NUREG/CR-2549
SAND82-0324
RI, RD
Printed May 1982

Background Study and Preliminary Plans for a Program on the Safety Margins of Containments

Thomas E. Blejwas, Albert W. Dennis,
Ronald L. Woodfin, Walter A. Von Riesemann

Prepared by
Sandia National Laboratories
Albuquerque, New Mexico 87185 and Livermore, California 94550
for the United States Department of Energy
under Contract DE-AC04-76DP00789

Prepared for
U. S. NUCLEAR REGULATORY COMMISSION

NOTICE

Available from

GPO Sales Program
Division of Technical Information and Document Control
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

and

National Technical Information Service
Springfield, Virginia 22161

NUREG/CR-2549
SAND82-0324
RI, RD
Printed May 1982

Background Study and Preliminary Plans for a Program on the Safety Margins of Containments

Thomas E. Blejwas, Albert W. Dennis,
Ronald L. Woodfin, Walter A. Von Riesemann

Date Published: May 1982

Sandia National Laboratories
Albuquerque, New Mexico 87185
operated by
Sandia Corporation
for the
US Department of Energy

Prepared for
Mechanical/Structural Engineering Branch
Division of Engineering Technology
Office of Nuclear Regulatory Research
US Nuclear Regulatory Commission
Washington, DC 20555
Under Memorandum of Understanding DOE 40-550-757
NRC FIN No. A1249

Acknowledgment

Portions of the material presented in this report were provided by Southwest Research Institute, San Antonio, Texas, under contract to Sandia National Laboratories.

Executive Summary

This report describes a background study and preliminary plans for a program to develop methodology for predicting the capacity of containment structures under severe accident and environmental conditions. The work reported here was completed and presented to representatives of the NRC and an advisory panel in March 1981. Refinements and modifications in the program after March of 1981 will be reported in future documents.

Both analytical and experimental efforts were considered in the background study. It is concluded that the end results of the program should be

- Bench-mark data from scale-model tests on selected classes of containments, and
- A set of qualified computer programs that can be used to determine the ultimate capacity of steel, reinforced concrete, and prestressed concrete containments subjected to internal pressurization and seismic loadings.

Containment structures are designed, using elastic methods, to withstand design basis accidents without any loss of function. The ultimate capacity of containment structures is generally greater than the design level. The difference between design and ultimate capacity represents a margin of safety for which no reliable estimate is presently available. However, the margin of safety is available for protection against the effects of accidents beyond the design basis (e.g., degraded core accidents like TMI-2). This program will establish analysis procedures that may be used to predict, with known confidence, the ultimate capacity of a containment and thus allow the information to be integrated into reactor-risk analysis and accident-response planning.

Loadings in excess of containment design loading originate from two sources. First, the original design criterion may be rendered obsolete by new knowledge. For example, a geologic fault indicating greater seismic risk may be located in the vicinity of the plant and the safe-shutdown earthquake criterion may then be revised. Second, a loading condition that was not considered during the design process or that was deemed too improbable to warrant integration into the design conditions may be introduced. Examples of such loadings are internal static pressure generated by a hydrogen deflagration, a molten-core quenching event, or a core-concrete interaction event. Internal dynamic pressures caused by hydrogen detonation are also an example of this second type of loading.

Containment structures are designed to meet the requirements of each particular plant and are not standardized. Therefore, it is necessary to qualify analytical methods that may then be used to determine the ultimate load capacity of the containments. The accepted procedure for qualification of an analytical method is to experimentally establish the behavior and failure modes of representative structures when they are subjected to specified loadings; then, to demonstrate that the analytical method is able to predict (with the accuracy desired) the behavior, failure mode, and ultimate loading of the structures. A literature search of containment structural testing indicates this has been done only once. A 1/14-scale nonreplica model of a prestressed concrete CANDU containment was pressurized to failure. The measured responses were in agreement with those predicted using the BOSOR5 computer code. In this case the containment model was axisymmetric except for prestressing buttresses; that is, there were no penetrations in the containment's cylinder or dome.

In addition to the test of a CANDU containment, a limited amount of ultimate-load testing of reactor containments that did not include analytical correlation has been conducted. Of the four scale-model tests surveyed, all were static internal-pressure tests and all models were of concrete containments. Two were conducted in Japan, one in India, and one in Poland. None of the published results indicate an attempt at analytical correlation.

Investigations indicate that three loading conditions (internal static pressure, internal dynamic pressure, and seismic) and three types of containment construction (welded steel, reinforced concrete, and prestressed concrete) merit experimental investigation. The use of replica scale models was investigated and found suitable for the qualification of analytical methods. Ultimate load testing of models subjected to internal static pressure presents no problem at this time; however, experimental methods will have to be developed for both internal dynamic pressure and seismic loading.

Multiple testing at several scales is required to establish the repeatability and credibility of the experimental program. This demonstration of repeatability and replication of results at different scales is lacking in previous work.

Contents

1. Introduction 9
   1.1 Background 9
   1.2 Objective 10
   1.3 Program Scope 10
   1.4 Structure of the Report 10
   References 11
2. Survey of Containments 13
   2.1 Containment Types 13
      2.1.1 Introduction 13
      2.1.2 Structural Configuration and Construction 13
      2.1.3 Containment Structure Design Pressure 14
      2.1.4 Seismic Design Parameters 16
      2.1.5 Summary 16
   2.2 Design Procedures 16
   2.3 Review of Previous Tests 17
      2.3.1 Model Tests 17
      2.3.2 Tests of Actual Containments 18
      2.3.3 Conclusions From Previous Tests 19
   References 19
3. Problem Definition 21
   3.1 Loading Conditions 21
   3.2 Assumptions and Exclusions 21
4. Analytical Methods 23
   4.1 Introduction and General Comments 23
   4.2 Steel Containments 23
   4.3 Concrete Containments 24
   References 25
5. Containment Modeling 27
   5.1 Introduction 27
   5.2 Model Scaling Laws 27
      5.2.1 Replica Model Analysis 27
      5.2.2 Inspection of Pi Terms 27
      5.2.3 Nonreplica Model Analysis 31
      5.2.4 Summary 32
   5.3 Identification of Critical Structural Elements 32
      5.3.1 Introduction 32
      5.3.2 Identification of Critical Structural Elements 33
      5.3.3 Adequate and Replica Models 33
      5.3.4 Relationship Between Failure Modes and Scale Factors 37
   5.4 Credibility of Test Results 37
      5.4.1 Introduction 37
      5.4.2 Required Number of Tests 37
   References 38

Contents (cont):

6. Experimental Alternatives ... 39
  6.1 Introduction ... 39
  6.2 Experimental Techniques ... 39
    6.2.1 Static Pressure Loading ... 39
    6.2.2 Dynamic Pressure Loading ... 39
    6.2.3 Seismic Loading ... 40
    6.2.3.1 Seismic Input Definition ... 41
    6.2.3.2 Seismic Loading Devices ... 41
  6.3 Survey of Test Facilities ... 42
    6.3.1 Pressurization Facilities ... 42
    6.3.2 Seismic Facilities ... 42
  6.4 Instrumentation ... 44
  References ... 44
7. Summary and Preliminary Plans ... 45
APPENDIX A—Reactor Containment Structures ... 47
APPENDIX B—Full-Size Containment-Like Structures ... 53
APPENDIX C—Estimates of Model Fabrication Costs ... 55

Figures

2.1-1 Containment Structural Configurations ... 13
6.2-1 Test Facility for Static Pressurization ... 40
6.2-2 Pulse Calibration Chamber ... 40

Tables

2.1-1 Containment Structural Configuration and Construction Materials ... 14
2.1-2 Summary of Operating US Power Reactor Containment Structures ... 14
2.1-3 Summary of Future US Power Reactor Containment Structures ... 15
2.1-4 Summary of Operating and Future US Power Reactor Containment Structures ... 15
2.1-5 Near-Future Commercial Reactors ... 15
2.1-6 Design Pressures ... 15
5.2-1 List of Parameters ... 28
5.2-2 List of Pi Terms ... 29
5.2-3 Replica Modeling Law to Satisfy Pi Terms ... 30
5.2-4 General Modeling Law to Satisfy Pi Terms ... 31
5.2-5 Major Pi Terms for Pressurization Loading ... 32
5.2-6 Major Pi Terms for Seismic Excitation ... 32
5.3-1 Failure Modes and Critical Components for Free-Standing Steel Containments ... 34
5.3-2 Failure Modes and Critical Components for Reinforced Concrete Containments ... 35
5.3-3 Failure Modes and Critical Components for Prestressed Concrete Containments ... 36
6.3-1 Characteristics of Seismic Shaker Test Facilities ... 43
6.3-2 Minimum Input Requirements to Replicate a 1.0-g Horizontal Response Spectrum ... 44

Background Study and Preliminary Plans for a Program on the Safety Margins of Containments

1. Introduction

This report describes a background study completed in March 1981. Preliminary program plans to determine the capacity of containment structures under severe accident and environmental conditions are presented. The study considered both analytical and experimental efforts. Proposed work includes testing models of containment structures to failure. Results from these tests will be compared with those obtained using analytical methods (including computer codes) in order to determine the extent to which these methods can predict ultimate failure levels and modes. Computer code improvements will be made as necessary.

1.1 Background

The function of the containment system is to prevent the escape to the atmosphere of fission products that may be released from the reactor vessel. The containment structure is designed to prevent leakage for a variety of environment- and accident-loading conditions. Design procedures are generally based upon linear-elastic theory with factors of safety incorporated at various stages in the design process. Although the design procedures appear adequate to ensure safe operation for design loadings, extending these procedures to predict ultimate capacity is not feasible. However, loadings far more severe than design loadings can be postulated. For example, hydrogen deflagrations/detonations, which recently have received considerable attention because of the accident at Three Mile Island, may cause structural loadings in some containments that are higher than design loadings. Also, the magnitude of extreme environments (specifically earthquake loadings) cannot be predicted with confidence. If the potential risk associated with these extreme environments and accident conditions is to be assessed, knowledge of the ultimate capacity of the containment structure is essential.

Although a number of computer codes are available for analyzing the nonlinear, inelastic behavior of steel structures, the application of these programs to the prediction of the ultimate capacity of containment structures has limitations. In general, the existing codes have not been qualified by comparision with tests-to-failure of containment-like structures. No ultimate failure mechanism for biaxial and triaxial stress is universally accepted. Also, steel containments include a number of features (such as stiffeners and penetrations) that may have a significant effect upon the overall structural behavior in the inelastic range. Only by obtaining experimental information can proper judgments be made about how to apply existing computational methods to the configurations of interest.

The analytical modeling of the ultimate behavior of concrete containments with high confidence is not possible at present and may not be practical in the near future. In addition to difficulties similar to those for steel containments, some phenomena in the response of reinforced/prestressed concrete containments are not well enough understood to be modeled with confidence. Modeling using simplified approaches such as layering of concrete and steel is possible, but the adequacy of these methods for handling large-scale structures with realistic design features has not been established.

Tests around the world of the ultimate capacity of containment models have been limited. The large amplitude, nonlinear response of steel containments has not been investigated experimentally. Pressure tests of models of concrete containments indicate that the models behave, in a gross sense, as predicted. However, the only reported program that combined extensive analytical effort with testing[1] used a concrete shell with butresses (i.e., penetrations and other details were not included). Tests by others[2,3] indicate that large penetrations have a significant effect upon containment response and initial leakage. A steel liner, which is required for all concrete containments in the US, was included in only one of the model tests;[2] other types of liners or the lack of a liner caused problems in many tests. Finally, none of the reported model test programs included comparisons of results with tests at different scales.

In conclusion, although containments can be expected to function adequately for design conditions, the ultimate capacity cannot be predicted with confidence. In general, previous testing has not been directed at qualifying the complex nonlinear computer codes required for predicting the ultimate capacity. A program of combined experimental and analytical efforts is required.

1.2 Objective

The overall objective of this program is to develop methodology for reliably predicting the ultimate capacity of steel and concrete containment structures under loadings caused by accidents and severe environments. The proposed approach is to test models of containment structures to failure. The suitability of existing analytical methods for predicting ultimate-failure levels will be determined by comparisons with the test results. Improvements to computer codes will be made as necessary.

The specific loading conditions to be considered, in order of anticipated investigation, are

- Static-internal pressurization
- Dynamic-internal pressurization
- Seismic loadings

A survey of US operating and future plants (180 total) showed a large variety of containment types. The following three types, encompassing 75% of the containments, will be considered:

- Hybrid steel (ice condenser and MK-III)
- Reinforced concrete
- Prestressed concrete

Because it is impractical to test full-size containment structures, scale models will be used. Several requirements are necessary to achieve credible results. Each of the items below is an integral part of this program.

- A sound theoretical basis for the model design, the test loading, and interpretation of test data must be established.
- The model scale must be chosen such that the characteristics to be determined in the test are accurately represented in the model.
- The scale model must be fabricated with sufficient care to insure that failure mechanisms not present in the prototype are not introduced into the model.
- The test methods must be such that failure modes are not introduced or eliminated in the structure as a result of test procedures or test facilities.
- Instrumentation of sufficient accuracy and sensitivity to record all phenomena of interest must be employed.
- Adequate analytical support must be incorporated into all phases of the experimental plan.
- Repeatability of experimental results must be demonstrated.

Because the ultimate capacity and modes of failure of steel or concrete containments cannot be predicted with confidence using existing computer codes, an analytical task that parallels the experimental effort will be undertaken. Qualification of existing codes will be attempted; modifications of existing codes and development of new codes may be required.

The end results of this program will be:

- Bench-mark data from scale-model tests of selected classes of containments.
- A set of qualified computer programs that can be used to determine the ultimate capacity of containment structures.

1.3 Program Scope

The overall objectives of this program are extensive and will require several years to accomplish. The static pressurization of free-standing steel containments will constitute the first phase of the program. Detailed plans for this phase and later phases of the program will be reported prior to their initiation.

1.4 Structure of the Report

Section 2 contains a listing of containments for operating and future US nuclear power plants, design procedures used for containments, and a review of previous failure tests of pressurized containments. The containments (Section 2.1) are categorized by type of plant and method of construction (steel and reinforced and prestressed concrete with a liner). Design pressures are also given.

The three types of loading that will be considered in this program (static and dynamic pressurization and seismic) are discussed in Section 3. Although the testing program will not consider the combination of an earthquake load with internal pressurization, analytical efforts will be directed to this possibility.

Existing methods of analysis for concrete and steel containments are reviewed in Section 4. Although analysis of the failure level of steel containments is in a better state than that for concrete, modeling with confidence is not possible at present.

Requirements for models to be used in the experimental effort are described in Section 5. Scaling laws and the effect of using different scale factors are discussed. The requirements for obtaining credible results are presented.

In Section 6, experimental loading techniques and facilities are discussed. Some static pressurization tests can be conducted with existing Sandia facilities. Modest new facilities are required for static tests of large-scale models. Dynamic pressurization and seismic loadings require additional developmental work.

Chapter 7 contains a brief summary and preliminary plans for the program. Detailed plans for each phase of the program will be formulated as the program progresses.

Three appendices are included. Appendix A contains details of the survey of existing and planned nuclear containment structures. Results of a brief search to identify full-size, containment-like structures that might be suitable and available for use as test specimens are presented in Appendix B. The costs of fabrication of steel, reinforced concrete, and prestressed concrete containment models at scales from 1/50 to 1/4 are presented in Appendix C.

References

[1]R. J. Atchison, G. J. K. Asmis, and F. R. Campbell, "Behavior of Concrete Containment Under Overpressure Conditions," *Transactions of the 5th SMiRT*, Paper No. J3/2, Berlin, August 1979.

[2]K. Donten et al, "Results of Strength Tests on a 1:10 Model of Reactor Containment," *Transactions of the 5th SMiRT*, Paper J4/8, Berlin, 1979.

[3]T.V.S.R. Appa Rao, "Behaviour of Concrete Nuclear Containment Structures Up to Ultimate Failure With Special Reference to MAPP-1 Containment," *Inelastic Behavior*, Report 4-SM-THEME/75, Structural Engineering Research Centre, Madros, India, 1975.

2. Survey of Containments

2.1 Containment Types

2.1.1 Introduction

In order to ascertain the different types of containments in use and proposed for use, a survey of US Light Water Reactor (LWR) containment structures was conducted. This information will be used as input in determining the priorities of this program. A discussion of containment types, such as single and double barrier, is not given herein. The interested reader is directed to a recent survey article.[1]

As of June 30, 1980 there were 69 LWRs licensed for commercial operation in the US.[2] There are an additional 116 land-based commercial LWR containments docketed with the NRC but not licensed for commercial operations as of June 30, 1980.[3] Of the 116 proposed containments, information was obtained on 112 and this information is presented in Appendix A. The four containments for which information was not available are Douglas Point 1, Haven 1, and NYSE&G 1 and 2. Containments are catalogued in Appendix A by reactor type, pressure-suppression system, principal construction material, and structural configuration.

2.1.2 Structural Configuration and Construction

There are six major structural configurations (without regard for type of construction material) used by the 181 containments listed in Appendix A.

- A vertical cylinder with a hemispherical dome and flat base; 69 units or 38% use this configuration, Figure 2.1-1 (a).
- A vertical cylinder with a shallow (usually ellipsoidal or torospherical) dome and flat base; 50 units or 28% use this configuration, Figure 2.1-1 (b).
- A "light-bulb"/torus (MARK I); 25 units or 14% use this configuration, Figure 2.1-1 (c).
- A sphere; 13 units or 7% use this configuration, Figure 2.1-1 (d).
- A vertical cylinder with hemispherical dome and ellipsoidal base; 12 units or 7% use this configuration, Figure 2.1-1 (e).
- A truncated cone with or without a vertical cylinder and flat base (MARK II); 11 units or 6% use this configuration, Figure 2.1-1 (f).

This list accounts for 180 of the 181 containments. The remaining containment, Humbolt Bay, is an early design.

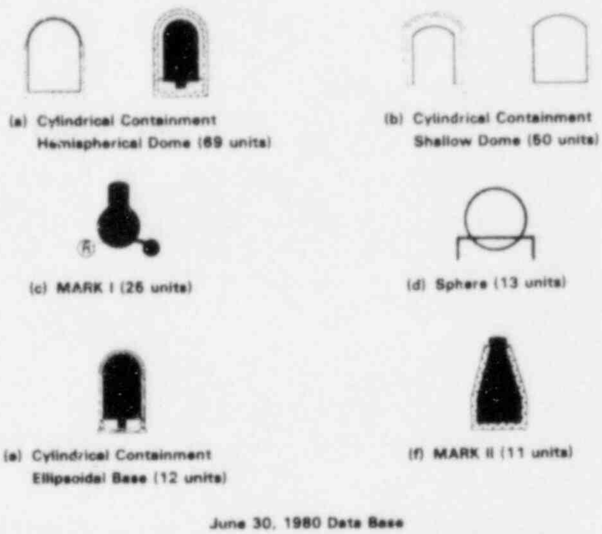

Figure 2.1-1. Containment Structural Configurations

The containment structures are fabricated of steel and/or concrete. The construction can be classified into four major categories:

a. Prestressed reinforced concrete body with a deformed-bar reinforced concrete base; 59 units or 33% use this construction.

b. Reinforced concrete*; 47 units or 26% use this construction.

---

*The term "reinforced concrete" is used to identify conventional deformed-bar reinforced concrete.

c. Welded steel; 49 units or 26% use this construction.

d. Welded steel body with reinforced concrete base; 23 units or 13% use this construction.

There are 178 units in the above list. The three units not included are Humbolt Bay, Ginna, and Robinson 2. Humbolt Bay has a welded steel dry well and deformed-bar reinforced concrete wet well. Ginna and Robinson 2 are constructed of reinforced concrete with vertical prestressed reinforcement and deformed-bar hoop reinforcement.

Table 2.1-1 contains a summary of the reactor containment structures by configuration and construction materials. Tables 2.1-2, 2.1-3, and 2.1-4 contain the same information with the addition of reactor type (BWR or PWR), pressure-suppression system, and modified configuration descriptions. Table 2.1-2 is for reactors that have operating licenses; Table 2.1-3 is for reactors that are docketed, but did not have operating licenses as of June 30, 1980. One plant (Washington 2) is omitted from Table 2.1-3 because its containment structure is unique. It is the only MARK II plant with a steel containment. Table 2.1-4 contains a composite of the information in Tables 2.1-2 and 2.1-3.

Twenty-two power reactors are expected to be placed in commercial operation between June 30, 1980 and December 31, 1982.[4] Fifteen of these reactors are PWRs and the remaining seven are BWRs. The reactors and their expected operational dates are listed in Table 2.1-5. This information was abstracted from Appendix A.

2.1.3 Containment Structure Design Pressure

The containment structure design pressure is based on a severe (Class 8) accident, usually a loss of coolant accident (LOCA), the pressure suppression system (if any), and the containment structure free volume. The design pressures for 48 containment structures were obtained from a Chicago Bridge and Iron Company publication;[5] 57 containment structure design pressures were obtained from Preliminary and Final Safety Analysis Reports; 13 were obtained from Reference 6 and an additional 8 design pressures from Reference 7 for a total of 126 out of the 181 containments listed in Appendix A. There are some discrepancies between the design pressures stated for the same containments in the references; however, it is conceivable that in some cases the conflicting references stated cold test or calculated maximum pressures rather than design pressure. The design pressure information is presented in Table 2.1-6.

Table 2.1.1. Containment Structural Configuration and Construction Materials

JUNE 30,1980 DATA BASE, Sandia National Laboratories	PRESTRESSED CONCRETE BODY WITH REINFORCED CONCRETE BASE	REINFORCED CONCRETE	STEEL	STEEL BODY WITH REINFORCED CONCRETE BASE	OTHER
HEMISPHERICAL DOME CYLINDRICAL BODY FLAT BASE	20	39		8	2
SHALLOW DOME CYLINDRICAL BODY FLAT BASE	35			15	
MARK I LIGHT BULB/TORUS		2	23		
SPHERE			13		
HEMISPHERICAL DOME CYLINDRICAL BODY ELLIPSOIDAL BASE			12		
MARK II TRUNCATED CONE	4	6	1		
OTHER					1

Table 2.1.2. Summary of Operating US Power Reactor Containment Structures

JUNE 30, 1980 DATA BASE, Sandia National Laboratories			CONTAINMENT STRUCTURES: CONCRETE			CONTAINMENT STRUCTURES: STEEL						
			PRESTRESSED VERTICAL CYL. W DOME WITH FLAT BASE	DEFORMED BAR HEMISPHERICAL DOME, VERT. CYL. BODY AND FLAT BASE	OTHER CONCRETE	LIGHT BULB TORUS CONTAINMENT	STEEL SPHERE	HEMISPHERICAL DOME, CYL. BODY AND ELLIPSOIDAL BASE	HYBRID STEEL STEEL DOME AND BODY REINFORCED CONCRETE BASE	SUB TOTAL	SUB TOTAL	TOTAL
COMMERCIAL U.S. POWER REACTORS	PWR'S	ATMOSPHERIC CONTAINMENT STRUCTURES WITHOUT PRESSURE SUPPRESSION SYSTEMS	22	6	2		3	5		37	43	69
		SUBATMOSPHERIC CONTAINMENTS		4						4		
		ICE-CONDENSER CONTAINMENTS		2						2		
	BWR'S	MARK I			2	20				22	26	
		MARK II										
		MARK III										
		PRE MARK			2		2			4		
		SUB-TOTAL	22	11	6	20	5	5				
		SUB-TOTAL	39			30						
		TOTAL	69									

Table 2.1.3. Summary of Future US Power Reactor Containment Structures

JUNE 30, 1980 DATA BASE

Sandia National Laboratories

			CONTAINMENT STRUCTURES									
			CONCRETE			STEEL						
			PRE-STRESSED VERTICAL CYL. & DOME WITH FLAT BASE	DEFORMED BAR HEMISPHERICAL DOME VERT CYL BODY AND FLAT BASE	OTHER CONCRETE	LIGHT BULB TORUS CONTAINMENT	STEEL SPHERE	HEMISPHERICAL DOME CYL BODY AND ELLIPSOIDAL BASE	HYBRID STEEL STEEL DOME AND BODY REINFORCED CONCRETE BASE	SUB-TOTAL	SUB-TOTAL	TOTAL
COMMERCIAL U.S. POWER REACTORS	PWR'S	ATMOSPHERIC CONTAINMENT STRUCTURES WITHOUT PRESSURE SUPPRESSION SYSTEMS	33	15			8	6		62	77	111
		SUBATMOSPHERIC CONTAINMENTS*		7						7		
		ICE-CONDENSER CONTAINMENTS							8	8		
	BWR'S	MARK I				3					34	
		MARK II			10					10		
		MARK III		6					15	21		
		PRE-MARK										
		SUB-TOTAL	33	28	10	3	8	6	23			
		SUB-TOTAL	71			40						
		TOTAL	111									

*WASHINGTON 2 IS NOT INCLUDED IN THE ABOVE TABLE

Table 2.1.4. Summary of Operating and Future US Power Reactor Containment Structures

JUNE 30, 1980 DATA BASE

Sandia National Laboratories

			CONTAINMENT STRUCTURES									
			CONCRETE			STEEL						
			PRE-STRESSED VERTICAL CYL. & DOME WITH FLAT BASE	DEFORMED BAR HEMISPHERICAL DOME VERT CYL BODY AND FLAT BASE	OTHER CONCRETE	LIGHT BULB TORUS CONTAINMENT	STEEL SPHERE	HEMISPHERICAL DOME CYL BODY AND ELLIPSOIDAL BASE	HYBRID STEEL-STEEL DOME AND BODY REINFORCED CONCRETE BASE	SUB-TOTAL	SUB-TOTAL	TOTAL
COMMERCIAL U.S. POWER REACTORS	PWR'S	ATMOSPHERIC CONTAINMENT STRUCTURES WITHOUT PRESSURE SUPPRESSION SYSTEMS	55	20	2		11	11		99	120	180
		SUB-ATMOSPHERIC CONTAINMENTS		11						11		
		ICE-CONDENSER CONTAINMENTS		2					8	10		
	BWR'S	MARK I			2	23				25	60	
		MARK II			10					10		
		MARK III		6					15	21		
		PRE-MARK			2		2		*	4		
		SUB-TOTAL	55	39	16	23	13	11	23			
		SUB-TOTAL	110			70						
		TOTAL	180									

*WASHINGTON 2 IS NOT INCLUDED IN THE ABOVE TABLE

Table 2.1-5. Near-Future Commercial Reactors

Plant Name	Pressure Suppression System	Expected Operation
Fermi 2	MARK I	1982
Susquehanna 1	MARK II	1982
La Salle 1 & 2	MARK II	1981/1982
Zimmer	MARK II	1981
Clinton 1	MARK III	1982
Grand Gulf 1	MARK III	1982
Sequoyah 1 & 2	Ice Condenser	1980/1981
McGuire 1 & 2	Ice Condenser	1981/1982
Watts Bar 1 & 2	Ice Condenser	1981/1982
Commanche Peak 1	None	1981
Diablo Canyon 1 & 2	None	1981
Salem 2	None	1981
Calloway 1	None	1982
Summer	None	1981
Farley 2	None	1981
San Onofre 2	None	1981
Waterford 3	None	1982

Table 2.1.6. Design Pressures

JUNE 30, 1980 DATA BASE

Sandia National Laboratories

			CONTAINMENT STRUCTURES						
			CONCRETE			STEEL			
			PRE-STRESSED VERTICAL CYL. & DOME WITH FLAT BASE	DEFORMED BAR HEMISPHERICAL DOME VERT CYL BODY AND FLAT BASE	OTHER CONCRETE	LIGHT BULB TORUS CONTAINMENT	STEEL SPHERE	HEMISPHERICAL DOME CYL BODY AND ELLIPSOIDAL BASE	HYBRID STEEL STEEL DOME AND BODY REINFORCED CONCRETE BASE
COMMERCIAL U.S. POWER REACTORS	PWR'S	ATMOSPHERIC CONTAINMENT STRUCTURES WITHOUT PRESSURE SUPPRESSION SYSTEMS	47-60	42-55	42-60		25-46*	34-44*	
		SUB-ATMOSPHERIC CONTAINMENTS		45					
		ICE-CONDENSER CONTAINMENTS		12					11-15*
	BWR'S	MARK I			56	56-62			
		MARK II			45-56				
		MARK III		15					15
		PRE-MARK			52		27-30*		

* DESIGN PRESSURE ROUNDED TO WHOLE NUMBERS.

2.1.4 Seismic Design Parameters

The major seismic design parameters for currently licensed power plants (as of May 1980) are given in Reference 8. Information for proposed plants will be obtained later if required.

2.1.5 Summary

There are 181 commercial nuclear-power reactors listed in References 2 and 3 as operating or planned. These reactors are cataloged in Appendix A by containment structure type. The expected operational date of the power reactors (other than those already in operation) is also listed. Twenty-one of 112 proposed plants have been cancelled or have an indefinite starting date. The elimination of these plants from the data set would not change the priorities of the program.

2.2 Design Procedures

The purpose and goals of this program are better understood if there is a general understanding of the procedures used in the design of containment structures. Because of the number of different types of containment structures and loading conditions, several design procedures are used. However, certain generalities in the design process are evident; some of the general design procedures will be reviewed in this section. The inappropriateness of extrapolating design procedures to predict ultimate capacity will also be discussed.

Containment vessels in the US are designed to satisfy the criteria in Section III of the ASME Pressure Vessel Code.[9,10] The NRC's Standard Review Plan[11] references the ASME code but also modifies some criteria and references other NRC regulation documents. During the time from preliminary design to operation of a nuclear power plant, the ASME codes and NRC regulations may change several times. For example, the 1963 to 1971 editions of the ASME code specify the design pressure as no less than 90% of the maximum containment internal pressure; however, in subsequent editions, the design pressure is specified as 100% of the maximum pressure (certain allowable stresses were also raised by 10%). Thus, the reviewer must know which edition of the code was used in order to properly interpret the design procedure.

A review of the 1980 edition of the ASME Code reveals that the analysis of containments (both steel and concrete) is based generally upon elastic (usually assumed linear) behavior; thin-shell theory; and membrane response except at discontinuities and penetrations. Although some provisions are made for the plastic analysis of steel containments (Section NE-3228), elastic analyses are emphasized; in fact, elastic analyses are allowed even when the calculated stresses exceed the yield strength of the material (Section NE-3227.6). For concrete containments, Section CC-3310 states that consideration must be given to the redistribution of forces caused by concrete cracking, yet Section CC-3320 states: "Elastic behavior shall be the accepted basis for predicting internal forces, displacements, and stability of thin shells." It is reasonable to expect designers to estimate a level of concrete cracking and perform a linear analysis.

The *steel containment code* (subsection NE) uses allowable/working-stress concepts as opposed to load factor-strength design. The absence of factored loads for steel containments is a notable feature of this code. Although the Standard Review Plan specifies load combinations, all of the combinatorial factors are one or zero. Thus, no explicit safety factor is included in loading conditions, although certain unlikely load combinations may necessitate a design that has a high margin of safety for other combinations. The specification of allowable stresses controls more directly the margin of safety. The allowable stress is a function of the type of stress (primary, secondary, or peak) and the type of loading (design or service limits A, B, C, or D). It is worthy of note that, for primary plus secondary stresses due to service limit A or B loadings, the allowable stress is three times the stress intensity S_{ml}; for many materials, 3 S_{ml} is equal to the specified ultimate stress capacity of the material. Therefore, although elastic analyses are suggested, the code relies upon a redistribution of stresses caused by material yielding at discontinuities and penetrations (i.e., a shakedown to elastic behavior). As an alternative procedure for penetrations, the area replacement rule (NE-3332.2) eliminates generally the necessity for analysis by attempting to insure that sufficient additional material is added around a penetration so that a general yield state will not occur prematurely; however, it is clear that local yielding will occur for some design conditions.

The *concrete containment code* (subsection CC, Division 2, of Section III) which is a joint ASME-ACI effort, combines service and factored loads with allowable stresses. The specified load combinations (Table CC-3230-1) contain combinatorial factors for service loads that are all one or zero, but for factored loads the combinatorial coefficients are as high as 1.5. Thus, load-combination factors of safety are included explicitly for some loadings and not for others. This is a departure from the usual ACI philosophy where even dead loads have a load factor of 1.4. Additional factors are included in the specification of allowable tresses. Values of compressive strength of concrete vary from 0.3 f_c to 0.85 f_c, depending upon type of force and loading (see Tables CC-3421-1 and CC-3431-1). For steel reinforcement, the maximum allowable stress is 0.9 f_y for factored loads, but local yielding is allowed under certain conditions even for primary forces (section CC-3422). Lower allowables stresses are specified for service loads.

Although properly implemented design procedures are likely to result in a containment structure that functions satisfactorily for design conditions, they do not insure predictable behavior for loadings of higher magnitude or different characterization. The codes are aimed at achieving ductile behavior up to design limits but, as the yield capacity of materials is exceeded, the design procedures generally become invalid.

For overpressure loading of steel containments, areas near structural discontinuities and penetrations will yield before other areas. Although the yielding is generally self-limiting for a given level of loading, strains in the area of discontinuities will be higher than in other areas as pressure is increased. Whether these strains will be significantly higher and cause rupture before a general yield condition of the containment is reached is not evident. In some containments there is no thickening around penetrations because the shell wall is much thicker than required for design pressure alone. Although these containments will function adequately for pressures up to design pressure, a possibility exists that the area around the penetration will fail before a general yield state of the shell is reached.

For internal overpressurization of concrete containments, material behavior and interaction of the steel and concrete provide analytical difficulties as pressures are raised above the design levels. In particular, the ultimate capacity of reinforcement-concrete bonding, the capacity of concrete in biaxial and triaxial stress, or the interaction of the concrete and steel liner are too complex to allow the prediction of the ultimate functional capacity with certainty. Significant leakage of the containment may occur at pressures well below those predicted to cause the steel reinforcement to reach its ultimate capacity.

For earthquake loadings, additional analytical difficulties are present because of the necessity of determining the dynamic response of the containment. The accepted dynamic model for design has been a linear lumped-mass beam model in which the beam stiffness is primarily a function of the tangential shear stiffness of the shell. Nonlinearity/cracking of concrete is not treated explicitly, although some level of concrete cracking is assumed in the determination of stiffness. By placing a very low limit on allowable shear stress of concrete, the NRC has insured that a concrete containment is not likely to fail during a safe-shutdown earthquake, despite the simple modeling techniques. However, it should be emphasized that the design analyses for earthquakes are not accurate (the uncertainty of future ground motions makes the use of more elaborate procedures questionable). Therefore, extending the current design procedure to predict ultimate capacity is not feasible.

Although the procedures used in designing containment structures appear to be adequate for insuring safe operation for design loadings, they provide little insight into the margin of safety. The response of containments to loadings well above design is too complex to be predicted with current design procedures. The most important point to be made is that a consistent pattern of built-in conservatism does not exist in the present codes.

2.3 Review of Previous Tests

Tests of containment structures around the world have been limited in numbers, but some lessons can be learned from a brief review of the more significant tests. Although tests of actual full-size containments have been conducted, only models have been tested to levels of response significantly above the design conditions. Generally, model test programs have consisted of one test or a short series of tests with rather limited objectives. In particular, testing for overpressurization has been a significant part of many of the test programs. As will become evident, only models of reinforced and prestressed concrete containment structures have received significant attention.

2.3.1 Model Tests

A Canadian test program to study overpressure conditions[12] has been coordinated with analytical development.[13] The test structure was a 1/14 scale, prestressed concrete model of a CANDU containment structure. However, the model was not a replica of the prototype. In particular, no penetrations were included in the cylinder or spherical dome; the walls were thicker than replica scale (5 in. vs 3 in.) and the ratio of dome to cylinder thicknesses (4.0/5.0) was higher than replica scale (1.71/3.0); also, reinforcement and prestressing were not replicated. However, the model was not axisymmetric; i.e., prestressing buttresses were modeled. Also, the model did include a ring beam at the cylinder-dome junction and plastic liner, as is present in Canadian containment structures. Pressurization of the model was achieved hydraulically with water. After several liner failures and accompanying leakage at about 80 psig, the final test with a strengthened liner produced initiation of tendon failure at 142 psig and a large through-the-wall blow out at 159 psig. The measured response agrees very well with results from a modified version of the axisymmetric code BOSOR5.[14]

Japanese tests for overpressure conditions[15] were not replicas of an actual containment structure. The reinforced concrete models, which contained no penetrations in the walls, had an outer diameter of 2.4 m (94.5 in.) and a wall thickness in both the cylinder and dome of 0.10 m (3.94 in.). Although the diameter corresponds to a scale of about 1/17, the walls were thicker than would be present in a similar US containment structure. The construction consisted of micro concrete with vertical and horizontal deformed-bar reinforcements of 6 mm (0.24 in.) dia and ties between the inner and outer layers. Water pressurization was used and a neoprene rubber liner prevented leakage through cracks. Two models were tested: one at room temperature throughout, and one with a thermal gradient of 40° to 50°C across the wall. Results at yield and ultimate were very similiar for the two tests. The structures were designed for a pressure of 4 kg/cm² (60 psig). Yielding first occurred at about 90% of the value predicted by an elastic analysis. Ultimate capacity was reached when a punching shear failure occurred in the dome at about 9 kg/cm² (130 psig).

Tests of the lateral load capacity of reinforced concrete models of about 1/25 scale have also been conducted by the Japanese.[16] Only the cylindrical portion of the containment with no penetrations was tested. Again, the wall thickness was larger than would be scale for a US containment.

In India a 1/12-scale replica model of a prestressed containment has been loaded by overpressurization.[17] The model was constructed of micro concrete, annealed-steel wires for reinforcement, and prestressing wires for tendons. The stress-strain relationships for all materials closely approximated those of the prototype structure. As used on the prototype, vinyl paint was applied on the inside for leak-tightness. The six largest penetrations were simulated in the model. Pressurization was achieved with air. At a pressure of 19.5 psig, significant leakage occurred through cracks in the dome. Other than concrete cracking, structural failure of the containment could not be achieved because of leakage and equipment limitations. Interestingly, cracking occurred at a pressure below that predicted for the prototype (25 psig). Penetrations and other discontinuities greatly affected local deformations and had a noticeable effect on the overall response of the structure. Nonlinear material behavior was important at low levels of loading.

The largest model test to date was a 1/10-scale test conducted in Poland.[18] The model was a prestressed concrete replica constructed of the same materials used in the prototype. Details in the model included prestressed vertical and hoop tendons, buttresses, ring girder, equipment hatch, and personnel locks (including a thickened shell around the openings). Significantly, the model included a 1-mm (0.04-in.) thick mild-steel liner connected to the concrete with wire anchors. The ultimate capacity of the model, based upon limit strength of the tendons and yield limit for the reinforcing bar was 0.49 MPa (71 psig) or 0.62 MPa (90 psig) with the liner. Elastic tests were conducted with pressurized air while water was used in the destructive tests. Failure of the model occurred at a pressure of 0.52 MPa (75 psig) at the top and 0.58 MPa (84 psig) at the bottom; the difference is due to the head created by the water. Leakage at these pressures occurred because of bending near the buttresses and near the ends of extra reinforcement around penetrations. It is not clear whether or not the leakage was associated with fabrication of the very thin liner.

2.3.2 Tests of Actual Containments

All containments in the US are tested for strength by application of a small overpressure loading (presently 115% of design pressure) and for leakage at design pressure. In addition to verifying the function of the containment for design pressure, the tests provide data for verification of linear-elastic computer analyses. Unfortunately, the tests provide little insight into the large deformation, nonlinear response of the containment. As noted in Section 2.2, extrapolating linear response to obtain ultimate capacity is not practical.

Dynamic tests have been conducted on a few containments around the world. In 1979 the Heissdampfreaktor (HDR) containment in the Federal Republic of Germany was sinusoidally tested with shakers to a level equivalent to a 0.06-g ground motion.[19] Similar tests have been conducted in Japan on the Fukashima 1 and Shimane containments (10^{-4}g sinusoidal),[20] Hanaoka 1 and 2 (2.5×10^{-4} g at 2.5 Hz),[21] and Tokai II. In the US, sinusoidal tests of containments at very low force levels were conducted between 1969 and 1972 at San Onofre Unit 1[22] and in 1972 at Quad Cities. Testing in the Federal Republic of Germany has also included explosive tests at HDR with measurements of response at the nearby VAK plant.[19] In general, all of the above tests were conducted to verify analytical models and to obtain damping values. However, all the reported tests resulted in very low levels of response. The large amplitude, nonlinear behavior of the structures was not investigated.

2.3.3 Conclusions From Previous Tests

The large amplitude, nonlinear response of steel containments has not been investigated experimentally. The pressure tests of concrete containments indicate that the models behave, in a gross sense, as predicted.

However, the tests conducted in India and Poland indicate that large penetrations have significant effect upon containment response. Indeed, leakage in the Polish model occurred near the heavily reinforced area around penetrations and near the buttresses, despite the presence of a steel liner. Other types of liners (or the lack of a liner) tended to be a problem in many tests; i.e., liners had to be reinforced/replaced so that ultimate structural capacity could be reached. The Indian model, which had only vinyl paint for a liner, could not be loaded above 20 psig because of heavy leakage through cracks. Therefore, if functional failure of containments is to be simulated, the model tests must include liners, penetrations, and other structural discontinuities.

The tests and analytical efforts by the Canadians demonstrate that a computer program can be qualified for concrete shells of revolution. Unfortunately, the importance of asymmetries in tests by others suggests that more general 3-D shell programs must be qualified if the ultimate capacity of containments is to be predicted with confidence.

None of the reported model test programs compared results with models of other scales or with a prototype structure. For the subject test program, the approach to be used to achieve credibility for model results is to conduct experiments on models at each of several scale factors. Then, the behavior of the full-size structure can be predicted with a stated confidence.

References

[1]A. Walser, "An Overview of Reactor Containment Structures," *Nuc Eng and Design*, Volume 61, No. 1, November 1980.

[2]*Nuclear Safety*, Vol 21, No. 3 (Oak Ridge, TN: Nuclear Safety Information Center, Oak Ridge National Laboratories, May-June 1980) pp 399-405.

[3]Ibid., pp 429-434.

[4]"World List of Nuclear Power Plants: Operable, Under Construction, or on Order as of June 30, 1980," *Nucl N*, August 1980, pp 102-107.

[5]*Nuclear Containment Vessels*, Bulletin No. 8510 (Oak Brook, IL: Chicago Bridge and Iron Co).

[6]J. D. Stevenson, "Containment Structures for Pressurized Water Reactor Systems - Past, Present, and Future State-of-the-Art" (Pittsburgh, PA: Westinghouse Nuclear Energy Systems).

[7]F. A. Heddleson, *Design Data and Safety Features of Commercial Nuclear Power Plants*, ORNL/NSIC-96 (Oak Ridge, TN:Oak Ridge National Laboratories, June 1976).

[8]M. Subudhi et al, *Seismic Review Table*, NUREG/CR-1429 (Upton, NY: Department of Nuclear Energy, Brookhaven National Laboratory, May 1980).

[9]ASME Boiler and Pressure Vessel Code, "Section III, Nuclear Power Plant Components, Division 1 - Subsection NE, Class MC Components" (New York: The American Society of Mechanical Engineers, July 1980) (earlier editions may be applicable).

[10]ASME Boiler and Pressure Vessel Code, "Section III, Nuclear Power Plant Components, Division 2 - Code for Concrete Reactor Vessels and Containments" (New York: The American Society of Mechanical Engineers, July 1980) (earlier editions may be applicable).

[11]*Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants*," LWR Edition, NUREG-75/087 (Washington, DC: US Nuclear Regulatory Commission, 1975).

[12]R. J. Atchison, G. J. K. Asmis, and F.R. Campbell, "Behaviour of Concrete Containment under Over-Pressure Conditions," *Transactions of the 5th SMiRT*, Paper No. J3/2, Berlin, August 1979.

[13]S. Rizkalla, S. H. Simmonds, and J. G. MacGregor, "A Test of a Model of a Thin-Walled Prestressed Concrete Secondary Containment Structure," *Transactions of the 5th SMiRT*, Paper No. J4/2, Berlin, August 1979.

[14]D. W. Murray, L. Chitnuyanondh, and C. Wong, "Modelling and Predicting Behavior of Prestressed Concrete Secondary Containment Structures Using BOSOR5." *Transactions of the 5th SMiRT*, Paper J3/5, Berlin, August 1979.

[15]Y. Aoyagi et al, "Behaviours of Reinforced Concrete Containment Models Under Thermal Gradient and Internal Pressure," *Transactions of the 5th SMiRT*, Paper J4/5, Berlin, August 1979.

[16]T. Uchida et al, "Behavior of Reinforced Concrete Containment Models Under the Combined Action of Internal Pressure and Lateral Force," *Transactions of the 5th SMiRT*, Paper J4/4, Berlin, August 1979.

[17]T. V. S. R. Appa Rao, "Behaviour of Concrete Nuclear Containment Structures Up to Ultimate Failure With Special Reference to MAPP-1 Containment," *Inelastic Behavior*, Report 4-SM-THEME/75 (Madros, India: Structural Engineering Research Centre, 1975).

[18]K. Donten et al, "Results of Strength Tests on a 1:10 Model of Reactor Containment," *Transactions of the 5th SMiRT*, Paper J4/8, Berlin 1979.

[19]A. Jehlicka and L. Malcher, "Low-Level Earthquake Testing of the HDR: Comparisons of Calculations and Measurements for the Reactor Building," *Transactions of the 5th SMiRT*, Paper No. K13/2, Berlin, 1979.

[20]K. Muto et al, "Comparative Forced Vibration Test of Two BWR-Type Reactor Building," *Transactions of the 2nd SMiRT*, Paper No. K5/3, Berlin, 1973.

[21]N. Mizuno et al, "Forced Vibration Test of BWR-Type Nuclear Reactor Buildings Considering Through Soil Coupling Between Adjacent Buildings," *Transactions of the 5th SMiRT*, Paper No. K13/4, Berlin, 1979.

[22]J. Hornbuckle et al, "Forced Vibration Tests and Analysis of Nuclear Power Plants," *Nucl Eng and Design*, No. 25, 1973, pp 51-93.

3. Problem Definition

3.1 Loading Conditions

The number of loading conditions to which a containment structure can be exposed is virtually limitless when one considers all possible accident scenarios; also, the number of types of containment structures is large. It is unusual to find two plants with containments truly alike in all structural details. In an experimental program, only a limited number of tests can be performed; therefore, the selection of loading conditions is critical.

The types of loading conditions used in this program will be representative of loadings on actual containment structures and will be reproducible in a test environment. It is desirable that results obtained using the loading conditions will provide insight into the capacity of containment structures to function properly during a full spectrum of environmental or accident conditions. The function of the containment is to prevent the escape to the atmosphere of fission products that may be released from the reactor; thus, functional failure occurs when significant leakage to the atmosphere is possible. Since design procedures generally are based upon linear-elastic theory, they have a great deal of built-in conservatism for design conditions. Furthermore, pressure tests on completed containments assure low-leakage rates for design static pressure. Therefore, containment buildings can be expected to function satisfactorily at loading conditions more severe than the design conditions. Types of loading conditions that can be postulated to cause functional failure include (1) conditions which are included in the design specifications but which exceed the specified magnitude and (2) loading conditions caused by accidents that are deemed too improbable to warrant consideration during the design process.

Many of the loading conditions used in design have upper bounds that encompass, with very high probability, the loads that the actual structure will experience (e.g., the gravity load on a structure can be predicted with high accuracy). However, the prediction of severe environment conditions (e.g., earthquake and tornado) is imprecise and, although conservative design conditions are generally imposed for these events, uncertainties exist. Also, methods of dynamic analysis for earthquake loadings include many approximations that can affect the predicted behavior of a containment structure. Therefore, earthquake-like loadings will be included in this program. The use of these loadings is aimed at the determination of the capacity of containment structures to function during and after an earthquake.

Many of the postulated accidents, for which a containment structure is not designed, result in high internal pressure, both quasi-static and dynamic. A knowledge of the ultimate pressure capacity of containment structures would assist the nuclear community in assessing the possible dangers caused by accidents other than those considered during the design process; thus, this program will include quasi-static and dynamic pressurization as loading conditions. Dynamic loadings may be symmetric or asymmetric.

This program will consider the following loading conditions:

- Static Pressurization
- Dynamic Pressurization
- Earthquake-Like loadings.

As much as possible, loadings that are generic rather than plant-specific and independent of actual accident or environmental events will be used.

3.2 Assumptions and Exclusions

The results of this program should be applicable to a variety of accidents and environmental events. However, because only a limited number of loading conditions can be included, many conditions will of necessity be excluded. Thus, the loading conditions will be selected in an effort to determine the "ultimate capacity" of the containment structure without attempting to simulate a particular accident or event. It is planned to identify the characteristics of a general type of loading, the results of which can be extrapolated to a variety of scenarios. In this regard, the loading conditions will be selected so that computer programs can be qualified (see Section 4.1 for a definition of qualification); the qualified program can then be used to analyze other similar events.

Complex combinations of specified conditions will not be emphasized in this program; e.g., although it is possible that a LOCA could occur in conjunction with an earthquake event, the combination will not be studied experimentally. By verifying the analysis of each type of event independently, the analysis of combined events can be conducted with confidence. External explosions (such as caused by transportation accidents) and missile impacts will not be included in the study.

Boundary conditions also significantly effect the results of structural analyses. The emphasis is placed upon determining the "ultimate capacity" of the containment structure. Therefore, soil-structure interaction will not be emphasized; rather, for earthquake-like excitations, the response of the structure relative to the motion of the base-mat will be emphasized. Although internal structures can interact structurally with the containment structure in a dynamic event, the internal structures will not be included in all models. One exception to this is dynamic internal pressurization during which the internal structures affect greatly the form and magnitude of pressure pulses.

Modeling characteristics (specifically, the modeling of penetrations) must be limited in this program. Usually very small penetrations do not affect significantly the structural response of the containment as a whole; separate tests of small penetrations with a section of a containment vessel are contemplated. Large penetrations such as the equipment hatch and personnel lock will be included in large scale models, but their inclusion in smaller models may not be desirable. In any case, the seals around the hatches will not be included. Although the seals are susceptible to leakage, they are not structural in the usual sense and their analytical modeling to predict leakage would not be possible with the present state-of-the-art. Separate tests of seals are recommended.

4. Analytical Methods

4.1 Introduction and General Comments

A brief review of the analytical methods that are available for performing analyses of containments up to ultimate capacity is presented in this section. The choice of an analytical procedure/computer program for structural analysis depends generally upon the type of loading condition, type of structure (including material properties), and type of mathematical modeling or discretization. As indicated, the loadings of interest for this program are internal static pressurization, dynamic pressurization, and earthquake-like loadings. Containment structures can be divided according to construction materials into two categories: steel and reinforced and prestressed concrete. Procedures that are particularly dependent upon this categorization are discussed in Sections 4.2 and 4.3.

The type of analysis is dependent upon the degree to which details are included in the structural model. If asymmetric features such as penetrations are neglected, axisymmetric analyses can be performed for static and symmetric dynamic pressurization. Generally, computer codes that are tailored specifically to axisymmetric problems require less user effort for modeling and shorter computer run times. For certain asymmetric loadings, a symmetric model allows for simpler analytical procedures. When penetrations or other asymmetries are included in the structural model, three-dimensional analyses are necessary for all loading conditions; however, some approximate techniques are possible. For example, the results from an axisymmetric analysis can be used as boundary conditions in an analysis of a section of the structure that includes a penetration. However, tests in Poland and India (discussed in Section 2.3) indicate that the overall behavior of the structure is affected by large penetrations. Therefore, approximate procedures must be used with caution.

Computer programs that are believed suitable for analyzing containment structures to ultimate capacity have not, in general, been qualified* by comparisons with tests of containment structures or models. The only reported exception is the modified version of BOSOR5,[1] which has been compared with good results to the Canadian tests of a model of a CANDU containment.[2] Other computer analyses of containments to determine ultimate capacity have been performed but without test comparisons. These analyses include the studies of the pressurization of the Zion and Indian Point containments that used the NONSAP-C[3] and HONDO[4] codes, and analyses of the Sequoyah containment that include the use of ANSYS.[5]

Dynamic loadings can be treated in many of the computer programs that are suitable for static loads; however, because the response will be nonlinear (usually eliminating the use of modal solutions/truncation), time-history analyses can require prohibitive computer time and cost. In particular, the refined discretization required to model penetrations and other details will cause the dynamic model to have very high-frequency content. If an explicit time-integration scheme such as central difference is used, the time step must be extremely small to insure stability. If an implicit integration such as the Wilson-Theta method is chosen, the run cost and time at each step will be very large for nonlinear problems because a very large number of simultaneous algebraic equations must be solved.

As noted in Section 2.2, design procedures for earthquake loadings use simple linear beam models to represent the containment structure. The extension of these procedures to consider nonlinear ultimate behavior is impractical because determining the nonlinear force-displacement relationships is not possible at this time. In conclusion, a study of analytical procedures for containments must include techniques to reduce cost/run time or the resulting procedures may be too expensive to use.

4.2 Steel Containments

Because steel containments are generally homogeneous in material properties and because generally accepted constitutive models for steel are available for loadings past yield, several computer programs are available for both static and dynamic loadings. These codes include MARC,[7] ADINA,[8] and ANSYS.[5] How-

*According to the ASME,[6] qualification is concerned with whether a verified computer program adequately solves a particular problem.

ever, none of the codes have been qualified for application to steel containment vessels. Although the post-yielding plastic behavior of steel is well understood, no ultimate failure mechanism for biaxial and triaxial stress has been accepted. Crack-propagation theory requires that the location of an initial crack be known; this location can not be predicted with the existing programs. Therefore, predicting functional failure due to crack initiation, crack growth, and resulting leakage will not be attempted at this time. Although a numerical instability may develop in the mathematical model at some level of loading, functional failure may occur prior to this level.

The modeling of penetrations and other details requires very fine discretizations. The resulting models contain many elements (finite-element approach) and/or nodes (finite-difference approach). The required set-up time and computer run time is very large; thus, for the analysis of entire containments, the inclusion of such details may be impractical. The problem of cost/time is more severe for dynamic analyses.

Circumferential ring and vertical stiffeners add complexity to the analysis of steel containments. Stiffeners can be "smeared out" or treated as discrete stiffeners. However, smearing of stiffeners is generally inappropriate for containment structures because stiffeners are spaced far enough apart that significant variations of displacements and stress occur.[9] Clearly smearing is particularly inappropriate when nonlinear behavior is present; therefore, stiffeners must be treated discretely to obtain meaningful results in ultimate analyses. However, predictions of stress and dynamic response are sensitive to seemingly minor changes in the mathematical modeling of discrete stiffeners. In particular, the location of the stiffener relative to the shell reference surface and differences in shear center and centroidal axes can be very important.[9] The treatment of these details in existing codes must be investigated further.

If only circumferential rings are present and penetrations are neglected, axisymmetric codes/elements can be used. Of particular note are the BOSOR codes[10,11] which are widely used and which have been compared with tests on a variety of axisymmetric metal structures. If vertical stringers are included, a code with general shell capabilities such as MARC,[7] ANSYS,[5] or ADINA[8] is required. Although axisymmetry is not present, vertical stiffeners are usually evenly spaced circumferentially. If penetrations are neglected, it is possible to model one segment (vertical-stringer to center of panel) from top to bottom. If penetrations are included, a full finite-element model of the entire containment structure is required. With nonlinear behavior, such full analyses may be impractical because the time and cost become prohibitive. Further investigation of analytical techniques is required.

4.3 Concrete Containments

Many complex components and interactions affect the response of concrete containments, including

- Steel liners, which are required on all US concrete containments
- Liner-concrete interaction, including attachment studs
- Steel reinforcement and prestressing tendons
- Reinforcement-concrete interaction (bond strength/slippage)
- Tendon-concrete interaction, including friction along the length of the tendon
- Complex nonlinear behavior of concrete.

With the present state-of-the-art, it is not practical (if indeed even possible) to fully account for all of these factors in a computer analysis of an entire containment structure. Until experiments provide better insight into the failure mechanisms for concrete containments, it is not possible to determine which factors can be neglected or treated in an approximate manner; thus, evaluating the credibility of existing codes is not possible at present. However, some general observations of analytical procedures are helpful for planning future efforts; brief observations are included in this section.

At least two general approaches are possible in the application of the finite-element method to reinforced concrete shells: the discrete-element approach and the layered approach. In the former, the concrete and steel reinforcing are modeled as discrete elements that are connected at nodal points or through bond linkage elements. Although the discrete-element approach is practical for evaluating single structural components (such as a reinforced concrete beam), the approach is impractical at present for concrete containments because an extremely large number of discrete elements are necessary. However, the discrete-element approach may be the only approach suitable for predicting ultimate capacity if element interaction (such as between studs and concrete) prove to be failure mechanisms. In this case, considerable development will be required to make the approach practical.

In the layered approach, the finite element is divided into a number of steel and concrete layers over the depth. A review of the manner in which this approach and the discrete element approach have been implemented has been presented by Litton and Gidwani.[12] It is clear that the manner of implementation can have a significant effect upon results. None of the researchers[12] has qualified this approach with tests of containments. However, the finite difference code BOSOR5, which also uses a layered approach and which has been modified to include a concrete element[11] has been favorably compared to tests of an axisymmetric nonreplica model of a Canadian CANDU containment. As noted in Section 2, the test model did not contain a steel liner or penetrations. It should be emphasized that the BOSOR codes require axisymmetric modeling.

Although considerable progress has been made over the last 10 yr in modeling of concrete, some phenomena are not well enough understood to be modeled explicitly with confidence. For example, shear strength/stiffness after cracking and the interaction of the concrete with reinforcement including doweling action can not, at present, be modeled with confidence. Also, the interaction of tendons with concrete, including frictional forces along the length of the tendon, can not be modeled in detail, particularly for dynamic loadings. Even constitutive models for concrete without reinforcement are either approximate or so complex as to provide great difficulties in implementation. The high cost of implementation is probably not justified in most cases because none of the constitutive models is widely accepted.

The modeling of concrete containments in fine detail with high confidence is not possible at present and may not be practical in the near future. Modeling, using simplified approaches such as layering of concrete and steel, is possible, but qualification of existing or new codes is required before simplified procedures can be used with confidence.

References

[1]D. W. Murray, L. Chitnuyanondh, and C. Wong, "Implementation of an Elastic-Plastic Concrete Relationship," *Computer Methods in Applied Mechanics and Engineering*, No. 23, 1980, pp 35-57.

[2]D. W. Murray, L. Chitnuyanondh, and C. Wong, "Modeling and Predicting Behavior of Prestressed Secondary Containment Structures Using BOSOR5," *Transactions of the 5th SMiRT*, Paper J3/5, Berlin, August 1979.

[3]P. D. Smith and C. A. Anderson, *NONSAP-C: A Nonlinear Stress Analysis Program for Concrete Containments Under Static, Dynamic, and Long-Term Loadings*, NUREG/CR-0416, LA-7496-MS (Los Alamos, NM: Los Alamos Scientific Laboratory, October 1978).

[4]S. W. Key, Z. E. Beisinger, and R. D. Krieg, *HONDO II, A Finite Element Computer Program for the Large Deformation Dynamic Response of Axisymmetric Solids*, SAND78-0422 (Albuquerque, NM: Sandia Laboratories, October 1978).

[5]G. J. DeSalvo and J. A. Swanson, *ANSYS, Engineering Analysis System User's Manual* (Houston, PA: Swanson Analysis Systems, Inc., August 1978).

[6]I. Berman, "Introduction," *Engineering Computer Software, Verification, Qualification, and Certification* (New York, NY: American Society of Mechanical Engineers, 1971) pp ix-x.

[7]*MARC-CDC User Information Manual, Volume I - General Purpose Finite Element Analysis Program*, Publication No. 17309500 (Minneapolis, MN: Control Data Corporation, 1977).

[8]K. J. Bathe, *ADINA, A Finite Element Program for Automatic Dynamic Incremental Nonlinear Analysis*, Report 82448-1 (Cambridge, MA: Massachusetts Institute of Technology, May 1977).

[9]D. Bushnell, "Thin Shells," *Structural Mechanics Computer Programs*, edited by W. Pilkey, K. Saczalski, and H. Schoeffer (Charlottesville, VA: University Press of Virginia, 1974) pp 277-358.

[10]D. Bushnell, "BOSOR4 - Program for Stress, Buckling, and Vibration of Complex Shells of Revolution," *Structural Mechanics Software Series*, Vol. 1, edited by N. Perrone and W. Pilkey (Charlottesville, VA: University Press of Virginia, 1977) pp 11-143.

[11]D. Bushnell, "BOSOR5 - Program for Buckling of Elastic-Plastic Complex Shells of Revolution Including Large Deflections and Creep," *Computers and Structures*, Vol. 6, pp 221-239, 1976.

[12]R. W. Litton and J. M. Gidwani, *Failure Criteria of Reinforced Concrete Structures*, Vol. I, AFWL-TR-77-239 (Kirtland AFB, NM: Air Force Weapons Laboratory, May 1979).

5. Containment Modeling

5.1 Introduction

The high costs associated with conducting full-scale tests on reactor containments make scale-model tests an attractive alternative. A model analysis for three types of reactor containment structures subjected to three different types of loadings is presented in Section 5.2. The three loading conditions are

- Internal static pressure
- Internal dynamic pressure
- Seismic loading

The three types of reactor containment structures are

- Hybrid steel
- Reinforced concrete
- Prestressed concrete

The failure of structural elements under a given load condition (identified in Section 5.3) will establish the ultimate capacity of the containment. The conditions required to establish a credible experimental program are discussed in Section 5.4.

In the most general sense, an experimental result is credible if it is shown to offer "reasonable grounds for being believed." In this chapter, a more limited definition (better suited to our problem) has been employed. The experimental results will be considered "credible" if it is shown that the failure mode and ultimate load-carrying capacity of the individual models tested can be correlated with those of the other models. However, the credibility of an experimental program cannot be established prior to obtaining and evaluating the data. It is, therefore, normal to start with small-scale tests and expand the size of the experimental program after preliminary tests indicate that correlatable results can be expected.

There are many factors that affect the credibility of a test program. However, the primary basis for judgment in model testing is the same as in other fields of scientific and engineering experimentation.

- Repeatability of experimental test results must be demonstrated
- Test results must be independent of the size (scale) of the experimental item.

Thus, if a reasonable number of tests are conducted in a well-planned experimental program, there is every reason to expect that high credibility will be accorded to the program.

5.2 Model Scaling Laws

5.2.1 Replica Model Analysis

In a replica model all parameters relating to geometry, material properties, loading conditions, etc, are scaled according to generally accepted procedures. Table 5.2-1 lists those parameters that are judged to be relevant to the pressure or seismic testing of containment models. From this list, 28 nondimensional pi terms were generated, using the Buckingham Pi Theorem;[1] these terms are presented in Table 5.2-2.

5.2.2 Inspection of Pi Terms

For a model and prototype system to be equivalent, only the pi terms (and not each individual parameter) have to be identical in both systems. This principle introduces the concept of scale factor for the various parameters in the problem.

The type of model normally considered in this type of study is a replica one. A replica model is built with the same materials in corresponding model and prototype locations, but is smaller by a geometric scale factor λ. In a replica model, all material properties such as density ρ, strength σ, and strain rate coefficient K_i are the same as in the prototype. However, not all phenomena are scaled without distortion in a replica model. This statement can be demonstrated by investigating each pi term. Each pi term in Table 5.2-2 is investigated by setting the ratio of the values for the model and prototype equal to unity (i.e., by equating the pi terms for model and prototype). For example, consider pi term 13 which is concerned with gravitational effects.

$$\left(\frac{\rho g L}{\sigma}\right)_m = \left(\frac{\rho g L}{\sigma}\right)_p$$

where the suscript m refers to the model and p refers to the prototype.

Table 5.2-1. List of Parameters

Symbol	Parameter	Fundamental Units of Measure	Reason for Using in Analysis
L	Characteristic Length	L	Geometry
ℓ_i	Other Lengths Relative to L	-	
θ_i	Angles	-	
ρ	Mass Density of Concrete or Other Reference Material	FT^2/L^4	Inertial Effects
ρ_i	Density of Steel or Other Materials Relative to Reference ρ	-	
σ	Characteristic Strength of Structure	F/L^2	Strength of Structural Materials
E_i	Moduli Relative σ	-	
σ_i	Strengths Relative σ	-	
ν	Poisson's Ratio	-	
N_i	Number Reinforcing Bars	-	Prestress in Concrete
F	Prestress Force in Bar	F	
F_i	Prestress Force Other Bars Relative F	-	
g	Acceleration Gravity	L/T^2	Dead Weight Effects
K_i	Strain Rate Coefficient	FT/L^2	Strain Rate
ρ_o	Density of Air	FT^2/L^4	Atmospheric Conditions
p_o	Atmospheric Pressure	F/L^2	
β_i	Coefficient of Equivalent Viscous Damping	FT/L	Equivalent Viscous Damping
ϵ	Strain Anywhere on Structure	-	Response Being Observed
x	Displacement of Any Point on Structure	L	
v	Velocity of Point on Structure	L/T	
a	Acceleration of Point on Structure	L/T^2	
t	Time	T	
$\dot{m}$	Leakage Rate	FT/L	
P	Applied Maximum Pressure	F/L^2	Internal Pressure Loading
T	Duration of Pressure Loading	T	
$p(t)$	Applied Pressure History	F/L^2	
A	Acceleration Amplitudes	L/T^2	Applied Earthquake Loading
V	Velocity Amplitudes	L/T	
X	Displacement Amplitudes	L	
ω	Frequency	$1/T$	
T_e	Earthquake Time Duration	T	

Table 5.2-2. List of Pi Terms

Symbol	Parameter	Fundamental Units of Measure	Reason for Using in Analysis
$\pi_1 = \ell_i$	Geometric Similarity	$\pi_{15} = \varepsilon$	Interpret Response
$\pi_2 = \theta_i$		$\pi_{16} = \frac{x}{L}$	
$\pi_3 = \rho_i$	Similar Densities	$\pi_{17} = \frac{v\ \rho^{1/2}}{\sigma^{1/2}}$	
$\pi_4 = N_i$	Number of Reinforcing Bars	$\pi_{18} = \frac{a\ \rho\ L}{\sigma}$	
$\pi_5 = \sigma_i$	Constitutive Similarity	$\pi_{19} = \frac{\dot{m}}{\rho^{1/2}\ \sigma^{1/2}\ L^2}$	
$\pi_6 = E_i$		$\pi_{20} = \frac{t\ \sigma^{1/2}}{L\ \rho^{1/2}}$	
$\pi_7 = \nu$		$\pi_{21} = \frac{P}{\sigma}$	Excitation for Pressurization
$\pi_8 = \frac{P_o}{\sigma}$	Scale Atmospheric Conditions	$\pi_{22} = \frac{p(t)}{\sigma}$	
$\pi_9 = \frac{\rho_o}{\rho}$		$\pi_{23} = \frac{T\ \sigma^{1/2}}{L\ \rho^{1/2}}$	
$\pi_{10} = \frac{\beta_i}{L^2\ \rho^{1/2}\ \sigma^{1/2}}$	Scale Equivalent Viscous Damping	$\pi_{24} = \frac{A\ \rho\ L}{\sigma}$	Excitation for Earthquake
$\pi_{11} = \frac{F}{\sigma\ L^2}$	Scale Prestress in Tendons	$\pi_{25} = \frac{V\ \rho^{1/2}}{\sigma^{1/2}}$	
$\pi_{12} = F_i$		$\pi_{26} = \frac{X}{L}$	
$\pi_{13} = \frac{\rho\ g\ L}{\sigma}$	Scale Gravitational Effects	$\pi_{27} = \frac{\omega\ L\ \rho^{1/2}}{\sigma^{1/2}}$	
$\pi_{14} = \frac{K_i}{\sigma^{1/2}\ \rho^{1/2}\ L}$	Scale Strain Rate Effects	$\pi_{28} = \frac{T_e\ \sigma^{1/2}}{L\ \rho^{1/2}}$	

Rearranging

$$\left(\frac{\rho_m g_m L_m \sigma_p}{\rho_p g_p L_p \sigma_m}\right) = 1.0$$

For a replica model, the ratio of the lengths (L_m/L_p) is the scale factor (λ) while the density (ρ) and strength (σ) are the same for model and prototype. Thus, this equation becomes

$$\left(\frac{g_m}{g_p} \cdot \lambda\right) = 1.0$$

or

$$\frac{g_m}{g_p} = \frac{1}{\lambda}$$

Thus, gravity scales as the inverse of the scale factor.

In a manner similar to this, each pi term was investigated. The results of this effort are given in Table 5.2-3.

If all parameters could be scaled as in Table 5.2-3, all pi terms would be satisfied and no problems would arise if the models were well built and tests carefully conducted. Close inspection of the parameters in Table 5.2-3 shows that three potential problem areas arise. These are given below the dashed line.

Because a 1.0-g gravity field exists on the surface of the earth, it is difficult to test a model scaling gravity as $1/\lambda$. Thus, pi term 13 will not scale and dead weight effects are incorrect in a replica model. A check of the magnitude of the dead-weight effect showed that, for a reinforced concrete containment, stresses at the bottom of the walls are approximately 200 psi. For a free-standing steel containment, stresses are on the order of 750 psi. These stresses are sufficiently small to be considered insignificant relative to the strengths of the structural materials. Therefore, distortions caused by gravitational effects in scaling should be small.

Inspection of pi term 14 reveals that the strain-rate coefficient effects in the model should be smaller than in the prototype by factor of λ. For many materials, increases in the rate of strain will result in slight changes in yield point as well as other stress-strain characteristics. This influence increases with the magnitude of the scale factor. No easy resolution exists for correcting this influence. Separate effects tests for prototype and model materials at the correct respective strain rates should be conducted to determine strain-rate significance. Slight variations in the heat treatment of the model steel and slight variations in the composition of the concrete can be made, if necessary, to establish the same stress-strain curves at scaled rates in the model. If materials being used are not very rate sensitive, then the normal dispersion in response measurements may be larger than strain rate effects.

Damping is difficult to scale properly. As used in this analysis, equivalent viscous damping must scale as λ^2, but this will not occur if the same materials are used in corresponding locations in model and prototype. Damping will be too high in the model. Inability to scale damping is not generally important in dynamic internal pressurization tests. However, damping is important in earthquake-loading experiments. During fabrication of a model, damping can be controlled to a limited extent, but exact duplication of damping will not be realized. One possible procedure for treating the influence of damping involves measuring the damping in the model and adjusting the input to an appropriate level.

Table 5.2-3. Replica Modeling Law to Satisfy Pi Terms

Parameter	Symbol	Scale Factor
Lengths, displacements	L, x, X	λ
Angles	θ_i	1.0
Times, duration	t, T, T_e	λ
Velocities	v, V	1.0
Accelerations	a, A	$1/\lambda$
Stresses	σ	1.0
Densities	ρ, ρ_o	1.0
Strains	ϵ	1.0
Pressures	P, p(t), P_o	1.0
Frequencies	ω	$1/\lambda$
Forces	F	λ^2
Number of reinforcing bars	N_i	1.0
Leakage rate	$\dot{m}$	λ^2
- - - - - - - -	- - -	- - -
Acceleration of gravity	g	$1/\lambda$
Strain-rate coefficient	K_i	λ
Equivalent viscous damping	β_i	λ^2

NOTE: As an example of how to use this table, consider a scale model 1/20 the size of the prototype. This means that the scale factor λ equals 1/20. Durations in the model will be 1/20 those in the prototype, but frequencies in the model will be 20 times higher. Forces in reinforcing bars will be 1/400 those in a prototype as will be the mass leakage rate. All stresses and strains at scaled times will be the same in model and prototype systems.

5.2.3 Nonreplica Model Analysis

The replica modeling law is not the only method of satisfying all of the pi terms in Table 5.2-2. Table 5.2-4 presents a general solution in which the densities are scaled by a factor γ, the strength and stress are scaled by a factor α, and all significant lengths are scaled by a factor λ. When the factors α and γ equal 1.0, this general solution reduces to the replica modeling law that has already been summarized in Table 5.2-3. For static and dynamic pressure-loading models, a replica model works well. Hence, there is no need to change materials by making α and γ factors other than 1.0.

Table 5.2-4. General Modeling Law to Satisfy Pi Terms

Parameter	Symbol	Scale Factor
Lengths,displacements	L, x, X	λ
Angles	θ_i	1.0
Times, duration	t, T, T_o	$\frac{\gamma^{1/2}\lambda}{\alpha^{1/2}}$
Velocities	v, V	$\frac{\alpha^{1/2}}{\gamma^{1/2}}$
Accelerations	a, A	$\frac{\alpha}{\gamma\lambda}$
Stresses	σ	α
Densities	ρ, ρ_o	γ
Strains	ϵ	1.0
Pressures	P, p(t), P_o	α
Frequencies	ω	$\frac{\alpha^{1/2}}{\gamma^{1/2}\lambda}$
Forces	F	$\alpha\lambda^2$
Number of reinforcing bars	N_i	1.0
Leakage rate	$\dot{m}$	$\gamma^{1/2}\alpha^{1/2}\lambda^2$
Acceleration of gravity	g	$\frac{\alpha}{\gamma\lambda}$
Strain-rate coefficient	K_i	$\gamma^{1/2}\alpha^{1/2}\lambda$
Equivalent viscous damping	β_i	$\lambda^2\alpha^{1/2}\gamma^{1/2}$

On the other hand, earthquake-vibration studies conducted on shaker tables have problems whenever small replica models are used. In a replica model, frequency scales as $1/\lambda$, which in a 1/16-scale model would mean that frequencies are 16 times greater. No currently available shaker tables can provide the required spectra.

The general modeling solution presented in Table 5.2-4 presents an opportunity for testing vibration models at frequencies lower than those associated with replica models. If the strength factor (α) is less than 1.0 and the density factor (γ) is greater than 1.0, models can be built whose frequencies will be lower than those in a replica model. As an illustration, assume $\alpha = 1/9$ and $\gamma = 4.0$, then, for a 1/16 scale model, the scale factor for frequency would be only 2.7 times the frequency in a prototype rather than being 16 times greater as in a replica model.

This general solution is not without problems. By obtaining a better scale factor for frequency, a requirement for changing materials when building a model has been added. In an elastic model, a change is less difficult than in an inelastic model. The use of material substitutions is conceptually possible, but many practical problems arise when the entire stress-strain curve must be simulated. In addition, a material such as reinforced concrete would require substitute materials for the concrete and steel. Since material failure, which is an integral part of this investigation, could not be accurately modeled, the utility of the model would be quite restricted.

A commonly used type of nonreplica model scaling is the so-called Froude scaling. Fundamentally this requires that the Froude number, V^2/gL, scale identically. The effect is that gravity scales without distortion but time scales as λ. This type scaling is useful for rigid body problems, but not for problems where material flexibility and fracture are involved; the material parameters such as E_i and σ_i must be distorted. Los Alamos[2] planned the use of this type scaling in a 1976 proposal to NRC for a seismic test facility. No use of Froude scaling is seen for this program.

5.2.4 Summary

For pressurization loading, the major pi terms are shown in Table 5.2-5. These pi terms can be satisfied using a replica model, and no major instrumentation or loading problems exist. The scale factors for the replica model were presented in Table 5.2-3.

For seismic excitation, the major pi terms are listed in Table 5.2-6. Satisfying these pi terms is more difficult. Damping (scaled according to pi term 10) presents particular problems. Rigorously scaled damping requires a distorted model and special testing techniques. Another problem with the use of replica models for earthquake studies occurs when shaker tables are used for the excitation. When the input spectrum is scaled, some conflicting requirements for shaker capabilities are generated. Further discussion of this problem is in Section 6.1. Clearly seismic-model studies are more complex than those for internal pressurization.

Table 5.2-5. Major Pi Terms for Pressurization Loading

$\pi_1 = l$	$\pi_{12} = F_i$
$\pi_2 = \theta_i$	$\pi_{15} = \epsilon$
$\pi_3 = \rho_i$	$\pi_{16} = \frac{x}{L}$
$\pi_4 = N_i$	$\pi_{17} = \frac{v\rho^{1/2}}{\sigma^{1/2}}$
$\pi_5 = \sigma_i$	$\pi_{18} = \frac{a\rho L}{\sigma}$
$\pi_6 = E_i$	$\pi_{19} = \frac{\dot{m}}{\rho^{1/2}\sigma^{1/2}L^2}$
$\pi_7 = \upsilon$	$\pi_{20} = \frac{t\sigma^{1/2}}{L\rho^{1/2}}$
$\pi_8 = \frac{P_o}{\sigma}$	$\pi_{21} = \frac{P}{\sigma}$
$\pi_9 = \frac{P_o}{\sigma}$	$\pi_{22} = \frac{p(t)}{\sigma}$
$\pi_{11} = \frac{F}{\sigma L^2}$	$\pi_{23} = \frac{T\sigma^{1/2}}{L\rho^{1/2}}$

Table 5.2-6. Major Pi Terms for Seismic Excitation

$\pi_1 = l$	$\pi_{15} = \epsilon$
$\pi_2 = \theta_i$	$\pi_{16} = \frac{x}{L}$
$\pi_3 = \rho_i$	$\pi_{17} = \frac{v\rho^{1/2}}{\sigma^{1/2}}$
$\pi_4 = N_i$	$\pi_{18} = \frac{a\rho L}{\sigma}$
$\pi_5 = \sigma_i$	$\pi_{19} = \frac{\dot{m}}{\rho^{1/2}\sigma^{1/2}L^2}$
$\pi_6 = E_i$	$\pi_{20} = \frac{t\sigma^{1/2}}{L\rho^{1/2}}$
$\pi_7 = \upsilon$	$\pi_{24} = \frac{A\rho L}{\sigma}$
$\pi_8 = \frac{P_o}{\sigma}$	$\pi_{25} = \frac{V\rho^{1/2}}{1/2}$
$\pi_9 = \frac{\rho_o}{\rho}$	$\pi_{26} = \frac{X}{L}$
$\pi_{10} = \frac{\beta_i}{L^2\rho^{1/2}\sigma^{1/2}}$	$\pi_{27} = \frac{\omega L\rho^{1/2}}{\sigma^{1/2}}$
$\pi_{11} = \frac{F}{\sigma L^2}$	$\pi_{28} = \frac{T_e\sigma^{1/2}}{L\rho^{1/2}}$
$\pi_{12} = F_i$	

5.3 Identification of Critical Structural Elements

5.3.1 Introduction

It is the purpose of this section to identify those structural elements whose failure, under a given loading condition, will establish the ultimate load-carrying capacity of a containment structure. Once the failure modes have been identified, the model designer may design model structures that will adequately demonstrate the behavior of the prototype structure and satisfy all of the essential terms in the scaling laws.

5.3.2 Identification of Critical Structural Elements

Each of the previously discussed structural types and loading conditions have been considered in the determination of critical elements, the failure of which could lead to containment failure. The failure modes (Tables 5.3-1 to 5.3-3) are cataloged in general terms such as meridional tension or tangential shear for structural elements. Where it is applicable, nonstructural failures such as a seal failure at a pipe penetration are also identified. Some categories overlap (e.g., meridional and/or circumferential bending will be present in a buckling "failure," but all three are listed in separate catagories). After general failure modes have been identified, individual structural components are identified (e.g., the steel liner plates in a reinforced concrete containment; the welds joining the liner plates together; and the individual reinforcing bars).

The critical components were selected after a detailed review of the construction drawing of three plants and a general review of the Safety Analysis Reports of similiar plants. The prototype plants for determination of critical components are:

Free-Standing Steel – Watts Bar 1 and 2,
Reinforced Concrete – Salem 1 and 2, and
Prestressed Concrete – South Texas 1 and 2.

5.3.3 Adequate and Replica Models

Before the different types of containment models are discussed, certain terms will be defined.

Model – "A model is a device which is related to a physical system such that observations on the model may be used to predict accurately the performance of the physical system in the desired respect."[3]

Prototype – "A physical system for which the predictions are to be made is called the prototype."[3]

Replica Model – "A (replica model is a) physical model of a prototype which is geometrically similar in all respects to the prototype and employs identically the same materials at similar locations."[1]

Adequate Model – "Adequate Models are models from which accurate predictions of one characteristic of the prototype may be made, but which will not necessarily yield accurate predictions of other characteristics."[3]

Dissimilar Material Models – "A dissimilar material model is a model which is geometrically similar to a prototype but made of different material. The material must, however, have properties which can be correlated with those in the prototype."[1]

Model experiments are an accepted method for problem solution where prototype testing is too costly or is impractical for other reasons. Replica or adequate models are generally employed in structural tests that involve static and impulsive loads; dissimilar material models are often employed in structural tests that involve vibratory and thermal loadings within the elastic range. Many examples are available.[1,3-5]

The need for adequate models in the program becomes apparent when one considers the replica modeling cost of the welds that join the plates in free-standing steel containments. These welds are made in the following manner. A certified welder places a section of weld bead. The bead is then cleaned and ground to present a surface suitable for nondestructive testing. If nondestructive testing indicates cracking, the weld bead is removed and replaced. This process is difficult, costly, and time consuming on the prototype. When it is recognized that many properly sequenced passes are required on both sides of the steel plate at each joint, the difficulty and cost of replica modeling these welds becomes apparent. It is estimated that the cost of a ¼-scale free-standing steel containment would increase by \$3M if replica welding of joints is required. Therefore, "adequate" modeling of the weld will be used and separate effects tests will check the adequacy of the model welds.

While welding is the largest single-cost item, there are many other areas where model cost reduction can be achieved by the use of adequate models. For example, the Watts Bar containment structure is attached to its concrete base by 360 anchor bolts. These bolts provide only a tensile load capability. Under static-pressure and impulsive-loading conditions, it would be sufficient to model the distributed load-deflection characteristics of the hold-down bolts rather than replicating each of the 360 bolts (assuming that bolt failure is not being investigated). Seismic loading presents a more complicated problem. At this time, adequacy of not modeling individual bolts is questionable for seismic tests and would not be recommended without further study.

Considerable economy can be achieved through the use of adequate scale models. For example, a "replica" 1/20-scale model of a free-standing steel containment is estimated to cost \$167 000 in 1981. An adequate 1/20-scale model for pressurization tests of the same structure is estimated to cost as little as \$50 000, depending upon which features are not replicated but treated in an "adequate" manner.

Table 5.3-1. Failure Modes and Critical Components for Free-Standing Steel Containments

Failure Modes and Critical Components	Loading	¼ Scale	⅛ Scale	1/10 Scale	1/20 Scale	1/50 Scale
Hoop Tension						
Plate	E, P	✓	✓	✓	✓	✓
Welds	E, P	✓[1]	x	x	x	x
Stiffeners	E, P	✓	✓	✓	✓	x
Meridional Tension						
Plate	S,E,P	✓	✓	✓	✓	✓
Welds	S,E,P	✓[1]	x	x	x	x
Stiffeners	S,E,P	✓	✓	✓	✓	x
Meridional Bending						
Plate	S,E,P	✓	✓	✓	✓	✓
Welds	S,E,P	✓[1]	x	x	x	x
Stiffeners	S,E,P	✓	✓	✓	✓	x
Local Bending @ Penetrations						
Plate	S,E,P	✓	✓	✓	✓	✓
Welds	S,E,P	✓[1]	x	x	x	x
Stiffeners	S,E,P	✓	✓	✓	✓	x
Tangential Shear						
Plate	S	✓	✓	✓	✓	✓
Welds	S	✓[1]	x	x	x	x
Radial Shear @ Access Ports						
Plate	S,E,P	✓	✓	✓	✓	✓
Welds	S,E,P	✓[1]	x	x	x	x
Stiffeners	S,E,P	✓	✓	✓	✓	x
Radial Shear @ Base						
Plate	S,E,P	✓	✓	✓	✓	✓
Welds	S,E,P	✓[1]	x	x	x	x
Stiffeners	S,E,P	✓	✓	✓	✓	x
Shell Buckling						
Plate	S, E	✓	✓	✓	✓	✓
Stiffeners	S, E	✓	✓	✓	✓	x
Pipe Penetration-Seal	S,E,P	✓	✓	✓	x	x
Equipment Hatch						
Membrane Tension						
Plate	E, P	✓	✓	✓	✓	x
Welds	E, P	✓[1]	x	x	x	x
Support Ring Collapse	E, P	✓	✓	✓	✓	x
Bolt Overstress						
Bolt	S,E,P	✓	✓	✓	✓	x
Support	S,E,P	✓	✓	✓	✓	x
Shell Buckling	S,E,P	✓	✓	✓	✓	x
Personnel Lock						
Bending						
Plate	S,E,P	✓	✓	✓	✓	x
Welds	S,E,P	✓[1]	x	x	x	x
Shear						
Plate	S,E,P	✓	✓	✓	✓	x
Welds	S,E,P	✓[1]	x	x	x	x
Locking Mechanism	S,E,P	✓	✓	✓	x	x
Foundation Settlement	S	x	x	x	x	x

S = Seismic
E = Internal Explosion
P = Internal Static Pressure

[1]Conductivity is not scaled; i.e., heat effected zones, residual stresses, etc. are not the same in model and prototype unless the parts are stress-relieved.

✓ = Will scale
x = Will not scale

Table 5.3-2. Failure Modes and Critical Components for Reinforced Concrete Containments

Failure Modes and Critical Components	Loading	¼ Scale	⅛ Scale	1/10 Scale	1/20 Scale	1/50 Scale
Meridional Tension						
Rebar	S,E,P	✓[1]	✓[1]	✓[1]	✓[1]	x
Hoop Tension						
Rebar	E, P	✓[1]	✓[1]	✓[1]	✓[1]	x
Longitudinal Compression (concrete crushing, spalling, scabbing)	S	✓	✓	✓	x	x
Tangential Shear						
Concrete	S	✓	✓	✓	x	x
Lacing	S	✓	✓	✓	x	x
Stirrups	S	✓	✓	✓	x	x
Radial Shear						
Concrete	S,E,P	✓	✓	✓	x	x
Lacing	S,E,P	✓	✓	✓	x	x
Stirrups	S,E,P	✓	✓	✓	x	x
Longitudinal Bending	S,E,P	✓	✓	✓	x	x
Circumferential Bending	S	✓	✓	✓	x	x
Shell Buckling	S	✓	✓	✓	✓	x
Pipe Penetration-Seal	S,E,P	✓	✓	✓	x	x
Equipment Hatch						
Membrane Tension						
Plate	E, P	✓	✓	✓	✓	x
Welds	E, P	✓[2]	✓	✓	✓	x
Support Ring Collapse	E, P	✓	✓	✓	✓	x
Bolt Overstress						
Bolt	S,E,P	✓	✓	✓	✓	x
Support	S,E,P	✓	✓	✓	✓	x
Shell Buckling	S,E,P	✓	✓	✓	✓	x
Personnel Lock						
Bending						
Plate	S,E,P	✓	✓	✓	✓	x
Welds	S,E,P	✓[2]	x	x	x	x
Shear						
Plate	S,E,P	✓	✓	✓	✓	x
Welds	S,E,P	✓[2]	x	x	x	x
Locking Mechanism	S,E,P	✓	✓	✓	x	x
Liner Failure						
Plate	S,E,P	✓	✓	✓	✓	x
Welds	S,E,P	✓[2]	x	x	x	x
Foundation Settlement	S	x	x	x	x	x

S = Seismic
E = Internal Explosion
P = Internal Static Pressure

[1]Cadwelds are not simulated exactly.

[2]Conductivity is not scaled; i.e., heat effected zones, residual stresses, etc, are not the same in model and prototype unless the parts are stress-relieved.

✓ = Will scale
x = Will not scale

Table 5.3-3. Failure Modes and Critical Components for Prestressed Concrete Containments

Failure Modes and Critical Components	Loading	¼ Scale	⅛ Scale	1/10 Scale	1/20 Scale	1/50 Scale
Meridional Tension						
Rebar	S,E,P	✓[1]	✓[1]	✓[1]	✓[1]	x
Tendons	S,E,P	✓	✓	✓	x	x
Hoop Tension						
Rebar	E, P	✓[1]	✓[1]	✓[1]	✓[1]	x
Tendons	E, P	✓	✓	✓	x	x
Longitudinal Compression (concrete crushing, spalling, scabbing)	S	✓	✓	✓	x	x
Tangential Shear						
Concrete	S	✓	✓	✓	x	x
Stirrups	S	✓	✓	✓	x	x
Radial Shear						
Concrete	S,E,P	✓	✓	✓	x	x
Stirrups	S,E,P	✓	✓	✓	x	x
Longitudinal Bending	S,E,P	✓	✓	✓	x	x
Circumferential Bending	S	✓	✓	✓	x	x
Shell Buckling	S	✓	✓	✓	✓	x
Tendon Anchor						
Bearing Failure (concrete)	S,E,P	✓	✓	✓	x	x
Tendon Terminator	S,E,P	✓	x	x	x	x
Pipe Penetration-Seal	S,E,P	✓	✓	✓	x	x
Equipment Hatch						
Membrane Tension						
Plate	E, P	✓	✓	✓	✓	x
Welds	E, P	✓[2]	x	x	x	x
Support Ring Collapse	E, P	✓	✓	✓	✓	x
Bolt Overstress						
Bolt	S,E,P	✓	✓	✓	✓	x
Support	S,E,P	✓	✓	✓	✓	x
Shell Buckling	S,E,P	✓	✓	✓	✓	x
Personnel Lock						
Bending						
Plate	S,E,P	✓	✓	✓	✓	x
Welds	S,E,P	✓[2]	x	x	x	x
Shear						
Plate	S,E,P	✓	✓	✓	✓	x
Welds	S,E,P	✓[2]	x	x	x	x
Locking Mechanism	S,E,P	✓	✓	✓	x	x
Liner Failure						
Plate	S,E,P	✓	✓	✓	x	x
Welds	S,E,P	✓[2]	x	x	x	x
Foundation Settlement	S	x	x	x	x	x

S = Seismic
E = Internal Explosion
P = Internal Static Pressure

[1]Cadwelds are not simulated exactly.

[2]Conductivity is not scaled; i.e., heat effected zones, residual stresses, etc, are not the same in model and prototype unless the parts are stress-relieved.

✓ = Will scale
x = Will not scale

Adequ... models do have limitations. The most severe limitation is the elemination of some failure modes. For example, if the containment liner plates are not joined by replica welds, the failure of welded joints will not be modeled properly. Therefore, it is possible to unknowingly eliminate a potential failure mechanism in an adequate model.

5.3.4 Relationship Between Failure Modes and Scale Factors

Results of failure mode investigations are summarized in Tables 5.3-1 through 5.3-3. The components of concrete containments that may fail in meridional or hoop tension are the rebar and tendons. The rebar can be properly scaled to models of 1/20-scale except that the Cadwelds that join the large rebar cannot be duplicated in model scale; however, these joints have been established to be stronger than the rebar. Therefore, the Cadwelds can be omitted from the model. Tendons can be scaled in models as small as 1/10-scale. Failures associated with the tendon anchors are listed separately. It is not expected that the tendon anchors can be duplicated in models smaller than 1/4-scale. Similar considerations also apply to the reinforced-concrete containments. As noted previously, welds can not be practically replicated; however, the liner can be modeled in scales as small as 1/50-th.

Failure modes for steel containment structures are described in Table 5.3-1. The fine detail required to model stiffeners makes these costly and perhaps impractical to include in models smaller than 1/20-scale.

Many items may be adequately represented in small scale models, even though they have not been replicated. The tie-down bolts mentioned previously are an example of this. Thus, one must view the Xs in Tables 5.3-1 through 5.3-3 as question marks. At least one test at a sufficiently large scale coupled with separate effects tests and analysis are needed to eliminate questions concerning the credibility of small scale models.

5.4 Credibility of Test Results

5.4.1 Introduction

In the development of a model test program, there are several requirements that lead to credible tests results.

- A sound theoretical basis for the model design, the test loading, and interpretation of test data must be established.
- The model scale must be chosen such that the characteristics that are to be determined in the test are accurately represented in the model.
- The scale model must be fabricated with sufficient care to insure that failure mechanisms which are not present in the prototype are not introduced into the model.
- The test methods must be such that failure mode(s) are not introduced or eliminated in the structure as a result of test procedures or test facilities.
- Instrumentation of sufficient accuracy and sensitivity to record all phenomena of interest must be employed.
- Adequate analytical support must be incorporated into all phases of the experimental plan.
- Repeatability of experimental results must be demonstrated.

Each of the above items form an integral part of this program.

5.4.2 Required Number of Tests

An important element of an experimental program is repetition of experiments to prove reproducibility of results. It cannot be overemphasized that reproducibility of results must be demonstrated before conclusions can be drawn. For each individual experiment, a sound statistical treatment of all measured data (both control and response) is required prior to considering the experiment complete. The results of this complete experiment must then be demonstrated as reproducible by repetition; i.e., the experiment must be conducted again in as nearly the same manner as possible, limited only by the experimenter's ability and random factors.

Random factors will always be present. They may be associated with model fabrication, load application, response measurement, material variability, or other variables. The variations between experiments will produce a scatter in the results of seemingly identical experiments. Therefore, the results are reproducible if the measured response in all attempts fall within an acceptable scatter band. Careful error analysis is necessary to determine an acceptable scatter band. In complicated experiments with multiple responses it is possible that only some responses will be reproducible.

While the principle of reproducibility must be followed, restraint with regard to economics and time factors must be exercised. Therefore, this program will begin with two or three experiments at the smallest scale. If these give reproducible results in all principal responses (notably failure), the experiments at the next larger scale will then begin.

In the event that reproducibility cannot be demonstrated at the smaller scale, the larger scale experiments will be deferred pending resolution of the problem. Additional smaller scale experiments may be required before the larger tests are conducted.

The above procedure will insure that results obtained represent behavior to be expected in prototype containments and do not merely represent some anomaly resulting from model fabrication, experimental techniques, or other variables. For those features that are included in the scale models, conducting several reproducible model tests at different scales yields a higher level of confidence in the results than would be obtained from conducting a single test on an actual full-size containment structure.

References

[1]W. E. Baker, P. S. Westine, and F. T. Dodge. *Similarity Methods in Engineering Dynamics* (Rochelle Park, NJ: Hayden Book Company, Inc., 1973).

[2]C. A. Anderson, R. C. Dove, and R. L. Rhorer, *A Proposal for a Seismic Facility for Reactor Safety Research*, LA-NUREG-6388-P (Los Alamos, NM: Los Alamos Scientific Laboratory, July 1976).

[3]G. Murphy, *Similitude in Engineering* (New York: Roland Press, 1950).

[4]H. L. Langhaar, *Dimensional Analysis and Theory of Models* (New York: John Wiley & Sons, Inc., 1951).

[5]"Models of Concrete Structures - State-of-the-Art," Report No. ACI 444-79, *Concrete International*, January 1979.

6. Experimental Alternatives

6.1 Introduction

Useful experiments on containment models require accurate knowledge and control of the applied loadings and careful measurement of the resulting model response. Loading techniques for each of the three categories (static pressurization, dynamic pressurization, and seismic) considered for this program are discussed in Section 6.2. Suitable facilities are considered in Section 6.3 and Appendix B. Instrumentation and techniques for measuring model response are outlined in Section 6.4.

6.2 Experimental Techniques

6.2.1 Static Pressure Loading

Of the three types of loadings discussed in this plan, procedures for static pressurization are the most straightforward. Several methods of loading are available. Pressurization can be accomplished pneumatically with a gas such as nitrogen, or hydraulically with water. Although pressurization with water is safer, it has certain disadvantages for this program. The static-head differential from top to bottom of a large model would be significant (on the order of 10 psi for a ⅛-scale model). Also, the leakage characteristics of water are different from those of air; as noted previously, leakage is an important response parameter.

Some of the hazards associated with pneumatic pressurization can be minimized by the use of a gas (such as nitrogen) that will not support combustion. However, some hazard will still exist since the large potential energy stored in a gas, which is highly compressible, can lead to missile generation if a sudden rupture occurs. Therefore, a remote or protected site is required if gas pressurization is used. We believe that the inconvenience of protected siting is a small penalty to pay to achieve the realism offered by gas pressurization over hydraulic loading. A conceptual design of a suitable test setup is presented in Figure 6.2-1.

Testing will include an initial pressurization to 1.15 of design pressure and a leak check at design pressure. In addition to simulating the pressure testing of actual containments, this pretest will provide an opportunity for a function check of test facilities, instrumentation, and pressure-sealing interfaces between the model and the test equipment. Also, checks for manufacturing defects in the model can be conducted during this pretest period.

Pressurization to failure will be conducted in steps so that the rate of leakage or lack of measurable leakage can be determined. The pressure will be slowly raised to a specified value; the input will be shut off; the pressure will be monitored for a length of time; and the test will continue to the next pressure level. If high-leakage rates are encountered before a rupture, the specified pressure level will be maintained, using a pressure control device; the mass flow rate into the model will be measured.

6.2.2 Dynamic Pressure Loading

The dynamic pressure loadings of interest in this program are characterized by a pressure spike of large magnitude and short duration (on the order of a few milliseconds), followed by a period of transients and reflected pulses that are superimposed on the residual pressure level. Quantities that can have a profound effect upon the structural response include the duration and peak magnitudes of the initial spike, the magnitude of the initial and final static-pressure levels, and the frequency and magnitude of the reflected spikes. These quantities, which may vary spatially in the structure, are scenario and structure dependent. Indeed, defining all these quantities with confidence for a given hydrogen detonation in a prototype containment may not be possible. Therefore, experimentally modeling a particular hydrogen accident in a prototype containment is not possible at this time. In any case, the goal of determining the "ultimate capacity" of the containment could not be reached if specific scenarios were replicated. Thus the dynamic loadings used in this program will have characteristics similar to those of hydrogen-detonation accidents, but specific events will not be replicated.

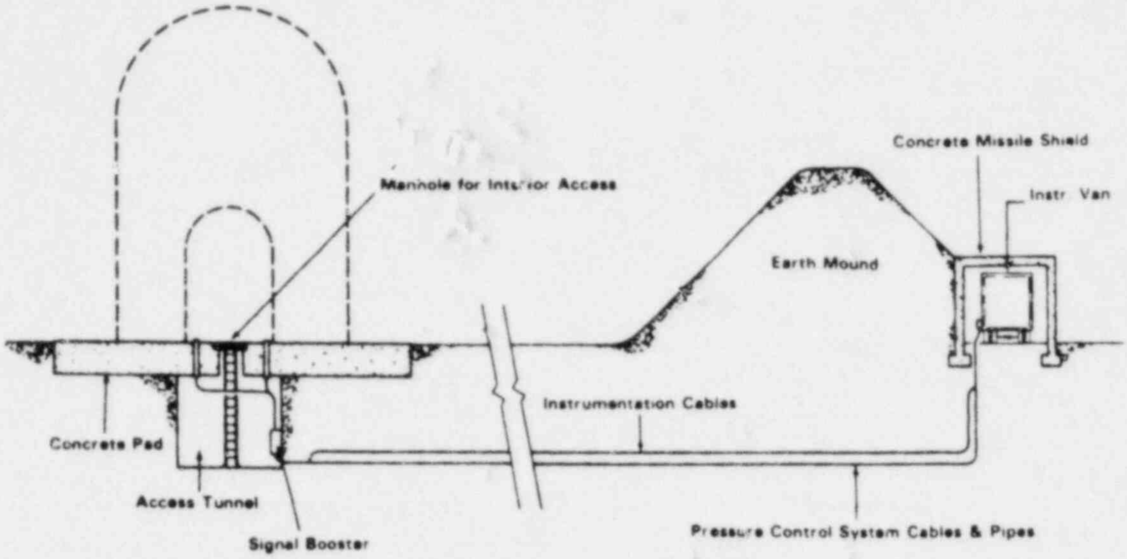

Figure 6.2-1. Test Facility for Static Pressurization

A single loading that ruptures a model provides little insight into the capacity of the model. The model experiments with dynamic pressurization will, therefore, begin with tests involving small* pressure pulses and progress to tests with increasingly larger pulses; thus the ultimate capacity of the containment can be established. Before dynamic tests on models begin, a loading procedure that is reliable and that will allow the formation of pressure pulses of varying and predictable intensity must be developed. A pulse-calibration chamber, as illustrated in Figure 6.2-2, will be required. The chamber will allow experimentation with a variety of gaseous mixtures or solid explosives; it will be instrumented sufficiently to allow a complete definition of the pressure pulse. Different size chambers may be required so that scaling effects for the test technique can be determined.

Detailed experimental techniques for dynamic pressurization can not be established until a loading technique has been developed. Experience gained from static tests of containment models will be useful in developing the experimental techniques. It is clear, however, that remote or protected sites are required. It is anticipated that the sites used for static pressurization will (with minor modification) be suitable for the dynamic pressurization tests.

*The size of a pressure pulse can be changed by varying the magnitude, duration, or the shape. Until further investigation is conducted, it is assumed that the pulse can be characterized by the total impulse.

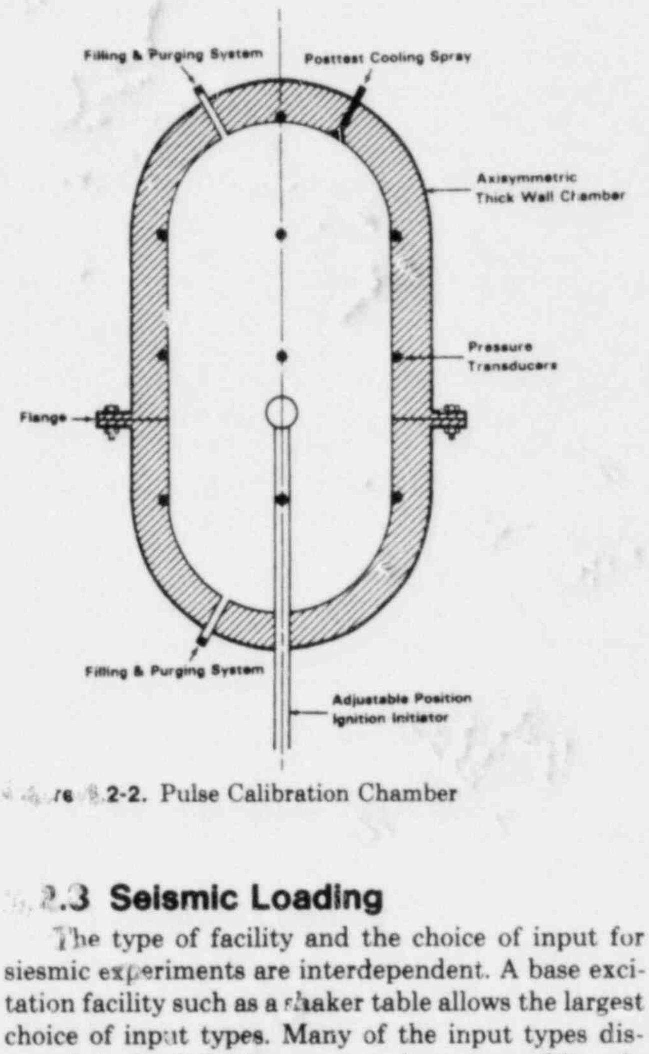

Figure 6.2-2. Pulse Calibration Chamber

6.2.3 Seismic Loading

The type of facility and the choice of input for siesmic experiments are interdependent. A base excitation facility such as a shaker table allows the largest choice of input types. Many of the input types discussed in the following paragraphs are possible only with a shaker facility.

6.2.3.1 Seismic Input Definition

The input used for the seismic loading of containment models must be representative of actual earthquakes and, more important perhaps, representative of the input specifications for a seismic design. Obtaining the proper input on models of appropriate scale will be difficult.

The current design practice for defining seismic excitation is to use a design response spectrum that has been normalized to a maximum ground acceleration.[1] The maximum ground displacement is proportional to the maximum ground acceleration (presently 36 in. for 1.0 g for site-independent spectra).[2] Established procedures exist for normalizing the spectrum to any maximum ground acceleration/displacement.[2] However, a unique transformation between a response spectrum and base input to the structure does not exist. The duration of the earthquake is not specified by a response spectrum; strong-motion duration is particularly important when inelastic response and cyclic degradation occur, as they will in tests to failure of containment models.

Several procedures exist for generating artificial time histories that are compatible with a specified response spectrum (see Reference 3 for a short review). A single time-history (either artificial or real) could be specified for this program; however, to determine the "ultimate capacity" of the containment model, tests of varying intensity must be conducted. The acceleration at each point in time of a base accelerogram can be amplified by a constant factor. Alternatively, the response spectrum can be scaled according to established procedures and time-histories of varying intensity generated.

An alternative to using time-history input is to specify the power-spectral density, which can be approximated from the response spectrum.[3] The spectral density can be multiplied by a time function to account for the nonstationary nature of an actual earthquake; by changing the magnitude of the time function, the intensity of the earthquake can be varied for different tests. Unfortunately, the automatic application of a time function that modifies the spectral density is not a standard feature on existing shaker facilities.

Other types of input are possible but they can not be directly correlated with design spectrum. If sinusoidal-dwell tests are conducted, the shaker facility must have sophisticated output control because, for inelastic response with cyclic degradation, the frequency of the input must be adjusted continually to obtain a pseudo-resonance condition. Although dwell tests will yield insight into the lateral capacity of the containment model, relating the output to seismic response may not be possible. Quasi-static and impulsive tests can also be used to investigate the capacity of the structure to one or more lateral loadings. However, quasi-static tests will not produce a distribution of forces that is the same as the inertial forces in an earthquake.

6.2.3.2 Seismic Loading Devices

Two classes of devices are used for earthquake loadings: base excitation and forcing devices. The latter category includes a variety of eccentric mass devices, hydraulic actuators, cutters, and pulsers that apply time-varying (often sinusoidal) forces to one or more points on the structure while the base is constrained. Although these devices are suitable for investigating the modal properties of elastic structures they are generally unsuitable for this program because the manner and location of the input affects the output and the eventual failure mechanism.

For base excitation, two types of devices are currently used: shaker facilities and explosives. It is not presently clear which is more suitable for this program or whether another technique must be developed.

Shaker tables can be controlled to provide time-history, power-spectral-density, and sinusoidal dwell inputs; however, each shaker facility has kinematic and force limitations that limit the size of the model and/or the magnitude of input that can be used. As will be discussed in Section 6.3, no existing shaker facility has the capacity to fail the medium to large-scale models anticipated for this program. Another disadvantage of shaker tables is that, for the larger models, it may be necessary to construct the models adjacent to or on the shaker table.

Although high-intensity ground motions are possible with buried explosives, the durations of large excitations to date have been very short.[4] A technique developed by SRI International[5] uses relatively small amounts of explosives and sequencing of arrays may be a feasible approach to extending the duration of excitation. Another disadvantage of explosive testing is that remote siting is required for testing with buried explosives. To limit soil damping, it may be desirable to anchor the foundation of the model to rock beds.

Underground nuclear tests also provide significant ground motions, but the intensity at sites that have suitable frequency content is too low.[6] Also, only one event is available at each test site and logistics make testing very difficult.[7]

Additional study is required before an input and loading device for the earthquake experiments can be selected or developed for this program.

6.3 Survey of Test Facilities

A limited survey has been conducted to determine the capabilities of existing test facilities. Facilities for conducting pressurization experiments and seismic experiments are discussed in Sections 6.3.1 and 6.3.2, respectively. Results of a survey of existing full-size structures that might be suitable for use as test specimens are contained in Appendix B.

6.3.1 Pressurization Facilities

As noted previously, the static and dynamic pressurization tests are hazardous and will require remote or protected facilities. Facilities that were designed for testing components using high explosives are available at Sandia, Albuquerque. Models on the order of 1/50 to 1/25 scale can be accommodated. Indoor laboratory facilities can be used for 1/50-scale models. Existing outdoor facilities consisting of five-sided "bang-boxes" may be used for models up to 1/25-scale. Instrumentation bunkers, communications facilities, electric power, and access roads are available at these sites.

Models larger than 1/25 scale can not be accommodated in the existing Sandia test facilities; however, models to approximately 1/8 scale could be constructed and tested in Sandia's Area III complex. Facilities such as bunkers, electrical power, and access roads must be provided. Such facilities have frequently been constructed for other Sandia tests.

If a 1/4-scale model is tested, a site more remote than Area III is required. Candidate test areas are Sandia's Coyote Canyon Test Area located approximately 12 miles southeast of the laboratory in the Manzano Mountains and Sandia's Tonopah Test Range located near Tonopah, Nevada. Both of these areas would require site work, but support facilities and experienced personnel are available.

6.3.2 Seismic Facilities

Results of a survey of existing shaker facilities (from Reference 8, with additions and modifications) are presented in Table 6.3-1. Minimum input requirements for replicating a design-response spectrum from Regulatory Guide 1.60[2] are presented in Table 6.3-2. A 1.0-g normalization has been selected to demonstrate trends; however, to be reasonably confident of inducing failure in a containment, a capability to replicate a full-scale spectrum approaching 4 g is probably required (4 g at full-scale corresponds to 32 g at 1/8 scale when replica scaling is used). Therefore, although precise facility requirements for this program have not been formulated, it is clear that no single existing or planned facility can meet the requirements for the range of scales discussed in this plan. Even the impressive facility under construction at Tadotsu Engineering Laboratory in Japan may be incapable of failing a 1/4- or 1/8-replica scale model.[9,10]

A facility large enough to use a base-excited, response-controlled, hydraulically actuated table for tests to failure at 1/4 or 1/8 scale is probably within current technology; however, such a facility does not presently exist and would cost several hundred million dollars to construct. The Japanese Tadotsu facility, which has been several years in construction, was built at a cost between $80M and $200M. Therefore, it is anticipated that shaker tables will not be used for the large scale tests in this program. For small scale experiments, some existing shaker facilities may be suitable; additional investigation is needed to determine precise facility requirements and to match available facilities with these requirements.

Table 6.3-1. Characteristics of Seismic Shaker Test Facilities

Facility		Control Axes	Table Size (ft)	Max Weight Test Item (klb)	Max 2-g Weight Test Item (klb)	Max Displ. (in.)	Max Force (klb)	Approx Freq Maximum (Hz)
Tadotsu Eng Lab		2	49x49	2,210		3.9 V 7.9 H	7,280 V 6,600 H	30
Corps of Eng Construction Eng Research Lab		2	12x12	810	13	2.75 V 5.75 H	810 V 450 H	200
Hill AFB	A	2	262 ft²		87		200	55
	B	2	108 ft²		4.5	5	100	500
Univ. of CA		2	20x20	120	30	4.0 V 12.0 H	100 V 210 H	50
Wyle Labs	A	2	4x4	3		3 (both)	25 V	500
	B	1	6.7x8.3	12		3	80	250
	C	1	5x17	40	40	5.5	150	500
	D	2	9x12	10		8 (both)	31	70
	E	2	8x8	6		9 V 12 H	36 V 29 H	70
Westinghouse Astronuclear Lab		1	8x16	180	20	20	55	33
Acton		2	36 ft²		8	26	26	200
SNL		1	4x6	10		8	40	0.1 - 500
White Sands	A	1	5x5	24	12	10	24	>100
Missile Range	B	1	None	40	20	4	40	>100
Southwest Research Inst.		2	4x4	6		6.6 V 6.6 H	20 V 10 H	100
Battelle		2	28 ft²		1.5	4	6	50
Systems Controls		1	35 ft²		2	6	2	33

Table 6.3-2. Minimum Input Requirements to Replicate a 1.0-g Horizontal Response Spectrum

Scale of Replica Model	Acceleration[1] (g)	Displacement[1] (in.)	Maximum Frequency[2] (Hz)
Full	1.0	36	33
1/4	4.0	9.0	130
1/8	8.0	4.5	260
1/16	16.0	2.2	530
1/32	32.0	1.1	1100

[1]Full scale site-independent spectrum normalized to 1.0 g from Req. Guide 1.60.[2]

[2]Maximum required frequency for full scale is assumed to be the lowest frequency at which no amplification of ground motion is specified in Reference 2.

6.4 Instrumentation

Instrumentation of the modeled containment structure has three purposes. The instrumentation must

- Detect failure of the containment structure
- Provide adequate data to evaluate the validity of computer codes to predict the static and dynamic response of the containment structure
- Provide accurate data to evaluate the actual applied loads for correlation with the responses.

High demand will be placed upon the instrumentation, because both elastic and plastic deformation of the structure must be measured. In local regions of the structure, the tests may completely destroy the integrity of the structure.

Failure-detection instrumentation may include photographic/video recording for visual reconstruction; acoustic monitoring for indications of leaking or cracking; pressure and mass-flow monitoring for leak-rate determination; sampling of trace elements for leak detection; and kinematic measurements for detection of sudden changes that imply failure. These and other kinematic measurements will establish the response of the model to the pressure loading. Response measurements will use displacement and strain gages as the fundamental transducer types with other techniques such as birefringent coatings and interoferrometric or laser-ranging systems as auxiliary devices to permit certain areas to be investigated more thoroughly or more conveniently.

Pressure measurement for accurate monitoring of loading will be required. Accurate pressure measurements, and even some response measurements, will necessitate temperature measurements in suitable locations.

All the foregoing measurements will be recorded in the appropriate way, either on film, magnetic tape, or as digitized data for computer storage. Thorough and consistent error analysis will be performed on each piece of measured data so that the results derived from the data may be stated precisely with known errors and confidence.

Data measured and recorded will be reduced to engineering units, the appropriate errors will be associated with the measurement, and the results presented in a form that best describes the relationship between fundamental quantities. The confidence and accuracy associated with each relationship will be established and stated.

References

1"Seismic Analysis and Design," Ch 5 of *Structural Analysis and Design of Nuclear Power Plant Facilities*, Manual No. 58 (New York: American Society of Civil Engineers, 1980).

2"Design Response Spectra for Nuclear Power Plants," *Regulatory Guide 1.60* (Washington, DC: US Atomic Energy Commission, December 1973).

3A. Preumont, "A Method for Generation of Artifical Earthquake Accelerograms," *Nucl Eng and D*, 59, 1980, pp 357-368.

4C. J. Higgins, R. L. Johnson, and G. E. Triandafilidis, "The Simulation of Earthquake-Like Ground Motions With High Explosives" (Albuquerque, NM: University of New Mexico, July 1978).

5H. E. Lindberg, G. R. Abrahamson, and J. R. Bruce, untitled paper in *Proceedings, Sixth National Meeting of the Universities Council for Earthquake Engineering Research* (Urbana-Champaign, IL: University of Illinois, May 1-2, 1980) pp 174-176.

6T. C. Bache, W. E. Farrell, and D. G. Lambert, *Block Motion Estimates From Seismological Observations of Mighty Epic and Diablo Hawk*, DNA 5007F (La Jolla, CA: System Science and Software, June 1979).

7S. L. Blouin, J. L. Bratton, and E. H. Bultmann, *Earthquake Ground Motion Simulation Study*, EPRI NP-1387, TPS 79-734 (Albuquerque, NM: Civil Systems Incorporated, April 1980).

8C. A. Anderson, R. C. Dove, and R. L. Rhorer, *A Proposal for a Seismic Facility for Reactor Safety Research*, LA-NUREG-6388-P (Los Alamos, NM: Los Alamos Scientific Laboratory, July 1976).

9T. Ohmori, and N. Kobayashi, "Large-Scale High Performance Vibration Table in Japan," unpublished paper, 1980.

10Y. Ohsaki, "New Approaches to a Seismic Design and Testing in Japan," *Nuclear Engineering International*, January 1980, pp 47-49.

7. Summary and Preliminary Plans

Three different types of containment structures will be considered in this program:

- Hybrid steel (ice condenser and MKIII)
- Reinforced concrete
- Prestressed concrete

The specific loading conditions to be considered are

- Static internal pressurization
- Dynamic internal pressurization
- Seismic loadings

Although nine combinations of containment types and loading conditions can be postulated, not all combinations will be investigated. Funding constraints, NRC priorities, and other factors will dictate priorities and an order of investigation. At the present time it is anticipated that the static pressurization of hybrid-steel containments will be investigated first. Static pressurization is the most straightforward of the loading conditions; hybrid-steel containments have the lowest design pressure and may be the most susceptible to failure caused by a hydrogen deflagration/detonation.

Additional work is necessary to develop loading techniques for the dynamic-pressurization and seismic experiments. Indeed, a meaningful seismic test to destruction of a concrete containment model may not be possible with existing techniques and equipment. Compromises in experimental objectives and model configurations will be considered along with a further investigation of testing techniques.

The experimental portion of the program will use scale models. Many features in the models will replicate features in full-size containments. Adequate modeling will be used where necessary because of economic constraints. At present, scales of between ⅛ and 1/32 are anticipated for the steel models. Models of about 1/10 scale are anticipated for the concrete models. Concrete presents modeling difficulties at smaller scales; further investigation will be conducted prior to concrete-model design and construction.

Analytical effort will parallel the experiments. Preliminary analyses will be conducted to gain insight into the nonlinear behavior of containments at loadings approaching those required to cause rupture and to identify problem areas for future analytical and experimental work. After testing of a configuration is completed, the analytical and experimental results will be compared in an attempt to qualify the analytical methods for structures and loadings of the type tested. Computer code modifications will be undertaken if necessary. In addition to investigating the adequacy of complex computer codes, the applicability of hand-calculations based upon the properties and loading of gross sections will be investigated.

Detailed plans for the analysis and testing of each combination of loading and containment type will be formulated prior to initiation of testing of that configuration. Modifications and additions to the work presented herein will be reported as appropriate.

APPENDIX A

Reactor Containment Structures

A1 Classification of Containment Structures

This appendix contains a tabulation of 181 LWR containment structures. The basis for the tabulation is the set of land-based light-water commercial-power reactors listed in Reference 1 as either operating reactors or docketed proposed plants. The containment structures are cataloged in Section A3 by

- LWR Type (BWR or PWR)
- Type of Pressure Suppression System
  MARK I
  MARK II
  MARK III
  Pre-MARK
  Ice-Condenser
  Subatmospheric Operation
  Atmospheric Operation Without Pressure Suppression
- Containment Structure Configuration
- Containment Structure Construction Material
  Steel
  Reinforced Concrete
  Prestressed Concrete
  Hybrid Steel (steel shell and reinforced concrete base)*

The information contained in Section A3 is summarized in Tables A1-1 through A1-5.

The primary sources of information for containment structure classifications were the plant Final Safety Analysis Reports (FSAR), if available; if not, the plant Preliminary Safety Analysis Reports (PSAR). In a number of cases (approximately 85 out of 181), other sources[2-10] were used when FSAR and PSAR information was not available or could not be used. The scheduled completion dates for units that are in the planning or construction phase were obtained from Reference 11 and have a June 30, 1980 base date. The subsequent cancellation of two plants (North Anna 4 and Forked River) is also noted in Section A3.

*The terms hybrid steel and free-standing steel are used interchangeably.

Table A1-1. Steel BWR Containments

	Light-Bulb/Torus MARK I	Other Steel MARK II & Pre-MARK	Freestanding Steel MARK III
Operating plants	20	4 Humbolt Bay is in this category	
Plants expected to be placed in commercial operation between 6/30/80 and 12/31/82	1		
Plants expected to be placed in commercial operation after 12/31/82	2	1	8
Plants which are planned, but have the commercial operation date listed as indefinite			7

*The terms hybrid steel and free-standing steel are used interchangeably.

Table A1-2. Concrete BWR Containments

	Deformed-Bar Reinforcing MARK I	Deformed-Bar Reinforcing MARK II	Deformed-Bar Reinforcing MARK III	Prestressed Reinforcing MARK II
Operating plants	2			
Plants expected to be placed in commercial operation between 6/30/80 and 12/31/82		1	2	3
Plants expected to be placed in commercial operation after 12/31/82		5	1	1
Plants which are planned, but have the commercial operation date listed as indefinite			3	

Table A1-3. Prestressed Concrete PWR Containments (atmospheric)

	Six Buttress Design	Three Buttress Design		
	Shallow Dome	Shallow Dome	Hemi Dome	Other Designs
Operating plants	15	5	1	3
Plants expected to be placed in commercial operation between 6/30/80 and 12/31/82		2	2	
Plants expected to be placed in commercial operation after 12/31/82		9	14	3 Forked River Cancelled
Plants which are planned, but have the commercial operation date listed as indefinite			3 Erie 1 & 2 Cancelled	

Table A1-4. Reinforced Concrete PWR Containments

	Deformed Bar Reinforced Concrete		
	Ice Condenser	Subatmospheric	Atmospheric
Operating plants	2	4	5
Plants expected to be placed in commercial operation between 6/30/80 and 12/31/82			4
Plants expected to be placed in commercial operation after 12/31/82		6	9
Plants which are planned, but have the commercial operation date listed as indefinite		1 North Anna 4 Cancelled	2 New England 1 & 2 Cancelled

Table A1-5. Steel PWR Containments

	Freestanding (ice condenser)	Sphere (atmospheric)	Cylinder With Domed Closures (atmospheric)
Operating plants		3	5
Plants exp...ed to be placed in commercial operation between 6/30/80 and 12/31/82	6		1
Plants expected to be placed in commercial operation after 12/31/82	2	4	5
Plants which are planned, but have the commercial operation date listed as indefinite		4	

A2 Accuracy of Classifications

The information contained in Section A3 was obtained from the sources referenced above. However, there were conflicts in the classification of some of the containments. Where possible the FSAR or PSAR on the plant in question was reviewed to determine containment classification; in some cases the plant AE was contacted[9] to determine containment classification. It is our opinion that Section A3 is generally correct, but may, in a few cases, be in error. The data sample is large enough (181 containments) so any error that would result in changing a plant containment classification would not significantly alter the data base presented in Section A3 nor the conclusions derived therefrom.

A3 Containment Structure Catalog

A3.1 BWR Commercial Reactor Containments

MARK I Containments

Steel containment structures in the light-bulb/torus configuration.

Licensed Plants
Arnold
Browns Ferry 1, 2, and 3
Cooper
Dresden 2 and 3
Fitzpatrick
Hatch 1 and 2
Millstone Point 1
Monticello
Nine-Mile Point 1
Oyster Creek
Peach Bottom 2 and 3
Pilgrim 1
Quad Cities 1 and 2
Vermont Yankee

Future Plants	Scheduled Completion
Fermi 2	1982
Hope Creek 1	1986
Hope Creek 2	1989

Deformed-bar reinforced-concrete containments with steel liner in both the dry well and torus-shaped suppression chamber.

Licensed Plants
Brunswick 1 and 2

Future Plants
None

MARK II Containments

Deformed-bar reinforced-concrete with steel-domed closure cap, steel liner, and flat base.

Truncated-cone and vertical-cylinder body.

Licensed Plants
None

Future Plants	Scheduled Completion
Limerick 1	1985
Limerick 2	1987
Nine Mile Point 2	1986
Susquehana 1	1982
Susquehana 2	1983

Truncated-cone body

Licensed Plants
None

Future Plants	Scheduled Completion
Shoreham	1983

Prestressed concrete body, steel cap, steel liner, and flat base.

Truncated-cone and vertical-cylinder body.

Licensed Plants
None

Future Plants	Scheduled Completion
La Salle 1	1981
La Salle 2	1982
Zimmer 1	1981

Truncated-cone body

Licensed Plants
None

Future Plants	Scheduled Completion
Bailly 1	1987

Steel containment with truncated-cone and vertical-cylinder body, domed-closure cap, and ellipsoidal base.

Licensed Plants
None

Future Plants	Scheduled Completion
Washington 2	1983

MARK III Containments

Deformed-bar reinforced-concrete vertical cylinder, hemispherical dome, flat base, and steel liner.

Licensed Plants
None

Future Plants	Scheduled Completion
Clinton 1	1982
Clinton 2	Indefinite
Grand Gulf 1	1982
Grand Gulf 2	1986
Skagit 1	Indefinite
Skagit 2	Indefinite

Free-standing steel cylinder and shallow dome with deformed-bar-reinforced base. Base has a steel liner.

Licensed Plants
None

Future Plants	Scheduled Completion
Allens Creek	1987
Black Fox 1	1985
Black Fox 2	1988
Hartsville 1	1986
Hartsville 2	1987
Hartsville 3	Indefinite
Hartsville 4	Indefinite
Montague 1	Indefinite
Montague 2	Indefinite
Perry 1	1984
Perry 2	1988
Phipps Bend 1	Indefinite
Phipps Bend 2	Indefinite
River Bend 1	1984
River Bend 2	Indefinite

Other BWR Containments (Pre-MARK)

Steel containment with a hemispherical dome, vertical-cylinder body, and ellipsoidal base.

Licensed Plants
La Crosse

Free-standing steel sphere

Licensed Plants
Big Rock Point
Dresden 1

Steel dry well, deformed-bar-reinforced concrete wet well with steel liner.

Licensed Plants
Humboldt Bay

A3.2 PWR Commercial Reactor Containments

Ice Condenser Containments

Deformed-bar reinforced-concrete vertical cylinder, hemispherical dome, and flat base with steel liner.

Licensed Plants
D.C. Cook 1 and 2

Future Plants
None

Free-standing steel cylinder and hemispherical dome with deformed-bar reinforced concrete base and steel-base liner.

Licensed Plants
None

Future Plants	Scheduled Comnpletion
Catawba 1	1983
Catawba 2	1985
McGuire 1	1981
McGuire 2	1982
Sequoyah 1	1980
Sequoyah 2	1981
Watts Bar 1	1981
Watts Bar 2	1982

Subatmospheric Containments
Deformed-bar reinforced-concrete cylinder, hemispherical dome, flat base, and steel liner.

Licensed Plants
Beaver Valley 1
North Anna 1
Surry 1 and 2

Future Plants	Schedule Completion
Beaver Valley 2	1986
Jamesport 1	1988
Jamesport 2	1990
Millstone 3	1986
North Anna 2	1981
North Anna 3	1987
North Anna 4	Cancelled

Atmospheric Containment Structures (without pressure-suppression features)

Deformed-bar reinforced-concrete cylinder, hemispherical dome, flat base, and steel liner.

Licensed Plants
Haddam Neck (Connecticut Yankee)
Indian Point 2 and 3
Main Yankee
Salem 1

Future Plants	Scheduled Completion
Comanche Peak 1	1981
Comanche Peak 2	1983
Diablo Canyon 1	1981
Diablo Canyon 2	1981
Harris 1	1985
Harris 2	1988
Harris 3	1994
Harris 4	1992
New England 1	Cancelled
New England 2	Cancelled
Salem 2	1981
Seabrook 1	1983
Seabrook 2	1985
Washington 1	1985
Washington 4	1986

Concrete vertical cylinder with prestressed vertical reinforcement and deformed-bar hoop reinforcement. Deformed-bar reinforced-concrete hemispherical dome and flat base. Complete steel-lined structure.

Licensed Plants
Ginna
Robinson 2

Prestressed concrete vertical cylinder and dome, deformed bar reinforced flat base, and steel liner.

No buttresses, shallow dome, and diagonal reinforcing pattern.

Licensed Plants
Fort Calhoun

Three-buttress design

Hemispherical dome

Licensed Plants
Trojan

Future Plants	Scheduled Completion
Calloway 1	1982
Calloway 2	1987
Erie 1	Cancelled
Erie 2	Cancelled
Greenwood 2	1990
Greenwood 3	1992
Palo Verde 1	1983
Palo Verde 2	1984
Palo Verde 3	1986
Pebble Springs 1	1988
Pebble Springs 2	1990
Pilgrim 2	Indefinite
South Texas 1	1984
South Texas 2	1986
Sterling	1988
Summer	1981
Vogtle 1	1985
Vogtle 2	1988
Wolf Creek	1983

Shallow dome

Licensed Plants
Arkansas 1 and 2
Farley 1
Millstone 2
Rancho Seco

Future Plants	Scheduled Completion
Braidwood 1	1985
Braidwood 2	1986
Byron 1	1983
Byron 2	1984
Farley 2	1981
Marble Hill 1	1986
Marble Hill 2	1987
Midland 1	1984
Midland 2	1983
San Onofre 2	1981
San Onofre 3	1983

Four-buttress design with shallow dome

Licensed Plants
None

Future Plants	Scheduled Completion
Bellefonte 1	1983
Bellefonte 2	1984
Forked River	Cancelled

Six-buttress design with shallow dome

Licensed Plants
Calvert Cliffs 1 and 2
Crystal River 3
Oconee 1, 2, and 3
Palisades
Point Beach 1 and 2
Three Mile Island 1 and 2
Turkey Point 3 and 4
Zion 1 and 2

Spherical-Steel Containments

Licensed Plants
Indian Point 1
San Onofre 1
Yankee Rowe

Future Plants	Scheduled Completion
Cherokee 1	1990
Cherokee 2	1992
Cherokee 3	Indefinite
Perkins 1	Indefinite
Perkins 2	Indefinite
Perkins 3	Indefinite
Yellow Creek 1	1985
Yellow Creek 2	1988

Steel containment with a hemispherical dome, vertical-cylinder body, and ellipsoidal base.

Licensed Plants
Davis Besse 1
Kewannee
Prairie Island 1 and 2
St. Lucie 1

Future Plants	Scheduled Completion
Davis Besse 2	1988
Davis Besse 3	1990
St. Lucie 2	1983
Washington 3	1986
Washington 5	1987
Waterford 3	1982

References

[1]*Nuclear Safety,* Vol. 21, No. 3 (Oak Ridge, TN: Nuclear Safety Information Center, Oak Ridge National Laboratories, May-June 1980) pp 399-403 and 429-434.

[2]*Nuclear Containment Vessels,* Bulletin No. 8510 (Oak Brook, IL: Chicago Bridge and Iron Co.).

[3]*Commercial Nuclear Power Plants,* Ed 12 (Rockville, MD: NUS Corporation, January 1980).

[4]M. Subudhi et al, *Seismic Review Table,* NUREG/CR-1429 (Upton, NY: Department of Nuclear Energy, Brookhaven National Laboratory, May 1980).

[5]J. D. Stevenson, *Containment Structures for Pressurized Water Reactor Systems, Past, Present and Future - State of the Art* (Pittsburgh, PA: Westinghouse Nuclear Systems).

[6]SECY-80-107-A NRR, Memo dated 4/22/80, H. Denton to The Commissioners. Subject: "Proposed Interm Hydrogen Control Requirements."

[7]F. A. Heddleton, *Design Data and Safety Features of Commercial Nuclear Power Plants,* ORNL/NSIC-96 (Oak Ridge, TN: Oak Ridge National Laboratories, June 1976).

[8]Letter, T. J. Ahl, Chicago Bridge and Iron Co., to W. A. Von Riesemann, Sandia National Laboratories. Subject: "Containment Vessels for U.S.A. Nuclear Plants," dtd 12/5/80.

[9]Personal communication, W. S. Mott, EBASCO, with A. W. Dennis, Sandia National Laboratories. Subject: "Containment on Harris 1, 2, 3, and 4," 12/10/80.

[10]Personal Communication, J. Ucciferro, United Engineers and Constructor, with W. A. Von Riesemann, Sandia National Laboratories. Subject: "MARK I Containments."

[11]"World List of Nuclear Power Plants, Operable, Under Construction, or on Order (30 MWe and Over) as of June 30, 1980," *Nuclear News,* August 1980, pp 102-107.

APPENDIX B

Full-Size Containment-Like Structures

B1 Introduction

A brief search was conducted to identify full-size, containment-like structures that might be suitable and available for use as test specimens in this program. The search consisted of telephone inquiries to a number of potential sites throughout the United States. No facilities were visited. The most suitable structures for use in parts of the containment tests may be (1) the Experimental Boiling Water Reactor (EBWR) facility located at Argonne National Laboratory; (2) the decommissioned Saxton PWR facility located in Saxton, PA; and (3) the decommissioned Carolinas Virginia Tube Reactor located in Parr, SC. Data obtained from these and the other facilities for which inquiries were made is presented on the following pages.

B2 List of Facilities

B2.1 Argonne National Laboratory

Argonne National Laboratory
Argonne, IL
Principal Contact: John Honekampe
Phone: 8-972-4483

Facilities:

CP-5 Nuclear Reactor Facility

- Not in use at present
- Reactor still in building
- Building has high bay with dome
- Reactor is 5-MW pool type
- Geometry not known at present
- Nearest structure 500 ft away

EBWR Facility (Experimental Boiling Water Reactor)

- Reactor decommissioned and removed
- Building details
  Cylindrical steel shell
  Diameter: 80 ft
  Height: 119 ft
  56 ft of building is below ground level
  Bottom and sides are 5/8-in.-thick steel
  Dome is 3/8-in.-thick steel
- Some small amount of Pu containment
- Vibration laboratory located nearby

B2.2 Hanford Engineering Development Laboratory

Hanford Engineering Development Laboratory
Hanford, Washington
Principal contact: Roy E. Dunn
Phone: 8-444-7258

Facilities:

PRTR Building (Plutonium Recycle Test Reactor)

- Reactor not operational
- Building now used for temporary laboratory space
- Building has steel dome

200 Area Building

- Building has domed-shape roof
- Steel tank inside of building
- Size of building is "small"

B2.3 Idaho Operations Office

Idaho Operations Office
Idaho Falls, Idaho
Principal contacts: Ray McCord, Stu Milam
Phone: 8-583-2466
Phone: 8-583-1618

Facilities:

EOCR Building (Experimental Organic Coolant Reactor)

- Building will not hold pressure
- Large steel vessel in building
- Presently not in use

B2.4 Los Alamos Scientific Laboratory

Los Alamos Scientific Laboratory
Los Alamos, NM
Principal contact: Jim Jackson
Phone: 313-7-1211

Facilities:

UTREX Reactor Building

- Reinforced concrete with steel liner
- Presently used for laboratory space
- Personnel building located nearby

TA-55 Pu Processing Building

- In use full time

B2.5 Nevada Test Site

Nevada Test Site
Las Vegas, Nevada
Principal contact: Hank Kerr (Sandia)
Phone: 311-6-0420

Facilities:

DoD Domes

- At least two available
- Presently not in use
- Need cleaning
- Building details
  Reinforced concrete
  Diameter: 50 ft
  Shell thickness: 24 in.
  Max rise 9 ft 8 in.

CETG Domes

- Similar to DoD domes except for 6-in. shell thickness
- Unknown number available

B2.6 White Sands Missile Range

White Sands Missile Range
White Sands, NM
Principal contact: John McDougall
Phone 1-678-2443

Facilities:

Climatic conditioning igloos (3)

- Not presently in use
- Reinforced concrete
- Covered by 2-ft earth (not underground)
- Inside height (max) 12 ft
- Aluminum liner

Smaller igloos (3)

- Similar to above but smaller
- Specifications not known

Underground bunkers

- Reinforced concrete
- Presently not in use
- Rectangular shape

B2.7 Decommissioned Facilities

Decommissioned Facilities:

Name: Saxton
NSSS Type: PWR
Location: Saxton, PA

Name: Carolinas Virginia Tube Reactor
NSSS Type: PWR
Location: Parr, SC

APPENDIX C

Estimates of Model Fabrication Costs

Estimates were made of the fabrication costs associated with various sized models of the containment structures. These estimates were made for an urban construction site, such as Albuquerque, and must be increased for a remote construction site. A summary of these costs are given in Table C-1. Three containment types were considered: (1) a free-standing steel containment, (2) a reinforced concrete containment, and (3) a prestressed concrete containment. In order to estimate costs, specific containment designs were chosen as typical. Watts Bar Units 1 and 2 were chosen to represent the free-standing steel type; Salem Units 1 and 2, the reinforced concrete containment type; and South Texas Units 1 and 2, the prestressed concrete type. These designs were chosen because of readily available design information; they do not necessarily represent the best choices for the program. However, they appear to be typical of their respective classes of designs and are appropriate for deriving preliminary cost estimates. Five scale sizes were considered in this early study, including 1/4, 1/8, 1/10, 1/20, and 1/50. Interpolation provided estimates for 1/16 and 1/32 scales.

The following paragraphs describe the limitations assumed for estimating the costs of the various sized models in the preliminary study described above.

1/4-Scale Replica Model

It was assumed that very close replicas of the prototype containments will be fabricated. Concrete, liner plate, reinforcement, and weld lines will be duplicated. Welds themselves will not be replicated. Sizes are sufficiently large to allow the use of standard rebar and steel sheet of the same type and grade. Concrete can be fabricated having similar strength and aggregate properties. All major penetrations will be included.

1/8-Scale Replica Model

All concrete, rebar, plate reinforcement, and weld lines will be duplicated. Commercially available standard rebar will be used. In some cases, special steel plate may be substituted for the designated ASTM steels if thin plate (80 to 170 mils) is unavailable in the same type and grade as used in the prototype. Concrete will be fabricated with scaled strength and aggregate size although aggregate strength may not be replicated exactly. All major penetrations are included.

1/10-Scale Replica Model

Same details as 1/8 model.

1/20-Scale Replica Model (also applies to 1/16 scale as well)

All concrete, rebar, and plate reinforcement will be duplicated. Because of the smaller scale, fewer plates will be used in fabricating the free-standing steel containment liners of the concrete containments and the free-standing steel containment with the consequent reduction in welds. Concrete strength will be replicated, but aggregate properties will not be. Only the major penetrations will be included.

1/20-Scale Nonreplica Model

Because of the high cost of deplicating rebar placement, another model was costed in which steel mesh would replace the rebar. The reinforced concrete would be fabricated to retain the correct cross-sectional strength and stiffness. All other details would be the same as the 1/20-scale replica model.

1/50-Scale Nonreplica Model (also applies partially to 1/32 scale)

Because of the size, rebar placement and plate reinforcement will not be replicated, but the effective strengths and stiffnesses will be. The steel wall of the free-standing steel containment and the cylinder wall liners in the concrete containment will be fabricated from a single sheet. The steel dome will be spin formed in a single piece. Concrete strength will be replicated but aggregate properties will not be.

DISTRIBUTION:

US NRC Distribution Contractors (CDSI) (435)
7300 Pearl St
Bethesda, MD 20014
410 copies to R1 and RD
25 copies to NTIS

US Nuclear Regulatory Commission (25)
Mechanical/Structural Engineering Branch
5650 Nicholson Lane
Rockville, MD 20852
Attn: J. F. Costello

Iowa State University,
420 Town Engineering Bldg
Ames, IA 50011
Attn: L. Greimann

TVA
400 Commerce Ave
Knoxville, TN 37902
Attn: D. Denton, W9A18

Los Alamos National Laboratories
PO Box 1663
Mail Stop J 576
Los Alamos, NM 87545
Attn: C. Anderson

Nuclear Design Group
800 Jorie Boulevard
Oak Brook, IL 60521
Attn: T. J. Ahl

Southwest Research Institute
Department of Ballistics & Explosives Sciences
6220 Culebra Rd
PO Drawer 28510
San Antonio, TX 78284
Attn: W. E. Baker

Bechtel Power Corporation
12400 East Imperial Highway
Horwalk, CA 90650
Attn: A. H. Hadjian

ANCO Engineers, Inc.
1701 Colorado Ave
Santa Monica, CA 90404
Attn: G. E. Howard

University of Illinois
Department of Civil Engineering
Urbana, IL 61801
Attn: M. A. Sozen

Stevenson & Associates
3645 Warrensville Center Rd
Cleveland, OH 44122
Attn: J. D. Stevenson

United Engineers & Constructors, Inc.
Structural Analysis Group
30 South 17th St
Philadelphia, PA 19101
Attn: J. J. Ucciferro

School of Civil and Environmental Engineering
Hollister Hall
Cornell University
Ithaca, NY 14853
Attn: R. N. White

Electric Power Research Institute (3)
3412 Hillview Ave
PO Box 10412
Palo Alto, CA 94304
Attn: I. Wall
H. T. Tang
G. Sliter

Battelle Columbus Laboratories
505 King Ave
Columbus, OH 43201
Attn: R. Denning

University of Alberta
Department of Civil Engineering
Edmonton, Alberta, CANADA T6G 2G7
Attn: D. W. Murray

Technology for Energy Corporation
PO Box 22996
10770 Dutchtown Rd
Knoxville, TN 37922
Attn: E. P. Stroupe

EG&G Idaho
Willow Creek Bldg, W-3
PO Box 1625
Idaho Falls, ID 83415
Attn: S. Sadik

DISTRIBUTION (cont):

Fauske & Associates, Inc.
16W070 West 83rd St
Burr Ridge, IL 60521
Attn: H. K. Fauske

Sargent & Lundy Engineers
55 East Monroe St
Chicago, IL 60603
Attn: A. Walser

Offshore Power Systems, Inc.
PO Box 8000
Jacksonville, FL 32225
Attn: J. Tsai

General Electric Company (2)
175 Curtner Ave
San Jose, CA 95112
Attn: R. F. Gou
J. Love

R. F. Reedy, Inc.
236 N. Santa Cruz Ave
Los Gatos, CA 95030

US Department of Energy
Office of Nuclear Energy
Mail Stop B-107
NE-540
Washington, DC 20545
Attn: A. Millunzi

1521 W. W. Shurtleff
4400 A. W. Snyder
4410 D. J. McCloskey
4412 J. W. Hickman
4412 R. G. Spulak
4415 D. C. Aldrich
4415 J. L. Sprung
4440 D. A. Dahlgren
4441 M. Berman
4441 M. P. Sherman
4442 W. A. Von Riesemann (20)
4442 T. E. Blejwas (20)
4442 A. W. Dennis (3)
4442 D. L. Goodwin
4442 D. S. Horschel
4442 J. Jung
4442 R. D. Meyer
4442 W. A. Sebrell (10)
4442 R. P. Toth
4442 R. L. Woodfin
4443 D. A. Dahlgren (actg)
4443 J. A. Letz
4444 S. L. Thompson
4444 W. H. Schmidt
4445 L. O. Cropp
5520 T. B. Lane
5521 C. M. Stone
5521 D. V. Swenson
5522 R. Russo
8214 M. A. Pound
3141 L. J. Erickson (5)
3151 W. L. Garner (3)

Org.	Bldg.	Name	Rec'd by	Org.	Bldg.	Name	Rec'd by

120555078877 1 ANRORI
US NRC
ADM DIV OF TIDC
POLICY & PUBLICATIONS MGT BR
PDR NUREG COPY
LA 212
WASHINGTON DC 20555

Sandia National Laboratories