# APPENDIX 5D

# REPORT ON STRESS ANALYSIS OF BUTTRESS

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# 1 DESIGN DETAILS

## 1.1 TENDON ANCHORAGE HARDWARE

The prestressing system employs the BBRV anchorage system utilizing 169-1/4" diameter wires conforming to ASTM A421-65, Type BA. The dimensions and capacities of the tendons used hereafter for the anchorage zone analysis are more fully described in Appendix 5B, Part D.

## 1.2 ANCHORAGE ZONE REINFORCEMENT

The mild steel reinforcement in the tendon anchorage zones of the buttress and the base mat are shown on GAI Drawings 421004, 421005, 421006, 421031,421032, and 421033. This reinforcement was conservatively designed by the methods enumerated hereafter in Section 3 of this Appendix and is judged to be sufficient to accommodate all predicted stress states without jeopardizing the ability to satisfactorily place the concrete.

# 2 TENDON LOADS AND ALLOWABLE STRESSES

## 2.1 TENDON LOADS

The minimum guaranteed ultimate tensile strength of the tendons is approximately 2,000 kips. Before the tendons are anchored (locked off) at 70 percent of the ultimate tendon force, which is equal to 1,400 kips, they may be temporarily overstressed to 1,600 kips in order to reduce friction losses. The anchor zones are designed for anchor forces of 1,400 kips per tendon. The anchor zones are also checked to resist safely temporary anchor force of 1,600 kips per tendon.

## 2.2 ALLOWABLE STRESSES

The allowable stresses for concrete and steel will conform to those given in Section 5.2.2 and 5.2.3 of the FSAR.

Due to the specialized nature of anchor zone design the following conditions and stress criteria will be used in the design. These stress criteria include adequate conservatism in their application.

First, verify that the allowable concrete compression and tensile stresses are not exceeded due to bearing and bursting stresses respectively.

Second, check the tensile stresses in reinforcing steel when no credit is taken for concrete tensile capacity in the investigated anchor zones.

Under these conditions, the allowable stresses will be shown as follows:

CONCRETE

1. Allowable bearing stress

$f_{cp} = 0.6 f'_{ci} \sqrt[3]{A'_b / A_b}$

according to ACI/318-63, [1]
Paragraph 2605(C)

$f'_{ci} = 5.0$ ksi.

2. Allowable compression stress in the structure beyond the anchorage zone $f_c = 0.45 f'_c$

$f'_c = 5.0$ ksi.

3. Allowable Tension Stress

$3\sqrt{f'_c}$

$f'_c = 5.0$ ksi.

REINFORCING STEEL

Allowable tension for

A615-Grade 40, $f_s$ = 20 ksi.

3 ANCHOR BLOCK STRESSES

The investigation of the buttress anchorage zones for the hoop tendons is described in the following sections. The analytical work as presented, is based on the end anchor zone stresses and stress flow pattern studies documented by Y. Guyon (Reference 2) and F. Leonhardt (Reference 3). Their work has been substantiated through extensive strain measurements and stress flow pattern established by studies utilizing photoelastic techniques.

The design presented herein is quite conservative; nevertheless, it has been adopted as the basis for determination of required reinforcing steel in the end anchor zones to overcome tensile splitting stresses and to provide for spalling stresses near the concrete surface.

The existance of a three-dimensional stress field including high shear, thermal and shrinkage tensile stresses is influenced by hairline cracking due to creep and thermal stresses. In order to accommodate the most unfavorable stress combination in the anchorage zones, the following design goals were established:

1. Maximum concrete bursting tensile stresses have been kept below maximum allowable tensile stress of conservatively $3\sqrt{f'_c}$ using Y. Guyon's symmetrical prism method. This would indicate that no special spiral and bursting reinforcement is required. The

combined stresses during normal operation and test loading also indicate that the concrete will remain uncracked in the anchorage zone.

2. However, due to the existance of complex thermal stresses and possible microcracking, the ability of concrete to carry tensile stresses has been discarded, and mild steel reinforcing has been provided for:

   a. the equivalent fluid pressure
   b. transverse tensile stresses
   c. unbalanced vertical forces
   d. unbalanced horizontal forces
   e. equilibrium condition

   The combined stresses during the design pressure for the Reactor Building (p = 55 psig) indicate that the concrete will remain uncracked in the anchorage zone. The combined stresses during the factored accident loading (refer to 5.2.3.2, cases a. and b. indicate that the concrete in the anchorage zone may develop hairline cracks.

3. Existing theoretical solutions for anchorage zone design do not agree with more recent experimental results (References 17 and 18); maximum transverse tensile forces were made up to 5.8 times larger than determined by Y. Guyon.

4. Possible failure mechanisms due to spalling and bursting stresses were evaluated to verify static equilibrium and acceptable stress limits under such hypothetical conditions.

5. Bursting stresses in a large body are much smaller than in a member bounded by planes (Reference 19). Therefore, while it was found that two-dimensional theories underestimate the bursting stresses (Reference 17), analytical results based on isolated prisms or better, experimental values such as those developed by J. Zielinski and R. E. Rowe can be used for massive structures (Reference 19).

6. The design of the anchorage zones provides a margin of safety similar to the entire reactor building. Furthermore, the reinforcement is designed to control cracking due to spalling, bursting, or shear of the concrete under sustained load.

3.1 <u>BEARING STRESSES</u>

Figures 5D-7 through 5D-10 show the typical hoop tendon arrangement and buttress anchor zone.

Bearing area: $\mathrm{A_b} = 24^2 - 11^2\left(\frac{\pi}{4}\right) = 481\,\mathrm{in.}^2$

Concrete anchor surface: $\mathrm{A'_b}\,33^2 - 11^2\left(\frac{\pi}{4}\right) = 994\,\mathrm{in.}^2$

Allowable bearing stress: $\mathrm{f_{cp}} = 0.6(5.0)\left(\frac{994}{481}\right)1/3 = 3.82\,\mathrm{ksi}$

Use of 28 day concrete strength is conservative since initial tensioning will occur significantly later in time.

Bearing pressure for anchor force:

a. $P = 1633.5\ \text{kips},\ f_b = \dfrac{1633.5}{481} = 3.40\ \text{ksi} < f_{cp} = 3.82\ \text{ksi}$

b. $P = 1866.8\ \text{kips},\ f_b = \dfrac{1866.8}{481} = 3.88\ \text{ksi}$

The depth below the bearing plate, at which concrete stresses are identical to those allowed beyond the anchorage zone area, is computed on the conservative assumption that the load spreads at a 30$^{o}$ angle from the plane normal to the bearing plate surface.

Obtain the required concrete area to satisfy the above condition for:

P = 1633.5 kips, $A_g = \dfrac{1633.5}{2.25} = 727\ \text{in}^2$

If it is conservatively assumed that the anchor force spreads at an angle of 30$^{o}$, the depth (h) below the bearing plate at which the concrete stress will have been reduced to the allowable compression stress as shown above, can now be computed. See Figure 5D-8b.

$$A_g = (20.5 + 2h\tan 30^\circ)^2 - \left(7^2\left(\frac{\pi}{4}\right)\right)$$

$$622 = \left(20.5 + \frac{2h}{\sqrt{3}}\right)^2 - 38.5,\ h = 4.5\ \text{in.}$$

Special provisions must be made to confine the concrete within the 4.5 in. below the bearing plate. From then on, spiral reinforcement meeting the requirement for tensile splitting stresses will be investigated as described in part 3.3.

3.2 EQUIVALENT FLUID PRESSURE METHOD

Directly below the bearing plate the necessary reinforcement for confining the concrete is computed for an assumed equivalent fluid pressure "p". The equivalent fluid pressure is equal to the concrete compression stress multiplied by the Poissons Ratio, which is assumed to be 0.2.

The concrete compression stress 2 in. below the bearing plate,which is about at the center of the critical 4.5 in. depth, will be used to determine the equivalent fluid pressure. The initial 1,400 kips anchor force is considered. See Figure 5D-8b.

$$A_g = \left[(20.5 + 4(\tan 30^\circ)\right]^2 - \left[(7^2(\frac{\pi}{4}))\right] = 481.5\ \text{in}^2$$

$$f_c = \frac{1400}{481.5} = 2.91\ \text{ksi}$$

The equivalent fluid pressure will be:

p = 2.91 (0.2) = 0.58 ksi

The reinforcement is arranged as shown on GAI Drawing 421031 . The #5 spiral has a diameter of 20 inches with a 3 1/2 in. pitch, and a total length of 36 inches. Horizontal ties 2-#10 and vertical reinforcement 1-#10 plus 1-#8 are also available to resist this equivalent fluid pressure.

The required steel area for the 20 inch diameter spiral was computed for an allowable steel stress of $f_s$ = 20 ksi.

Required steel area:

$$A_s = \frac{0.58(4.5)(20)}{20} = 2.61 \text{ in}^2$$ (steel area req'd. over a 4.5 in length below the bearing plate)

Steel provided:

a. Radially, Spiral 2-#5 = 0.62 in$^2$
Ties 2-#10 = <u>2.54</u>
3.16 in$^2$ > 2.61 in$^2$

b. Vertically, Spiral 2-#5 = 0.62
Vert. 1-#10 = 1.27
Vert. 1-#8 <u>0.79</u>
2.68 in$^2$ > 2.61 in$^2$

## 3.3 TRANSVERSE TENSION-PRISM METHOD

Transverse stresses are produced when the anchor force spreads out into the surrounding concrete. Directly behind the bearing plate the stress lines are parallel to the tendon axis. The angle change of the stress-flow-lines, where they bend outwards, produces in these regions compression in the concrete which is of no concern. Further away, the stress-flow-lines tend to bend back and tensile transverse stresses are created as a result of this angle change. Figure 5D-9 and 5D-10 show the idealized compression stress-flow-lines in plane and the approximate location of transverse stresses.

Further, it can be seen that the angle changes will be small, and that the stress intensity along the compression stress-flow-lines reduces as they spread out into the concrete mass. From these considerations it can be concluded that the resulting tensile transverse stresses will be small and within the tensile capacity of the concrete. Deformed vertical, circumferential, and radial reinforcement is provided throughout the buttress in addition to the spiral reinforcement at each bearing plate. The reinforcement will tie the concrete mass together and ensures that thermal cracking of the concrete mass, if it should occur, is of no significance as far as structural integrity of the Reactor Building is concerned.

Figure 5D-10 shows that the critical condition, with respect to transverse tensile stresses in the vertical plane, exists when only individual tendons are stressed. When all tendons are stressed, the stress-flow-lines are more or less parallel. Figure 5D-9 and 5D-10 show, superimposed over the anticipated stress-flow-lines, "equivalent beams" for which the transverse stresses can be computed. Computation for transverse stresses in radial planes are performed in tabulated form in Table 5D-1. Three cases are analyzed as follows:

<u>Case 1</u>

Finds the maximum transverse radial stress in symmetrical equivalent beam. Variation la investigates anticipated conditions during normal operation. Variations 1b and 1c are made for extremely shallow and extremely deep "equivalent beams" to establish how variations of beam depths affect the magnitude and location of the transverse stresses.

<u>Case 2</u>

Investigates the effect of eccentric loads on equivalent beams. Guyon's "Symmetrical Prism Method" is used. According to this method the depth of the eccentric beam does not affect the magnitude of radial transverse stresses; only the dimension of the prism drawn symmetrical to the tendon center line determines their magnitude. Possible variations in eccentric beam depths are therefore of no concern.

All stresses were computed on the conservative assumption that the load spreads at 30$^{o}$ in the circumferential direction. The stress computations confirm the previously made prediction that transverse stresses are within the acceptable concrete stress range.

<u>Case 3</u>

Finds the maximum transverse vertical stresses in symmetrical equivalent beam. Variations 3a and 3b investigate anticipated conditions during normal operation. Vertical transverse forces counteract each other under operating conditions largely through vertical compression forces in the buttress concrete. See Figure 5D-10b. Variation 3c investigates anticipated conditions if only one anchor force acts. See Figure 5D-10a.

3.3.1 REINFORCEMENT FOR TRANSVERSE FORCES

The allowable concrete tensile stresses in the radial and vertical planes are not exceeded and no special transverse reinforcement is required for design loads. However, reinforcement is available in the form of buttress reinforcement in the hoop and meridional direction consisting of a radial tie-reinforcement anchored into the shell and spiral reinforcement in the maximum bursting zone.

The buttress is reinforced sufficiently to resist all transverse tensile forces Zy (design) acting within the limits of the buttress without relying on concrete tensile resistance.

Spiral: 5/8" ?, 3 1/2" pitch, d = 20", length = 36"
Maximum tensile splitting stress, Table 5D-1, $f_{by}$ =0.178 ksi

$$T = 3.5\,(0.178)\,\frac{(20)}{2} = 6.24^{K},\quad f_s = \frac{6.24}{0.31} = 20.1 \text{ ksi}$$

The spiral is designed for Zy (design) = 420 kips

Maximum transverse tensile force as computed is $Z_y$ = 311 kips.

Steel provided:

| | | |
|---|---|---|
| Spiral | 22 - #5 | = 6.82 $in^2$ |
| Ties | 10 - #10 | = <u>12.70</u> |
| | | 19.52 $in^2$ |

$$f_s = \frac{420}{19.52} = 21.5 \text{ ksi}$$

Temperature changes in the concrete during the life of the plant or at accident condition will produce a maximum of less than 1 ksi stress in the bursting zone reinforcement.

## 3.4 UNBALANCED FORCES

Figure 5D-11 shows the tendon anchor forces for six buttresses in the Reactor Building.

The solid line horizontal force vector in Figure 5D-11 represents the tangential components of the anchor forces. The anchor forces are applied on opposite faces of the buttresses and counteract each other. However, since the vertical spacing of the anchors varies, it is not possible to balance all the forces. The unbalanced horizontal forces must be transferred from the buttress into the Reactor Building wall.

The two dashed force vectors, Figure 5D-11, adjacent and parallel to each anchor force vector are obtained by proportioning the anchor force according to its position with respect to two opposing anchors similar to determining the reactions of a simply supported beam with an unsymmetrical concentrated load. The dashed horizontal force vector on the buttress center line indicates the unbalanced force between two opposing anchor forces. How these horizontal forces are resisted is discussed later.

Since the horizontal anchor forces on opposing sides are offset from each other, the forces must change direction in order to balance each other. Transverse forces in vertical direction are a result of this angle change. The vertical dashed force vectors shown in Figure 5D-11 represent the unbalanced vertical forces behind each anchor zone.

Vertical forces in Figure 5D-11 were computed for the simplified assumption that the anchor forces travel in straight lines from anchor plate to anchor plate after first moving 18 in. horizontally into the concrete and gradually bend over into the inclined position. See Figure 5D-12.

The check analysis employing the Finite-Element Method did reveal a limited region of surface tensile stresses which presumable would produce cracking. However after concrete cracking has occurred with the extent limited by supplemental reinforcing a new state of equilibrium will exist. The following section discussing vertical and horizontal unbalanced forces verifies that a mechanism does exist to maintain equilibrium.

### 3.4.1 VERTICAL UNBALANCED FORCES

During initial prestressing and during tendon inspection the balance of opposing tendon anchors might not exist and the total vertical transverse stress must be resisted. The maximum concrete transverse

tensile stress for this condition was computed in Table 5-2, Case 3c, $f_{by}$ = 115 psi. Figure 5D-10a shows the anticipated typical vertical stress-flow-lines for such conditions. The figure also shows the "equivalent beam" for which transverse stresses were determined according to Guyon's Symmetrical Prism Method. Sufficient reinforcing steel in form of a spiral has been provided for Case 3c without relying on concrete tensile resistance. In addition, the vertical reinforcement in the buttress, consisting of 7-#10, As = 8.8 in.,(Reference 2) can resist a transverse splitting force of Z = 8.8 (0.95) (40) = 335 kips which is slightly more than the expected force Zy = 311 kips shown in Table 5D-1. The reinforcement is placed mainly on the buttress surfaces in order to provide access for concrete placement. There is no need to distribute this reinforcement throughout the cross-section of the buttress or to place it where the vertical transverse forces actually act. Wherever the reinforcement is placed, it will not prevent the concrete from cracking and will only tie the concrete together preventing the development of uncontrolled cracks. Consequently, surface reinforcement is sufficient. It restrains the buttress in the vertical direction and no failure mechanism caused by vertical transverse stresses can develop.

The highest vertical in-plane shear stresses at the buttress-cylinder intersection occur directly beneath the ring-girder and at the cylinder-base slab intersection haunch area. An amplification factor of 1.64 was applied to the shear stresses in both areas to account for the increased stiffness at the buttress. Sufficient reinforcing was provided between the buttress and cylinder to take the shear stresses.

3.4.2 HORIZONTAL UNBALANCED FORCES

Unbalanced horizontal forces shown in Figure 5D-11 must be transferred into the wall.

During initial prestressing and tendon inspection the counteracting effect of opposing anchor forces might be lost at any any anchor zone position. All anchor zones will therefore be reinforced as the analysis requires for such conditions. Figure 5D-9a shows the anticipated typical horizontal stress-flow-lines for such conditions. The figure also shows the "equivalent beams" for which transverse stresses were determined according to Guyon's "Symmetrical Prism Method."

The analysis for horizontal transverse forces is tabulated in Table 5D-1. The critical loading condition exists when a tendon is stressed without being counterbalanced by opposing anchor forces. The maximum concrete transverse tensile stress for this condition is represented by Case 2a, $f_{by}$ = 178 psi. Sufficient reinforcing steel in form of a spiral has been provided for Case 2a without relying on concrete tensile resistance. In addition, the radial tie reinforcement in the buttress, consisting of 10-10#, $A_s$ = 12.7 in., (Reference 2) can resist a transverse splitting force of Z = 12.7 (0.95) (40) = 482 kips which is larger than the expected force Zy = 311 kips shown in Table 5D-1. The tie reinforcement restrains the buttress in radial direction and no failure mechanism caused by radial transverse stresses can develop.

The highest shear stresses for the case of no opposing prestress force in the buttress occurs at the buttress-cylinder discontinuity on the side of the buttress opposite the existing prestress force. Reinforcing of #10 at 12 in. (See Figure 5D-4) are provided to take the resulting tensile stresses.

### 3.4.3 EQUILIBRIUM CONDITION IN THE ANCHOR BLOCK

The reinforcement is checked based on the assumption of an existing spalling crack in the vertical direction. The forces and the extent of cracking in the anchor block were based on a method developed by Gergely, Soyen, and Siess of arresting cracks due to transverse stresses (Reference 4). The idealized mathematical model used for this approach, see Figure 5D-13, is quite conservative. The computation is shown in Table 5D-2. It should be noted that zero moment exists at B' when c = 27 in., and that the governing moment, Mm, prevails at c = 42 in., that is, far beyond the face of the wall.

Concerning the visco-elastic behavior of concrete, it can be stated that the resulting redistribution of stresses developed by Gergely will not significantly affect reinforcing requirements (Reference 5). The super-position of bursting stresses and normal stresses from vertical tendons results in a tri-axial state of stress which is less critical than during construction stages. No additional investigations for the final state of stress in the anchorage zones, therefore, appear necessary. The various sources of errors in a numerical tri-axial analysis (uncertainties of material properties, element definition, etc.) may lead to less accurate values than were obtained by two-dimensional methods.

The shear across the cylinder at the buttress-cylinder intersection for the case of unbalanced prestressed force has been checked. Sufficient reinforcing had been provided to account for the shear stresses. For accident temperature and pressure conditions a crack across the entire cylinder at the buttress cylinder discontinuity has been assumed. Sufficient steel reinforcing is provided to take any tension where the compressive stresses due to prestress have been overcome.

## 4 CHECK ANALYSIS BY FINITE-ELEMENT METHOD

### 4.1 INTRODUCTION

The finite-element method has been employed to check the design of tendon anchorage zones in the Reactor Building. The purpose of this analysis was not as a design basis but simply as a check to demonstrate the conservatism of the method described here before. This section presents the method of analysis, load conditions, and results for the buttress and adjacent wall, and in conclusion to correlate these results with the design of the buttress reinforcement which has been presented in Section 3.

The dominate loads are:

a. Prestress forces

b. Thermal loads

c. Pressure forces due to loss of coolant accident (LOCA)

Although the stress analysis of the buttress and adjacent wall is a three dimensional problem, it can be reduced to a two dimensional problem, as the meridional stress of the cylindrical portion of the vessel is usually constant. The finite-element method of plane stress analysis (References 6 and 7) which can account for complex loadings, arbitrary geometric configurations, and arbitrary boundary conditions, assumes a homogeneous material (i.e., no

cracks) and is an ideal tool for dealing with such a problem. Therefore, a plane stress finite-element computer program (Reference 7) was used in the buttress analysis.

For the plane stress finite-element program the thermal loads were obtained from another computer program. The results from this program, for the steady state winter normal operation and the transient state during winter start-up, winter shut-down, and winter loss of coolant accident were used as input data in the finite- element program.

As described later, a system of 768 direct stiffness simultaneous equations required solution. To save computer storage, the Gauss-Seidel iterative method (Reference 8), which requires the storage of only the non-zero elements of the stiffness matrix, was employed. In order to expedite convergence, an over-relaxation factor (Reference 9) was also used in Reference 7; this factor usually ranges from 1.0 to 2.0.

Cracking of the concrete for the winter LOCA load condition is predicted. Areas of high tensile stresses have been defined and reinforcement capable of resisting these provided.

## 4.2 PLANE STRESS FINITE-ELEMENT COMPUTER PROGRAM

Due to the fact that the meridional stresses of the cylindrical portion of a containment vessel are essentially uniform and constant, the horizontal cross section of the cylinder may be considered as a section subjected to plane stresses only. The finite-element method is a versatile method of analysis and is capable of providing states of stress in the buttress for complex loadings, arbitrary geometric configurations, and arbitrary boundary conditions. Tendon holes are not included in the buttress geometry.

In the plane stress finite-element analysis, the horizontal cross sectional plane of the cylindrical portion of the vessel is replaced by a finite number of elements interconnected at a finite number of nodal points. Continuity between elements of the system is maintained by requiring that within each element, "lines initially straight remain straight in their displaced positions." This requirement is satisfied if the strains within each element are assumed to be constant. Therefore, the stresses within each element are also constant. Distributed loads are replaced by equivalent nodal forces which act at the three nodes of the element. Based on these assumptions, it is possible to derive the stiffness matrix of a typical element, which is an expression for the nodal forces resulting from unit nodal displacements:

$$\{p\} = [k]\ \{?\} \qquad (1)$$

where {p} is the vector of six nodal forces which corresponds to the vector {?} of six nodal displacements (Figure 5D-14), and (k) is the stiffness matrix of the element.

The nodal force vector {p} can be expressed as:

$$\{p\} = \{PX_i, Px_j, Px_k, Py_i, Py_j, Pyk\}) \qquad (2)$$

and the nodal displacements vector {?} can be expressed as,

$$\{?\} = \{u_i, u_j, u_k, v_i, v_j, v_k\} \qquad (3)$$

which are shown in Figure 5D-14.

Formulation of Element Stiffness Matrix

Corresponding to the above assumptions, the displacements of a point at (x,y) are written as linear functions of the coordinates:

$$u = u_i + C_1x + C_2y$$
$$v = v_i + C_3x + C_4y \qquad (4)$$

The stiffness matrix [k] can be obtained by the following steps:

a. Substitute the nodal coordinates into Equation(4) to obtain the displacement functions in terms of the nodal displacements.

b. Substitute results of (a) into the linear strain-displacement equations to obtain the strains in terms of nodal displacements.

c. Substitute the results of (b) into the stress-strain equation to obtain the stress in terms of nodal displacements.

d. Find the nodal resultant forces which are statically equivalent to the stresses found in (c) so that the relationships between the nodal forces and the nodal displacements can be written in a matrix form as shown in Equation (1).

The explicit formulations of the matrices which compose the element stiffness matrix can be found in Reference 7.

Solution of the Total Stiffness Equations
After the stiffness matrices are formulated for each of the elements,
they are assembled to form a total system stiffness matrix. If the boundary conditions and the loading conditions are properly specified, the total stiffness simultaneous equations are readily solvable to give the nodal displacements. Then, the nodal displacements are used to generate the stresses in each element and at each node. The directions and magnitudes of the principal stresses can also be computed by using the Mohr's circle technique.

Iterative Method and Optimum Over-Relaxation Factor
The Gauss-Seidel iterative procedure (Reference 8) is applied to solve the stiffness simultaneous equations. The method requires the storage of only the non-zero elements of the stiffness matrix during the iterative process.

Since the stiffness matrix [k] is positive definite,the iterative method will always converge (Reference 10). The covergence rate is, however, often rather slow. An over-relaxation factor (Reference 9) can be applied which greatly increases the convergence rate. The determination of the proper over-relaxation factor is difficult and is based mostly on experience and trial and error.

4.3 GEOMETRY OF BUTTRESSES WITH TENDONS

The pertinent geometry of the Reactor Building as it affects the buttress analysis is shown in Figure 5D-15. In Figure 5D-15a, a sectional view illustrates the vessel with six buttresses. Figure 5D-15b, shows the arrangement of the hoop tendons on a stretch out of the cylindrical

wall. Figure 5D-15c, shows the dimensions of one buttress and the arrangement of the hoop and vertical tendons.

## 4.4 THERMAL LOADS AND TEMPERATURE PROFILES

If the temperature profile in a buttress is known, the difference between the temperature of any point and the reference temperature can be determined. The temperature difference, with the modulus of elasticity and thermal expansion coefficient of the concrete, provides the equivalent constant stress for each finite-element. The constant thermal stress of each element can be converted to equivalent nodal resultant forces which can be considered as external forces applied at the nodal points of a non-heated element.

Temperature Profiles

The temperature profiles and iso-thermal curves were calculated by a computer program developed by Brunnel (Reference 11) for the solution of steady state and transient state problems in one, two, or three dimensional Cartesian or cylindrical systems. The governing differential equations were solved by the method of finite differences, using standard relaxation techniques.

The convergence criterion:

$$\frac{T_j\text{ (step i)} - T_j\text{ (step i-1)}}{Y_j\text{ (step i)}} \leq 0.00001 \qquad (5)$$

where $T_j$ is the temperature at node j, was applied at every node of the difference lattice. To ensure stability of the transient solutions, the time increment was selected according to the method presented in Reference 11.

The buttress and wall temperatures were determined for the following conditions:

a. Normal operation in winter
b. Start-up in winter
c. Shut-down in winter
d. Loss of coolant accident during winter

The winter season was chosen because it produces the greatest thermal gradients in the shell of the Reactor Building.

The mean monthly temperature was chosen due to the fact that day- night fluctuations had little effect on the interior portion of the concrete. According to Carslow and Jeager, (Reference 12) the amplitude of the temperature diminishes as:

$$\exp\left[X \frac{wP_{con} \quad C_{p\ con}}{2\ k_{con}}\right] \qquad (6)$$

Thus a distance of 12.8 inches from the outer wall of the Reactor Building the effect of the oscillation is only 0.1 what it is on the outer face, and the middle and inner portion of the vessel will be unaffected by day-night temperature oscillations.

For the start-up condition, it was assumed that the inside air temperature of the Reactor Building rose from 60 F to 110 F in one hour, and for shut-down the reverse was used. The inside film coefficient was taken as 1.490 Btu/hr/sq ft/F for cases a, b, and c above. This was modified film coefficient, which took into account the steel liner and the still air between the liner and the concrete:

$$h_m = \frac{1}{\frac{1}{h_i} + \frac{X_{steel}}{k_{steel}} + \frac{X_{air}}{k_{air}}} \qquad (7)$$

Equation 7 neglects the circumferential conduction and heat capacity in the steel liner and air gap. However, due to the relatively small thickness of the liner and the air gap when compared with the thickness of the concrete wall, these effects may be neglected. The preliminary analyses verified this assumption. The following values, obtained by theoretical analysis (Reference 13), were used in the thermal analysis:

inside film coefficient = 2.0 Btu/hr/sq ft/F

specific heat, $C_p$ con = 0.210 Btu/lb/F

thermal conductivity, $k_{con}$ = 10.0 Btu in./hr/sq ft/F

$k_{steel}$ = 300.0 Btu in./hr/sq ft/F

$k_{air}$ = 1.836 Btu in./hr/sq ft/F

The density of the concrete, $P_{con}$ was assumed to be 140 lbs/cu ft.

The above values agree closely with the results from experimental research carried out in Japan (Reference 14).

$C_{p\ con}$ = 0.200 Btu/lb/F

$k_{con}$ = 11.28 Btu in./hr/sq ft/F

The density of the concrete $P_{con}$, was assumed to be 146 lb/cu ft.

The thickness of the steel and air gap was:

$X_{air}$ = 1/32 inch

$X_{steel}$ = 3/8 inch

Using Reference 15, the local film coefficient for air at 20 F and a free stream velocity of 20 mph flowing past a concrete cylinder was calculated to be:

h = 0.56 Btu/hr/ft$^2$/F

As a safe approximation the outside film coefficient was chosen as:

$h_?$ = 1.0 Btu/hr/sq ft/F

The buttress model was replaced by a mesh of 61 nodes, with adiabatic boundaries at the centerline of the buttress and at the radial mid-section of the wall between two neighboring buttresses (Figures 5D-16 and 5D-17). Due to symmetry, there is no heat transferred across these boundaries. For Case d the liner temperature during LOCA was used as the inner boundary temperature in conjunction with a film coefficient of 5.880 Btu/hr/sq ft/F, which simulated the conduction through the air gap:

$$h_m = \frac{k_{air}}{X_{air}} \qquad (8)$$

The transient state temperature analyses were performed at various time steps.

The temperature profiles for Cases a, b, c, and d are shown in Figure 5D-18. Because the most severe temperature gradients occur for case a and d, only these two cases were considered in the stress analyses. Iso-thermal curves for cases a and d are shown in Figures 5D-19 and 5D-20.

The design pressure inside the Reactor Building during LOCA is equal to 55 psig and the maximum pressure was initially calculated to occur at approximately 200 seconds after initiation of the accident. The maximum liner temperature of 280F was initially calculated to occur at approximately 650 seconds after the initiation of the accident. However, the maximum temperature gradient, which causes the most severe thermal stresses, was calculated to occur at 10,000 seconds after initiation of the accident.

Consequently, the determination of the most severe stress state, due to thermal and pressure loads, requires that a number of stress analyses be conducted at various elapsed times after commencement of the accident. A conservative estimate of the stresses was obtained, by superimposing directly the maximum values for accident temperature and pressure. Consequently although subsequent calculations alter the time at which minimum pressure and liner temperature occur, these changes do not affect this analysis.

4.5 NON THERMAL LOADS

In addition to the thermal loads, there are other loads which act on the structure:

a. Hoop Tendon Anchor Pressure

The tendon force was assumed to be distributed uniformly up the face of the buttress. The resulting average force (=$p_a$) was applied to each of six nodes. The location of the nodes approximately corresponded to the position of the tendon.

b. Hoop Tendon Pressure in a Radial Direction

Radial pressures, due to the hoop tendons in the wall, were calculated as average stresses in pounds/in.

The two radial pressures applied (refer to Figure 5D-21):

1. From point A to the tangent point T. Over this length the tendon spacing causing radial pressure (i.e., continuous past the buttress) is 33 in., giving a pressure of $p_t$, and

2. For the length from point T to point B (midway between buttresses) the average tendon spacing causing radial pressure is 16 1/2 in., giving a pressure of $2p_t$.

c. Internal Pressure Due to LOCA

Internal pressure acting on the inside face of the Reactor Building Wall was considered as a combination of two separate effects:

1. Pressure due to LOCA. The maximum pressure due to LOCA was taken at 82.5 psig. (1.5 P)

2. Pressure due to expansion of the liner caused by the increase in temperature during the LOCA condition (110F to 280F) based upon a limiting yield strength of 1.2 $f_y$.

The above two loads were assumed to act simultaneously during the LOCA condition, giving an equivalent internal pressure of $p_i$.

The magnitude and combination of the above non-thermal loads is noted in section 4.7

## 4.6 MATERIAL PROPERTIES

### Prestressing Steel Wire

The type of prestressing wire to be used in the construction is a stress relieved wire. Dimensions, properties, etc., of the wire as used in the analysis were:

a. Size of tendon = 169 wires
b. Diameter of wires = 1/4 in.
c. Minimum guaranteed ultimate tensile stress = 240,000 psi
d. Tendon force @ 70 percent GUTS = 1400 kips
e. Modulus of elasticity = 29 x $10^6$ psi
f. Coefficient of friction = 0.16
g. Coefficient of wobble friction = 0.00030

Concrete

The assumed properties of the concrete used in the analysis were:

a. Compressive strength @ 28 days = 5000 psi

b. Modulus of elasticity = 4 x $10^6$ psi

c. Poissons ratio = 0.20

Liner

The liner is fabricated from 3/8 inch thick steel plate manufactured to ASTM A-283 Grade C. The minimum yield stress for this material is 30,000 psi. In determining the equivalent liner pressure LOCA, the yield stress was assumed to be 36,000 psi.

## 4.7 LOADING CASES AND COMBINATIONS

The buttress and wall were analyzed for four loading cases. The first two cases considered the buttress and wall being subjected to the maximum prestress forces, i.e., prestress forces allowing only those losses that would occur between prestressing and start-up (beginning of plant life.) The latter two cases considered the buttress and wall being subjected to the minimum prestress forces, i.e., prestress forces after 40 year losses (end of plant life).

The load cases were:

a. Winter normal operation with maximum prestress force.

b. Winter LOCA with maximum prestress force.

c. Winter normal operation with minimum prestress force.

d. Winter LOCA with minimum prestress force.

These cases were considered to be more critical than the summer normal operation/LOCA conditions. The summer condition results in a smaller temperature differential in the wall which in turn results in maximum compressive and tensile stresses less than the winter condition. Hence, the summer condition was not considered in the analyses.

The numerical value of the applied loads was:

| LOAD CASE | a | b | c | d |
|---|---|---|---|---|
| $t_{LT}$ | 110 | 150 | 110 | 150 |
| $t_{LP}$ | 110 | 280 | 110 | 280 |
| $t_e$ | 20 | 20 | 20 | 20 |
| $p_a$ | 6.25 | 6.73 | 5.52 | 6.03 |
| $pt_{A-T}$ | 46.07 | 49.75 | 39.25 | 42.2 |
| $2p_{tT-B}$ | 92.13 | 99.50 | 78.5 | 84.4 |
| $p_i$ | 0.0 | 96.1 | 0.0 | 96.1 |

where:

$t_{LT}$ = Temperature (F) of liner for thermal gradient in wall.

$t_{LP}$ = Temperature (F) of liner for equivalent pressure of liner due to expansion during LOCA.

$t_e$ = External air temperature (F)

$p_a$ = Prestress force at bearing plate (kips/inch)

$pt_{A-T}$ = Radial prestress force from point A to tangent point T (lbs/inch)

$2p_{T-B}$ = Radial prestress force tangent point T to point B (lbs/inch)

$p_i$ = Equivalent internal pressure (LOCA pressure + liner expansion pressure lbs/inch)

Figure 5D-21 shows the application of these loads on the buttress and wall.

Load $p_a$ $p_t$, and $2p_t$ for Case a incorporate losses due to elastic shortening of the concrete, friction and creep, shrinkage, and steel relaxation losses that occur between prestressing and start-up. Case b prestress forces reflect increases from Case a, due to elastic elongation of the structure during LOCA. Thus, Case a and b reflect the maximum prestress force that will exist in the structure, at the earliest time in the life of the plant, that a LOCA could occur.

Loads $p_a$, $p_t$, and $2_{pt}$ for Cases c and d are similar to Cases a and be respectively, except that the losses due to creep, shrinkage, and steel relaxation are those that are assumed to occur over 40 years to end of plant life. Case d prestress forces reflect an increase from Case c due to elastic elongation of the structure during LOCA.

## 4.8 STRESS ANALYSIS

Due to the symmetrical nature of the structure only one-twelfth of the cylinder, as shown in Figure 5D-16, need be analyzed. Preliminary analyses showed that beyond a certain point in the wall, the stress state, for all loading cases, was uniform. To conserve computer storage and give a more accurate solution, it was decided to reduce the size of the model to that shown in Figure 5D-22. Nodal displacements of both boundaries were specified. The displacements were obtained form a computer program as developed by Kalnins (Reference 16) using a shell of revolution theory.

There were a total of 672 elements and 384 nodal points, so that 768 simultaneous equations required solution. For the computations, the Gauss-Seidel iterative solution procedure was used.

## 4.9 ANALYSIS OF RESULTS

Figures 5D-23 through 5D-30 show iso-stress curves for the maximum and minimum principal stresses for the four loading conditions outlined in Section 4.7. It should be noted that these curves represent the algebraic maximum and minimum values of the principal stresses. The do not show the orientation of the stress trajectories.

Inspection of the iso-stress curves shows that, as expected, high compressive stresses exist directly beneath the bearing plate for all load conditions. Under winter normal conditions, a region of tensile bursting stress occurs in the buttress at some distance removed from the bearing plate; during LOCA, however, these bursting stresses merge with those resulting from the pressure and temperature loads.

The variation of temperature through both the wall and buttress leads to substantial hoop stress gradients. It should be noted that in the curves for LOCA, the direction of the stresses in the wall changes in the vicinity of the centerline.

Of primary interest, however, is the high localized tensile stress that occurs at the buttress-wall re-entrant corner during LOCA, which results from both the thermal gradient as well as the local bending arising from the variation of resistance of the buttress and wall to internal pressure.

## 4.10 CONCLUSIONS

The results indicate that for the most severe loading conditions, the maximum stresses in the buttress and adjacent wall are within safe limits, except for the localized area at the buttress wall corner where tensile stresses greater than $6 ?f_?$ occur. Reinforcement to stabilize cracking in this local region are provided. All other results would indicate that the design method employed was conservative.

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TABLE 5D-1
BUTTRESS ANCHOR ZONE
TRANSVERSE STRESSES IN RADIAL AND VERTICAL PLANES

| Case | Radial Plane | | | |
|---|---|---|---|---|
| | 1a | 1b | 1c | 2a |
| 2a'(in) | 20.5 | 20.5 | 20.5 | 20.5 |
| 2a (in) | 52 | 26 | 78 | 32 |
| 2a'/2a | 0.40 | 0.79 | 0.26 | 0.64 |
| fy[1] | 0.27 | 0.10 | 0.34 | 0.15 |
| z (in) [2] | 0.80 | 0.90 | 0.70 | 0.9 |
| 2b' (in) | 20.5 | 20.5 | 20.5 | 20.5 |
| 2b=2b'+(2az)tg30$^o$ | 44.5 | 33.5 | 51.5 | 37.0 |
| A=2a2b (in$^2$) | 2315 | 870 | 4150 | 1182 |
| P (kips) [3] | 1400 | 1400 | 1400 | 1400 |
| $f_{bz}$ = P/A (ksi) | 0.605 | 1.610 | 0.338 | 1.182 |
| $f_{by}$ = $f_{bz}$ fy (ksi) [4] | 0.164 | 0.161 | 0.115 | 0.178 |
| Zy = 0.3P (1-a'/a) (kips) [5] | 252 | 88 | 311 | 151 |
| $Zy_{(design)}$ = 0.3P (kips) [6] | 420 | 420 | 420 | 420 |
| az (in) | 20.8 | 11.7 | 27.3 | 14.4 |

[1] Guyon, "Prestressed Concrete", Figure 9.[2]
[2] Guyon, "Prestressed Concrete", Figure 9.[2]
[3] 1600 Kips and 2000 kips anchor forces is not critical.
[4] Changes in tensile splitting stresses with varying "2a" dimension.
[5] Zy = total transverse force Leonhardt "Prestressed Concrete" p. 271.[3]
[6] For design purpose "(a'/a)" is conservatively assumed to be equal to zero.

TABLE 5D-1 (continued)
BUTTRESS ANCHOR ZONE
TRANSVERSE STRESSES IN RADIAL AND VERTICAL PLANES

| Case | Ultimate Load- Radial Plane | | Vertical Planes | |
|---|---|---|---|---|
| | 2a | 3a | 3b | 3c |
| 2a' (in) | 20.5 | 20.5 | 20.5 | 20.5 |
| 2a (in) | 32 | 33 | 26 | 78 |
| 2a'/2a | 0.64 | | | |
| fy [1] | 0.15 | | | |
| z (in) [2] | 0.9 | | | |
| 2b' (in) | 20.5 | | | |
| $2b=2b' + (2az)tg30^0$ | 37.0 | See Case 2a, 1b, and 1c | | |
| $A=2a2b$ ($in^2$) | 1182 | | | |
| P (kips) [3] | 2000 | | | |
| $f_{bz}$ = P/A (ksi) | 1.690 | | | |
| $f_{by} = f_{bz}$ fy (ksi) [4] | 0.254 | | | |
| Zy=0.3P (1-a'/a) (kips) [5] | 216 | | | |
| $Zy_{(design)}$=0.3P (kips) [6] | 600 | | | |
| az (in) | 14.4 | | | |

[1] Guyon, "Prestressed Concrete", Figure 9.[2]
[2] Guyon, "Prestressed Concrete", Figure 9.[2]
[3] 1600 Kips and 2000 kips anchor forces is not critical.
[4] Changes in tensile splitting stresses with varying "2a" dimension.
[5] Zy = total transverse force Leonhardt "Prestressed Concrete" p. 271.[3]
[6] For design purpose "(a'/a)" is conservatively assumed to be equal to zero.

TABLE 5D-2
BUTTRESS ANCHOR ZONE
SPALLING
(Refer to Figure 5D-13)

$$M = P \{c - e - (c/h)^2 [2h - 3e - c + \frac{2ec}{h}]\}$$

c= varies    e = 15.5 in.    h = 69 in.

$$M = P \{c - 15.5 - (c/69)^2 [2 \times 69 - 3 \times 15.5 - c ( 1 - \frac{2 \times 15.5}{69})]\}$$

| C (in.) | c - 15.5 | $(c/69)^2$ | $-c( 1 - \frac{2 \times 15.5}{69})$ | M/P | $M_m$ (in. - K) |
|---|---|---|---|---|---|
| 5 | -10.5 | 0.00525 | - 2.75 | -11.0 | |
| 10 | - 5.5 | 0.02100 | - 5.50 | - 7.3 | |
| 15.5 | 0 | 0.05040 | - 8.50 | - 4.4 | |
| 20 | + 4.5 | 0.08400 | -11.00 | - 2.3 | |
| 25 | + 9.5 | 0.1310 | -13.80 | -0.7 | zero at 27 in. |
| 30 | +14.5 | 0.1890 | -16.50 | + 0.3 | |
| 42 | +26.5 | 0.370 | -23.10 | + 1.2 | 1680 |
| 50 | +34.5 | 0.525 | -27.50 | + 0.9 | (in. - kips) |
| 69 | +53.5 | 1.00 | -38.00 | 0 | |

$F_t = \frac{M_m}{h-z}$    Z = 3 in    h = 69 in.

$f_s$ allowable $= 10^5 \sqrt{\frac{W}{A_s}}$

$A_s$ = 1.27 sq in. (#10 bars)

W = 0.005

$f_s = 10^5 \sqrt{0.005/1.27} = 6.28$ ksi

$F_T = \frac{1680}{69 - 3} = 25.5^K$

$A_s = F_T/f_s = 25.5/6.28 = 4.05$ sq in.

$A_s$ provided: #10 @ 12 in. + 1 #10 Loop = 6.03 sq in.