APPENDIX 5C

GILBERT ASSOCIATES, INC., REPORT

TO

METROPOLITAN EDISON COMPANY

ON

DESIGN OF LARGE OPENING REINFORCEMENTS

FOR REACTOR BUILDING

FOR

THREE MILE ISLAND NUCLEAR STATION
UNIT 1

This Report Contains:

44 pages of text
8 tables
29 figures
2 drawings

Appendix A (8 pages)
Appendix B (3 pages)

DESIGN OF

LARGE OPENING REINFORCEMENTS

FOR REACTOR BUILDING

METROPOLITAN EDISON COMPANY

THREE MILE ISLAND NUCLEAR POWER STATION - UNIT NO. 1

BY

GILBERT ASSOCIATES, INC.

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SUMMARY

DESIGN BASES

The Reactor Building large openings, consisting of a 22 feet 4 inches diameter opening (equipment access hatch) and a 10 feet 0 inches diameter opening (personnel lock), are designed generally in accordance with the criteria as detailed in The Three Mile Island Nuclear Station Unit 1 Safety Analysis Report. The steel reinforcements used in the vicinity of the openings have a minimum yield stress of 40,000 psi.

GENERAL DESCRIPTION

The vertical and horizontal prestressing tendons are draped around the holes and are continuous (i.e., no tendons are terminated at the hole). Circular rebar rings are located around the hole as principal reinforcement. Horizontal and verical rebars are provided to the extent required by calculated radial (to the hole) stresses. Normal shear forces, due to pressure on the hatch, at the interface between penetration barrel and concrete are resisted by steel shear plates.

STRESS DISTRIBUTION AROUND A CIRCULAR HOLE IN A CIRCULAR CYLINDRICAL SHELL

A survey was made of the available theoretical solutions and experimental techniques for determining stress distributions around circular holes in a shell structure. This survey indicated that an available programmed finite-element solution was most practical for this application.

ANALYSIS OF STRESSES AROUND LARGE OPENINGS

A. Method of Analysis
The adequacy of the design of the openings was verified by the use of Computer Program FELAP (Reference 12) developed at the Franklin Institute Research Laboratories (FIRL). This program provides for the representation of the shell by flat rectangular panels with multiple layers and is used on the assumption that the perturbation on the shell introduced by the presence of the opening is local. The basis for the derivation of this program is described in Reference 13.

B. Verification of Method Accuracy
In order to evaluate the accuracy of the solution method, a test problem was solved to develop stresses which could be directly compared with other theoretical solutions and experimental results. Very satisfactory results were obtained as evidenced by the data presented in Section 4.1. This study also verified the adequacy of the grids used on the analysis of the openings for the Three Mile Island - Unit No. 1 Reactor Building.

C. Basis for Analytical Model
For purposes of the analysis, the shell was idealized by representing (1) the liner as an isotropic steel layer, (2) the horizontal or vertical reinforcement as an orthotropic layer with no Poisson's ratio effect and no shear stiffness, and (3) the circular ring reinforcement with zero shear stiffness. A more complete description of the model idealization is contained in Section 4.2.1.

D. Load Combinations

The basic loads, including dead, internal pressure, earthquake, prestress, and thermal (operating and accident) loads, were combined in accordance with the basic criteria and more fully tabulated as shown in Table 1. All loading conditions are more fully tabulated as shown in Table 1. All loading conditions are more fully described in Section 4.2.2.

E. Creep and Shrinkage Effects

The effect of load redistribution due to concrete shrinkage and creep was investigated as described in Section 4.2.3 and found to be negligible.

F. Summary of Results

The stress-resultants and stress-couples for panels which were of interest in evaluating the adequacy of the design are summarized in Tables 2 through 8. The correctness, of the values computed by the finite-element method was verified in several ways, as described in Section 4.1 (c).

VERIFICATION OF REINFORCEMENT ADEQUACY

A. General Method

Principal stress-resultants and stress-couples were computed for all panels. Interaction diagrams were prepared based upon procedures for ultimate strength design of ACI 318-63. Interaction diagrams for those panels found to be significant

are included as Figures 20 through 28, indicating the stress state for all pertinent loading combinations. The interaction diagrams show that sufficient reinforcement has been provided to carry all loads, including the full thermal stress-resultants and stress-couples.

B. Additional Considerations

Additional studies were performed to evaluate the acceptability of the penetration barrel and reinforcement for normal shears, which are described in Sections 5.4 and 5.5.

C. Design Drawings

Design drawings providing details of the opening reinforcement are included at the end of this report.

1 DESIGN BASES

1.1 General

The large openings in the Reactor Building are designed generally in accordance with the criteria as detailed in the Safety Analysis Report, and as more completely detailed hereafter. The large openings consist of the opening for the equipment access hatch, with a diameter of 22 feet 4 inches, and the opening for the personnel lock, with a diameter of 10 feet 0 inches Although these criteria and the analytical and design methods described hereafter apply to both openings, detailed results are provided only for the larger opening.

1.2 Design Loads

The following loads were considered in the structural design of the Containment Vessel:

a. Internal Pressure-Design Pressure is 55 psig
b. Test Pressure - 63.3 psig (1.15 times design pressure)
c. Live Loads
d. External pressure - 2.5 psig
e. Wind Load
f. Thermal Loads

    1. Accident
    2. Operating
    3. Test

g. Seismic Ground Accelerations
h. Dead Load
i. Prestressing Load
j. Tornado Load

The thermal loads on the Reactor Building and their variation with time, are developed on the basis of the blow-down transients as detailed in the Safety Analysis Report. The openings, as a portion of a Class I structure, are designed on the basis of a horizontal ground acceleration of 0.06g as the design earthquake, and of 0.12g as the maximum hypothetical earthquake. The vertical component is taken as 2/3 of the horizontal component.

The equipment access hatch consists of a single door located outboard of the Reactor Building shell with a personnel lock inset in the door. The barrel (penetration sleeve) for the hatch is consequently subjected to the accident pressure. The isolated personnel lock consists of double doors, one located inboard and the other outboard of the Reactor Building shell. The design considers the consequence of both doors being closed during the accident as well as the extremely remote possibility that either one of the doors is opened during the accident.

1.3 Load Combinations

The design is based upon limiting load factors which are used as the ratio by which accident, earthquake, and wind loads are multiplied for design purposes to ensure that the

load deformation behavior is essentially elastic. The loads utilized to determine the required limiting capacity of any structural element are computed as follows:

a. $C=0.95D + 1.5P + 1.OT$
b. $C=0.95D + 1.25P + 1.OT' + 1.25E$
c. $C=0.95D + 1.OP + 1.O\underline{T} + 1.OE'$
d. $C=0.95D + 1.OW_t + 1.OP_t$

Symbols used in the above equations are defined as follows:

C: Required load capacity of section
D: Deal load of structure
P: Accident pressure load - 55 psig
T: Thermal loads based upon temperature transient associated with 1.5 times accident pressure.
T': Thermal loads based upon temperature transient associated with 1.25 times accident pressure.
$\underline{T}$: Thermal loads based upon temperature transient associated with accident pressure.
E: Seismic load based upon 0.06g ground acceleration.
E': Seismic load based upon 0.12g ground acceleration.
W_t: Wind loads based on 300 mph tornado.
P_t: Pressure load based on an internal pressure of 3 psig difference between inside and outside of the Reactor Building.

The acceleration response spectra and structural damping are as detailed in the Safety Analysis Report.

1.4 Material Stress/Strain Criteria

a. Concrete Reinforcement
The deformed bars used for concrete reinforcement in the Reactor Building, are intermediate grade billet steel conforming to ASTM A615-68 Grade 40 with a guaranteed minimum yield strength of 40,000 psi.

The design limit for tension members (i.e., the capacity required for factored loads) is based upon the yield stress of the reinforcing steel. No mild steel reinforcement is designed to experience strains beyond the yield point at the factored loads. The load capacity so determined is reduced by a capacity reduction factor "f" which provides for the possibility that small adverse variation in material strengths, workmanship, dimensions, and control, while individually within required tolerances and the limits of good practice, occasionally may combine to result in an under-capacity. The coefficient "f" is 0.95 for tension, 0.90 for flexure, and 0.85 for diagonal tension, bond and anchorage.

b. Prestressing Tendons

The steel tendons for prestressing consist of 169 - 1/4 in. diameter wires using a BBRV anchorage system and high tensile, bright, cold drawn, and stress-relieved steel wires conforming to ASTM A421-59T, Type BA, "Specifications for Uncoated

Stress-Relieved Wire for Prestressed Concrete" with a minimum ultimate tensile stress of 240,000 psi.

The steel tendons are stressed during the post-tensioning operation to a maximum of 80 percent of ultimate strength and locked-off for an initial stress of 70 percent of ultimate strength. The maximum effective prestress is determined taking into consideration allowances for the following losses, which are deducted from the transfer prestress:

1. Elastic shortening of concrete
2. Creep of concrete
3. Shrinkage of Concrete
4. Steel relaxation
5. Frictional loss due to intended or unintended curvature of the tendons

In no event does the effective prestress exceed 60 percent of the ultimate strength of the prestressing steel, or 80 percent of the nominal yield point stress of the prestressing steel, whichever is smaller. The design is based upon the steel tendons not being stressed beyond the yield point as defined by ACI 318-63 when subjected to the factored loads.

c. Structural Concrete

The structural concrete will have a minimum compressive strength of 5,000 psi in 28 days. Under operating conditions the allowable concrete stresses are in accordance with

ACI 318-63 Part IV-A, "Structural Analysis and Proportioning of Members-Working Stress Design," and Part V, Chapter 26, "Prestressed Concrete."

The Reactor Building, including the large openings, is checked for the factored load combinations and compared with what is generally defined as the yield strength of the structure. For concrete, the yield strength is defined, except as described hereafter by the allowable stresses in ACI 318-63, Part IV-B, "Structural Analysis and Proportioning of Members - Ultimate Strength Design." Concrete cracking is assumed when when the principal tensile stress based upon all loads including thermal loads, exceeds $6\ddot{O}f_C$ or 424 psi.

d. Liner

The liner was designed as participating with the concrete shell in carrying membrane forces (See 5.1). The stress limits established for the liner are consistent with those limits for stress intensity (i.e., the difference between the algebraically largest principal stress and the algebraically smallest principal stress at a given point) defined in Section III of the ASME Boiler and Pressure Vessel Code and based upon a working strength design consistent with that code.

The liner is carbon steel plate conforming to ASTM A283-67, grade C. This material has a minimum yield stress of 30,000 psi and a minimum ultimate tensile strength of 55,000 psi.

The liner is normally 3/8 inch thick in the cylindrical portion of the structure. In the immediate vicinity of the large opening it is increased to a 1 1/8 inch thick reinforcing plate.

The barrel and reinforcing plate are carbon steel conforming to ASTM A516 Grade 60 Fire box Quality modified to ASTM A300. This material has a minimum yield stress of 32,000 psi and a minimum ultimate tensile strength of 60,000 psi.

1.5 Test Condition

No specific stress or strain limits are established for the test condition. The factored load combinations previously described have been established so as to ensure that the response due to design loads is essentially elastic. A check is made to ensure that no significant permanent deformation of the structure occurs during the test. This means that following the test there should be no visible permanent distortion of the liner and only small hairline cracks should exist in the concrete.

1.6 Operating Condition

The load combinations relevant to operating conditions are determined as reflected in Section 4.2.2. Allowable stresses are based upon those stipulated in ACI 318-63.

2 GENERAL DESCRIPTION OF OPENING REINFORCEMENT

2.1 Introduction

The normal flow of stresses in the hoop and meridional direction is obstructed by the large access holes in the Reactor Building. During normal operation of the plant, horizontal and vertical prestress forces are the single largest stress contributor. These forces do not create any difficulty in transferring horizontal and vertical compressive stresses around the opening, but give the necessary reserve compressive stresses to counterbalance the horizontal and vertical strains resulting from the accident pressure load. The horizontal and vertical stresses in the reinforcing steel are very small during normal operation, but will theoretically increase up to almost yield stress due to the factored pressure load. These hoop and meridional forces under pressure load will be transferred around the opening by counter acting prestress forces and by hoop, meridional, and circular ring reinforcing steel.

2.2 Ring Reinforcement Splices

The circular ring reinforcing steel will be spliced by use of a Cadweld Rebar Splice. These splices are located at points of low rebar stresses, and no more than 1/3 of the ring steel will be spliced in one section. This should eliminate any slip between Cadweld and rebar that could occur at high rebar stresses.

Radial tensile stresses out from the center of the access opening are created due to the pressure load. Hoop and meridional reinforcement is provided to carry these stresses out from the opening and will be terminated where "pure" membrane stresses exist in the wall.

2.3 Normal Shear at Edge of Opening

The peripheral or normal shear reinforcement in the concrete around the Penetration Barrel is designed for the computed shear at a distance equal to or greater than, d/2 = 42.0 in., from the edge of the opening per ACI 318-63, or for twice the normal shear due to internal pressure on the hatch, whichever is the larger of the two valves.

2.4 Prestressing

The maximum curvature for any horizontal or vertical tendon, around an opening, is at a radius of not less than 25.0 feet. When tendons are draped around penetrations in a manner in which a radial force is exerted on the penetration, within the diameter of the penetration, the spacing of the tendons are not less than 10 inches. The distance between the penetration and the centerline of the first tendon for the equipment access and personnel access penetrations is not less than 12 inches. The distance between tendons when two layers are required is not less than 18 inches. The total angular change of the tendons was laid out to keep frictional losses within a satisfactory margin, and to satisfy a practical execution of the job in the field.

3 STRESS DISTRIBUTION AROUND A CIRCULAR HOLE IN A CIRCULAR CYLINDRICAL SHELL

3.1 Introduction

The first theoretical treatment of the problem presented by openings in thin shell structures is commonly attributed to Lur'e[1], who obtained an approximate solution for the stress distribution around a very small circular cut-out in a thin circular cylindrical shell subjected to a homogeneous biaxial stress state [x'[P. Lur'e derived the following expression for the edge stress:

Equation 3.1

[= ([x + [P) - 2([x + [P) cos2f

+ Ö3(1 - v²) X d² [2[x + ([P - 3[x) cos2f]
16Rt

in which:

[x = meridional stress in the shell without the opening
[P = hoop stress in the shell without the opening
f = angular coordinate (see Figure 2)
R = shell radius
t = shell thickness
d = diameter of cut-out = 2r

Equation (3.1) does not include bending effects, which can be quite important in thin shells. Therefore, Lur'e's solution represented a relatively minor improvement on the flat plate solution (Reference 2). In fact, for many years the design of reinforcement around shell openings was based on the classical stress concentration factors of flat plate theory. In this area solutions for openings reinforced by means of a symmetrical circular doubler plate were obtained by Sezawa and Kubo (Reference 3), Gurney (Reference 4), and Beskin (Reference 5). Solutions for the stress distribution around unreinforced and ring-reinforced holes in flat plates can be found in Savin's (Reference 6) extensive treatment of the subject.

The plane stress approach is not satisfactory, however, for large openings. In this case large stress-couples may appear around the edge of the opening even when the shell elsewhere is in a pure membrane state of stress. Fortunately, very valuable results have recently

become available. Withum (Reference 7) investigated the stress distribution in a cylinder weakened by a hole, subjected to torsion, by using a perturbation scheme. This technique was extended by Kline 'et al'(Reference 8) to determine the stresses around a circular cut-out in a pressurized circular cylindrical vessel. This work, carried out at the General Technology Corporation with the support of the Bureau of Ships of the U. S. Navy, was part of a systematic theoretical investigation of two problems: (References 9 and 10).

a. determination of the state of stress in the vicinity of a circular hole on a circular cylindrical shell subject to internal pressure.

b. determination of the stress distribution in two normally intersecting cylindrical shells.

The solution of problem (a), for internal pressure as well as other practical loading conditions, has been presented by Naghdi and Eringen (reference 10). Lekkerkerker (Reference 11), extending Lur'e's approach, solved the same problem for axial tension and torsion. He found excellent correlation between his solution and experimentally determined stresses (using electric resistance strain gages) in a mild steel tube subject to torsion. Stress concentration factors from References 8 to 11 are presentated in Figures 1 and 2. The coefficients given may be directly applied to the calculation of maximum stresses at the edge of the opening.

Unfortunately, no theoretical solution is available for reinforced openings or for non-isotropic shells (e.g., orthotropic reinforced concrete shells). The complexity of these problems is such that they can only be dealt with by means of numerical or experimental methods. Among the many possible approaches, the finite-element method and the stress-freezing technique of three-dimensional photoelasticity have become especially attractive in recent years and are briefly evaluated in Sections 3.2 and 3.3.

3.2 Finite-Element Method

Steady progress in the finite-element approach has led to the possibility of determining the stress distribution around reinforced openings in shell structures with good accuracy. The most important advantage of this method is its generality: reinforcing rings, variable shell thickness, and material orthotropicity, for example, may be incorporated into the analysis without difficulty. Unfortunately, the accuracy of the solution must still be evaluated by test: For instance, results obtained using two different grids may provide an indication of covergence. Alternatively, results obtained using a prescribed grid size and pattern may be checked against some of the theoretical solutions of Section 3.1 or against experimental values. The latter approach was followed in connection with the analysis of the stresses around the openings for the Three Mile Island Reactor Building.

The designs of the reinforcement around the openings in the Three Mile Island Reactor Building were verified by using Computer Program FELAP (Reference 12), developed at Franklin Institute Research Laboratories (FIRL). The solution is based on a representation of the shell as an assembly of flat rectangular panels. In the first order shell theory described by Zudans (Reference 13), which formed the basis for the FIRL finite-element solution, the rotation about the normal to the shell middle surface is taken equal to zero. Consequently, this leaves only five degrees-of-freedom associated with each nodal point. It must be noted that the model with five degrees-of-freedom at a corner point, while "compatible" for plate problems, is unbalanced for shell problems in the third rotation (Reference 14). Although usually this unbalance does not affect the accuracy of the

solution, it can lead to unrealistic results (Reference 14). Therefore, in applications to nuclear power plants, a verification of the results becomes desirable.

Connor and Brebbia (Reference 15) developed a stiffness matrix for a thin shell element of rectangular plan and also noted that good results can be obtained with the finite-element method using non-compatible displacement expansions, which do not include all the rigid body displacements (Reference 15). According to Connor and Brebbia (Reference 15), it appears that curved elements lead to better results, for the same element size and shape, and are therefore more efficient than flat elements. However, comparison of results obtained using two different types of curved triangular elements (Refences 16 and 17) with flat elements (References 12 and 14) for the test problem discussed in Section 4.1 did not substantiate that belief (See Figure 10). In fact, all finite-element results were close to Eringen, Naghdi, and Thiel's (Reference 9) solution, which can be considered, within the limitations of shallow shell theory, an "exact" solution.

3.3 Applications of Three-Dimensional Photoelasticity

Until very recently, experimental methods constituted the only feasible approach to the stress analysis of geometrically complicated reinforced openings in shell structures. In this area, the stress-freezing technique of three-dimensional photoelasticity appears to be the most suitable experimental method. Outstanding studies of stresses around reinforced openings in pressure vessels were carried out by Levan (Reference 18), Taylor and Lind (Reference 19), and Takahashi and Mark (Reference 20). In the latter, comparison of the photoelastic results with a finite-element analysis of the axisymmetric thick-walled reinforced concrete vessel showed very close agreement between the two solutions. Durelli, del Rio, Parks, and Feng (Reference 21) carried out an experimental evaluation of the stress around an opening in a thin shell by means of brittle coatings, electrical and mechanical strain gages, micromenters, and photoelasticity with the objective of comparing the accuracy and advantages of each technique.

Photoelasticity was concluded to be the most effective experimental approach in this type of problem (Reference 21). In the fabrication of the model Durelli 'et al' (Reference 21) used a Hysol 4290 epoxy resin which was found to give poor performance in recent tests (Reference 22). Mark and Riera (Reference 22) believe that the use of the improved model materials will lead to a considerable reduction in the dispersion of photoelastic readings, which was large in the past (Reference 23), and which still is the most important argument against this experimental approach. In spite of the difficulties associated with the model material, the photoelasticity determined in Reference (21) shows good correlation with Eringen and Naghdi's (Reference 9) theoretical solution. Stress concentration factors determined photoelastically by Durelli 'et al' and by Houghton and A. Rothwell (Reference 24) by means of electric resistance strain gages mounted on circular cylindrical aluminum shells are given in Figure 3(c). Figure 3(c) also shows the stress concentration factors computed by Eringen and Naghdi (Reference 9).

The photoelastic approach presents several advantages (Reference 25): 1) full field observations give clear understanding of overall behavior and permits recognition of unsuspected critical regions, 2) measurements are made with very small effective gage lengths so that high gradients can be studied in small models, and 3) basic instrumentation is simple and inexpensive. Traditionally, models have been machine finished, but improving casting techniques have already permitted the fabrication of complicated models without any noticeable edge effect. This represents two important steps forward: 1) model fabrication

cost can be drastically reduced and 2) the technique may now be applied to complicated models that cannot be readily machined.

Ducts for prestressing "tendons" have successfully been incorporated into epoxy models by using nylon piano cords, which are set in the model prior to casting as if they were rebars in a conventional reinforced concrete element (Reference 26). After the epoxy has hardened they can be easily pulled out, leaving a perfect duct without residual stresses around the walls. The photoelastic method, therefore, offers the possibility of determining in one single study the stresses around openings in prestressed vessels in "the large," as well as in "the small." The latter, which includes stresses around curved tendons, corners, possible non-linear distributions through the wall thickness, etc., may demand separate analyses when a solution based on thin shell theory is used as the basis for design.

Further progress in theoretical or numerical analysis of stresses around shell openings, which must account for non-linear distributions through the shell thickness near the opening and for other effects that cannot be predicted by thin-shell theory, can be fostered by adequate experimental verification of the results, or by purely experimental studies that may help define the areas that require additional investigation. This is illustrated by a photoelastic investigation of stresses around reinforced openings in plates due to Lerchenthal (Reference 27), which indicates that the departure from a plane stress distribution near the openings may be significant. For this purpose, the stress-freezing technique may yet prove to be an invaluable tool.

4. ANALYSIS OF THE STRESSES AROUND LARGE OPENINGS IN THE THREE-MILE ISLAND UNIT NO. 1 REACTOR BUILDING

4.1 Verification of Finite-Element Method of Analysis as outlined in Section 3.1, a finite-element solution was chosen to determine stresses around large openings in the Reactor Building. To evaluate the accuracy of the solution method (FIRL Program FELAP, as described in Section 3.2), a test problem was solved with the same grid used in the finite-element analysis of openings for the Three Mile Island Reactor Building. The grid is shown in Figure 4. In the finite-element analysis the following assumptions were made:

a. The perturbation introduced by the presence of the opening on the shell state of stress is localized.

b. According to (a.), stresses and displacements some distance away from the region surrounding the opening are not affected by the opening.

c. A panel of rectangular plan, which is centered around the opening, is removed from the shell (Figure 4). The displacements corresponding to the shell without the opening along the boundary lines of this panel constitute the boundary conditions for the finite-element analysis. The analysis is correct if the panel boundaries are sufficiently far from the opening so that (a.) and (b.) apply. This assumption can be verified 'a posteriori' by comparing stresses along the boundary lines with those existing in the shell without the opening.

d. Each element can be idealized as a layered plate. Three typical elements, each with four layers, for representation of three regions of different thickness of the finite element model, as shown in Figure 5.

e. Because of symmetry, only one quadrant need be analyzed.

Since reliable experimental or theoretical results for reinforced openings in shells similar to that under consideration were not available, it was decided to check the solution method against the shell tested by Durelli 'et al' (Reference 21), for which other theoretical solutions were also available. The example problem is defined by:

$R = 430.00$ in.
$r = 85.70$ in.
$t = 18.04$ in.
$v = 0.30$
$p = 100.00$ psi

The computed tangential membrane stresses around the opening edge are compared in Figure 6 with those given in Reference (9) and with the experimental stresses determined by Durelli 'et al' (Reference 21). Similarly, Figure 7 shows the tangential surface stresses around the edge. Additional comparisons between the finite-element solution and the experimental values are given in Figures 8 and 9. Finally, Figure 10 shows the variation along the symmetry axes of the stress resultant N_P determined by different approaches, including another finite-element solution using triangular shell elements based on Prato's work (Reference 16). The correlation of the FIRL finite-element solution with the other results is good. It must be noted that the finite-element results are particularly close to the

solution of Eringen 'et al' (Reference 9), which was regarded as the most accurate. It should also be pointed out that the correlation is good in spite of the fact that the panel boundaries were not sufficiently removed from the opening, as revealed by Figure 9 which shows the existence of a small stress couple M_P at $\frac{s}{r}$ = 3.5 that had not been predicted by the model study. This result was verified by another finite-element analysis of the test problem using Prato's triangular shell elements (Reference 16).

It was pointed out in Section 3.1 that the stress concentration factors under loading conditions such as internal pressure, axial tension or torsion can be computed at different locations in terms of the nondimensional parameter T (see Figure 2). As T ® 8 we approach the plane stress solution and the convergence problem for the finite-element solution disappears. It can be concluded, therefore, that satisfactory results in the test problem (T = 1.17) constitute adequate verification of the solution method for the large openings in the Three Mile Island Reactor Building for which T = 62 (equipment hatch, based on typical shell thickness).

4.2 Stress Analysis Under Operating and Accident Loads

4.2.1 Representation of the Shell Around the Opening

The finite-element grid used to solve the test problem (Section 4.1) was also employed to determine the stress distribution around the large openings in the Reactor Building. The shell was idealized as follows:

a. The liner was represented as an isotropic steel layer (E_{st} = 30,000 ksi, U = 0.3). Composite action was assumed in the determination of the stress resultants and stress couples.

b. The vertical and horizontal steel reinforcement was represented as a layer of an orthotropic material having no Poisson's ratio effect, no shear stiffness (G_{12} = 8), and no meridional stiffness(E_1 = 8). This layer was located at the center of gravity of the meridional or hoop reinforcement.

c. The ring steel rebars, providing additional reinforcement around the opening, were represented as a layer located at the center of gravity of the ring reinforcement. The shear stiffness of this layer was set to equal zero (G_{12} = 8). In Computer Program FELAP the axes of orthotropicity are oriented in the hoop and meridional directions. Consequently, since the axes of orthotropicity of the ring reinforcement only coincide with those directions in the regions around the opening's vertical and horizontal axes, the following approximation was made: at every panel type the hoop and meridional stiffnesses were directly proportioned to the projected steel area.

d. The concrete layers were idealized as follows:

E_1 = 4000 ksi
E_2 = 4000 ksi
G_{12} = 1740 ksi
U_{12} = 0.15

e. The barrel (penetration sleeve) was represented on contributing, with 50 percent of its area, to the stiffness of the elements adjacent to the opening. In other words, a stiffener of 3/8 inch thickness was included in the model along the periphery of the opening.

Note that although the penetration barrel was incorporated into the finite-element model as explained above, it was not regarded as contributing to the load-carrying capacity of the shell (See Section 5.2).

4.2.2 Basic Loading Conditions

The stress distribution around the opening was determined for the loading conditions described in Section 1 and those loading combinations more completely described hereafter. The specific loads are defined as follows:

a. Dead Load

The stress distribution around the opening due to dead weight was calculated assuming a uniform meridional compression in the cylinder equal to the stress resultant at the elevation of the opening axis (in the shell without the opening). The weights of the equipment hatch and personnel lock were neglected. Since dead weight stresses in the typical shell wall at the elevation of the equipment and personnel access are low (less than 100spsi), the above simplifications will not significantly influence the final stresses in the pressurized vessel.

b. Internal Pressure

Internal pressure was assumed to act on the interior of the shell as well as on the barrel (penetration sleeve) of the equipment access hatch. The internal pressure on the hatch was assumed transferred to the shell by a uniform normal shear Q_p around the opening. The effect of non-uniform distributions of Q_p were analyzed separately.

c. Earthquake

Seismic stresses around the openings were investigated for two directions of the horizontal component of motion:

1. Earthquake motion oriented in the direction normal to the openings.

2. Earthquake motion oriented at 900 with the direction normal to the openings.

Seismic loads were evaluated as described in Appendix B.

d. Prestress

Prestressing loads are represented as two independent loading conditions by a uniform hoop and meridional compression in the shell (away from the opening) equal to the stress resultant corresponding to initial and final prestress. Curved tendons around the opening were considered in the analysis as line loads P = T/r where T is the total prestressing force per tendon and r is the radius of curvature at the location under consideration. These line loads were integrated within each panel and applied as nodal forces on the shell, as shown in Figures 11 and 12.

e. Thermal Loads

Operating Temperature

The steady-state temperature distribution in the reinforced area around the opening was obtained using "Heating 2" a computed code developed by Mr. R. D. Brunell of Atomics International (Reference 28). This particular computer program is applicable for the solution of transient and/or steady state problems in one, two, or three dimension cartesian or cylindrical coordinate systems.

The steady state temperatures were determined using the formula:

$$\sum_{i=l}^{M} {}_{j} K_i (T_i - T_j) = 8$$

with convergence criterion

$$\frac{T_j \text{ step} - T_j \text{ step}^A - 1}{T_j \text{ step}^A} \text{ £ } .00001$$

for all nodes.

The transient temperatures were determined using the formula:

$$\sum_{i=l}^{M} {}_{j} K_i (T_i - T_j) = C_j \frac{\text{delta } T_j}{\text{delta } T}$$

to ensure stability of the transient solution the time step was chosen as:

$$\text{delta } T \text{ £ } \left(\frac{C_j}{\sum_{i=l}^{M} {}_{j} K_i} \right)$$

which was calculated for all nodes and the smallest value chosen.

The opening temperatures were determined for:

1. normal operation in winter

2. start-up in winter
3. shut-down in winter
4. accident condition in winter

The winter season was chosen because at this time the largest thermal gradients are present in the Reactor Building. The temperature on the external surface of the Reactor Building was taken as 21.4F. This is the lowest average monthly temperature ever recorded in the area by the United States Weather Bureau. Lower temperatures will be experienced for shorter periods of time, however these only affect a small portion of the wall thickness, as proved by preliminary tests, and were not considered.

It was assumed that during start-up, the normal operating temperature 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 .010354 Btu/hr in^{2}F for cases 1, 2, and 3 which is actually a modified film coefficient which takes into account the steel liner and the stagnant air gap between the liner and the concrete:

$$h_M = \frac{1}{\frac{1}{h_i} + \frac{X_{STEEL}}{K_{STEEL}} + \frac{X_{AIR}}{K_{AIR}}}$$

This does neglect any circumferential conduction, however, due to the small comparative thickness of the liner and air gap the effect will be negligible. This was also proven in preliminary tests.

In case 4 the liner temperature was known therefore as a film coefficient of .040766 Btu/hr in^2 F was used which simply simulated the conduction through the air gap:

$$h_M = \frac{k_{AIR}}{X_{AIR}}$$

The outside film coefficient in all cases was taken as .0069555 Btu/hr in^2 F.

The opening itself was broken into 61 nodes with adiabatic boundaries at the radial midpoint of the buttress and the radial midpoint of the plane wall. This was done because considering symmetry, there is no heat transfer across these boundaries.

The following material values were used for the concrete:

P = .08102 lb/in.^{3}
C_p = .210 Btu/lb F
k = .06944 Btu/hr in F.

The input for the finite-element stress analysis was prepared on the basis of the above results.

Summer thermal stresses are conservatively computed as equal to 27 percent of the peak winter stresses.

f. Accident Temperature

The effect on the barrel (penetration) resulting from an accident temperature of 281F (net increase: 281 - 110 = 171F) was representated by internal pressure acting on the barrel. The magnitude of this pressure was determined on the basis of a two dimensional analysis, which assumed the barrel (t = 0.75 in.) plus 0.55 in. of concrete were suddenly heated to 171 F. The equivalent pressure on the concrete was found to be equal to 183 psi.

Definitions for the foregoing variables are:

$j^K i$ = thermal conductivity between node j and i
T_i = temperature at node i
T_j = temperature at node J
M = number of nodes adjacent to node j
A = interation number
C_j = compositance of node j
T = time step
X = thickness
k = thermal conductivity
h_M = modified film coefficient, inside
h_i = inside film coefficient
h_8 = outside film coefficient
P = density
C_P = specific heat
P_1 = radial distance between boundaries
P_2 = radial distance opening covers

The temperature gradients obtained by the aforemention computer program are presented as 6 figures (Figures 13 through 18). Figures 13, 14, and 15 refer to the temperature gradients under winter operating temperature conditions at region near the opening, normal wall section, and the barrel-normal wall transition zone, respectively. Figures 16, 17, and 18 show the temperature gradients under winter accident temperature condition at region near the opening, normal wall section, and the barrel-normal wall transition zone, respectively.

4.2.3 Effect of Creep and Shrinkage

It appears that shrinkage of concrete can only introduce compressive stresses into the steel rebars. These stresses will largely disappear after the internal pressure in the shell takes place, and need not be considered in the stress analysis of the opening.

The load redistribution due to concrete creep (i.e., the redistribution of Nf , NfP, NP , Mf, MP, etc.) is expected to be small. This conclusion is sustained by a finite-element plane stress analysis, which indicates that the stress concentration factor for uniaxial compression changes by only 5 percent when the hoop 'in-plane' stiffness is reduced by 100 percent. If the 'effective modulus' approach is used to determine the final stress distribution under operating stresses, a final modulus equal to 65 percent of its initial value would lead to a final ratio between vertical and hoop 'in-plane' stiffness equal to:

$$\frac{42 \times 1600}{41.875 \times 1600 + .125 \times 30{,}000} = \frac{67.3}{70.65} = .950$$

the initial ratio is:

$$\frac{42 \times 4000}{41.875 \times 4000 + .125 \times 30{,}000} = \frac{168}{170.8} = .985$$

Therefore, a change in the ratio between vertical and hoop 'in-plane' stiffnesses of about 4 percent is not expected to have any significant effect on the stress-resultant and stress-couple distribution.

The effect of creep and shrinkage on prestress losses is taken into account as indicated in Section 5.8. Likewise, the transfer of load from concrete to steel in any given cross-section is considered in the verification of rebar stresses when applicable.

With reference to the discussion of Appendix A, it may be concluded that the load redistribution due to concrete creep in the unpressurized vessel will not affect in any significant degree the load distribution in the structure under test or accident pressure.

5. VERIFICATION OF DESIGN CRITERIA

5.1 Basis for Verification of Shell Loading Capacity Due to Primary Loads (Principal Stress-resultants and Principal Stress-couples)

The loading capacity at any point of the shell was verified according to the following procedure:

a. The principal stress-resultants N^1 and N^2 were computed in terms of Nf, NP, and NfP.

b. The principal stress-couples M^1 and M^2 were computed in terms of Mf, MP, and MfP.

c. Considering that throughout the critical regions of the shell (both axes of symmetry and along the edge of the opening) the orientations of N^1, N^2, and M^1, M^2 coincide with the orientation of the reinforcement and that in the rest of the shell they nearly coincide with each other and with the orientation of the steel rebars. Systems N^1, M^1, and N^2, M^2 are treated independently. Since in the regions in which the directions of principal stress-resultants and stress-couples are not colinear or do not coincide with the orientation of the steel rebars, stresses are low, the error introduced by assuming them colinear will not affect the conclusion concerning the load-carrying capacity of the shell. In the latter case, steel rebars not oriented in the direction of principal stress-resultants or stress-couples were conservatively neglected in the computation of the strength of the section. In other words, it is proposed that forces in the 1-direction will not affect the strength of the section in the 2 direction and vice versa. This is in full agreement with a square failure criterion for concrete in biaxial compression, which appears to be very conservative. Stresses in rebars oriented along a principal stress direction obviously will not be affected by forces in the other principal direction.

d. The computed ultimate capacity of any section of the shell satisfies the requirements of ACI 318-63, Sections 1600, 1700, 1800, and 1900. Only deformed bars as defined in ACI 318-63, Sections 301 are used.

e. Composite action between the shell and the liner is neglected in the computation of ultimate moments. The liner is regarded as carrying only its share of the principal stress-resultants.

Interaction diagrams were prepared as described in Section 5.2 for elements located along both symmetry axes and along a 45 degree line. Principal stress-resultants and stress-couples corresponding to all critical load combinations are shown in Figures 20 thru 28 on the interaction diagrams corresponding to elements 44, 55, 66, 77, 73, 74, 99, 100, and 101. The position of a point representing a stress state within the diagram gives a clear indication of the "local safety factor" at that location (i.e., at the center of the element). It should be pointed out that all points on the interaction diagram are computed with respect to shell reference surface as shown in Figures 20 thru 28. That is, under compression and bending, "N" and "M" were transferred from the plastic centroid to the shell reference surface. Under tension, "N" was transferred from the center of gravity of the reinforcing steel to the shell reference surface. It must be emphasized that even if a point representing a stress state fell outside the diagram, that would not indicate a critical or nearly critical condition for two reasons:

a. The interaction diagrams were determined on the basis of conservative assumptions and it is expected that the 'true' failure envelopes lie a certain distance away from the computed envelopes.

b. A point outside the interaction diagram would merely indicate local yielding of one or more rebars at that location, which would cause a load redistribution towards less highly stressed regions. A point representing a stress state contained within an interaction diagram indicates that stresses in all steel rebars at that section are below yield stress, and that concrete stresses, where applicable, are below code allowable stresses.

5.2 Interaction Diagram

a. Axial Compression and Bending (See Figure 19)

1. Concentric Compression

$P_8 = f\,[0.85\, f_C\, A_C + A_{st}\, f_y]$,

$A_{st} = A_{s1} + A_{s2} + A_{s3}$

2. Simple Bending

$$a = \frac{A_{s1}\, f_y + A_{s2}\, f_{s2} - A'_{s3}\, f_{s3}}{0.85\, f_C\, b}$$

$$M_8 = [T_1\, d_1 + T_2\, d_2 + C_{s3}\, d'_3 + C_C\, (C - \frac{a}{2})]$$

$C \quad d$

$M_C \quad M_{s1} + M_C$

$C_C = 0.85\ f_C\ a\ b$

3. Bending and Axial Compression

$$C_b = \frac{d(87000)}{87000 + f_y}$$

$a_b = k_1 a.b$

$P_b = f\ [0.85\ f_C\ (a_b - a)\ b]$

$M_b = M_8 + P_b\ d_b$

4. Determine P_a and M_a

$e_a = 0.1\ t$; ACI 318-63, Section 1901 (a)
$M_a = P_a e_a$

By similar triangles, $P_a = \dfrac{P_8\ M_b}{e_a(P_8 - P_b) + M_b}$

b. Axial Tension and Bending

1. Concentric Tension

$P_8 = f\ A_{st}\ f_y$

Note: For purpose of clarity, only three steel layers were included in the preceding equations.

5.3 Reinforcing Steel

a. The opening steel reinforcement will be as shown on Drawing Nos. D-421-035 and D-421-036.

b. The amount of reinforcement, which includes regular hoop, meridional, and circular ring steel around the opening, equals or exceeds that shown to be required by calculations. In no event is the liner assumed to contribute more than its yield strength.

c. The clear distance between parallel bars is not less than 2 times the maximum size of coarse aggregate, 2 times the bar diameter, nor less than 2 inches.

d. Vertical shrinkage or temperature reinforcement is placed at the outside face of wall. The minimum amount of such reinforcement on the outside concrete face wall is greater than 0.0015 of the gross cross-sectional area of concrete.

5.4 Penetration Barrel

The portions of the Equipment Access Hatch and Personnel Lock extending beyond the concrete shell were designed and fabricated by Chicago Bridge and Iron Company in accordance with the ASME Nuclear Vessels Code. The barrel of each of these penetrations within the limit of the concrete thickness was investigated for the following loads:

a. Meridional membrane stresses in the barrel due to internal pressure on the hatch.

b. Hoop membrane stresses in the barrel due to the 'in-plane' deformation of the opening.

c. Meridional bending stresses in the barrel caused by meridional shear transfer from the barrel to the concrete.

d. Thermal Stresses.
The objective of this investigation was to verify that the stresses in the barrel are within allowable limits. Refer to ASME Nuclear Vessels Code, Article 4, Par. N-414. It should be noted that the allowable stresses referred to are based on working strength design.

5.5 Concrete Shear

Splitting planes were hypothesized parallel to the surface of the shell through the various layers of concrete reinforcement and tendon conduit. The 'in-plane' shear stresses are produced by radial forces produced by circular rebar rings and draped tendons. Sufficient steel has been provided in the form of straight or hooked vertical and horizontal bars and ties to develop the total shear stress across the hypothesized planes (Reference 29). The shear stresses are conservatively assumed to be the summation of the loads resisted by the circular bars on the vertical axis due to the factored pressure load. That is to say, the shearing force exerted across a plane through Layer 15 (see attached Drawing No. D-421-036) is equal to the summation of rebar forces on Layers 15 and 16 on the vertical axis. The maximum shear stress on the dowels due to the aforementioned load does not exceed the yield stress of the dowels. The dowels are anchored by mechanical anchorage (900 hooks) and/or sufficient bond development length which is determined on the basis of Ultimate Strength Design provision of ACI 318-63. All rebars provided to resist the aforementioned loads consist of A615 material with a 40,000 psi minimum guaranteed yield strength.

5.6 Shear-Diagonal Tension

The ACI 318-63 recognizes the punching shear to be critical
at a distance "d/2" out from the periphery for slabs and footings (See Sections 1207 and 1707). A beam type of diagonal tension failure is impossible due to the geometry and two directional stresses in the shell. We believe that a punching mode of failure is the type of failure that should be and has been investigated. These shear stresses included in the computer output take into account the effect of the pressure on the door plus the reinforced area.

Two modes of shear transfer are considered. First, shear transfer through concrete without shear reinforcement. Second, disregarding the shear capacity of concrete, enough reinforcing steel is provided to carry the normal shears by steel alone.

Ultimate peripheral or normal shear stress carried by concrete is computed by:

$$v_u = \frac{Q}{d}$$

$$v_c = 4 f^{d}(f_C)^{1/2}$$

(ACI 318-63, Section 1707)

where:

Q = normal shear stress-resultant at the critical section

v_u = nominal ultimate shear stress as a measure of diagonal tension

v_c = allowable ultimate shear stress to be carried by concrete

d = distance from extreme compression fiber to centroid of tension reinforcement

f_C = 28 days compressive strength of concrete

f = 0.85

Shear reinforcement requirements

The ultimate shear capacity of the reinforcing steel alone is computed by:

$$v_u = f\,[a_{sv}\, f_y]$$

where:

A_{sv} = cross-sectional area of reinforcing steel acting in tension across a potential diagonal tension crack

f_y = yield strength of reinforcement

f = 0.85

5.7 Shear Transfer Plates

Steel shear transfer plates are provided at the intersection of the barrel penetration sleeve and liner, for both equipment and personnel accesses, as shown in Figure 29. The shear plates, made of A516 Gr. 70 steel, are designed to resist a total shear force produced by an internal pressure of 1.5 accident pressure acting outward from the barrel. The shear force is transferred from the barrel penetration sleeve to the liner and concrete by means of the shear transfer plates. It is conservatively assumed that the welded tee joint, at the intersection of the liner and barrel penetration, does not resist any of the shear force.

5.8 Radial Reinforcement

Draped horizontal and vertical tendons produce radial forces which act toward the opening as shown in Figures 11 and 12. These radial forces result in producing tensile stresses, in the concrete, in the vicinity of the curved tendons. Steel reinforcement is provided in the form of hook and straight bars to resist radial forces, as can be seen in Drawing D-421-036. Hoop reinforcing resist the radial forces pro- duced by curved vertical tendons while meridional reinforcing resist radial forces produced by curved horizontal tendons.

The unit radial forces and total radial forces are given by the following equations:

Unit Radial Force $p = T/r$

Total Radial Force $p = \sum_{i=1}^{N} \frac{T_i}{r_i} L_i$

p = unit radial force kips/inch
P = total radial force KIPS
T = initial prestressing tendon force KIPS
r = radius of curvature of tendon
N = total number of curved tendons

5.9 Summary of Design and Conclusions

In selecting the analytical method used for determining the stress-resultant and stress-couple distributions in the shell under all critical loading conditions an extensive bibliographic search was conducted and available methods were evaluated. In our judgment the analysis of the stresses around the openings for the Three Mile Island Reactor Building has been based on the most satisfactory of available methods.

The design was guided by the basic proposition that the best reinforcement is in fact the least reinforcement that will satisfy the requirement for carrying the shell loads around the opening and the normal shear applied along the opening edge into the shell out to a distance from the opening until a membrane state of stress is reached. Although not directly applicable the IITRI studies on steel containment structures conclusively showed that a stiff reinforcement around openings substantially reduces the burst strength of circular plates (Reference 30). In our judgment this design as evidenced by the data included in Section 5 provides the required reinforcement strength and further conservatism in determining reinforcement requirements is not prudent in that the ultimate load capacity might be thereby reduced.

LIST OF REFERENCES

1. A. I. Lur'e: "Concentration of Stresses in the Vicinity of an Aperture in the Surface of a Circular Cylinder," Prikl. Mat. Mekh. 10, 1946, pp. 397-407, (English translation by N. Brunswick, N.Y. Univ., Inst. of Math. Science, 1960)

2. S. Timoshenko: "Strength of Materials, Part II," D. Van Nostrand Company, Inc., New York, N.Y., 1930, p. 457.

3. K. Sezawa and K. Kubo: "Stresses in a Plate with a Flanged Circular Hole," Report of the Aeronautical Research Institute, Tokyo Imperial University, Tokyo, Japan, No. 84, September, 1932.

4. G. Gurney: "Analysis of the Stresses in a Flate Plate with a Circular Hole Under Edge Forces," British Rep. Memo No. 1834, February, 1938.

5. Leon Beskin: "Strengthening of Circular Holes in Plates Under Edge Loads," Journal of Applied Mechanics, Trans. ASME, Vol. 66, 1944, p. A-140.

6. G. N. Savin: "Stress Concentration Around Holes," Pergamon Press, 1961.

7. D. Withum: "Die Kreiszylinderschale mit Kreisformigem Ausschnitt unter Schubbean-Spruchung," Ingerieur Archiv, XXVI, 1958, pp. 435-446.

8. V. Kline, R. C. Dixon, N. F. Jordan, and A. C. Eringen: "Stresses in Pressurized Cylindrical Shell with Cut-out," General Technology Corp. Tech, Report 3-1, 1961.

9. A. C. Eringen, A. K. Naghdi, and C. C. Thiel: "State of Stress in a Circular Cylindrical Shell with a Circular Hole," Welding Research Council Bulletin 102, 1965.

10. A. K. Naghdi and A. C. Eringen: "Stress Distribution in a Circular Cylindrical Shell with a Circular Cut-out," Ingenieur Archiv.

11. J. G. Lekkerkerker: "Stress Concentration Around Circular Holes in Cylindrical Shells," Eleventh Int. Congress on Applied Mechanics, Munich, 1964.

12. FELAP - Finite Element Analysis Program for Complex Structures, Franklin Institute Research Laboratories, Philadelphia, Pa.

13. Z. Zudans: "New Formulation and Evaluation of Elastic Shell Theory," Ph. D. Dissertation, University of Pennsylvania, May 1966.

14. Z. Zudans: "Analysis of Asymmetric Stiffened Shell Type Structures by the Finite-Element Method," paper to be published in "Nuclear Engineering and Design."

15. J. J. Connor and C. Brebbia: "Stiffness Matrix for Shallow Rectangular Shell Elements", ASCE Journal, Vol. 93, EM5, pp. 43-65, 1967.

16. C. Prato: "A Mixed Finite-Element Method for Shell Analysis," Ph.D. Dissertation, M.I.T., June 1968.

17. F. Yao "Finite-Element Method for Shell Analysis," Ph.D. Dissertation, M.I.T., January 1968.

18. M. M. Leven: "Photoelastic Determination of Stresses in Reinforced Openings in Pressure Vessels," Welding Research Council Bulletin No. 113, N.Y., 1966, pp. 25-52.

19. C. E. Taylor and N. C. Lind: "Photoelastic Study of the Stresses Near Openings in Pressure Vessels," ibid., pp. 1-24.

20. S. K. Takahashi and R. Mark: "Analysis and Photoelastic Investigations of Stress Concentrations in Sphere-Cylinder Configurations," U. S. Naval Civil Engineering Laboratory Report.

21. A. J. Durelli, D. J. del Rio, V.J. Parks, H. Feng: "Stresses in a Pressurized Cylinder with a Hole," Journal ASCE, Vol. 93, ST5, 1967, pp. 383-399.

22. R. Mark and J. D. Riera: Discussion of Ref. (19), Journal ASCE, Vol. 94, ST4, pp. 1075-76.

23. J. L. Mershon: "Preliminary Evaluation of PVCR Photoelastic Test Data on Reinforced Openings in Pressure Vessels," Welding Research Council Bulletin No. 113, 1966.

24. D. S. Houghton and A Rothwell: "The Influence of Curvature on the State of Stress Ahead of Cracks in Cylindrical Shells," Non-Classical Shell Problems, Proc. of the Int. Symposium on Non-Classical Shell Problems, Warsaw, Poland, 1963, North Holland, 1964, pp. 685-700.

25. R. Mark: "Photomechanical Model Analysis of Concrete Structures," ACI Symposium on Models, 1968.

26. J. H. Barnwell and C. H. Edwards: "The Development and Testing of Three-Dimensional Photoelastic Models of Pretensioned, Prestressed Concrete Beams," paper presented at the 1968 SESA Spring Meeting, May 1968, Albany, N.Y.

27. Ch. H. Lerchenthal: "A Photoelastic Investigation of Stress Concentration Around Reinforced Cut-outs in Stresses Sheets," ibid.

28. R. D. Brunell "Computer Program Heating and Heating 2," Atomic International Division Report No. AI-64-MEMO-177, North American Aviation Inc., Detroit, Michigan, USA, 1964.

29. R. F. Mast: "Auxiliary Reinforcement in Concrete Connections," Journal ASCE, Vol. 94, No. ST6, June 1968, pp. 1485-1504.

30. M. A. Salmon: "Studies of Reactor Containment Structures," IITRI Final Report for U.S. AEC, Dept. of Commerce, Washington, D. C., 1966.

TABLES

TABLE 5C-1		FUNDAMENTAL LOADS								
		Load No.	DL (1)	H&VP (2)	OT (3)	AT (4)	IP (5)	E_1 (6)	E_2 (7)	(8)
Loading Combinations	Normal Operation	1	1.000	1.260	0.0	0.0	0.0	0.0	0.0	0.0
		2	1.000	1.110	0.270	0.0	0.0	0.0	0.0	0.0
		3	1.000	1.110	1.000	0.0	0.0	0.0	0.0	0.0
		4	1.000	1.110	0.270	0.0	0.0	0.625	0.0	0.0
		5	1.000	1.110	1.000	0.0	0.0	0.625	0.0	0.0
		6	1.000	1.110	0.270	0.0	0.0	-0.625	0.0	0.0
		7	1.000	1.110	1.000	0.0	0.0	-0.625	0.0	0.0
		8	1.000	1.110	0.270	0.0	0.0	0.0	0.625	0.0
		9	1.000	1.110	1.000	0.0	0.0	0.0	0.625	0.0
		10	1.000	1.110	0.270	0.0	0.0	0.0	-0.625	0.0
		11	1.000	1.110	1.000	0.0	0.0	0.0	-0.625	0.0
		12	1.000	1.000	0.270	0.0	0.0	0.0	0.0	0.0
		13	1.000	1.000	1.000	0.0	0.0	0.0	0.0	0.0
		14	1.000	1.000	0.270	0.0	0.0	0.625	0.0	0.0
		15	1.000	1.000	1.000	0.0	0.0	0.625	0.0	0.0
		16	1.000	1.000	0.270	0.0	0.0	-0.625	0.0	0.0
		17	1.000	1.000	1.000	0.0	0.0	-0.625	0.0	0.0
		18	1.000	1.000	0.270	0.0	0.0	0.0	0.625	0.0
		19	1.000	1.000	1.000	0.0	0.0	0.0	0.625	0.0
		20	1.000	1.000	0.270	0.0	0.0	0.0	-0.625	0.0
		21	1.000	1.000	1.000	0.0	0.0	0.0	-0.625	0.0
	Test Condition	22	1.000	1.110	0.270	0.0	1.150	0.0	0.0	0.0
		23	1.000	1.110	1.000	0.0	1.150	0.0	0.0	0.0
		24	1.000	1.000	0.270	0.0	1.150	0.0	0.0	0.0
		25	1.000	1.000	1.000	0.0	1.150	0.0	0.0	0.0
	Accident Pressure Loadings	26	1.000	1.110	0.270	1.000	1.000	0.0	0.0	0.0
		27	1.000	1.110	1.000	1.000	1.000	0.0	0.0	0.0
		28	1.000	1.110	0.270	1.000	1.000	1.000	0.0	0.0
		29	1.000	1.110	1.000	1.000	1.000	1.000	0.0	0.0
		30	1.000	1.110	0.270	1.000	1.000	-1.000	0.0	0.0
		31	1.000	1.110	1.000	1.000	1.000	-1.000	0.0	0.0
		32	1.000	1.110	0.270	1.000	1.000	0.0	1.000	0.0
		33	1.000	1.110	1.000	1.000	1.000	0.0	1.000	0.0
		34	1.000	1.110	0.270	1.000	1.000	0.0	-1.000	0.0
		35	1.000	1.110	1.000	1.000	1.000	0.0	-1.000	0.0
		36	1.000	1.000	0.270	1.000	1.000	0.0	0.0	0.0
		37	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0
		38	1.000	1.000	0.270	1.000	1.000	1.000	0.0	0.0
		39	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0
		40	1.000	1.000	0.270	1.000	1.000	-1.000	0.0	0.0
		41	1.000	1.000	1.000	1.000	1.000	-1.000	0.0	0.0

TABLE 5C-1 (continued)		FUNDAMENTAL LOADS (continued)								
		Load No.	DL (1)	H&VP (2)	OT (3)	AT (4)	IP (5)	E_1 (6)	E_2 (7)	(8)
Loading Combinations (continued)	Accident Pressure Loadings (continued)	42	1.000	1.000	0.270	1.000	1.000	0.0	1.000	0.0
		43	1.000	1.000	1.000	1.000	1.000	0.0	1.000	0.0
		44	1.000	1.000	0.270	1.000	1.000	0.0	-1.000	0.0
		45	1.000	1.000	1.000	1.000	1.000	0.0	-1.000	0.0
		46	1.000	1.110	0.270	1.000	1.250	0.0	0.0	0.0
		47	1.000	1.110	1.000	1.000	1.250	0.0	0.0	0.0
		48	1.000	1.110	0.270	1.000	1.250	0.625	0.0	0.0
		49	1.000	1.110	1.000	1.000	1.250	0.625	0.0	0.0
		50	1.000	1.110	0.270	1.000	1.250	-0.625	0.0	0.0
		51	1.000	1.110	1.000	1.000	1.250	-0.625	0.0	0.0
		52	1.000	1.110	0.270	1.000	1.250	0.0	0.625	0.0
		53	1.000	1.110	1.000	1.000	1.250	0.0	0.625	0.0
		54	1.000	1.110	0.270	1.000	1.250	0.0	-0.625	0.0
		55	1.000	1.110	1.000	1.000	1.250	0.0	-0.625	0.0
		56	1.000	1.000	0.270	1.000	1.250	0.0	0.0	0.0
		57	1.000	1.000	1.000	1.000	1.250	0.0	0.0	0.0
		58	1.000	1.000	0.270	1.000	1.250	0.625	0.0	0.0
		59	1.000	1.000	1.000	1.000	1.250	0.625	0.0	0.0
		60	1.000	1.000	0.270	1.000	1.250	-0.625	0.0	0.0
		61	1.000	1.000	1.000	1.000	1.250	-0.625	0.0	0.0
		62	1.000	1.000	0.270	1.000	1.250	0.0	0.625	0.0
		63	1.000	1.000	1.000	1.000	1.250	0.0	0.625	0.0
		64	1.000	1.000	0.270	1.000	1.250	0.0	-0.625	0.0
		65	1.000	1.000	1.000	1.000	1.250	0.0	-0.625	0.0
		66	1.000	1.000	0.270	1.000	1.150	0.0	0.0	0.0
		67	1.000	1.260	1.000	1.000	1.150	0.0	0.0	0.0
		68	1.000	1.000	0.270	1.000	1.150	0.0	0.0	0.0
		69	1.000	1.000	1.000	1.000	1.150	0.0	0.0	0.0
	Normal Operations	70	1.000	1.110	0.270	0.0	0.0	1.000	0.0	0.0
		71	1.000	1.110	1.000	0.0	0.0	1.000	0.0	0.0
		72	1.000	1.110	0.270	0.0	0.0	-1.000	0.0	0.0
		73	1.000	1.110	1.000	0.0	0.0	-1.000	0.0	0.0
		74	1.000	1.110	0.270	0.0	0.0	0.0	1.000	0.0
		75	1.000	1.110	1.000	0.0	0.0	0.0	1.000	0.0
		76	1.000	1.110	0.270	0.0	0.0	0.0	-1.000	0.0
		77	1.000	1.110	1.000	0.0	0.0	0.0	-1.000	0.0

DL = Dead Load
H & VP = Horizontal and Vertical Prestress
OT = Operating Temperature

AT = Accident Temperature
IP = Internal Pressure
E1 = Earthquake 1
E2 = Earthquake 2

Note: Coefficients are based on:
Accident Pressure, Winter Temperature,
Final Prestress, Maximum Hypothetical Earthquake

TABLE 5C-2
(Sheet 1 of 1)

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 1
DEAD LOAD

	AXIAL DIRECTION		HOOP DIRECTION		MEMBRANE SHEAR	TWISTING MOMENT	NORMAL	SHEARS
	$N\phi$	$M\phi$	$N\theta$	$M\theta$	$N\phi\theta$	$M\phi\theta$	$Q\phi$	$Q\theta$
11	-5.55	14.78	-0.15	3.58	0.12	-0.36	0.42	0.00
22	-5.05	-17.67	-1.05	-0.60	0.21	-0.25	0.46	-0.01
33	-4.61	-59.53	-0.94	-10.57	0.43	5.09	0.27	0.26
44	-2.06	-8.70	1.36	25.65	0.79	15.23	-1.26	0.10
55	-0.50	8.10	4.01	77.44	0.71	10.54	-0.66	-0.54
66	0.30	13.15	7.17	139.22	0.45	3.47	-0.48	-1.01
77	0.44	4.67	12.26	241.50	-0.63	-16.09	0.65	-0.88
25	-8.70	-22.61	0.78	3.48	-0.32	-9. 85	0.23	0.08
49	-12.24	-210.41	2.13	64.37	-0.28	-16.70	0.82	-0.53
73	-14.62	-259.86	1.26	48.47	0.30	-10.33	0.86	-0.25
74	-11.32	-198.03	0.58	10.96	2.43	31.48	0.42	0.86
94	-8.11	11.44	0.79	-8.53	-0.12	8.44	-2.32	-0.12
97	-11.96	-142.28	-1.19	5.85	-0.30	5.98	0.03	0.43
99	-16.70	-279.92	-3.20	-45.84	-0.38	-10.72	0.11	1.25
100	-21.34	-366.85	-3.39	-56.70	0.28	0.89	1.12	1.51
101	-29.35	-504.98	-1.93	-11.53	2.56	48.08	2.72	-2.36

NOTE: $N\phi$, $N\theta$, $N\phi\theta$, $Q\phi$, and $Q\theta$ in Kips/in.
$M\phi$ and $M\theta$ in Kips-in/in.
Values computed by finite-element analysis (Sheet 1 of 7)

TABLE 5C-3
(Sheet 1 of 1)

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 2
HOOP AND VERTICAL PRESTRESS

	AXIAL DIRECTION		HOOP DIRECTION		MEMBRANE SHEAR	TWISTING MOMENT	NORMAL	SHEARS
	$N\phi$	$M\phi$	$N\theta$	$M\theta$	$N\phi\theta$	$M\phi\theta$	$Q\phi$	$Q\theta$
11	-31.79	-142.30	-72.32	55.63	0.14	-1.20	-0.44	0.12
22	-29.51	-118.97	-67.36	61.36	0.73	-2.52	-0.35	0.05
33	-24.90	-167.75	-91.91	-789.23	2.69	12.76	-0.50	32.16
44	-8.42	2.42	-113.30	-1906.16	6.02	64.67	-3.67	2.62
55	-1.13	-13.56	-112.86	- 1752.00	9.22	107.44	-10.39	2.84
66	-9.01	-80.17	-118.33	-1631.09	10.90	122.42	-18.66	0.59
77	-10.19	-17.42	-125.59	-1570.90	14.17	124.75	-27.16	3.44
25	-39.83	-117.50	-74.70	-241.10	1.72	-150.89	2.59	4.00
49	-45.42	-379.25	-69.20	-751.08	16.13	33.18	0.22	3.20
73	-52.07	-458.64	-53.36	-659.38	29 .47	224.47	0.31	2.26
74	-53.81	-488.44	-60.14	-809.52	42.35	392.40	10.25	6.37
94	-39.10	47.94	-55.55	9.88	0.52	-4.06	0.73	3.52
97	-60.31	-475.61	40.43	-255.04	2.58	22.90	-0.42	-4.61
99	-68.06	-543.16	-17.65	-86.70	7.30	84.43	2.09	-6.53
100	-72.22	-416.40	-6.67	-30.10	9.89	93.83	-1.10	10.62
101	-76.92	-219.28	-7.43	26.58	10.27	46.46	-4.02	14.42

NOTE: $N\phi$, $N\theta$, $N\phi\theta$, $Q\phi$, and $Q\theta$ in Kips/in.
$M\phi$ and $M\theta$ in Kips-in/in.
Values computed by finite-element analysis (Sheet 2 of 7)

TABLE 5C-4
(Sheet 1 of 1)

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 3
OPERATING TEMPERATURE (WINTER)

	AXIAL DIRECTION		HOOP DIRECTION		MEMBRANE SHEAR	TWISTING MOMENT	NORMAL	SHEARS
	$N\phi$	$M\phi$	$N\theta$	$M\theta$	$N\phi\theta$	$M\phi\theta$	$Q\phi$	$Q\theta$
11	-3.10	514.41	10.23	433.38	-0.17	0.10	0.20	-0.01
22	-3.96	529.61	7.96	440.66	-0.12	-0.61	-0.69	-0.02
33	0.24	694.27	0.39	984.76	0.11	-7.74	-5.00	-33.98
44	-4.29	490.90	-14.95	1582.80	0.27	-38.15	-1.04	0.08
55	-4.07	323.86	-18.51	1652.76	0.08	-97.75	1.43	-3.91
66	-3.40	268.22	-22.05	1771.96	0.25	-132.39	3.86	1.59
77	-1.16	145.44	-25.85	1932.96	2.60	-156.72	3.68	12.47
25	0.16	514.79	2.91	571.08	-3.92	101.63	-1.85	-2.50
49	-5.83	1144.06	-11.13	759.89	1.09	-38.77	-1.25	0.41
73	-6.41	1346.10	-6.66	730.25	4.13	-294.50	5.43	0.54
74	-5.76	1196.31	-7.02	955.61	4.50	-606.81	-3.38	-3.62
94	-17.97	489.90	-6.49	479.64	-4.98	102.04	-24.95	-0.16
97	-13.53	924.93	-4.45	642.63	0.50	-2.91	0.46	9.88
99	-8.48	1683.20	-3.26	478.10	0.57	-51.10	0.58	-1.90
100	-9.16	1866.96	-2.10	341.98	0.68	-106.07	0.01	-3.47
101	-12.72	2053.34	-0.91	35.57	1.61	-187.64	-0.52	7.54

NOTE: $N\phi$, $N\theta$, $N\phi\theta$, $Q\phi$, and $Q\theta$ in Kips/in.
$M\phi$ and $M\theta$ in Kips-in/in.
Values computed by finite-element analysis (Sheet 3 of 7)

TABLE 5C-5
(Sheet 1 of 1)

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 4
ACCIDENT TEMPERATURE (WITH BARRELL EFFECT)

	AXIAL DIRECTION		HOOP DIRECTION		MEMBRANE SHEAR	TWISTING MOMENT	NORMAL	SHEARS
	$N\phi$	$M\phi$	$N\theta$	$M\theta$	$N\phi\theta$	$M\phi\theta$	$Q\phi$	$Q\theta$
11	-2.86	217.29	0.09	186.63	-0.17	0.73	0.74	0.00
22	-3.74	170.86	0.68	176.19	-0.24	0.58	0.70	0.00
33	-4.34	140.97	4.97	333.57	-0.62	-8.70	0.40	-11.43
44	-9.82	-78.19	13.42	666.08	-1.80	-44.84	2.41	-0.05
55	-14.81	-278.23	17.75	787.17	-3.77	-108.26	3.15	-0.94
66	-20.71	-431.10	15.33	785.34	-6.40	-154.00	1.21	1.63
77	-27.65	-638.49	17.94	927.02	-10.08	-217.19	-2.52	18.04
25	-1.56	179.23	-1.02	205.89	-0.29	20.06	-0.44	-0.51
49	2.31	357.30	0.12	260.03	-4.03	-174.35	-1.44	-0.38
73	7.88	563.30	-0.85	185.09	-10.00	-353.01	-2.05	-0.07
74	0.99	356.46	0.62	274.74	-15.13	-534.57	0.72	0.32
94	-1.48	182.56	-1.67	168.19	-0.03	-0.75	0.08	-0.45
97	5.09	435.56	-4.30	164.76	-0.68	-17.07	-0.28	3.49
99	13.85	856.12	-12.86	129.17	-2.98	-87.04	-0.45	3.00
100	15.84	956.25	-20.01	-343.88	-6.20	-164.51	-0.64	5.65
101	7.86	892.45	-26.94	-613.86	-9.40	-255.04	16.46	-2.69

NOTE: $N\phi$, $N\theta$, $N\phi\theta$, $Q\phi$, and $Q\theta$ in Kips/in.
$M\phi$ and $M\theta$ in Kips-in/in.
Values computed by finite-element analysis (Sheet 4 of 7)

TABLE 5C-6
(Sheet 1 of 1)

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 5
55 psig INTERNAL PRESSURE

	AXIAL DIRECTION		HOOP DIRECTION		MEMBRANE SHEAR	TWISTING MOMENT	NORMAL	SHEARS
	$N\phi$	$M\phi$	$N\theta$	$M\theta$	$N\phi\theta$	$M\phi\theta$	$Q\phi$	$Q\theta$
11	23.12	88.28	45.64	-37.45	-0.12	1.52	0.81	-0.04
22	22.51	35.40	43.89	-56.46	-0.13	2.98	1.00	-0.02
33	22.43	29.84	58.58	423.42	-0.39	6.13	-0.18	-18.10
44	18.98	-5.22	74.08	1070.76	-1.22	4.56	0.85	-0.06
55	14.66	-55.50	82.16	1157.09	-3.64	-54.47	6.79	-3.58
66	8.84	-90.60	93.92	1304.67	-6.74	-96.68	8.83	-3.47
77	-1.05	-194.66	111.53	1589.83	-12.51	-160.33	13.96	2.52
25	20.98	52.28	47.42	169.43	-0.63	78.06	-1.62	-2.49
49	23.20	115.32	47.20	639.42	-10.54	-98.54	-1.12	-2.14
73	25.75	267.38	36.81	532.50	-20.32	-324.05	-3.19	0.21
74	25.24	213.57	39.74	607.13	-28.21	-427.25	-12.22	-0.96
94	20.55	-38.89	39.23	-110.38	-0.20	-7.25	1.36	-3.85
97	34.65	531.60	29.36	346.81	-1.70	-21.80	-0.58	5.64
99	36.80	701.25	13.72	103.07	-4.49	-81.21	-1.10	5.31
100	34.78	625.07	3.72	-34.14	-6.12	-109.36	-0.53	7.93
101	29.14	475.54	-5.16	-172.79	-6.51	-115.10	6.09	6.90

NOTE: $N\phi$, $N\theta$, $N\phi\theta$, $Q\phi$, and $Q\theta$ in Kips/in.
$M\phi$ and $M\theta$ in Kips-in/in.
Values computed by finite-element analysis (Sheet 5 of 7)

TABLE 5C-7
(Sheet 1 of 1)

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 6
EARTHQUAKE #1

	AXIAL DIRECTION		HOOP DIRECTION		MEMBRANE SHEAR	TWISTING MOMENT	NORMAL	SHEARS
	$N\phi$	$M\phi$	$N\theta$	$M\theta$	$N\phi\theta$	$M\phi\theta$	$Q\phi$	$Q\theta$
11	-4.07	10.83	-0.11	2.62	0.09	-0.26	0.31	0.00
22	-3.70	-12.95	-0.77	-0.44	0.15	-0.18	0.34	-0.01
33	-3.38	-43.64	-0.69	-7.75	0.32	3.73	0.20	0.19
44	-1.51	-6.38	1.00	18.80	0.58	11.16	-0.92	0.07
55	-0.37	5.94	2.93	56.76	0.52	7.73	-0.48	-0.40
66	0.22	9.64	5.26	102.05	0.33	2.54	-0.35	-0.74
77	0.32	3.42	8.99	177.02	-0.46	-11.79	0.48	-0.65
25	-6.38	-16.57	0.57	2.55	-0.23	-7.22	0.17	0.06
49	-8.97	-154.23	1.56	47.18	-0.21	-12.24	0.60	-0.39
73	-10.72	-190.48	0.92	35.53	0.22	-7.57	0.63	-0.18
74	-8.30	-145.16	0.43	8.03	1.78	23.07	0.31	0.63
94	-5.94	8.39	0.58	-6.25	-0.09	6.19	-1.70	-0.09
97	-8.77	-104.29	-0.87	4.29	-0.21	-4.38	0.02	0.32
99	-12.24	-205.18	-2.35	-33.60	-0.29	-7.86	0.08	0.92
100	-15.64	-268.90	-2.48	-41.56	0.21	0.65	0.82	1.11
101	-21.51	-370.15	-1.41	-8.45	1.88	35.24	1.99	-1.73

NOTE: $N\phi$, $N\theta$, $N\phi\theta$, $Q\phi$, and $Q\theta$ in Kips/in.
$M\phi$ and $M\theta$ in Kips-in/in.
Values computed by finite-element analysis (Sheet 6 of 7)

TABLE 5C-8
(Sheet 1 of 1)

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 7
EARTHQUAKE #2

	AXIAL DIRECTION		HOOP DIRECTION		MEMBRANE SHEAR	TWISTING MOMENT	NORMAL	SHEARS
	$N\phi$	$M\phi$	$N\theta$	$M\theta$	$N\phi\theta$	$M\phi\theta$	$Q\phi$	$Q\theta$
11	-0.56	1.48	-0.02	0.36	3.59	-0.04	0.04	0.00
22	-0.51	-1.77	0.69	-0.06	2.72	-0.03	0.05	0.00
33	-0.46	-5.95	1.41	-1.06	2.04	0.51	0.03	0.03
44	-0.21	-0.87	2.14	2.57	1.58	1.52	-0.13	0.01
55	-0.05	0.81	2.90	7.74	1.07	1.05	-0.07	-0.05
66	0.03	1.32	3.72	13.92	0.55	0.35	-0.05	-0.10
77	0.04	0.47	4.81	24.15	-0.06	-1.61	0.07	-0.09
25	-0.87	-2.26	0.08	0.35	3.55	-0.99	0.02	0.01
49	-1.22	-21.04	0.21	6.44	3.55	-1.67	0.08	-0.05
73	5.54	-25.99	7.13	4.85	7.03	-1.03	0.09	-0.03
74	7.77	-19.80	8.96	1.10	9.14	3.15	0.04	0.09
94	-0.81	1.14	0.08	-0.85	3.57	0.84	-0.23	-0.01
97	-0.20	-14.23	-0.12	0.59	2.47	-0.60	0.00	0.04
99	0.13	-27.99	-0.32	-4.58	1.76	-1.07	0.01	0.13
100	0.37	-36.69	-0.34	-5.67	1.03	0.09	0.11	0.15
101	0.64	-50.50	-0.19	-1.15	0.26	4.81	0.27	-0.24

NOTE: $N\phi$, $N\theta$, $N\phi\theta$, $Q\phi$, and $Q\theta$ in Kips/in.
$M\phi$ and $M\theta$ in Kips-in/in.
Values computed by finite-element analysis (Sheet 7 of 7)

FIGURES

DRAWINGS

APPENDIX A

Effect of Concrete Creep and the Sustained Operating Stresses on Stress Distribution Around Openings in a Rapidly Pressurized Reinforced Concrete Vessel.

APPENDIX A

EFFECT OF CONCRETE CREEP AND THE SUSTAINED OPERATING STRESSES ON THE STRESS DISTRIBUTION AROUND OPENINGS IN A RAPIDLY PRESSURIZED REINFORCED CONCRETE VESSEL

Consider the simple structure shown below:

Figure A-1

Column 1 is a reinforced concrete column with net concrete area A^1_C and longitudinal steel areas A_{st}^1. A^2_C and A^2_{st} denote the net concrete area and the logitudinal steel area, respectively, of reinforced concrete columns 2. The system is loaded at time t = 0 with a vertical load P. Let us determine the initial load distribution:

$$P = T_1 + 2T_2 \sin 600 = T_1 + Ö3\, T_2 \qquad (1)$$

$$\text{delta } L_2 = \text{delta } L_1 \sin 600;\ \text{delta } L_2 = \frac{Ö3}{2}\ \text{delta } L_1 \qquad (2)$$

$$\text{in which } L_1 = \frac{T_1}{A^1_C E_C + A_{st} E_{st}^1} \qquad (3) \qquad L$$

$$\frac{Ö3\ \text{delta } L_2}{2\ L} = \frac{T_2}{A_C^2 E_C + A_{st}^2 E_{st}} \qquad (4)$$

E_C and E_{st} denote the "effective" modulae of elasticity of concrete and steel, T_1 and T_2 the loads carried by columns 1 and 2, respectively. From equations (1) to (4) we obtain:

$$T_1 + Ö3\, T_2 = P$$

$$\frac{Ö3}{2}\ \frac{\text{delta } L_1}{A_C E_C + A_{st} E_{st}}\ T_1 - \frac{2}{Ö3}\ \frac{L}{A_C^2 E_C + A^2_{st} E_{st}}\ T_2 = 0$$

or

$$\frac{T_1}{T_2} = \left\{ \frac{J}{1.5I + J} \right\} \left\{ \frac{Ö3}{2}\ \frac{I}{1.5I + J} \right\} P \qquad (5)$$

In which

$$I = \frac{L}{A_C^1 E_C + A_{st}^1 E_{st}}$$

$$J = \frac{2}{Ö3} \frac{L}{A_C^2 E_C + A_{st}^2 E_{st}} \qquad (6)$$

Equation (5) gives the loads acting on Columns 1 and 2 in terms of the "effective" modulae E_C and E_{st}

In general, T_1 and T_2 will change with time (Load redistribution) due to concrete creep. Under the assumption that concrete behaves like a Kelvin-type material, the load distribution may be calculated exactly for t ® ¥ by resorting to the creep-limit modulus $E_{cu.}$ (Time-dependent behavior of steel is neglected).

Note that even when there is no load redistribution (for example, when I = B) there will be some stress redistribution. In other words, T_1 an T_2 may remain constant, but the percentage of both carried by the steel reinforcement will increase as concrete creeps. As a result, steel stresses will increase and concrete stresses will decrease to final values which may be easily computed.

Let us now assume that at time t_1 a load P_1 is superimposed on the existing load P_8 giving a total load ($P_8 + P_1$). (See Figure A-1). let T_1 and T_2 be the column loads immediately before the load P_1 is applied. To compute the actual column loads T_1 and T_2 after P_1 is applied, we would be tempted to determine the column loads T_1* and T_2* corresponding to P_1 acting alone on the structure (On the basis of the initial modulus of elasticity) and then add them to T_1^1 and T_2^1:

$$T_1^2 = T_1^1 + T_1^* \qquad (7)$$

$$T_2^2 = T_2^1 + T_2^* \qquad (8)$$

The approach is valid if there is no concrete cracking. If either column 1 or 2 (or both) cracks due to the resulting tensile stresses, the results obtained by restoring to equation (7) will be incorrect. The situation will be best illustrated by an example. Let:

E_C^o = initial modulus of elasticity of concrete = 4000 ksi

E^u_C = "final" effective modulus of elasticity of concrete = 2000 ksi

$E_{st} = 7.5E_C^o = 30000$ ksi

$A_C^1 = 100$ in.^{2}

$Ac^2 = 100 \text{ in.}^2$

$A_{st}^1 = 2 \text{ in.}^2$

$A_{st}^2 = 4 \text{ in.}^2$

$L = 1000 \text{ in.}$

A load P_8 = -200 kips is applied at T = 8 and kept constant.
At time $t = t_1$ a second load P_1 = +300 kips is applied.

Initial Load Distribution under $P = P_8$

$$i = \frac{1000}{100 + 2 \times 7.5} \frac{1}{E_C^8} = \frac{8.69}{E_C^8} \qquad (9)$$

$$J_i = \frac{2}{Ö3} \frac{1000}{100 + 4 \times 7.5} \frac{1}{E_C^8} = \frac{8.875}{E_C^8} \qquad (10)$$

$$T_1^8 = \frac{8.875}{1.5 \times 8.69 + 8.875} P_8 = 0.404P_8$$

$$T_2^8 = \frac{8.69 \times 0.866}{1.5 \times 8.69 + 8.875} P_8 = 0.343P_8 \qquad (11)$$

Load Distribution under $P = P_8$ Immediately Before Application

of P_1 (t_1 ® ¥)

$$f = \frac{1000}{0.5 \times 100 + 2 \times 7.5} \frac{1}{E_C^8} = \frac{15.4}{E_C^8} \qquad (12)$$

$$^{J}f = \frac{1.155 \times 1000}{0.5 \times 100 + 4 \times 7.5} \frac{1}{E_C^8} = \frac{14.45}{E_C^8}$$

$$T_{11}^{1} = \frac{14.45}{1.5 \times 15.4 + 14.45} P_8 = 0.385P$$

$$T_2^{1} = \frac{15.4 \times 0.866}{1.5 \times 15.4 + 14.45} P_8 = 0.355P_8 \qquad (13)$$

Concrete Stresses Due to $P = P_8$

Initial Stresses

$$f_{C1}^{8} = \frac{Ii^T\ 1E^8e}{L} = -704 \text{ psi}$$

$$f_{C2}^{8} = \frac{Ji^T\ 2^8E_C}{L} = -608 \text{ psi} \qquad (14)$$

$$f_{C1}^{1-} = \frac{If^T I^u E_C}{L} = -598 \text{ psi}$$

Final Stresses

$(t_1 \text{ ® ¥ })$ { $$f_{C2}^{1-u} = \frac{Jf^T 2E_C}{L} = -514 \text{ psi} \qquad (15)$$

Consequently, if t_1 is large, concrete stresses immediately after P_1 is applied would be:

$$f_C{}^{1+}{}_1 = -598 + 1055 = +457 \text{ psi}$$

$$f_C{}^{1+}{}_2 = -514 + 912 = +398 \text{ psi} \qquad (16)$$

If the tensile strength of concrete is f_t = 420 psi, then column 1 should be expected to crack, i.e., the entire load T_1 would be carried by the steel reinforcement. Under such conditions, a load redistribution would occur, resulting in a large increase in T_2, with subsequent cracking of columns 2 as well. The final load distribution will therefore depend on $A_{st}{}^1$ and $A_{st}{}^2$ only. That is to say, if both columns crack it is irrelevant whether t_1 is small or large. Moreover, $A_C{}^1$ and $A_C{}^2$ will no longer play any role in the problem. In fact:

$$I^{1+} = \frac{1000}{2 \times 30000} = 0.01667$$

$$J^{1+} = \frac{1.155 \times 1000}{4 \times 30000} = 0.00962 \qquad (17)$$

$$T_1{}^{1+} = \frac{0.00962\ P}{1.5 \times 0.01667 + 0.00962} = 0.278P$$

$$T_2{}^{1+} = \frac{0.01667 \times 0.866}{1.5 \times 0.01667 + 0.00962} = 0.416P \qquad (18)$$

It has been shown that if our sample structure is fully cracked, then creep has no influence whatsoever on the final load distribution. It may be hypothesized, however, that the concrete tensile strength in Column 2 is higher, say f_t = 1500 psi. The question is than asked, what is now the load distribution. The problem may be solved by computing the total load "C" carried by concrete in Column 1 at time shortly before $t = t_1$, and assuming that, as Column 1 cracks, load will be transferred simultaneously to joint and to the steel of column 1.

Initial Load Distribution with Concrete in Column 1 Fully Cracked

$$I\,c = \frac{1000}{2 \times 7.5} \frac{1}{E_C{}^8} = \frac{66.6}{E_C{}^8}$$

$$J_8 = \frac{1.155 \times 1000}{100 + 4 \times 7.5} \frac{8.875}{E_C{}^8} \qquad (19)$$

$$T_1 = \frac{8.875}{1.5 \times 66.6 + 8.875} P = 0.082\ P$$

$$T_2 = \frac{0.866 \times 66.6}{1.5 \times 66.6 + 8.875} P = 0.531 P \qquad (20)$$

$$T_1 = 0.385 P_8 + 0.082 (P_1 + C) - C$$

$$T_2 = 0.355 P_8 + 0.531 (P_1 + C) \qquad (21)$$

with C = -.598 x 100 = -59.8 kips we get:

$$T_1 = +2.6 \text{ kips}$$
$$t_1 \; ® ¥$$
$$T_2 = +56.9 \text{ kips} \; \} \qquad (22)$$

If t_1 is sufficiently small, it may be assumed that P_8 and P_1 are applied simultaneously at t = 0, in which case:

$$T_1 = 0.082 \times 100 = +8.2 \text{ kips}$$
$$t_1 = 0$$
$$T_2 = 0.531 \times 100 = +53.1 \text{ kips} \; \} \qquad (23)$$

The difference between (22) and (23) is not large. Note that superimposing the load distribution after creep due to P_8 (equation 13) with the load distribution corresponding to the cracked structure under P_1 leads to:

$$T_1 = -0.385 \times 200 + 0.082 \times 300 = -42.4 \text{ kips}$$
$$T_2 = -0.355 \times 200 + 0.531 \times 300 = 88.3 \text{ kips}$$
$$(24)$$

which are entirely unrealistic figures. Note also that an "exact" stress analysis for the case when t_1 is large (equations (19) and (20)) would <u>not</u> be feasible for moderately complex structures.

APPENDIX B

Earthquake Analysis

APPENDIX B

EARTHQUAKE ANALYSIS

The computation of seismic stresses was carried out on the basis of the fundamental mode of the containment structure associated with maximum response. The peak of the response curve (0.31g) for 2 percent critical damping and 0.12g peak ground acceleration was used to determine:

(1) The stress-resultant Nf at the center of the opening (in the shell without the opening) for the horizontal component of earthquake motion oriented in the direction normal to the openings.

(2) The in-plane shear stress-resultant NfP at the center of the opening (in the shell without the opening), for the horizontal component of earthquake motion oriented at 900 with the direction normal to the opening.

(3) The stress-resultant Nf at the center of the opening (in the shell without the opening) for the vertical component of motion associated with 0.12g peak ground acceleration.

The influence of the opening on the above seismic loads was evaluated as follows:

a. The stress-resultant and stress-couple distributions and, therefore, the stress-concentration factors corresponding to (1) and (3) were conservatively computed on the basis of the finite-element results for dead load.

b. The stress concentration factors corresponding to (2) were determined on the basis of Lekerkkerker's solution (Reference 11) (See Figure 3) for a shell with a hole subjected to torsion, i.e., to a pure membrane shear NfP at the location of the opening, (in the shell without the opening). Note that the stress concentration factors are slightly larger than those corresponding to the plate solution. In computing stresses at elements away from the edge of the opening, however, the stress concentration factor was assumed to decrease as in the plane solution.

In determining the values shown in Table (6) and (7) the absolute value of the contribution of the vertical component of motion (3) was added to the absolute value of the contributions of the two horizontal components [(1) and (2)].