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ASSESSMENT OF BEHAVIOR AND DESIGN OF STEEL LINERS FOR CONCRETE REACTOR VESSELS

Prepared for:
U.S. Atomic Energy Commission
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Ti-ta Lee and Orhan Gurbuz

Engineering Research Institute
Iowa State University
Ames, Iowa

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1. INTRODUCTION

The design of prestressed concrete reactor vessels (PCRV's) presents many challenges to engineers. One of the important design problems is the provision for leak-tightness of the vessel. Since the radioactive coolant must be contained within the vessel cavity, the cavity is always lined with a suitable material. In the United States and abroad, steel liners are used as the primary leakage barrier.

Because of the lack of definitive design code, different organizations here and abroad have undertaken different and individual approaches to the liner design problems. These approaches have usually included research and development programs, both analytical and experimental. The result has been an accumulation of a great deal of knowledge on the behavior and design of liners. Unfortunately much of the information accumulated is proprietary in nature, and only a limited number of publications are available on the subject of steel liner design and analysis.

Recently, ACI-ASME Technical Committee on Concrete Pressure Components for Nuclear Service issued "Proposed Standard Code for Concrete Reactor Vessels and Containments" (1)* for trial use and comment. "British Standard Specification for Prestressed Concrete Pressure Vessels for Nuclear Reactors" (2) was also published recently. Design requirements for liners for PCRV's are included in these codes.

Although both the ACI-ASME proposed code and the British Standard Specification are invaluable to designers and design reviewers of liners

* Numbers in parentheses refer to the references in Chapter 8.

alike, of necessity, the statements in these documents tend to be general and descriptive. For efficient and thorough review of a design, specific information such as capabilities and restrictions of various liner design analysis techniques, behavior of liner as influenced by design parameter variations, design review information formats and requirements, etc. should be made readily available to a reviewer.

As a part of the HTGR (High Temperature Gas Cooled Reactor) - PCRVT Technical Assistance Program, a research was conducted with a primary goal of developing a procedure for accurate independent assessment of the behavior and design of steel liners for concrete reactor vessels. Specifically, the study included the following:

1. An interpretive review of available analytical and experimental studies on liner behavior and design.
2. A comparative evaluation of the capabilities and limitations of suitable selected analytical methods to predict rapidly and accurately the liner response.
3. Development of design review information requirements and guidelines.

Results of this research program are presented in this final report. Where appropriate, reference is made to previous reports submitted during the course of this study in order to avoid repetition of detailed data. However, the main findings of the study are included in this report even though they may have been included in earlier reports.

Contents of this final report are arranged in the following order: current design practice, evaluation of current design practice, design review information requirements, and design review guidelines.

2. SUMMARY OF CURRENT LINER DESIGN PRACTICE

An in-depth, interpretive, state-of-the-art review on various problems related to liner design procedures was conducted. The results of this review is summarized in this chapter.

2.1. Design Data on Existing PCRV Liners

Available data on PCRV liners are given in Table I. Included in this table are: (a) liner assembly parameters and (b) related vessel design parameters. Sources of information from which the data in the table were gathered are given in Column (14) at the end of the table.

Table I includes only the vessels which are considered to be prototypes. Vessels which are similar to the prototypes (with minor modifications) and their respective prototypes are listed below:

<u>Vessel</u>	<u>Prototype</u>
France - St. Laurent II	EDF-4
Great Britain - Hunterston	Hinkley Point B
Great Britain - Heysham	Hartlepool

Table I does not include those being constructed or designed in other countries.

Following conclusions can be drawn from an evaluation of the available information:

- a. Liner parameters show a substantial variation among the different PCRV's designed to date.
- b. Vessel design parameters (mainly, pressure and thermal gradient), which essentially control the nominal design strains or stresses for liners, also show substantial variation among different

Table I. General data on PCRV liners.

1 No.	2 PCRV (country)	3 Inside shape	4 Inside diam (ft)	5 Inside height (ft)	6 Normal operating pressure (psi)	7 Design pressure (psi)	8 Effect. temp at or near liner ($^{\circ}$ F)	9 Thermal gradient across the wall ($^{\circ}$ F)	10 Liner thickness (in.)	11 Liner steel properties	12 Anchor type and spacing	13 Cooling system	14 References
1.	G2 & G3 (France)	Horizontal cylinder	42.5	62.5 (length)	220	220	122	54	1.18	*	0.79 and 1.57 in. round bars	Gas cooled	3, 4, 5
2.	EDF-3 (France)	Vertical cylinder	58	64.5	384	423	167	90	0.98 - 1.38	Soft steel	3.15 x 0.49 in. bars spaced at about 12 in.	Cooling tubes welded to the liner	3, 4, 6
3.	EDF-4 (France)	Vertical cylinder	58	110.5	384	423	122	63	0.98 - 1.38	Soft steel	Same as EDF-3	Cooling tubes welded to the liner	3, 4, 6, 7
4.	Oldbury (Gr. Britain)	Vertical cylinder	77	60	350	385	149	54	0.44 - 0.5	Mild steel	Bottom: 3 x 3/8 in. tees at 12 in. and angles at 60 in. Wall: vertical 4 x 12 in. ribs at 12 in., ties at 12 in., circumferential ribs near top and bottom	Cooling pipes welded to the ribs. At the bottom tees carry the pipes	3, 5, 8, 9, 10, 11
5.	Wylfa (Gr. Britain)	Spherical	96	-	384	423	113 6 in. from liner	27	0.5 - 1.25	$\sigma_y^{(c)} = 37 - 40$ ksi	6 x 3 x 5/8 in. meridional angles at 24 in. max. Special provisions at the bottom	Cooling pipes welded to the liner	3, 9, 11, 12, 13, 14
6.	Dungeness B (Gr. Britain)	Vertical cylinder	65.5	58	435	478	131	63	0.5 - 1.5	$\sigma_y = 42 - 50$ ksi	Bottom: 6 x 1/2 in. ribs at 10 in. Wall: closely spaced studs Top: studs of larger diameter (outside the central region)	Cooling pipes welded to the ribs at the bottom. Cooling pipes clipped to the liner on the wall	5, 15
7.	Bugey I (France)	Vertical cylinder	52	125.5	637	676	158	63	1.0 - 1.38	Ductile steel	*	Cooling tubes welded to the liner	4, 16
8.	Fort St. Vrain (U.S.A.)	Vertical cylinder	31	75	688	845 ^(a)	130	50	0.75	$\sigma_y = 56 - 71$ ksi	Stud anchors at 7 1/2 in. square pitch, closer spacing in the haunch area.	Circumferential cooling tubes welded to the liner	17, 18, 19, 20
9.	Hinkley Pt. B (Gr. Britain)	Vertical cylinder	62	63.5	560	* ^(b)	*	*	0.5	Mild steel	Stud anchors spaced at 6 to 9 in.	Cooling pipes welded vertically	10, 11
10.	Hartlepool (Gr. Britain)	Multi- penetration	Main: 43 Boiler liner: 9	60	575	644	158	*	0.75 - 1.0	Mild steel	Stud anchors	*	11, 13, 21

(a) Reference pressure.

(b)* Indicates that the information is not available.

(c) Liner yield stress.

designs. The earlier vessels were designed for a relatively low pressure; whereas the operating pressure of the advanced PCRV's is around 500-700 psi. It is anticipated that the use of PCRV's for GCFR's (Gas Cooled Fast Breeder Reactors) will further increase the operating pressure to 1250 psi.

- c. The British and French design practice has been to use structural carbon steel for liner plates. In the Fort St. Vrain reactor, heat-treated carbon-manganese-silicon steel was chosen as the liner material.
- d. Anchor types and spacing varies among designs. The more recent British designs and the Fort St. Vrain design provide stud anchors whereas the usual French practice has been to use bars. Anchor spacing-liner thickness ratio varies from 10 to 24 or more.
- e. In earlier designs cooling tubes or pipes are either welded to the liner, or the anchor ribs, or not welded at all. In the more recent designs, however, cooling tubes are always welded to the liner.

2.2. Modes of Failure

The failure modes considered in different designs include:

- a. buckling
- b. excessive deflections
- c. rupture of liner
- d. rupture of anchor
- e. concrete failure

- f. high strain-low cycle fatigue
- g. brittle failure due to irradiation.

The most important one among these failure modes is considered to be buckling and the ensuing excessive deflections (3). For this reason, prevention of liner buckling by limiting the anchor spacing - plate thickness ratio has been a major design criterion (8, 12, 15, 20).

Design against rupture of anchor involves limiting anchor stress under extreme conditions (20). Rupture of liner may be prevented by limiting anchor size-liner thickness ratio (22). Concrete failure may be avoided by limiting stresses in concrete (5, 20). High strain-low cycle fatigue is considered highly unlikely to occur (12, 23). Exposure to radiation increases the nil ductility temperature (NDT) substantially (23), and thus selection of low NDT steels for liners is considered to be of primary importance (18). Irradiation increases the yield strength of steel. This fact is to be taken into account in anchor system design (2).

2.3. Design Bases

2.3.1. Design Analyses

The most important structural consideration in the design of liners without anchors (such as penetration liners) is the stability of the liner shell. The diameter-thickness ratio of the shell should be such that buckling (elastic or inelastic) of the liner will be prevented under all possible loading conditions. Thus, the thickness of the liner depends on stability considerations for a given cavity size. Stability problems in liners are discussed in the next subsection.

In the design of liners with anchors, the usual practice is to first select the liner thickness and cooling tube spacing considering the construction and cooling requirements (11). The anchor spacing, in the direction perpendicular to the cooling tubes, is in turn determined by the cooling tube spacing since stud anchors are usually placed between cooling tubes. The anchor spacing in the other direction is usually the same as above, thus forming a square anchor pattern.

Design analyses of liners with anchors involves consideration of both stability and stress problems. The former is treated in the next section, together with stability problems in unanchored liners. The latter is discussed below.

For stress analysis it is possible to include the liner in the analysis of the entire vessel, representing it as a shell element subject to membrane forces (10, 11). However, it is not practical to include shear anchors in such an analysis (10). Furthermore, it is difficult to take into account local buckling and variations which affect behavior of the liner in the analysis of a complete vessel. For these reasons, stress analysis of the liners has been considered as a separate problem, divorced from the analysis of the vessel (10, 19).

In this "uncoupled" approach, the first step is to determine liner "design" stresses or strains from an analysis of the vessel under different loading conditions. Finite difference (8, 15), dynamic relaxation (14), and finite element (20) methods have been used for this purpose. In the analysis of the vessel, the liner may or may not be taken into account (10, 20). When the liner is included in the analysis, it is assumed that full strain compatibility exists between the liner and the concrete surface.

The second step in this approach involves stress (load-dissemination) analysis of a segment of the liner. For this analysis, stiffness characteristics of liner components are required. Stiffness characteristics of anchors and cooling tubes are determined experimentally (10, 20). In British practice, the liner characteristics are usually based on experimental results also (10). In the Fort St. Vrain design however, theoretical stiffness of the liner was used in the load dissemination analysis.

The load dissemination analysis results in the final design stresses or strains for anchors, cooling tubes, and liner plates. A description of stress analysis methods is given in Subsection 2.3.3.

2.3.2. Stability Problems

2.3.2.a. Nature of the Problem

Liners in prestressed concrete reactor and containment vessels are in biaxial compression under most loading conditions. For this reason, buckling of liners has been a major design consideration (24).

Compressive stresses are introduced into the liner in two different ways:

- a) Compressive straining as a result of reduction in length or diameter (e.g., due to prestressing).
- b) Compressive straining due to cavity restraint when the liner temperature increases.

It has been shown that for a ring strip in a rigid cavity these two different ways of loading result in the same buckling strain (25). In view of this no differentiation will be made in regards to how the strains are introduced in the following discussion.

The behavior of thin, concrete-encased shells is quite different from that of shells without concrete encasement (26). Therefore, buckling studies on thin shells (without concrete support) cannot be realistically applied to shells which are encased. In this study only concrete-encased shells are considered.

The behavior of liners anchored to concrete with discrete (e.g., studs) and continuous (e.g., welded angles, tees) ties is also different from that of liners without anchors. These two cases are treated separately in the following discussion.

2.3.2.b. Buckling of Liners without Anchors

If the liner is not anchored to the surrounding concrete, the failure under compressive stresses will be of a general instability type wherein a large portion of the shell collapses.

In order to determine the magnitude of uniform strains at which general instability occurs, a strip of the liner (usually a ring or a ring segment), supported by a rigid cavity, has been used as the analytical model. The strain at which there is an alternate (buckled) equilibrium position is determined for this model. This strain, which is termed minimum buckling strain, then may be used as a design criterion; i.e., if the computed strain is greater than the minimum buckling strain, anchors should be provided. Studies in which the above approach has been adopted is summarized in Ref. 27.

As an example, the results given by Chan and McMinn (28, 29) will be discussed here. The buckled form of a ring considered by Chan and McMinn is shown in Fig. 1. Curves showing the relations between uniform strains and angle subtended by the buckled length are shown in Fig. 2. Referring to Fig. 2 it was concluded that:

- a) For small ϵ_0 (below point b), where ϵ_0 is the uniform circumferential strain in the ring, there is only one possible equilibrium position, namely the unbuckled state.
- b) For ϵ_0 beyond b, three equilibrium positions are possible. These are the unbuckled shape, and the buckled shapes along ab and bc. However, the position along ab, which involves small deflections, is unstable and if there are no restraints, the strip will take the position along bc which is stable.
- c) The equilibrium position is independent of the modulus of elasticity.

It should be noted that some external disturbance (i.e., initial imperfection) must exist for the strip to snap into the alternate equilibrium position. It was stated that, as an example, random vibrations (such as that due to motion of circulators in a penetration) may cause the liner to leave the backing surface far enough to reach the unstable position.

In order to confirm their analysis, Chan and McMinn conducted some tests on steel strips, backed by a rigid cavity. The experimental results showed somewhat lower buckling strains than those predicted, the difference being attributed to friction. Sometimes "explosive" buckling was observed in their tests. Such buckling was stated to be quite unpredictable.

From a design viewpoint, curves such as those shown in Fig. 2 are of practical value. As an example, if the radius-liner thickness ratio of a cylindrical vessel is 400, the minimum buckling strain is about 520 $\mu\text{in./in.}$ Therefore, if the design strains are expected to exceed

that value, anchors should be provided in order to prevent buckling. This example, of course, does not consider any factor of safety nor imperfections in the vessel which influence buckling response.

It is to be noted that the theory given by Chan and McMinn was also supported by the tests conducted at Case Western Reserve University (27, 30). In these tests, thin shells encased in thick concrete shells were strained thermally. It is stated that one specimen, in which there were no anchors attached to the thin shell, gave results which agree very well with the theory presented by Chan and McMinn.

2.3.2.c. Buckling of Liners with Anchors

If the liner is anchored to backing concrete, the failure mode will no longer be general instability. Instead, the following three types of panel buckling should be considered (27, 30):

- a) Circumferential buckling (ring mode)
- b) Axial buckling (strip mode)
- c) Combined buckling (lobar mode).

From a practical point of view, possible effects of buckling of anchored liner rather than the buckling phenomenon itself are of importance because:

- a) Buckling may result in excessive lateral deflections which may interfere with functional requirements of vessel components.
- b) Tensile stresses may develop in the buckled region which, in combination with increased brittleness due to irradiation, may lead to brittle fracture of the liner.
- c) Inplane load carrying capacity of buckled sections will be decreased due to bending, resulting in differential forces between buckled and unbuckled regions. These forces will

cause shear forces in the anchors which must be evaluated for proper anchor design.

The case of stud-anchored liners has been studied theoretically and experimentally by Kicher and his associates at Case Western Reserve University (25, 27, 30). Solutions for two models of liner elements have been obtained: a model of a two-dimensional cylindrical shell element (panel) supported by rigid studs and a ring model (with finite or infinite radius of curvature) representing a segment of a liner supported by elastic studs. The main results obtained from the first model are the minimum buckling strains while the second model, additionally, gives anchor forces.

In the panel model, the liner is assumed buckled in the ring, strip or lobar modes. It was found that the minimum buckling strain for the ring mode is always less than or equal to minimum buckling strain for any other mode. The conclusion was that, in a cylindrical shell, the minimum buckling strain is independent of the axial spacing of studs. This conclusion is valid so long as the axial strain is equal to or less than the circumferential strain.

Minimum buckling strain in the ring mode for a specific example is given graphically in Fig. 3 based on data from Ref. 30. In this example the radius (R) to thickness (h) ratio, R/h , is 750 with h equal to 0.5 in. It is to be noted that for lower R/h ratios the predicted minimum buckling strains would be higher. Similar data using the second model (e.g. liner strip with flexible anchors) are not available.

Chan and McMinn also gave an approximate equation with which the required spacing of anchors to prevent buckling can be determined. This

approximate expression is based on buckling of the flat strip as a strut in the third mode (see Fig. 1) and agrees fairly well with more refined theories. The expression is

$$a = \frac{2.5 h}{\sqrt{\epsilon_c}} \quad (1)$$

where a is anchor spacing, h is liner thickness, and ϵ_c is uniform design strain. The minimum buckling strain obtained from the above formula is also shown graphically in Fig. 3 for comparison.

2.3.3. Stress Analysis Methods

The term "stress analysis methods" as used here refers to those techniques with which a section of the liner assembly is analyzed to determine forces and displacements under an assumed loading condition. Thus, it excludes techniques which are concerned with the stability conditions of the idealized model.

The main methods found in the literature are (31):

1. One-dimensional analysis:

- a) Parker's method (8, 10)
- b) Doyle and Chu's method (32)
- c) Gulf General Atomic's (GGA) method (19, 20)
- d) Bechtel's method (33)
- e) Iterative Force Distribution method (31)
- f) Initial Stress method (34).

2. Two-dimensional analysis:

The finite element method developed at the Franklin Institute Research Laboratories (35).

Among the methods listed above, Parker's and GGA's methods have been used in the design of PCRV liners. Doyle and Chu's, Bechtel's, and the two-dimensional methods have been applied to the design of containment vessel liners. Iterative Force-Distribution and Initial Stress methods were developed during the course of this study.

All methods in the category of one-dimensional analysis consider an idealized segment of the liner either in the circumferential or in the meridional direction. The model consists of a strip of liner and a series of anchors (Fig. 4). In the two-dimensional analysis, a section of the liner together with anchors and surrounding concrete is considered.

It should be noted that the main differences among the one-dimensional methods are solution techniques employed, assumed component behavior, and assumed boundary conditions. A brief description of all stress analysis methods is given in the following paragraphs.

For convenience, in the discussion hereafter, segments of liner between anchors are referred to as "panels." Those panels which are assumed to have lower yield point and modulus of elasticity are referred to as "weak" panels, the remainder being named "strong" panels (see Fig. 4a).

2.3.3.a. Description of Individual Methods

a) Parker's Method: The analysis procedure involves the following steps:

1. Determine biaxial design strains from vessel analysis.
2. Determine weak panel characteristics and anchor characteristics from test results. Both weak and strong panel characteristics are idealized as elastic-perfectly plastic.

3. Using the recurrence relationship of the type

$$F_{i+1} - c_1 F_i + c_2 F_{i-1} = c_3 \bar{F}_i \quad (2)$$

set up a set of simultaneous equations. In the recurrence formula:

F_i is the final force in the i th panel; c_1, c_2, c_3 are constants related to panel and anchor stiffness and curvature; $\bar{F}_i$ is the i th panel design force based on design strains (Fig. 6b). This recurrence formula is obtained by substituting equilibrium relations and panel and anchor stiffnesses into the compatibility equation for node displacements. (See Ref. 10 or 24 for derivation.)

4. Since the material properties are nonlinear, solve the simultaneous equations for the final panel forces by a step-by-step approach.

5. The anchor forces are then determined using equilibrium conditions at the nodes $S_n = F_{i+1} - F_i$, where S_n is the force in the anchor between panels i and $i + 1$.

As this brief description indicates, Parker's method is a flexibility approach in which the recurrence equations are compatibility conditions in terms of unknown panel forces.

b) Doyle and Chu's Method: Although this method has been used in the design of containment vessel liners, it should be applicable to PCRV liners as well. The procedure is summarized as follows:

1. Assume that all panels are initially at yield and that one panel buckles. Since the load-axial deformation characteristics of panels initially at yield are not known (32), the strut analogy is used for the weak panel, i.e., the buckled panel.
2. Assume that the model is symmetrical with respect to the weak panel. Thus only one-half of the section need be analyzed (Fig. 4b).

3. The equilibrium conditions applied at each node result in the following set of equations:

$$u_1 = u_2 + \frac{a(\sigma_o - \sigma_{pb})}{E} - \frac{a}{Ebh} S_1 \quad (3)$$

$$u_n = \frac{u_{n+1} + u_{n-1}}{2} - \frac{a}{2Ebh} S_n \quad (4)$$

where: u_n = displacement of the nth anchor, S_n = force in the nth anchor, σ_o = initial panel stress, σ_{pb} = final stress in the buckled panel, h = liner thickness, a = anchor spacing along the model, b = anchor spacing perpendicular to the model, and E = modulus of elasticity.

4. Since the above set of equations involve nonlinearity, an iterative procedure is used to determine anchor displacements. Experimentally obtained anchor force-deformation characteristics are used.

5. The final force in any unbuckled panel is given by

$$F_i = F_i^o - Ehb \frac{u_{n-1} - u_n}{a} \quad (5)$$

where: F_i = compressive force in panel i , F_i^o = initial force in panel i , and the other variables are as defined above.

c) Gulf General Atomic's Method: The method used by Gulf General Atomic is described in Ref. 19 without detailed formulation. The procedure given is as follows:

1. Determine the initial panel forces using design strains obtained from analysis of the PCR.V.

2. Assume that all panels remain elastic and that there is no "weak" panel. Determine inplane stiffnesses of panels by theoretical calculation. The panels are called "main springs."

3. Define anchor stiffnesses, using linear segments to describe the experimentally obtained load-deformation characteristics. The

anchors are termed "side spring." (The anchor system includes both studs and cooling tubes.)

4. With the initial panel forces and the main and side spring stiffnesses thus determined, an analysis is made using "NONLIN" computer code which, presumably, is based on stiffness formulation of the nonlinear problem.

5. The NONLIN computer program gives the final anchor forces.

d) Bechtel's Method: The one-dimensional model used in Bechtel's relaxation method is shown in Fig. 5a. The method is developed for design of containment vessel liners. A brief summary of the development of theory is given below:

Assuming that anchor movements are completely prevented, the stress in the strong panel will be:

$$\sigma_{\theta} = \frac{E}{1 - \mu^2} (\epsilon_{\theta} + \mu \epsilon_z) \quad (6)$$

where: σ_{θ} = circumferential stress, ϵ_{θ} = circumferential design strain, ϵ_z = axial design strain, and μ = Poisson's ratio. The computed stress is assumed to exist regardless of whether it exceeds the actual yield point. This assumption implies that all strong panels remain elastic. The initial membrane force is:

$$N = h \sigma_{\theta} \quad (7)$$

If the anchor at one end of the weak panel is released, the displacement at this anchor (anchor 1) will be

$$\delta_{11} = \frac{N}{K_C + K_B + K_R} \quad (8)$$

where: δ_{ij} = displacement at anchor i due to release of anchor j, K_C = anchor stiffness, K_B = weak panel stiffness, and K_R = strong panel stiffness.

Now assume that the second anchor (anchor 2) is released, leaving the first anchor free to deform. It can be shown that the total displacement at anchor 1 due to movements at anchors 1 and 2 will be

$$\delta_{11} + \delta_{12} = \frac{N}{K_C + K_B + K_R} (1 + B) \quad (9)$$

where:

$$B = \frac{K_R^2}{(K_C + K_B)K_R + (K_C + K_R)(K_C + K_B + K_R)}$$

In a similar manner, the total displacement at anchor 1 due to movements at anchors 1, 2, and 3 is found to be

$$\delta_{11} + \delta_{12} + \delta_{13} = \frac{N}{K_C + K_B + K_R} (1 + B + B^2). \quad (10)$$

In obtaining the last relationship, the following term appears in the denominator and is disregarded, conservatively, since it is greater than unity for the range of variables involved:

$$A = \frac{1}{K_C + K_B + K_R} \left[(K_C + K_R) + \frac{K_R [(K_C + K_B)K_R + K_C(K_C + K_B + K_R)]}{(K_C + K_B)K_R + (K_C + K_R)(K_C + K_B + K_R)} \right] \quad (11)$$

Considering an infinite number of anchors, the total displacement at anchor 1, δ , will be

$$\delta = \frac{N}{K_C + K_B + K_R} \left[1 + \sum_{n=1}^{\infty} B^n \right] \quad (12)$$

Now let

$$N' = N \left[1 + \sum_{n=1}^{\infty} B^n \right] \quad (13)$$

Then the problem is reduced to solving the following relationship:

$$\delta = \frac{N'}{K_C + K_B + K_R} \quad (14)$$

The actual computation involves the following steps:

1. Determine biaxial strains from vessel analysis.
2. Determine K_C and K_B from test results. (K_C is taken to be the initial slope of the load-displacement curve and K_B is twice the initial slope since both ends of the weak panel move toward each other.) Calculate $K_R = Eh/a$ (ignoring the effect of Poisson's ratio).
3. Determine imaginary force N' to be applied at anchor 1.
4. Find δ , displacement at anchor 1 due to movements of all

anchors:

$$\delta = \frac{N'}{K_C + K_B + K_R} \quad (15)$$

In determining δ , first solve for δ using initial stiffnesses for K_C and K_B . If the computed value of δ exceeds the elastic limit for any component, then a plastic analysis is required. For example, if δ thus computed exceeds the elastic limit, δ_e , for the anchor, the above equation is rewritten as

$$\delta_e K_C + (\delta - \delta_e) K'_C + \delta(K_B + K_R) = N' \quad (16)$$

where K'_C is the anchor stiffness in the inelastic range, and the equation is solved for δ .

5. Using the displacement thus obtained, the anchor force in anchor 1 is obtained from

$$F_C = K_C \delta \quad (17)$$

if $\delta \leq \delta_e$; or a similar relation, if $\delta > \delta_e$. For example, for the case illustrated in step 4, the anchor force would be

$$F_C = \delta_e K_C + (\delta - \delta_e) K'_C \quad (18)$$

As evidenced from the discussion here, Bechtel's method gives the anchor force in anchor 1 only. Forces at other anchors and in liner panels can be determined once the force in anchor 1 is known.

e) Iterative Force-Distribution Method: The compatibility conditions between a panel and two adjacent anchor may be expressed by (31)

$$\epsilon_{si} = \epsilon_{ci} + \frac{u_n - u_{n-1}}{a_i} \quad (19)$$

where: ϵ_{si} = average biaxial liner panel strain, in panel i positive if compressive,
 ϵ_{ci} = average biaxial design strain, positive if compressive,
 u_n = relative tangential displacement at node n , positive if toward negative x -direction (see Fig. 6), and
 a_i = panel length.

The characteristics of panels and anchors are given by:

$$F_i = f_i(\epsilon_{si}) \quad (20)$$

and

$$S_n = g_n(u_n) \quad (21)$$

where: F_i = panel force per unit width for panel i , positive if compressive,
 S_n = shear force per unit width for anchor n , positive if in positive x -direction, and
 $f_i(\epsilon_s)$ and $g_n(u_n)$ are nonlinear stiffness functions for panel i and anchor n , respectively.

The equilibrium equation at node n is

$$S_n = F_{i+1} - F_i \quad (22)$$

The liner analysis problem is thus reduced to the solution of a set of nonlinear simultaneous equations defined by the above four equations.

In the technique developed, the nodes are fixed and nodal forces necessary for equilibrium (due to differential panel strength and/or design strain) are determined. The nodes are released one at a time, distributing negative of the nodal (holding) forces at the node. After the completion of distribution of nodal forces, a second set of holding forces are computed at the nodes and the procedure is continued until all holding forces become negligible. Details of the solution technique were described in Ref. 31.

The actual computation involves the following steps:

1. Determine the biaxial design strains from vessel analysis.
2. Determine the weak panel and anchor characteristics.
3. Determine the geometry of the model to be analyzed (e.g., number of panels, boundary conditions, etc.).
4. Using the above as input data, final forces and displacements are obtained from a computer program which will carry out the calculation using the solution technique described in the preceding paragraph.

f) Initial Stress Method: The model used in the initial stress method is the same as the one used in the iterative force-distribution method. The same nonlinear simultaneous equations defined in e) are solved using "initial stress" method of nonlinear analysis (34).

The actual steps involved in the analysis are also the same. Details of the theory and a computer program developed for stress

analysis using the initial stress method are included in Ref. 34 and will not be repeated here.

g) Finite Element Method: Unfortunately, details of the finite element method, developed at the Franklin Institute Research Laboratories, are not available. The significant differences between this approach and one-dimensional analysis appear to be the following:

1. One-dimensional analysis shows that the number of panels included in the model has an appreciable effect on the results. The predicted anchor forces and displacements increase with increasing number of panels (31). In the finite element approach, which was used in the design of the Fort Calhoun PCCV liner, only five panels and four anchors were considered. The accuracy of the method, using relatively fewer panels, should be evaluated.
2. In finite element approach, stiffnesses of all elements are determined theoretically (assuming lower modulus of elasticity and lower yield point for the weak panel). In the one-dimensional analysis experimentally determined anchor and weak panel characteristics are usually used.
3. The finite element method takes into account concrete behavior behind the liner and anchors. In one-dimensional methods, on the other hand, the effect of concrete behavior is assumed to be indirectly taken into account by using experimentally determined component characteristics.
4. In the two-dimensional analysis, there is a need to use a yield criterion for steel and concrete because of the existence

of a biaxial stress field. In one-dimensional analysis, obviously, such a problem does not exist.

h) Other Methods: Except for e) and f), the stress analysis methods briefly described above are those which have been actually used in the design of reactor or containment vessel liners. There are other methods used in the design of liners which are not discussed in this report (15, 36-38).

2.3.4. Other Structural Considerations

In the preceding section, methods with which an idealized model may be analyzed to determine forces and displacements in the liner components were described. It is essential that the behavior of the components be evaluated in the light of predicted forces and displacements and other structural considerations. Such an evaluation will also be helpful in justifying and/or improving the criteria used in idealizing the model.

2.3.4.a. Assumed Behavior of Liner Panels

a) Strong Panels: In the iterative force distribution and initial stress methods of analysis briefly discussed in the preceding section, strong panel behavior is assumed to be elastic-perfectly plastic. The same assumption was made by some British designers (10, 11).

In the method used by Bechtel for containment liner analysis, it is assumed that strong panels remain elastic, regardless of the magnitude of design strains (33). A third approach, adopted by Doyle and Chu in their method of analysis which was also developed for containment liners (32), assumes the panels to be initially yielded. This implies that design strains beyond yield have no effect on the model behavior.

These three different approaches will result in different anchor forces and displacements if the design strains (ϵ_{ci}) are greater than the yield strain (ϵ_y) for the liner material. This point is illustrated in Fig. 7, using a specific example (31). In this example, using either Bechtel's approach or the elastic-perfectly plastic approach has resulted in much larger anchor forces than those based on Doyle and Chu's approach, if the design strains are beyond the yield strain of the weak panel. It was shown (31) that this difference is largely dependent on the assumed behavior of the weak panel. If the weak panel is assumed to carry no load, the difference between the elastic-perfectly plastic assumption and Doyle and Chu's approach is smaller, the latter still resulting in smaller anchor forces.

b) Weak Panels: The behavior of an assumed "weak" panel in the analytical models are determined either experimentally (10, 33) or theoretically using strut analogy (12, 15, 24, 32). The experimental studies conducted for this purpose are discussed in Subsections 2.5.1 and 3.4.1.

The use of the load-axial deformation relation of a column with rectangular cross section in place of that of an actual panel is called "strut analogy" in various publications. By introducing eccentricity and/or initial deflection, load-deformation relations for struts can be analytically determined (24, 31, 39-43).

In the course of this study a computer program was developed, based on Jezek's work, for the analysis of fixed-ended and simply-supported struts. In this program, the effect of uniform lateral loads (e.g., simulating internal pressure and water pressure behind

the liner) was included. Details of the theory and computer program were included in Ref. 31. It is to be noted that similar studies have been made in connection with the design of the Dungeness B Vessel in the United Kingdom (15), but no detailed formulation is available.

In the design analysis process the characteristics of weak panels, whether obtained from tests or analytically, are usually idealized. In some British designs this idealization results in elastic-perfectly plastic behavior, assuming no load drop-off, as shown in Fig. 8. It is to be noted that both the initial stiffness and the imaginary yield point of a weak panel are less than those of a strong panel.

Other investigators have assumed load-dropoff in the weak panel by making use of strut analogy (31, 32) or test results (11, 33). In the parametric studies conducted in the course of this research program, strut analogy was used and weak panel characteristics were described by a tri-linear approximation (31). This is also shown in Fig. 8.

2.3.4.b. Forces on Cooling Tubes

In one-dimensional analysis methods, the weak panel is usually assumed to occur between adjacent stud anchors, not between adjacent cooling tubes or between a cooling tube and the nearest stud. The frictional force between the tubes and concrete is considered to be insufficient and unreliable to resist the movement of the cooling tube in the direction normal to the liner surface, if such a tendency exists. Based on this reasoning, buckling or inelastic bending between cooling tubes is never considered in British design (43).

The shearing resistance of cooling tubes in the plane of the liner have been taken into account in some British designs (43) and in the Fort St. Vrain design (19, 20).

The shear stiffness of the cooling tubes for the Fort St. Vrain design is greater than that of stud anchors. Thus, if the tubes are included in the design analysis, the cooling tubes are predicted to carry a greater percentage of the total shear forces than the stud anchors (31).

The effect of both types of shearing forces discussed above were evaluated in the design of cooling tubes (19). Available data shows that in the Fort St. Vrain design stresses induced in the cooling tube walls and fillet welds are lower than allowable values (19).

2.3.4.c. Forces on Stud Anchors

In most load dissemination analysis methods, only inplane (shear-slip) characteristics of stud anchors have been considered (10, 12, 20, 31, 33). It was stated that the actual stress field in the anchors is very complicated and thus its behavior is best determined by experimental studies (10). For this reason, experimentally obtained stud shear-deformation data have been used in all designs. Stud anchor fatigue behavior was also experimentally determined (44) and the resulting data was used in design (20).

The load dissemination analysis methods considered in past designs give the anchor shear forces. In addition to shear forces, there will be axial tensile forces on the anchor due, mainly, to inelastic bending of a weak panel and water pressure behind the liner. The interaction of axial and shear forces does not appear to be discussed in past designs.

The embedment length of the anchors should be such that, if it were stressed to ultimate, failure would occur by rupture of the anchor (20). Tests have shown that, for concrete strengths of about 3000 psi, an embedment length of about nine stud diameters is sufficient to develop the full anchor strength (45).

Another consideration in the design of anchors is the effect of temperature on concrete strength and hence on the required embedment length of stud anchor. It was stated that temperatures up to 200 °F will have negligible effect on the strength of concrete (20).

2.4. Design Criteria

2.4.1. Loads

It was mentioned in Subsection 2.3.1 that the usual procedure in liner design is to first determine the liner design stresses or strains from overall vessel analysis and then to analyze the liner using design stresses or strains as the loading. In liner design analysis, local variations listed in Subsection 2.4.3 are also taken into account.

The types of loads considered in determining the liner design stresses or strains are:

- a. dead loads
- b. prestressing loads
- c. creep and shrinkage of concrete
- d. thermal loads
- e. internal pressure
- f. live loads
- g. wind loads.

The effects of the last two types of loads are considered during construction of the structure only. It is to be noted also that earthquake effects do not appear to be considered in past liner designs.

General discussion on these loads are given in the following paragraphs. Specific loading combinations are discussed in Subsection 2.4.2.

2.4.1.a. Dead Loads and Live Loads

The effects of dead and live loads during construction period are seldom discussed in the literature. It is stated that these loads would depend largely upon the method of construction and, thus, stress analysis at various stages of the construction is essential (20). Residual stresses induced in the liners during the construction (before the concrete has gained its strength) are not considered (12, 20), although compressive residual stress is considered to have a beneficial effect on fatigue strength (12).

Dead load stresses in the completed vessel (after concrete has gained its strength) were considered in the design of the Fort St. Vrain PCRV liner. Maximum average strain was shown to be on the order of 20 μ in./in. (46), which may be considered small in comparison with strains due to other loads. Live load stresses were negligible in the completed structure but were considered during construction period (20).

2.4.1.b. Wind Loads

Wind loads are normally considered during construction of the vessel only.

2.4.1.c. Prestressing Loads

The magnitude of required prestress in a vessel is based on design criteria adopted for the vessel. The design criteria, including the

magnitude of design pressure differ substantially among the vessels constructed. Furthermore, different prestressing systems have been used. As a result, magnitude and gradient of imposed liner strains under prestress vary considerably from design to design.

2.4.1.d. Creep Loads

Creep is defined as the increase in concrete strain under sustained loads. Although creep and shrinkage are interrelated, their effects are usually considered separately.

The magnitude of creep-induced strains (and stresses) in the liner are considerable and time- and temperature-dependent. For example, in the Fort St. Vrain liner design analysis, strains in the liner due to creep were twice (after five years) and three times (after 30 years) as much as the effective prestressing strain (46).

2.4.1.e. Shrinkage Loads

The amount of concrete shrinkage is usually determined based on test results. As indicated above, shrinkage strain in the concrete is considered additive to creep strain. In the Fort St. Vrain analysis, shrinkage-induced strains in the liner were assumed to be 150 $\mu\text{in./in.}$ and the resulting stress (4000 psi) was simply added to analytical results obtained from other loads (20).

In British practice, strains in the liner due to concrete shrinkage were assumed to be as high as 400 $\mu\text{in./in.}$ (13, 47).

2.4.1.f. Thermal Loads

Although the temperature in the liner plate is nonuniform, for design purposes, a uniform "effective" (design) liner temperature is usually assumed. The effective liner temperature, together with the

boundary conditions on exterior surface of the vessel, determine the thermal gradient across the vessel wall. The effect of thermal gradient is to induce further compressive strains in the liner. The magnitude of these strains have usually been determined from axisymmetric analysis of the vessel (10, 20).

Thermal effects due to restrained thermal expansion of the liner are also considered in liner design.

2.4.1.g. Internal Pressure

The effect of internal pressure, ranging from vacuum to design pressure, is considered in the design of liners. Internal pressure normally causes tensile stresses in the liner and thus is not included in the loading combination from which maximum design compressive strains in the liner are determined. However, variation in liner design strains due to pressure fluctuation is important for fatigue considerations. The possibility of tensile stresses in the liner under internal pressure, in combination with other loads, has been investigated. In the Fort St. Vrain design, it is concluded that the liner would be in biaxial compression throughout the vessel's design life (48). In the Hinkley Point B design, on the other hand, small tensile stresses in the liner were predicted under internal pressure, when the liner was assumed to be cold (10).

2.4.2. Loading Combinations

The design stresses and/or strains for the liners are obtained from a combination of the loads discussed in Subsection 2.4.1. The loading combinations used by British designers and Gulf General Atomic are summarized in Table II. Information regarding the criteria adapted by French designers is not available.

Table II. Liner design criteria - loading combinations.

Item	British criteria (a)	Gulf General Atomic (b)
1. Design basis	Nominal design strains obtained from an analysis of the vessel	Same
2. Construction category loading combinations		
a. During construction	Not known	$D + L + W$
b. After the completion of prestress	F_t	$D + F_t$
c. Design temperature	$F_t + T_d$	$D + F_t + T_d$
d. Design temperature and test pressure	$F_t + P_t + T_d$	$D + F_t + P_t + T_d$
3. Normal category loading combinations		
a. Normal operation	$F_t + P_o + T_o$	$D + F_t + P_o + T_o$
b. End of operation, with reference pressure	-	$D + F_t + P_r + T_o$
c. End of operation	-	$D + F_t$
<u>Selected references</u>	8-12, 15, 50	17-20, 46, 49

(a) It is not known whether British designers have taken into account the dead load effects.

(b) GGA design analysis includes additional load combination at different time points in vessel's life. These additional combinations are not included in the table since they are obviously not critical.

Notation:

D = dead loads,

F_t = prestressing forces at time of consideration (includes time-temperature dependent creep and shrinkage effects),

L = live loads,
 P_o = normal operating pressure,
 P_r = reference pressure,
 P_t = pre-operational test pressure,
 T_d = design temperature,
 T_o = normal operating temperature, and
 W = wind loads.

The loading combinations shown in Table II are based on the expected sequence of loading conditions the vessel experiences during and after the completion of construction. In the design process, the liner is analyzed under strains due to these loading combinations and the resulting stresses or strains are compared with appropriate allowable stresses or strains. This "working stress" approach does not seem to be supplemented by "limit" or "ultimate strength" design considerations. One exception is the design of Fort St. Vrain PCRV liner in which liner anchoring system was designed to ensure compatibility between the liner and concrete up to Limit Condition I* (20).

2.4.3. Local Variations

Types of postulated local variations and loads, and their combinations with design strains under different loading conditions, have varied considerably among past liner designs. The variations considered by British and GGA designers are shown in Table III.

There appears to be a significant difference between the local effects taken into account in stress analysis after the design stresses

* Limit Condition I is defined as the condition at which the vessel ceases to respond elastically (49).

Table III. Liner design criteria - local variations.

Item	British criteria	GGA criteria
1. Variation in liner thickness	Yes	No
2. Variation in liner yield point	Yes	No
3. Variation in spacing and stiffness of anchors	Yes	Yes
4. Initial inward curvature.	Yes	No (a)
5. Concrete void behind liner.	No	Yes
6. Water pressure behind liner	No	Yes
7. Local hot spots	Yes	Yes
8. Local loads	Yes	Yes
9. Loss of an anchor	Not known	Yes

(a) Effect of initial inward curvature is discussed qualitatively in Ref. 20.

or strains are obtained. The British designers have assumed the possibility of buckling and considered adverse effects of variation in liner properties and of initial inward curvature. In the Fort St. Vrain design, these factors were not considered.

In British design practice, water pressure and a concrete void behind the liner are not considered. Cooling tube leakage is regarded to be incredible. If unacceptable voids are detected behind the liner, the voids are filled with grout via holes drilled in the liner. GGA designers considered not only these two factors but also the effect of hot spots (20).

The effect of local variations listed in Table III is, in general, to increase forces and displacements in the liner components. A

quantitative example is included in Subsection 3.3.3 to indicate the extent of these effects.

2.4.4. Allowable Stresses and Displacements

Allowable stresses used in the design of the Fort St. Vrain liner are shown in Table IV. The reference codes, if any, are also shown in the table. It should be noted that these values are applicable to stresses resulting from credible loading conditions*.

The table shows that:

- a. Liner allowables were based on ASME Boiler and Pressure Vessel Code, Section III, Division 1 (1965).
- b. Allowable bearing stresses for concrete were taken from ACI Building Code (1963) requirements.
- c. There was no suitable code for establishing allowable values for shear anchor (studs and cooling tubes) and for shear forces adjacent to cooling tubes.

Information regarding the British allowable stress criteria is limited. It is understood that the British standard code of practice is generally used. In the design of the Wylfa PCRV liner, allowable bearing stress behind the ribs is taken as 1000 psi. The allowable loads on the shear anchors are based on ultimate strength of the member with a factor of safety of 2.2 (12).

*The credible loading conditions may be interpreted as corresponding to normal and upset stress conditions of the proposed code (1).

Table IV. Gulf General Atomic liner assembly allowables

Component	Allowable stress	Reference code
Liner:		
Primary stresses	$1.0 S_m$	ASME '65
Local stresses	$1.5 S_m$	ASME '65
Primary plus secondary stresses	$3.0 S_m$	ASME '65
Bearing on concrete:		
Behind the liner	$0.375 f'_c$	ACI '63
Behind penetration shear anchors ^(a)	$0.6 f'_c$	ACI '63
Shear force on:		
Cooling tubes	(b)	None
Studs	(c)	None
Shear force between cooling tubes ^(d) :		
Haunch area	800 lb/in.	None
Barrel area	380 lb/in.	None

(a) Based on lower limit of allowable bearing stress under tendon anchors.

(b) Not defined. Maximum computed value (2400 lb/in.) is smaller than the ultimate load capacity (8000 lb/in. from tests).

(c) Not defined. Maximum computed values (2700 and 4300 lb/stud) are shown to be much smaller than the ultimate load capacity (32,500 lb/stud, from tests).

(d) It appears that values shown were based on a study conducted by GGA, the details of which are not available ($f_{cua} = 6000$ psi).

Notation:

S_m = allowable stress intensity, and

f'_c = compressive strength of concrete.

2.5. Testing in Support of Design

Some experimental program has been undertaken in support of every PCRV liner design. Details of tests conducted are discussed in this section. Evaluation of these tests and recommendations as to the type of testing required for future prototype designs are given in Section 3.4.

2.5.1. Tests on Liner Segments

2.5.1.a. Tests Conducted

The most comprehensive test program on liner plate segments appears to be the one undertaken by Whessoe Limited in England (9, 11, 43). The main purpose of their test program appears to be the determination of the behavior of strong (i.e., flat) and weak (i.e., initially deflected) panels.

A square plate specimen (with or without an initially deflected panel), about 4 x 4 ft and supported by a thick plate (representing the backing concrete in the actual vessel), was used for determining the panel behavior. Plate material and thickness were the same as that of the prototype. Stud anchors were welded to the test plate and bolted on the opposite side of the back-up plate. Inplane forces were applied by edge compression and extensive strain and deflection readings were taken. Forces induced in the anchors were also recorded.

Similar specimens were tested by GGA in the design of the Fort St. Vrain PCRV liner (17). The main difference between the GGA and Whessoe specimens was that in the GGA specimen concrete was placed between the test plate and the back-up plate. Furthermore, in the GGA specimen, the concrete layer had a circular void at the center of the plate through which a lateral load was applied, simulating internal and water pressures. Stud anchors were wrapped in styrofoam in order to prevent loading of the anchors in shear. One of the four specimens tested had a weld offset and another had a cooling tube along the centerline.

The main purpose of GGA's test program was stated in Ref. 17 as follows:

"To demonstrate the ability of the selected liner and anchor design to accept the biaxial compression loading conditions without developing deformations that could adversely affect the performance of the liner, anchors, thermal barrier, or cooling tubes."

A different type of test conducted for the design of Wylfa liners was on a segment of liner together with anchors and back-up concrete (9, 12). The six-foot round specimen had a spherically dished liner plate and the inplane load was applied using hydraulic jacks. This was one of the few tests reported in which composite action of the liner and backing concrete was studied.

Two cylindrical models, with diameter-thickness ratios of 28 and 88, were also tested in the design of the Wylfa liners. Compressive strains of up to 3300 $\mu\text{in./in.}$ were applied to the specimens by means of shrink-fitting the specimen into thick steel cylinders. The effect of initial lateral deflection was also investigated in one of the cylindrical specimens.

Since the tests on large specimens were expensive and time consuming, additional tests were also conducted on strips of full-thickness plate (12) to supplement the data obtained in other tests. These strips were tested in axial compression with fixed-end boundary conditions in order to determine the dropoff in axial load in the plastic range. Specimens with different length-thickness ratios and initial lateral deflections were tested. This type of test is commonly called a strut test.

Bechtel has also tested rectangular plate segments (panels) in axial compression with fixed-end conditions. The resulting data have been used in the analysis of containment vessel liners (33).

There were other tests conducted, although not in support of a particular liner design. Chan and McMinn have tested ring segments in rigid circular cavities (28). Kicher and Esselman have tested cylindrical shells supported in a rigid cavity and flat and curved specimens with liner and backing concrete (27, 30). Some of these tests are briefly discussed in Subsections 2.3.2 and 3.2.2 as they pertain to buckling behavior of strips and shells.

2.5.1.b. Findings from Tests

The main findings from the tests summarized above are the following:

Tests Conducted in England:

- a) Stress-strain relationship of both flat and initially deflected panels may be approximated as elastic-perfectly plastic (10)*.

* See footnote on the next page.

- b) Load dropoff may occur in the initially deflected panel (11)*.
- c) For flat specimens (with anchor spacing-liner thickness ratio ranging from 18 to 38), the strain required to initiate buckling was the yield strain of steel (12).
- d) In the spherically dished specimens, the corresponding strain was about 2.5 times as much, indicating the beneficial effect of curvature.
- e) The composite test specimen showed that a very large barrel-shaped buckling (i.e., inelastic bending) takes place in the plate.
- f) Buckling of penetration liners, with a diameter-thickness ratio of up to 88, may be discounted.
- g) Strut tests showed that load carrying capacity of struts with initial deflection is smaller than that of similar struts without initial deflection.
- h) If the length-thickness ratio of struts with fixed-end conditions is less than about 16, no load dropoff occurs.

Tests by GGA:

- a) No load dropoff occurred in the panels even with small initial eccentricity**. It should be noted that the panel length-thickness ratio in these tests was 10.

*The no-load-dropoff and load-dropoff curves (apparently based on test results) given in Refs. 10 and 11 seem to be contradictory. However, this difference could be completely attributable to length-thickness ratios of the panels tested. Load dropoff may tend to occur for larger anchor spacing-liner thickness ratios. Unfortunately, the test panel data are not given.

**It should be noted here that the load considered is the total edge load. Whether changes take place in the stress field, especially during the applications of lateral load to an isolated area, was not discussed.

- b) It was concluded that the liner and anchor studs would withstand all potential loading conditions (including lateral pressures) without developing deformations that could adversely affect the performance of the liner assembly.

2.5.2. Tests on Anchor Systems

Tests have been conducted on components used as liner anchors and ties by numerous investigators. The purpose of these tests is usually to obtain data which is needed in the design analysis of the liner assembly. Specimens and testing procedures used depend on the type of anchor being studied and the specific test objectives.

It was reported previously (24) that bars, angles, and tees were commonly used as anchors in early designs. Accordingly, tests were conducted on tees (8), angles (12, 33), and bars (15). The most common testing procedure appears to be the so-called "push-out" tests. The specimen in this type of test is usually a plate with one or more anchors welded to it. The anchors are embedded in a concrete slab or block. The plate is pushed in its plane so the anchors embedded in concrete are loaded in shear. Bonding between the plate and backing concrete is usually prevented. In some cases the concrete was prestressed. By measuring shear deformations of the anchors corresponding to a series of load magnitudes, the load-deformation (slip) characteristics were obtained.

Shear load-deformation relationships of stud anchors have been extensively studied in the past (mainly at the University of Illinois and at Lehigh University) for application to conventional structures. These data were also used in the design of the Fort St. Vrain PCRV liner (19, 20).

Two types of tests are commonly used to evaluate the static and fatigue behavior of shear studs. These are composite beam tests and push-out tests. In the beam tests, load per shear connector cannot be measured directly. For this reason, investigators have conducted push-out tests to determine the properties of individual connectors.

A typical push-out test specimen usually consisted of a wide flange beam section, with stud anchors welded to its flanges and embedded in concrete (44, 51, 52). Two or four studs were welded to one flange (44) or both flanges (51). A concrete slab (or block) was placed on the flange, with or without reinforcement. Loads were applied either to the beam (51) or to the edge of the slab (44).

In static tests, the load carried by the studs and the corresponding shear deformation (slip) are measured at several load levels to give the stiffness characteristics of anchors needed in design analysis. Failure occurred at the shank of the stud or by pulling out a part of the flange to which the stud is welded when there was sufficient embedment length. Otherwise failure occurred by pulling out of the stud anchor from the backing concrete.

In fatigue tests, the load (with varying minimum stress and stress range) was cycled until failure occurred. Two different modes of failure were observed. Most fatigue failures were initiated at the reinforcement of the stud weld and penetrated into the beam flange, causing a concave depression into the beam flange. In a few cases, the fatigue failure initiated at the weld reinforcement and penetrated through the weld. Using the experimental data, S-N curves, where S is the range of stress on the nominal stud area and N is the number of cycles, have been developed (44).

2.5.3. Tests on Cooling Tubes

Tests on cooling tubes have been conducted both here (square cross section) and abroad (square and circular cross section). However, detailed data from the tests conducted outside the United States do not appear to be available.

Gulf General Atomic has tested cooling tubes, using two types of specimens, in support of the Fort St. Vrain PCRV liner design (17, 20). In the first type of specimen two cooling tubes, 1-in. square with 0.12-in. wall thickness, were welded on two sides of a 3/4-in.-thick plate. The plate was then embedded in concrete; the total assembly being cast in a steel cylinder. Bond between the plate and backing concrete was prevented. By applying an edge force to the plate, tubes were loaded in shear and displacements were recorded. The purpose of this type of testing was to determine behavior and ultimate load capacity of the cooling tubes. It was concluded that sizable factors of safety existed against failure under static and cyclic loads.

The second type of specimen used was a double plate, separated by a 1-in. gap and with two cooling tubes welded to the outside of the plates. Four studs were also welded to the two plates, two on each side, simulating the cavity liner design. These studs were wrapped in styrofoam so that only the cooling tubes would be loaded in shear. The specimen again was embedded in concrete, the total assembly being encased in a steel cylinder. Similar push-out tests were conducted on these specimens and load-displacement characteristics were obtained. Test results showed that static and cyclic load capacities were much greater than those required as predicted by analysis.

The cooling tubes in both specimens were considered failed when the load on the specimen, or when the pressure inside the tubes (constant 15 psi pressure was applied to the tubes during the tests), could not be maintained. Failure usually occurred by concrete failing in shear, at approximately a 30° angle to the plate.

3. EVALUATION OF CURRENT LINER DESIGN PRACTICE

Current liner design practice (both here and abroad), analytical and experimental techniques employed, and design criteria adopted are evaluated in this chapter. The order of subjects discussed is the same as that in Chapter 2. At the end of each section, recommendations concerning the subject matter of the section are included. These recommendations are based on available data (both experimental and theoretical) on liner design and engineering principles.

3.1. Modes of Failure

3.1.1. Buckling

It was stated in Subsection 2.3.2 that buckling behavior of liners with closely spaced anchors is quite different from that of liners without anchors. Stability of liners without anchors is indeed an important design consideration. On the other hand, the case of stud-supported liners may be treated as a stress problem. In this approach possible local variations, including buckling of a weak panel, can be fully taken into account as described in other parts of this report.

3.1.2. Excessive Deflections

Excessive lateral deflections of liner segments (normally as a result of buckling) may be detrimental to the integrity of reactor internal components and therefore should be considered in design analysis.

3.1.3. Rupture of Liner

There are two conditions in which failure of the liner by rupture is considered possible. The first of these has to do with ultimate

load conditions and is discussed in Subsection 3.2.5. The second is concerned with the anchor diameter-liner thickness ratio which is briefly discussed in the following paragraphs.

Push-out tests on stud anchors have exhibited three types of failure. These are (a) pulling-out of the stud anchor from the concrete, (b) shearing of anchor, and (c) pulling-out of the anchor from the base metal (beam flange in tests) to which it is welded. It was mentioned earlier that type (a) failure may be disregarded so long as sufficient embedment length is provided as is usually the case in PCRV's.

If an anchor happens to be loaded to its ultimate capacity, type (b) failure would be preferred to type (c) failure simply because of leak-tightness considerations. Pulling-out of the anchor from the liner plate would leave a hole in the plate and thus would conceivably lead to pressurized crack conditions.

In order to prevent type (c) failure mode, it has been suggested that the anchor diameter-liner thickness ratio be limited to about 2 (32). Push-out tests have shown that if this ratio is greater than about 2.7, the failure mode is usually type (c) (22). Therefore, there is an inherent factor of safety against type (c) failure mode, if the limiting ratio of 2 is adopted.

3.1.4. Rupture of Anchor

The possibility of anchor failure is another important design consideration which must be fully investigated. It has been shown that stress analysis methods may be used to predict anchor forces and displacements (8, 10, 11, 15, 19, 31-33). The predicted values may then be evaluated in view of the ultimate experimental values thus determining a safety factor against anchor failure.

3.1.5. Concrete Failure

Concrete failure is usually associated with the cooling tubes. Failure may occur due to high bearing stresses and shear stresses in the concrete between two cooling tubes. Therefore, limitations on the magnitude of these stresses are necessary to prevent concrete failure.

Local failure of concrete adjacent to stud anchors does not appear to be considered in past designs. This is probably due to the fact that high stress concentrations at these locations are unavoidable. It is usually assumed that such local concrete crushing will not affect the performance of the liner system.

3.1.6. High Strain-Low Cycle Fatigue

Fatigue of anchors and liner plate due to cyclic loads has also been an important consideration in past designs. The load-dissemination analysis methods have been used to determine the cyclic load range in the anchor (20) and in the liner panels (12). It was concluded that this failure mode should be investigated in all designs.

3.1.7. Brittle Failure of Liners

Neutron irradiation will produce changes in the mechanical properties of the liner and its attachments. In particular a rise in the ductile/brittle transition temperature and an increase in yield strength (2, 23) will result due to irradiation.

In order to prevent brittle failure of liners in areas where tensile stresses may occur, the transition temperature should be below the minimum predicted liner temperature. This is generally achieved by specifying suitable low-temperature impact properties for the liner

materials. Thus, it is important to determine the possible shift in nil ductility transition temperature (NDT) due to irradiation (1, 2, 18).

3.1.8. Recommendations

Based on the available data and past design experience, the failure modes to be considered in the design process are listed below. Qualitative criteria for preventing each type of failure are also listed. Quantitative design criteria are discussed in other parts of this report.

<u>Mode of failure</u>	<u>Design criteria</u>
Buckling	Radius-thickness ratio of the liner shell without anchors should be limited. No similar limitation is needed for liners with closely spaced anchors.
Lateral deflections	These should be limited to values specified in the design specification for the vessel.
Rupture of liner	Anchor size - liner thickness ratio should be limited.
Rupture of anchor	Anchor forces and displacements as determined by stress analysis should be limited.
Concrete failure	Concrete bearing stresses and shearing stresses between cooling tubes should be limited.
Fatigue failure	Cyclic load on the anchors and cyclic strains for the liners should be limited.
Brittle failure	The notch toughness of the selected liner material should be such that the possibility of brittle failure is precluded.

3.2. Design Bases

3.2.1. Design Analyses

As mentioned in Subsection 2.3.2, stability of liners without anchors is an important problem. Unfortunately, there does not appear to be an adequate theoretical method with which behavior of such liners may be predicted accurately, taking into account probable loading conditions, local variations, initial imperfections, and inelasticity. Therefore, the design of liners without anchor (with moderate shell radius-thickness ratios) should probably be substantiated by adequate testing (see also Subsection 3.2.2).

The uncoupled approach commonly used for the design analysis of anchored liners (with high shell radius-thickness ratios or flat) appears to be adequate. In this way it is possible to analyze sections of the liner, taking into account concrete-imposed loads and postulated local variations.

In the design analysis it is important to accurately calculate the strains which may be imposed on the liner since the magnitude of these strains affects the predicted stresses and displacements in liner components substantially. It is also important to determine accurately the behavior of liner components. Effects of various local variations must be considered for adequate design (31).

3.2.2. Stability Problems

3.2.2.a. Buckling of Liners without Anchors

In the case of liners without anchor, buckling is the primary design consideration. As pointed out in Subsection 2.3.2, only those

methods in which the effect of backing concrete (e.g. rigid cavity) is considered are applicable to liners.

The analytical methods mentioned previously give, as is usual in stability problems, the load at which buckling may take place with external disturbance. In the problem at hand, the load is in the form of concrete imposed and thermal strains in the liner. Buckling may also occur due to random vibrations or initial lateral deflections (28).

It is to be noted that the stability analysis methods used in the past (25, 27-30) consider a perfectly elastic response. On the other hand, as can be surmised from Fig. 3, elastic buckling may occur only if the radius-thickness ratio is very large. In cases where radius-thickness ratio is relatively small, buckling can be precipitated by yielding (26). It appears that yielding of steel liner has not been considered in the available stability analysis methods.

Another difficulty with the analytical methods is that few mathematical formulations of the buckling problem exist which include the effects of fabrication imperfections such as out-of-roundness, flat spots, and residual stresses (26). Such factors will influence minimum buckling strains and post-buckling behavior of the liners.

It is probably due to the above-mentioned reasons that, in British practice, anchors are provided for all liners (including circulator penetration liners) unless the cavity radius-liner thickness ratio is very small. It appears that available theoretical and experimental data are not sufficient to set a limiting value for the radius-thickness ratio beyond which anchors should be provided. It is noted that tests conducted on a cylindrical shell with radius-thickness ratio of 44 showed no buckling problem under uniform strains beyond

yield (12). Therefore, it may be suggested that in liners with radius-thickness ratio less than 44 there should not be any buckling problem.

3.2.2.b. Buckling of Liners with Anchors

The elastic buckling studies for liners with closely spaced anchors conducted by Kicher and his associates (25, 27, 30) and by Chan and McMinn (28, 29) as described in Subsection 2.3.2 are not directly applicable to the PCRV liners for the following reasons: a) As shown in Fig. 3, the strain at which buckling is predicted is much greater than the yield strains of steels commonly used as liners. Therefore, the liners will yield long before any elastic buckling occurs. This is supported by the experiments on liner shell and ring models (27) in which plastic deformation was observed even though anchor spacing-liner thickness ratio was from 39 to 141. b) Even if one assumes that elastic buckling analysis is still acceptable, the theoretical minimum buckling strains are still so much greater than the design strains for liners designed to date, thus making the results of such theoretical analysis of little practical value. The anchor forces predicted in Ref. 27 are also too small, much smaller than the shear anchor forces obtained from one-dimensional load dissemination analysis considering adverse variations between panels (31).

Although it is conceivable to extend the above-mentioned buckling analysis into inelastic range, such a study probably will have a limited practical application on account of the following reasons:

a) Stability methods cannot take into account some of the local variations (e.g., variations of yield point, liner thickness, etc.) which should be considered in liner design.

b) In cylindrical PCRV's, the design strains in the vertical direction are not uniform. In the new multi-penetration designs, even the circumferential design strains are not uniform due to large penetrations which extend through most of the vessel height. As pointed out earlier (31), nonuniform design strains introduce differential panel forces which must be resisted by anchor forces. Available stability methods, however, do not consider nonuniform strains.

c) From a practical point of view, fabrication of a PCRV liner to an assumed geometric perfection is a difficult task. It may be expected that some liner panels will have negative curvature. As an example considering the Fort St. Vrain design, the theoretical distance from the periphery of the liner cylinder to the chord between two stud anchors is about 0.036 in. If the liner has an initial inward deflection, exceeding this theoretical value, between two studs, the liner strip then becomes a column with initial lateral deflection.

d) Liner forces are partially induced by shear anchors which constitute eccentric loading on the panel.

Whether a panel is subject to initial inward deflection or eccentricity, such a panel will develop bending stresses when the concrete-imposed strains are applied. Then the problem is reduced to inelastic bending (due to a small length-thickness ratio) rather than that of stability.

As discussed in Subsection 2.3.1 in the design of liners with anchors, the anchor spacing is usually determined on the basis of cooling tube spacing selected. This usually results in a rather close anchor spacing. In earlier liner designs, tests were conducted to insure

that "buckling" was not a problem for the close anchor spacing determined (more properly, for the anchor spacing - liner thickness ratio selected). Since in most liner designs the anchor spacings usually stay within the range established by previous tests, "buckling" of liner with closely spaced anchors is not really a problem. Since a stress analysis must be conducted for a liner with closely spaced anchors, it may be stated that a buckling study of liner panel should be conducted in order to establish the panel characteristics as affected by unavoidable initial liner deflections which are needed in the stress analysis.

3.2.2.c. Recommendations

The following recommendations are offered as a general guide in view of the available data:

a) In the case of liners with cavity radius-liner thickness ratio (R/h) of up to about 44, there does not appear to be any need for providing shear anchors.

b) If the R/h ratio exceeds the above value, a design without anchors should probably be substantiated by testing.

c) In the case of large R/h ratios, the stability methods mentioned in Subsection 2.3.2 may be used as a guide in selecting anchor spacing. However, it can be seen from Fig. 3 that stability considerations will result in too large anchor spacing. Therefore, the usual practice of providing anchor between adjacent or every other cooling tube appears to be reasonable.

d) Once the liner thickness and anchor size and spacing are determined, the liner assembly should be analyzed by using one of the load dissemination methods discussed in other sections of this report.

3.2.3. Stress Analysis Methods

The capabilities and limitations of each of the one-dimensional methods described previously have been studied and are discussed below.

3.2.3.a. Parker's Method

This method of analysis is possibly the most versatile stress analysis method. In addition to variable design strains for panels, variations 1, 2, 3, 4, 7, and 9 (see Subsection 2.4.3) can be directly taken into account. These variations have been considered in the design of British PCRV's (10). It appears that the effect of variations 5 and 6 are not considered as was discussed in Ref. 24. The effect of local loads does not appear to be discussed in the literature; however, it is believed that local loads are taken into account using principles of theory of elasticity as was done in the design of the Fort St. Vrain liner.

One difficulty with Parker's method is in solving simultaneous equations which involve nonlinear coefficients (see Ref. 24). However, the theory can be reformulated using displacement principles and the resulting simultaneous equations solved by the iterative force-distribution or the initial stress methods described in Subsection 2.3.3.

3.2.3.b. Doyle and Chu's Method

This method of analysis is essentially a stiffness reformulation of Parker's approach (24). The basic difference is in the assumption of uniform initial panel forces made by Doyle and Chu. As a result of this assumption, their model is restricted to a liner segment symmetrical about a "weak" panel. Thus, this approach may be used for stress analysis in the circumferential direction. Modification of the technique

is necessary if it is applied in the meridional direction where design strain gradients exist.

3.2.3.c. Gulf General Atomic's Method

In the design of Fort St. Vrain liners, stress analysis was made in the meridional direction only. Details of the procedure are not available. However, it appears that variations 1, 2, 3, 7, and 9 can be readily taken into account in the computer code developed, although these were not considered in the design of Fort St. Vrain liner. The effects of water pressure and concrete void behind the liner in biaxial compression field were considered not amenable to analysis and were determined experimentally (20). Inward curvature was not considered and buckling of a panel (and thus losing some of its load carrying capacity) was considered unlikely. It was stated that complete loss of load carrying capacity of one panel will not result in the failure of other liner components (20).

3.2.3.d. Bechtel's Method

The analysis technique used by Bechtel in the stress analysis of containment vessel liners is a relaxation method based on uniform design strains. In this method, local variations in liner thicknesses and yield points can be taken into account but not variations in anchor spacing and stiffnesses. The effect of assumed inward curvature is included by using experimental characteristics for the weak panel. The effect of lateral pressure is considered by defining an equivalent inplane force on the basis of elastic relationships. The effects of all other variations were not considered in Ref. 33, probably because they do not affect the containment vessel liners.

In using Bechtel's relaxation method, the following points should be kept in mind:

1. This method assumes uniform design strain. If there is strain gradient, modification of the method is necessary.
2. The strong panel force, N , is determined on the basis of elastic relationship. This implies that all panels except the weak panel remain elastic regardless of the magnitude of design strains. It is shown in Subsection 3.2.4 that such an assumption may be too conservative.
3. In the derivation of the imaginary force, N' , which is applied at the node between the weak and the adjacent strong panel, the term "A" (see Subsection 2.3.3) was found to be greater than unity and was disregarded conservatively. The term "A" in PCRV liners, however, can be shown to be always less than unity and thus computing N' force from the relationship given in Subsection 2.3.3 may be on the unsafe side. Therefore, care should be taken in evaluating this force.
4. The equation for determining N' is based on linear elastic relationships. Therefore, the accuracy of the method is not established in the case of nonlinear (or inelastic) component behavior. This fact is of importance since the behavior of anchors under shear loading is nonlinear.
5. The stiffness of the weak panel is determined as twice the initial slope of the load-deformation curve for the panel. The value of K_B found in this manner is actually greater than strong panel stiffness, K_R .

3.2.3.e. Iterative Force-Distribution and Initial Stress Methods

Both of these methods are based on the same one-dimensional liner model used by Parker (Fig. 6). The main difference is in the technique used for the solution of nonlinear simultaneous equations.

All of the local variations listed in Subsection 2.4.3 can be taken into account in these methods. It is to be noted that variations 4, 5, and 6 are considered indirectly, e.g., by incorporating the effects of these variations on the stress-strain relationship of the assumed "weak" panel.

In both methods, average design strain in each liner panel is an input parameter. Therefore variation in the magnitude of design strains among the panels does not present any problem in the analysis.

3.2.3.f. Recommendations

It is recommended that load dissemination analysis methods used in any design be capable of dealing with nonlinear material properties and actual boundary conditions. The iterative force-distribution method or the initial stress method discussed previously may be used for general liner stress analysis.

It should be mentioned that it may be difficult to analyze some parts of the liner assembly (such as main cavity and gas duct liner intersections and internal component support points) using one-dimensional models. In such cases the state of stress and deformations should be investigated, preferably by a finite-element elasto-plastic analysis, by considering a suitable free body of the section and using forces and/or displacements obtained from the vessel analysis at the boundaries of the free body (53).

3.2.4. Other Structural Considerations

3.2.4.a. Assumed Behavior of Liner Panels

a) Strong panels: It was stated in Subsection 2.3.4 that three different assumptions have been used regarding the behavior of strong panels.

These assumptions are:

- 1) Elastic-perfectly plastic [Parker (10) and this study].
- 2) Elastic under all loading conditions [Bechtel (33)].
- 3) All panels yield before a weak panel is formed, implying that design strains beyond yield do not affect the behavior of the structure [Doyle and Chu (32)].

It is shown in Fig. 7 that the second assumption results in the highest anchor forces which are conservative. It can be shown that if Bechtel's assumption is used for a case with $\epsilon_{ci} > \epsilon_y$, the final strains in the panels away from the weak panel are greater than yield strain of the liner material. This indicates that these panels will actually be in yield, thus contradicting the assumption.

Doyle and Chu's assumption is correct when all panels yield under applied strains and then one panel (the weak one) buckles inelastically. However, in PCRV liners, such a behavior is unlikely and due to any adverse variation, at least some panels will deflect laterally introducing bending moments. The latter in turn reduces the axial load capacity of the panel and since the yield load for the adjacent flat panel is larger, the adjacent panel can never yield. Because of this reason, Doyle and Chu's approach actually leads to anchor forces which are on the unsafe side for design strains greater than biaxial yield strains, as shown in Fig. 7.

b) Weak panels: It is clear that behavior of weak panels is best determined from suitable tests (see Subsections 2.5.1 and 3.4.1).

However, it appears that, in lieu of testing, strut analogy may be used to evaluate the weak panel characteristics for use in analysis. The effect of lateral pressure on the weak panel behavior may be readily incorporated (31) when strut analogy is used.

The validity of the assumption that stress-strain behavior of a liner panel can be based on the behavior of an equivalent fixed-end strut is indicated in Fig. 9. The test result is taken from Ref. 10. It is understood that panel width-thickness ratio was between 12 and 18. Strut analysis for these two ratios gives adequate results as shown in the figure.

It must be noted that the above statement is based on one test result only. No other test data seem to be available. Furthermore, the test result is given only up to a maximum strain of 2500 $\mu\text{in./in.}$ (Fig. 9). In analysis however, the apparent strain may be much higher than this value (31). Therefore, there appears to be a need for more experimental research in this area in order to establish weak panel behavior and to verify the validity of strut analogy.

Idealizing stress-strain characteristics of weak panels (whether based on test results or strut analogy) is merely for convenience. This idealization may be bilinear (10) or trilinear (31) so as to describe the actual curve with sufficient accuracy. The important aspect of this process is that any load dropoff must be adequately described since the load dropoff has a significant effect on predicted forces and displacements (31).

3.2.4.b. Forces on Cooling Tubes

It was stated in Subsection 2.3.4 that buckling or inelastic bending of the liner between cooling tubes, which introduces torsional moments on the cooling tubes, has not been considered. It is assumed that sufficient shearing forces, in the direction normal to the plane of liner, between cooling tube-concrete interface cannot be developed to resist moments which will develop at the liner-cooling tube joint. This assumption is considered to be valid.

It appears to be debatable whether the contribution of cooling tubes in resisting differential panel forces in the plane of the liner should be taken into account. It was suggested by some designers that the shear resistance of cooling tubes is not reliable because concrete between the cooling tubes cannot develop predicted shear loads (43).

The problem may be better understood by considering the shear stresses in concrete between cooling tubes. If the friction between the liner and concrete is ignored, it is clear that bearing forces on the concrete in contact with the tube must be equilibrated by concrete shearing stresses between two tubes. It is possible that, in some cases, the magnitude of predicted shearing stresses will be greater than that allowed by the proposed code (1).

As an example, consider the Fort St. Vrain design with the following pertinent data: $a = 7.5$ in., $f'_c = 6000$ psi, i.e. $f_v = 0.075 f'_c = 450$ psi, where a is the cooling tube spacing, f'_c is the compressive strength of concrete, and f_v is the allowable shear between cooling tubes. Without assuming any weak panel, the predicted shearing stress in one case was found to be about 800 psi (19) which is greater than

the allowable. Introduction of any postulated adverse variation would result in even higher shearing stresses.

Even if the calculation indicates that the concrete shear stress is allowable, there is always the possibility of a concrete void being adjacent to the tube wall, especially underneath horizontal cooling tubes.

It may be concluded, therefore, that cooling tubes should not be considered as anchors in the load dissemination analysis. This will, of course, increase the calculated forces in stud anchors and is therefore conservative.

For the design of cooling tubes, however, the model should include the cooling tubes so that maximum forces on the tubes may be calculated. These forces must be adequately evaluated considering postulated adverse variations in order to achieve proper cooling system design, including weld design.

The predicted displacements should also be evaluated in view of the test results. For example, in the study reported in Ref. 31, the maximum inplane displacements were found to be about 0.02 in. for Fort St. Vrain liner anchors. The push-out tests conducted by GGA on cooling tubes showed that failure occurs at about 0.08 in. of shear slip (17, 20).

Cooling tubes can also be expected to displace in the radial direction, away from the concrete, if there is water pressure behind the liner. The radial displacement would be greater for larger anchor spacings. This possibility of radial displacements and its effects must also be considered in the design of cooling tubes.

3.2.4.c. Forces on Stud Anchors

As pointed out earlier, the practice in liner design is to use only shear-slip characteristics of anchors. Thus, the actual stress field in the anchors has been of little interest. This approach is justified since the allowable anchor shear forces are based on experimental results. It is essential that these experiments reflect the actual loading conditions in a prototype vessel (see Subsection 3.4.2).

In cases where a pressure build-up may occur behind the liner due to a leak in the cooling tubes, the stud anchors will be subject to pull-out forces. It was stated earlier that cooling tubes are not expected to function as an anchor in such an event. Thus, water pressure should be assumed to be resisted by studs alone. In the Fort St. Vrain design, nominal stud anchor spacing is 7.5 in. in both axial and circumferential directions. Therefore, neglecting the effect of liner curvature, a conservative estimate for anchor tensile forces due to a water pressure of 125 psi would be about 7 kips which is much less than the experimental ultimate strength of about 35 kips for 3/4-in. diameter studs (45).

Tensile forces will also be induced in anchors where an adjacent panel is buckled (more correctly, subjected to inelastic bending). The magnitude of such pulling-out forces may be estimated by approximate methods (33) or from the tests conducted on flat liner plate sections (20). The magnitude of these forces will naturally depend on the design strains. Thus, if such forces are determined experimentally, they should be obtained under appropriate biaxial strain fields. Interaction of tensile and shear forces is discussed in Subsection 3.3.4.

Another important consideration in stud design is the embedment length necessary to develop the full stud strength. As mentioned earlier,

tests have shown that a shank length of about nine stud diameters is sufficient to develop the full strength in concrete strengths of about 3000 psi (20). For higher concrete strengths the required embedment length would, of course, be shorter. The existence of prestress also has a beneficial effect in developing the anchor strength.

The above criterion on development length does not take into account the effect of temperature on concrete strength. It appears that research data on the interrelationship among development length, concrete temperature, and existence of prestress is lacking. It has been stated that a temperature of 200 °F will have a negligible effect on the strength of concrete (20). Since the predicted concrete temperatures in the Fort St. Vrain PCRV design were within 200 °F, the minimum length requirements given in Ref. 45 was appropriate for that design.

It is to be noted that the actual stud anchor lengths used in past designs were usually greater than the minimum required and thus no pull-out problem was expected.

3.2.4.d. Recommendations

The following guidelines on design assumptions and analytical procedures are offered for adequate design of the liner components:

Strong panels: Strong panel behavior may be assumed to be elastic-perfectly plastic or, conservatively, purely elastic. The assumption that all panels yield under applied strains and that one panel buckles afterwards may lead to unconservative results and therefore is not recommended.

Weak panels: The weak panel behavior should be determined by appropriate testing or, in lieu of such testing, by using strut analogy.

If tests are conducted, the effect of local variations (see Subsections 2.4.3 and 3.3.3) should be included in these tests whenever possible.

Cooling tubes: It is recommended that cooling tubes be considered not to participate in resisting moments, shears, and radial forces. For the design of cooling tubes however, the analytical models should include the cooling tubes in order to evaluate the magnitude and effects of forces and moments on the tubes.

Stud anchors: In design analysis, only stud anchors should be considered as effective in resisting differential panel forces and radial forces due to water pressure or vacuum pressure. It is appropriate to use experimental anchor characteristics in analytical methods.

3.2.5. Ultimate Loading Considerations

The behavior of liner under ultimate loading conditions is an important consideration in evaluating the ultimate strength of the overall vessel. This importance is not due to contribution of liner to vessel ultimate strength. Rather, it is the influence of liner fracture on the predicted vessel response near ultimate pressure.

It is generally accepted that, at or near the ultimate pressure, the liner may rupture and the coolant may leak through the concrete cracks. Gross liner rupture is undesirable since it will lead to pressurized crack conditions. It was stated that gross liner rupture may occur when only a few large cracks form in the concrete over which the liner plate may not be able to span and that in order to prevent a few large concrete cracks, adequate bonded reinforcement near the liner is needed (54). With substantial bonded reinforcement, it is reasoned,

large cracks cannot form because of the redistributing effects of the reinforcement.

It may be stated that, even if there are a few large cracks, there is very small probability of gross liner rupture. The reasons for this assertion are summarized in the following paragraphs.

The most critical case would come about if one panel reaches its ultimate strength before the adjacent panel yields. If several panels yield, providing sufficient elongation so as to conform the liner shell to vessel deformation, there can be no gross liner rupture. Referring to Fig. 6b and considering a unit width strip, assume that the liner panel ruptures adjacent to a cooling tube. If such a failure is to happen, the following inequality must hold:

$$V_u > F_u - F_y \quad (23)$$

where V_u is the ultimate shear force per unit width to be developed by the cooling tube and F_u and F_y are the ultimate and yield force per unit width, respectively, of the liner. The V_u force must in turn be developed by concrete shear between the cooling tube and the neighboring anchor. This shear force is

$$V_u \leq \frac{a}{2} f_v$$

where a is the anchor spacing (also the cooling tube spacing) and f_v is the ultimate shear strength of concrete. As an example, if $a = 8$ in., liner yield stress = 56 ksi, ultimate strength = 75 ksi, thickness = 0.75 in., concrete strength = 6000 psi, and $f_v = 3.5\sqrt{f_c} = 270$ psi, then

$$V_u \leq 1.08 \text{ kips/in.} \quad (24)$$

and

$$F_u - F_y = 16.25 \text{ kips/in.} \quad (25)$$

Obviously, concrete will fail in shear long before the adjacent liner panel reaches its ultimate strength.

If a panel between two stud anchors is to rupture, then

$$S_u > F_u - F_y \quad (26)$$

where S_u is the anchor ultimate shear force per unit width. Again using the above example with 3/4-in. diameter studs (ultimate shear strength = 32.5 kips)

$$S_u = 4.1 \text{ kips/in.} \quad (27)$$

Thus, shear anchor must fail before the liner ruptures.

It is clear that successive failure of stud anchors and concrete between cooling tubes will equalize the panel forces, thus redistributing the total vessel deformation over a large length of liner. Considering the fact that total vessel deformation prior to failure is within a few inches, it takes only several liner panels to provide for the total deformation, after the liner panels have yielded.

The conclusion from the above discussion is that, notwithstanding the steel reinforcement adjacent to the liner, gross failure of the liner under ultimate pressure is highly unlikely.

The above discussion is based on the premise that liner panel characteristics are uniform. Of course, if there is a liner panel weaker in tension than two adjacent panels and if there happens to be a large concrete crack adjacent to the weaker panel, then the weaker panel may rupture.

From a design viewpoint therefore, the following are of importance:

- a) Cavity liner should not have large abrupt changes in thickness and strength properties so that large differential panel forces may not develop.
- b) The liner material should have adequate ductility.

The first criterion is intended to minimize the probability of liner rupture under all conditions. The second criterion is intended to prevent crack propagation. It is most likely that, under ultimate loading conditions, local cracks will originate from very small liner defects or small cracks produced by stress concentration at liner attachments or at discontinuities (54). A ductile liner material may be expected to have sufficient strain energy to prevent small cracks from opening and propagating.

3.3. Design Criteria

3.3.1. Loads

The list of loads discussed in Subsection 2.4.1 essentially agrees with the list of loads^{*} which must be considered in liner design according to the proposed code (1). The only additional load which is included in certain loading combinations suggested in the proposed code is the earthquake load.

Earthquake load is rarely discussed in the design of liners. In the Fort St. Vrain PCRV liner design, seismic stresses were computed for concrete alone, neglecting the effect of tendons, liner, and reinforcement (20). The maximum stress in concrete near the liner at the

* Loads which are not relevant to liner design are excluded.

bottom haunch area was given as 270 psi for the working stress design condition (horizontal acceleration of 0.05 g and vertical acceleration of 0.033 g with 2% critical damping). If the modular ratio is assumed to be 8, the resulting principal stress in the adjacent liner will be about 2200 psi. This maximum stress corresponds to about 70 $\mu\text{in./in.}$ biaxial strain in the liner.

In the design of some containment vessel liners, strains induced in the liner due to seismic loads have been taken into account (33). The magnitude of these strains, under the maximum hypothetical earthquake conditions, were found to vary from approximately 0 to $\pm 76 \mu\text{in./in.}$ in the vertical direction and 0 to $\pm 23 \mu\text{in./in.}$ in the circumferential direction.

Although the magnitude of strains due to seismic loads is relatively low, they should nevertheless be considered and included in design strains.

3.3.1.a. Recommendations

It is concluded that the design strains should be determined considering the following loads:

- a. dead loads
- b. prestressing loads
- c. creep and shrinkage
- d. thermal loads
- e. internal pressure
- f. earthquake loads.

The effect of live loads (including hydrostatic pressure of wet concrete) and wind loads (depending on the construction sequence of the liner) should be investigated.

Combinations of various loads for design purposes are discussed in the next section.

3.3.2. Loading Combinations

The loading combinations adopted by British designers and Gulf General Atomic were given in Table II, Subsection 2.4.2. Following important points are apparent from that table.

- a. The loading conditions considered during the early life of the vessel are the same.
- b. In the Fort St. Vrain PCRV liner design, due attention was also given to the liner stresses which were expected at the end of the design life. Reference 12 reported that the minimum, probable, and maximum values of imposed strains in the liner at significant stages in the reactor life have been estimated. Thus it appears that British designers have also investigated liner assembly behavior throughout the design life of the vessel, although this is not apparent from the loading conditions given in Table II.

Table II also shows that, in past designs, only the following categories of loading combinations [using the terminology of the proposed Code (1)] have been considered:

- a. Construction category loading combinations
- b. Normal category loading combinations.

3.3.2.a. Recommendations

Based on previous designs, the proposed code and functional requirements of liner assembly, the following categories of loading

combinations should be considered in determining the design strains for the liners*:

- a. Construction category loading combinations
- b. Normal category loading combinations
- c. Abnormal category loading combinations
- d. Extreme environmental category loading combinations.

The severe environmental category load combinations are not included in the above list simply because the additional loads to be included in this category (1) do not affect the liner design strains appreciably. The failure category loading combinations are not believed to apply to liner design. In fact, the proposed code does not place any limit on the stresses and displacements for liner components under failure conditions.

It is important to note that in determining the design strains under various loading combinations due attention should be given to the expected loading sequence of the vessel throughout its entire design life.

3.3.3. Local Variations

It was pointed out earlier that the local variations considered in past designs vary considerably. In order to illustrate the effect of postulated adverse variations, the following example (from Ref. 31) is presented. In this example, Fort St. Vrain liner was analyzed in the circumferential direction, neglecting the resistance of cooling tubes. The method of analysis used was the iterative force-distribution method

* Specific loading combinations in each category and the load factors to be used in these combinations are beyond the scope of this study.

discussed previously. In the analysis, a one-dimensional model consisting of 31 panels, 30 interior nodes, and two fixed ends is considered. The 16th panel (middle panel) is assumed to be the "weak panel" (i.e., the panel with initial inward deflection). The weak panel characteristics are obtained from strut analysis and approximated using three line segments. The anchor load-deformation relationship is taken from Refs. 32 and 45 and approximated by four line segments.

The design data for the Fort St. Vrain liner, in the circumferential direction, is as follows:

Liner thickness	$h = 0.75 \text{ in.}$
Liner yield stress [*]	$\sigma_y = 60 \text{ ksi}$
Initial lateral deflection of weak panel (assumed)	$w_{om} = 1/16 \text{ in.}$
Anchor spacing (x- and y-directions)	$a = 7.5 \text{ in.}$
Anchor size and type	3/4-in. diam headed studs
Design strains ^{**}	$\epsilon_{ci} = 1731 \text{ } \mu\text{in./in.}$

The local variations introduced into the basic design (individually and in combinations) and related assumptions are shown in Table V as cases a to k. The design strains for case k are different as will be discussed later on. In all cases, the weak panel is assumed to have 1/16-in. initial inward deflection, except as noted.

* For SA-537 Gr. B steel. However, the specified minimum yield stress in the design was 56 ksi.

** Uniform design strain in the circumferential direction. This value is computed from the data given in Ref. 18 and corresponds to a loading case: prestress + shrinkage + creep + thermal loads.

Table V. Local variations studied.

Case	Design strain, in./in.	Initial defl., in.	Other local variations	Assumptions on local variations
a-1	1731 ^(a)	1/16	None	—
a-2	1731 ^(a)	1/8	None	—
b	1731 ^(a)	1/16	Thickness variation ^(b)	h = 0.74 in., weak panel h = 0.806 in., other panels
c	1731 ^(a)	1/16	Anchor stiffness	One stud stiffness is one-half of the others
d	1731 ^(a)	1/16	Loss of an anchor	Loss of an anchor doubles the spacing
e	1731 ^(a)	1/16	Water pressure	q = 125 psi, behind weak panel
f-1	1731 ^(a)	1/16	Hot spots	$\Delta T = 100^{\circ}\text{F}$, adjacent to weak panel
f-2	1731 ^(a)	1/16	Hot spots	$\Delta T = 100^{\circ}\text{F}$, two panels adjacent to weak panel
f-3	1731 ^(a)	1/16	Hot spots	$\Delta T = 200^{\circ}\text{F}$, two panels adjacent to weak panel
g	1731 ^(a)	1/16	Yield point variation ^(c)	$\sigma_y = 48$ ksi, weak panel $\sigma_y = 60$ ksi, other panels
h-1	1731 ^(a)	1/16	Combination of b, e, f-1	—
h-2	1731 ^(a)	1/8	Combination of b, e, f-1	—

Table V. Continued

Case	Design strain, μin./in.	Initial defl., in.	Other local variations	Assumptions on local variations
i-1	1731 ^(a)	1/16	Combination of b, d, e, f-1, g	—
i-2	1731 ^(a)	1/8	Combination of b, d, e, f-1, g	—
j	1731 ^(a)	—	Loss of panel load carrying capacity	Weak panel carries no load
k-1	981 ^(a)	1/16	Concrete void	q = 750 psi on weak panel
k-2	981 ^(a)	—	Concrete void	Weak panel carries no load

(a) From Refs. 18 and 20.

(b) From ASTM Standards, Part 4, A20.

(c) Strong panels are assumed to have 25% higher yield point.

3.3.3.a. Methods of Analysis of Local Variations

The computer program developed for the analysis of liner system by the iterative force-distribution method (31) requires separate input for each panel and each anchor. With this feature, any local variations of known locations can be easily accommodated.

In design analysis, the local variations should be assumed to exist in the most unfavorable manner. Usually a weak panel is identified and local variations are assumed to occur in or next to this weak panel in the following manners:

1. Initial inward deflection

Apply the assumed initial deflection to the weak panel and develop its characteristic by the strut analysis.

2. Variation in liner thickness

Establish the weak panel characteristic based on the thickness of the thinnest panel. Establish the strong panel characteristic based on the maximum panel thickness.

3. Variation in liner yield stress

Establish the weak panel characteristic based on the minimum yield stress. Establish the strong panel characteristic based on the maximum yield stress.

4. Variation in anchor spacing

Establish the weak panel characteristics using the largest anchor spacing. Possibility of a loss of an anchor can be treated by doubling the regular anchor spacing at the weak panel (i.e., the weak panel length is twice that of other panels).

5. Variation in anchor stiffness

Assume that one of the anchors adjacent to the weak panel has reduced stiffness. Use a different anchor characteristic to reflect the variation in anchor stiffness.

6. Water pressure behind the liner

Develop the weak panel characteristic considering water pressure as the lateral load, using the strut analysis.

7. Concrete void behind the liner

If the liner surface is not subjected to internal pressure, the existence of a concrete void has no effect in the analysis. If the internal pressure exists, the liner panel covering the concrete void should support the pressure by plate bending. For liner analysis, develop the weak panel characteristic by strut analysis, considering the internal pressure as the lateral load. It is noted that the bending deflection of the panel in this case is opposite in direction to those due to initial inward deflection and water pressure behind the liner.

8. Local high temperature (hot spots)

This effect is taken into account by adding the corresponding thermal strains to the panel design strains (10). It is likely that the increased local temperature distribution will be non-uniform. However for design purposes, one or more panels may be assumed to have uniform temperature increases. Also, thermal gradient in the thickness direction is usually ignored in the design of both PCRV and containment liners (10, 20, 33).

Combinations of various local variations can be accommodated by assuming that all local variations to be combined occur in the same weak

panel. This is justified since strains tend to accumulate in a weak panel. A new weak panel characteristic is then developed for the combination on hand (using strut analysis, if plate bending is involved).

When the design strain is uniform and no structural discontinuity exists, the location of the weak panel for design is immaterial. When the design strain is variable, the weak panel should be selected at the location of maximum strain gradient and maximum strain (8). It may be necessary at times to try several possible locations in order to identify the critical weak panel for a given liner under a given design strain condition.

3.3.3.b. Discussion of Results

The computed maximum anchor forces and displacement and locations of reference anchors are shown in Table VI. The reference anchor is defined as the anchor with displacement no more than 10% of the maximum anchor displacement. Its location is given by counting the anchors sequentially starting with the maximum stressed anchor (also with maximum displacement) as anchor number 1. Since, in most cases, the last anchor in a model has a displacement of approximately 2% of the maximum displacement, a rough indication on the extent of disturbance caused by a local variation is obtainable with added information from the reference anchor.

Discussion of the results in Table VI and their significance is given below in the order of cases in Table VI.

Case a. Initial inward deflection: If the initial lateral deflection of the weak panel is assumed to be 1/16 in., the resulting forces

Table VI. Effects of local variations in the circumferential direction.

Case	Maximum anchor force, kips	Maximum anchor displ., in.	Weak panel strain, in./in.	Approximate lateral deflection (a), in.	Location of ref. anchor (b)
a-1	2.75	0.0023	0.0024	0.025	10
a-2	10.64	0.0090	0.0041	0.094	11
b	11.28	0.0098	0.0043	0.083	11
c	2.82	0.0024	0.0024	0.025	10
d	19.00	0.0369	0.0066	0.532	12
e	11.12	0.0094	0.0042	0.117	11
f-1	11.01	0.0093	0.0035	0.061	6
f-2	13.36	0.0164	0.0047	0.094	4
f-3	18.75	0.0344	0.0079	0.194	5
g	16.36	0.0259	0.0096	0.227	11
h-1	18.91	0.0360	0.0105	0.329	11
h-2	19.00	0.0368	0.0108	0.298	11
i-1	23.54	0.0807	0.0121	0.927	12
i-2	23.62	0.0816	0.0122	0.892	12
j	24.83	0.0946	0.0270	—	13
k-1	0.20	0.0002	0.0010	0.013	9
k-2	19.45	0.0414	0.0120	—	12

(a) Additional to initial deflection.

(b) Reference anchor is defined as the anchor with displacement no more than 10% of the maximum anchor displacement. The number in this column gives the location of reference anchor, counting the maximum stressed anchor as number 1.

Note: The above values of anchor forces and displacements are to be compared with ultimate anchor strength of 32.5 kips and ultimate displacement of 0.341 in. (45).

and deformations are rather low. The reason for this result is that the design strains are lower than the yield strain of the steel used. If the design strains were assumed to be at yield strain level, the predicted maximum anchor force would be 11.76 kips which is much larger than the 2.75 kips shown in Table VI.

In the case of 1/8-in. initial inward deflection, anchor forces and displacements are predictably higher. Although the increase would largely depend on the design strain as discussed above, it may be concluded that larger initial inward curvature will result in greater anchor forces and displacements.

Case b. Effect of thickness variation: The thickness variation shown in Table V is based on tolerances for a 3/4-in. plate shown in ASTM Standards, Part 4, A20. The anchor force is seen to increase considerably, in fact, more than that due to 1/8-in. initial deflection.

Case c. Effect of anchor stiffness variation: In this case it was assumed that one anchor adjacent to the weak panel had one-half of the stiffness of other anchors. The result shows that such a variation is not significant. Thus, it may be concluded that lower stiffness of a single anchor is not important as long as it is considered to have sufficient tensile strength to hold the panel in place (see case d).

Case d. Effect of loss of an anchor: If the anchor is completely lost (i.e., no shear or tensile load capacity), the two adjacent panels would become one panel which is assumed to be the weak panel. The resulting anchor force is larger than those cases in which only a single variation is considered.

Case e. Effect of water pressure: In this case it was assumed that the weak panel had a lateral pressure of 125 psi in addition to the assumed

1/16-in. initial lateral deflection. The magnitude of the lateral pressure is stated to be the maximum credible condition for the Fort St. Vrain Vessel (20). The resulting anchor forces and displacements, although large, are not as great as those in case d. However, it should be pointed out that the effect of water pressure would be greatly increased if the anchor spacing is larger.

Case f. Effect of hot spots: There are two possible critical locations for hot spots:

1. The weak panel is the hot spot: If the weak panel is already strained beyond yield, the effect of temperature increase will be to increase lateral deflection thus reducing its load carrying capacity. This in turn will cause increases in the anchor forces. However, there does not appear to be any data available on the behavior of initially yielded struts or plates. For this reason the case of weak panel being subject to temperature increase is not studied.
2. The hot spot is away from the weak panel: In this case the worst effect will come about if hot spot occurs adjacent to the weak panel. Case f gives the results of analyses with this assumption.

In case f-1, it is assumed that the panel adjacent to the weak panel will be subject to 100 °F temperature increase. In case f-2, two consecutive panels adjacent to the weak panel are assumed to be hot. Finally, in case f-3, two consecutive panels adjacent to the weak panel are assumed to have 200 °F higher temperature. The magnitudes of differential temperatures are taken from the proposed code (1) and reflect the normal and the emergency loading conditions. Since the

area over which temperature increase occurs is not known, one and two panels are considered in this study.

The results show that anchor forces increase considerably under the assumed conditions. It is noted also that assuming hot spots on two consecutive panels is more critical than assuming hot spot on one panel.

One interesting observation is that the effect of hot spots seems to be more localized compared to other local variations. This is reflected by the low numbers for the location of reference anchors as given in Table VI.

Case g. Yield strength variation: Although the minimum yield strength of SA.537 Gr. B steel is 60 ksi (for 3/4-in. thickness), it was assumed in this case that the weak panel yield stress is 48 ksi. This is based on the assumption that yield stress variation may be as much as 25% of the minimum (35).

The anchor force obtained for this case is larger than other cases considered so far (except cases d and f-3).

Cases h and i. Combined effect of local variations: The combination of more than one adverse variation should be based on a probabilistic study. Since such a study is not available, four different but arbitrary combinations were studied.

In case h initial deflection, thickness variation, water pressure, and hot spot were combined. The significant results obtained may be summarized as follows:

1. The effect of adverse variations are not additive. In fact, the anchor force for the combined case is much smaller than the sum of anchor forces due to individual variations.

2. It was indicated in case a that the magnitude of initial lateral deflection, without any other local variation, affects the anchor forces substantially. However, case h shows that when initial inward deflection is considered together with other local variations, the magnitude of initial deflection does not appear to make much difference.

In case i, all adverse variations are combined (initial deflection, water pressure, hot spot, loss of an anchor, and yield strength variation). Such a condition is probably highly unlikely. The results show that anchor forces would be about 73% of anchor ultimate strength. The displacements are about 24% of the ultimate. It is noted that, as in case h, the magnitude of initial lateral deflection makes little difference when it is combined with other local variations.

Case k gives the results of two analyses for which design strains were taken from Refs. 18 and 24.

It is seen that the resulting anchor forces and displacements are negligible if the weak panel load-displacement characteristic is based on 1/16-in. initial deflection. However, if the weak panel is assumed to carry no inplane load (by assuming the formation of plastic hinges at three points), anchor forces and displacements are considerable.

3.3.3.c. Recommendations

Although the example given above is for a specific design, it may be concluded that the results do indicate, qualitatively, the importance of postulated local variations. The following recommendations are based on detailed stress analysis studies given in Ref. 31 from which the above example was taken.

a) The effect of the following local variations should be considered in liner design:

1. Variation in liner thickness
2. Variation in liner yield point
3. Initial inward curvature
4. Concrete void behind liner
5. Water pressure behind liner
6. Local hot spots
7. Loss of an anchor
8. Misalignment of liner seams
9. Local loads.

Although the effect of liner weld offset was not studied quantitatively, it may be stated that its effect will be similar to that of initial inward curvature. The local loads introduced at reactor internal component support points and at discontinuities are detail problems in nature and should always be considered in design.

The above list of local variations are essentially the same as that given in the proposed code (see CB-3810, Ref. 1) with the exception of items 5 and 7. In the design of the Fort St. Vrain liner however, the effects of water pressure behind the liner and loss of an anchor have been considered.

b) The combination of local variations and their relationship to loading categories discussed in Subsection 3.3.2 should probably be based on probabilistic studies. However, a suggested list of overall analysis conditions is given in Chapter 5.

3.3.4. Allowables Stresses and Displacements

As pointed out in the introduction, different organizations have adopted different criteria for liner assembly allowables in the past due to lack of an applicable code. The allowables adopted by GGA for the design of the Fort St. Vrain liner were given in Table IV of Subsection 2.4.4. The liner allowables therein were based on ASME Section III, Div. 1 (55) and the concrete allowables on American Concrete Institute Building Code (1963). No allowables for shear forces on stud anchors and cooling tubes were given. Instead, the predicted values were simply compared to experimentally obtained ultimate values in order to evaluate the factors of safety.

The recently proposed code (1) includes allowable stresses, strains and displacements for all liner components. These allowables are summarized in Table VII. For completeness, allowables for liner, shear anchor, and concrete are given in the same table although they appear separately in the code (Tables CB-3700-1, CB-3700-2, and CB-3422-1). It is assumed that allowable stresses given in Table CB-3422-1 are applicable to bearing stresses below the cooling tubes and shearing stresses between the adjacent cooling tubes.

3.3.4.a. Discussion of the Proposed Code Allowables

a. The proposed code permits tensile membrane stresses under normal operating conditions, although past design practice has invariably resulted in no tension in the plane of the liner under normal operating conditions (8, 49).

b. Steel liner is not a pressure vessel by itself and stresses in it are imposed by the backing concrete, not directly by the pressure. In the liner stress analysis, the design strains, which are the strains

Table VII. Proposed code allowables.

Stress condition	Loading category	Liner (a).				Anchors		Concrete	
		Compression		Tension		Mechanical loads	Displacement limited loads	Bearing below shear anchors	Shear between adjacent anchors
		Membrane	Membrane plus bending	Membrane	Membrane plus bending				
Normal and Upset	Construction Normal Abnormal Severe environmental	$\epsilon_{su}/30$	$\epsilon_{su}/20$	$1.0 S_m$	$1.5 S_m$	Lesser of $F_a \leq 0.67 F_y$ $F_a \leq 0.33 F_u$	$\delta_a = 0.25 \delta_u$	Confined concrete: $0.6 f_{cua}$ Unconfined concrete: $0.2 f_{cua}$	$0.075 f_{cua}$
Emergency	Extreme environmental	$\epsilon_{su}/20$	$\epsilon_{su}/10$	$\epsilon_{su}/20$	$\epsilon_{su}/10$	Lesser of $F_a = 0.9 F_y$ $F_a = 0.5 F_u$	$\delta_a = 0.5 \delta_u$	No limitation	
Faulted	Failure	No limitation							

(a) The relation between stress conditions and loading categories is not explicitly given in the table for liner plate allowables (CB-3700-1) of the proposed code.

The strain value to be compared to the strain allowables of this table shall be maximum biaxial or triaxial at the location of consideration.

Notation:

ϵ_{su} = steel ultimate strain in uniaxial tension
 F_a, F_y, F_u = allowable, yield and ultimate forces, respectively.
 δ_a, δ_u = allowable and ultimate displacements, respectively
 f_{cua} = compressive strength of concrete at time of test.

in concrete adjacent to the liner, are redistributed in the liner so as to maintain equilibrium and compatibility conditions. These redistributed strains, which may be obtained from a load-dissemination analysis, should be interpreted as the "membrane" strain except in areas where bending may occur (e.g., in an initially deflected panel).

With the above definition, limiting membrane stress and/or strains in the liner appear to be somewhat superfluous. The reason is that final panel strains are always less than "design" strains except in a weak panel where strains tend to accumulate. The concrete-imposed design strains are already limited by the provisions of section CB-3420 and therefore additional provisions are not needed.

Thus, it appears that limiting values for membrane plus bending would be sufficient for design purposes. These allowables would normally apply to those areas of the liner where strains are accumulated due to existence of a weak panel or where bending occurs due to discontinuities or liner attachments.

c. As pointed out earlier, design analyses in the past have usually been conducted for normal and upset stress conditions. In the proposed code, allowables are given for emergency stress condition (except for concrete bearing and shear allowables). This will probably result in a more comprehensive treatment of stress analysis problems.

d. The proposed code does not place any limitation on stresses and displacements (for liner assembly) under failure category loading combinations. This lack of provision appears to be in line with past design experience. In the past, liners have not been analyzed (nor designed) for failure category loading combinations.

e. The provisions for anchor forces and displacements also appear to be consistent with past design experience. The implied factor

of safety of 3 for anchors (mechanical loads - with respect to ultimate strength) under normal and upset conditions is somewhat greater than the value of 2.2 adopted in some British designs. In PCRV and containment vessel liner design practice in the United States, there does not appear to be a set factor of safety adopted in design criteria. In the Fort St. Vrain design, for example, the predicted anchor and cooling tube shear forces were simply compared to experimentally obtained ultimate strengths and the safety margins were indicated from this comparison.

In the containment vessel liner design practice adopted by Bechtel, the factor of safety for anchors is computed on the basis of the strain-energy of the anchors. In the proposed code, safety factors are defined in terms of displacements in the case of displacement limited loads. It can be shown that, for typical anchor stress-strain relationships, the factors of safety implied by the code displacement allowables are more conservative (i.e., it will result in a lower computed factor of safety for a given displacement) than Bechtel's approach.

3.3.4.b. Recommendations

Based on past design experience and discussions presented in Subsections 3.3.2 and 3.3.4, Table VII has been modified to obtain Table VIII. The modifications include types of liner stresses to be specified and loading categories corresponding to stress conditions. Allowable values have not been changed.

a. Liner stresses and strains: Since a liner plate is biaxially (or triaxially) loaded, the stress or strain value to be compared to the allowables in Table VIII should be the maximum biaxial (or triaxial) at the location of consideration.

Table VIII. Liner assembly allowables (as modified).

Stress condition	Loading category	Liner		Anchors		Concrete	
		Maximum compression	Maximum tension	Force	Displacement	Bearing	Shear between adjacent anchors
Normal and Upset	Construction Normal Abnormal	$\epsilon_a = \frac{\epsilon_{su}}{20}$	$1.5 S_m$	Lesser of: $F_a = 0.67 F_y$ $F_a = 0.33 F_u$	$\delta_a = 0.25 \delta_u$	Confined: $0.6 f_{cua}$ Unconfined: $0.2 f_{cua}$	$0.075 f_{cua}$
Emergency	Extreme environmental	$\epsilon_a = \frac{\epsilon_{su}}{10}$	$\epsilon_a = \frac{\epsilon_{su}}{10}$	Lesser of: $F_a = 0.9 F_y$ $F_a = 0.5 F_u$	$\delta_a = 0.5 \delta_u$	No limitation	No limitation
Faulted	Failure	No limitation					

Note: In a biaxial stress field, stress and strain values to be compared to the allowables of this table are as follows:

Tension-tension: $\sigma_1 \leq 1.5 S_m$

Compression-compression:

$$\frac{1}{1 - \mu^2} |(\epsilon_1 + \mu \epsilon_2)| \leq \epsilon_a$$

Tension-compression: $\sigma_1 - \sigma_2 \leq 1.5 S_m$

$$\left| \frac{\epsilon_1 - \epsilon_2}{1 + \mu} \right| \leq \epsilon_a$$

Notations:

ϵ_a = allowable strain

μ = Poisson's ratio

σ_1, σ_2 = principal stresses, $|\sigma_1| \geq |\sigma_2|$

(See Table VII for others.)

ϵ_1, ϵ_2 = principal strains, $|\epsilon_1| \geq |\epsilon_2|$

The maximum shearing stress theory of failure may be used in evaluating calculated stresses and strains. In the following, application of this theory to biaxial stress field is illustrated.

If σ_1, σ_2 ($|\sigma_1| > |\sigma_2|$) are principal stresses and S_a is the limiting stress intensity, the maximum shearing stress theory can be expressed as:

$$\text{if } \sigma_1, \sigma_2 \text{ are of the same sign: } |\sigma_1| \leq S_a \quad (28)$$

$$\text{if } \sigma_1, \sigma_2 \text{ are of opposite sign: } |\sigma_1 - \sigma_2| \leq S_a \quad (29)$$

Equations (28) and (29) can be expressed in strain forms. Using elastic theory:

$$\sigma_i = \frac{E}{1 - \mu} (\epsilon_i + \mu \epsilon_j), \quad i, j = 1, 2 \quad (30)$$

where: E = Young's modulus, and

μ = Poisson's ratio.

Substituting Eq. (30) in Eqs. (28) and (29):

$$\text{if } \sigma_1, \sigma_2 \text{ are of the same sign: } \left| \frac{\epsilon_1 + \mu \epsilon_2}{1 - \mu} \right| \leq \epsilon_a \quad (31)$$

$$\text{if } \sigma_1, \sigma_2 \text{ are of opposite sign: } \left| \frac{\epsilon_1 - \epsilon_2}{1 + \mu} \right| \leq \epsilon_a \quad (32)$$

where: ϵ_1, ϵ_2 = principal strains.

Thus, if σ_1, σ_2 are both tensile, then Eq. (28) applies with $S_a = 1.5 S_m$. If σ_1, σ_2 are both compressive, then Eq. (31) applies. If σ_1 and σ_2 are of opposite sign, then both Eqs. (29) and (32) presumably should be satisfied with $S_a = 1.5 S_m$.

The above interpretation of liner allowable stresses and strains is shown in Fig. 10 for the normal and upset stress conditions. In

this figure, allowable values are plotted in terms of principal strains. It is seen that Eq. (27) governs if σ_1 and σ_2 are of opposite sign. It is also seen that the allowable strain curve is discontinuous at the point where transition from tension compression to compression-compression stress field occurs.

b. Anchor forces and displacements: Anchor allowables are broken into two categories. Mechanical loads are those where deformation of the structure will not relax the condition causing the load. Loads transmitted by attached equipment and those due to water pressure behind the liner should probably be classified as mechanical loads.

The second category is the displacement limited loads which are self-limiting or self-relieving in nature. Most anchor loads probably fall into this category. For example, loads from which design strains are derived (i.e., prestressing, creep, thermal but probably not internal pressure) may be classified as displacement limited loads.

From a design viewpoint, it is important to correctly identify loading conditions and to use appropriate allowable criteria. Available experimental data indicate that, for stud anchors, allowable forces are more restrictive than displacements. Therefore, mechanical load criteria should probably be used whenever a loading combination includes a load which may be classified as a mechanical load.

Another important design problem is the interaction of calculated tensile and shear forces on the anchors. Unfortunately, the proposed code does not appear to address to this problem. It appears that tension-shear interaction has not been considered in past designs as well. Therefore, it is recommended that an interaction criteria be incorporated

in the code. Since there does not appear to be any data in this respect, it is suggested that an elliptic interaction relationship may be used. The tension-shear allowables for bearing type bolts are based on such an interaction curve in the Specifications of the American Institute of Steel Construction (AISC).

c. Concrete allowables: As indicated previously, the concrete bearing and shear allowables of Table CB-3422-1 are interpreted to apply to concrete stresses adjacent to cooling tubes. It was also pointed out (see Subsection 3.2.4) that it may be difficult to satisfy shear requirements between cooling tubes. However, there does not appear to be sufficient data to suggest modifications in these allowables.

3.4. Testing in Support of Design

3.4.1. Tests on Liner Segments

3.4.1.a. Evaluation of Testing Procedures

It is apparent from the review given in Subsection 2.5.1 that tests in the past have been conducted for two purposes:

- a) To observe the behavior of a particular specimen which represents a part of the actual liner assembly, and
- b) To develop stress-strain characteristics for strong and weak panels to be used in liner design analysis.

In general, the testing conditions (models, support conditions, loading devices) should simulate the actual conditions as closely as possible, in view of the nature of information to be obtained. For behavior observation, the entire liner system should be properly simulated. For obtaining panel characteristics, simulation of the liner panel is the important consideration.

Complete simulation of all actual conditions in a laboratory experiment is a very difficult and sometimes impossible task. Therefore, the models and test set-ups to be used usually involve many simplifications to the actual conditions. This was done by both British and GGA researchers. The main differences between actual structure and test conditions of the models are as follows:

a) In the prototype, stresses are induced in the liner plate by a combination of bearing, friction, and loading of anchors in shear. Further stresses are induced by restraints against thermal expansion and creep of concrete. In all models tested, the liner was stressed in edge compression.

b) The shear forces in the anchors will be resisted by adjacent concrete in compression. In the GGA's model on the other hand, anchors were wrapped in styrofoam in order not to load concrete. In Whessoe's test model there was no concrete and whether anchors were tightly fitted in the holes in the back-up plate to simulate shear loading is not known.

c) The adverse variations to be considered in stress analysis are rather arbitrary in nature. Some of these are based on permissible or possible variations from theoretical values. Simulation of these variations in tests would be somewhat difficult. Effects of the following variations appear to have been studied in liner segment tests:

1. Initial lateral deflection
2. Lateral pressure and concrete void
3. Misalignment of plate seam (1/8-in. weld offset)
4. Effect of cooling tube welded along the plate centerline
5. Loss of an anchor.

Other local variations (yield strength, thickness, and temperature) do not appear to be amenable to investigation by testing.

d) If the test results are to be used in design analysis, the stress-strain relationships for weak panels should be evaluated. In GGA's test program however, the characteristics of the overall plate specimen have been determined. It is stated that weak panel characteristics have been obtained from Whessoe's plate test program (10). Obviously, extensive instrumentation would be needed in order to determine actual stress distribution in the panel.

e) In GGA's model, stud anchors were not loaded in shear thus resulting in a uniform state of stress in all panels. If the studs were embedded in concrete, thus permitting shear loading, concrete cracking would probably have occurred presenting some problems. Therefore, it may be that, for the specific purpose of determining weak panel characteristics, the test plate should be supported directly by a thick steel plate without a concrete layer between the two plates.

3.4.1.b. Recommendations for Liner Plate Testing

In view of the past experiments and the necessity of a practical approach to the liner plate testing problem, the following recommendations are made:

a) In each prototype design (i.e., different material and geometric properties), sections of liner plate should be tested to determine weak panel behavior. In lieu of such tests, strut analogy may be used although testing is preferred.

b) In testing liner segments, the test set-up developed by GGA may be used. However, anchors should be permitted to be loaded in

shear so that actual conditions will be more nearly simulated, if this is possible without causing problems in the backing concrete.

c) The weak panel characteristics should include consideration of initial lateral deflection, lateral pressure (both toward and away from backing concrete or plate), loss of an anchor, and other probable local variations.

d) Extensive strain and deflection readings should be taken so that actual stress field in the weak panel can be established.

e) It is realized that obtaining the effects of some variations (such as thickness and yield strength variation between panels and hot spots) experimentally will be difficult. However, these variations can be incorporated in the design analysis, based on theoretical considerations as indicated in Ref. 31.

It is to be noted that the above recommendations are based on review of past experience, with the practical application of test data to design of liners in mind. In actual testing some of what is recommended may be too impractical or, conversely, other variations may be studied so that a more complete set of design data can be obtained. Therefore, the details of the test specimens and testing procedures should be left to the designer.

Tests of liner panels or systems for the purpose of behavior observation and evaluation, although not needed for liner design analysis, should be encouraged in any prototype design.

3.4.2. Tests on Anchor Systems

3.4.2.a. Evaluation of Testing Procedures

In order to evaluate the adequacy of past push-out tests for determining anchor shear characteristics and fatigue strength, it will be worthwhile to examine the factors which affect the behavior of anchors in such tests. These factors are:

- a) Dimensions of the model: slab thickness, stud embedment length, and distance from the stud to the edge of concrete. If the slab is shallow and thus embedment length is small, the failure will be by pulling out of the anchors. Since such a behavior is not expected (i.e., sufficient embedment length usually provided) in a reactor vessel, test results in such cases will not be useful (45). Therefore, a sufficient mass of concrete should be provided in order to prevent such a failure.
- b) Spacing of studs: Tests with 2 and 4 in. of spacing showed no effect of spacing on the stud shear capacity (51).
- c) Yield point and ultimate strength of studs: Tests should be conducted using the same studs as those which will be used in the vessel since the anchor characteristics depend on material properties.
- d) Concrete strength: It has been found that the magnitude of load for a given value of slip increases approximately in proportion to $\sqrt{f'_c}$ (51). On the other hand, concrete strength did not significantly influence the fatigue strength of stud shear connectors (44).
- e) Shape and thickness of the base material: The behavior (though probably not ultimate of fatigue strength) of studs also depends on the stiffness of the plate to which they are welded. In the tests mentioned above, studs were welded to a wide flange beam. The stiffness of a beam

flange would be different than that of a plate (with same thickness) because of the effect of beam web. Therefore, in determining the stud characteristics, plates should be used as the base material as was done in most British tests.

f) Existence of prestress: The stud behavior will also be influenced by existence of prestress. In some tests conducted in Great Britain, the concrete block in the push-out specimen was prestressed (8, 15). The effect of prestress, however, does not appear to be discussed in the literature.

g) In the fatigue tests reported in Ref. 44, minimum stress and stress range were such that failure occurred after a great number of load cycling (i.e., more than 10,000 cycles as is usual in conventional fatigue studies). On the other hand, in a stud which is welded to the reactor vessel liner, much fewer appreciable stress changes take place during the life span of the structure [for example, 125 cycles from atmospheric to normal working pressure for Fort St. Vrain (20)]. Fatigue data on low cycle-high stress behavior of anchors does not appear to be available.

3.4.2.b. Recommendations on Anchor Testing

The above review indicates that testing on stud anchor is needed in support of a prototype design. These tests should be conducted in such a manner that precise anchor characteristics and fatigue strength can be evaluated for use in liner design analysis.

a) Static tests: In push-out test specimens, actual variables should be simulated as much as possible. For example, studs may be welded to a box section made up of actual liner material rather than a wide flange section. Such a specimen, using 5/8-in. diameter studs,

has been tested. But the test results do not appear to be available (12). The concrete should be prestressed and be thick enough to accommodate the actual stud embedment length.

Another aspect of shear anchor testing is the effect of temperature on the behavior of studs. It has been stated that concrete temperatures up to 200 °F do not have any appreciable effect on the results. Since local temperatures greater than that are permitted by the new code (1), additional experimental work seems to be warranted in this respect.

b) Fatigue tests: Tests should also be conducted on studs, using similar push-out specimens as described above, in order to determine the endurance limit of individual studs. In these tests, minimum stress and stress range may be selected in such a way that failure will occur after fewer cycles than those reported in Ref. 44. The controlling factor in fatigue tests, however, may be the anchor displacement rather than stress. Thus it will be possible to relate predicted displacements to those obtained from tests, for a given number of cycles.

3.4.3. Tests on Cooling Tubes

3.4.3.a. Evaluation of Test Procedures

It appears that the specimens tested and the testing procedure used by GGA (Subsection 2.5.3) adequately simulates the actual conditions.

The most important aspect of the test results, so far as design analysis is concerned, is the magnitude of ultimate shear displacement of the cooling tubes. Test results showed that this displacement is about 0.08 in. (specimen type two which represents the cavity liner assembly). As pointed out earlier, for proper design of cooling tubes,

the shear forces on the tubes must be considered by including them in the analytical model. The predicted displacements can then be compared to those obtained from the tests in order to evaluate the factor of safety.

3.4.3.b. Recommendations on Cooling Tube Testing

There does not appear to be any need to conduct experiments on cooling tubes since there is adequate data in this regard. This conclusion, of course, applies to the future designs where design details are similar to that of the Fort St. Vrain PCRV liner. If a different design is used, additional testing, similar to that conducted by GGA, should be conducted in order to obtain reliable design data.

4. DESIGN REVIEW INFORMATION REQUIREMENTS

For an extensive and thorough review of a PCRV liner design, it is necessary that sufficient detailed design calculations be readily available to the reviewer. Inclusion of detailed design calculations in the safety analysis report (SAR) will make the SAR unduly voluminous. Furthermore, this is unnecessary since not all designs are prototype.

It is suggested, therefore, that the design review information be submitted as follows:

- a. For a prototype design: Submit safety analysis reports and a design report.
- b. For a nonprototype design: Submit safety analysis reports and a supplementary design report, if necessary. The design report for the prototype should be clearly indicated.

The design review information to be included in the SAR's is described in Section 4.1 and the details of actual data required are given in Section 4.2. The information generally consists of a description of the design, design bases, design analysis methods, and results of analysis. In essence, the SAR consists of the design specifications and a summary of the contents of the design report. The information in SAR's should be complete and self-contained but no detailed analysis or calculation should be necessary. Sufficient cross references to the design reports should be reported.

The design report is to be used to support the contents of SAR's. It should include detailed analyses for cavity liner, penetration liners, and local critical areas. In this report, it is not sufficient to merely indicate the method of analysis and/or computer code used, even though

the accuracy of these methods and computer codes may have been previously established. Rather, it is important that the details of how the analysis was conducted be described adequately. The details should include assumptions involved, description and justification of the analytical model used in each analysis, boundary conditions, input data, and the like. The contents of the design report may in general follow that of SAR's, but it is more important that each description of design analyses be provided with sufficient input data so that the design analysis can be evaluated with minimum reference to SAR's or other sources. If any test result is used in liner assembly design, details of the testing should be documented in the design report.

The supplementary design report should follow the format of the corresponding prototype design report and include only details of the design variations from the prototype and how they are handled in the design of the new liner.

It is believed that the above suggestion concerning design review information submittal will considerably reduce the amount of reporting and the review time required for nonprototype designs.

It is to be noted that the review, as discussed herein, is limited to the "Structural Design" aspects of the PCRV liner. Other aspects such as construction, inspection, nondestructive testing, and surveillance are excluded.

4.1. Narrative Description

4.1.1. Summary Description

The liner assembly should be described in sufficient detail and be compared to similar designs that have been recently used in other nuclear facilities. Adequate drawings showing the liner configuration and design details should be provided. If there has been any testing conducted to support the adequacy of the configuration being used, it should be briefly described. If there is development work being accomplished in support of the prototype design, it should be so indicated.

4.1.2. Codes and Procedures

Codes, standards, specifications, safety guides, office practices, and the like, which serve to guide and restrict the design and analysis processes, should be summarized. Documents referenced should be properly identified in the design information provided, whenever appropriate.

4.1.3. Material Properties

The liner component materials and their characteristics relevant to design should be presented. These characteristics should be given for both unirradiated and irradiated cases. Specific testing conducted to establish the material characteristics to be used in design should also be presented.

4.1.4. Design Bases

4.1.4.a. Functional Requirements

The design requirements imposed on the liner resultant from the predicted sequential facility operation should be described.

4.1.4.b. Performance Characteristics

The required and expected behavior of the liner assembly under constructional, operational, and extreme loads should be described. The possible or postulated failure modes under any condition should be indicated and discussed.

4.1.4.c. Loads

The loads which impose strains and displacements on the liner components should be discussed. These will normally include dead weight, construction loads, prestressing, pressure, seismic, thermal, creep, shrinkage, and local loads. If any load which might influence the performance of the liner is not included, the reasons for doing so should be discussed.

4.1.4.d. Loading Combinations

Combinations of loads which result in the design strains and displacements for the liner components should be specified. The manner of combination of loads and the load factors applied in each loading category should be discussed and justified.

4.1.5. Design Analysis

All critical areas in the liner assembly considered in the design should be identified. If a particular design is based on test results, applicability of and justification for using the tests in lieu of design analysis should be discussed. If analysis is not conducted for any possibly critical area, the reasons for that should be discussed.

The design analyses normally include the following:

- a) General analyses for cavity and penetration liners and
- b) Analyses of local critical areas.

In describing the design analysis conducted for each critical area, the information listed below should be provided.

4.1.5.a. Design Strains and/or Displacements

The design strains and/or displacements, obtained from overall vessel analysis, should be given, preferably in a tabular or graphical form. Separate data should be provided for each loading combination. The method or methods with which these strains and displacements are obtained should be described.

4.1.5.b. Analysis Model

The model of the section being analyzed (i.e., one-dimensional model; ring model, etc.) should be described in sufficient detail. Any assumption and idealization regarding the behavior of the model and its components should be described and justified.

4.1.5.c. Method of Analysis

The analytical technique used should be described in sufficient detail. If the same computer code is being used in several problems, it may be discussed only once. However, the input data for the problem at hand should be adequately discussed.

4.1.5.d. Local Variations

If, in addition to design strains and/or displacements, local variations are considered in the analysis, they should be listed and discussed. The manner in which local variations are determined and combined should be indicated and justified.

4.1.5.e. Analysis Results

The relevant analytical results should be given, preferably in tabular or graphical form. The code allowables should also be given in the table or graph.

4.1.5.f. Evaluation of Results

The significance of analytical results should be evaluated in view of code allowables and test results if available. Adequate discussion of the tests conducted should be provided.

4.2. Required Information Summary

General requirements for the design review information (primarily for SAR's) were described in the preceding section. In this section, specific data to be submitted is summarized. The data as used here should be interpreted as both descriptive and numerical. Whenever appropriate, the table form with check marks ("X") is used to indicate at a glance all data requirements belonging to a specific category. It is not implied that tabular forms are necessary in submitting these required data, although properly organized tables are often preferred.

The summary, as described in this section, applies directly to general cavity and penetration liner design. The format and scope of the data summary may be somewhat different in presenting the design analyses of local critical areas depending on specific conditions of the area in question.

4.2.1. Summary Description

Data to be provided are:

Item	Cavity liner	Penetration liners ^(a)
Vessel geometry showing liner locations	X ^(b) D ^(c)	X, D
Liner geometry	D	D
Diameter	X	X
Height	X	X
Thickness	X	X
Anchors		
Types	X, D	X, D
Dimensions	X, D	X, D
Spacing	X	X
Arrangement	X, D	X, D
Cooling tubes		
Types	X, D	X, D
Dimensions	X, D	X, D
Spacing	X	X
Arrangement	X, D	X, D
Length unbacked by concrete		X, D

(a) Data to be provided for each group of penetration liners.

(b) Data required.

(c) Reference drawing required.

4.2.2. Codes and Procedures

These would normally include the following:

Liner design codes,

AEC safety guides, and

Design standards.

The following information should be provided for each document:

Reference number for the document,

Name of the document,

Organization which published the document, and

Year published.

4.2.3. Material Properties

Material properties of primary importance in structural design of liners are listed below:

Item	Cavity liner ^(a)	Penetration liner ^(b)	Shear anchors
Material identification	X	X	X
Unirradiated properties			
Yield point	X	X	X ^(c)
Ultimate strength	X	X	X
Ultimate uniaxial elongation	X	X	
Ultimate shear slip			X
Coefficient of thermal expansion	X	X	
Modulus of elasticity	X	X	X
Poisson's ratio	X	X	
Irradiated properties			
Yield point	X	X	
Ultimate strength	X	X	
Ultimate uniaxial elongation	X	X	
NDT shift	X	X	
Test references	X	X	X

(a) Data to be provided for each different liner material used.

(b) Data to be provided for each group of penetration liners.

(c) If the shear anchor does not exhibit a clearly defined yield point, the actual load-slip characteristics of anchors should be provided.

Material properties of secondary importance in liner design, such as those of backing concrete and weld metals, should be given in connection with the discussion of allowables or other appropriate items.

4.2.4. Design Bases

4.2.4.a. Functional Requirements

Data to be provided in this category are:

Pressure:

Testing pressure,
Normal operating pressure,
Peak operating pressure,
Maximum cavity pressure, and
Design pressure.

Liner temperature:

<u>Loading category</u>	<u>Effective liner temperature</u>	<u>Local at hot spots</u>
Normal operation	X	X
Abnormal	X	X
Extreme environmental	X	X

Total maximum neutron irradiation at liner.

In view of the fact that standard definitions of different types of pressure and temperature conditions are yet to be established, it is suggested that these pressure and temperature conditions be clearly defined.

4.2.4.b. Performance Characteristics

Data to be provided are:

Loading category	Cavity liner	Penetration liners (a)	Shear anchors
Construction	X	X	X
Normal	X	X	X
Abnormal	X	X	X
Extreme environmental	X	X	X

(a) Data to be provided for each group of penetration liners.

In evaluating performance characteristics of the liner components, the following failure modes should be considered:

Buckling,

Lateral deflections,

Rupture of liner,

Rupture of anchor,

Concrete failure,

Fatigue failure, and

Brittle failure.

4.2.4.c. Loads

Loads considered in liner design should be given:

Dead loads,

Prestressing loads,

Creep and shrinkage,

Thermal loads,

Internal pressure,

Seismic loads,
Live loads, and
Wind loads.

If appropriate, reference drawings should be provided.

4.2.4.d. Loading combinations

All loading combinations considered in design should be listed.

These include combinations in the following categories:

Construction,
Normal,
Abnormal, and
Extreme environmental.

4.2.5. Design Analyses

All critical areas analyzed, including cavity liner, each group of penetration liners, and other local areas, should be listed. This list should include:

Identification of each critical area,
Reference drawing number, and
Reference page number in the design report.

The following set of data should be provided for each case analyzed:

4.2.5.a. Design Strains and/or Displacements

Cavity and possibly most penetration liners:

Design strains and/or displacements in both circumferential
and axial directions.

Local critical areas:

Two- or three-dimensional strains or displacements.

4.2.5.b. Analysis Model

Each model should be defined by:

Type of model (one-, two-, or three-dimensional),

Dimensions of the model, including number of panels,

Boundary conditions, and

Reference drawing.

4.2.5.c. Method of Analysis

Liner analysis method should be identified in terms of:

Stability or stress analysis,

Elastic or inelastic,

Name identifying the method,

Assumptions used in the method,

Analysis technique,

Computer code,

Input data,

Previous application, and

References.

4.2.5.d. Local Variations

Data to be provided include:

Nature of local variations considered,

Extent of each local variation considered, and

Local variation combinations considered.

4.2.5.e. Analysis Results and Evaluation

Data in this group may be summarized in the following tabular form:

Item	Location	Computed	Allowable	Bases for allowable
Liner				
Maximum stress intensity or strain ^(a)	X	X	X	X
Cumulative usage factor	X	X	X	X
Maximum lateral displacement ^(a)	X	X	X	X
Anchors				
Maximum shear force	X	X	X	X
Maximum shear force range	X	X	X	X
Maximum shear displacement	X	X	X	X
Maximum tensile force	X	X	X	X
Maximum combined shear and tensile force	X	X	X	X
Cooling tubes				
Maximum shear force	X	X	X	X
Maximum shear displacement	X	X	X	X
Maximum lateral displacement	X	X	X	X
Concrete stresses				
Maximum bearing stress	X	X	X	X
Maximum shear stress	X	X	X	X
Maximum stress and displacements in other liner attachments	X	X	X	X

^(a) These could be based on stability or stress analysis.

It should be emphasized that a satisfactory SAR for expedient review of design must have a proper combination of description, numerical data, and reference drawings. Sections 4.1 and 4.2 should therefore be considered mutually supplementary. It is further noted that the required information as presented in Sections 4.1 and 4.2 may be construed as a minimum requirement. Additional data, if relevant to the behavior and design of the liner assembly, should be provided by the designer.

5. DESIGN REVIEW GUIDELINES

It was recommended in Chapter 4 that a Design Report be submitted for each prototype design and that Safety Analysis Reports be submitted for each prototype and nonprototype design. The review procedures for both are discussed in Section 5.1 and acceptance standards and bases are discussed in Section 5.2.

5.1. Review Procedure

During the review process, due attention should be given to the following criteria:

a) Completeness of design information provided

Information provided in SAR's and the Design Report should cover all aspects related to the design of the liner assembly in a well-organized manner in order to clearly and adequately portray the design product and its soundness. Information should be properly referenced where appropriate. Reports prepared in accordance with the requirements of Chapter 4 will normally meet this criterion.

b) Adequacy of design bases and design procedures

This includes considerations such as:

Are applicable and up-to-date codes and guides used?

Are functional requirements clearly stated and generally consistent with current practice?

Are performance characteristics generally consistent with current design practice?

Loads and loading combinations used.

Methods of design analyses used.

The acceptance standards and bases for the last two items are described in detail in Section 5.2.

c) Safety of structure designed

The liner assembly, in parts or as a whole, should meet all structural safety requirements stipulated in applicable codes, safety guides, and the like. When testing is employed to verify the safety of design, it should be clearly described and compared with the design product.

If the Safety Analyses Report is based on a Design Report which has already been reviewed and approved, the review of most of the items listed in b) above can be accomplished with a minimum of effort. If the SAR is for a prototype vessel for which a Design Report is yet to be submitted, a more careful review will probably be necessary. In this case, approval of the SAR may be granted contingent upon submittal of an acceptable Design Report.

In reviewing the Design Report, emphasis should be placed on evaluating design procedure adequacy. Design analyses necessarily involve many idealizations and assumptions regarding the behavior of the structure. These idealizations and assumptions are based on both engineering science and individual judgment. It is important that the judgment used in design analyses be both adequate and appropriate. It is believed that this can only be evaluated through careful examination of all aspects of analysis details.

The above description of review procedure was prepared with the assumption that the liner design information will be available in a self-contained form, independent from the overall PCRV design information. If this is not the case, a computer program might prove to be useful in extracting and listing the liner review data required.

It is also assumed that, in general, no numerical calculation will be performed by the reviewer in the reviewing process. The review procedure described above emphasizes the importance of conceptual aspects of the design procedures, adequacy of the input data on which the design is based, and acceptability of the experimental and analytical results obtained.

5.2. Acceptance Standards and Bases

The proposed ACI-ASME Code (1), once adopted, will be the official standard with which all designs must comply. The acceptance standards for various design information in the categories of design bases and design analyses as presented in this section are intended to comply with, and should be regarded as a supplement to, the proposed code. These standards are derived from the evaluation of current practice presented in Chapter 3 and should help expedite design review process since specific recommendations are given for many "requirements" which are only briefly mentioned in the proposed code. The acceptance bases are not fully given here since they were discussed in detail in Chapter 3. The order of subject matter discussed parallels that of the design information requirements presented in Section 4.2.

Since the proposed code should be readily available for reference purposes in the reviewing process, specific requirements appearing in the code (e.g., tables on allowables) are not repeated in this section.

5.2.1. Loads

The loads from which linear assembly design strains and/or displacements are determined should normally include dead weight, prestressing

loads, concrete creep and shrinkage, thermal loads, internal pressure, seismic loads, and local loads. Wind, construction live loads, and hydrostatic pressure should be considered in construction category loading combinations.

The local loads should include reactions at the reactor internal component support points, pipe reactions due to thermal expansion, and others. Another type of local effects is discussed under the heading of local variations.

5.2.2. Loading Combinations

The loading categories for which the liner assembly should be designed and analyzed are construction, normal, abnormal, and extreme environmental categories.

Severe environmental and failure loading categories are not normally applicable to liner assembly design. In the case of severe environmental loading category, the only additional load which imposes strains in the liner components is the Operating Basis Earthquake selected for the PCRV site. Considering the fact that earthquake-induced liner strains are very small compared to strains due to other loads, it is concluded that liner design analysis for this category is not needed since it will have little effect on the final magnitudes of forces and displacements in the liner assembly.

The failure category loading combination need not be considered in liner design either. It is reasonable to conclude that under faulted conditions, gross failure of the liner assembly is highly unlikely and that local failure will not have any meaningful effect on the overall vessel behavior (see Subsection 3.2.5).

Specific loading combinations in each category are considered to be beyond the scope of this study. However, as a general recommendation, it is suggested that the actual loading combination should reflect the planned sequential operation of the vessel. This implies that loading combinations should include the conditions where the vessel will be at design temperature with or without internal pressure and, possibly, the condition where the liner may be cold with internal pressure since this case may result in tensile stresses in the liner. The loading combinations should also include the condition of the vessel at the end of its design life. Appendix C in the proposed code gives a list of typical load combinations for the PCRVR in general.

5.2.3. Design Strains and/or Displacements

The liner design strains and displacements will normally be obtained from overall vessel analysis using a finite element computer program. In the vessel analysis, sufficient nodal points should be included so that an accurate design strain profile can be obtained. Analysis shows that nonuniform design strains induce appreciable forces in the anchor system and therefore accurate determination of design strains is essential.

The design strains and displacements should be determined for all loading combinations so that fatigue response of the liner assembly can be adequately evaluated.

In the analysis of the overall vessel, the liner may or may not be included. This is acceptable since the inclusion of the liner has little effect on design strains and displacements.

5.2.4. Analysis Model

5.2.4.a. General Cavity Liner Stress Analysis

For the general analysis of cavity liner, a one-dimensional model is adequate. In this model an assumed "weak" panel should be postulated and adverse variations as described in Subsection 5.2.6 should be applied.

The model should include a sufficient number of nodes. For this purpose, the criterion that the displacement of the last anchor should be within 2% of the maximum anchor displacement (adjacent to the weak panel) may be used. At least two models, one in the circumferential direction and the other in the axial direction, should be analyzed.

The boundary conditions for the circumferential model may be assumed to be fixed. In the axial direction, actual boundary conditions at midheight and at the periphery of top (or bottom) penetrations may be used. The effect of curvature and concrete bearing may be neglected since this will give conservative results.

In one-dimensional models, the liner panels and anchors may be represented by springs. Properties of these springs are discussed in Subsection 5.2.5.

5.2.4.b. General Penetration Liner Analysis

In the case of penetration liners anchored to the concrete in a manner similar to the cavity liner, there is no need for additional analysis if the geometric properties of the penetration liner assembly (other than curvature) are the same as the cavity liner. Otherwise, a general penetration liner stress analysis should be conducted in a manner similar to cavity liner stress analysis.

In a case where the penetration liner is not supported by closely spaced anchors, stability of the liner under concrete-imposed compressive

strains is the major problem. In such cases, stability analyses should be conducted, taking into consideration local variations discussed in Subsection 5.2.6, since these local variations will affect buckling response of the liner shell substantially.

The model used in stability analysis should be either a ring model (if the penetration liner length-diameter ratio is large enough) or the complete liner. In either case, the effect of rigid cavity in which the liner is placed should be taken into account, since the behavior of such shells is quite different than that of shells without concrete encasement.

In view of possible difficulties in stability analysis, design of unsupported penetration liners may be based on adequate testings.

5.2.4.c. Analysis of Local Critical Areas

The critical areas which should be considered in separate design analyses include, but are not limited to, the connections between gas duct liners and cavity or penetration liners, and the areas where the reactor internal component reactions are transmitted to the liner. The models to be used in these areas, in general, would depend on the geometry of the area being considered. As a general guide, it may be stated that the boundaries of the model should be sufficiently removed from discontinuities and stress concentration points. The resulting model would be either a two-dimensional (plane stress case) or a three-dimensional structure with known initial strains and/or boundary displacements. These strains and displacements, as before, should be determined from overall vessel analysis.

5.2.5. Method of Analysis

5.2.5.a. One-Dimensional Stress Analysis

The following one-dimensional analysis methods are considered to be acceptable (see Subsection 2.3.3 and 3.2.3):

- a) Parker's method,
- b) Iterative force-distribution method, and
- c) Initial stress method.

However, other stress analysis methods may be used provided that the following significant factors are properly taken into account:

- a) Material nonlinearities,
- b) Variable design strains, and
- c) Local variations.

In the method employed, the strong panel behavior may be assumed to be elastic or elastic-perfectly plastic. If elasticity is assumed, the results will be on the conservative side. If elastic-plastic behavior is assumed, the yield point should be established on the basis of biaxial test results.

In the case of a weak panel, plasticity must be considered since strains tend to accumulate in a weak panel. The weak panel characteristics should preferably be based on test results. In lieu of test results, theoretical load-inplane relationships for struts may be used (see Subsection 3.2.4).

The load-displacement relationships of anchors should be based on test results. The tests should simulate the actual conditions as much as possible (see Subsection 3.4.2).

5.2.5.b. Stability Analysis

Unfortunately, there does not appear to be a satisfactory method of stability analysis for liners without closely spaced anchors. Therefore, it is concluded that the design of such liners will probably have to be based on experimental studies, as mentioned previously. In this case, the applicability of experimental data to the problem at hand would require careful examination since exact simulation of actual liner conditions in an experiment would be rather difficult. In addition, due attention must be given to the effects of local variations discussed in Subsection 5.2.6.

If an analytical approach is attempted, due consideration should be given to plasticity of the liner as well as local variations.

5.2.5.c. Local Analysis

It is virtually impossible to prescribe rules for an acceptable method for the analysis of a local critical area. Furthermore, in some cases a two-dimensional plane stress method may be applicable whereas in other cases the idealized model may require a three-dimensional analysis. Whether a finite element method or an analytical method based on principles of theory of elasticity is suitable would also depend on the problem at hand. It is concluded therefore that each technique used in the analysis should be evaluated as to its adequacy and applicability in view of the physical conditions of the model and loading conditions to which such a model is subjected. It may be stated that the methods used in local analyses will normally be more refined and require complicated mathematical formulation.

5.2.6. Local Variations

All possible or postulated local variations in material properties and loading conditions should be considered in design analyses. These should include:

- 1) Variation in liner thickness.
- 2) Variation in liner yield point.
- 3) Initial inward curvature.
- 4) Concrete void behind liner.
- 5) Water pressure behind liner.
- 6) Local hot spots.
- 7) Loss of an anchor.
- 8) Misalignment of liner seams.

The manner in which the above variations should be combined and included in different loading categories should probably be based on planned sequential operation of the vessel and probabilistic studies. Suggested local variation combinations for the four loading categories mentioned previously are shown below:

Loading category		Local variation ^(a)							
		1	2	3	4	5	6	7	8
Construction		Consistent with planned fabrication and construction sequence.							
Normal	1	X	X	X			X	X	X
	2	X	X		X		X	X	
Abnormal	1	X	X	X			X	X	X
	2	X	X		X		X	X	
Extreme environmental		X	X	X		X	X	X	X

^(a) Local variation numbers refer to the above listing.

The bases for the above suggestions are discussed below:

a) Variations to be considered in construction category loading combinations: The liner loads and local variations to be considered depend on the fabrication and construction sequence which may vary for a different design. Therefore, no specific combination is given.

b) Variations to be considered in normal category loading combinations: The first combination in this category should be used in conjunction with the loading combination in which internal pressure is zero or a vacuum condition exists. Normally, in this case, the design strains will be near maximum.

The second combination is intended for the condition where internal pressure exists. In this case, the panel adjacent to a concrete void will be the weak panel with considerably smaller load carrying capacity. There is no need to consider initial inward deflection and misalignment of liner seams in this case. This is reasoned from the observation that the influence of initial deflection and misalignment on weak panel characteristics is small when a significant lateral pressure exists on the weak panel.

The condition where a panel with initial inward deflection exists next to a panel with initial outward deflection (and concrete void behind it) need not be considered. In such a case, there will be two weak panels to take up the strains from the strong panels (when the strong panels unload) and thus it will not be critical.

c) Variations to be considered in abnormal category loading combinations: The combinations and their purposes given in this category are the same as those of the preceding category. It is to be noted that the magnitudes of differential thermal strains to be applied at hot spots are different for different loading categories.

d) Variations to be considered in extreme environmental category loading combinations: In this category, only the combination which is to be used in connection with the condition of no internal pressure is suggested. A second combination, with internal pressure, is not needed since the increase in forces and displacements (compared to the abnormal category) will not be nearly as much as the increase in allowables.

The water pressure behind the liner due to cooling-tube leakage is considered to be an emergency condition since it is extremely unlikely that such a condition will occur.

The above combinations should be interpreted as examples and may be applied to the case of general cavity and penetration liner stress analysis. The designer should take into account all possible combinations and justify any combination which is not considered.

In the case of local analysis, the type and combination of local variations to be included should be judged based on the nature of the problem at hand. No specific recommendation will be attempted here in this regard.

The manner of incorporating local variations in design analysis was illustrated in Subsection 3.3.3, using an actual design example. Similar design calculations are considered to be acceptable.

The magnitudes of local variations to be considered in analysis should be based on codes, the design specification, and relevant vessel design data as illustrated in Subsection 3.3.3.

One last remark worth mentioning at this point is that, obviously, the design analyses to be conducted following the recommendations of this section will be quite voluminous. It may be stated that some of

the analyses need not be done, provided a worse condition has been analyzed and found satisfactory under the same criteria.

5.2.7. Analysis Results

5.2.7.a. Liner

Maximum stress intensity or strain: In evaluating the liner stresses and strains, only the combined membrane plus bending case needs to be considered since membrane stresses and strains are already limited by concrete vessel provisions.

In biaxial stress fields, tension allowables should be used if principal stresses are tension-compression. The reason is that in the proposed code tension allowables are more restrictive. In triaxial stress fields, similar provisions should be applied.

Cumulative usage factor: In fatigue considerations, the cumulative usage factor should be computed per ASME code, Section III, Division 1.

Maximum lateral displacement: Maximum inward liner displacement should be computed, possibly by approximate methods. This deflection should be limited to permissible values given in the Design Specification.

5.2.7.b. Anchors

Maximum shear force and displacement: Anchor shear forces and displacements computed under the loading categories listed previously should be limited to the proposed code allowables.

In evaluating the predicted maximum anchor forces and displacements, the displacement limited-load allowables may be used in cases where internal pressure and water pressure behind the liner do not exist. This is permissible since all the other loads may be classified as

displacement limited loads. However, it is acceptable to use mechanical load allowables in all cases. In typical PCRV liner anchors, the mechanical load allowable is more restrictive than displacement load allowables.

It is suggested that even in the case of internal pressure, displacement limited load allowables should be applicable in concrete-backed sections of the liner. This is reasoned from the fact that the backing concrete limits lateral (out-of-plane) displacements of the liner.

Maximum shear force range: The Design Specification for a PCRV will normally specify different magnitudes of cyclic loads with corresponding numbers of cycles. In evaluating the fatigue data, it is adequate to consider only the maximum load range (such as from zero to normal operating pressure) for the total number of cycles specified. If this is done, then only a set of shear force range - number of cycles needs to be considered in design analysis. If this simplification results in fatigue conditions beyond the allowable, then a cumulative usage approach, similar to that for liners, may be suggested.

In any event, the cyclic shear force range should be limited to values which are determined (preferably by experimental means) to be safe with a proper factor of safety.

Maximum tensile force: Tensile forces in the anchors will be induced due to buckling (or, more correctly, inelastic bending) of a panel and due to water pressure behind the liner. The first of these should be evaluated in all loading categories. The second, that due to water pressure, may be added in the extreme environmental category only.

The proposed code does not have any provisions for anchor tensile forces. Since an anchor length sufficient to develop the full strength of the anchor is usually provided, allowables on the bases of ultimate strength are adequate. As in the case of shear forces, a factor of safety of 3 for normal and upset conditions and 1.5 for the emergency condition are considered to be adequate. Note that these allowables are on the bases of mechanical loads.

Maximum combined shear and tensile force: In this case, as in the case of tensile forces, the proposed code does not contain any provisions. In the absence of any code allowable, the elliptical interaction approach of the American Institute of Steel Construction (AISC) specifications is considered to be adequate.

5.2.7.c. Cooling Tubes

Maximum shear force and displacement: The shear forces in the cooling tubes should be evaluated by including them in any analytical model. If the design is based on test results, adequacy of design should be evaluated in the light of test results.

The mechanical load-displacement limited load considerations for anchors are also applicable to cooling tubes. The cooling tube displacements in the plane of the liner are probably more critical in view of the fact that, in tests, cooling tubes have exhibited less ductility than the stud anchors.

In the design of cooling tube welds, stresses due to shear and radial forces should be considered.

If the allowable shear forces are based on test results, as is usual, the adequacy and applicability of the test procedure should be indicated.

Maximum lateral displacement: It is expected that cooling tubes may displace in the radial direction in regions where panel buckling occurs. This displacement will be compounded by possible water pressure behind the liner. However, there is no need to limit such displacements if the following rules are observed:

a) The strength of the tube weld should be larger than concrete shear forces which resist the movement of the tube in the radial (out-of-liner plane) direction.

b) The adjacent liner plate lateral deflection is within the allowable value.

5.2.7.d. Concrete Stresses

Maximum bearing stress: The bearing stress under the cooling tube side wall should be within the code allowable.

Maximum shear stress: The concrete shear stresses between adjacent cooling tubes should be evaluated from the computed cooling tube shear forces. The allowable of Table CB-3422-1 in the proposed ACI-ASME code, if it is assumed to be applicable to this case, appears to be somewhat too restrictive. It is concluded that higher stresses may be permitted since the loading condition is displacement limited and any micro-crack in the concrete will be confined to a small area.

5.2.7.e. Maximum Stresses and Displacements in other Liner Attachments

Stresses and displacements in other liner attachments, mainly in local areas for which local analysis is required, should be adequately determined and evaluated in view of appropriate code allowables.

6. SUMMARY AND CONCLUSIONS

An overview of current PCRV liner design practice was presented in Chapter 2. Evaluation of current design practices, including some analytical methods in use, was given in Chapter 3. Based on this evaluation, information requirements for an efficient and complete design review were listed in Chapter 4. In Chapter 5, guidelines for the review process and standards for acceptance and their bases were presented.

The scope of the liner design problem extends far beyond the expertise of any one or two individuals since the liner assembly is a complicated structure with many parts, each having different properties, different functional requirements, and different performance criteria. The materials presented in this report resulted from an effort made to review the behavior characteristics and design problems of liners in a broad sense and from a structural engineer's viewpoint.

Because of limitations on time and manpower available for this study, the emphasis was placed on the behavior and design of general cavity and penetration liners. The importance of local critical areas, of which there are so many, and their associated problems were recognized and indicated without attempting to draw conclusions and to develop guidelines for design assessment.

The following summary conclusions, therefore, are intended to apply to general design and analysis of cavity and penetration liners.

1. There does not appear to be any need, nor reason, to place limitations on the type of materials and geometry of the total assembly, as long as the resulting design can be shown to be satisfactory by adequate analytical and/or experimental methods.

2. In general, closely spaced anchors will be needed for shells with a large diameter-thickness ratio. There can be no prescribed values for the two adjectives used in the preceding sentence, namely, "closely" and "large." Each design must be substantiated by adequate means.

3. The proposed code, with a few exceptions, provides sufficient and acceptable design criteria. Lack of explicitness on the definition of mechanical and displacement limited loads and on the allowables for cooling tubes and adjacent concrete and a lack of criteria on shear-tension interaction for anchors are to be noted. Also, specific membrane allowables on concrete-backed liners are considered to be superfluous. A clear indication of how the allowables should be applied in the case of combined tension-compression in biaxial or triaxial field is also needed.

4. For an efficient and thorough review of a design, the detailed design should be submitted in a "Design Report." The review of the Design Report should be done manually, making certain that analytical models and methods, related assumptions, derivation and application of input data, and evaluation of results in view of code allowables and test data are all adequate and appropriate.

5. Such a review can be accomplished by following the guidelines developed in this report. It is to be noted that, in the opinion of the authors, there is no need for the reviewer to make detailed calculations, so long as the approaches used are considered to be acceptable and the results presented are complete.

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APPENDIX A. FIGURES

Fig. 1. Usual buckling form of a ring (from Ref. 28).

Fig. 2. Circumferential strain vs included angle for equilibrium states (from Ref. 28).

Fig. 3. Variation of uniform minimum buckling strains with anchor spacing.

Fig. 4. One-dimensional models used by Parker (10) and Doyle and Chu (32).

(a)

(b)

Fig. 5. Model used by Bechtel (33).

Fig. 6. One-dimensional stress analysis model used in this study.

Fig. 7. Anchor force-design strain relationships as influenced by assumptions regarding strong panel behavior.

Fig. 8. Idealized panel characteristics.

Fig. 9. Comparison of panel (experimental) and strut (theoretical) characteristics.

REGION SEGMENT	STRESS FIELD	EXPRESSION*
1	TENSION-TENSION	$\sigma_1 \leq S_a, \sigma_2 \leq S_a$
2	TENSION-COMPRESSION	$ \sigma_1 - \sigma_2 \leq S_a$
3	COMPRESSION-COMPRESSION	$\left \frac{\epsilon_1 + \mu \epsilon_2}{1 - \mu^2} \right \leq \epsilon_{ac}, \left \frac{\epsilon_2 + \mu \epsilon_1}{1 - \mu^2} \right \leq \epsilon_{ac}$

* BASED ON MAXIMUM SHEARING STRESS (TRESCA) THEORY

NOTE: $S_a = 1.5 S_m$, $\epsilon_{at} = \frac{S_a}{E}$, $\epsilon_{su} = 20\%$, $\epsilon_{ac} = \frac{\epsilon_{su}}{30}$, $\mu = 0.3$

Fig. 10. Graphical representation of proposed code allowables in bi-axial stress field-normal and upset conditions.

APPENDIX B. NOMENCLATURE

A, B	= constants used in describing Bechtel's method
a	= anchor spacing
a_i	= panel length for panel i
b	= anchor spacing perpendicular to one-dimensional model for liner stress analysis
c_1, c_2, c_3	= constants used in describing Parker's method
D	= dead load
E	= modulus of elasticity
F_a	= allowable force in an anchor
F_c	= anchor force in anchor 1 in Bechtel's method
F_i	= final force per unit width in panel i
$\bar{F}_i$	= design force per unit width in panel i
F_i^0	= initial force per unit width in panel i
F_t	= prestressing force at time of consideration
F_u	= ultimate force
F_y	= yield force
f'_c	= compressive strength of concrete
f_v	= allowable concrete shearing stress between cooling tubes
f_{cua}	= compressive strength of concrete at time of test
h	= thickness of liner plate
K_B	= weak panel stiffness
K_C	= anchor stiffness
K'_C	= anchor stiffness in inelastic range
K_R	= strong panel stiffness
L	= live load

N	= initial membrane force; number of cycles
N'	= imaginary force to be applied at anchor 1 in Bechtel's method
P_o	= normal operating pressure
P_r	= reference pressure
P_t	= pre-operational test pressure
q	= lateral pressure
R	= radius
S	= stress range
S_a	= allowable stress intensity
S_m	= design stress intensity per ASME Section III, Division 1
S_n	= shear force in anchor n which is between panels i and i + 1, per unit width
S_u	= ultimate anchor shear force per unit width
T_d	= design temperature
T_o	= normal operating temperature
u_n	= tangential displacement of the nth anchor (or node)
V_u	= ultimate shear force
W	= wind load
w_{om}	= initial lateral deflection of a weak panel
α	= included angle for defining buckled form
ΔT	= temperature increase
δ	= total displacement at anchor 1 in Bechtel's method
δ_a	= allowable displacement for anchor
δ_e	= elastic limit displacement for anchor
δ_{ij}	= displacement at anchor i due to release of anchor j
δ_u	= ultimate anchor displacement

ϵ_1, ϵ_2	= principal strains
ϵ_a	= allowable strain
ϵ_{ac}	= allowable compressive strain
ϵ_{at}	= allowable tensile strain
ϵ_c	= design strain
ϵ_{ci}	= average biaxial design strain in panel i
ϵ_{si}	= average biaxial liner panel strain in panel i
ϵ_{su}	= ultimate uniaxial tensile strain in steel
ϵ_x, ϵ_y	= strains in x- and y-direction, respectively
ϵ_y	= yield strain
ϵ_z	= axial strain
ϵ_θ	= circumferential strain
μ	= Poisson's ratio
σ	= stress
σ_1, σ_2	= principal stresses
σ_{bp}	= final stress in the buckled panel
σ_o	= initial panel stress
σ_y	= liner yield stress
σ_θ	= circumferential stress