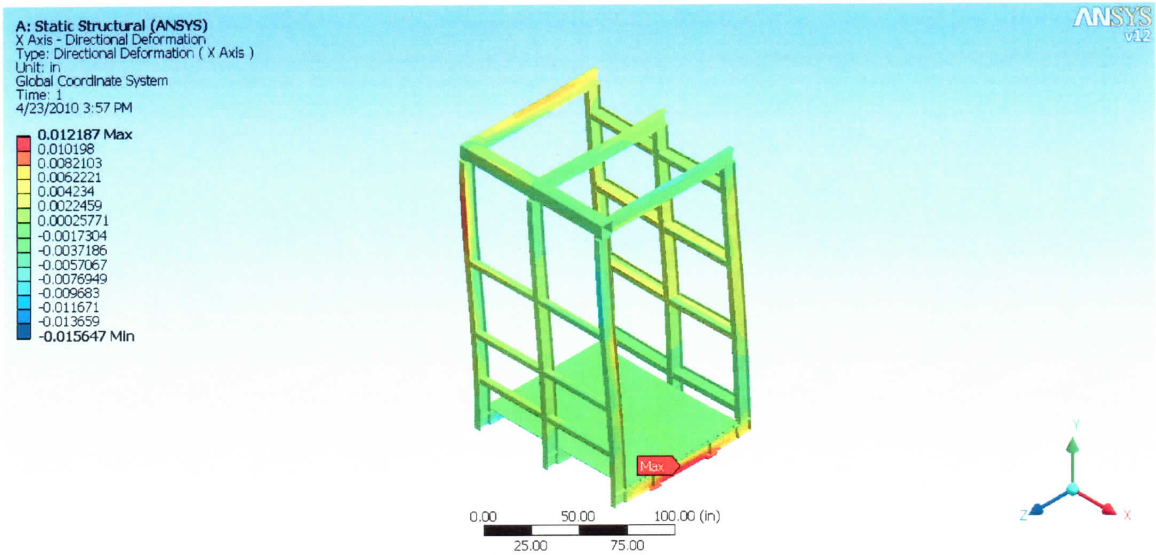


Figure 7. Deformation Ux in Case3

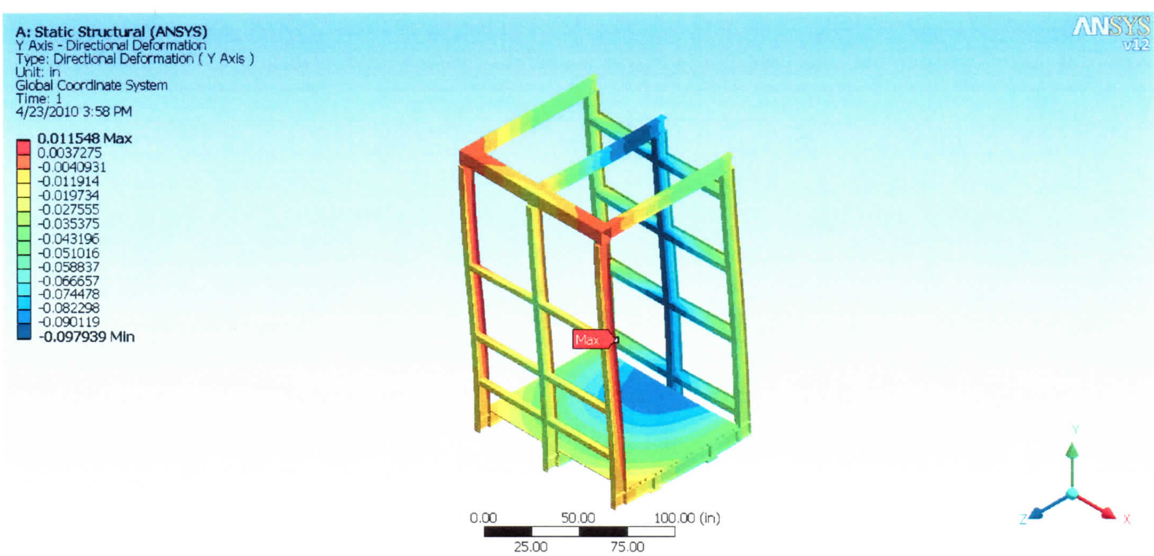


Figure 8. Deformation Uy in Case3

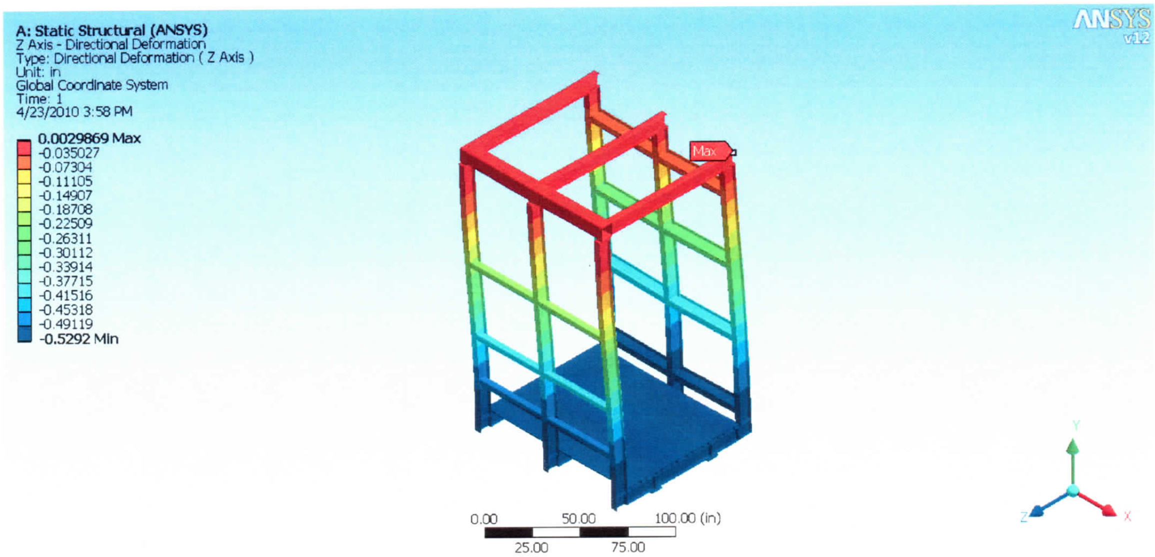


Figure 9. Deformation Uz in Case3

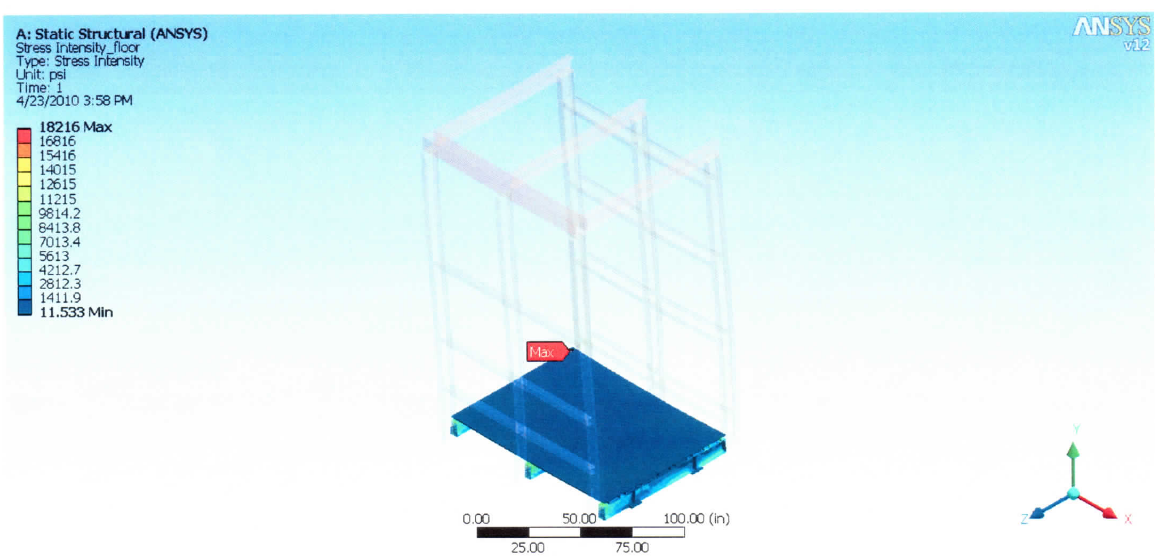


Figure 10. Bottom Stress Intensity in Case3

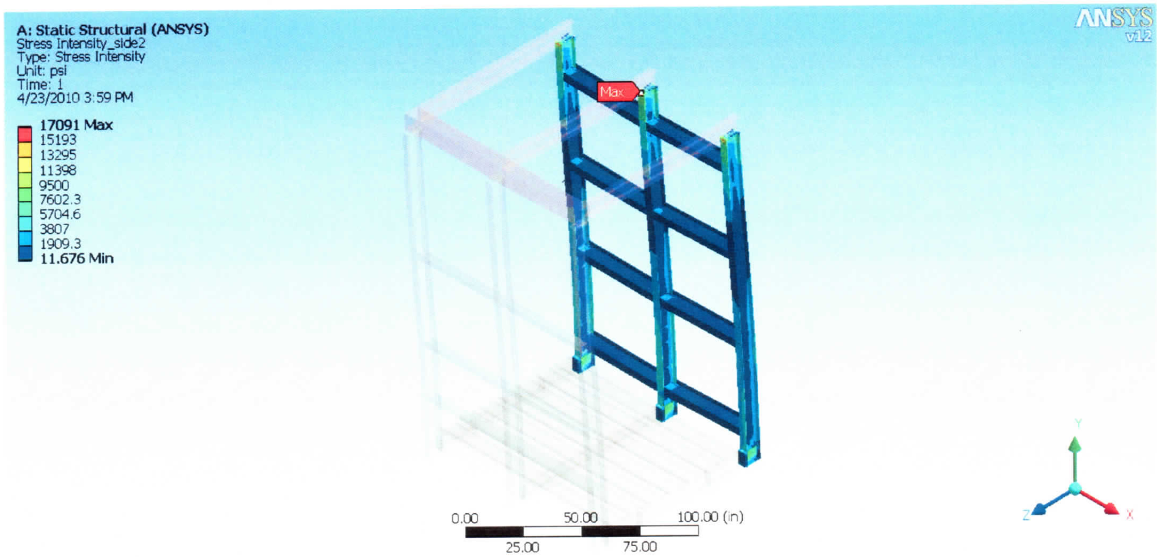


Figure11. Side1 Frame Stress Intensity in Case3

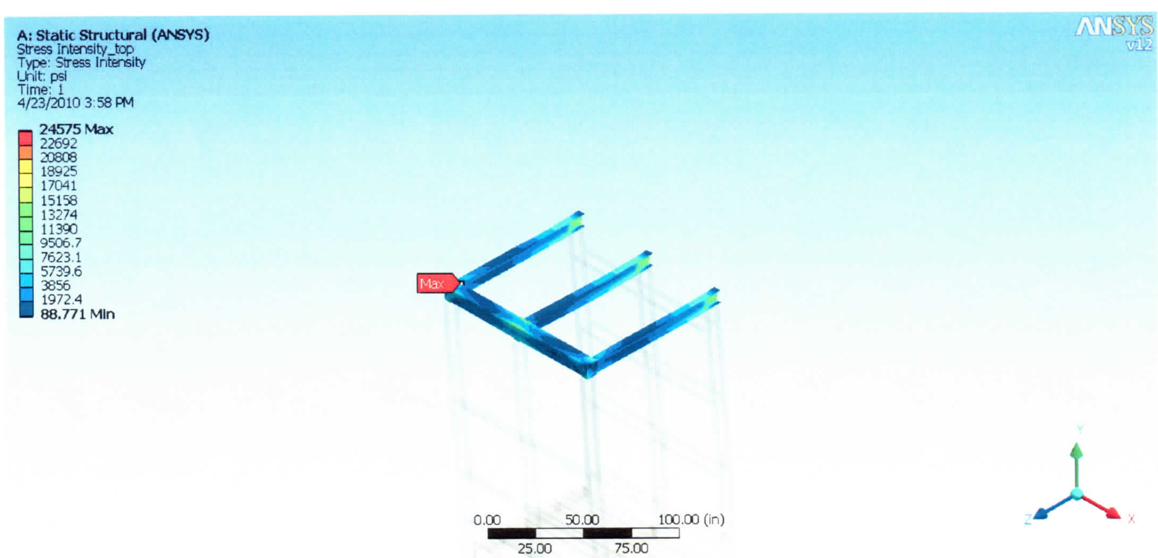


Figure12. Top Frame Stress Intensity in Case3

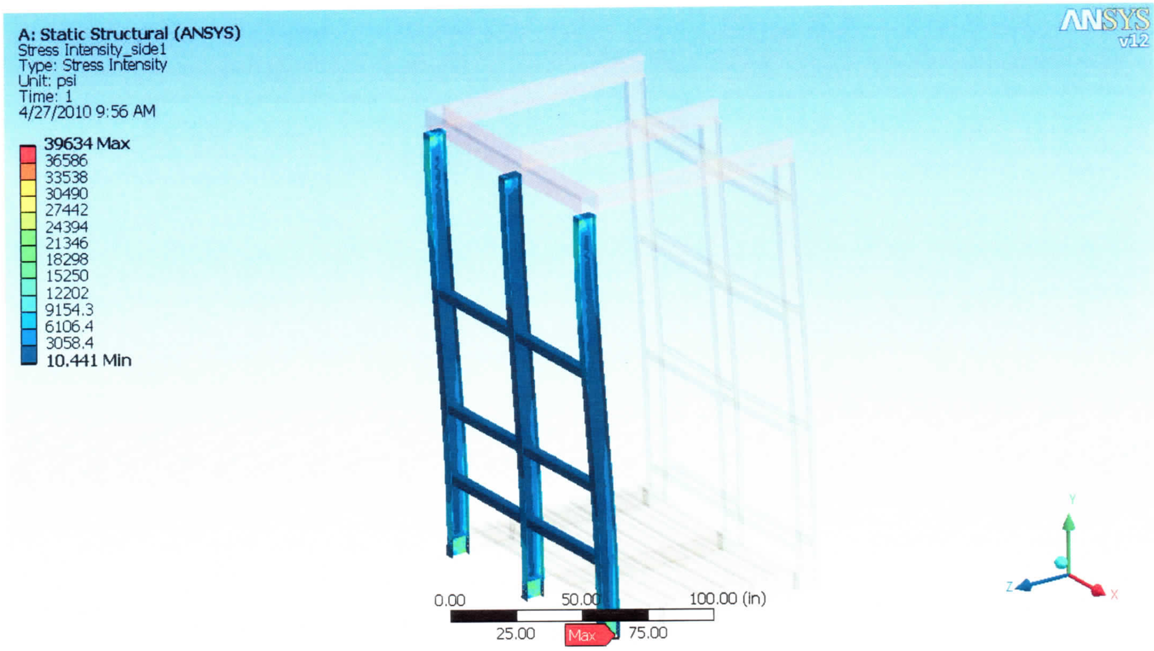


Figure11. Side2 Frame Stress Intensity in Case3

Appendix I

Design of welded and bolted connections

Welds between 1/4 inch plate ,W8x10 and 1/2 inch plate

3/16 inch filet welds @ 3 places (changed from old 5/16 inch to new 3/16)

Load on joint from Table1 in Caes3 using the worst combination in case3:

Fx= 7 lbs (shear)
Fy=4214 lbs (tension)
Fz=1032 lbs (shear)
Mx=80108 lbs-in (bending)
My=96 lbs-in (twisting)
Mz=331 lbs-in (bending)

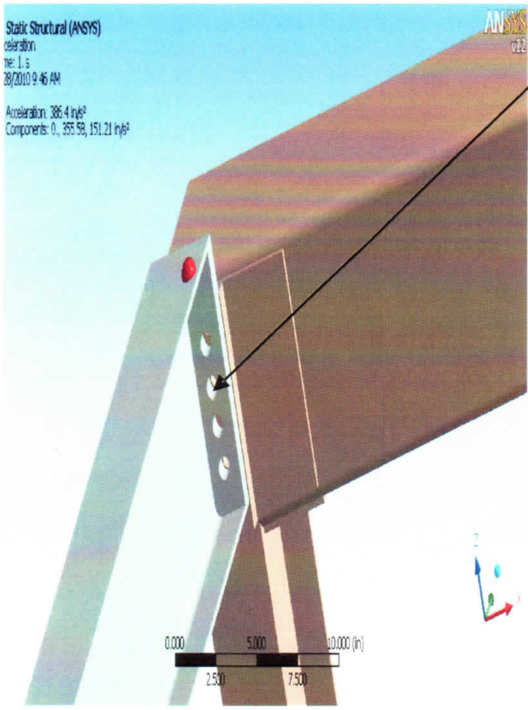
Force on weld
$\sigma sx = Fx/(7.5*2) = 0.47$ lbs/in
$\sigma t = Fy/[2*(3.83x2+7.5*2)] = 93$ lbs/in
$\sigma sz = Fz/(7.5*2*2) = 34.4$ lbs/in
$\sigma tx = Mx/Swx = (80108/2)/18.75 = 2136$ lbs/in
where $Swx = d^2/3 = 7.5^2/3 = 18.75 in^2$
$\tau = My/Jw = (96/2)/70.42 = 0.68$ lbs/in
where
$Jw = (b^3+3bd^2)/6 = (7.5^3+3*7.5*0.17^2)/6 = 0.6 in^2$
$\sigma tz = Mz/Swz = (331/2)/1.275 = 132$ lbs/in
where $Swz = bd = 0.17*7.5 = 1.275 in^2$

Resultant force on weld
$fr = Sqrt[(\sigma t + \sigma tx + \sigma tz)^2 + (\sigma sx + \tau)^2 + (\sigma sz + \tau)^2] = 2361$ lbs/in << 2.5kips/in

Allowable force on weld
$Fr = E*0.3*70 = 0.12*0.3*70 = 2.524$ kip/in
Where $E = 0.707*0.17 = 5/8x3/8 = 0.12$ in

Table 1

Case 3	Fx (lbs)	Fy (lbs)	Fz (lbs)	Mx (lbs-in)	My(lbs-in)	Mz (lbs-in)
Connection1	7	3884	1032	80108	15	105
Connection2	2	4214	445	35388	14	114
Connection3	1	3884	1011	77582	96	331



Bolts between W8x10 and (1/4 plate+1/2 plate)

5/8 "Bolt A325x6@3 places
Stress area = 0.3068 in^2
Bolt tensile stress =92/3 =30.6 ksi
Bolt shear stress =17 ksi
Allowable load in tension=30.6*0.3068=9.4kips
Allowable load in shear=17*0.3068=5.22kips

Load on joint from Table1using the worst combination in case 3:

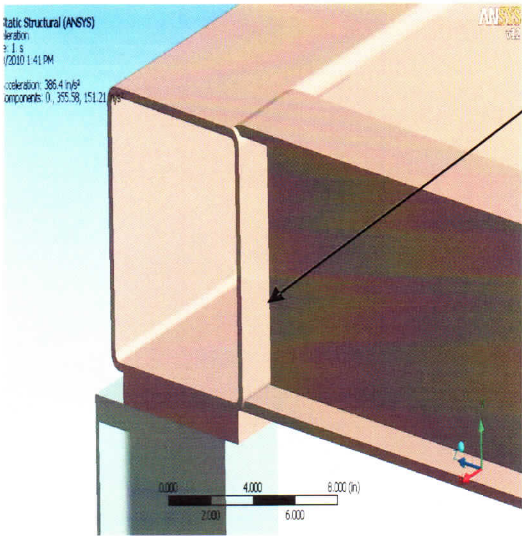
Fx= 7 lbs (shear)
Fy=4214 lbs (tension)
Fz=1032 lbs (shear)
Mx=80108 lbs-in (bending)
My=96 lbs-in (twisting)
Mz=331 lbs-in (bending)

Force on each bolt
fxs=Fx/6=1.1 lbs
ft=Fy/6=702 lbs
fzs=Fz/6=172 lbs
ft'=Mx/5.25/2=7629 lbs
fys'=My/2/2=24lbs
ft"=Mz/2 /2=83lbs

ft+ft'+ft"=702+7629+83=8.4kips <<9.4 kips

fs=sqrt [(fxs+fys')^2+fzs^2)]=0.282 kips << 5.22 kips

No Change!



Welds between 8x8x1/4 tube and w8x15 beam

1/4 inch filet welds @ 3 places (changed from old 5/16 inch to new 1/4 inch. And two new plates shall be welded to the beam)

Load on joint from Table1 using the worst combination in Case3:

Fy=4214 lbs (shear)
Fz=1032 lbs (tension)
Mx=80108+Fz*4=84236 lbs-in (bending)

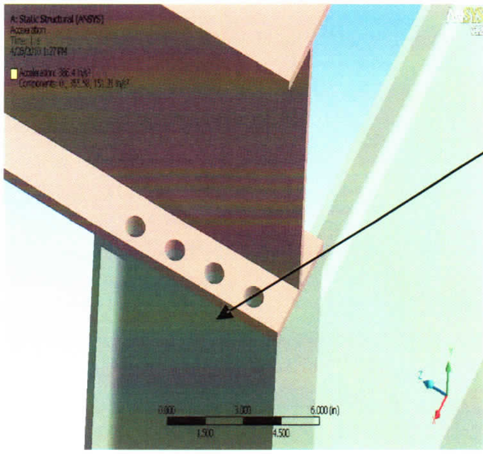
Force on weld
$\sigma sy=Fy/(7.5*2)=280$lbs/in
$\sigma t=Fz/(3.83+7.5*2)=54$lbs/in
$\sigma tx=Mx/Swx=84236/18.75=4493$ lbs/in
where Swx=d^2/3=7.5^2/3=18.75in^2

Resultant force on weld
fr=Sqrt[(σt+ σtx)^2+(σsx)^2]=4.6 kips/in >3.57 kips/in --- No Good!
Allowable force on weld
Fr=E*0.3*70=0.17*0.3*70=3.57kips/in
Where E=0.707*0.245=0.17 in

I propose to use 2 x ½ inch plates which will be welded to the inside of the W8x15 beam to increase the weld capability.

New force on weld
$\sigma sy=Fy/(7.5*4)=140$lbs/in
$\sigma t=Fz/\{[(3.83+7.5*2)]/2\}=27$lbs/in
$\sigma tx=Mx/Swx=(84236/2)/18.75=2246.5$ lbs/in
where Swx=d^2/3=7.5^2/3=18.75in^2

Resultant force on weld
fr=Sqrt[(σt+ σtx)^2+(σsx)^2]=2.3 kips/in <3.57 kips/in -- Ok!



Bolts between W8x15 beam and W8x15 beam

3/4 "Bolt A490 x 8 @3 places (changed from grade5 to grade8)
Stress area = 0.4418 in^2
Bolt tensile stress =130/3 =43.33 ksi
Bolt shear stress =21 ksi
Allowable load in tension=43.33*0.4418=19.4kips
Allowable load in shear=21*0.4418=9.28kips

Load on joint from Table1using the worst combination in case 3:

Fx= 3 lbs (shear)
Fy=1738 lbs (tension)
Fz=2057lbs (shear)
Mx=96621 lbs-in (bending)
My=270 lbs-in (twisting)
Mz=61 lbs-in (bending)

Force on each bolt
fxs=Fx/8=0.375 lbs
ft=Fy/8=217 lbs
fzs=Fz/8=257 lbs
ft'=Mx/[(1.75*2)/2]=13803 lbs
fys'=My/2/2=67.5bs
ft"=Mz/2 /2=15lbs

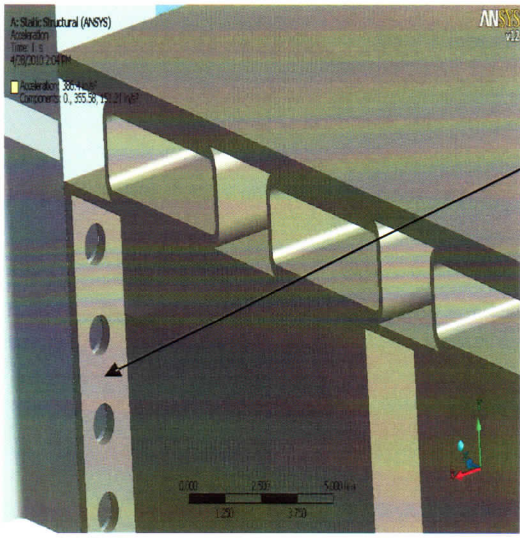
ft+ft'+ft"=217+13803+15=14.035 kips <<19.4 kips

If grade5 bolts were used, the allowable load in tension would be 30.6*0.4418=13.510kips which is less than the actual applied load of 14 kips.

fs=sqrt [(fxs+fys')^2+fzs^2)]=0.392 kips << 9.28 kips

Table 2

Case 3	Fx (lbs)	Fy (lbs)	Fz (lbs)	Mx (lbs-in)	My(lbs-in)	Mz (lbs-in)
Connection11	1	1730	1988	92800	203	47
Connection22	2	1428	2057	96621	28	8
Connection33	3	1738	2002	93532	270	61



Bolts between bottom frame and W8x15 beam

3/4 "Bolt A325 x 8 @6 places

Stress area = 0.4418 in^2
Bolt tensile stress =92/3 =30.6 ksi
Bolt shear stress =21 ksi
Allowable load in tension=30.6*0.4418=13.5kips
Allowable load in shear=21*0.4418=9.28kips

Load on joint from Table3 using the worst combination in case 3:

Fx= 118 lbs (shear)
Fy=4822 lbs (shear)
Fz=848lbs (tension)
Mx=57316 lbs-in (bending)
My=683lbs-in (bending)
Mz=4281 lbs-in (twisting)

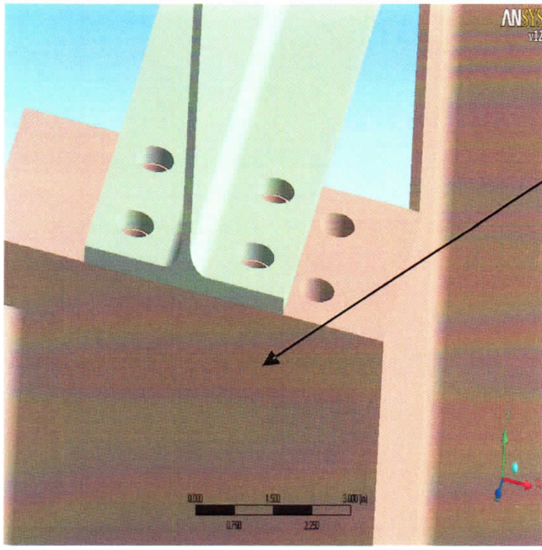
Force on each bolt
fxs=Fx/8=15 lbs
fys=Fy/8=602lbs
ft=Fz/8=106 lbs
ft'=Mx/[(1.75*2)/2] =8188lbs
ft''=My/2.25/2=152Ibs
fzs=Mz/[(1.75*2)/2]=611lbs

ft+ft'+ft"=106+8188+152=8.4 kips <<13.5 kips
fs=sqrt [(fys+fzs")^2+(fxs)^2]=1.2 kips << 9.28 kips

No Change!

Table 3

Case 3	Fx (lbs)	Fy (lbs)	Fz (lbs)	Mx (lbs-in)	My(lbs-in)	Mz (lbs-in)
Connection b1	28	1148	374	48242	457	2290
Connection b2	67	1391	532	54678	178	441
Connection b3	99	1176	353	49845	683	4281
Connection b11	118	3282	678	55588	125	2277
Connection b22	13	4822	848	57316	37	116
Connection b33	108	3256	698	56292	207	1480



Bolts between W8x10 beam and W8x10 beam

5/8 "Bolt A325 x 4@6 places

Stress area = 0.3068 in^2
Bolt tensile stress =92/3 =30.6 ksi
Bolt shear stress =17 ksi
Allowable load in tension=30.6*0.3068=9.4kips
Allowable load in shear=17*0.3068=5.22kips

Load on joint from Table 4 and Table 5 using the worst combination in case1 and Case2:

Fx= 73 lbs (shear)
Fy= 5856 lbs (tension)
Fz=59 lbs (shear)
Mx=2282 lbs-in (bending)
My=20 lbs-in (twisting)
Mz=243 lbs-in (bending)

Force on each bolt
fxs=Fx/4=18 lbs
ft=Fy/4=1464lbs
fzs=Fz/4=15 lbs
ft'=Mx/2.38/2=480 lbs
fxs'=My/2/2=5lbs
ft"=Mz/2/2=61lbs

ft+ft'+ft"=1464+480+61=2kips <<9.4 kips

fs=sqrt [(fxs+fxs')^2+fzs^2)]=0.023 kips << 5.22 kips

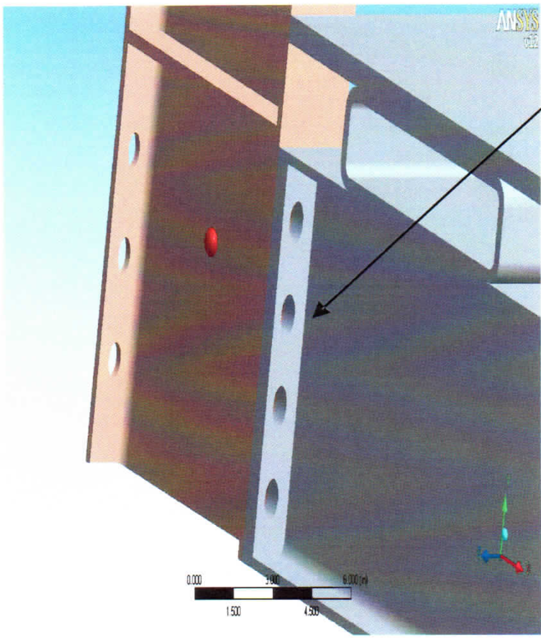
No change!

Table 4

Case 1	Fx (lbs)	Fy (lbs)	Fz (lbs)	Mx (lbs-in)	My(lbs-in)	Mz (lbs-in)
side A	44	1592	57	2280	17	244
side B	73	5856	1	36	2	202
side C	45	1583	58	2266	19	253
side 2A	46	1589	57	2282	18	251
Side 2B	72	5854	1	35	1	243
side 2C	44	1589	59	2273	20	223

Table 5

Case 2	Fx (lbs)	Fy (lbs)	Fz (lbs)	Mx (lbs-in)	My(lbs-in)	Mz (lbs-in)
side A	8	84	6	204	3	40
side B	14	13	0	7	0	41
side C	8	84	6	203	4	42
side 2A	8	81	6	197	4	42
Side 2B	14	19	0	7	0	32
side 2C	8	81	6	194	4	36



Bolts between bottom frame and W8x15 beam

3/4 "Bolt A325 x 8 @6 places

Stress area = 0.4418 in^2
Bolt tensile stress =92/3 =30.6 ksi
Bolt shear stress =21 ksi
Allowable load in tension=30.6*0.4418=13.5kips
Allowable load in shear=21*0.4418=9.28kips

Load on joint from Table6 and Table7 using the worst combination in case1 and Case2:

Fx= 200 lbs (tension)
Fy=4877 lbs (shear)
Fz=44 lbs (shear)
Mx=1235 lbs-in (twisting)
My=209 lbs-in (bending)
Mz=2203 lbs-in (bending)

Force on each bolt
ft=Fx/8=25 lbs
fys=Fy/8=609lbs
fzs=Fz/8=6 lbs
fys'=Mx/[(1.75*2)/2] =176 lbs
ft'=My/2.25/2=46 Ibs
ft"=Mz/[(1.75*2)/2]=315 lbs

ft+ft'+ft"=25+46+315=0.386 kips <<13.5 kips
fs=sqrt [(fys+fys')^2+(fzs)^2]=0.785 kips << 9.28 kips

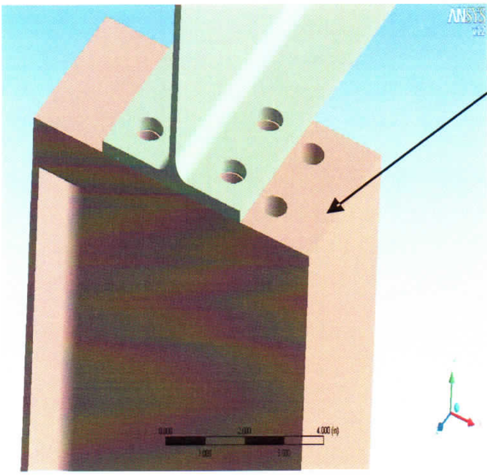
No change!

Case 1	Fx (lbs)	Fy (lbs)	Fz (lbs)	Mx (lbs-in)	My(lbs-in)	Mz (lbs-in)
floor side 2	16	1724	16	320	157	1767
floor side 22	199	4877	7	55	21	819
floor side 23	20	1732	11	600	209	2203
floor side	19	1735	8	618	170	2165
floor side 12	200	4873	5	46	15	744
floor side 13	18	1724	14	318	161	1894

Table 7

Case 2	Fx (lbs)	Fy (lbs)	Fz (lbs)	Mx (lbs-in)	My(lbs-in)	Mz (lbs-in)
floor side 2	10	357	27	468	49	246
floor side 22	9	178	12	71	40	1300
floor side 23	11	346	39	1085	7	248
floor side	11	353	44	1235	6	216
floor side 12	9	162	16	124	53	1404
floor side 13	9	367	28	432	59	186

WELD ~~ARE~~ 5/16 FILLET
AND WERE LEFT AS THEY
WERE. E. McV.
04/29/2010



Welds between 0.25 plate and w8x15 beam

3/16 inch filet welds @ 6 places (change from ¼ inch weld to 3/16 inch weld)

Load on joint from Table 4 and Table 5 using the worst combination in Case1 and Case2:

F_x= 73 lbs (shear)
F_y=5856 lbs (tension)
F_z=59 lbs (shear)
M_x=2282 lbs-in (bending)
M_y=20 lbs-in (twisting)
M_z=241 lbs-in (bending)

Force on weld
$\sigma sx=Fx/(7.5*2)=4.86 lbs/in$
$\sigma t=Fy/[(3.83*2+7.5*2)]=258 lbs/in$
$\sigma sz=Fz/(3.83*2)=7.7$ lbs/in
$\sigma tx=Mx/Swx=2282/1.275=1790 lbs/in$
where $Swx= bd=0.17*7.5=1.275 in^2$
$\tau=My/Jw=20/0.6=33$ lbs/in
where
$Jw=(b^3+3bd^2)/6=(7.5^3+3*7.5*0.17^2)/6=0.6 in^2$
$\sigma tz=Mz/Swz=241/18.75=132$ lbs/in
where Swz=d^2/3=7.5^/3=13in^2

Resultant force on weld
$fr=Sqrt[(\sigma t+\sigma tx+\sigma tz)^2+(\sigma sx+\tau)^2+(\sigma sz+\tau)^2]=2061$ lbs/in << 2.5kips/in

Allowable force on weld
$Fr=E*0.3*70=0.12*0.3*70=2.524$ kip/in
Where $E=0.707*0.17=5/8x3/8=0.12$ in