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Insights Into the Behavior of Nuclear Power Plant Containments During Severe Accidents

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Insights Into the Behavior of Nuclear Power Plant Containments During Severe Accidents

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Abstract

The containment building surrounding a nuclear reactor offers the last barrier to the release of radioactive materials from a severe accident into the environment. The loading environment of the containment under severe accident conditions may include much greater than design pressures and temperatures. Investigations into the performance of containments subject to ultimate or failure pressure and temperature conditions have been performed over the last several years through a program administered by the Nuclear Regulatory Commission (NRC). These NRC sponsored investigations are subsequently discussed. Reviewed are the results of large scale experiments on reinforced concrete, prestressed concrete, and steel containment models pressurized to failure. In conjunction with these major tests, the results of separate effect testing on many of the critical containment components; that is, aged and unaged seals, a personnel air lock and electrical penetration assemblies subjected to elevated temperature and pressure have been performed. An objective of the NRC program is to gain an understanding of the behavior of typical existing and planned containment designs subject to postulated severe accident conditions. This understanding has led to the development of experimentally verified analytical tools that can be applied to accurately predict their ultimate capacities useful in developing severe accident mitigation schemes. Finally, speculation on the response of containments subjected to severe accident conditions is presented.

MASTER

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We have made maximum use of the work perviously funded by the U.S. Nuclear Regulatory Commission. The technical and financial support of the NRC has allowed the state of the art in predicting ultimate behavior of containment structures to be improved significantly. The level of validation of analytical techniques used for ultimate behavior analyses has impacted not only the nuclear community, but also the structural community.

Additionally, we would like to recognize the past and present members of the Containment Technology Division at Sandia National Laboratories and thank them for their contributions to the research that has been highlighted in this report. A special thanks is due to Mr. D. Clauss who has participated in many of the testing programs, as well as contributing to many of the references covered in this report. We would like to thank Dr. R. Weatherby for his help in reviewing this report, and especially for his insights in and analysis of reinforced concrete models, and his continuing work in the area of liner tearing analysis. Also, we thank Dr. R. Watson for his timely editorial and technical contributions to this report.

We also thank the many analysts throughout the world who participated in the pretest and posttest evaluations. Their contributions have gone to further our understanding of ultimate containment performance and have in a marked way improved the scope and impact of this research.

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ACRONYMS

AEA	Atomic Energy Authority
ANL	Argonne National Laboratory
BNL	Brookhaven National Laboratory
BWR	Boiling Water Reactor
CEA	Commissariat a L'Energie Atomique
CEGB	Central Electricity Generating Board
CTL	Construction Technology Laboratories
EJMA	Expansion Joint Manufacturers Association
ENEA	Comitato Nazionale per la ricerca e per lo sviluppo dell'Energia Nucleare e delle Energie Alternative
EPA	Electrical Penetration Assemblies
EPDM	Ethylene Propylene
EPRI	Electric Power Research Institute
GRS	Gesellschaft fur Reaktorsicherheit
ILRT	Integrated Leak Rate Test
INEL	Idaho National Engineering Laboratory
LOCA	Loss of Coolant Accident
LWR	Light Water Reactor
NII	HM Nuclear Installations Inspectorate
NRC	Nuclear Regulatory Commission
PWR	Pressurized Water Reactor
SIT	Structural Integrity Test
SNL	Sandia National Laboratories
SNUPPS	Standardized Nuclear Unit Power Plant System
SSE	Safe Shutdown Earthquake
TMI	Three Mile Island

Insights Into the Behavior of Nuclear Power Plant Containments During Severe Accidents

1. INTRODUCTION

In recent years, severe reactor accidents have been postulated that could result in pressure and temperature inside a containment building greater than its design basis loads. The performance of a containment building in the event of a severe accident must be known in order to assess the potential consequences and risks to public health and safety; it can also be used to improve the design of new containments. In particular, we need to reliably predict a containment's pressure capacity and the timing, mechanism, and location of a potential failure. Because its function is to contain radioactive materials, failure of a containment is defined as the loss of integrity (i.e., the release of a significant amount of radioactive material).

The Nuclear Regulatory Commission (NRC) is sponsoring a number of programs, collectively termed the Containment Integrity Programs, that address the issue of Light Water Reactor (LWR) containment performance for loads beyond the design basis. Sandia National Laboratories (SNL) is managing four of these programs: (1) scale model tests of containment buildings, (2) tests of seals and mechanical penetrations, (3) development of analytical techniques, and (4) tests of electrical penetration assemblies (EPAs). NRC-sponsored program at Idaho National Engineering Laboratory (INEL) involves an investigation into the performance of isolation valves.¹

1.1 Program Objectives

The overall objective of the program is to develop methods and guidelines for reliably predicting the performance of nuclear containment buildings subjected to hypothesized severe accident, seismic, and other extreme loads. We are using data generated by experimental programs to assess analytical methods and to make improvements and modifications to the methods as necessary. We have also gained important insights into which structural details significantly affect containment performance and how to best represent those details in an analytical model. The intent of this paper is to combine information from the four Containment Integrity Programs, and other related programs being managed by SNL, and to draw inferences about the performance of containment buildings.

1.2 Report Organization

In this paper, we first give an overview of the experimental programs at SNL as they relate to reactor containments, that is:

- the static overpressurization tests of seven containment models,
- the seal and mechanical penetration tests,
- the personnel air lock test,
- equipment hatch testing,

- plans for testing bellows, and
- the EPA tests.

Then we discuss our perspective of state-of-the-art methods for predicting the performance of reactor containments. We present criteria for a number of potential failure mechanisms for containments and relate the importance of these criteria to the necessary detail of the model. We submit that the finite element method may be the only means to accurately calculate the global nonlinear response for the complex geometry present in nuclear reactor containments. We discuss the features of a finite element code that are needed to predict the nonlinear structural response, and we provide guidelines for determining which structural details should be included in finite element models. Finally, we draw conclusions about the performance of existing containments subject to severe accident conditions.

The approach outlined in this paper can be used, in our opinion, to generate state-of-the-art estimates for the capacity of a containment building without introducing any intentional conservatism. If bounding calculations are needed to know the minimum capacity with high confidence, then simpler approaches may be applicable. It should also be noted that most steel containment buildings are enclosed by a concrete shield wall. We do not address the issue of how to analyze the behavior of a steel containment building if and when it comes into contact with the shield wall.

2. CONTAINMENT TEST RESULTS

Seven containment model tests are discussed in this section. Six of the seven tests were conducted by SNL, with the remaining test conducted by Central Electricity Generating Board (CEGB) in the U.K. Included in the testing program were:

- Four 1:32-scale steel containments,
- One 1:8-scale steel containment,
- One 1:6-scale reinforced concrete containment, and
- One 1:10-scale prestressed containment that was tested in the U.K.

2.1 Steel Containments

2.1.1 Background of the Test Program

The objective of the scale steel model tests was to generate data that could be used to qualify methods for predicting the response of steel LWR containment buildings to greater than design pressure loads. The models were intended to represent features commonly found in steel [pressurized water reactor (PWR) ice condenser and boiling water reactor (BWR) Mark III] containments. Four 1:32-scale steel models and one 1:8-scale steel model were statically pressurized to failure. Details of the test procedures and experimental data are reported in Ref. 2 and 3 for the small steel models and in Ref. 4 for the large steel model.

2.1.2 1:32-Scale Model Test

As indicated by Figure 2-1, three configurations of 1:32-scale models were tested: a clean shell (two models, SC0 and SC1), a ring-stiffened shell (SC2), and a shell with three penetrations (SC3). The clean shell models were used as a baseline to measure the effect of the addition of ring stiffeners in SC2 and of the penetrations in SC3. For each of the models, the base of the cylinder was welded to a mounting ring, which was in turn bolted to a rigid test fixture. In all models, the discontinuities at the base and springline did not affect the final failure mechanism or the capacity.

We found that the pressure at which general yielding began could be calculated accurately by a simple closed form solution. The results of this simple approach agreed well with the results of the more elaborate finite element technique. Using the von Mises yield criterion along with the simple equilibrium equations for the meridional and circumferential stresses, the general yield pressure of a thin-walled cylinder is given by

$$p_{gy} = \frac{2}{\sqrt{3}} \frac{t}{r} \sigma_y \quad (2-1)$$

where σ_y is the uniaxial yield stress, t is the cylinder wall thickness, and r is the cylinder radius.

For pressure greater than the general yield pressure, bulging (radial expansion) of the cylinder became noticeable, as shown in Figure 2-2. The 1:32-scale models generally failed by a tensile instability or by exceeding the effective ductility limit of the material. The failures initiated near midheights of the cylinders.

Figure 2-1. 1:32-Scale Model Configuration

Figure 2-2. Deformed Shape of SC0 after General Yielding

The capacity of SC1, which did not rupture, was nearly 20% less than that of SC0. In SC1, leakage from a meridional through-wall crack in the cylinder near the midheight caused depressurization of the model. Away from the crack, strains near midheight of the cylinder were about 6%, which is considerably less than those attained in the other models. The crack was in the parent material adjacent to a weld repair area.

We found that the wall adjacent to the weld repair area had been severely thinned by excessive grinding of the original weld. Because of the large depth of the surface flaw, the resulting through-wall crack did not grow.*⁵ Large surface flaws like these would not be expected in a full-size steel containment.

The ring-stiffened shell, SC2, was identical to the clean shell models with the exception of 10 stiffening rings brazed to the cylinder. The effect of the stiffeners on the global post-yield behavior of the cylinder was equivalent to an increase in the cylinder wall thickness, equal to the stiffener cross-sectional area divided by the cylinder length. The added material in the stiffeners was equivalent to an 18% increase in shell thickness. The 18% increase is approximately equal to the predicted and actual increase in the pressure at general yield. The general yield pressure for SC2 determined using equation 2-1 (with the thickness increased to account for the stiffeners) was nearly identical to the value determined with finite element models. Representing the stiffeners by increasing the cylinder wall thickness is referred to as "smearing." The failure pressure of SC2 was almost 5% higher than that of SC0, the difference was not as large as the 18% difference that might be predicted using the smearing technique. Other features of the ring-stiffened model SC2 were not adequately addressed by the smearing assumption.

The penetration model, SC3, included representations of an equipment hatch and two personnel air locks. The dimensions and wall thicknesses of the cylinder and dome were the same as those used in the clean shell models. The penetrations did not affect the global response of the cylinder; the general yield pressure for SC3 was approximately the same as that for SC0 and SC1. However, the capacity of SC3 was about 10% less than that of SC0. This has been attributed to local strain concentrations in the cylinder caused by the penetrations.

2.1.3 1:8-Scale Model Test

The 1:8-scale steel model had many of the structural features found in a typical containment, as shown in Figure 2-3. The model was fabricated by Chicago Bridge and Iron Company to the ASME code with a design pressure of 40 psig (0.276 MPa).⁶

The ellipsoidal base, which was used for access and support, was not considered to be part of the model. There were 11 penetrations, including two equipment hatches with single O-ring seals, two representations of personnel air locks, and two constrained penetrations. The penetrations were constrained with rigid welded pipe across the diameter of the containment connected to the inside of each penetration. This prevented their expansion in the radial direction. Reinforcement of the cylinder at all penetrations was provided as required by the area replacement rule of the ASME code. Twelve stiffening rings were attached to the cylinder at approximately equal intervals. Over 800 channels of instrumentation were used in the model.

As in the 1:32-scale models, bending at the base of the cylinder and the response of the dome and the springline region did not affect the failure mechanism or the capacity. We did not observe a single distinct pressure at which general yielding occurred for the whole cylinder during the test. Strain concentrations were measured in the cylinder adjacent to the reinforcement around the penetrations. The most severe were those adjacent to the personnel air lock and equipment hatches at the cylinder midheight, where local strains were up to 1.7 times the free field strain. Due to the eccentric geometry of

* D. B. Clauss, "Small Steel Model Failures," Internal Memorandum (Albuquerque: Sandia National Laboratories, November 1, 1985).

Figure 2-3. 1:8-Scale Steel Containment Model

Figure 2-4. Geometry of Formed Stiffener Near Equipment Hatch of 1:8-Scale Containment

the stiffener pattern around the equipment hatch, Figure 2-4, there was a breakdown in the membrane load carrying action in the stiffeners. Of note: we found that the constrained pipe within the model had little or no effect on the capacity of the model, or on the failure mechanism and location.

2.1.4 Test Limitations

The performance of full-size steel containments may not be the same as that of the scale models. Our primary objective for the steel scale model tests was to generate data that could be used to qualify prediction methods when environments more severe than the design basis loads are encountered.

The scale models were not replicas of actual steel containments. The variation in design from one plant to the next is by itself enough to warrant separate structural analyses and independent conclusions about performance for each containment.

2.2 Reinforced Concrete Containment

We tested a 1:6-scale reinforced concrete containment model in July 1987. The model was constructed by United Engineers and Constructors to the ASME boiler and pressure vessel code, Section III, Division 1 and 2, using a design pressure of 46 psig (0.317 MPa). The design of this containment is based on a composite of many existing containments in the United States. It is the most recent and the largest model we have tested to date in the SNL Containment Integrity Program. A photograph of the model is shown in Figure 2-5.

2.2.1 Model Geometry

The model consists of a 40-in.- (1.02-m-) thick flat basemat, a 9-3/4-in.- (0.248-m-) thick circular cylinder wall with a radius of 11 ft (3.36 m) and a height of 22.25 ft (6.79 m), and a hemispherical dome that has a wall thickness of 7-3/4 in. (0.197 m) and a radius of 11 ft (3.36 m). A steel liner, 1/16 in. (1.59 mm) thick along the inside of the basemat and cylinder wall and 1/12 in. (2.12 mm) thick along the dome, is incorporated into the model to provide leak tightness.

The steel liner is attached to the concrete with headed studs. The concrete wall contains eight layers of reinforcement, including four layers in the hoop direction, two layers in the vertical direction, and two layers of diagonal (seismic) reinforcement. Additional reinforcement is provided near penetrations and at the junction of the cylinder wall and basemat. Detailed geometry and material property information are provided in the pretest and posttest analysis reports.^{7,8}

2.2.2 Reinforced Concrete Containment Testing

Prior to high pressure testing, we performed a Structural Integrity Test (SIT) and an Integrated Leak Rate Test (ILRT) on the model. The procedures and standards followed in these tests were the same as those used on actual containments. The high pressure test to failure was conducted using nitrogen gas in pressure increments ranging from 2 to 5 psi, with the smallest increments at the end of the test. Pressure was normally increased by the desired step size in 10 to 20 sec. and then held for about 20 min. before a data scan of the 1200+ transducers was initiated. This delay was sufficient for the model to essentially reach a static equilibrium condition.

Figure 2-5. The 1:6-Scale Reinforced Concrete Containment Model

2.2.3 Test Results

No significant leakage was detected from the model until the pressure was raised to 135 psig (0.931 MPa), at which point a leak rate of 11% mass/day (8 scfm) occurred. At 140 psig (0.965 MPa), leakage had increased to 13% mass/day (10 scfm) and, again, at 143 psig (0.986 MPa) to 62% mass/day

(50 scfm). The test was terminated after more than one hour at 145 psig (1.0 MPa) due to leakage greater than the capacity of the nitrogen gas supply, which was approximately 5000% mass/day (4000 scfm).

The initial leak rate measurement recorded at 145 psig (1.0 MPa) was 234% mass/day (185 scfm), but the leak rate increased steadily with time during this last pressure hold. Failure of the model occurred near the thickened insert plate of an electrical feed penetration with the formation of a large (22-in-long) tear in the liner at 145 psig (1.0 MPa) or 3.15 times its design pressure of 46 psig (0.317 MPa). We observed many other smaller liner tears and distress points upon posttest examination, as depicted in Figure 2-6. All of the liner tears appear to have been affected by the studs. However, virtually all of the model leakage was due to the single 22-in.- (0.56-m-) long tear. Figure 2-7 shows photos of the large tear and two of the typical small liner tears.

2.2.4 1:6-Scale Containment Analyses

Analyses of this model were conducted extensively both before and after the pressure test by a number of organizations in the U.S. and abroad. The pretest analyses and predictions for the capacity and failure mode of the 1:6-scale model were published in May 1987 (before the test) in NUREG/CR-4913.⁷ The posttest document on the round-robin analysis, NUREG/CR-5341,⁸ describes comparisons of the pretest predictions with the experimental results and posttest evaluations. With the posttest analyses an opportunity was afforded to introduce improvements in the analytical models and to develop insights into the failure mechanisms.

The organizations that participated in the round-robin analysis represented a diverse group of regulators, national laboratories, and industry. Those that contributed to both the pretest predictions and posttest evaluations are:

- Sandia National Laboratories (SNL) - U.S.A.
- Argonne National Laboratory (ANL) - U.S.A.
- Electric Power Research Institute (EPRI) - U.S.A.
- Commissariat a L'Energie Atomique (CEA) - France
- HM Nuclear Installations Inspectorate (NII) - U.K.
- Comitato Nazionale per la ricerca e per lo sviluppo dell'Energia Nucleare e delle Energie Alternative (ENEA) - Italy
- Atomic Energy Authority, Safety & Reliability Directorate (AEA) -U.K.
- Gesellschaft für Reaktorsicherheit (GRS) - F.R.G.

In addition to those listed above, the CEGB U.K. and Brookhaven National Laboratory (BNL)-U.S.A. also participated in the pretest analysis report. However, CEGB and BNL did not contribute to the posttest evaluations. Analyses were also conducted by the Japan Atomic Energy Research Institute and Taiwan Electric Power Company (both organizations have information exchange agreements with the NRC), but because they did not begin their involvement until much later than the others, they were not formal participants in the round-robin effort.

Figure 2-6. Cylinder Stretchout Showing Liner Tears

A number of benefits were obtained from the round-robin analyses. First, the large number of analytical approaches and interpretations led to a comprehensive instrumentation plan and improved the conduct of the test. Second, a large number of state-of-the-art finite element codes were applied to the analysis of the model (ABAQUS, ADINA, CASTEM, NEPTUNE, NFAP, PAFEC, and TEMP-STRESS) and validated for global response. Third, sources of uncertainty and error in the analyses were identified and improvements have been made in modeling. Finally, a consensus has emerged on the modeling techniques to accurately predict global response. The insights obtained from the posttest evaluations have been greater than any one group could have accomplished alone.

Figure 2-7. The Major Liner Tear and Two Typical Smaller Tears

Pretest predictions for the capacity of the model varied from 130 to 190 psig (0.896 to 1.31 MPa). Analysts were also asked to provide the maximum pressure at which they had high confidence that there was a low probability of failure. With just one exception, these numbers were less than the pressure at which the model failed (145 psig, 1.0 MPa).

2.2.5 Conclusions from the 1:6-Scale Containment Analyses

The main sources of error in the pretest analyses were the assumptions of how failure occurs and the lack of detailed local analyses, not the global response calculations. The global response of the cylinder and dome, in particular the hoop behavior of the cylinder, was predicted with reasonable accuracy by virtually all participants using two-dimensional axisymmetric finite element models. The variations in calculated global response measures were mostly associated with differences in concrete constitutive models and in the way that rebar is accounted for in the models. This is true primarily in bending regions such as in the basemat, at the wall-base, and at the springline. Specific guidelines for the axisymmetric analyses, which have evolved from this work, are described later in this paper.

More important than the choice of a finite element code or constitutive model were the assumptions adopted by the analyst. Most pretest analyses were two-dimensional based on axisymmetric models. The use of axial symmetry with the corresponding loss in model detail is probably the single most important reason that failure was not correctly interpreted by most analysts. Although most recognized the possibility of liner tearing, the prevailing subjective judgment was that local strain concentrations would not be great enough, given the high ductility of the liner (ultimate strain > 25%), to tear the liner before some other failure mode occurred. The analysts that overpredicted the capacity of the model overlooked the liner tearing mode, in part because they relied on axisymmetric models. For example, a number of analysts predicted hoop rebar failure in the range of 180 to 190 psig (1.24 to 1.31 MPa). These analyses were not so much in error as they were incomplete, due to their failure to make a detailed assessment of liner tearing. Additionally, underpredictions were the result of very conservative criteria for shear or rebar failure (such as general yielding).

Clearly, one lesson learned is that detailed local models, with boundary conditions based on the global response, are necessary to accurately predict failure. EPRI/ANATECH followed this approach in their pretest predictions. They were able to accurately predict the range for the capacity (140 to 150 psi) and the mode of failure (liner tearing). They found that their predictions consisted of a list of possible liner tearing locations rather than the precise location or mode of the largest tear. Unfortunately, EPRI's approach entailed a good deal of subjective judgment, as their calculated strains were below the ductility of the material. Since the test, however, posttest evaluations at SNL, CEA, GRS, and EPRI have led to considerable refinement of this approach, especially into the magnitude of liner strain concentrations and the design features that cause them.⁹

In the case of rebar failure, a failure criterion based on rebar yield is clearly inappropriate. With the possible exception of large diameter bars that are bent around major penetrations, the test results suggest that the ultimate strength of the splice or the rebar will be developed. In the case of shear failure, there is simply no generally recognized method for determining the shear capacity of a reinforced concrete section under simultaneous application of tensile loads and bending moments.

Design codes are intended to be conservative, but the degree of conservatism is not quantitatively known. The nominal shear stress in the concrete at the base of the cylinder wall has been estimated at 450 psi (3.10 MPa), or $5.7\sqrt{f'_c}$, at the maximum test pressure of 145 psig (1.0 MPa) (Ref. 8, Section 5.9). This is significantly higher than the allowable shear stress allowed for in the ASME boiler and pressure vessel code of around $3.5\sqrt{f'_c}$ depending on the specific design parameters. An accurate evaluation criterion for shear failure remains a difficult challenge to concrete containment analysts.

2.3 Prestressed Concrete Containment

A 1:10-scale model of the Sizewell-B prestressed concrete containment building was constructed and tested for the CEGB in the U.K. SNL participated in a cooperative effort between the NRC and the AEA of the U.K. by providing insight during the model's design and construction. Much of the insight was based on lessons learned during the 1:6-scale model test at SNL.

SNL also was asked to perform pretest predictions of the model response. These analyses would become part of the technical exchange and were carried out in the first half of 1989 by ANATECH prior to the experiment being completed in July 1989. Within the framework of SNL's Containment Integrity Program, the principal objectives of the pretest analyses were:

- to validate analytical methods used to predict the global structural response of a prestressed concrete containment, and
- to gain insight into the potential structural failure modes (not including liner tearing) of a prestressed concrete containment.

2.3.1 Model Geometry

The model geometry, illustrated in Figure 2-8 and based on the Sizewell-B containment design, consists of a 14.4-ft- (4.4-m-) diameter cylinder with a wall thickness of 5.1 in. (0.13 m), a 16.5-in.- (0.42-m-) thick basemat, a 3.9-in. (0.10-m) hemispherical dome, and a rubber bladder in place of a steel liner. The geometry, material property information, and rebar layout* are similar to the Standardized Nuclear Unit Power Plant System (SNUPPS) design used in the U.S.

The model rests on a plinth that does not extend all the way to the radius of the meridional tendon anchorages in order to facilitate tendon jacking. The cylinder wall contains a scaled version of an equipment hatch, a personnel air lock, and several smaller penetrations. The meridional tendons are a hairpin design laid out orthogonally in plan and anchored under the basemat. The hoop tendons all span 240°, and the anchorages are staggered between three wall buttresses located at 41°, 161°, and 281°.

Participation in the experiment represented a valuable opportunity to examine the ultimate pressure capacity of a prestressed concrete containment model in a manner similar to SNL's 1:6-scale reinforced concrete containment model. The methodology we used in the 1:10-scale containment analysis is similar to that employed in the analysis of the 1:6-scale containment.¹⁰ However, the Sizewell-B model introduces new elements in the analysis due to the prestressed design and the absence of a steel liner (neglected because of the test goals in addition to scaling difficulties).

In a steel-lined concrete containment, the liner accounts for a percentage of the pressure capacity of the containment. Elimination of the steel liner would logically reduce the ultimate capacity of the containment. We believe this reduction in pressure capacity may be more than offset by the elimination of premature liner tearing failure, and that the capacity of the rubber-lined containment could be higher than a steel-lined one.

* Based on a written communication from R. Crowder, Nuclear Design Associates, to D. Horschel, Sandia National Laboratories, with accompanying test data, Sept. 12, 1989.

Figure 2-8. Schematic of 1:10-Scale Prestressed Containment
(Not to Scale)

Even with the replacement of a steel liner with a rubber bladder, the results of the prestressed model test were judged to be significant for three reasons:

- Far too little is known about the nature of shear failure in combination with large bending in concrete containments and about constitutive modeling of shear behavior. Shear failure in the 1:10-scale model was deemed to be of relatively high probability, particularly due to the absence of the steel liner. It was noted that the steel liner accounts for about 40% of the moment of inertia of the cracked section at the base of the cylinder wall and, thus, is a major contributor to the shear/bending resistance in the ultimate pressure regime.
- Global analyses of prestressed containments¹¹ showed that, at high pressures approaching global failure, prestressed containments may have less residual strength than reinforced containments and thus may have a narrow pressure range between liner tearing/leakage and ultimate burst pressure.
- With increased emphasis placed on rapid overpressurization scenarios in the United States, burst mode failures from dynamic loads should be reinvestigated. This is significant; a leak-before-break failure mode might not have time to develop in dynamic pressurization cases. The only way to induce a burst mode failure with certainty in a quasi-static pressure test is with a rubber-lined model, provided that premature rupture of the rubber liner does not occur. Because the rubber liner does not contribute to the strength of the containment, emphasis is placed on the behavior of the concrete, reinforcement, and prestressing tendons.

2.3.2 Test Results

The test pressurization consisted of four hydrostatically applied pressure cycles. The first three cycles went to 1.15 times the design pressure of 50 psig (0.345 MPa), with the fourth cycle being an ultimate pressure test. The pressures referenced in this paper are gage pressures at the apex of the dome. Pressures at lower elevations in the model were subjected to higher total pressure due to hydrostatic head. During the high pressure test, a structural instability in the basemat occurred at a pressure of 112 psig (0.77 MPa) ($2.24 \times$ design) and the vessel began to tip noticeably to one side. The test was terminated at this pressure.

Upon inspection, it was deemed that failure of the model had occurred due to large basemat uplift, which caused tilting of the model, concrete spalling on the underside of the basemat, and basemat bottom rebar failure. Some ratcheting or nonelastic behavior was observed in the basemat uplift during the three low pressure testing cycles. The basemat uplift was 18% higher at the end of the third low pressure load cycle than after the first. The pretest analyses ignored the low pressure tests and began at the start of the ultimate pressure test. As such, the ratcheting behavior was not accounted for. The barrel, however, behaved elastically during the low pressure tests.

Basemat uplift was the largest displacement response component (1.6 in. (40 mm) at 112 psig (0.77 MPa) compared to a maximum radial displacement of 0.53 in. (13.5 mm) at the barrel midheight). A deformed structure plot of a pretest analysis at 110 psig (0.76 MPa) ($2.2 \times$ design) is shown in Figure 2-9. The displacements shown in this plot match the test measurements well, except in the dome where the analytical model appeared to be overly stiff. The hoop and meridional tendons appeared to have performed satisfactorily in the test. The only exception to this is that the data shows evidence of significant friction bonding and lack of slippage in areas of large shear and bending.

As we described in the pretest predictions report,¹² the primary basis for the predictions were two finite element axisymmetric global analyses. One analysis was with a frictionless-tendon representation to simulate greased frictionless tendons. The other axisymmetric analysis was with a bonded-tendon representation that simulated a no-slip condition due to friction. The bonded-tendon model matched the

Figure 2-9. Deformed Shape Using Bonded-Tendon Analyses at 125 psig
(Displacements multiplied by 10)

experimental results well, whereas the frictionless-tendon model did not show a good agreement with some of the measurements, particularly with regard to basemat/wall-base response. No meridional tendon yielding was observed in the gages during the test. However, no tendon gages were located in the critical wall-base juncture region, and it is postulated that, because of friction bonding, local tendon yielding may have occurred at that location. The hoop tendon strains show small amounts of yielding beginning to occur by the end of the test.

2.3.3 Comparisons with the Pretest Analysis

In general, the bonded-tendon analysis provided the best agreement with the measured results. Both models give similar results in the barrel and dome, but the bonded-tendon model showed much closer agreement with the measured basemat response. Similarly, detailed local equipment hatch and local buttress finite element model results also displayed important local effects due to tendon bonding.

The bonded-tendon assumption appears to be justified by the significant level of friction evidenced in the tendon gage measurements. This is so particularly after the concrete was significantly damaged. The reason for the difference between basemat response in the bonded and unbonded models stems from the early occurrence of a plastic hinge in the bonded model. The bonded-tendon analysis also predicted the elevated basemat bottom bar strain, the spalling of concrete near the basemat anchorage, and the major crack at the inside base of the wall, which were all observed in the test.

The 1:10-scale containment test results have provided another source of data for verification of analysis procedures. It has been speculated that, because of basemat scaling difficulties and construction details, the basemat bending/uplift failure mode is not representative of what would occur in a real containment. Nonetheless, the test results are still useful for further validation of analytical methods.

Of note: CEGB is considering the possibility of modifying the model's basemat and re-testing the 1:10-scale containment.

3. SEPARATE EFFECTS TEST RESULTS

Several experiments have been conducted or are planned on containment components, which complement the testing results gained from the testing of the containment models. Many of the tests included the effects of temperature, pressure, and aging of the components.

3.1 Electrical Penetration Assemblies

3.1.1 Background Information

EPAs are used to provide a leak-tight pass-through in nuclear containment buildings for electrical cables with power, control, and instrumentation applications. Typical PWR and BWR containments include anywhere from 30 to 70 EPAs per unit.^{13 14}

3.1.2 Test Program

Three full-size EPA designs -- one each by Conax, D. G. O'Brien, and Westinghouse -- were tested at SNL as a part of the Electrical Penetration Assemblies Program. These three EPAs were built to industry specifications and provide a good representation of the different materials and designs used in existing containments. The EPA test specimens were subjected to a combination of radiation and thermal aging to simulate end-of-service-life conditions. After aging, each EPA was exposed to assumed "worst-case" severe accident conditions representative of the containment type in which they are most commonly employed. Table 3-1 provides a list of the accident conditions applied during testing.

Table 3-1. Summary of Electrical Penetration Assembly Test Conditions

EPA Design	Containment Type	Maximum Temperature & Pressure
D.G. O'Brien	PWR	361°F, 155 psia
Westinghouse	BWR Mark III	400°F, 75 psia
Conax	BWR Mark I	700°F, 135 psia

3.1.3 Test Results

The conditions listed in Table 3-1 were applied for a period of 10 days. No significant leakage was measured during any of the tests. The temperature distribution and electrical continuity of each EPA was also monitored during the tests. Because there are a large number of different EPA designs, the exceptional leak integrity of the three EPAs in this test program should not be assumed to necessarily apply to all designs. EPAs manufactured before 1971 were not regulated by any national codes or standards and were often fabricated in the field, whereas EPAs tested in this program were subject to rigorous quality assurance and were designed to meet the standards of IEEE 317-1976 and IEEE 323-1974.

If information concerning the containment pressure and temperature conditions is known, a heat transfer analysis can be used to determine the approximate temperature profiles in the EPA. Knowing the

temperature distribution and exercising engineering judgment, a reasonable evaluation of the performance of other EPA designs can be made. The above tests have provided a good basis for such an evaluation.

3.2 Personnel Air Lock

3.2.1 Background Information

The complex structural interaction between the door, gasket, and bulkhead in personnel air locks makes analytical evaluation of leakage difficult. In order to provide data to validate methods for evaluating the leakage potential, a full-size personnel air lock was subjected to simulated severe accident loads of up to 300 psig (2.07 MPa) and 800°F (427°C). The test was conducted at the CBI Industries research facility in Plainfield, Illinois under contract to SNL.

3.2.2 Test Program

The air lock tested in this program was designed and partially fabricated for the canceled Callaway Unit 2 nuclear power plant. The construction of the air lock was completed for this test program. The air lock and related test hardware is shown schematically in Figure 3-1. A double dog-ear gasket made of Presray ethylene propylene (EPDM) E603 was used to maintain a seal between the door and bulkhead. Details of the sealing surfaces are shown in Figure 3-2.

Before beginning the tests, we subjected the gaskets to an accelerated thermal aging process intended to simulate in-service radiation and thermal aging over a 40-year service life. The accelerated aging conditions involved heating of the gaskets in place to a temperature of approximately 370°F (188°C) for 172 hours. The aging process resulted in a significant change in the gasket cross section, to the point that the original double dog-ear cross section was not recognizable. At several locations the gasket material had flowed into the space between the door and bulkhead. The displaced gasket material prevented full compression of the gasket into the 1.344 × 0.688 in. gland (which the gasket is designed to do) and precluded metal-to-metal contact between the door and bulkhead under pressurization.

We instrumented the test specimen with over 300 sensors. The majority of the instrumentation was strain gages, thermocouples, and displacement transducers. Chamber V-1, shown in Figure 3-1, was constructed to form a pressure chamber with the inner door. Access manholes, nozzles for instrumentation feed-throughs, and pipe fittings were also installed. Additional information on the instrumentation and data acquisition, along with the heating and pressurization systems, can be found in the report by Julien and Peters.¹⁵

We made the following simplifications for the purposes of this test:

- Based on analytical investigations, we concluded that structural deformations in the containment wall had a negligible effect on the deformation of the sealing surfaces and thus no effect on the leakage potential. We made no attempt to model the effects of the containment wall in this test.
- We used heated air rather than steam to simplify leakage measurements. Also, for safety reasons, we oriented the air lock vertically instead of horizontally. These differences primarily affect the heat transfer mechanisms within the test. On the inner door, we used forced convection to approximate the heat transfer properties of a steam environment given a horizontal orientation.

Figure 3-1. Schematic of Air Lock and Test Chambers in Test Cell

Figure 3-2. Seal Gland and Gasket Cross-Section for Inner Door
(Dimensions in inches, not to scale)

Organic gasket materials exhibit a threshold temperature at which their performance changes dramatically. Above the threshold temperature, changes occur in the composition and integrity of the material. The threshold temperature for the Presray EPDM material used to fabricate the double dog-ear seals is approximately 650°F (343°C).¹⁶ In many severe accident sequences, the maximum temperature is less than this threshold temperature, but there are some accident scenarios that may result in higher temperatures. Time is also recognized as an important parameter, but its effect is not quantified.

We utilized three loading cycles that take into account the leakage potential and the threshold temperature of the gasket material:

- The first load cycle consisted of heating the air above the inner door to 400°F (204°C). In the test configuration, "above the inner door" is equivalent to inside the containment building. The air temperature was allowed to stabilize and the pressure was increased to 300 psig (2.07 MPa) in 10-psi (69-kPa) increments. Pressure was decreased in 25-psi (170-kPa) increments.
- The second load cycle consisted of increasing the air temperature above the inner door from 400° to 800°F (204° to 427°C). The air temperature was allowed to stabilize, and the pressure was increased to 300 psig (2.07 MPa) in 10-psi (69-kPa) increments. Pressure was decreased in 25-psi (170-kPa) increments. As pressure was increased, the air temperature above the door dropped below 600°F (316°C).
- The third load cycle was implemented since the gasket survived the first two load cycles. The air temperature above the door was increased to 850°F (454°C) and allowed to thermally soak for approximately 10 hours. Pressure was increased in 25-psi (170-kPa) increments until the gasket seal failed. The larger pressure increments were used to minimize heat loss during pressurization. The air lock was then pressurized to 300 psig (2.07 MPa).

3.2.3 Test Results and Analytical Comparisons

During the first two load cycles of the severe accident test, we detected no leakage past the inner door. As expected, the relative deformation of the sealing surfaces was minimal and the gasket material had enough resiliency to maintain a seal. The gasket temperature in the first cycle did not exceed the threshold temperature; brief exposures to above-threshold temperatures as in the second cycle may also be tolerated.

During the final cycle, a large portion of the gasket on the inner door was ejected from its groove at 150 psig (1.03 MPa). From this point on, the inner door was effectively bypassed. The load cycle continued to a pressure of 300 psig (2.07 MPa). Due to the inner door seal failure, the air space between the doors was also pressurized to 300 psig (2.07 MPa). However, the integrity of the air lock was maintained; the outer door did not fail because it did not experience the extreme temperature loads found on the inside of the containment.¹⁷

The condition of the inner door gasket after the test is shown in Figure 3-3 and Figure 3-4. Along the bottom of the door, the gasket material was missing. In other areas, the gasket was charred and badly cracked. An oily, sooty coating was found on the fixtures and piping that was in the path of leakage from the inner door. We concluded that the Presray EPDM gasket material suffered some form of combustion damage during the test.

In the test, we found the structural integrity of this air lock to be exceptional. The maximum strains at 300 psig (2.07 MPa) were only about 0.3% with no significant risk of structural failure. As indicated by Figure 3-5, our pretest three-dimensional finite element predictions were reasonably accurate for the response of the bulkheads. The response was essentially linear and the effect of elevated temperature on the bulkhead deformation was small. However, comparisons for the sealing surface deformations are

Figure 3-3. Seal Failure Location at Bottom of Door

Figure 3-4. Charred Remains of Gasket near Bottom Door Corner

Figure 3-5. Out-of-Plane Bulkhead Displacement Transducer TOB2-080

not very meaningful because we neglected the changes in the gasket properties that resulted from the thermal aging in the pretest predictions. Also, thermal expansion of the gasket material, which we neglected in the analysis, apparently had a significant effect on the deformations of the inner door. We need additional study of the door and sealing surface deformations to validate the analysis methods. Future evaluations of leakage from penetrations should take the thermal expansion of the gasket material into account.

Temperature histories for the severe accident test, at several locations along the air lock, are plotted in Figure 3-6. We can make two significant observations with respect to the temperature profiles:

- The temperature of the gaskets and the inner door were considerably less than the air temperature inside pressure chamber V-1.
- Due to the limited heat transfer ability of the air lock, the outer door temperature remained much cooler than the inner door. Even when the inner door seal failed, there was not enough mixing between the air in the air lock and that inside the containment to significantly raise the temperature of the outer door. It is true that the temperature of the outer door was affected in a nonconservative way (too low) by the orientation of the air lock in this test. However, it is believed that even for horizontally oriented air locks in actual containments, there will be enough reduction in temperature between the inside of the containment and the outer door to prevent degradation of the EPDM seal material in the outer door (assuming a maximum containment temperature of 800°F (427°C)).

Figure 3-6. Temperature Histories at Selected Locations

3.2.4 Conclusions

The deformations of the sealing surface of the personnel air lock tested in this program were markedly small even for extreme values of pressure and temperature. As a result, the gasket material was able to prevent leakage except when the temperature was beyond the threshold stability limit of the gasket material. Although leakage past the inner door may occur under these conditions, the redundancy and reduced temperature loading of the outer door precluded leakage to the outside environment. Additionally, we need to study and validate analytical methods for evaluating the leakage potential of personnel air locks and to determine if these conclusions extend to other personnel air lock designs.

3.3 Equipment Hatch Seal Testing

3.3.1 Background Information

Equipment hatch seal tests are an ongoing program being conducted at SNL. Twelve tests are planned, five having been completed at the present time. Correspondingly, information presented in this section needs to be considered preliminary.

3.3.2 Testing Program

We are in the process of conducting equipment hatch seal tests on the pressure-unseating equipment hatch used in the 1:6-scale reinforced concrete containment model.* We refer to unseating as the decrease in seal compression as the pressure on the hatch is increased. A schematic of the containment building, including equipment hatch B, is shown in Figure 3-7; the hatch details are shown in Figure 3-8. The outer door of this hatch is the pressure unseating hatch. Since the inner cover was not designed

* L. D. Lambert, *Test Plan for Evaluating Leakage Behavior of Equipment Hatch Seals*, Revision 8, Internal Document (Albuquerque: Sandia National Laboratories, 1989).

Figure 3-7. Schematic of the 1:6-Scale Containment Showing Equipment Hatch Chamber

Figure 3-8. Hatch Chamber Detail

to be loaded with pressure between the two hatches, it has been welded closed for these tests and will not factor into the test program. These tests will provide data that will be useful in the prediction of leakage from similar equipment hatch designs.

The leakage potential for many penetrations depends strongly on the performance of its seal. Table 3-2 displays the test matrix that we have formulated for evaluating seal performance. The test matrix includes variations in the gasket material, gasket aging, bolt stiffness, bolt preload, and temperature of the hatch chamber.

During a test, pressure is incremented in steps while strains, displacements, temperatures, and leakage are monitored and recorded. We have conducted five seal tests at ambient temperature and plan to do seven more that will include elevated temperatures. During the tests that involve high temperatures, the in-line heaters will heat the incoming nitrogen gas. Radiant heaters mounted inside the equipment hatch will heat the gasket and surrounding area to accident temperature conditions.*

3.3.3 Test Results

Comparisons between the measured and calculated behavior for the completed tests provide confirmatory evidence that the gross structural response can be used to predict the initiation of significant leakage. This is possible when the available springback is used as the gasket performance parameter. The structural response can be calculated as shown by Clauss.** The gap, g , separation displacement, s , and separation pressure, p_s , are given as:

$$g = \begin{cases} 0 & \text{for } s < S_p \\ s - S_p & \text{for } s > S_p \end{cases} \quad (3-1)$$

$$s = \begin{cases} 0 & \text{for } p < p_s \\ \pi r^2 (p - p_s) \left[\frac{1}{k_B} + \frac{2\nu}{k_F} \right] & \text{for } p > p_s \end{cases} \quad (3-2)$$

$$p_s = \frac{\left[\frac{k_B + k_F}{k_F} \right] F_i + k_B (\epsilon_{TF} - \epsilon_{TB}) L}{\left[1 + \frac{2\nu k_B}{k_F} \right] \pi r^2} \quad (3-3)$$

S_p is the available gasket springback, r is the inside radius of the equipment hatch sleeve, p is the test pressure, k_B is the elastic axial bolt stiffness (which should be determined from tensile tests), k_F is the elastic axial flange stiffness, ν is Poisson's ratio, F_i is the mechanical bolt preload, ϵ_{TF} and ϵ_{TB} are the average thermal strains in the flange and bolt, respectively, and L is bolt grip length. The available gasket springback can be approximated given dimensional measurements of the gasket and sealing surface features. Other factors such as thermal expansion, pressure, and temperature will also affect gasket performance.

* L. D. Lambert, *Safe Operating Procedure for Testing a 1:6-Scale Containment Equipment Hatch at the Containment Test Site*, SOP 60008 9002, Internal Document, (Albuquerque: Sandia National Laboratories, 1990).

** D. B. Clauss, "Comparison of Experiment and Analysis for the Behavior of Penetrations in the 1:6-Scale Reinforced Concrete Containment Model," Internal Memorandum (Albuquerque: Sandia National Laboratories, May 28, 1987).

Table 3-2. Proposed Test Matrix for Investigating the Leakage Potential of a Pressure-Unseating Equipment Hatch

Test Designator	Gasket Material ¹	Aging Duration ² (hours)	Bolt Preload (kips)	Number of bolts	Test Load ³
LP3 ⁴	SI	118	45.7	10	A
HT1	SI	144	45	10	A
HT2	EP	Unaged	55	10	A
HT3	EP	Unaged	72	20	A
HT4	EP	168	72	20	A
HT5	EP	Unaged	72	20	B
HT6	EP	Unaged	57	20	B
HT7	EP	168	57	20	B
HT8	SI	168	57	20	B
HT9	EP	Unaged	57	20	C
HT10	EP	168	57	20	C
HT11	SI	168	57	20	C

1. SI = to silicone; EP = ethylene propylene.
2. Gaskets will be aged in place at 300°F (149°C) for the indicated time to simulate both radiation and thermal aging. Orwick¹⁸ has shown that compression set retention of EP and SI gaskets is most sensitive to radiation aging. Exposure to as little as 50 Mrads results in compression set retention of 75% for EP and 90% for SI. Typically, a radiation dose of 200 Mrads has been used in other experiments and, although it may represent an overtest, the compression set retention is about 95% for EP and about 97% for SI at this level of exposure. Since only thermal aging is practical for the equipment hatch tests, the aging time (and possibly temperature also) should be adjusted to achieve compression set retention of the gaskets between 80% and 95%.
3. A = Stepwise pressurization at ambient temperature.
 B = Stepwise pressurization and heating; temperature held equal to the steam saturation temperature at the current pressure.
 C = Hold gas temperature at level sufficient to maintain the gasket at or above its threshold instability temperature as defined in Ref. 16 for at least two hours; maintain temperature and initiate a stepwise pressurization.
4. This test was performed July 10, 1987, on the SNL 1:6-scale reinforced concrete containment model. The gasket was not aged in place.

3.3.4 Difficulties in Measuring Leakage

We have experienced some difficulty in leakage measurements due to movement of the gasket as the separation gap widens during the course of a test. As the gap increases, the measured leakage may actually decrease due to the gasket partially moving out of the groove and wedging itself into the gap between the equipment hatch cover and sleeve. In practice, the calculated results are subject to large uncertainty because important parameters such as the bolt stiffness, bolt preload, and available springback are not as well characterized as they are in these tests. Good engineering judgment should be used in actual applications by assigning appropriate values to these parameters.

3.4 Bellows

We have identified bellows as an area of the containment system boundary that is not well understood from an ultimate behavior standpoint. We have designed a series of tests that will be performed in the near future. The following subsections describe the function of bellows and the motivation for bellows testing.

3.4.1 Background Information

Piping penetrations of containment buildings employ bellows to minimize any possible structural loading of both the piping systems and the containment shell. Penetration bellows are primarily used in steel containments in which they are an integral part of the containment boundary. Should a leak develop in these bellows the containment would be breached. The majority of reinforced and prestressed concrete containments do not use bellows connections between the process pipes and containment shell. Instead, a "rigid" connection is used.

Process piping bellows vary in size from 6 in. (15.24 cm) up to approximately 60 in. (1.5 m) in diameter. They are normally constructed of two plies of SA240, Type 304, stainless steel that are separated by a thin wire mesh. The primary purpose of the outer ply is to provide a means to check for leakage of the bellows. This is done by pressurizing the space between plies and noting any drop in pressure. Much of the information on the use of bellows in containment penetrations was obtained from Refs. 19 and 20. Figure 3-9 illustrates a typical application of bellows at a process piping penetration.

Bellows are also used at the penetration of each vent line into the suppression chamber in BWR Mark I containments. Vent line bellows range in diameter from approximately 65 in. (1.65 m) to as much as 125 in. (3.2 m). About 90% of the vent line bellows are one-ply; the remaining are two-ply. The vent line bellows are also constructed of SA240, Type 304, stainless steel. Details of vent line bellows penetrations are similar to those shown in Figure 3-9 for process piping penetrations.

3.4.2 Bellows Design

Containment penetration bellows are designed to absorb differential movement between the containment shell and the pipe to which they are attached. The design movements result from the following conditions: normal operation, safe shutdown earthquake (SSE), and loss of coolant accident (LOCA). In most cases, the bellows design movements are obtained by summing the maximum deformations associated with each of these conditions as if they occurred simultaneously. The provisions included in the Standards of the Expansion Joint Manufacturers Association (EJMA)²¹ are used for bellows design. Normally, containment bellows are designed to resist about 5,000 cycles of the above "worst case" loading. Also, each is sized to resist the full containment design pressure. Using the EJMA approach, a minimum factor of safety of four against burst due to internal pressure only must be obtained. Thus, especially for the two-ply bellows, a considerable margin should exist between normal containment design conditions and those conditions that would cause a failure of the bellows pressure boundary.

3.4.3 Performance of Bellows During a Severe Accident

In the event of a severe accident, pressure and temperature conditions within containment may reach levels that are well beyond the design basis. In most cases, we note that radial growth of the containment wall due to internal pressure imposes axial compression on the bellows. There are a few cases in which bellows are placed inside the containment and such that radial growth elongates the bellows. For cylindrical containments, the bellows must also absorb lateral deformation caused by the vertical growth of the containment shell due to increasing temperature and the "uplift" caused by the pressure acting on the dome. If the containment pressure is sufficient to cause general yielding of the

Figure 3-9. Typical Application of Process Piping Bellows

containment in the hoop direction, the radial growth increases rapidly for small additional increases in pressure. Figure 3-10 illustrates, in generic terms, the increase in axial compression and lateral deformation that will be imposed on a bellows for increasing containment pressure.

As shown, if the containment pressure is high enough, it is possible that the bellows could become fully compressed (i.e., each convolution is fully closed so that there is no remaining flexibility of the bellows). Upon reaching full compression, it is extremely likely that a tear in the bellows will occur resulting in a potentially large leak path through the containment pressure boundary.

An extensive review of bellows-related literature has been conducted and several bellows manufacturers and designers have been interviewed. However, there are no known past research efforts in which bellows have been subjected to the extreme loading conditions that might occur during a severe accident.

3.4.4 Future Work

At this time, we see ourselves faced with unanswered questions regarding the performance of containment bellows during a severe accident. Two of the foremost questions arising are:

- Are containment bellows capable of being fully compressed when subjected to simultaneous lateral deformation, internal pressure, and elevated temperature without developing a tear in the bellows material or in the connection of the bellows to the end spool?
- Can containment bellows withstand the postulated combinations of elongation, lateral offset, external pressure, and elevated temperature without a failure of the bellows pressure boundary?

If containment bellows cannot remain leaktight under these conditions, then methods must be developed to predict the possible deformation and pressure conditions that would likely cause a failure of the bellows pressure boundary.

Figure 3-10. Bellows Deformations Caused by Increasing Containment Pressure

Currently, we are developing plans to conduct a series of tests in which representative bellows are subjected to postulated severe accident loadings. During the tests, various combinations of internal and external pressure, axial compression, elongation, and lateral deformation will be applied to typical bellows geometries. The magnitudes of deformations to be applied will be determined from global shell analyses of typical containments at the position of the most critical bellows. The goal of these tests is to develop methods to predict the pressure and deformation conditions that will likely cause a tear in the bellows, which could produce a large leak path through the containment boundary.

3.5 Seals and Gaskets

3.5.1 Background Information

Compression seals and gaskets form an important part of the containment pressure boundary in many operable penetrations, such as equipment hatches, personnel air locks, and drywell heads. If metal-to-metal contact does not exist between the sealing surfaces of these penetrations, the gaskets represent the only barrier to prevent leakage through the containment boundary at these locations.

3.5.2 Testing Programs

Severe accident testing of typical gasket cross sections and materials has been conducted at SNL¹⁶ and at INEL (under contract to SNL).²² The seals tested were made from EPDM, silicone, or neoprene. Both new and aged (thermal only or thermal plus radiation aging environments) gaskets were included in the test programs. Steam, heated dry air, or heated dry nitrogen were used as the test environments. During these tests, the gaskets were subjected to constant internal pressure (either 143 or 160 psig (0.98 or 1.1 MPa)) while the seal temperature was slowly increased up to as much as 700°F (371°C). A general summary of the test results obtained both at SNL and INEL is provided in Table 3-3.

Table 3-3. Summary of Compression Seal Test Results

Material	No. of Tests	Test Environment	Failure Temperatures (°F)	Mean Failure Temperatures (°F)
EPDM	5	Steam	626-670	647
	8	Nitrogen	577-667	613
	1	Air	651	651
Silicone	8	Steam	486-592	512
	2	Nitrogen	>700	>700
	1	Air	681	681
Neoprene	3	Nitrogen	460-500	487

3.5.3 Test Results

For each test, seal failure was recorded as the temperature at which significant leakage past the seals began. After testing, many of the gaskets were charred and were of a powdery consistency, suggesting combustion of the gasket material occurred during the tests. The test results indicate that the failure temperature is independent of applied aging or gap between sealing surfaces, or angle of the sealing surfaces relative one to another. However, aging tends to reduce the available gasket springback, which may affect the leakage behavior of some containment penetrations. As shown, silicone appears to have a lower failure temperature in a steam environment than in air or nitrogen, whereas Presray EPDM is not strongly affected by the applied test environment.

We need to mention that, for all test results in Table 3-3, a fixed gap was imposed between the metal sealing surfaces. A few tests were also conducted in which there was metal-to-metal (no gap) contact of carefully finished sealing surfaces. No leakage was observed for these tests up to the maximum test temperature of 700°F (371°C). It should be noted that any out-of-flatness of the sealing surfaces in an actual penetration could prevent metal-to-metal contact and, thus, should be considered when evaluating leakage. More detailed information regarding the test conduct and results is available in the final test reports.^{16,22}

3.6 Inflatable Seals

3.6.1 Background Information

Inflatable seals are used to prevent leakage around the perimeter of personnel and escape lock doors in approximately 10% of U.S. LWR containments. All of the installations are in either PWR or BWR Mark III type containments. They are fastened to the outer edge of the air lock doors and, when pressurized with air, seal the gap between the door and the bulkhead. When deflated, there is a 3/8-in. (0.95-cm) gap between the sealing surface of the seals and the bulkhead. A typical application is shown in Figure 3-11. The air lock doors are rectangular with rounded corners and vary in size from 8 ft x 5 ft (2.43 x 1.52 m) to 6 ft 6 in. x 3 ft 6 in. (2 m x 1.1 m). Typically the corner radius is about 12 in. (30.5 cm).

Figure 3-11. Typical Application of Inflatable Seals in Personnel Air Lock Door
(Seals shown fully inflated)

According to instructions from the supplier, the seal pressure must be at least 30 psi (207 kPa) greater than the containment design pressure in order to ensure that leakage does not exceed design allowables. A survey of the plants currently using inflatable seals revealed that the normal operating seal pressure varies from plant to plant, with a minimum seal pressure of 50 psig (0.35 MPa) and a maximum of 110 psig (0.76 MPa). In all cases, the seal pressure is at least 30 psi (207 kPa) greater than the containment design pressure.

3.6.2 Test Program

A series of tests were conducted to determine the leakage behavior of typical inflatable seal designs when subjected to containment pressure and temperature conditions that exceed the design basis. These included tests performed at both room temperature and elevated temperatures representative of postulated severe accident conditions. The maximum test temperature was 400°F (204°C). Both aged (radiation and thermal) and unaged seals were included in the test program. A total of four pairs of seals were tested.

At the beginning of each test, we varied the internal seal pressure to cover the range of seal pressures actually used in containments. For each seal pressure level, the external (containment) pressure was increased until significant leakage (defined as 10,000 std. ft³ per day) past the seals was observed. Parameters, which we closely monitored and recorded during the tests were the internal seal pressure, external pressure, and leakage past the seals, along with temperatures of the test chamber and the fixture to which the seals were attached.

3.6.3 Test Results

The test results indicate that, regardless of the tested seal design, initial seal pressure, applied aging, or test temperature, significant leakage did not occur until the applied external (containment) pressure exceeded the initial seal pressure. Initial seal pressure is defined as the seal pressure at the beginning of each test at room temperature with no applied external pressure. It is analogous to the normal operating seal pressure in an actual containment. A test validated procedure has been developed²³ to estimate the containment pressure and temperature at which significant leakage past inflatable seals will occur.

4. RECOMMENDATIONS TO THE ANALYST

Our approach in this section is to examine three basic tasks that must be completed in order to predict containment performance. These tasks require:

- developing a model that represents the important as-built structural details, material properties, and geometry,
- calculating the response, and
- correctly interpreting the result.

By necessity, these tasks must be well integrated. For instance, the failure criteria must be consistent with the geometric details in the model(s) and the physical phenomena (i.e., plasticity, large deformations, etc.) accounted for in the calculations. We propose failure criteria and describe calculation methods and the geometric details that should be considered to be consistent with the proposed criteria. But first, we examine how predictions for containment performance will be used, because this strongly influences our choice of methods.

4.1 What Do We Need to Predict?

Our concern at this point focuses on how the results will be used. We feel that in any structural analysis the tools and the methods we choose are an important feature of the work. Predictions for the performance of a containment subject to severe accident loads affect risk estimates. This, in turn, could help with emergency preparedness planning for a severe accident. In the event of a severe accident, the consequences of an early containment failure appear to be considerably greater than those of a late failure. Further, the consequences of rupture are worse than those of leakage.

For our analyses to be useful in estimating risk, we need to develop the capability to:

- determine the margin of safety between the design pressure and the pressure at which a loss of containment integrity occurs, and
- to distinguish between a failure due to leakage and one due to rupture as defined by NUREG-1150.

We believe that there will not be significant leakage in steel containments until large deformations associated with post-yield behavior arise. Therefore, fulfillment of our capability to distinguish failure due to leakage or rupture implies a need for nonlinear analysis and failure criteria that account for the material's ductile behavior

We need to assume a different perspective than that commonly taken, that is, we do not wish to know the minimum threshold pressures for rupture or for leakage with high confidence. Rather, we need to try to make a best estimate. As a matter of course, this approach inherently carries some uncertainty.

In that we are trying to make a best estimate, and with regards to our supposition that leakage is not likely prior to the onset of general yielding, we can eliminate some methods as candidates for use in predicting containment performance. For instance, a failure criterion often used in the past is based on rupture occurring at general yield or 1% strain. Very simple axisymmetric models, either finite element or closed form formulations, could be used in conjunction with such a failure criteria.

This method tends to underestimate the rupture pressure. Yet, it is acceptable if it can be demonstrated that significant leakage occurs (past penetration seals, bellows, or isolation valves, for example) at a lower pressure. The same holds true if for some other reason only bounding calculations are necessary. However, if leakage does not occur before general yielding, a method based on a simple model and failure criteria like that described above is not adequate. This is due to it not allowing for the possibility of leakage (by virtue of an overly conservative rupture criterion). Therefore, it could change estimates of risk due to a severe accident.

The methods we propose are discussed in the following sections. They are based on the premise that significant leakage will not arise before the onset of general yielding. However, it is possible that leakage could arise before the onset of general yielding for certain types of penetrations (such as bellows or personnel air locks with inflatable seals) and valves.

The failure criteria, structural response calculations, and modeling have all benefited from comparisons with experimental data. Comparisons of pretest predictions with experimental results for the scale models, as well as posttest analyses, are discussed in numerous articles and reports.^{2, 3, 24, 25, 26} The greatest uncertainty has been in our understanding of how forming and fabrication procedures affect the material properties at test time and of which structural details are important. Posttest analyses have helped explain the discrepancies that existed and led to improvements in modeling techniques.

4.2 Steel Containment Modeling

4.2.1 The Analyst's Role (How to Make a Model)

In creating the model, we find there are two basic issues: first, what geometric details of containment need to be represented (i.e., the options of a solitary shell, major penetrations, and welds), and second, what the proper finite element mesh discretization should be. Again, the models need to be consistent with the failure criteria and the analytical methods that are adopted.

In terms of time, cost, and accuracy, we feel it is most efficient to evaluate criteria based on separate structural calculations for each pressure release mechanism (i.e., shell rupture, liner tearing, seal failure, etc.). With this approach, each model can be simpler and focus only on the details important to the pressure release mechanism of interest.

In order to evaluate shell rupture, at least two structural models, one including an equipment hatch or a personnel air lock and one modeling the general shell behavior is necessary. We have learned that it is very important to include any details that may be related to the result of a local breakdown in the membrane action of the shell or stiffeners. In the 1:8-scale steel model, the strength of the stiffeners was reduced by an eccentricity in the stiffener pattern around the equipment hatches, which caused primary bending in the stiffener.²⁴ As a result, when the stiffener failed, the load was transferred into the shell, and the shell failed at a lower pressure than expected. We have supported an investigation that identifies eccentricities and other small details that affect the capacity and the failure mechanism of in-service steel containments.

Ensuring that the model has been properly discretized depends largely on the analyst's ability to anticipate the type of behavior (i.e., membrane action, bending, shear, etc.) that will occur in any given location. In this respect, a background in plate and shell behavior is invaluable, as well as an understanding of the differences in element formulations. For example, bending occurs at discontinuities such as intersections between penetrations and pipes and the containment shell. It occurs where there are changes in the shell geometry and thickness.

The elastic bending response of a shell typically has the form of an exponentially decaying sinusoid. The maximum size of the elements in areas characterized by bending can be estimated if the characteristic wavelength of the elastic response, which depends on the radius and thickness of a shell, and the

element shape functions are known. For instance, to represent the bending response of a shell with a characteristic wavelength, at least four or five linear elements are needed per wavelength.

Obviously, the analyst must exercise considerable engineering judgment in constructing the model and discretizing a finite element mesh. The efficacy of the mesh and element choices must be assessed by verifying that the solutions do not violate fundamental mechanics (for example, force equilibrium), and perhaps even with sensitivity studies with element size and order as the parameters.

Predictions for leakage from pressure seating equipment hatches can be based in part on the same three-dimensional model of the shell and equipment hatch used in the analysis for rupture. In order to accurately predict ovalization of the sleeve, the stiffener pattern around an equipment hatch must again be modeled explicitly.²⁷ We note, clearly, the "smearing" technique described earlier in Section 2 has only limited application, as it cannot be used to obtain accurate predictions of leakage from ovalization of pressure seating equipment hatches.

We have determined that the deformations of the equipment hatch sleeve can be thought of as being independent of the cover deformations. This means that the shearing forces transmitted between sealing surfaces are negligible. A separate axisymmetric analysis of the equipment hatch cover can be conducted, and the relative change in the diameter of the sleeve can then be computed for use in the leakage criterion.

We have used closed form solutions to determine the response of pressure unseating hatches, based on equations for preload in bolts.²⁸ The equations must be corrected to account for differential thermal expansion between the bolts and the compression members. We see this as essentially a linear, force equilibrium problem. It is possible, however, in the presence of very large temperature differentials between the bolts and the compression members, for the bolts to yield. Kulak²⁹ used a finite element approach to calculate the flange separation of a BWR drywell head. The results were very similar to those we obtained with the closed form solution. Some data on pressure unseating hatches was obtained from the test of a 1:6-scale reinforced concrete model (see Section 2.2.4).

We believe that the deformation of the sealing surfaces of a personnel air lock can be calculated with a model that includes only the bulkhead and door. The inner bulkhead and door resist internal pressurization of the containment in the same way as flat plates, which is by a bending action. The bulkhead can be treated as simply supported or be treated as fixed at its intersection with the sleeve. Considering that separation of the sealing surfaces is caused by a mismatch in the bending displacements of the door and bulkhead, the contact problem must be investigated.

4.3 Performance of Steel Containments Under Severe Accident Conditions

Estimates of risk from a severe accident must be based on predictions of structural performance specifically for the containment of interest. In Section 4.3.1, we emphasize this approach by looking at two factors that could have significantly changed the outcome of the 1:8-scale model test. In Section 4.3.2, we speculate on the probable behavior of steel containments under accident conditions. Our speculations are intended to provide a focus for future research and other investigations, and are obviously not plant specific.

4.3.1 Outcomes May Differ

We have found that the deformation of the equipment hatch sleeve can be related to the average membrane strain in the cylinder. The ovalization of the equipment hatch sleeve is primarily inextensional (no stretching of the sleeve perimeter). As a result, the equipment hatch sleeve basically conforms to the membrane displacements of the cylinder. The increase in the horizontal radius of the equipment hatch can be approximated as

$$\Delta r_H = \epsilon r \quad (4-1)$$

where ϵ is the average membrane circumferential strain at the elevation of the equipment hatch centerline and r is the original hatch radius.

At the sealing surface, the decrease in the vertical radius of the sleeve is approximately equal to the increase in its horizontal radius. The deformation is greatest along these two radii. Assuming the cover is essentially rigid, the leakage criterion for pressure-seating equipment hatches is expressed mathematically as:

$$\Delta r_H > t \quad (4-2)$$

where t is the thickness of the sleeve at the sealing surface. Note that leakage may occur prior to this condition if the seals fail.

Combining equations 4-1 and 4-2, the leakage criterion for pressure seating equipment hatches can be expressed as:

$$\epsilon r / t > 1. \quad (4-3)$$

A critical strain, ϵ' , which is defined as the average membrane strain corresponding to the onset of leakage, can be determined with equation 4-3 for a pressure seating equipment hatch given its geometry. The leakage potential of equipment hatches from eight steel containments, identified in a survey conducted by ANL,^{19, 20} were evaluated in terms of this critical strain. As shown in Table 4-1, the critical strain ranged from 2.5% to 7.3%. This compares to the 7.3% critical strain calculated for the 1:8-scale model.

Table 4-1. Critical Strain for Leakage of Pressure Seating Hatches in Eight In-Service Steel Containments and the 1:8-Scale Containment

Unit	t	r	ϵ'	Seal Type
1	5-1/4"	72"	7.3%	gumdrop
2	4-1/2"	72"	6.3%	gumdrop
3	4-1/2"	84"	5.4%	gumdrop
4	4-1/2"	84"	5.4%	gumdrop
5	4-1/2"	126"	3.6%	gasket
6	4-1/2"	126"	3.6%	gasket
7	4-1/2"	126"	3.6%	gasket
10	3"	120"	2.5%	gumdrop
scale model	1-1/8"	15-1/2"	7.3%	O-ring

If the 1:8-scale model had been subject to elevated temperature, we believe the outcome of the test may have been different. The yield strength for A516 Gr 60 steel decreases with temperature up to 600°F (316°C), where the yield is 30% lower than at room temperature. At the same temperature, the ultimate strength has increased by nearly 20% over its room temperature value. The elongation is constant or increases slightly in this temperature range. This suggests that, although the strength of the model may be higher in the presence of elevated temperatures, general yielding of the cylinder will occur at lower pressures. Since large deformations are associated with the onset of general yielding, the potential for leakage is increased by elevated temperature.

4.3.2 A Look into the Future (Assessing Possibilities)

There is a substantial margin between the design loads for steel containments and the threshold environment for rupture. We expect that steel containments can withstand internal pressures several times greater than their design pressures at temperatures up to 600°F (316°C). This is a direct result of design procedures; ASME code incorporates a factor of safety of 3.64 (the ratio of allowable stress to ultimate stress), and the design is based on minimum allowable material properties (actual tensile strength may be 15-30% higher).

The actual wall thickness may be greater than that required by design, because designers round up to obtain standard thicknesses or to satisfy design requirements other than internal pressure. In addition, design requirements for internal pressure do not allow for the increase in strength due to the ring stiffeners. However, the margin is reduced by combined loads and by penetration and stiffener failures that introduce strain concentrations into the shell.

The average membrane strains in the cylinder at the threshold environment for rupture have important ramifications with regard to leakage potential of pressure seating hatches. As we discussed in Section 4.3.1, leakage depends on the average membrane strain at the hatch elevation, and the diameter and thickness of the sleeve. The mechanics and location of rupture determine the maximum attainable average membrane strain. Since higher membrane strains occur near midheight of the cylinder, the elevation of the hatch is also an important factor.

If a premature failure of a stiffener does not occur, a rupture is most likely to initiate adjacent to a major penetration near midheight due to the strain concentration at that point. In this case, average membrane strains near midheight could reach 7.5% to 10% (assuming strain concentration factors from 1.5 to 2.0, and an ultimate strain of 15%), and we would expect leakage from pressure seating equipment hatches, probably sufficient to preclude rupture.

If a stiffener fails prematurely, it is expected to be near a penetration because of the complex patterns at these locations. Should this be the case, a very severe strain concentration can occur, and the average membrane strains may reach only 1% to 5%. The strain concentration produced by the combination of a penetration and failure of a stiffener near that same penetration is greater than the product of the two individually, because of the shape of the stress-strain curve. The likelihood of leakage from pressure seating equipment hatches is reduced in this scenario.

Stiffeners and elevated temperature both affect the leakage potential of pressure seating equipment hatches. The relationship of the general yield pressure to the rupture pressure and the shape of the stress-strain curve help to determine the average membrane strain at rupture. In general, the larger the ratio of rupture pressure to general yield pressure, the higher the strains. Stiffeners increase the general yield pressure but do not necessarily increase the rupture pressure, thereby decreasing the leakage potential. On the other hand, up to 600°F (316°C) the minimum specified yield strength of A516 steel decreases while the ultimate strength increases. Therefore, for elevated temperature, the general yield pressure is lower and the average membrane strains at rupture are larger than at room temperature. This, thereby, increases the leakage potential of pressure seating equipment hatches.

For the other types of penetrations, including drywell heads, pressure unseating equipment hatches, and personnel air locks, leakage depends strongly on seal performance. Seal performance is, in turn, highly dependent on temperature. The maximum severe accident temperature for steel containments used in PWR and BWR Mark III plants is estimated to be 400°F (204°C). The seals we have tested so far have not shown significant degradation at this temperature. There may still be some reduction in seal performance in these plants due to radiation or thermal aging. On the other hand, steel containments in BWR Mark I and Mark II plants may be subject to temperatures near 700°F (371°C), which is sufficient to cause degradation of every seal tested. Therefore, in BWR Mark I and Mark II plants, any loss of metal-to-metal contact at sealing surfaces may result in leakage.

Analyses show that the combination of bolt preload and the forces produced by the differential thermal expansion of the bolts and flanges is sufficient to maintain metal-to-metal contact in BWR drywell heads for an internal pressure of 120 psig (0.83 MPa) and temperature of 750°F (400°C). We expect to verify this analysis, at least in part, by applying the same method to the pressure unseating equipment hatch being tested in the 1:6-scale reinforced concrete model.

For personnel air locks, we believe separation of the sealing surfaces on the inner door and bulkhead is likely below the rupture pressure. Consequently, we feel there may be leakage past the inner door in BWR Mark I and Mark II containments. In PWR and BWR Mark III containments, leakage past the inner door may occur if thermal or radiation exposure causes a loss of resiliency in the seal. It is likely that the outer door and bulkhead, which are not subject to the maximum accident temperature, will prevent leakage.

Some personnel air locks use inflatable seals, which require a much different approach. In these cases, leakage depends only on the performance of the seal, which may be compromised when the internal pressure exceeds the normal operating seal pressure.

4.3.3 Failure Criteria

Our failure criteria define a threshold environment, that is, the pressure and temperature at which leakage or rupture is imminent. In a LWR containment building, there are a number of potential failure mechanisms, or release paths. The release paths we have considered are described in Table 4-2. Separate structural calculations may have to be made for each release path, each with its own unique modeling requirements.

To date, we have proposed criteria for rupture of steel shells, leakage in pressure seating equipment hatches, and pressure unseating hatches. We will propose criteria to evaluate leakage from other types of penetrations upon completion of testing. The test results for EPAs apply directly to full-size containments* since they are conducted on full-size, vendor-supplied EPAs subject to environments that envelop the accident environments. The leakage characteristics of isolation valves have been investigated by INEL.

The rupture criterion for steel shells that we propose is empirically based. Despite this, we recognize tensile instability as the actual failure mechanism. Calculating the loads at which a tensile instability occurs is a difficult problem. For steel containments, we believe that the tensile instability will generally occur when the strain is near the material's ultimate strain. Thus:

- Rupture will occur if, at any point on the pressure boundary, the equivalent strain exceeds the material's ultimate strain (the strain at maximum load from a uniaxial tensile test). If the

* Early containments with in-field EPA designs are excluded.

Table 4-2. Release Paths in Steel Light Water Reactor Containments

Rupture of the Containment Shell
Leakage past Sealing Surfaces (Operable Penetrations)
 Pressure Seating Equipment Hatches
 Pressure Unseating Hatches (Drywell Heads, Equipment Hatches)
 Personnel Air Locks
Leakage past Purge and Vent Valves
Leakage from Electrical Penetration Assemblies
Leakage due to Failure of a Bellows (Expansion Joint)

strain is due primarily to bending, somewhat higher strain may be allowable (to account for crack growth through the thickness).

We believe through-wall cracks in a steel containment resulting from quasi-static internal pressurization will grow in a stable fashion for a short period of time until a critical length is reached, at which point rupture occurs. The stored energy in a containment at the rupture pressure is so great that venting through the growing crack does not cause a significant drop in pressure before the critical crack length is reached. Consequently, the stresses at the crack tip are not relieved and the crack cannot be arrested. We conclude that a through-wall crack that would cause a venting of the containment precluding rupture is extremely unlikely. Therefore, our rupture criterion is really meant to represent any cohesive shell failure (tearing).

For a pressure seating equipment hatch, separation (gapping) of the sealing surfaces does not occur. However, due to ovalization of the sleeve while the door (cover) remains circular, the sealing surfaces can lose contact at some point around the circumference. In this situation, leakage is actually independent of seal performance, and the leakage criterion for pressure seating equipment hatches can be stated simply:

- Leakage will occur if the radial displacement of the sleeve at the sealing surface relative to the cover exceeds the thickness of the sleeve at any point around its circumference.

In Section 4.3.1, we showed that this criterion cannot be satisfied unless significant plastic strains are developed in the cylinder (from 2-8%). Some researchers have suggested that rotation of the sealing surfaces relative to each other may also increase the likelihood of leakage. We believe just the opposite is true, because rotation may help to maintain metal-to-metal contact, which precludes leakage. On the other hand, out-of-flatness of the sealing surfaces would tend to increase the potential for leakage.

In pressure unseating equipment hatches and drywell heads, the internal pressure works to separate the sealing surfaces. Preload in the bolts attaching the hatch to the sleeve (or the head to the lower shell) resists the tendency to separate. In many cases the bolts may be at a lower temperature than the flanges, which, for bolt temperatures not exceeding 800°F (427°C), has the effect of additional preload and effectively raises the internal pressure necessary to unseat the hatch.²⁹

The failure criterion for pressure unseating equipment hatches and drywell heads is more complicated because it depends on deformation and seal performance. Since data on seal performance are not yet complete (particularly with regard to aging phenomena), the following criterion should be regarded as a working model:

- If temperatures are high enough to cause seal degradation, leakage will occur when the pressure is sufficient to cause separation of the sealing surfaces. (Leaking may begin at very low pressures, much lower than the unseating pressure, if out-of-flatness of the sealing surfaces prevents uniform metal-to-metal contact.)
- If the seals are not degraded, but aging causes a loss of resiliency, at some point into the accident any subsequent increase in the separation will cause leakage.
- If the seals have not been degraded or lost their resiliency, leakage will occur when the seal squeeze becomes equal to zero (when the separation is equal to the initial seal compression).

Some investigators have suggested that the seals may flow or move outward to fill small gaps. If this phenomenon can be substantiated, we will suggest appropriate modifications to the leakage criterion. It should also be noted that ovalization may also lead to leakage for pressure unseating equipment hatches, just as for pressure seating equipment hatches.

Seal performance will also affect leakage rate. In cases where the seal is degraded, we believe that what remains of the seal will strongly influence the leakage rate. We have only proposed methods to predict the onset of leakage. With this in mind, we hope that as additional data from the seal and penetration tests becomes available, approaches to predicting leakage rates will be developed.

We have not formalized a failure criterion for personnel air locks yet, but it will be similar to that for pressure unseating hatches. For personnel air locks, leakage again depends on separation of the sealing surfaces and seal performance. In this case, separation is caused by a mismatch in the bending displacements of the door and bulkhead. Results from the test of a full-size personnel air lock will be used to modify and refine a leakage criterion for personnel air locks. A failure criterion for bellows will not be postulated until tests are completed.

4.3.4 Features of an Analytical Tool for Calculating Steel Containment Response

We stress, again, the need for structural analysis that is consistent with the relevant failure criterion. In order to accurately predict the threshold environment for rupture of a steel containment, we need to calculate the post-yield response of the shell. Additionally, nonlinear calculations for the structural response of numerous penetrations are necessary if leakage is to be accurately estimated.

The experimental results and nonlinear analyses of the scale models demonstrated that the post-yield behavior cannot be extrapolated from linear analyses. Linear analyses cannot account for the effects of load redistribution, plastic flow, or the shape of the stress-strain curve for A516 steel. For instance, at the base of the cylinder where radial displacement is constrained, an elastic stress concentration associated with bending is observed. The stress at the base is nearly twice that at midheight while the response is still elastic. Yielding occurs near the base of the cylinder well below the general yield pressure, but the stress and strain do not increase appreciably until general yield occurs. Then, as the pressure is increased beyond general yield, the strains at midheight increase more rapidly than those at the base. In the scale model tests, rupture was always associated with high strains near the cylinder midheight. Failure was unrelated to the elastic stress concentration at the base of the cylinder.

Figure 4-1. Deformed Shapes of Ring-Stiffened Shell

Another example of how elastic analysis can be misinterpreted is illustrated by Figure 4-1. Elastic analyses show that the radial displacement of the cylinder midway between stiffeners is nearly twice that adjacent to a stiffener. Yet, after general yielding, the cylinder deforms into a barrel shape with the relative difference in displacement midway between stiffeners and behind stiffeners becoming almost insignificant.

In the scale models, the penetrations at cylinder midheight affected the failure pressure and location, but not in the manner one might have expected from an elastic analysis. The elastic stress concentrations above and below penetrations were relieved as plastic flow occurred, and at the same time, circumferential stretching generated strain concentrations in the cylinder along the horizontal diameter of the penetrations. Strains adjacent to penetrations may be as much as twice that of "free-field" strains. This does not, however, necessarily cause a substantial drop in capacity because the stress-strain curve has a relatively flat slope near the ultimate load.

For pressure unseating equipment hatches and drywell heads, the structural response parameters used in the leakage criterion can generally be calculated by linear elastic methods. In this case, there are a number of suitable approaches for calculating the response. However, to predict leakage from many types of penetrations and to predict rupture of the shell, the nonlinear response must be known.

We believe that the only approach that can be used to reliably predict the nonlinear response, especially for the complex geometry of containments, is the finite element method. There is a restricted class of computer codes that has the features needed to represent the mechanics of large strain and large deformation typical of containment response. We used MARC, a general-purpose finite element code,³⁰ to calculate the structural response of the steel models. Chicago Bridge and Iron and Southwest Research Institute also did calculations for the small steel models using ADINA.^{2, 31, 32} Based on comparisons of the calculated response with experimental data, we identified the features of the analysis that are needed to obtain reliable results.

4.3.4.1 Material Modeling

The stress-strain relationship for each type of steel should be represented by a multilinear curve up to at least the ultimate strain. Minimum specified properties are not normally representative of the actual materials; properties may vary as a function of heat (i.e., production run) and thickness. The specified properties for the material also give little indication of the shape of the stress-strain curve. We have found that variations of properties within a particular heat are negligible. However, the temperature dependence of the yield strength, tensile strength, and work hardening slopes of steels can have an important effect on performance, and should be represented in the material model.

Material properties should be determined from uniaxial tensile tests on the actual fabrication materials if at all possible. However, we found that the uniaxial yield stress was about 10% too high in the model tests because of strain rate effects and the Bauschinger effect.^{3, 24} Of course, these factors introduce uncertainty into the predictions for structural performance.

4.3.4.2 Geometric Nonlinearities

Second-order and large-rotation terms need to be included in the strain-displacement relations. The stiffness matrix should be updated to reflect the changes in geometry, including thinning of the wall. Distributed forces should be formed on the basis of current geometry (i.e., loads from pressure should remain normal to the surface).

4.3.4.3 Element Formulation

Thin shell elements appear to work well. We used MARC element 72, which is a bilinear constrained 8-node shell element,³³ for our three-dimensional (3D) analysis. Other elements should be benchmarked using data from the steel scale models.

The effects of elevated temperature were not incorporated into the scale model tests, and therefore, the analytical methods are not benchmarked for elevated temperature. Nevertheless, we feel that the effects of elevated temperature can be handled by most general-purpose finite element codes. The main difficulty is in determining the actual temperature profiles in the shell and penetrations. Thermal stresses tend to be self-limiting, and consequently they are not likely to threaten containment integrity.

4.4 Concrete Containment Modeling

In order to establish the framework for discussing containment performance evaluation, it should be stated at the outset that there are two distinct objectives and corresponding approaches involved in containment evaluation. The first objective and approach is design analysis; the second objective and approach is failure analysis.

4.4.1 Containment Design Analysis

Concrete containments are constructed to withstand design pressures while experiencing stresses no greater than those specified in codes such as ACI-349. Historically, design pressures are based on energy balance calculations from the assumption of a LOCA. Guidelines established by the Standard Review Plan,³⁴ which references 10 CFR 50, Appendix K, give the heat flux parameters and energy transfer assumptions that are to be used in the calculation. In short, the underlying principle that dictates design pressure is the assumption that, when vessel blowdown occurs, all available water in the system within the containment is converted to steam. The pressure in the containment is a result of this total steam production less the amount of energy dissipated through the heat sinks, assuming a closed free volume.

As has been established by the recent studies conducted in the development of NUREG-1150,^{35, 36} there are many uncertainties involved in the calculation of the pressure. Significant probabilities exist for the occurrence of pressures greatly in excess of the design pressure due to severe accidents conditions not previously considered.

Once design pressure is established, the containment is typically analyzed by routine elastic analysis to ensure that code allowable stresses are not exceeded. It was not until the 1980s that containment failure analyses were conducted for overpressure conditions. Because of the highly nonlinear behavior of concrete containment structures, the analytical requirements for failure analysis differ significantly from those for design analysis. The absence of containment failure analyses from general practice has been due mainly to the more demanding analytical requirements involved. It was not until the Three Mile Island (TMI) accident, which created the need for better understanding of containment overpressure behavior, that such investigations were undertaken. However, despite major progress, failure analysis of containments is still a relatively new structural engineering activity, and it is in this light that we must view the emerging methods for evaluating containment overpressure behavior.

4.4.2 Concrete Containment Failure Analysis

As the 1:6-scale model and a variety of containment specimen tests¹⁰ have demonstrated, to predict containment failure requires explicit analytical treatment of local liner details and stiffness discontinuities. At high pressures, cracking-induced liner-anchorage-concrete interaction is similar in both reinforced and prestressed containment structures even though it is initiated earlier in reinforced structures. As the concrete cracks, the liner begins to deform plastically (generally earlier in reinforced containments than in prestressed containments). Liner-concrete interaction (including anchorage effects) becomes the dominant local effect mechanism that will eventually govern the subsequent liner tearing behavior.

As previously mentioned, as recently as five years ago, it was generally assumed that concrete containments would fail in global hoop expansion of the barrel (a burst mode). But the latest experiments and detailed local analyses have shown that, even though the liner is quite ductile, the strain concentrations that occur next to discontinuities or at anchorages will cause liner tears prior to the occurrence of barrel burst mode failure. It is far more difficult to predict when and where a tear will occur and what the leakage behavior will be than it is to calculate the ultimate burst pressure of the containment.

4.4.3 Developing the Finite Element Model

There are two major issues involved here: determining what geometric details of a containment need to be represented and establishing proper mesh discretization. Again, the models need to be consistent with the failure criteria and the analytical methods that are adopted.

Various finite element modeling strategies have been used by different groups for axisymmetric two-dimensional (2D) global analysis. From the experience gained in the current research program and for the reasons cited in the preceding discussion, guidelines can be implemented for 2D grid development. A grid that adequately captures global response for a typical prestressed concrete containment is shown in Figure 4-2. It consists of 231 8-node continuum elements for modeling concrete, 53 three-node shell elements for the liner, 44 beam elements to model the liner angle embedments, and ABAQUS **"*REBAR"** subelements to model tendons and rebar. This grid size is quite manageable and is computationally inexpensive on most computers.

Similar global analyses have also been performed for reinforced containment prototypes such as the 1:6-scale model. An example of this recommended grid is shown in Figure 4-3. The grid consists of 243 8-node, second order, axisymmetric elements for the concrete, 62 3-node shell elements for the liner with 5-point integration through the thickness, and ABAQUS **"*REBAR"** subelements. Grid convergence studies were conducted for this grid in Ref. 37.

4.4.3.1 Basemat

We found the proper modeling of the basemat to be important to accurately predict the potential failures at the wall-base juncture. It is generally not important in predicting the response of the barrel and the dome. Accuracy is required in predicting basemat liftoff that can occur at high pressures and control the global behavior, as in the case of the Sizewell-B 1:10-scale model. Local basemat details, such as sumps, cutouts, and keyholes, have been ignored in the global analyses performed to date, particularly because they have not been included in the test models. The interaction of the foundation with the tendon gallery has yet to be studied in detail. However, for a detailed evaluation of an individual plant, it may be necessary to account for major cutouts, keyholes, and the tendon gallery.

For axisymmetric analyses, regularly spaced grid is recommended over to the symmetry line to adequately calculate the bending moment in the basemat, which is a maximum at the axis of symmetry. This bending moment causes the possibility of yielding the basemat bottom bars that, in the cases of the 1:10-scale containment and the 1:6-scale containment, led to significant basemat liftoff (more so in the 1:10-scale containment). In most actual reinforced or prestressed containments, leakage is likely to occur before yielding of the basemat bottom bars in bending.

"No-tension" springs (sometimes referred to as "Winkler" springs) should be used along the bottom of the basemat when it is supported on soil to allow the basemat to lift off and redistribute the soil contact pattern. Spring stiffnesses should represent available soil stiffness measurements for the particular containment site being analyzed, but approximation of this stiffness should be adequate for predicting accurate global response. The global response has not been found to be very sensitive to soil stiffness. Rock foundations can usually be treated as rigid foundations.

4.4.3.2 Wall-Basemat Juncture

Specific modeling details of the wall-basemat juncture in a global analysis grid are described in Ref. 38. For our discussion at this point, it suffices to say that some grid refinement is mandatory to capture the large shear and moment gradients that occur at the base of the wall. At a region of high bending moment, it is important to calculate the strain in the various rebar layers correctly by explicit modeling of discrete layers. Rather than smearing properties for the entire wall section. We note that it is not necessary nor is it recommended to predict liner strain concentrations across cracks or at liner embedment discontinuities.

Figure 4-2. Computational Grid for Typical Prestressed Containment

Figure 4-3. Computational Grid for Typical Reinforced Concrete Containment

4.4.3.3 Wall and Dome

For grid uniformity and simplicity, three elements through the thickness are maintained up to the dome apex. The grid spacing is reduced in the springline region to ensure adequate calculation of bending moments that occur due to the dome-to-cylinder stiffness transition. In the main wall and dome regions, use of 8-node elements with quadratic displacements along element sides allows relatively large element spacing. For concrete cracking analyses, higher order elements with lower order integration (i.e., 8-node quadrilaterals with 2×2 integration) are preferred because of their superior numerical behavior under cracking conditions. Finer element integration typically leads to poorer convergence characteristics because of the significant increase in unique cracking states that can occur in any given element.

The apex of the dome is often a subject of modeling difficulty in both reinforced and prestressed containments. In reinforced containments, bars are normally terminated at some type of cap plate because of the construction constraints. In prestressed geometries, dome tendons often are not draped axisymmetrically, which leads to a modeling discrepancy in axisymmetric grids. Simplified treatment of both of these problems is justified by the fact that no significant liner strain concentration has ever been demonstrated at or near the apex in experiments or analyses of either prestressed or reinforced concrete containments. Computational modeling of the apex does affect the level of vertical displacement of the apex relative to the rest of the dome. Overly "stiff" models, particularly for prestressed containments, lead to a downward dimple at the apex. Overly flexible apex models lead to larger vertical apex displacements than the rest of the dome. However, in modeling parameter studies investigating these scenarios, liner strain concentration factors were always low; therefore, premature leakage due to liner rupture at the apex is not a consideration that could be affected by variations in computational modeling.

4.4.3.4 Rebar Modeling

The modeling of rebars in a finite element code varies widely from code to code. Displacement compatibilities between the rebars and the concrete require that the rebars be represented by the same displacement shape functions as the continuum element in which they reside. This is not generally observed in all computer codes. Another difficulty is the translation of rebar drawings to computer code input. For 2D axisymmetric grids, rebar modeling is relatively straightforward. However, for 3D continuum analysis, the rebar arrangements are very complex. As such, it is virtually impossible to develop computer input without preprocessing software such as the one described in Ref. 39.

Some work has been performed studying tendon friction losses and the effects of modeling tendons as rebars with concrete strain compatibility. This contrasts to modeling them with frictionless rebar elements. In the 1:10-scale prestress containment analyses, we found that a bonded-tendon assumption (versus a frictionless tendon assumption) improved the predicted response of the wall-base region. Yet, it had little effect elsewhere in the structure.

In an axisymmetric analysis, strain compatibility of hoop tendons is automatic. To the contrary, with meridional tendons, one has a choice of modeling tendons as ordinary rebar, or as separate truss elements external to the grid and attached to the grid with linkage elements that only transmit forces normal to the tendon. For a prestressed containment analysis, one should take this into consideration for choosing a meridional tendon modeling strategy; the current study of this modeling technique is still ongoing, and conclusions have not yet been finalized. Predictions for leakage from hatches and penetration seals can be based on the methods recommended in the steel containment research reports.^{40, 1, 2, 3}

4.4.4 Features of Analytical Tools for Calculating Response

Reliably predicting the threshold environment for leakage or failure of a concrete containment requires accurately calculating the post-yield response. Nonlinear 3D calculations for the structural response are

also necessary for local stiffness discontinuities and penetrations if liner tearing and leakage is to be accurately predicted.

Methods for performing such analyses will almost certainly require the use of submodels of local regions rather than a 3D analysis of the entire containment. Local submodels can be loaded via specified boundary displacements generated from coarse global analyses. These boundary conditions need to include the effects of the interaction of the penetration or discontinuity with the free field structure. For example, penetration regions have been observed to displace smaller radial distances than the free field, causing out-of-plane relative motion between the penetration and the free field. Examples of analyses of this type are described in Ref. 7.

For equipment hatches and drywell heads, the structural response parameters used in the seal leakage criterion developed in Ref. 40 can generally be calculated by linear elastic methods. In this case, there are a number of approaches applicable to calculating the response. However, to predict leakage from many types of penetrations, the nonlinear response of the supporting wall must be known.

For most of the analyses described herein, ABAQUS, a general-purpose finite element code,⁴¹ was used to calculate structural response. Analysts participating in the 1:6-scale model round-robin analysis also did calculations using ADINA, CASTEM, NEPTUNE, NFAP, PAFEC, and TEMP-STRESS. Based on comparisons of the calculated response with experimental data, the features of the analyses that are needed to obtain reliable results have been identified.

4.4.4.1 Material Modeling

Pertinent details for modeling steel materials were discussed previously in Section 4.3.4.1. These factors must also be included in reinforced and prestressed concrete containment analyses. In particular, the low-wave hardening of liner steels in the yield plateau requires careful verification of the nonlinear strains.

Concrete constitutive models should include tensile cracking (normally treated as occurring in the principal stress directions at the integration points). Alternately, the concrete can be treated as a no-tension material in analyses aimed at predicting global response. Some method for simulating post-cracking shear retention should also be included. Shear retention is particularly important at the wall base, and numerical solutions in this region are found to be sensitive to the shear retention algorithm used. Advanced algorithms including "rough crack" models of crack face friction have been developed in the research programs, as described in Ref. 42. Description of these methods, however, is beyond the scope of the current paper.

While most of the concrete elements in the computational grid will probably experience no compressive stress at high pressures, a few elements at the outer wall base and along the top of the basemat will experience compression, sometimes large enough to approach f_c' . Thus, it is important to model concrete crushing. An elastic/perfectly plastic stress-strain curve has shown reasonable results in most cases as compressive strains are not expected to reach the strain-softening regime. Inclusion of strain-softening is preferred for completeness, yet the associated plasticity algorithm is more complex. As complex behavior is to be expected at the wall base, it is important that the concrete compressive yield algorithm properly treat the effective stress and strain calculation. Compressive yield is more likely to be reached by the combination of vertical compressive stress and shear stress rather than any one component stress reaching f_c' . This condition is essential for predicting potential wall-base shear failure.

4.4.4.2 Geometric Nonlinearities and Other Issues

Contrary to the recommendations for steel containment analysis, we found the second order and large rotation terms need not be included in the strain-displacement relations. The stiffness matrix need not be

updated to reflect the changes in geometry or thinning of the wall, as it is in steel shells. The effects of elevated temperature discussed in Section 4.3.4.3 are relevant to concrete containments.

4.4.5 Computational Methods Required for Concrete Failure Analysis

The present state of the art of modeling and analysis of concrete structures consists of finite element computer codes in which: (a) plain concrete and steel reinforcement are modeled as two materials joined together as structural elements; (b) plain concrete material models account for compressive plasticity and tensile cracking (smeared over the element) with post-cracking tension stiffening and shear retention capabilities; or (c) reinforcement is treated as simple tension-compression members fully bonded to the concrete. In addition to these three areas of global analysis capability, calculation of local liner strain states requires analytical treatment of the liner-anchorage-concrete and even the rebar-concrete interfaces (although the latter is of secondary importance). Analysis of local details near penetrations also requires complex labor and computer-intensive three-dimensional modeling. Local effects and 3D local analyses can be conducted based on the displacement results of global analyses used as boundary conditions. Results of local effects analyses conducted for the 1:6-scale containment are discussed in detail in Refs. 43, 44, and 45.

4.5 Interpreting Containment Failure, Leakage, and Leakage Rates from Analytical Results

4.5.1 Definition of Failure and Leakage

Answering the liner tearing question alone does not address the issue of quantification of risk. The questions implied when clarifying a tear prediction are: does this mean the containment has failed and what is the release?

Recent publications regarding containment overpressure behavior, including the latest draft of NUREG-1150, show a lack of agreement between structural experts regarding the definition of containment failure. The lack of consensus is evidence of the general gap in our knowledge of how incipient liner tears will manifest themselves in terms of leak areas and release rates.

In the NUREG-1150 draft, a "leak" is described as a containment breach that would arrest a gradual pressure buildup but would not result in depressurization in less than two hours, whereas a "rupture" would depressurize the containment within two hours. These distinctions have been made because: (1) the consequences of an early failure appear to be considerably greater than those of a late failure, and (2) for a late failure, the consequences of rupture are worse than those of leakage. Leak areas of 0.1 and 1.0 square feet were assigned to the leak and rupture definitions, respectively. In the NUREG Appendices, these definitions are used interchangeably with "failure mode."

Containment performance discussions historically have also been divided between performance during quasi-static pressurization, and performance during postulated steam explosions or dynamic pressurization. The structural behavior discussion for the latter category is generally non-quantitative because little work has been done to analytically or experimentally investigate containments subjected to internal explosions or deflagrations. The focus in that area generally is on quantifying radiological releases from the containment. This remains a challenge in predicting containment performance.

4.5.2 Liner Tearing Criteria

The tearing criterion for the liner plate was developed in the EPRI research³⁹ from a multiaxial stress and ductility formulation. Tests on various ductile materials were cited by Manjoine⁴⁰ and a ductility ratio formula was proposed where the ductility can be determined using the following equations:

$$\mu = \frac{\epsilon_{eff}}{\% \text{ Uniaxial Elongation}} \quad (4-4)$$

$$\mu = 2(1-TF) \quad (4-5)$$

where μ is the ductility ratio and TF is the Davis triaxiality factor defined by:

$$TF = \frac{\sqrt{2} (\sigma_1 + \sigma_2 + \sigma_3)}{[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]^{1/2}} \quad (4-6)$$

The percent uniaxial elongation shall herein be called ϵ_{uf} for uniaxial failure strain. According to the theory, a tear will initiate at a point when the equivalent plastic strain ϵ_p , satisfies $\epsilon_p > \mu\epsilon_{uf}$. The ductility relationship, Equation 4-5, has been found to be a reliable formulation, particularly for the triaxiality factors that are most likely to occur in containment liners. While Equation 4-5 was proposed by Manjoine for the range $0 < TF < 5$, containment analysts need only be concerned with $1 < TF < 2$ to justify this statement. It has been assumed that, at stress states of interest, no membrane compressive stress exists in the liner, and out-of-plane stresses (σ_3) have a negligible effect on TF. This reduces Equation 4-6 to:

$$TF = \frac{(\sigma_1 + \sigma_2)}{(\sigma_1^2 - \sigma_1\sigma_2 + \sigma_2^2)^{1/2}} \quad (4-7)$$

With this formula it is straightforward to calculate ductility ratios. For example, for uniaxial tension, $\sigma_2 = 0$: $TF = 1$ and $\mu = 1$. For $\sigma_1 = \sigma_2$ (i.e., the approximate stress condition in the containment dome): $TF = 2$ and $\mu = 0.5$; this means that the liner will tear at an effective plastic strain of only half the uniaxial failure strain.

4.5.3 "Facts" About Failure Modes and Leakage

In light of the lack of consensus on the definitions of failure modes and leakage, we find that it is appropriate to summarize what is and is not known. Herein, we do so with regards to failure modes and leakage.

4.5.3.1 Failure Mode

The term "failure mode" is often misused in containment evaluation. "Failure mode" pertains, to a greater extent, to the structural limit state under a given loading. For example, "alpha mode" of a vessel head projectile impact would be expected to cause a different type of failure than pressurization and is, thereby, an appropriate use of the term failure mode. However, it is more appropriate to refer to leakage as a local loss of function at a particular location, whereas "burst" can properly be referred to as a failure mode.

4.5.3.2 Liner Tearing

When subjected to quasi-static pressurization, concrete containments with steel liners should be expected to develop leakage. There is no known experimental or analytical evidence to the contrary. It should, however, be emphasized that all tests conducted on concrete pressure vessels known to the authors, have

used hydrostatic pressurization. The only exception to this is the SNL test. Hydrostatic testing significantly decreases the probability of a catastrophic rupture of the pressure vessel.

Detailed local analyses demonstrating potential liner tearing mechanisms have been carried out by various groups for many stiffness discontinuity configurations. Numerous concrete vessel and local specimen tests with steel liners have demonstrated the liner tearing and leakage phenomena; a list of known tests is given in Table 4-3. Note that, in many concrete vessel tests, leakage of steel-lined vessels is such a common result that rubber bladders are used instead to try to circumvent the leakage that is recognized as being almost certain to occur.

It is not known at the present time if a leakage failure will reach an equilibrium state or if it will lead to a burst failure. The arguments for each follow.

4.5.3.3 Equilibrium Leakage

The equilibrium leakage concept first introduced in WASH-1400 and related work⁴⁶ has been covered in numerous reports over the last decade. With this in mind, our quasi-static pressurization failure scenario for concrete containments follows below.

As a concrete containment building (either reinforced or prestressed) begins to expand in response to quasi-static pressurization, liner strain concentrations occur at local stiffness discontinuities. The strain concentrations can be strong or weak depending on the type of discontinuity and the type of liner to concrete anchorage system. The strength of the discontinuity affects the level of global strain needed to tear the liner. These global strains have been found (from testing and analysis) to range from 0.2% (just above yield) to about 2%. With regards to the weakest discontinuities, 2% global strain is still well within the ultimate capabilities of the concrete containment reinforcement; thus, a liner tear is deemed to be almost certain to occur.

Once the strain concentration causes a tear to initiate, the tear will not grow beyond the size required to maintain equilibrium between pressure increase and leakage. This is because: (a) the structure's residual stiffness at initial rupture is sufficiently large to restrain the liner from rapid expansion, and (b) the liner tearing is a stable fracture process due to the highly redundant crack arrest mechanism offered by the concrete backing. This leads to the conclusion that liner crack growth beyond the equilibrium leak area requires further pressure rise.

4.5.3.4 No Equilibrium Leakage

The high pressure expulsion of molten material from the reactor vessel, the deflagration of combustible gases, and the rapid generation of steam through the interaction of molten fuel with water in the containment are phenomena that could lead to pressure rises in the containment over a period of a fraction of a second. While this pressure rise time is still effectively quasi-static, some experts feel that the rapid rise time would not allow large enough tears to develop. This would lead to a more severe failure of the liner. We need to note that proponents of this viewpoint still tend to support giving high probability of occurrence of leakage. They merely add that, for certain pressurization rates, more severe failure modes involving higher pressures can also be reached.

4.5.3.5 Reinforced Versus Prestressed Containment Behavior

Based on the 1:6-scale containment model, the 1:10-scale containment model, and several global analyses of other reinforced and prestressed concrete containment prototypes,⁴⁷ significant insight has been gained regarding the behavior of both containment types. The global behavior of reinforced concrete containments differs significantly from the behavior of prestressed concrete containments in that a reinforced containment will develop hoop cracking much earlier than a prestressed containment.

Table 4-3. Experiments for Liner Tearing and Leakage

Test	Scale	Shape	Radius-to-Thickness Ratio	Pressure Ratio (Leakage-to-Design)	Homogeneous Barrel Strain at Leakage	Liner	Remarks
Fort St. Vrain G. A. Model 1, U.S.A.	1:4.5	Cylinder	5	3.5,2.5	0.22%	Steel	
Wylfa Vessel, U.K.	1:12	Sphere	5	See remark	0.1%	Rubber	2.7 pressure ratio reached with no leakage
Oldbury Vessel, U.K.	1:8	Cylinder		3	0.4%	Steel	
"	1:12	Sphere	5	2 - 3	0.4%	Rubber	
"	1:8	Cylinder	3.5	2 - 3	-	Steel	Shear-type failure artificially induced
1000 MWe HTGR G. A. Model 2, U.S.A.	1:4	Cylinder	2.4	2	0.06%	Steel	
Sandia Reinforced Containment Model, U.S.A.	1:6	Cylinder with dome	13.5	3.0	1.3%	Steel	
CTL Spec. 2.5	Full	Pre-stressed wall-base juncture		2.6*	0.2%	Steel	Several tears at wall-skirt juncture
CTL Spec. 2.4	Full	Pre-stressed wall with penetration		2.4*	2.2%	Steel	Large tear at penetration
CTL Spec. 3.2	Full	Pre-stressed wall with penetration		2.9*	2.7%	Steel	Severe liner necking next to anchorage
CTL Spec. 2.2	Full	Pre-stressed wall with initial liner flaw		-	-	Steel	Pre-existing flaw extended in ductile manner
CTL Spec. 3.3	Full	Reinforced		-	1.6%	Steel	Strain concentration measured near penetration (4.3% max.) but tearing not reached

* Leakage versus design pressures for Construction Technology Laboratories specimens are best estimates based on global strain levels reached; these specimens were not pressure vessels.

Calculations show that substantial hoop cracking can occur in a reinforced concrete containment between 70% and 90% of the design pressure, depending on the percentage of steel. Therefore, a reinforced containment, unlike a prestressed containment, will experience extensive cracking before the design pressure is reached. When a reinforced containment cracks, the main reinforcement is stressed only to approximately 30% of its yield, whereas tendon reinforcement in a prestressed containment is at 80% or 90% of yield when the concrete cracks.

This, combined with the fact that conventional rebar has greater ductility than prestressing tendons, gives reinforced containments an after-cracking pressure capability range that is significantly larger than in prestressed containments. This implies that leakage before catastrophic rupture is more likely to occur in a reinforced containment than in a prestressed containment.

On the other hand, some typical details of prestressed containment designs include stronger liner anchorages and more severe stiffness discontinuities than in reinforced containment designs, as evidenced by the Construction Technology Laboratories (CTL) specimen test series.¹⁰ Therefore, with the site specific nature of containment designs, it is not possible to generalize what pressures cause leakage in prestressed containments as compared to reinforced containments. The relative behavior of the two containment types can be summarized by Figures 4-4 and 4-5 that show the stress-strain behavior of the primary load carrying elements for reinforced and prestressed concrete containments, respectively.

The liner and rebar behavior is basically the same in both cases except that, in the prestressed case, the liner and rebar stresses remain compressive until approximately 1.4 times the design pressure (shown as 1.4 DP on Figure 4-5). The significance of the figures lie in the relative positions of the liner yield range versus the yield range of the primary load carrying components and the ultimate strain capacity of the materials. The S-shaped curves shown represent the general shape and relative location of the expected probability of leakage and probability of catastrophic rupture curves.

After liner yielding, the probability of leakage becomes significant due to liner strain concentrations. We note that it is not until much larger global strain (and pressure) that the probability of catastrophic rupture becomes significant. And in conjunction with this, if the failure pressure is approached from below in quasi-static fashion, the range of pressure needed for catastrophic rupture might not be reached.

4.5.3.6 Temperature

The subject of temperature remains largely an untested issue with regard to global containment behavior. This is due to the experimental difficulties involved with modeling elevated temperatures. Conclusions about global behavior must be taken primarily from analytical studies. Experiments have, however, been conducted for local structural details and penetrations by both SNL in the NRC research and CTL in the EPRI research. Results of the analytical and experimental studies are summarized below.

Studies of temperature effects in the EPRI research included a test of a prestressed containment specimen (specimen 2.6 in the CTL test series¹⁰), a reinforced containment specimen (specimen 2.7), and analyses of these specimens. Typical prestressed and reinforced geometries have also been investigated by analysis for combined pressure and global temperature up to 400°F (204°C), plus the addition of a local temperature spike simulating a local hydrogen burn.

The specimen tests and analyses were aimed at inducing thermal buckling in the liner. The results of that work showed that temperatures in the ranges predicted to occur (by NUREG-1150, for example, 350° to 400°F (177° to 204°C)) have only a small effect on the ultimate pressure capacity for both reinforced and prestressed containment structures because the cracked concrete carries no tension regardless of temperature. Also, temperatures in this range have only a minor effect on typical rebar properties.

Figure 4-4. Idealization of Reinforced Concrete Containment Behavior

Figure 4-5. Idealization of Prestressed Containment Behavior

The increase in temperature changes the liner stress-strain behavior in the low pressure range, but the end results near failure are essentially the same as in the pressure only cases. The addition of local temperature spikes to the temperature history cause small local liner-concrete separation (a precursor to liner buckling) in prestressed containments. However, this is not so in reinforced containments. The tendency for liner buckling in prestressed containments is viable only for temperature spikes above 400°F (204°C) and pressures at or below design pressures; this may not be a plausible combination, thus effectively eliminating liner buckling as a realistic concern.

The liner buckling findings were suggested by the experiments mentioned above. In both prestressed and reinforced containments subjected to elevated temperature, the liner experiences compressive yield at low pressures, but eventually develops tension at higher pressures regardless of added temperature. Similar conclusions have been reached by other researchers, such as Wesley et al., at NTS/Structural Mechanics Associates.³⁷ It was concluded for a reinforced concrete containment that temperature effects are primarily limited to their effect on steel material properties.

If temperatures were developed in the range of 700° to 800°F (370° to 427°C) and existed long enough to heat up the embedded rebar or tendons, ultimate pressure capacities would be dramatically reduced because of degradation of the modulus and yield strength of the steel elements. With regard to liner tearing and leakage, the effect of temperature can delay tearing until slightly higher pressures, but at temperatures of 400°F (204°C) and below, it is generally believed that the effect is not enough to change the characteristic of leakage before catastrophic burst for gradual pressurization. Once liner yield is reached during pressurization, the global structure strain would need to be approximately 0.2% larger with a 400°F (204°C) temperature to cause the same global liner strain that would occur without temperature. The effect of temperature on leakage pressure predictions is easily handled by incorporating temperature distribution into a global containment analysis such as that recommended herein.

4.5.3.7 Leak Rates and Leak Areas

With the inception of the postulating of leak before catastrophic rupture, several groups have focused on the prediction of the onset of liner tearing. They have accomplished this by improving and refining advanced structural analysis techniques, and computing very localized liner strains with greater and greater precision. However, to our knowledge, these studies have not included fracture mechanics refinements to estimate crack size and trajectory. These exclusions are due to the complexity of the liner-concrete-anchorage interaction.

As we described in an earlier discussion, the fundamental gap in knowledge for estimating release risks during loss of coolant accidents is in estimating leak areas and leakage rates. In order to interface with regulatory requirements, leak areas and leakage rates must eventually be addressed. Some preliminary work has been conducted in this area by different groups, but the subject of estimating leak rates through liner tears and cracked concrete is particularly complex.

Nevertheless, based on existing work in this area, some fundamental conclusions can be drawn regarding leakage through cracks in concrete containments. In most cases, if liner tearing occurs in a region experiencing far-field rebar yield, leakage rates can be assumed to be governed by the liner tear aperture itself. This will not be greatly affected by any obstruction to the leak path posed by the concrete. Such a conclusion is supported by experiments performed at the University of Alberta, EPRI leak rate experiments, and leak rate measurements taken from the SNL 1:6-scale model.

To illustrate this further, for a concrete containment undergoing global hoop strain of 0.5% at the barrel midheight (still well within its ultimate barrel capacity) and using a concrete cracking strain of 100×10^{-6} , the sum of all the primary crack widths around the circumference of the barrel of 150-ft (45.7-m) diameter containment is 28 in. (71cm). At 1.3% strain, as was the case when the 1:6-scale containment

liner tears occurred, the sum of concrete crack widths in a full-scale containment is 72 in. (1.8 m.). Once the liner tears at far field strains of this magnitude, it is clear that the leak paths through the concrete will be quite large and the obstruction to flow through the concrete to be negligible in comparison to the throttling of flow through a small liner aperture. These conclusions are described in more detail by Dameron et al.,³⁸ in which approximate leak area and leak rate prediction rules have been established based on the Canadian tests, an EPRI test, and the 1:6-scale containment measurements.

5. SUMMARY

The NRC has sponsored a number of programs over the last several years that are collectively known as the Containment Integrity Program. This program investigates the behavior of LWR containments when loadings exceed the design basis. The SNL Containment Integrity Program has included: (1) scale model testing of containment buildings, (2) tests of seals and mechanical penetrations, (3) development of analytical techniques, and (4) electrical penetration testing. The objective of the program is to develop methods and guidelines for reliably predicting the performance of nuclear containment buildings subjected to severe accident loading conditions.

Seven tests of scale model containments were conducted within the test program. Six of these tests were conducted by SNL, with the seventh test conducted by CEGB in the U.K. The containment models tested at SNL had features commonly found in LWR containments, but did not model any specific containment. All of the tests were statically tested to failure using pressurized nitrogen gas with the exception of the 1:10-scale prestressed containment, which used water pressure contained in a rubber bladder inside the model. The data gathered from these tests was used primarily to qualify methods for predicting the response of LWR containments under ultimate loading conditions. The following paragraphs briefly describe the conclusions from the tests.

One 1:8-scale steel containment and four 1:32-scale steel containments were tested by SNL. The 1:8-scale steel containment and three of the four smaller 1:32-scale models failed in a burst mode. The three 1:32-scale models failed when the steel material experienced a tensile instability or the effective ductility of the material was exceeded. The one model that did not burst had a tear that formed in an area of the wall that was thinned due to grinding of a weld near a repair in the containment. In the 1:8-scale model, a small eccentricity in the stiffener pattern near an equipment hatch caused a stress concentration that led to the burst failure. The tests pointed out the need to consider the actual as-built details in the prediction of ultimate behavior in full-size steel containments.

A 1:6-scale reinforced concrete containment was built and tested to failure in July 1987. This test highlighted the need to evaluate in detail the potential failure mechanisms that a reinforced concrete containment can experience. The model failed when the steel liner tore near a reinforced electrical feed penetration and allowed leakage of approximately 5000% mass/day. The failure occurred at 145 psi (1.0 MPa) or about three times the design pressure. Pre-test analysis failed to identify the actual failure mechanism or location of the failure.

A 1:10-scale prestressed concrete containment was tested by the CEGB in the U.K.. This model incorporated a rubber bladder on the inside of the containment model to facilitate the use of testing with water pressure. The containment basemat failed at 112 psi (0.77 MPa) or 2.24 times design pressure and resulted in significant basemat uplift and tipping of the model. Concrete spalling also occurred on the outside containment walls near the basemat at the peak pressure loads due to a hinge mechanism that formed at the base of the cylinder wall.

Separate effects testing of containment equipment that is part of the pressure boundary has been performed under the Containment Integrity Program at SNL. Several conclusions can be drawn from the testing that include:

Three full-size electrical penetrations were tested under combined pressure and temperature loads. The pressure integrity of the penetrations was maintained throughout the range of severe accident pressures and temperatures used in the tests. The performance of other EPAs is not known but may be deduced from these tests.

A full-size personnel air lock was tested under severe pressure and temperature loads. This test showed that although the inner door of the air lock may leak due to temperature related breakdown of the seals, the outer door will likely maintain containment integrity during postulated severe accident conditions due to its reduced temperature environment.

Testing on the pressure unseating equipment hatch of the 1:6-scale reinforced concrete containment model is ongoing at the present time. This testing program will subject the model equipment hatch to different combinations of pressure and temperature loads utilizing different gasket materials. Preliminary comparisons between the measured and calculated behavior for the completed tests show that the prediction of the onset of the leakage can be predicted using the structural response of the hatch and available gasket springback. Prediction of leak rate is more difficult to quantify and is related to seal behavior.

Testing of bellows used at containment piping penetrations is in its initial stages of implementation at SNL. Various combinations of internal and external pressure, axial compression, elongation, and lateral deformations will be applied to typical bellow geometries to assess their ability to maintain pressure integrity.

Testing of seal and gasket materials has been completed. This program investigated the performance of different gasket materials used in containments under different loading and aging conditions. The tested gasket materials exhibited a threshold temperature above which significant leakage past the seals occurred in the tests.

Testing of inflatable seals that are used around the perimeter of personnel and escape lock doors has been completed. These seals were tested at elevated pressures and temperatures in both the aged and unaged condition. For all test conditions, significant leakage did not begin until the chamber (containment) pressure exceeded the initial seal pressure. A test validated procedure has been developed to estimate the internal pressure and temperature at which significant leakage past these seals occurs.

Analytical predictions and techniques that have been used as part of this program were presented. The results from the scale model containment tests and separate effect tests were used to verify the analytical techniques. These analytical techniques can then be applied to full size containments. Analytical techniques suitable for the failure analysis of full-size steel, reinforced concrete, and prestressed concrete containments were covered in detail.

5.1 Limits of the Work

Some aspects of containment integrity have not been addressed within the programs covered in this report. There are areas of behavior and response that have not been thoroughly investigated at this time. Some of these topics are described below:

The behavior of the containment under elevated temperature and pressure loads has not been thoroughly investigated. The containments tested in this program have ignored the effects of temperature on the material properties and thermal induced stresses associated with elevated temperatures.

The effect of aerosols within the containment atmosphere during an accident has not been investigated. The aerosols may plug holes in the containment that may lead to a higher pressure capability, but have the potential to change the mode of failure from a possible benign mode to a burst mode. This applies to unlined concrete containments and lined containments when the liner has failed.

Liner-anchorage-concrete interaction is significant in determining how liners tear in concrete containments. SNL is currently initiating a separate effects testing program that will investigate liner-anchorage-concrete interaction.

Seismic loadings coupled with severe accident loads is an area where its impact on containment performance has not been investigated in any detail.

Some other areas that effect the containment integrity have been looked at by other organizations. It is beyond the scope of this paper to cover these topics, but they are listed here with appropriate references:

The buckling of steel containment shells under many different combinations of pressure and membrane loadings has been investigated by Los Alamos National Laboratories and others. This topic is addressed in Refs. 48-53.

Impact of the containment by many different types of projectiles have been investigated by different organizations and individuals. Some of the work specifically addressed turbine and aircraft impact into containment structures.⁵⁴⁻⁵⁶ Refs. 57 and 58 provide information on other work related to impact of projectiles on structures.

5.2 Remaining Questions

Before applying the lessons learned from the concrete containment integrity research, a number of issues still need to be resolved. Although there has been much progress in developing analytical models for evaluating liner tearing, we feel that the following questions remain:

- Under what conditions will stud or other anchorage shear failure occur rather than liner tearing?
- How is the failure mode of the liner-anchorage system affected by scaling? For example, it could be affected by the ratio of liner thickness to stud diameter and by membrane loading of the liner (before the development of high stud shear forces).
- To what extent is the magnitude of liner strain concentrations affected by friction and bond between the concrete and liner? An example of this could be by differential radial motion at a major concrete crack, or by stud or anchorage spacing.
- How is the magnitude of liner strain concentrations and the resulting tear length and trajectory affected by the size and shape of insert plates? What is the best way to extend the current analysis techniques to predict tear areas?

Investigation and validation of methods for evaluating other potential failure modes is still needed. Shear failure is particularly difficult to evaluate; there is no generally recognized, reliable method of determining shear capacity of a reinforced concrete section under simultaneous application of tensile load and bending moment. Repressurization of the 1:6-scale model (after repairs are made to the liner) is one potential means for obtaining much needed data on the shear behavior of the cylinder-basemat intersection. Failure of large rebars where they are bent around penetrations, as occurred in an EPRI test,¹⁰ are also of some concern. The effects of cold working on the available ductility of these bars should be studied further. Rates of pressurization and effects of temperature must be considered in more detail. At high rates of pressurization, there is a possibility that sequential failure modes could occur; that is, for very high rates of pressurization, liner tearing may not arrest the pressure buildup within the containment, and another failure mode could occur at slightly higher pressure. This is the basic reason for emphasizing on the development and validation of analysis methods. A reliable evaluation of containment performance must be based on careful, detailed analysis of the specific containment geometry and loading of interest.

6. CONCLUSION

We have examined methods for predicting the performance of steel containments subject to severe accident loads, with emphasis on the finite element approach. Release paths and failure modes were discussed in some detail. Some of the more important aspects of modeling the nonlinear response characteristic of steel containments were highlighted by experiences from testing and analysis of scale models.

Steel containments have significant capacity beyond the design basis loads. With a few possible exceptions, we think it is unlikely that any significant leakage will arise before the onset of general yielding in the containment. Leakage from pressure seating equipment hatches and rupture may occur within a relatively narrow range of pressure. For equipment hatches with large radius-to-thickness ratios, or for containment shells that can develop large free-field membrane strains prior to rupture, or given both conditions, significant leakage is likely to precede and possibly preclude rupture. However, there is enough uncertainty in the calculations to suggest that probabilities for failure by each potential mechanism, such as leakage from pressure seating equipment hatches or rupture, be estimated for use in consequence analysis.

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