**CONTROL DEWAR AND VLPC BAYONET CAN
PLATFORM CONNECTION
DESIGN AND ANALYSIS**

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ABSTRACT

The four connections for the control dewar and VLPC bayonet can platform are designed, using finite element analysis, to carry all dead weight and live loads. Based on the loads applied to the platform, two 1 inch thick plates and two 3/4 inch thick brackets made of ASTM A572-Grade 42 are required. The 1 inch thick plate requires a 3/8 inch thick intermediate steel material, between the 8"x4"x1/4" boom and the plate, for load reinforcement as well as weld area reinforcement. Both the plates and the brackets require 3/4 inch steel bolt connections.

INTRODUCTION

The new solenoid, which just arrived from Japan, will be tested while the detector is positioned outside the collision hall to assure that the solenoid operates correctly before it is rolled back into the collision hall. In order for these tests to begin, the proper cryogenics must be made available. The components needed to operate the solenoid are a control dewar, vacuum pump and controllers. All of these components, along with a VLPC bayonet can, have been designated to sit on a platform which will be mounted onto the cryobridge. The cryobridge is essentially a rectangular column that houses cryogenic piping and runs around the perimeter of the detector. The focus of this report is the platform connections.

First, the load estimates, due to the above components, are considered. Second, the platform dimensional outline and the loads, as applied to the platform structure, are presented. Third, the reaction forces and moments, generated by a finite element analysis, are presented for all four connection points. Once these three steps are complete, the platform connection design begins.

The platform connection designs start by first explaining the initial conditions governing the design. Second, the design analysis method is presented. Third, the platform connection design is presented. And finally, the recommendations for the connections, the bolt patterns and the weld calculations are presented.

INITIAL CONDITONS

The platform design begins with a load analysis, which is based on the components mounted on the platform as seen in Appendix A¹. This section of the analysis specifies the platform dimensions and the type of structural material chosen. Page A3 is the dimensional outline of the platform structure, consisting of 2"x 4"x 3/16" and 4"x 8"x 1/4" rectangular tubing. After these types of tubing were chosen, the loads were distributed as seen on page A4. This model is then entered into the computer and a finite element analysis (FEA) is performed. A finite element analysis is an approximate method for calculating the behavior of the real structure. Points A, B, C, and D on the model are considered the connection points of the platform. The results from the FEA are shown on page A5, where the reaction forces and moments are drawn at all four connection points. In order to maintain continuity with the FEA model, the connection points will no longer be labeled A, B, C, D, but will rather be designated as nodes 1, 26, 313 and 288 due to the meshing process generated by the computer. Page A6 shows the assigned nodes in a simple wireframe sketch and presents the reaction forces and moments in table form at the top of the page.

¹ Appendix A authored by Russell Rucinski, Mechanical Engineer.

METHOD OF ANALYSIS

The analysis required for the four connections involves analyzing the forces acting on the connections as well as the moments acting on those connections. Therefore, the correct analysis must include both reactions. SDRC I-DEAS 3-D modeling has been chosen to perform the finite element analysis. However, a limitation arises in that this software does not allow for a direct application of a moment onto a solid part. Since I am modeling all four connections as solid parts, I must devise a method that allows me to completely and correctly model my connections. I experimented with numerous elements and meshing techniques in order to find the best analysis method. I also consulted with SDRC. After trying various techniques I found a method that yielded acceptable solutions. This method will be called the lumped mass model. The lumped mass model allows for a moment to be applied to solid and the creation of this lumped mass model is outlined in, "How to Create Moments on a Solid." The outline is written in SDRC I-DEAS commands and is presented on the following page.

Lumped Mass Model

The lumped mass model begins by creating a structure, which is also referred to as a solid part. The structure is then meshed, where the meshing process involves subdividing the structure into nodes and finite elements in order to perform finite element analysis. A finite element is a discrete entity used to subdivide the geometry of the structure and each element is a simple shape such as rectangle or a triangle. The number of finite elements created is determined by the shape and size of the elements. This in turn determines the number and location of the nodes. In a finite element model, nodes are the points where the elements are connected. The nodes are what is needed to continue the development of the lumped mass model.

The moment application process begins by choosing a node on the surface of the structure near the location where the moment is to be applied. The selected node is then copied at some distance away from the structure's surface. The distance chosen is irrelevant since the lumped mass model translates the forces directly to the surface and

does not require a moment arm length. This new node is designated as the lumped mass and allows for six degrees of freedom(DOF).

Constraint Elements

Now that the lumped mass is created, it must be constrained to the surface of the structure. Constraining the lumped mass to the structure allows the moment, which is applied to the lumped mass, to translate to the structure. The leading candidate for constraint is a constraint element. A constraint element connects a single node to a set of nodes and transmits all translational and rotational forces from the single node to the set of nodes chosen. Thus, the constraint element originates from the lumped mass and connects to the elements on the surface of structure, thereby translating the moment acting on the lumped mass to the elements on the surface of the structure. However, in order to transmit the moment from the elements on the surface of the structure to the elements making up the entire structure, the elements on the surface must have six degrees of freedom as well.

Thin Shell Coating

Thin shell coating has been chosen to transmit the moment from the elements on the surface of the structure to the elements making up the entire structure. The thin shell coating performed on the surface of the structure is done for two reasons. First, it is used to change the elements on the specified surface from three DOF elements to six DOF elements. This allows for the transmission of the moment from the lumped mass through the constraint elements to the elements on the surface, where the elements on the surface can now accept rotational degrees of freedom as well as translational degrees of freedom.

Second, the thin shell coating method provides for the transmission of the moment throughout the entire structure. Since the structure already consists of elements that are similar in size and shape, any force or moment applied to one element will automatically transmit that same force or moment to adjoining elements. Thus, by creating a thin shell coating on the surface of the structure, I am allowing the surface elements to receive force and moment reactions which are in turn transmitted to all the elements in the structure. However, there is one precaution that must be mentioned.

Precaution

The constraint elements, used to constrain the lumped mass to the elements on the surface of the structure, create a dissimilar mesh between. This occurs because the geometry of the elements on the surface of the structure are different from the geometry of the constraint elements. According to I-DEAS, Creating Elements with Special Techniques, the precaution for joining dissimilar meshes is that the "results for any elements near [this] mesh interface should be suspect." In order to avoid suspect results for elements near the mesh interface, these elements are not selected for display during post processing.

CANTILEVER BEAM ANALYSIS

In order to validate the lumped mass method results, I created several cantilever beam models. First I modeled a cantilever beam, as seen below in Fig. 1. The cantilever beam is 10" x 2" x 1" and is made of steel. The left end of the beam is held rigid while a 10,000 in-lb. moment is applied to the other end acting in the z-direction. The resulting bending stress should be a maximum at 15 ksi, based on static calculations, where the moment is the force multiplied by the distance and the bending stress is the moment multiplied by the distance from the neutral axis to the outer most fiber divided by the moment of inertia ($\sigma_b = \frac{Mc}{I}$). The results from the finite element analysis (FEA) are 15 ksi, which agrees with the calculated bending stress of 15 ksi.

Fig. 1. Cantilever Beam 10"x2"x1"
with 10,000 in-lb. Moment Applied

Fig. 2. Cantilever Beam 10"x2"x1"
with Extension and Lumped
Mass Applied

The second cantilever beam model, Fig. 2, begins with the same dimensions used in Fig. 1, but now there is an additional beam section which extends off the end of the cantilever beam. The additional material allows me to (1) avoid suspect results near the mesh interface, and (2) post process the original cantilever beam section. Now I can constrain the lumped mass, where the 10,000 in-lb. moment in the z-direction is applied, to the surface of the additional material. Thus, when I post process my model to determine the maximum stress, I can choose to post process only a portion of the extended

material. This allows me to avoid the suspect results completely while still incorporating the effects of the moment applied to the lumped mass.

The results from the lumped mass method as applied to the cantilever beam are shown in Table 1. The first maximum Von Mises stress for the cantilever beam, 15 ksi, is the theoretical stress which all the models should predict. I used this theoretical stress as comparison for all the cantilever beam models tested.

I post processed the second model of the cantilever beam which includes the extended beam section and the lumped mass interface. Figure 1-B in Appendix B shows that this maximum Von Mises stress is 29 ksi, a stress that is 93.33% higher than the theoretical stress. This result is precaution mentioned early stating that the "results for any elements near a mesh interface should be suspect." And, as cautioned, the high stresses occur at the mesh interface.

For the third model, I post processed only a portion of the extended beam section, and excluded the lumped mass interface as seen in Fig. 2-B. The maximum Von Mises stress dropped to 24 ksi. With this post processing method I have been able to minimize the suspect results near the mesh interface and the maximum Von Mises stress is now only 60% higher than the theoretical stress. However, the true shape of the cantilever beam must be analyzed as closely as possible. Thus I post processed a fourth model.

The fourth model eliminates the entire extended beam section on the side of the applied moment as seen in Fig. 3-B. The result is a maximum Von Mises stress of 17.6 ksi. This maximum Von Mises stress is only 17.33% higher than the theoretical maximum stress of 15 ksi.

I used a fifth model to determine whether or not the extended beam section on the opposite side of the lumped mass affects the results. Figure 4-B shows the maximum Von Mises stress for the fifth post processed model which post processes only the original cantilever beam. The stress remained the same at a maximum at 17.6 ksi.

TABLE 1. Moment Results for all Cantilever Beam Lumped Mass Models

PART	CONDITION	MAX. VON MISES STRESS (ksi)	% DIFFERENCE FROM THEORETICAL
Cantilever Beam	10,000 in-lb moment applied to end of beam	15	0
	With Lumped Mass and Extensions	29	93.33
	Without Lumped Mass	24	60.00
	Without Elements on Lumped Mass Side	17.6	17.33
	Without Lumped Mass and Extensions	17.6	17.33

CANTILEVER BEAM RESULTS

From the cantilever beam models, I found that the lumped mass model produces a stress that is conservatively higher than that of the theoretical stress value. Therefore, this method will only increase the factor of safety in my design. Thus, I will proceed with the method of adding material to the original design, then applying a lumped mass to the additional material, and finally post processing only the original shape of the platform connections.

PLATFORM CONNECTION DESIGN

The design of all four platform connections begins by following the method used for the cantilever beam. The connections are drawn first, and then the additional material is added. I followed the procedure, "How to Create Moments on a Solid," and applied the reaction moments to the lumped mass corresponding to each of the four connections. The connections at nodes 1 and 288 are the plate connections for the platform, and nodes 313 and 26 are the bracket connections for the platform.

Boundary Conditions

The boundary conditions are comprised of three parts. The first boundary condition applied to the connections is the reaction forces. The reaction forces at the connections are shown in a previous analysis by Russ Rucinski in Appendix A. Page A6 shows a summary of all the reaction forces and moments as they pertain to each node connection.

The second boundary condition specifies which surfaces will be held rigid. The rigid surfaces chosen are the surfaces of the plates and brackets which are in contact with the cryobridge. These surfaces will have no rotation or translation.

The third boundary condition is the application of the 28000 lb. minimum fastener tension which is applied to all bolt holes as pertaining to the requirements of the American Institute of Steel Construction (AISC).

Case Scenarios

The analysis for the four node connections begins with three different case scenarios for each connection. The first case scenario presents the post processing of each connection design including the extended material and the lumped mass. The

extended material is the rectangular 8" x 4" x 1/4" steel tubing, known as a boom. The addition of the boom not only provides a surface to constrain the lumped mass, but is also a true representation of the assembled platform. The second case scenario post processes the connections excluding the lumped mass and a partial section of the boom. The final case presented post processes the connection excluding the lumped mass, a partial section of the boom and the fastener tension. All three case scenarios post processed follow the same case scenarios used to test the cantilever beam discussed previously. Table 2 presents the results of these cases for all four connections and Appendix B presents the stress plots for all the models. Once a maximum Von Mises stress is found, the material selection process can begin.

TABLE 2. Comparison of Moment Results for all Lumped Mass Models

PART	CONDITION	MAX. VON MISES STRESS (ksi)
Cantilever Beam	10,000 in-lb moment applied to end of beam	15
	With Lumped Mass and Extensions	29
	Without Lumped Mass	24
	Without Elements on Lumped Mass Side	17.6
	Without Lumped Mass and Extensions	17.6
Node 288	Lumped Mass and Boom	4860
	Lumped Mass and Partial Boom Excluded	81.1
	Lumped Mass, Partial Boom, and Fastner Tension Excluded	25.7
Node 1	Lumped Mass and Boom	4220
	Lumped Mass and Partial Boom Excluded	81
	Lumped Mass, Partial Boom, and Fastner Tension Excluded	17.8
Node 313	Lumped Mass and Boom	6050
	Lumped Mass and Partial Boom Excluded	81.4
	Lumped Mass, Partial Boom, and Fastner Tension Excluded	10.9
Node 26	Lumped Mass and Partial Boom Excluded	80.7
	Lumped Mass, Partial Boom, and Fastner Tension Excluded	9.51

Node Connection 288

The plate connection at node 288 is modeled first. Fig. 5-B shows the Von Mises stress results for the first scenario; the post processing of the plate including the boom section and the lumped mass. With a maximum Von Mises stress of 4860 ksi, this model is clearly beyond the acceptable stress range since the targeted maximum stress should be less than 36 ksi. At a maximum stress less than 36 ksi I can utilize ASTM A36 steel which is abundantly available.

The second scenario results, plotted in Fig. 6-B, show a dramatic decrease in the maximum Von Mises stress. The second scenario post processes the plate excluding a portion of the boom and the lumped mass and should be in agreement with the results from the cantilever beam analysis. The stress level dropped as expected, from 4860 ksi to 81.1 ksi.. However, the high stress concentration area changed from the mesh interface between the lumped mass and the boom to the fastener tension area around the bolt holes. Since these stresses are compressive stresses and not bending stresses I can neglect their presence when determining the maximum bending stress. Also, if the plate connection was to fail at the bolts, the bolts would fail first during the pre-tension installation period. Therefore, one more scenario must be presented to better approximate the stresses acting on the plate.

The final scenario is the post processing of the plate excluding the boom section on the applied moment side, the lumped mass interface and the fastener tension applied around the bolt holes. Fig. 7-B shows that the maximum Von Mises stress is now only 25.7 ksi.

Node Connection 1

The results presented for node connection 1 are very similar to those for node connection 288 since their geometry is exactly the same. However, due to the locations of the plate connections on the platform, node 1 has a lower maximum Von Mises stress. All three scenario results for node 1 follow the same trends as the scenario results for

node 288 and the cantilever beam. The maximum Von Mises stresses are 4220, 81, and 17.8 ksi as seen in Figures 8-B, 9-B, and 10-B respectively.

In order to verify the results found for node 1, I calculated the stress in the plate and compared it to the FEA results. The stress in the plate is mostly due to bending caused by the 16,320 in-lb. moment acting in the y-direction. The calculation for the stress in the plate can be followed below.

$$\sigma_{\text{BENDING}} = \frac{Mc}{I} = \frac{(16,320 \text{ in-lb})(.5 \text{ in})}{\frac{8 \text{ in}(1 \text{ in})^3}{12}} = 12.24 \text{ ksi}$$

The maximum calculated bending stress for the 8" wide, 1" thick plate is 12.24 ksi. The FEA model result, as seen in Fig. 10-B for case scenario three, shows a maximum Von Mises stress of 17.8 ksi. Both results closely agree, however, the stress comparisons also show that the stresses produced by the FEA will be conservatively high via the lumped mass method.

Node Connection 313 and 26

Node connections 313 and 26 show similar results to those of node connections 288 and 1 in that the stresses on the bracket are best approximated when post processing the model excludes the lumped mass interface and the additional material. For the third case scenario, the bracket's maximum Von Mises stresses are near 10 ksi.

RECOMMENDATIONS

The plate and bracket connections are best approximated by the lumped mass method and the material selection is based on the FEA results presented in Table 2. The maximum Von Mises stress for the four connections, excluding the fastener tension, is 25.7 ksi for node 288. The maximum allowable stress is a combination of bending stress and tensile stress and is 0.66 of the yield strength for a given material per. AISC 1.5.1.4. ASTM A572-Grade 42 steel, or a grade of steel with a higher yield strength, is recommended. The yield stress for this steel is 42 ksi and according the AISC standard for tension and compression on extreme fibers, the maximum allowable bending stress is 27.7 ksi. Therefore, the stresses in all four connections are below the allowable when using ASTM A572-Grade 42 steel or greater.

BOLT PATTERN DESIGN

The bolt patterns are chosen in accordance with American Institute of Steel Construction standards for minimum spacing and minimum center-to-center distance for each hole. AISC specifies, for minimum spacing in 1.16.4.1, that the minimum distance between the centers of holes "shall not be less than $2\frac{2}{3}d$, where d is the nominal diameter of the fastener." In this design, the nominal diameter is .75 inches. Therefore, the minimum allowable distance is 2 inches. I chose to use 2.5 inches as my minimum distance for the design of the 1" plates(nodes 1 and 288). Also, the minimum allowable edge distance is 1-1/4" according to AISC Table 1.16.5.1. However, in my design I will use an edge distance of 1.5 inches. Both the center spacing distance and the edge distance are chosen to be larger than the allowable minimums in order to increase the reliability of the design.

After choosing the bolt spacing, I analyzed the fastener group using the elastic method.² Each fastener group is made up of 3/4" bolts, A325 Grade. Here, the allowable tension stress due to bending is 44 ksi and the allowable shear stress is 17.5 ksi. The detailed calculations in Appendix C show that the maximum tensile and shear loading will be less than the allowables. For the node 1 and 288 connection, the tension due to bending is 8.7 ksi and the shear stress is 0.213 ksi. For node 26 and 313 connection, the tension due to bending is 2.72 ksi and the shear stress is 2.0 ksi. The results of these calculations show that the fastener groups can withstand the reaction forces and moments caused by the loads acting on the platform. Therefore, the fastener groups shall be manufactured as designed in Figures 3 and 4, using 3/4" bolts, grade A325.

² AISC page 4-58 and "Steel Structures, Design and Behavior." 2nd Edition, Section 4-8, by Charles Salmon and John E. Johnson, 1980.

Figure 3. Assembly of Plate Connection at Nodes 1 and 288.

DETAIL 1

1. INTERMEDIATE WELDS 1/2" LONG FROM EDGES.

2		IMP. TEL. REG. NO.	4
1		OFFICIAL USE ONLY	5
UNIT NO.		RECEIVED ON	6
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WELD SPECIFICATIONS

Plate Connection: Nodes 1 and 288

The two 1" plates for the connections at nodes 1 and 288 are identical in size and shape. Each plate shall follow the dimensions as shown in Fig. 3. According to AISC, the criterion for a fillet weld between two materials is based on the material thickness of the thicker material. However, there appears to be no preference for welding thick material to very thin material. But, there may be limitations for a fillet weld between a 1" thick plate and the 1/4" thick boom.

Charles G. Salmon and John E. Johnson, in Steel Structures, speak of size limitations which could apply to a weld between 1" thick and 1/4" thick material. The size limitations apply to the welding process. Since the welding process produces heat energy, the heat energy is mostly absorbed by the thicker of two plates being joined. Therefore, one can see that the thicker material allows for more heat energy dissipation vertically as well as horizontally. Thus, the thicker the plate, the faster the heat energy will be removed from the welding area. This in turn produces lower temperatures at the region of the weld. Since a minimum temperature is required to provide a cohesive connection between the two plates, a weld of sufficient size is needed. In other words, the thickness of the two plates needs to be comparable in size, because, "unless a proper temperature is maintained in the area being welded a lack of fusion will result."

Due to possible limitations of a fillet weld, based on the ratio of material thickness, a solution would be to weld a 3/8" thick material to the 1/4" thick boom, and then weld the 3/8" thick material to the 1" thick plate. This approach is beneficial for two reasons. First, this approach provides a reasonable material thickness ratio and thus more adequately provides for the minimum temperature requirements for proper fusion. By welding an intermediate material thickness to the 1/4" thick boom and to the 1" thick plate, we avoid the issue of excessive heat dissipation. Secondly, this approach provides reinforcement for the 1/4" thick boom at the point where the plate is welded to the boom. The detail of the 3/8" thick reinforcement material, as welded to the boom, is seen in Figure 4.

Bracket Connection: Nodes 26 and 313

The two brackets for the connections at nodes 26 and 313 are identical in size and shape. Each bracket will be made of a 1" thick base plate with the dimensions as shown in Fig. 5. Four individual 3/4" plates or webs, with dimensions shown in Fig. 5, will be welded to the 1" thick plates as designed. The fillet welds shall be at least 5/16" according to AISC Table 1.17.2A, pertaining to the material thickness of the thicker part joined. The fillet welds lengths shall include the complete contact surface between the 3/4" webs and the 1" plate.

Figure 5. Assembly of Bracket Connection at Nodes 26 and 313

WELD RECOMMENDATIONS

The fillet welds between the 3/4" base plate of the brackets and the 3/4" webs at nodes 26 and 313, shall be at least 3/8 inches. The weld shall run around the entire connecting surfaces. The fillet welds concerning nodes 1 and 288 are as follows. The fillet weld between the 1/4" thick boom and the 3/8" thick steel material shall be a 3/16" weld. And the fillet weld between the 3/8" thick steel material and the 1" thick plate shall be a 3/8" weld.

FERMILAB
ENGINEERING NOTE

SECTION

RD/DX

PROJECT

DCRYO. PLATEFORMS

SERIAL CATEGORY

3823.118

PAGE

A1

SUBJECT

PLATFORM DESIGN CALCS.
LOAD ESTIMATE

NAME

RUSS RUCINSKI

DATE

11-12-96

REVISION DATE

LOAD ESTIMATES UPPER PLATFORM

CONTROL DEWAR:

VAC. VESSEL:

RESERV.

4.6 mm

REF. TOSHIBA DES.
OKN000608 2/3
REV. A

$$\text{VOLUME STN. STL} = 2 \times \frac{\pi (850)^2}{4} (30 \text{ mm}) + \pi (850) (1650) (5) \\ + 2 \times \frac{\pi (324)^2}{4} (25) + \pi (324) (500) (4.6)$$

$$V_{SS} = 62,540,942 \text{ mm}^3 = 3816.5 \text{ in}^3$$

$$WT_{SS} = \rho V_{SS} = .287 \frac{\text{lb}}{\text{in}^3} (3816.5 \text{ in}^3) = 1095 \text{ lbs.}$$

3 ACTUATORS, BAYONETS, U-TUBES, VAPOR COOLED CURRENT LEADS ETC.
SAY $\approx$ 600 lbs ADDITIONAL.

CONTROL DEWAR WT. = 1700 lbs.

VLPC BAYONET CAN:

VAC VESSEL =

$$\text{VOLUME} = 2 \times \frac{\pi (20)^2}{4} (.75) + \pi (20) (30) (.188) \\ = 471.2 \text{ in}^3 + 354.4 \text{ in}^3 = 825.6 \text{ in}^3$$

$$WT_{VV} = 237 \text{ lbs.}$$

ADD U-TUBES INTERNAL PIPES, VAC. PIPE BELOW - 6" SCH 5 x 10' ^{76 #5}

VLPC BAYONET CAN WT. = 500 ~~45~~

FERMILAB
ENGINEERING NOTE

SECTION
RD/DØ

PROJECT
CRYO
PLATFORM

SERIAL CATEGORY
3823.115 A2

SUBJECT

PLATFORM DESIGN CALCS.
LOAD ESTIMATE

NAME
RUSS RUCINSKI

DATE
11-20-96

REVISION DATE

DEAD LOAD OF STRUCTURE

MAIN BOOMS : $4 \times 8 \times \frac{1}{4}$: $19.02 \text{ lb/ft} \times 10.9 \text{ ft. (UPPER)} = 207$

$7.4 \text{ ft (LOWER)} = 141$

$10.0 \text{ ft (DIAGONAL)} = 190$

RAILS : $2 \times 4 \times \frac{3}{16}$: $6.87 \text{ lb/ft} \times 9.1 \text{ ft.} \times 4 \text{ (SIDES)} = 250$

$\times 3.3 \text{ ft} \times 2 \text{ (ENDS)} = 45$

$\times 12.9 \text{ ft} \times 2 \text{ (DIAGONALS)} = 177$

$\times 8.3 \text{ ft} \times 3 \text{ (VERTICALS)} = 171$

TOTAL = 1181 lbs

HANDRAILS : $2 \times 2 \times \frac{3}{8}$ ANGLES (4.7 lb/ft) PER OSHA 29 CFR 1910.23, e, i

RAILS : $9.1 \text{ ft} \times 8 \text{ SIDES} = 342$

$3.3 \text{ ft} \times 8 \text{ ENDS} = 124$

$3.5 \text{ ft} \times 6 \text{ VERTICALS, UPPER PLATFORM ONLY} = 99$

TOTAL = 565 lbs

DECK PLATE : $\frac{1}{4}$ " THICK $\times 40$ " $\times 70$ " UPPER

40 " $\times 100$ " LOWER TOTAL = 482 lbs

TURBO PUMPS & CONTROLLERS : $100 \text{ lbs} \times 2 \text{ UNITS} = 200 \text{ lbs}$

CHIMNEY HANGERS : 18 ' CHIMNEY $\times 20 \text{ lbs/ft} = 360 \text{ lbs}$

LIVE LOAD = SAY 2 PEOPLE $\times 200 \text{ lbs EACH PLATFORM} = 800 \text{ lbs}$

TOTAL LOADS : UPPER : $1700 + 500 + \frac{1181}{2} + \frac{565}{2} + 241 + 360 + 400 = 4074$

LOWER : $\frac{1181}{2} + \frac{565}{2} + 241 + 200 + 400 = 1714$

TOTAL = 5788 lbs

FERMILAB
ENGINEERING NOTE

SECTION
RD/100

PROJECT
CRYO
PLATFORMS

SERIAL CATEGORY
3823.115

PAGE
A4

SUBJECT

PLATFORM DESIGN CALCS.

DIMENSIONAL OUTLINE & LOAD APPLIC.

NAME

RUSS RUCINSKI

DATE

11-20-96

REVISION DATE

LOADS APPLIED TO BASE:

LOADS ARE PLACED AT THE APPROPRIATE LOCATIONS
ON THE STRUCTURE.

FREE BODY DIAGRAM OF PLATFORMS

APPLIED LOADS & REACTIONS

Group ID : None
Result Set : 2 - B.C. 1, LOAD 1, REACTION FORCE_2
Report Type : Contour Units : IN
Result Type : REACTION FORCE
Frame of Reference: Part Data Component: Norm to Screen

Node	Reacti-X	Reacti-Y	Reacti-Z	Reacti-RX	Reacti-RY	Reacti-RZ
1	0.000E+00	-5.655E+02	2.844E+03	0.000E+00	-1.632E+04	-3.415E+03
26	-2.518E+03	1.478E+03	-4.847E+03	-1.706E+04	0.000E+00	0.000E+00
288	1.451E+03	1.286E+03	-2.831E+03	4.784E+02	1.600E+04	7.227E+03
313	1.067E+03	3.372E+03	4.835E+03	-1.966E+04	0.000E+00	0.000E+00
Total	0.000E+00	5.570E+03	-4.883E-04	-3.624E+04	-3.194E+02	3.811E+03
Maximum	288 1.451E+03	313 3.372E+03	313 4.835E+03	288 4.784E+02	288 1.600E+04	288 7.227E+03
Minimum	26 -2.518E+03	1 -5.655E+02	26 -4.847E+03	313 -1.966E+04	1 -1.632E+04	1 -3.415E+03
Average	0.000E+00	9.237E+00	-8.098E-07	-6.010E+01	-5.298E-01	6.320E+00

RESTRAINT SPECIFICATIONS

NODES 1 & 288 ; X,Y,Z TRANSLATIONS & ROTATIONS FIXED

NODES 26 & 313 ; X,Y,Z TRANSLATIONS FIXED

X ROTATIONS FIXED

Y,Z, ROTATIONS FREE

RESTRAINT SPECIFICATIONS BASED ON 4 BOLT CONNECTION
AT NODES 1 & 288, 2 BOLTS ALONG Y AXIS @ 26 & 313.

Figure 1-B. CANTILEVER BEAM: STRESSES WITH LUMPED MASS AND EXTENSIONS ($M_2 = 10,000 \text{ in-lb}$)

Figure 2-B. CANTILEVER BEAM: STRESSES WITHOUT LUMPED MASS & PARTIAL BOOM EXCLUDED

Figure 3-B. CANTILEVER BEAM: STRESSES WITHOUT ENTIRE VOLUME ON LUMPED MASS SIDE

Figure 4-B. CANTILEVER BEAM: STRESSES WITHOUT LUMPED MASS & EXTENSIONS

Figure 5-B. NODE CONNECTION 288: LUMPED MASS & BOOM

/dks4d3s7/ms_rucinski/akuwazaki2.mfl

RESULTS: 2- B.C. 1, LOAD 1, STRESS_2

STRESS - VON MISES MIN: 1.89E+01 MAX: 4.86E+06

FRAME OF REF: PART

VALUE OPTION: ACTUAL
SHELL SURFACE: TOP

Figure 6-B. NODE CONNECTION 288: LUMPED MASS & PARTIAL BOOM EXCLUDED

/dks4d3s7/ms_rucinski/akuwazaki2.mfl

RESULTS: 2- B.C. 1, LOAD 1, STRESS_2

STRESS - VON MISES MIN: 1.89E+01 MAX: 8.11E+04

FRAME OF REF: PART

VALUE OPTION: ACTUAL

8.11E+04

7.30E+04

6.49E+04

5.68E+04

4.87E+04

4.05E+04

3.24E+04

2.43E+04

1.62E+04

8.12E+03

1.89E+01

Figure 7-B NODE CONNECTION 288: LUMPED MASS, PARTIAL BOOM & FASTENER TENSION EXCLUDED

Figure 8-B. NODE CONNECTION 1: LUMPED MASS & BOOM

Figure 9-B. NODE CONNECTION 1: LUMPED MASS & PARTIAL BOOM EXCLUDED

Figure 10-B. NODE CONNECTION 1: LUMPED MASS, PARTIAL BOOM & FASTENER TENSION EXCLUDED

Figure 11-B. NODE CONNECTION 313: STRESSES WITH LUMPED MASS & BOOM

Figure 12-B. NODE CONNECTION 313: STRESS WITH LUMPED MASS & PARTIAL BOOM EXCLUDED

/dks4d3s7/ms_rucinski/akuwazaki2.mf1

RESULTS: 2- B.C. 1, LOAD 1, STRESS_2

STRESS - VON MISES MIN: 6.87E+01 MAX: 8.14E+04

DEFORMATION: 1- B.C. 1, LOAD 1, DISPLACEMENT_1

DISPLACEMENT - MAG MIN: 0.00E+00 MAX: 8.81E-03

FRAME OF REF: PART

VALUE OPTION: ACTUAL

Figure 13-B. NODE CONNECTION 313: STRESSES WITH FASTENER TENSION, LUMPED MASS, & PARTIAL BOOM EXCLUDED

Figure 14-B. NODE CONNECTION 26: STRESSES WITH LUMPED MASS & PARTIAL BOOM EXCLUDED

Figure 15-B. NODE CONNECTION 26: STRESSES WITH FASTENER TENSION, LUMPED MASS & PARTIAL BOOM EXCLUDED

USE 3/4" BOLTS, A325 GRADE

$$A_b = .4418 \text{ in}^2$$

FROM AISC, PART 4, TABLE 1-A:

$$F_t = 44 \text{ KSI}$$

$$\text{ALLOWABLE LOAD} = 194 \text{ KSI (TENSION)}$$

TABLE 1-C:

$$F_{ys} = 92 \text{ KSI}$$

$$F_{us} = 120 \text{ KSI}$$

$$\text{PROOF LOAD} = 85 \text{ KSI}$$

TABLE 1-D:

$$F_v = 17.5 \text{ KSI (SHEAR)}$$

$$\text{ALLOW LOAD} = 7.7 \text{ KIPS}$$

NODE 1

PER AISC TABLE 1.6.3 COMBINED TENSION & SHEAR

$$F_t = 55 - 1.8 f_v \leq 44 \text{ KSI} \quad \left\{ \begin{array}{l} \text{ALLOW. TENSION STRESS} \\ \text{IN BEARING-TYPE CONNECTION} \end{array} \right\}$$

REACTION AT 4 BOLT PATTERN

ANALYSING THE FASTENER GROUP BY THE ELASTIC METHOD
 (REF. AISC DG. 4-58 AND STEEL STRUCTURES DESIGN & BEHAVIOR,
 2ND EDITION, SECTION 4-8, BY CHARLES SALMIN & JOHN E.
 JOHNSON, 1980)

$$d_1 = 2.75 \quad d_2 = 3.72$$

REACTION R_y (DIRECT SHEAR):

$$f_1 = \frac{R_y}{6} = \frac{565.5 \text{ lb}}{6} = \underline{94.25 \text{ lb}} ; f_{\text{shear}} = \frac{94.25 \text{ lb}}{.4418 \text{ in}^2} = \underline{\underline{213 \text{ PSI}}}$$

TENSION FROM BENDING CONSIDERING INITIAL TENSION (AISI "STEEL...")

INITIAL BEARING PRESSURE:

$$f_{bi} = \frac{\sum T_i}{bd} ; T_i = 28000 \text{ lb (AISC TABLE 1.23.5)}$$

PRETENSION LOAD

 $b \equiv \text{PLATE WIDTH} = 8 \text{ IN}$ $d \equiv \text{PLATE LENGTH} = 8 \text{ IN}$

$$f_{bi} = \frac{28000 \text{ lb (6 bolts)}}{(8 \text{ IN})(8 \text{ IN})} = 2.625 \text{ ksi}$$

TENSILE STRESS

$$f_{eb} = \frac{6M}{bd^2} = \frac{6(16320 \text{ in-lb})}{(8 \text{ IN})(8 \text{ IN})^2} = 192 \text{ PSI}$$

f_{eb} SHOULD NOT EXCEED f_{bi} IF COMPRESSION BETWEEN THE
PIECES IS TO REMAIN AT THE TOP

$$f_{eb} = 192 \text{ PSI} < f_{bi} = 2625 \text{ PSI}$$

$$T_{\text{NET}} = f_{eb} b p = (192 \text{ PSI})(8 \text{ IN})(2\frac{1}{2} \text{ IN}) = 3840 \text{ lb}$$

$$f_t = \frac{T_{\text{NET}}}{A_b} = \frac{3840 \text{ lb}}{.4418 \text{ in}^2} = \underline{\underline{8.692 \text{ ksi}}} \left\{ \begin{array}{l} \text{NOMINAL TENSILE} \\ \text{STRESS IN TOP BOLT} \end{array} \right\}$$

SUMMARY

TENSION DUE TO BENDING:

$$f_t = 8.7 \text{ ksi} < F_t = 44 \text{ ksi} \quad \text{OK}$$

SHEAR:

$$f_v = .213 \text{ ksi} << F_v = 17.5 \text{ ksi} \quad \text{OK}$$

REACTION AT & BOLT PATTERN: ELASTIC METHOD (FASTENER GROUP ANALYSIS):

$$d_1 = 2.75''$$

$$d_2 = 3.72''$$

PROPORTIONAL SHARE OF M_x :

$$\frac{M_x}{6 \text{ fasteners}} = \frac{478.4 \text{ in-lb}}{6} = 80 \text{ in-lb} = M_x'$$

EVALUATE f_1 :

$$4 f_1 (d_2) = M_x'$$

$$f_1 = \frac{M_x'}{4 d_2} = \frac{80 \text{ in-lb}}{4 (3.72 \text{ in})} = 5.4 \text{ lb}$$

$$f_{1y} = f_1 \sin 45 = 3.8 = f_{1x}$$

EVALUATE f_3 :

$$2 f_3 (d_1) = M_x'$$

$$f_3 = \frac{M_x'}{2 d_1} = \frac{80 \text{ in-lb}}{2 (2.75 \text{ in})} = 14.5 \text{ lb}$$

REACTION R_y :

$$F_2 = \frac{R_y}{6} = \frac{1286 \text{ lb}}{6} = 214 \text{ lb}$$

MAX. SHEAR LOAD PER FASTENER:

$$f_{\text{SHEAR}} = \sqrt{(f_{1x})^2 + (f_{1y} + f_2)^2} = 217 \text{ lb}$$

OR

$$f_{\text{SHEAR}} = \sqrt{(f_2 + f_3)^2} = \underline{229 \text{ lb}}$$

TENSION FROM BENDING MOMENT (INITIAL TENSION)

$$f_{bi} = \frac{Z T_i}{bd} = 2625 \text{ psi}$$

TENSILE STRESS

$$f_{tb} = \frac{6 M}{bd^2} = \frac{6(7227 \text{ in-lb})}{(8 \text{ in} \times 8 \text{ in})^2} = 85 \text{ psi}$$

NO FURTHER CALCS ARE NEEDED. NODE 288 CONNECTION STRESSES ARE LOWER THAN NODE 1 CONNECTIONS. THEREFORE NODE 288 CONNECTIONS ARE GOOD.

ALL FASTENERS NEED TO WITHSTAND $R_{Y26} = 1478 \text{ lbs.}$

$$\frac{R_{Y26}}{6} = \frac{1478 \text{ lb}}{6} = .25 \text{ KIPS/BOLT}$$

ALLOWABLE LOAD FROM TABLE 1-D: 7.7 KIPS.

- SINCE THE CALCULATED LOAD IS LESS THAN THE ALLOWABLE, $3/4"$ BOLTS ARE OK.
- THE NODE 313 CONNECTION REACTIONS DOMINATE OVER THE NODE 26 REACTIONS.

NODE 313

TENSION FROM BENDING MOMENT CONSIDERING INITIAL TENSION

$$f_{bi} = \frac{\sum T_i}{bd} = \frac{28000 \text{ lb}(4)}{(3.95 \text{ in})(14 \text{ in})} = 2,025 \text{ psi}$$

TENSILE STRESS DUE TO MOMENT

$$f_{tb} = \frac{6M}{bd^2} = \frac{6(19,660 \text{ in.-lbs})}{(3.95 \text{ in})(14 \text{ in})^2} = 152 \text{ psi}$$

$$f_{tb} < f_{bi} \rightarrow 152 \text{ psi} < 2,025 \text{ psi} \quad \text{OK}$$

$$T_{NET} = f_{tb} b p = (152 \text{ psi})(3.95 \text{ in})(2 \text{ in}) = 1201 \text{ lb}$$

$$f_t = \frac{T_{NET}}{A_b} = \frac{1201 \text{ lb}}{.4418 \text{ in}^2} = \underline{\underline{2.72 \text{ KSI}}}$$

SHEAR COMPONENT (NODE 313):

$$f_1 = \frac{R_x}{4} = \frac{1067 \text{ lb}}{4} = 267 \text{ lb}$$

$$f_2 = \frac{R_y}{4} = \frac{3372 \text{ lb}}{4} = 843 \text{ lb}$$

$$V = \frac{\sqrt{(1067)^2 + (3372)^2}}{4} = 884 \text{ lb/BOLT}$$

$$f_v = \frac{884 \text{ lb}}{.4418 \text{ in}^2} = 2001 \text{ psi}$$

SUMMARY

TENSION DUE TO BENDING:

$$f_t = 2.72 \text{ ksi} < F_t = 44 \text{ ksi} \quad \text{OK}$$

SHEAR:

$$f_v = 2.0 \text{ ksi} < F_v = 17.5 \text{ ksi} \quad \text{OK}$$

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1. American Institute of Steel Construction, Steel Construction Manual, American Institute of Steel Construction, Illinois, 1980.
2. Salmon, G. Charles and Johnson, E. John, Steel Structures: Design and Behavior, 2nd Edition. Harper and Row, Publishers, New York, 1980.