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**Westinghouse Electric Corporation
Fusion Power Systems Department**

**STRUCTURAL ANALYSIS
AND
OPTIMIZATION PROCEDURE OF THE
TFTR DEVICE SUBSTRUCTURE**

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OF THE TFTR DEVICE SUBSTRUCTURE

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ABSTRACT

A structural evaluation of the TFTR device substructure is performed in order to verify the feasibility of the proposed design concept as well as to establish a design optimization procedure for minimizing the material and fabrication cost of the substructure members. A preliminary evaluation of the seismic capability is also presented. The design concept on which the analysis is based is consistent with that described in the Conceptual Design Status Briefing report dated June 18, 1975.

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1.0 INTRODUCTION

This study has been conducted during the conceptual design phase of the TFTR Project. The TFTR device substructure design configuration has been in a continual stage of development and it is for this reason that the analytical models were defined in a conservative fundamental form and the mathematics kept tractable. The validity and relevance of the design concept feasibility verification and especially the design optimization procedure have been therefore established for a wide range of design configurations. The design concept on which the included sample calculations are based is that described in the Conceptual Design Status Briefing report dated June 18, 1975.

The TFTR Device substructure serves as the support structure for the vacuum vessel, toroidal field coils, neutral beam injectors, vacuum pumps, and much of the device shielding. A sketch of a substructure concept is shown in Figure 1, and a discussion of its performance requirements can be found in reference 1. The substructure itself is composed of a glass fiber reinforced epoxy sheet that is directly supported by transverse I-beam joists which in turn are supported by I-beam girders resting on roller bogies. All I-beams are made of ARMCO 18-3Mn or equivalent.

2.0 SUMMARY

An outcome of this investigation is that the design concept feasibility has been verified and a design optimization procedure has been outlined for future reference. This procedure will be of help as the need for further design changes becomes evident. Based on the evaluated design concept, an allowable combined Vacuum Pump and Neutral Beam Injector load was calculated for the worst case support bed locations.

Since a seismic qualification of the substructure is also required, a conservative natural frequency calculation is made in this report as well as a discussion of the effects of a potential seismic condition. The seismic specification of reference 2 (Richter Magnitude 5) is a mild condition, but definitive structural effects cannot be evaluated analytically until a floor response spectrum has been generated for the TFTR Device location in the test cell building.

3.0 METHOD OF ANALYSIS

In order to minimize the fabrication cost of the substructure, it has been decided to minimize the amount of required welding by eliminating the need for any stiffener webs in the I-beams. This then was a factor to be considered in the design optimization process. This process consisted of selecting the most efficient I-beam sizes based on minimizing the amount of Armco 18-3Mn material to be purchased while assuring adequate structural bending and shear resistant properties of the beams. The design procedure used conforms to that of the AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings (see references 3, 4, 5, and 6).

A number of criteria for welded plate girders were adhered to in the design optimization process. In the evaluation of these criteria several mechanical properties were used which for Armco 18-3Mn (reference 7) include

$$\begin{aligned}\sigma_y &= \text{yield strength (annealed @ 0.2\% offset)} \\ &= 68,000 \text{ psi}\end{aligned}$$

$$E = \text{Young's Modulus} = 28.8 \times 10^6 \text{ psi}$$

Utilizing this material property data, the design criteria become:

- (1) To avoid reaching critical shear stress levels in the web it is suggested that the depth to span ratio of a joist or girder be about [3]*

$$\frac{d}{L} \leq \frac{\sigma_y}{800,000} \leq \frac{1}{11.8}$$

Deviations from this value are permitted as long as beam shear stress levels are checked and found acceptable.

* Numbers in brackets denote references.

- (2) The maximum width to thickness ratio of a compression element (flange) should be [4]

$$\frac{b_f}{t_f} = \frac{6000}{\sqrt{\sigma_y}} = 23.0$$

- (3) The maximum width to thickness ratio of a web, necessary for no reduction in allowable (compressive) bending stress due to possible lateral displacement of the web, should be [3]

$$\frac{d_w}{t_w} = \frac{24,000}{\sqrt{S_b}} = 119$$

- (4) Deflections in support beams should be held to $(1/2)(1/360)$ of the span length where machinery is to be supported [5].
- (5) The allowable beam tensile bending and shear stresses as defined by AISC are [3]

$$S_b = 0.60\sigma_y = (0.6)(68,000) = 40,800 \text{ psi bending}$$

$$T_a = 0.40\sigma_y = (0.4)(68,000) = 27,200 \text{ psi shear}$$

- (6) The allowable compression flange stress is the larger of (but not to exceed $0.60\sigma_y$) [3,4]

$$S_b = \left[1.0 - \frac{(L/r)^2}{2C_c^2 C_b} \right] .66\sigma_y$$

$$\text{and } S_b = \frac{12 \times 10^6}{Ld/A_f}$$

$$\text{where } C_c = \sqrt{\frac{2\pi^2 E}{\sigma_y}}$$

$$C_b = 1.0$$

L = span length

d = beam depth

r = radius of gyration

A_f = flange cross
sectional area

- (7) The compressive stress at the web toe of the beam fillet welds due to concentrated loads or reactions shall not exceed $0.75\sigma_y$. The stress levels are computed as follows [4]:

$$S_R = \frac{R}{t_w(N+k)} \quad \text{for reactions}$$

$$S_L = \frac{R}{t_w(N+2k)} \quad \text{for concentrated loads}$$

where R = concentrated load or reaction

t_w = web thickness

N = length of bearing for load or reaction (not less than k)

k = flange thickness plus leg size of fillet weld.

4.0 DISCUSSION OF ANALYSIS

The design optimization procedure can most meaningfully be described by presenting an evaluation of the optimum girder and joist sizes based on supported equipment dead weight considerations. The stiffness of the fiber reinforced epoxy sheet in the substructure was felt to be small in comparison to the joist and girder stiffness and was therefore neglected. The design parameters used in the analysis originate from the currently available TFTR device substructure conceptual design data [1]. Sections 4.1 to 4.3 of this report summarize the equipment weight, TF coil support ring, and roller bogie evaluations. Sections 5 and 6 document the optimization procedure for joist and girder selection. Section 7 briefly discusses Vacuum Pump and Neutral Beam Injector support integrity. Section 8 contains a discussion of the potential effects of a seismic event.

4.1 DESIGN LOADS

A layout drawing of the substructure is shown in Figure 2. The weight of the vessel, TF coils, and some shielding is applied to the support bed through the support

ring. The postulated worst case loading condition is the equivalent weight of two TF coils, one tenth of the vessel, and the applicable shielding all being supported by a single I-beam. This assumption applies in both the girder and joist evaluations because of the manner in which the dead weight loads are transmitted to the support bed (through the support ring). This can be seen through an inspection of Figure 2.

This load was computed to be

$$W = 1/10 \left[\begin{array}{l} \text{vessel wt.} + \text{TF coil wt.} + \text{internal shielding wt.} + \text{external} \\ \text{shielding wt.} \end{array} \right]$$

$$W = 1/10 [66,700 + 802,000 + 234,000 + 93,600] = 120,000 \text{ lbs.}$$

4.2 SUPPORT RING

The adequacy of the support ring can be demonstrated through a stress calculation.

$$\text{Ring stress} = \frac{(120,000 \text{ lbs.})}{(1/10)(\pi)(198'' + 14'')(2'')} \\ = 901 \text{ psi}$$

4.3 ROLLER BOGIES

The required roller supports for the substructure were selected as stock items from reference 8. Assuming, conservatively, that the support bed itself weighs 1.2×10^5 lbs and the device weight is 1.2×10^6 lbs, the maximum roller load would be

$$Q(U) = \frac{1.2 \times 10^5 \text{ lbs.}}{20 \text{ rollers}} = 6,000 \text{ lbs./roller (uniform load)}$$

$$Q(E) = \frac{1.2 \times 10^6 \text{ lbs.}}{28 \text{ rollers}} = 42,800 \text{ lbs./roller (equipment load)}$$

$$Q_{\max} = Q(U) + Q(E) = 48,800 \text{ lbs./roller}$$

Reference 8 indicates that 4" single and double rollers would be adequate for this application in that the rolling design life distance is about 250 kilometers under the load $Q_{\max}$.

5.0 GIRDER SELECTION

The design optimization procedure is discussed in this section through an evaluation of the most efficient girder design based on equipment dead weight considerations.

5.1 SIZING

To assure conservative results, the girder span was assumed to have simple supports. The maximum bending moment in any girder was then computed to be

$$M_{\max} = \frac{WL}{4} = \frac{(120,000)(30.2)}{4}$$

$$= 906,000 \text{ in-lbs.}$$

The required section modulus for a girder is

$$S(\text{req'd}) = \frac{M_{\max}}{S_b} = \frac{9.06 \times 10^5}{4.08 \times 10^4}$$

$$= 22.2 \text{ in}^3$$

Reference 3 has indicated that in order to eliminate the need for I-beam stiffening webs, the standard method in bridge design (American Association of State Highway Officials) is to limit the depth to thickness ratio of the web to less than some peak value. This peak value is based on the material yield strength data as shown in Figure 3. For Armco 18-3Mn it would appear to be

$$K = \frac{d_w}{t_w} \approx 40 \quad (\text{Armco 18-3Mn})$$

A plot of the girder weight in lbs/ft vs. web depth indicates the most efficient web depth to produce a given section modulus. This plot is shown in Figure 4 for different values of K. It is interesting to note from the figure that lower beam weights can be obtained if larger values of K would be chosen with the effect of requiring web transverse stiffeners. The optimum web depth can be found from the following formula as well as from Figure 4:

$$d_w = \sqrt[3]{\frac{3KS}{2}} \quad [\text{reference 3}]$$

$$= \sqrt[3]{\frac{(3)(40)(22.2)}{2}}$$

$$d_w = 11.0 \text{ in.} \quad \text{web depth}$$

The web thickness was computed to be

$$t_w = \frac{d_w}{K} = \frac{11.0}{40} = .275 \text{ in.} \quad \text{web thickness}$$

Assuming all of the shear resistance is in the beam web, then the required web area is

$$A_w (\text{req'd}) = \frac{V}{\tau_a} = \frac{120,000}{(2)(27,200)} = 2.20 \text{ in.}^2$$

A computation for the existing web area leads to

$$A_w = t_w d_w = 3.025 \text{ in.}^2$$

Therefore,

$$A_w > A_w (\text{req'd})$$

which indicates that the girder beam will be adequate in shear resistance capabilities.

Reference (3) has indicated that a characteristic of the efficient I-beam is that the cross sectional area of either flange will be equal to one half of the web cross sectional area. Thus

$$A_f(\text{reg'd}) = \frac{A_w}{2} = \frac{3.025}{2} = 1.512 \text{ in}^2$$

$$\text{where } A_f = b_f t_f$$

If we arbitrarily select a flange thickness of 0.50 in., then

$$t_f = 0.50 \text{ in.}$$

and

$$I_t = S(\text{reg'd}) c$$

$$= (22.2)(6.0)$$

$$\text{where } c = 1/2 (d_w + 2t_f) = 6.0 \text{ in.}$$

$$I_t = 133 \text{ in}^4$$

$$I_w = 1/12 t_w d_w^3 = (1/12)(.275)(11.0)^3 = 30.5 \text{ in}^4$$

$$I_f = 2b_f t_f c_f^2$$

$$\text{where } c_f = 1/2(d_w + t_f) = 5.75 \text{ in.}$$

$$I_f = I_t - I_w = 102 \text{ in}^4$$

$$b_f = \frac{I_f}{2t_f c_f^2}$$

$$b_f = \frac{102}{(2)(.50)(5.75)^2} = 3.08 \text{ in} \approx 3.1"$$

$$A_f = b_f t_f = (3.1)(0.5) = 1.55 \text{ in}^2$$

Thus, the available A_f is approximately equal to that of the efficient beam [$A_f(\text{reg'd})$].

The section modulus for this beam was then computed to be

$$S = \frac{I_t}{c}$$

$$= \frac{I_f + I_w}{c}$$

$$= \frac{2b_f t_f (c_f)^2 + 1/12 t_w d_w^3}{c}$$

$$= \frac{(2)(3.10)(.50)(5.75)^2 + (1/12)(.275)(11.0)^3}{6.0}$$

$$S = 22.2 \text{ in}^3 = S(\text{reg'd})$$

The ratio of flange width to thickness is

$$\frac{b_f}{t_f} = \frac{3.10}{0.50} = 6.2$$

which is within the safe limits designated by design criterion #2.

According to design criterion #6, the allowable compression flange stress should be checked to verify the validity of assuming $0.6\sigma_y$ as the allowable compressive stress. The larger value (not to exceed $0.6\sigma_y$) of the two computations below is the approved allowable stress.

$$S_b = \left[1.0 - \frac{(L/r)^2}{2C_c^2 C_b} \right] [0.66\sigma_y] = 0.62\sigma_y$$

$$\text{and } S_b = \frac{12 \times 10^6}{Ld/A_f} = 51,300 = 0.75\sigma_y$$

$$\text{where } C_c = \sqrt{\frac{2\pi^2 E}{\sigma_y}} = 91.4$$

$$C_b = 1.75$$

$$L = 30.2 \text{ in.}$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{(1/12)(0.5)(3.1)^3 + (1/12)(1.83)(.275)^3}{(.5)(3.1) + (1.83)(.275)}}$$

$$r = .778 \text{ in.}$$

$$A_f = b_f t_f = 1.55 \text{ in}^2$$

$$d = 2c = 12.0 \text{ in.} \quad d_w/6 = 1.83''$$

$$\sigma_y = 68,000 \text{ psi}$$

$$E = 28.8 \times 10^6 \text{ psi}$$

The assumed maximum value of $0.6\sigma_y$ for an allowable compressive flange stress has therefore been justified.

5.2 GIRDER DEFLECTION

The deflection of the girder span was computed to be

$$\delta_{\max} = \frac{WL^3}{48EI} = \frac{(120,000)(30.2)^3}{(48)(28.8 \times 10^6)(133)}$$
$$\delta_{\max} = 0.0180 \text{ in.}$$

Design criterion #4 suggests that deflections be limited to the following level:

$$\delta_{\text{allowable}} = (1/2)(1/360)(30.2) = 0.0419 \text{ in.} > \delta_{\max}$$

Thus, the expected girder deflection is within an acceptable level.

5.3 GIRDER FILLET WELD SPECIFICATION

In the interests of minimizing design fabrication costs it is desirable to minimize the size of the fillet welds joining the web and flanges. This will also serve to limit distortion during the welding process. Intermittent fillet welds may be used if desired, but continuous welds should be specified in the areas of the applied loads and beam supports. If intermittent fillet welds are used, the maximum weld spacing should be

$$D_{\max} (T) = \text{gap between welds} = 24 t_w$$
$$= (24)(.275)$$
$$= 6.60 \text{ in. at tension flange}$$

$$D_{\max} (C) = \frac{4000 t_w}{\sqrt{f_y}}$$
$$= \frac{(4000)(.275)}{\sqrt{68,000}}$$

$$D_{\max} (C) = 4.22 \text{ in. at compression flange}$$

The required weld size depends on the weld load.

$$f = \frac{VA_f c_f}{I_t n}$$
$$= \frac{(60,000)(1.55)(5.75)}{(133)(2)}$$

$f = 2010 \text{ lbs/in}$ for each weld

where $V = \text{shear load} = 120,000/2 = 60,000 \text{ lbs.}$

$n = \text{number of fillet welds holding flange area}$

The required fillet weld leg size was computed to be

$$\text{req'd leg} = \frac{f}{.707T_a} = \frac{2010}{(.707)(27,200)} = 0.104 \text{ in.}$$

assuming weld material has an allowable shear strength of 27,200 psi.

Therefore, the weld size selected should be 3/16" fillet welds, continuous or

$$\frac{.104}{.188} = 55.3\%$$

intermittent

staggered

based on good welded joint design practice.

5.4 LOAD BEARING AND REACTION BEARING SUPPORT ADEQUACY

Concentrated loads and beam end reactions can cause high compressive stress to occur at the web toe of the fillet in the area of the loads. Since no transverse stiffeners are to be placed in these regions, the following stress levels should not exceed $0.75\sigma_y$ (criterion #7):

$$S_R = \frac{R_R}{t_w(N_R + k)} = 28,400 \text{ psi} \quad \text{for reaction area}$$

$$S_L = \frac{R_L}{t_w(N_L + 2k)} = 28,400 \text{ psi} \quad \text{for loaded area}$$

where $R_R = 60,000 \text{ lbs}$ reaction

$R_L = 120,000 \text{ lbs}$ load

$N_R = 7''$ (reference 8)

$N_L = 14''$ (support ring width)

$k = \text{weld leg} + t_f = 3/16 + .50 = 0.69''$

Both of these stresses are less than $0.75\sigma_y$ (51,000 psi), therefore transverse stiffeners are not needed in the beam support and load bearing regions.

5.5 OPTIMUM GIRDER DESIGN

Having satisfied the beam design criteria, the optimum girder design based on dead weight considerations is as follows:

assuming weld material
allowable shear strength
equals 27,200 psi

where optimum girder design weight = 20.6 lb/ft (see Figure 4)

6.0 JOIST SELECTION

The optimization method is summarized by performing the same analysis in the joist selection.

6.1 SIZING

$$M_{\max} = \frac{(120,000)(96)}{4} = 2.88 \times 10^6 \text{ in-lbs}$$

$$S(\text{req'd}) = \frac{2.88 \times 10^6}{40,800} = 70.6 \text{ in}^3$$

Since $K = 40$,

$$d_w = \sqrt[3]{\frac{(3)(40)(70.6)}{2}} = 16.2 \text{ in} \quad \text{web depth}$$

$$t_w = \frac{16.2}{40} = 0.405 \quad \text{web thickness}$$

The required web shear area is

$$A_w (\text{reg'd}) = \frac{120,000}{(2)(27,200)} = 2.20 \text{ in}^2$$

The available web shear area is

$$A_w = (16.2)(0.405) = 6.56 \text{ in}^2 > A_w (\text{reg'd})$$

The flange area of each flange should be about

$$A_f (\text{reg'd}) = \frac{6.56}{2} = 3.28 \text{ in}^2$$

for efficient use of the flanges. If we set

$$t_f = 0.5 \text{ in.}$$

$$\text{then } c_f = (1/2)(16.2 + 0.5) = 8.35 \text{ in.}$$

$$c = 1/2 [16.2 + (2)(0.5)] = 8.6 \text{ in.}$$

$$I_t = (70.6)(8.6) = 607 \text{ in.}^4$$

$$I_w = (1/12)(0.405)(16.2)^3 = 143 \text{ in.}^4$$

$$I_f = 607 - 143 = 464 \text{ in.}^4$$

$$b_f = \frac{464}{(2)(0.5)(8.35)^2} = 6.65 \text{ in.}$$

$$\text{Therefore, } A_f = (6.65)(0.5) = 3.32 \text{ in.}^2 \sim A_f (\text{reg'd})$$

$$S = \frac{(2)(6.65)(0.5)(8.35)^2 + (1/12)(.405)(16.2)^3}{8.6} = 70.6 \text{ in}^3$$

$$\text{Therefore, } S = S (\text{reg'd})$$

The ratio of flange width to thickness is

$$b_f/t_f = 6.65/0.5 = 13.3$$

which is within the safe limits designated by design criterion #2.

The assumed allowable compressive flange stress of $0.6\sigma_y$ was shown to be less restrictive than the following, by noting

$$S_b = \left[1.0 - \frac{(L/r)^2}{2C_c^2 C_b} \right] [0.66\sigma_y] = 0.58\sigma_y$$

$$\text{and } S_b = \frac{12 \times 10^6}{Ld/A_f} = 24,100 \text{ psi} = 0.35\sigma_y$$

$$\text{where } C_c = \sqrt{\frac{2\pi^2 E}{\sigma_y}} = 91.4$$

$$C_b = 1.75$$

$$L = 96 \text{ in.}$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{(1/12)(.5)(6.65)^3 + (1/12)(2.7)(.405)^3}{(.5)(6.65) + (2.7)(.405)}}$$

$$r = 1.67 \text{ in.}$$

$$A_f = 3.32 \text{ in.}^2$$

$$d = 2c = 17.2 \text{ in.}$$

$$d_w/6 = 2.7''$$

$$\sigma_y = 68,000 \text{ psi}$$

$$E = 28.8 \times 10^6 \text{ psi}$$

Since one of the original assumptions was shown to be invalid, another iteration was necessary. Beginning with the required section modulus, then

$$S (\text{req'd}) = \frac{2.88 \times 10^6}{(0.58)(68,000)} = 73.0 \text{ in.}^3$$

Since $K = 40$,

$$d_w = \sqrt[3]{\frac{(3)(40)(73.0)}{2}} = 16.4 \text{ in.}$$

web depth

$$t_w = \frac{16.4}{40} = 0.41 \text{ in.}$$

web thickness

The required web shear area is

$$A_w (\text{req'd}) = \frac{120,000}{(2)(27,200)} = 2.20 \text{ in.}^2$$

The available web shear area is

$$A_w = (16.4)(0.41) = 6.72 \text{ in.}^2 > A_w \text{ (reg'd)}$$

The flange area of each flange should be about

$$A_f = (\text{reg'd}) = \frac{6.72}{2} = 3.36 \text{ in.}^2$$

Setting $t_f = 0.5 \text{ in.}$, implies

$$c_f = (1/2)(16.4 + 0.5) = 8.45 \text{ in.}$$

$$c = (1/2) [16.4 + (2)(0.5)] = 8.7 \text{ in.}$$

$$I_t = (73.0)(8.7) = 635 \text{ in.}^4$$

$$I_w = (1/12)(0.41)(16.4)^3 = 151 \text{ in.}^4$$

$$I_f = 635 - 151 = 484 \text{ in.}^4$$

$$b_f = \frac{484}{(2)(0.5)(8.45)^2} = 6.78 \text{ in.} \sim 6.8 \text{ in.}$$

Therefore,

$$A_f = (6.8)(0.5) = 3.40 \sim A_f \text{ (reg'd)}$$

$$S = \frac{(2)(6.8)(0.5)(8.45)^2 + (1/12)(.41)(16.4)^3}{8.7} = 73.1 \text{ in.}^3$$

Therefore $S \text{ (available)} \approx S \text{ (reg'd)}$

The ratio of flange width to thickness is

$$b_f/t_f = 6.8/0.5 = 13.6$$

which is within the safe limits designated by design criterion #2.

6.2 JOIST DEFLECTION

The deflection of the joist span was computed to be

$$\delta_{\max} = \frac{(120,000)(96)^3}{(48)(28.8 \times 10^6)(636)} = 0.121 \text{ in.}$$

This is less than the design allowable of

$$\delta_{\text{allowable}} = (1/2)(1/360)(96) = 0.133 \text{ in.}$$

6.3 JOIST FILLET WELD SPECIFICATION

Gap between intermittent welds

$$D_{\text{max}}(T) = (24)(.41) = 9.84 \text{ in.} \quad \text{at tension flange}$$

$$D_{\text{max}}(C) = \frac{(4000)(.41)}{\sqrt{68,000}} = 6.29 \text{ in.} \quad \text{at compression flange}$$

The weld load is:

$$f = \frac{(60,000)(3.40)(8.45)}{(636)(2)} = 1360 \quad \text{lbs/in. for each weld}$$

The required weld leg size would be

$$\text{req'd leg} = \frac{1360}{(.707)(27,200)} = 0.071 \text{ in.}$$

for a weld material with an allowable shear strength of 27,200 psi. Probably, 3/16" continuous welds would be a good selection. For intermittent welds,

$$\frac{.071}{.188} = 37.8\%$$

therefore intermittent

or staggered

6.4 LOAD BEARING SUPPORT ADEQUACY

$$S_R = \frac{R_R}{(t_w)[N_R + k]} = 38,600 \text{ psi} \quad \text{for reaction area}$$

$$S_L = \frac{R_L}{(f_w) [N_L + 2k]} = 19,000 \text{ psi} \quad \text{for loaded area}$$

where $R_R = 60,000 \text{ lbs.}$

$R_L = 120,000 \text{ lbs.}$

$N_R = 3.1$ (girder width)

$N_L = 14$ (support ring width)

$k = 3/16 + .5 = 0.688"$

Both of these stresses are less than $0.75\sigma_y$ (51,000 psi). Therefore, transverse stiffeners are not needed in the beam support and load bearing regions.

6.5 OPTIMUM JOIST DESIGN

Having satisfied the beam design criteria, the optimum joist design based on dead weight considerations is as follows:

$$\begin{aligned} \text{where optimum joist design weight} &= (A_w + 2A_f) \left(.283 \frac{\text{lb}}{\text{in}^3} \right) \left(12 \frac{\text{in}}{\text{ft}} \right) \\ &= [6.72 + (2)(3.40)] [.283] [12] \\ &= 45.9 \text{ lbs/ft} \end{aligned}$$

7.0 NEUTRAL BEAM INJECTOR AND VACUUM PUMP SUPPORT

Although the size, weight, and locations of the eight Neutral Beam Injectors and eight Vacuum Pumps have not been finalized as of this writing, it is possible to evaluate the maximum load from these components that the support bed can support. The allowable loads are computed based on the maximum bending moment as well as the maximum deflection that the any given joist and girder can tolerate.

By inspection of the joist spans shown in Figure 2, the worst case location for a Vacuum Pump, together with a Neutral Beam Injector, would be midway along the 139-inch joist span (see Figure 5). The maximum allowable moment in this case would be

$$\begin{aligned}M_{\max}(\text{joist}) &= SS_b \\&= (73.0 \text{ in.}^3)(0.58)(68,000 \frac{\text{lb}}{\text{in}^2}) \\&= 2.88 \times 10^6 \text{ in-lbs.}\end{aligned}$$

$$\text{Since } M_{\max} = \frac{WL}{4}$$

$$\text{Then } W = \frac{(4)(2.88 \times 10^6)}{139} = 82,900 \text{ lbs.}$$

The joist deflection under such a VP and NBI load would be

$$\begin{aligned}\delta_{\max}(\text{VP \& NBI}) &= \frac{(82,900)(139)^3}{(48)(28.8 \times 10^6)(636)} \\&= 0.253 \text{ in.}\end{aligned}$$

This compares to an allowable deflection of

$$\begin{aligned}\delta_{\text{allowable}}(\text{VP \& NBI}) &= (1/2)(1/360)(139) \\&= 0.193 \text{ in.}\end{aligned}$$

Since $\delta_{\max} > \delta_{\text{allowable}}$, the deflection criterion will govern the allowable VP and NBI load for the joist.

$$\begin{aligned}
 W_{\text{allowable}} (\text{VP \& NBI}) &= \frac{48EI \delta_{\text{allowable}}}{L^3} \\
 &= \frac{(48)(28.8 \times 10^6)(636)(0.193)}{(139)^3} \\
 &= 63,200 \text{ lbs.}
 \end{aligned}$$

A check on the allowable VP and NBI load for a girder indicates that the allowable combined load should be 11,500 lbs.

$$\begin{aligned}
 W &= \frac{(4)(9.05 \times 10^5)}{149} = 24,300 \text{ lbs.} && \text{based on bending moment} \\
 &\text{assuming } L = 149 \text{ in.} && \text{(longest girder span length} \\
 &&& \text{supporting VP and NBI load)} \\
 W &= \frac{(48)(28.8 \times 10^6)(133)(.207)}{(149)^3} = 11,500 \text{ lbs.} && \text{based on deflection}
 \end{aligned}$$

$$\text{where } \delta_{\text{allowable}} = (1/2)(1/360)(149) = .207 \text{ in.}$$

8.0 SEISMIC EVENT

In the final design optimization, consideration will have to be given to the occurrence of an earthquake as defined by reference 2. Reference 2 indicates that New Jersey building codes dictate that the potential effects of exposure to a Richter 5 seismic event should be accounted for in all construction design projects. This is not felt to be a severe seismic design requirement.

Some preliminary information must be supplied by the architect-engineer before such a seismic qualification could be undertaken. The Richter 5 acceleration magnitude of

0.05 g applies to ground motion at the TFTR site. Since a magnification of this excitation will occur in the TFTR device response, it is necessary for the architect-engineer to supply a floor response spectrum for the device location in the containment building. This data would include damping effects of the building.

The most convenient way of designing for a seismic event is to use the equivalent static load method. The vertical motion effects can be accounted for by establishing an adequate design using an appropriate factor applied to the dead weight of the TFTR device. This factor is dependent on the natural frequency of the support bed and will be given by the floor response spectrum. Some idea of the support bed natural frequency is obtained by considering 1/10 of the total device weight supported by the combined stiffness of the joist and girder.

$$K_{eq} = \frac{k_1 k_2}{k_1 + k_2}$$

$$k_1 = \frac{(48)(28.8 \times 10^6)(133)}{(30.2)^3}$$

$$k_1 = 6.68 \times 10^6 \text{ lb/in.}$$

$$k_2 = \frac{(48)(28.8 \times 10^6)(636)}{(96)^3}$$

$$k_2 = 9.94 \times 10^5 \text{ lb/in.}$$

$$K_{eq} = 8.65 \times 10^5 \text{ lb/in.}$$

$$m = \frac{120,000}{386} = 311 \frac{\text{lb}_f \text{ sec}^2}{\text{in.}}$$

$$f = \frac{1}{2\pi} \sqrt{\frac{K_{eq}}{m}} = 8.4 \text{ Hz}$$

Even if fixed beam supports are assumed, then

$$f = \sqrt{\frac{192}{48}} (8.4) = 16.8 \text{ Hz}$$

$$k = \frac{48EI}{L^3} \text{ simply supported beams}$$

Generally speaking, if the equipment natural frequency is in the range of three to twenty-five Hertz, there occurs a magnification of the ground excitation in the equipment response. However, even a fairly realistic magnification factor of 5 on the ground excitation, which would imply a response of 0.25g, could be easily handled by another I-beam design optimization iteration.

The lateral horizontal seismic motion will have to be resisted by the set of dual roller bogies located beneath the center of the TFTR device. Any potential problems in this area could be easily handled by increasing the number of roller supports. The I-beam web and flange integrity would also have to be checked for horizontal motion effects.

9.0 REFERENCES

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FIGURE 1. Device Substructure

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S= SINGLE ROLLER
 D= DUAL ROLLER
 C= CONCENTRATED LOAD
 U= UNIFORM LOAD

FIGURE 2. Layout Drawing of Substructure

FIGURE 3. Maximum Allowable K vs Material Yield Strength
When No Beam Transverse Stiffeners are to be Used

FIGURE 4. Relationship of Efficient Girder Weight and Web Depth

FIGURE 5. Vacuum Pump and Neutral Beam Injector
Supported by Substructure