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USE OF SMALL MODELS IN DESIGN AND
ANALYSIS OF PRESTRESSED-CONCRETE
REACTOR VESSELS

J. M. Corum
J. E. Smith

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Reactor Division

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J. M. Corum J. E. Smith

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ABSTRACT

A series of small prestressed-concrete reactor vessel (PCRv) models was designed, analyzed, constructed, and tested to investigate the suitability of small-scale ($\leq 1/20$) mortar models and epoxy models for determining elastic stress distributions and cracking and failure modes of concrete vessels, and to demonstrate the adequacy and relative accuracy of two- and three-dimensional elastic finite-element structural analyses when applied to relatively complex vessel shapes. Five models were studied. The first was a small concrete vessel that served as the prototype for the other four. There were two 1/2.75-scale mortar models of the prototype; a 1/5-scale prestressed epoxy model; and a 1/5-scale, idealized, unstressed, axisymmetric epoxy model. Each model was extensively instrumented, and the test results were compared with finite-element predictions. The prototype and the two mortar models were tested to failure with internal pressure. The prototype and one mortar model were pressurized hydraulically but, to examine possible differences in structural behavior, the other mortar model was tested pneumatically. The epoxy models were tested in the elastic range only.

Structural behavior of the small-scale mortar models closely duplicated that of the concrete prototype, and the behavior characteristics of the two mortar models indicated that failure, whether caused by pneumatic pressure or hydraulic pressure, was gradual and not catastrophic. The elastic stress distributions, the load-deformation responses at higher pressures, the modes of cracking and the pressure levels at which they occurred, and even the number and location of the major cracks were essentially the same in prototype and models. The models were analyzed elastically by using both two-dimensional (axisymmetric and plane) and three-dimensional finite-element stress analysis computer programs developed for PCRv's. Comparison of the predicted behavior with test results showed reasonably good agreement for most areas of the vessels, even with the two-dimensional analyses. The more involved three-dimensional analyses predicted the elastic stresses and deflections in the heterogeneous vessels quite well. The close agreement between model and prototype

behavior points up the potential of small-scale models for studying PCRV's. Further, the good agreement between analytical predictions of the elastic behavior of the models and the test results demonstrated the effectiveness and reliability of the finite-element computer analyses currently being used for evaluating PCRV's.

Keywords: pressure vessels, concrete (prestressed), nuclear reactors, models, testing, behavior, failure, photoelasticity, finite-element analysis, reactor safety, nuclear power.

1. INTRODUCTION

1.1 Background

Large-model testing and finite-element structural analyses have been important aids in the design and development of prestressed-concrete vessels for nuclear reactors (PCRV). However, since the construction of large models is costly and time-consuming, such models are of somewhat limited use in research and development and in the early stages of design. As a result, increased attention is being given to small-scale models and, in addition, increased reliance is being placed on more detailed finite-element analyses.

Historically, structural model testing has played a prominent role in the development of much of the general technology of reinforced and prestressed concrete. Although relatively large-scale structural models, often approaching the size of the prototype structure, were utilized in much of this work, small-scale laboratory models were also used and received widespread acceptance.^{1,2} As the interest in small-scale models increased, the theory of models received added attention, and improved techniques for designing, building, and testing small models of reinforced- and prestressed-concrete structures were developed.³ These developments gave further impetus to the use of small-scale models.

In contrast to the models used in the development of the general technology, the models used in the relatively recent development of prestressed-concrete pressure vessels for nuclear reactors⁴ have been predominantly very large scale, with little advantage being taken of the apparent potential of small-scale models. The primary reason for this is that a PCRV is a much more complex structure than those commonly dealt with, and thus there is a marked lack of confidence in the results of small-scale model tests.

The concept of using prestressed-concrete vessels was first applied, in the late 1950's, in the design of the G2 and G3 reactors at Marcoule in France. Since then several PCRV's have been built, or are under construction, for gas-cooled reactors in the United Kingdom and France, and one is under construction in Spain. The first PCRV in the United States

is under construction at the Fort St. Vrain Nuclear Power Generating Station, which is being built for the Public Service Company of Colorado by Gulf General Atomic, Inc., and is to have a capacity of 330 Mw(e).

In the case of each of these vessels, either completed or under construction, extensive model testing was used to demonstrate the adequacy of the design methods and/or the design itself.⁵⁻¹³ Although several very small-scale laboratory models of various types have been used to demonstrate the adequacy of various analytical approaches, to develop vessel configurations, and to qualitatively demonstrate the structural behavior of a particular design concept, they have generally been fairly simple. Consequently only secondary importance has been attached to them, and in all cases where detailed "proving" models have been required to quantitatively demonstrate the predictability and adequacy of a final design, large-scale ($\geq 1/10$) models have been used.

1.2 Purpose and Scope of PCRV Model Study

Costs of several hundred thousand dollars per model have been quoted for some of the large PCRV models that have been built and tested. Such costs, coupled with the time and effort required for construction, generally limit the use of large models to proving designs. Small-scale models that can be used freely in research and development and in the early stages of design are therefore of interest.¹⁴ Slightly simplified microconcrete PCRV models are being given increased consideration, and they show considerable potential. What is lacking, however, as previously stated, is adequate confidence in the information obtained with these small-scale models. If the validity of the results can be adequately demonstrated and if confidence in the use of small models can be established, it may well become possible to abandon the present scale limitations on proving models.¹⁴

The principal purposes of the model study program described in this report were (1) to investigate the suitability of small-scale microconcrete, or mortar, models and epoxy elastic models for determining elastic stress distributions and cracking and failure modes of concrete vessels, and (2) to demonstrate the adequacy and relative accuracy of two- and

three-dimensional elastic finite-element structural analyses when applied to relatively complex vessel shapes requiring simplifications in arriving at the idealized structure actually analyzed. Microconcrete, or mortar, models and epoxy models are typical of the two classes of models that are of interest in prestressed-concrete reactor vessel work and are commonly used in reinforced- and prestressed-concrete investigations. Epoxy models are direct elastic models capable of furnishing the elastic stress and strain distributions within a PCRV structure or component. Generally an elastic PCRV model would be very small scale (on the order of 1/50 to 1/100). In a direct elastic model study the model material is usually chosen on the basis of ease of fabrication for the particular structural configuration under study. For this reason, and also because of the desirability of operating at low applied loads, a plastic is frequently chosen.^{3,15,16}

Studies with direct elastic models are usually limited to the elastic domain. Such models cannot be used to predict or study the ultimate strength characteristics of reinforced- and prestressed-concrete structures in which failure involves cracking and various inelastic actions. The direct inelastic model, on the other hand, is used in an attempt to duplicate the behavior of prototype structures through all stages of loading up to and including failure. In terms of concrete structures, failure may mean excessive cracking or spalling, yielding or separation of the reinforcing steel, failure of the prestressing steel, crushing of concrete, excessive deflections, or leakage.

The large concrete PCRV models that have been tested were direct inelastic models, and small-scale microconcrete models are also direct inelastic models. For a PCRV microconcrete model, a scale of approximately one-twentieth appears to be near optimum. Larger models would approach the size and cost of the large-scale concrete models, while smaller models would be more costly because of the increased difficulties involved in including detailed features to an unnecessarily small scale. In any case, a microconcrete model would be a slightly simplified version of the prototype in the sense that details that would not affect the structural behavior of the prototype would be omitted or represented in a simplified manner in the model to facilitate its construction and hence reduce costs.

Five models were designed, analyzed, built, and tested to accomplish the goals of the model study program. These five models are depicted in Fig. 1. The first was a small concrete vessel that served as the prototype for the other four. The prototype represented the top half of a relatively simple cylindrical vessel with flat heads; its dimensions were on the order of one-tenth those of a full-size PCRV. It was designed for an internal pressure of 500 psi and was prestressed, or more correctly, posttensioned, axially, circumferentially, and through the head with Stressteel bars.

The next two models depicted in Fig. 1 were identical 1/2.75-scale microconcrete, or mortar,* models of the concrete prototype. They were built at Cornell University from design drawings of the concrete prototype that were furnished by Oak Ridge National Laboratory. The materials for the mortar models were chosen to duplicate the material properties of the prototype as closely as possible, and all but a few minor details of the prototype were reproduced to scale in the models. Consequently they were nearly true-to-scale direct inelastic models of the prototype, and although their overall dimensions were smaller than those of a 1/20-scale microconcrete model of an actual PCRV, the sizes of the detailed features -- prestressing elements, reinforcing elements, anchorages, penetrations, etc. -- were of the same order as those envisioned for a 1/20-scale model.

The fourth model was a 1/5-scale prestressed epoxy model of the concrete prototype and can be classified as a slightly simplified direct elastic model. Finally, the fifth model depicted in Fig. 1 was an unstressed, axisymmetric epoxy model having the same principal dimensions (i.e., inside diameter, head thickness, and minimum wall thickness) as the prestressed epoxy model.

These five models were analyzed by using both two- and three-dimensional finite-element computer programs and then tested. The test results were compared with the prototype test results and with the analytical

*Although the 1/2.75-scale models were actually made of a microconcrete, this type of model is commonly referred to as a mortar model, and consequently throughout this report the 1/2.75-scale models are referred to as mortar models.

Fig. 1. Models Used in PCRV Model Study.

predictions. In this manner the validity of the test results and of the analyses could be evaluated.

The concrete prototype vessel and the second mortar model were each tested hydraulically to failure. They were nearly identically instrumented with approximately 375 strain gages, of various types, and load transducers on the prestressing tendons. The first mortar model, on the other hand, was less extensively instrumented and was tested pneumatically to failure with nitrogen gas. The purpose of two mortar models rather than one was to determine the differences, if any, between failures produced hydraulically and pneumatically in these particular structures.

The concrete prototype and the two mortar models were subjected to almost identical tests. They were prestressed in the same sequence of steps, and then each underwent a time-dependent test for a period of approximately one month before the pressure testing began. Experimental data were recorded throughout the prestressing period, the time-dependent test period, and the period of pressure testing.

The 1/5-scale prestressed epoxy model, being an elastic model, was tested only in the elastic range. Two separate loadings consisting of prestressing only and then of prestressing plus internal pressure were examined. The model was instrumented with 194 strain gages located at essentially the same points as the surface strain gages on the prototype vessel.

Finally the axisymmetric epoxy model was loaded with internal pressure only and examined both photoelastically by using the scattered-light technique and with strain gages. It was used first to evaluate the suitability and accuracy of the scattered-light photoelasticity technique for determining the elastic stresses in small epoxy models of concrete vessels or of regions of such vessels and then to demonstrate the reliability and accuracy of finite-element techniques when applied to analyses of an idealized model of a thick-walled concrete vessel. Being perfectly axisymmetric the model was essentially identical to the idealized model treated by the axisymmetric finite-element program that was used to analyze the more complicated prestressed models in the model study program. The details of the photoelastic investigation are the subject of a separate report.¹⁷

1.3 Report Arrangement

Large amounts of information and data were generated in the course of analyzing, building, and testing the five models and comparing the test results with analytical predictions. In this report, an attempt has been made to describe in adequate detail what was done and to present significant and representative results, while at the same time maintaining a thread of continuity and purpose.

Chapter 2 describes the design philosophy and design details of the concrete prototype vessel, as well as its fabrication, which was carried out at a commercial plant. In Chapter 3 the design and fabrication of the mortar models, the prestressed epoxy model, and the axisymmetric epoxy model are described. Particular attention is given to the techniques used in fabricating the mortar models and to the planning and procedures required to successfully construct such small-scale models.

A materials testing program was carried out to obtain data that were needed for correctly modeling the materials used in the prototype and for design, analysis, and experimental data reduction. The materials used in the concrete prototype, the mortar models, and the epoxy models are described in Chapter 4, as are the materials tests that were performed and the results that were obtained.

The finite-element analyses of the models are described in Chapter 5, and plots of the predicted stress levels are presented for the design loadings. An elastic two-dimensional axisymmetric analysis was performed for each model and, in addition, various nonaxisymmetric features of the prestressed models were approximately examined by using plane elastic analyses. The prototype vessel was also examined by using a three-dimensional elastic analysis. Finally an approximate two-dimensional cracking analysis was performed for the prototype. In all cases finite-element computer programs developed for the U.S. Atomic Energy Commission by Gulf General Atomic, Inc., were utilized.

Chapter 6 describes the instrumentation and data-acquisition methods employed. Surface strain gages, embedded strain gages, crack-detection gages, load transducers, deflection gages, and a photoelastic coating were all used on the concrete prototype vessel; each type is described and

locations are given. Instrumentation for the other prestressed models is described relative to that for the prototype.

The actual model tests and the test results and comparisons with analytical predictions are discussed in Chapters 7 and 8. The results and comparisons were selected to give representative and realistic pictures of the behavior of the models and of the accuracy and adequacy of the analytical predictions.

Chapter 9, the final chapter, presents a brief summary of the principal observations and findings of the model study program and a concise summary of the conclusions drawn.

2. CONCRETE PROTOTYPE DESIGN AND FABRICATION

2.1 Requirements and Design Philosophy

The basic requirement for the concrete prototype vessel design was that it be representative of the principal features of existing prestressed concrete reactor vessel designs and yet be as simple as possible. For the purpose of studying the use of small models and the applicability of analytical methods, it was not felt necessary to have a completely realistic vessel. However, the design chosen was similar in many respects to PCRV's being built at that time.

Recent trends in PCRV design have been toward idealized geometries that are more amenable to analysis and prediction. In contrast to the earlier designs after which the concrete prototype vessel was patterned and which tended to be approximately hexagonal in outer shape, with the circumferential prestressing tendons anchored at the vertices, more recent designs have approached a circular cylindrical outer shape, with the circumferential tendons anchored at three or four small vertical pilasters on the outer surface. The earlier designs are typified by the French St. Laurent I (EDF-4) and II vessels,¹⁸ while the more recent designs are typified by the French Bugey I vessel.^{19,20} Presently vessel designers are tending toward a perfectly circular vessel exterior with wire-winding used to apply the circumferential prestressing. Also, the present trend is to not use prestressing tendons running through the heads but, rather, to apply an effective prestress in the heads by additional circumferential prestressing in the head regions. Thus the trend in present designs is toward very simple, ideal axisymmetric configurations.

Because the concrete prototype vessel used in the model study program was patterned after earlier PCR designs, it deviated significantly from a perfectly axisymmetric configuration and was consequently less amenable to prediction and analysis than are more recent PCRV designs. Also, because the prestressing forces were applied by a relatively small number of tendons through relatively large massive anchorage plates and blocks, the prototype vessel was more heterogeneous than an actual vessel in which hundreds of prestressing tendons create, in a gross sense, a more

homogeneous or uniform structural system. Because of its relative heterogeneity the prototype vessel was again less amenable to prediction and analysis than are actual PCRV's.

The concrete prototype vessel design was thus simple in one sense and complex in another. In terms of studying the applicability of models and analytical methods, the gross complexity of the prototype was considered to be advantageous, since in some respects the requirements on the models and analyses were more demanding than in the case of many actual vessel designs.

2.2 Vessel Details

The principal features of the concrete prototype vessel are shown in Fig. 2. The internal cavity of the prototype was a circular cylinder with flat ends, while the outer shape of the vessel was basically a hexagonal cylinder. The internal diameter was $32 \frac{3}{4}$ in., the head thickness was $11 \frac{1}{16}$ in., and the minimum wall thickness was $7 \frac{5}{8}$ in. The vessel represented only the top half of a PCRV and was closed off by a $4 \frac{1}{2}$ -in.-thick steel base plate. This removable plate provided for easy access into the vessel for strain gaging and liner application.

The vessel was prestressed with regular-grade Stressteel bars. There were twelve $1 \frac{1}{8}$ -in.-diam bars axially through the cylinder, six $1 \frac{3}{8}$ -in.-diam bars through the head, and four bands of $1 \frac{3}{8}$ -in.-diam bars circumferentially around the cylinder. There were six bars in each circumferential band, and these were anchored at heavy steel anchor blocks embedded in the vertices of the outside hexagonal surface. The inner surfaces of these anchor blocks were cut on a 48-in.-diam circle, which was equivalent to the across-the-flats dimension of the outer hexagonal surface. The anchorage plates for the axial tendons and the head tendons were standard Stressteel items. The bars were unbonded, since they were sheathed in thin-walled electrical metallic tubing that was nominally $1/8$ in. larger in inside diameter than the bar diameter.

Standard Stressteel anchorages were used. One end of each bar had a nut and thread anchorage, while the other end had a Howlett Grip Nut anchorage. The Howlett Grip Nut consists of an outer nut threaded onto a

Fig. 2. Design Details of Concrete Prototype Vessel.

slotted inner sleeve that fits around the bar. As the anchorage picks up load the inner surface of the sleeve grips the bar with buttress-like teeth and is progressively compressed and embedded into the bar by the action of inclined cam-like threads on its outer surface that move against matching threads on the inner surface of the nut body.²¹

The guaranteed minimum ultimate strength of regular-grade Stressteel bars is 145,000 psi, and the resulting design load levels were those listed in Table 1. Actual stress-strain curves for the bars used are included in Chapter 4.

Table 1. Design Load Levels for Stressteel Bars Used in Prototype

Bar Diameter (in.)	Loads (kips)		
	Temporary Jacking, ^a $0.8f'_s$	Initial Tensioning, $0.7f'_s$	Final Design, $0.6f'_s$
1 1/8	115	101	87
1 3/8	172	151	129

^aThe guaranteed minimum ultimate strength of the tendons, in terms of load, is denoted by f'_s .

Very little conventional reinforcing steel was used in the prototype vessel. An exception was the base region, which was heavily reinforced to guard against premature failure in this region, since it was of no real interest in the investigation. Inner and outer cages of reinforcing bars were used. These cages consisted of number 4 (1/2-in.-diam) bars meridionally and circumferentially on 2-in. centers. Structural-grade deformed bar conforming to ASTM Specification A15-58T (Bars, Billet-Steel, for Concrete Reinforcement) was used.

Reinforcing-bar mesh weldments were also used under each anchor plate to limit possible anchor-zone cracking. Weldments made up of number 2 (1/4-in.-diam) bars, which are not deformed, were used on 2-in. centers.

The concrete for the prototype was specified to have a 28-day compressive strength, f'_c , of 6000 psi, and the maximum aggregate size was specified to be 1/2 in. Other than these two requirements, the constituents and properties of the concrete were left to the discretion of the commercial fabricator who built the vessel.

To study the effects of head penetrations, a simple seven-hole array of penetrations was included in the top head of the prototype. Each penetration was lined with a 1/8-in.-thick steel cylinder having an inside diameter of 3 in. The liner thickness was chosen to compensate, approximately for the stiffness of the concrete removed. The outer surfaces of the penetration liners were provided with raised shear rings on the compression side of the vessel head. The penetrations were closed by steel plugs with O-ring seals. Tie-rods anchored to the bottom steel plate held the penetration plugs in place.

No attempt was made to use a steel liner for the internal cavity of the vessel. Rather, a Fiberglass-reinforced epoxy liner was used as the gastight membrane. An advantage of this liner for model tests was that it could be applied over strain gages on the concrete surface.

2.3 Design Considerations

The head- and wall-thickness proportions of the prototype vessel were picked, a priori, as representative of existing vessels, particularly EDF-3 and EDF-4 and the British Oldbury vessels. Also the design pressure of 500 psi was picked as fairly typical of that of existing vessels. Although terminology and usage vary in the United States and Europe, the design pressure is usually chosen in the range from 1.1 to 1.2 times the normal working pressure of the PCRV. Thus the normal working pressure of the hypothetical vessel represented by the prototype might lie somewhere between 417 and 455 psi.

With these preestablished values, the prestressing system for the prototype was designed to meet the following simple criteria:

1. For internal pressures up to the design level, the compressive prestress should be sufficiently high for the average stress on any vessel cross section to remain compressive.

2. The distance between tendon groups, or in this case between tendon anchor blocks and plates, should not exceed the thickness of the section they act on, and over the distance between tendons there should be at least static equilibrium between the prestressing forces and the internal design pressure.

3. For internal pressures up to the design level, the principal compressive stresses at any point in the vessel, other than at points of obvious stress concentration, should remain less than $0.45f'_c$. For the 6000-psi concrete used in the prototype, this compressive limit is 2700 psi.

4. Principal tensile stresses at any point should remain less than $3\sqrt{f'_c}$, which for 6000-psi concrete is 232 psi.

5. Although the meaning and significance of an ultimate load factor is controversial, values of two to three times the design pressure have been required. Since the ultimate load capabilities of a vessel are, at the least, a measure of its reserve strength, it was intended that the ultimate strength of the prototype vessel be at least 2.5 times the design pressure. This value is 1250 psi. For purposes of defining an ultimate load factor for a PCRV, the usual definition of ultimate load is that pressure which causes structural failure by crushing of the concrete or rupture of the prestressing tendons or by the intervention of a mechanism that leads to local shear or flexural collapse.²² It is assumed in this definition that the vessel liner continues to retain pressure and transmit this to the vessel until the full ultimate strength is reached.

The number, size, and spacing of the prestressing tendons were tentatively established on the basis of the first two requirements. Simple static relations were used for this purpose. Then, a series of slightly more refined design analyses was carried out using thin-shell theory. The purpose of the relatively simple thin-shell analyses was first to locate the axial tendons and the head tendons with respect to the neutral surfaces of the cylinder and head, respectively, and second to approximately check the stress magnitudes for any gross violation of the third and fourth requirements.

For the thin-shell analyses, axisymmetric conditions were assumed, and a circular cylindrical shell representing the cylindrical walls was connected to flat plates at each end that represented the top concrete head

and the bottom steel base plate. The loadings considered were (1) an internal pressure, (2) an external pressure on the cylinder to represent the circumferential prestressing, (3) an axial force and couple uniformly distributed around both ends of the cylinder to represent the axial prestressing in the cylinder and the couple it applies due to being offset from the neutral surface, and (4) a radial force and couple uniformly distributed around the edge of the top head to represent the head prestressing and the couple it applies due to being offset from the neutral surface.

By doing several analyses for prestressing alone and for prestressing plus the design pressure and using several sets of assumed values for the applied couples, the optimum location of the prestressing with respect to the neutral surfaces was determined. By this procedure the bending stresses estimated by shell theory were minimized at the head-to-cylinder junction. In the head the prestressing was assumed to act at the level of the middle pair of tendons (shown in Fig. 2), while in the cylinder the axial prestressing was assumed to act on a circle with radius equal to the average of the two tendon circle radii shown in Fig. 2.

Although the prototype vessel and PCRV's in general are much too thick for thin-shell theory to be applicable, the shape of the actual stress distributions is usually predicted fairly well by shell theory, and the magnitudes of the predicted stresses are not nearly so much in error as might be expected. This was borne out by early tests at Oak Ridge National Laboratory on epoxy PCRV models and comparisons of the experimental results with thin-shell²³ finite-element analysis predictions.²⁴

One additional simplified design analysis was performed to examine the circumferential bending stresses produced in the cylinder wall by the circumferential prestressing forces being applied at six discrete spots around the vessel circumference. These bending stresses tend to produce a circumferential tensile stress on the inner surface of the vessel under each anchor block. A thin circular ring analysis was used to determine the width of the anchor blocks necessary to limit the bending stresses to acceptable levels. It was assumed that the circular ring was subjected to six equally spaced groups of forces. Each group acting over the distance corresponding to the width of the anchor blocks represented the actual

distributed forces applied by the blocks. Finite-element analyses later verified that the resulting design was satisfactory, although there were circumferential bending moments remaining that produced fairly significant stress variations around the cylinder wall.

After the design details of the prototype were established a series of detailed finite-element stress analyses were performed. These included both approximate two-dimensional analyses and a three-dimensional analysis. The two-dimensional analyses included an axisymmetric analysis of the entire vessel, a plane-stress analysis of the top-head penetration region that gave an estimate of the stress concentrations around the penetrations, and a plane-strain analysis of the cylindrical vessel wall that gave an estimate of the circumferential stress variation around the wall. These two-dimensional analyses, taken together, provided a fairly good approximation to the stress variation throughout the vessel. However, to correctly treat the nonaxisymmetric structure the three-dimensional analysis was later carried out.

The finite-element analysis results verified the adequacy of the design and showed that the stress limits specified by requirements three and four, given previously, were satisfied during each step of the prestressing sequence and for internal pressures up to and including the design pressure of 500 psi. The finite-element analyses and the results are described in detail in Chapter 5.

The final area that was examined had to do with the expected modes of cracking and ultimate failure of the prototype when subjected to an increasing overpressure. Several simple approaches were used to estimate the maximum internal pressure that the prototype could withstand provided the cavity liner did not leak first. The most obvious possible modes of failure were investigated. These included rupture of the axial tendons, rupture of the circumferential tendons, and failure in the vessel head. The initiation and growth of cracking were predicted by using the finite-element approach described in Chapter 5.

The results of the cracking analysis indicated that the first structural cracking would occur at an internal pressure of 600 psi and would be located in the haunch that joins the head to the cylinder. This first crack was a circumferential one running completely around the haunch on the inner

surface of the vessel. As the pressure was increased, the crack grew slowly outward at an angle of about 45° with the horizontal. At a pressure of 940 psi, major vertical cracking was predicted to occur in the cylinder wall and the analysis was terminated. At that pressure the initial haunch cracking had extended about halfway through the vessel wall, and the analysis indicated that the stresses on the outer surface of the top head were near the point of initiating local flexural cracking in that region. Also, based on the results obtained at 940 psi, it was estimated that horizontal cracking would begin on the outer surface of the cylinder, near midheight, at about 1175 psi.

For rupture of the axial tendons it was assumed that the head was completely separated from the cylinder by the haunch crack and that the axial tendons were carrying the entire axial pressure load. On this basis axial tendon failure would occur at about 2250 psi. For failure of the circumferential tendons it was assumed that complete vertical cracking of the cylinder walls had occurred and that the walls were not restrained by the vessel heads. On this basis the circumferential tendons would carry the entire hoop load due to pressure, and failure would occur at 1560 psi. These tendon failure estimates were based on the actual observed ultimate strength of the tendons used. The values were furnished by the manufacturer and are given in Chapter 4.

The prediction of failure in the deep end slabs of cylindrical concrete vessels is difficult, and any calculations that are made contain elements of uncertainty. For this reason the ultimate load factors specified for heads are usually larger than those specified for other areas of a PCRV. Several research projects for investigating the failure behavior of heads have been carried out or are under way.

To estimate the ultimate failure pressure of the deep-slab head of the concrete prototype, published experimental results from a series of head tests by Campbell-Allen and Low²⁵ were utilized. The results show that for radially prestressed deep slabs, initial cracking occurs on the outer surface due to flexure. Primary failure, however, is due to a shearing action along inclined surfaces of revolution. Once this type of failure has been initiated, either a secondary tensile failure or a 'punch-out'

shear failure may occur. This ultimate failure behavior bears no relationship to the initial flexural crack patterns.

The head of the prototype had a thickness-to-inside radius ratio of 0.65 and an average radial prestress of 1000 psi. Curves presented by Campbell-Allen and Low based on their experimental results indicate that the ultimate failure pressure for this head would be approximately 4500 psi. This value can be considered, at best, only as a very rough estimate. It is, however, well above the failure pressures estimated for other assumed mechanisms.

In summary, cracking of the prototype was predicted to occur first in the haunch at about 600-psi internal pressure, which was 1.2 times the design pressure, and major structural cracking in the cylinder walls was predicted to occur at about 940 psi, which was nearly 1.9 times the design pressure. Finally, on the basis of the mechanisms considered and the assumption that the liner would not fail first, ultimate failure of the vessel would be expected to occur by rupture of the circumferential tendons at an internal pressure somewhat greater than 1560 psi. This value was over three times the design pressure.

2.4 Fabrication of Concrete Prototype Vessel

The concrete prototype was built by Concrete Materials of Georgia, Inc. (CMI), which fabricates precast and prestressed concrete structures at its large commercial plant located outside of Atlanta. The vessel was built with the manufacturer's normal forming and casting procedures and standard concrete constituent materials. Only the maximum aggregate size of 1/2 in. and the 28-day compressive strength of 6000 psi were specified.

The finished formwork is shown in Fig. 3 just prior to casting of the vessel. Except for a tubular fiber form used for the inner cylindrical wall the formwork was constructed of wood. The forms in the foreground of Fig. 3 were for standard 6- x 12-in. cylinders and flexural specimens that were cast along with the vessel and later used to characterize the concrete used in the prototype.

Figure 4 is a view looking into the top of the vessel form. The angle-iron structure at the top was simply bracing for the embedded steel parts.

PHOTO 74688

Fig. 3. Formwork for Concrete Prototype Vessel.

Fig. 4. Top View of Formwork Prior to Casting Concrete Prototype Vessel.

Under it, the anchor plates and anchor-zone reinforcing-bar weldments for the axial tendons and the head tendons can be seen, as can the top-head penetration liners and the thin-walled ducts for the tendons.

The actual pouring operation, shown in Fig. 5, followed CMI's normal procedures. The concrete was mixed at CMI's batch plant near the casting site and transported to the site in a ready-mix truck. There the concrete was transferred to a dump bucket. The vessel was poured in four or five main lifts and was vibrated after each lift with immersion-type vibrators. Vibration of the final lift is shown in Fig. 6. The entire casting operation required about 1 hr to complete.

The forms were stripped three days after casting, and at that time the vessel was wrapped tightly in several layers of burlap that were kept wet continually with a soaker hose. At the end of 14 days curing, the vessel was transported to ORNL, with the burlap being kept wet enroute. At ORNL the vessel was placed in a controlled-environment room having a temperature of 70°F, and the burlap was kept wet for an additional 14 days. Thus the vessel was wet cured for a total of 28 days. Throughout the instrumentation and test period the vessel remained in the controlled environment room, where the temperature was maintained at 70°F and the relative humidity at 50%. The finished vessel is shown assembled, instrumented, and ready for testing at ORNL in Fig. 7.

Fig. 5. Pouring Operation for Concrete Prototype Vessel.

PHOTO 74623

Fig. 6. Vibration of Vessel Casting with an Immersion-Type Vibrator.

Fig. 7. Instrumented Concrete Prototype Vessel Ready for Testing.

3. MODEL DESIGN AND FABRICATION

3.1 Mortar Models*

The two identical 1/2.75-scale mortar models of the concrete prototype vessel were designed and built at Cornell University where considerable research has been carried out on the use of mortar and microconcrete for small-scale direct inelastic models of concrete structures.³ Design drawings of the concrete prototype were furnished to Cornell, along with compressive stress-strain curves, splitting tensile strength data, and flexural strength data for the concrete mix that was chosen by CMI for the concrete prototype.

3.1.1 Design of Models

The design of the mortar models proceeded on the basis of a true structural model; that is, an attempt was made to select model materials with the same properties as those of the prototype materials. The only scaling involved was linear geometrical scaling. Mechanically induced stresses and strains throughout the models and the applied pressure loadings were thus intended to be identical for models and prototype, with the exception of deadweight effects, which cannot be easily modeled when the same types of materials are used in model and prototype. Deadweight effects are significant only in certain phases of the construction of PCRV's. Thermal loads were not considered in the model study program.

The geometrical scale factor for the mortar models was chosen as 2.75 (all prototype dimensions to be reduced by this factor). This scaling permitted the use of standard 1/2-in.-diam Stressteel bars to represent the 1 3/8-in.-diam Stressteel prestressing bars in the prototype. It also resulted in a model of nearly minimum workable size in terms of ease of fabrication, placement of model concrete, and application of

*The mortar models were designed and built under the direction of R. N. White, Associate Professor of Structural Engineering at Cornell University. The description of the design and fabrication of the mortar models presented in this chapter was prepared with his assistance, as was the section on mortar model materials presented in Chapter 4.

instrumentation. The 1/2.75-scale models had an internal diameter of 11.91 in., a head thickness of 4.02 in., and a minimum wall thickness of 2.77 in.

As in the prototype, regular-grade Stressteel prestressing bars were used. With the model scale set at 1/2.75 the axial tendons were required to be 0.41 in. in diameter. Since Stressteel bars smaller than 1/2 in. in diameter were not available, 1/2 in. bars were ground down to 0.41 in. in diameter to model the axial tendons. To keep seating losses to a minimum during the prestressing operation, standard Stressteel nut and thread anchorages were used at both ends of the model prestressing bars.

The guaranteed minimum ultimate strength of regular-grade 1/2-in.-diam Stressteel bars is 145,000 psi, the same as for the larger prototype tendons. For the 1/2-in.-diam head and circumferential model tendons, f'_g was thus 28,000 lb, and for the 0.41-in.-diam axial tendons, 19,000 lb. The design prestressing load levels for the mortar models based on these values of f'_g are given in Table 2. These design loads produced the same stresses and strains in the mortar models as did the corresponding design loads in the concrete prototype.

Table 2. Design Load Levels for Stressteel Bars Used in Mortar Models

Bar Diameter (in.)	Loads (kips)		
	Temporary Jacking, $0.8f'_g$	Initial Tensioning, $0.7f'_g$	Final Design, $0.6f'_g$
0.41	15.2	13.5	11.4
0.50	22.4	19.6	16.8

The modeling of reinforcing steel requires special attention, particularly for structures in which conventional reinforcing plays a significant role in the development of cracks. Considerable care is required to obtain

the proper bond characteristics and stress-strain behavior. Techniques for doing this are discussed in Ref. 3.

In the concrete prototype the only conventional reinforcing steel was located in noncritical areas. Furthermore, since relatively tight welded meshes were used, the bond problem was not critical. Consequently the reinforcing-steel modeling problem was simplified.

Prefabricated welded wire fabric was used for the required conventional reinforcing at the base of the models and under the steel anchor plates near the top. The wire fabric was used to eliminate the need for welding assemblies of small wires. The 1/2-in.-diam reinforcing in the base region of the prototype was simulated with 0.192-in.-diam wire fabric having a 0.94-in. center-to-center wire spacing. The lighter 1/4-in.-diam reinforcing under anchor plates of the prototype was simulated with 0.092-in.-diam wire fabric having a 0.75-in. center-to-center wire spacing. Although the wire diameter and center-to-center spacing of the model reinforcing bars were not individually scaled properly, the amount of reinforcing corresponded, on an area basis, fairly closely in model and prototype.

The wire fabric was obtained from the W. S. Tyler Co., Mentor, Ohio. To verify that the yield and ultimate strengths of the wire satisfied the requirements specified for the prototype reinforcing steel (ASTM A15-58T), tensile tests on three samples of the 0.192-in.-diam wire were performed at Cornell University. The average tensile strength was 72,000 psi (55,000 to 75,000 specified by ASTM A15-58T), and the average yield strength was 65,000 psi (33,000 minimum specified by ASTM A15-58T).

All other components in the model structures were essentially linearly scaled versions of the prototype components. This included the steel anchor blocks and plates for the prestressing, the penetration plugs and liners, and the thin and heavy steel plates at the model base. Scaling was distorted for some nonstructural items, such as the O-rings, the tie rods for the penetration plugs, and the access holes for instrumentation and pressurization in the bottom plate and in the top-head penetration plugs.

The model concrete is discussed in detail in Chapter 4. Six different mixes were designed and tested to obtain a suitable concrete with properties satisfactorily matching those of the prototype concrete. Emphasis was placed on matching the modulus of elasticity, the compressive strength and strain at failure, the splitting tensile strength, and the modulus of rupture.

The coarse aggregate was a nongraded, angular, crushed rock that passed a 5/16-in. sieve. The sand and aggregate dimensions were not geometrically scaled from the prototype concrete, since such scaling is not necessary to reproduce concrete properties; the relative quantities of materials are more important than the exact gradation of aggregates. The model concrete problem is further simplified in PCRV models, as compared, for example, with thin-shell models, because the thick walls of PCRV models permit the use of relatively coarse aggregate in the model mortar. Thus the "mortar" becomes a microconcrete.

3.1.2 Fabrication and Assembly of Forms

The complex outer surface of the vessel structure precluded simple construction of the forms. Close tolerances had to be maintained in order to produce high-quality models. Since the forms were to be used for casting two models and no expansion of the form material could be allowed because of possible difficulty in removal of the forms, it was decided that all basic form components would be made of Plexiglas. This material is waterproof, dimensionally stable, easy to machine, easily cleaned, and well suited for multiple casting use.

The exterior shape of the vessel required a transition from a circular cross section at the base to surfaces defined by three different hexagonal cross sections (as shown in Fig. 2) that formed (1) the outer surfaces of the circumferential prestressing anchor blocks, (2) the flat concrete pads between anchor blocks in each circumferential band, and (3) the main hexagonal outer surface of the vessel. The form for the circular base section was machined from a solid block of Plexiglas obtained by cementing together two sheets of Plexiglas with PS-18 cement. After turning on a lathe, the ring-shaped piece was cut into three segments to provide for

easy removal from the mortar models. These segments may be seen in Fig. 8 which shows the mortar model form partially assembled. The vertical joints between segments were sealed with 2-in.-wide aluminized Tuck tape.

Fig. 8. Partially Assembled Form for Mortar Models. Photograph made by Photo Science Service, Cornell University.

The hexagon formed by the outer surfaces of the anchor blocks was chosen as the control surface for form construction. The six outer sides were made of 3/4-in.-thick Plexiglas and were held together with aluminum angles as shown in Fig. 8. The forms for the actual hexagonal outer surfaces were made up of Plexiglas filler blocks and spacer blocks placed between the anchor blocks shown in Fig. 8. The entire form was placed on a 3/4-in.-thick 36- x 36-in. aluminum plate.

The inner circular form had the general shape shown in Fig. 8. Difficulty in obtaining the appropriate-size Plexiglas tubes led to the use of 1/2-in.-thick aluminum tubing as the basic material for this form. The smaller tube, which was for the base region, was fitted into the larger tube, which was for the main cavity wall, and the two were riveted together. After installing two interior rings, which formed the interior bevels, the entire form was cut into four segments by using a 1/8-in.-wide slitting saw. The locations and slopes of cuts were determined on the basis of ease in removal of the inner mold. All cuts were sealed with 2-in.-wide aluminized Tuck tape.

The top of the inner form was made of a Plexiglas disk cut into three pieces, which were turned on a lathe after cutting to assure a tight-fitting circular shape. The disk was held to the form walls by bolting it to the inner top ring of the main inner form.

For the first mortar model, the aluminum inner form was coated with petroleum jelly, which proved inadequate for completely preventing bonding between the concrete and aluminum. The aluminum surface was given four coats of Krylon spray paint and three coats of Butcher's wax before the forms were assembled for the second casting. Aluminum is less satisfactory than Plexiglas as a forming material for mortar or concrete because it requires special surface treatment.

The following sequence was used in the assembly of the complete form:

1. The steel bottom plate, with the base reinforcing cages welded to it, was positioned inside the three-piece plastic base form for the outer surface. The bottom surface of the plate was covered with five coats of clear Krylon plastic to protect it from rust, and the inner surface of the plastic base form was coated with a thin layer of petroleum jelly.

The plastic plates forming the outer walls of the form were positioned on the base with pins and then bolted to it.

2. The six anchor blocks in each circumferential band were arranged in proper position on a table and then joined together with the thin-walled tendon tubes, which were sealed in the holes with GE Clear Seal Translucent Sealer, a flexible silicone rubber compound. The sealant proved to be ideal because it remained elastic after setting and thus permitted minor adjustments in relative block positioning without destroying the required watertight seal.

3. Each layer of steel blocks and six filler blocks was placed, and six spacer blocks were placed between each layer. The steel anchor blocks were each bolted to plastic outer walls of the form. A 1/4-in. bead of ordinary white contractor's caulking compound was placed at all surface interfaces (plastic-to-plastic and plastic-to-steel) when building up the layers of the outside form. This material provided watertight joints, remained soft throughout the casting procedure, and enabled easy form removal. All surfaces that were to be in contact with the concrete were coated with petroleum jelly by brushing on a thin layer.

4. The interior mold was placed in position and clamped to the aluminum base plate. The machined head-penetration inserts were set into the plastic head of the interior mold and held in place by bolting.

5. After placement of the fourth and last layer of steel blocks and the interior mold, the exterior form was completed by placing the six side pieces that formed the outer hexagonal shape of the top head. These pieces also positioned and held the anchor-plate reinforcing-mesh tendon-tube assemblies for the six prestressing bars in the top head.

6. The assembly was completed by placing the 12 anchor-plate reinforcing-mesh tendon-tube assemblies for the axial prestressing bars. Each assembly was held at the top by a positioning jig. The entire form was held down on the aluminum base plate with four 3/8-in. vertical rods running from plates at the top of the form down through the base plate.

The finished complete form is shown in Fig. 9. The assembly was fastened securely with C-clamps to the table of a 750-lb capacity Syntron vibrator, as shown. Figure 10 is a view looking into the top of the form.

Fig. 9. Mortar Model Form in Casting Position on Vibrating Table.
Photograph made by Photo Science Studios, Cornell University.

Fig. 10. Top View of Mortar Model Form Prior to Casting. Photograph made by Photo Science Studios, Cornell University.

The positioning jig for the axial tendon assemblies may be seen, as well as the plates with the hold-down rods that run to the aluminum base plate. Under the positioning jig, the anchor-plate reinforcing-mesh tendon-tube assemblies for the head and axial prestressing may be seen. The penetration liners with their hold-down devices may also be seen.

3.1.3 Model Casting and Form Removal

The microconcrete for each model was mixed in three separate batches of about 1 ft³ each and was placed in the form by hand in layers several inches deep. About 6 in. of mix was needed in the first layer in order to get the mix through the reinforcing mesh at the base.

Each layer was vibrated until satisfactory compaction and elimination of air voids had been achieved; several minutes of vibration were required for each layer. Mechanical vibration was supplemented by rodding the microconcrete with 1/2-in. wooden dowels. The transparency of the base form was a real help in determining when the lower portion of the model mortar was in proper position; without the transparency there would have been no way of assessing the adequacy of placement and vibration. It should also be pointed out that casting of such a stiff mix in the congested model form would have been very difficult without the vibrating table.

The entire casting operation took about 45 min. Speed was essential in order to avoid vibration of partially set material at the bottom of the form as the top was approached. The top surface was finished and trowelled about 1 1/2 hr after casting. The model was then covered with wet burlap and a sheet of plastic film.

Test specimens (cylinders and flexural specimens) were cast along with each model and were subjected to the same curing conditions as the models. These specimens were used to characterize the actual microconcrete used.

The inner rings forming the bevels of the interior form were loosened 4 hr after conclusion of casting to prevent any restraint of possible shrinkage during the first day of curing. The four 1/8-in. cuts in the inner form walls were free to close after loosening of the inner rings.

All forms were removed 22 hr after casting. No difficulty was experienced in removing the walls and the top plate of the inner mold. The exterior form was also removed with no difficulty and with no damage to any of the concrete surfaces or edges. The thin layers of caulking compound on the spacer blocks and filler blocks were adequate in providing sufficient clearance for removal of the blocks from between the fixed-in-place steel anchor blocks.

Immediately after the forms were removed, small air bubble voids at the top surface of the concrete pads between anchor blocks were filled with neat cement paste. These voids were small in all cases and did not represent any defects of structural significance. Some very small voids on the interior surface were also filled.

The holes for the prestressing tendons and the penetrations in the top head were then covered with aluminized tape to prevent rusting in the curing room, and the exposed exterior steel surfaces were painted with hot paraffin. The interior surfaces of the head penetration liners and the exposed surface of the steel base plate were coated with a thick layer of petroleum jelly.

The models were then placed in a curing room under conditions of nearly 100% humidity for a period of 28 days. This corresponded to the 28-day period of wet curing for the concrete prototype vessel.

When the models were removed from the curing room the concrete surfaces were painted with several coats of shellac to limit the rate of moisture loss from the material. Shellac has been found to be a relatively effective moisture sealant for small mortar models and test specimens.³

Immediately following the 28-day cure the models were shipped by air to Oak Ridge, where they were placed in the controlled environment room having a temperature of 70°F and a relative humidity of 50%. Except for the first mortar model, which was removed from the room for the pneumatic pressure test, both models remained in the room throughout the instrumentation and test period.

One of the finished mortar models is shown in Fig. 11. This photograph was made before instrumentation. The same model is shown partially disassembled in Fig. 12. The base is at the right, with the top-head penetration plugs attached by tie rods. An axial tendon, a head tendon, and a circumferential tendon are shown in the foreground. The dark appearance of the model is due to the shellac coatings. The high quality of these small models is readily apparent in Figs. 11 and 12.

The second mortar model, which was fully instrumented to duplicate the prototype instrumentation and was tested hydraulically, is shown in Fig. 13. The model had been instrumented and was ready for testing when this photograph was taken.

3.2 Prestressed Epoxy Model

Design of the prestressed epoxy model of the concrete prototype vessel proceeded on the basis of a slightly simplified direct elastic model.

Fig. 11. Finished Mortar Model.

PHOTO 74618

Fig. 12. Disassembled Mortar Model.

Fig. 13. Instrumented Second Mortar Model Ready for Testing.

The resulting model was thus intended for testing only within the elastic range.

The geometrical scale factor for the prestressed epoxy model was chosen as 5.0 (all prototype dimensions were reduced by this factor). The model thus had an internal diameter of 6.55 in., a head thickness of 2.21 in., and a minimum wall thickness of 1.53 in.

It was decided that the stresses in the epoxy should be 1/10 those in the concrete prototype. This stress-scaling factor was chosen primarily for convenience in comparing results. With this factor, the design pressure for the epoxy model was $1/10 \times 500$, or 50 psi. The prestressing loads, P_m , for the model were related to the prestressing loads, P_p , for the prototype by the following relation:

$$\frac{P_m}{L_m^2} = \frac{1}{10} \frac{P_p}{L_p^2}$$

or

$$P_m = \frac{1}{10} \left(\frac{L_m}{L_p} \right)^2 P_p .$$

Here, the subscripts m and p refer to the model and prototype, respectively, and L_m and L_p refer to some characteristic linear dimension of the model and prototype, respectively. The ratio L_m/L_p is 1/5, the linear scale of the model. Therefore the prestressing forces in the model were related to those in the prototype by

$$P_m = \frac{1}{250} P_p .$$

Two elastic tests were performed on the epoxy model. The first was for the initial prestressing corresponding only to $0.7f'_s$. The second was for the design prestressing corresponding to $0.6f'_s$ plus the design pressure of 50 psi. The initial prestressing loads and the design prestressing loads for the concrete prototype tendons were given in Table 1. With

those values and the above relation, the required prestressing loads for testing the epoxy model were determined and are given in Table 3.

Table 3. Design Load Levels for
Prestressing in Epoxy Model

	Load (lb)	
	Initial Tensioning	Final Design
Axial tendons	404	348
Head and circumfer- ential tendons	604	516

There are two similitude requirements that should be fulfilled by the elastic properties of elastic models. First, Poisson's ratio for model and prototype materials should be equal, particularly for structures in which multiaxial stresses predominate. Epoxy has a significantly different Poisson's ratio from that of concrete. Thus the prestressed epoxy model did not meet this first requirement, and one of the objectives of comparing model and prototype test results was to evaluate the significance of this violation.

The second similitude requirement on elastic properties has to do with the relation between the ratio of elastic moduli of different materials within the model and within the prototype. This requirement can be stated as

$$\frac{(E_1)_m}{(E_2)_m} = \frac{(E_1)_p}{(E_2)_p},$$

where E is the elastic modulus and subscripts 1 and 2 refer to two different materials within the model or the prototype. In the concrete prototype vessel these two materials were steel and concrete. The heavy steel base plate in the prototype was simulated in the epoxy model by magnesium,

which has the same stiffness ratio to epoxy as steel does to concrete and thus satisfies the similitude requirement for elastic modulus ratios.

The 1/5-scale prestressed epoxy model is shown in Fig. 14 without its prestressing system and prior to being instrumented. The model was machined from a solid epoxy casting. It has essentially all the details of the concrete prototype, except for the embedded steel parts, such as the reinforcing in the base and under the anchor plates, the top-head penetration liners, and the anchor plates and blocks. To approximately account

Fig. 14. Prestressed Epoxy Model Before Being Instrumented.

for the latter, the steel anchor plates for the axial tendons and head tendon were simulated by built-up epoxy bosses having the same flexural rigidity, properly scaled, as the steel plates.

The prestressing tendons for the model consisted of 1/4-in.-diam drill rod. Individual small hydraulic rams were used to tension each tendon. These were connected to two common pressure supplies, one for the axial tendons and one for the head and circumferential tendons, so that the model could be loaded and unloaded rapidly in single steps. This feature minimized the time effects of creep and relaxation in the epoxy. One of the tendons, with its hydraulic ram, is shown disassembled in Fig. 15.

The finished model with the prestressing system in place is shown instrumented and ready for testing in Fig. 16. The instrumentation for this model essentially duplicated that on the surface of the concrete prototype vessel. Figure 17 shows the top of the epoxy model and the hydraulic rams

Fig. 15. Prestressing Tendon and Hydraulic Ram for Prestressed Epoxy Model.

PHOTO 75595

Fig. 16. Prestressed Epoxy Model Ready for Testing.

Fig. 17. Top View of Prestressed Epoxy Model.

for the axial prestressing tendons. The penetrations in the head were formed by drilling from both the inner and outer surfaces of the head and leaving a small amount of material where the two holes met. The resulting epoxy penetration plugs were at the same relative level within the head as the steel penetration plugs in the concrete prototype.

3.3 Axisymmetric Epoxy Model

The unprestressed axisymmetric epoxy model was a greatly simplified version of the prestressed epoxy model. It was simply a thick-walled circular cylinder with a thick flat head, as shown in Fig. 18. It had the same main dimensions, in terms of inside diameter, head and wall thicknesses, and haunch size, as the prestressed epoxy model. The internal

Fig. 18. Axisymmetric Epoxy Model.

diameter was 6.55 in., the head thickness was 2.21 in., and the wall thickness was 1.53 in.

Since only the top half of the model was of interest, the base region was simplified, as may be seen in Fig. 18. Also, the height of the model above the base plate was 10 in., in contrast to 12 in. for the prestressed epoxy model. The base plate was made of aluminum and was 1/2 in. thick. The axisymmetric model was machined from a solid casting of the same epoxy as used for the prestressed model.

The axisymmetric model was tested, elastically, with internal pressure only. The design pressure of 50 psi chosen for the prestressed epoxy model was also used for the axisymmetric model.

4. MATERIALS CHARACTERISTICS

The materials used in the concrete prototype, the mortar models, and the epoxy models are described in this chapter. Compressive stress-strain tests, flexural tests, and splitting tensile strength tests were performed on the concrete of the prototype and on the microconcrete of the mortar models. These tests were made at 28 days on wet specimens and at 90 days on both wet and dry specimens, and the results are summarized here. In addition, the stress-strain properties of the actual Stressteel tendons used in the prototype and the mortar models are presented, as are the stress-strain properties of the epoxy used in the epoxy models. Finally, the Fiberglas-reinforced epoxy liner used in the prototype and mortar models is described.

4.1 Concrete Prototype

4.1.1 Concrete

The specifications for the prototype called for a concrete with a 28-day compressive strength of 6000 psi and a maximum aggregate size of 1/2 in. Other than these requirements, the concrete mix to be used was left to the discretion of the vessel fabricator, Concrete Materials of Georgia, Inc.

The mix chosen was made up of the standard constituents used by CMI in their precast concrete components. Based on a one-bag batch, the mix proportions reduced to the following:

Water, gal	5.1
Cement, lb	94
Aggregate, lb	
Fine	160
Coarse	229

The resulting mix had a water-cement ratio of 0.46 and a slump of about 3 in.

Type III (high-early-strength) cement was used. The fine aggregate was Rollo sand obtained from the Atlanta Sand Company, and the coarse aggregate was crushed stone obtained from the Vulcan Materials Company's

quarry at Stockbridge, Georgia. The coarse aggregate was graded to conform to ASTM Specification C33-59 (Aggregates for Concrete) requirements for No. 7 aggregate, which has a nominal size ranging from 1/2 in. down to that retained by a No. 4 sieve.

Aggregate from the Stockbridge quarry has been fairly extensively used in U.S. Corps of Engineers construction projects and has consequently been carefully investigated by the Corps.²⁶ They define the crushed stone as being "about 90% fine to medium grained, relatively unweathered, foliated granite gneiss with hard, clean, angular to subangular rough surfaces, 9% pegmatite, and 1% weathered material." Granite gneiss is a banded metamorphic rock in which bands rich in granular minerals, mostly quartz and feldspars, alternate with bands in which schistose minerals, mostly micas, predominate.

Published tables²⁷ indicate that most gneisses, particularly granite gneisses, have much lower elastic moduli than most igneous rocks, such as granite, and the good limestones commonly used as concrete aggregates. Consequently, concretes made with a granite gneiss aggregate would be expected to also have an unusually low modulus of elasticity, as was true for the prototype vessel concrete.

A total of eighteen 6- x 12-in. cylinders for compressive tests, nine 6- x 12-in. cylinders for splitting tensile strength tests, and nine 6 x 6 x 20-in. beams for flexural tests were cast along with the prototype vessel. The specimens were removed from the molds 18 hr after casting and were placed in a water bath and shipped to Oak Ridge National Laboratory along with the vessel. At the end of 28 days after casting, one-third of the specimens were removed from the water and tested, one-third were removed from the water and stored in the 50% relative humidity, 70°F room, and one-third were left in the water. These latter wet and dry specimens were tested at 90 days.

Thus 28-day data were obtained on wet specimens, and 90-day data were obtained on both wet and dry specimens. The 28-day data were used as the prototype concrete compressive characteristics to be modeled in designing the mortar model microconcrete. The 90-day data were used to characterize the prototype vessel concrete near the time that the vessel was actually tested. The moisture condition of the concrete in the vessel was actually

somewhere between saturated and dried out. The dry specimens tested at 90 days were expected to be most representative of the vessel concrete, since their curing cycle duplicated that used for the vessel. Tests on the prototype vessel began at an age of 193 days. However, the stress-strain characteristics of concrete made with Type III cement would be expected to change very little with time after an age of 90 days, so the 90-day data were assumed to be applicable.

The compressive tests were performed in accordance with ASTM Specification C39-59 (Compressive Strength of Molded Concrete Cylinders). Complete stress versus strain curves to failure were obtained for each cylinder tested. Transverse strain measurements were also made during each test to obtain Poisson's ratio values.

Special strain transducers were developed and used for measuring the axial and transverse strains. These transducers, which are used in pairs, are shown in Fig. 19. One pair is shown attached to a specimen by rubber bands, and, to show the details of the transducer, another pair is shown removed. The basic strain-measuring elements are ordinary electrical-resistance strain gages bonded to thin cantilever beams. When the transducers are clamped lightly against a specimen being strained, the cantilever beams bend and thus produce a strain in the gages. The transducers are wired to read directly the averages of both the axial and the transverse strains on diametrically opposite sides of the specimen. Predetermined calibration factors are used to convert the recorded strains to actual concrete strains.

The axial gage length of the transducers was 2 in., and the transverse gage length was 1 in. It was felt that these small gage lengths, in contrast to the 10-in. gage length of a standard compressometer, were necessary to obtain representative strain values in the relatively small region of a specimen where the stress field is uniaxial.

Six specimens were tested for each of the three age and moisture conditions, and the results are summarized in Fig. 20. The curves shown for each condition are the average curves determined from six specimens. The results for each condition were consistent from specimen to specimen. For

Fig. 19. Strain Transducers Developed for Obtaining Compressive Stress-Strain Curves.

Fig. 20. Compressive Stress-Strain Curves for Concrete.

example, the range of compressive strengths was less than 10% of the mean in each case.

The 28-day compressive strength was 6810 psi, the modulus of elasticity was 3.2×10^6 psi, and Poisson's ratio was 0.22. As mentioned above, the modulus was quite low for the relatively high compressive strength. For a limestone aggregate concrete of this strength, a modulus of about 5×10^6 psi would be expected.

The splitting tensile strength tests were performed in accordance with ASTM Specification C496-66 (Splitting Tensile Strength of Molded Concrete Cylinders). Three specimens were tested for each of the three age and moisture conditions.

The flexural tests for determining the modulus of rupture were performed in accordance with ASTM Specification C78-59 (Flexural Strength of Concrete Using Simple Beam with Third-Point Loading). Three specimens were tested for each of the three age and moisture conditions. The span length used for the 6 x 6 x 20-in. beams was 18 in., and the two loads were applied at the third points. In all cases, fracture occurred in the center 6-in. section of the specimen.

The significant properties of the prototype vessel concrete are summarized in Table 4. The values given are the averages for the specimens tested at that condition. The ranges of the values are also indicated.

4.1.2 Stressteel Bars

Certified load-strain curves for each heat of Stressteel bars are prepared by an independent testing laboratory, and copies of these curves are furnished by the Stressteel Corporation to the purchaser for each heat shipped. The curves provided for the 1 3/8-in.-diam head and circumferential prototype tendons and the 1 1/8-in.-diam axial tendons are shown in Fig. 21. The temporary jacking load ($0.8f'_s$), the initial tensioning load ($0.7f'_s$), and the final design load ($0.6f'_s$) are marked on the curve for each size bar. The actual ultimate strength of the 1 3/8-in. bars was 22% higher than the guaranteed minimum of 215,000 lb, and for the 1 1/8-in. bars the actual ultimate was 9.4% higher than the guaranteed minimum of 144,000 lb.

4.1.3 Fiberglass-Reinforced Epoxy Liner

The Fiberglass-reinforced epoxy liner described here was used for the concrete prototype and for both mortar models. The characteristics desired for the liners were that they be able to span fairly sizable cracks while being simultaneously subjected to relatively high overpressures and that they then fail in a sudden, brittle manner to allow the pressurizing fluid to escape suddenly into the structural cracks. It was felt that if there were an effect of the pressurizing fluid acting in the structural cracks, these characteristics of the liner would impose the most stringent modeling conditions.

Table 4. Summary of Properties of Prototype Vessel Concrete

	28-Day-Cured Wet Concrete		90-Day-Cured Wet Concrete ^a		90-Day-Cured Dry Concrete	
	Average	Range (% of mean)	Average	Range (% of mean)	Average	Range (% of mean)
Modulus of elasticity, psi	3.2×10^6	25.0	3.4×10^6	8.8	3.5×10^6	8.6
Poisson's ratio	0.22	36.1	0.18	66.7	0.18	55.6
Compressive strength, psi	6810	8.3	7540	4.7	8060	7.7
Splitting tensile strength, psi	461	9.5	575	7.0	635	6.9
Modulus of rupture, psi	750	2.1	818	2.1	747	6.8
Unit weight, ^a lb/ft ³	147.3	1.0	147.8	0.7	145.3	1.2

^aUnit weights were based on 6- × 12-in. cylinders only.

Fig. 21. Tensile Stress-Strain Curves for Stressteel Bars Used in Concrete Prototype.

Preliminary tests were performed on several prospective liner formulations. In the tests, concrete disks with liners applied to one side were subjected to bending strains that caused visible cracking under the liners while the liners were simultaneously subjected to oil pressures up to 800 psi. In these preliminary tests the liner chosen spanned sizable concrete cracks before failure.

The liner and the steps used in applying it are described below.

1. The concrete surface flaws and voids were filled with an epoxy-sand mixture.

2. The first coating of epoxy was applied. The epoxy consisted of

100 parts of EPON 815 epoxy (Shell Chemical Corp.)
15 parts of polysulfide LP3 (Thiokol Chemical Corp.)
50 parts of Lancast A (distributed by CIBA Products Corp.)
3 parts of DMP-30 (Rohm and Haas Co.)
1 1/2 volumes of Cabosil (Cabot Corp.)
per volume of EPON 815

3. After the initial epoxy coating dried, the surface was waxed and polished by using a hard auto wax. Then it was coated with a film of polyvinyl alcohol parting agent.

4. Next a layer of 10-oz Fiberglas cloth (P-900-75 made by the Ferro Corp.) was applied to the surface.

5. Finally, two coats of the following epoxy were applied:

100 parts of EPON 815
15 parts of polysulfide LP3
15 parts of Lancast A
3 parts of DMP-30
1/2 volume of Cabosil per volume of
EPON 815

The purpose of the wax and the polyvinyl alcohol film was to provide a slip surface between the initial epoxy coating and the final Fiberglas-reinforced epoxy coating. This idea came from discussions with Davidson of the United Kingdom Atomic Energy Authority.²⁸ He and his associates had found that devising a ductile, nonmetallic liner that would span even the smallest of cracks in a PCRV model was a difficult task. Apparently the internal pressure pressed the liners to the concrete so tightly that, at a crack, an infinite strain had to be carried over a zero length. They found that the situation was much improved by providing a slip surface between inner and outer liners so that even after the inner liner cracked, the outer liner would slip over it and thus distribute the strain caused by the crack.

The Cabosil was added to the epoxy formulations simply to control the viscosity of the mix. The amount used can be varied without affecting other characteristics.

4.2 Mortar Models

4.2.1 Microconcrete

The basic requirements for the microconcrete were that it match the compressive strength, the tensile strength (splitting tensile strength and modulus of rupture), the modulus of elasticity, and the ultimate compressive failure strain of the prototype concrete. Because of the unusually low modulus of elasticity-to-strength ratio exhibited by the prototype concrete, it was difficult to match all properties. Six different mixes were designed and tested in the process of obtaining a suitable model concrete.

The final mix proportions for a water-to-cement ratio of 0.52 on a one-bag basis were

Water, gal	5.8
Cement, lb	94
Mortar sand, lb	282
Aggregate, lb	94

The resulting mix had a slump of about 3/4 in.

Type I (general purpose) cement was used. The mortar sand was from a glacial deposit found near Ithaca, New York. The sand particles had a rounded appearance similar to that of river sand and consisted mainly of quartz, with a specific gravity of 2.65. The following gradation was used:

Tyler Mesh	Opening (in.)	Percent Retained	Percent Passing
3	0.263	0	100.0
6	0.131	8.4	91.6
10	0.065	21.5	70.1
20	0.0325	20.3	46.6
35	0.0164	21.5	28.1

The coarse aggregate was a nongraded, angular crushed rock that passed a 5/16-in. sieve. It was obtained from Warren Brothers in Canoga, New York.

Since some scale effect exists with respect to both the compressive strength and tensile strength of concrete specimens,³ all model test specimen dimensions were scaled by approximately the same scale factor as used for the models. This resulted in 2- x 4-in. cylinders for compressive

tests and splitting tensile strength tests and $2 \times 3 \times 14$ -in. beams for modulus of rupture tests.

A total of 18 cylinders for compressive tests, nine cylinders for splitting tensile strength tests, and nine beams for flexural tests were cast with each of the two mortar models. The specimens were treated in a way equivalent to those for the prototype vessel concrete. After casting, all specimens remained in a fog room for 28 days. At the end of 28 days one-third of the specimens for each model were tested; one-third were removed from the fog room, dried for several days, and sealed with shellac; and one-third were left in the fog room. These latter wet and dry specimens were tested at 90 days. All the properties tests were performed at Cornell University. Standard ASTM procedures for preparing and testing specimens were followed as closely as possible, with appropriate adjustments being made for the reduced size of the specimens.

Complete compressive stress-strain curves to failure were obtained for about one-third of the cylinders tested in compression. Transverse strain measurements were not made on the cylinders. Poisson's ratio values were obtained later at ORNL on prismatic specimens cut from spare flexural specimens.

Strains were measured with a compressometer made specifically for the 2×4 -in. cylinders. The compressometer consisted of an assembly that was fastened to the specimen with turnscrews at three equally spaced points around the cylinder. It operated over a $2 \frac{1}{2}$ -in. gage length. Two 0.0001-in. dial gages, on opposite sides of the cylinder, measured the strain. Some difficulty was met in obtaining equal strains from the two dial gages, but consistent stress-strain curves were obtained by careful use of the instrument. All cylinders were tested at a loading rate of 0.015 in./min, and they were capped at one end with a thin layer of Hydrostone about 3 hr before testing.

The compressive stress-strain curves obtained for the microconcrete are shown in Fig. 22. Since the mix was identical for both mortar models and there was no observable difference in specimen test results for the two models, the results of specimens from both were averaged together. For each moisture and age condition, then, the compressive strength shown is

Fig. 22. Compressive Stress-Strain Curves for Microconcrete.

the average from 12 tests. The modulus of elasticity and shape of each curve were determined from the specimens for which strain measurements were made. The Poisson's ratio plot shown was averaged from three tests performed on dry specimens at an age of 270 days. The curves in Fig. 22 are directly comparable with those in Fig. 20 for the prototype concrete.

The splitting tensile strength tests utilized $1/4 \times 1/2 \times 4$ -in.-long strips of nontempered Masonite between specimen and testing machine heads. The specimens were loaded at a rate of 0.02 in./min. Considering both mortar models, a total of six specimens was tested for each of the three age and moisture conditions.

Six flexural specimens were also tested for each of the age and moisture conditions. The span length used for the $2 \times 3 \times 14$ -in. beams was

12 in., and the two loads were applied at the third points. The specimen loading rate was 0.02 in./min

The significant properties of the model microconcrete are summarized in Table 5. The values given are the averages for the specimens tested at that condition. The ranges of the values, where available, are also indicated.

Comparison of the values in Table 5 with those for the prototype vessel concrete given in Table 4 indicates the degree to which the model microconcrete duplicated the significant properties of the prototype concrete. The 90-day data are probably a more significant basis of comparison than the 28-day data. The comparison may be summarized as follows on the basis of both wet and dry 90-day data:

<u>Property</u>	<u>Model Compared with Prototype</u>
Modulus of elasticity	23% higher
Poisson's ratio	Same
Compressive strength	10% lower
Splitting tensile strength	24% higher
Adjusted modulus of rupture	23% higher

The prototype concrete properties were not modeled as well as they usually are in microconcrete modeling because, as discussed previously, the unusually low modulus of elasticity of the prototype concrete made the task more difficult. The differences, however, were not as significant as might have been expected, as was demonstrated by the final model test results. For mechanical loadings only (pressure and prestressing) a variation of the modulus of elasticity of the concrete relative to that of the steel in a vessel has a relatively insignificant effect on the elastic stress distributions. Likewise, in a prestressed structure a relatively small change in tensile strength has a very small influence on the failure strength of the structure. Since the microconcrete tensile strength was higher than that of the concrete, it would be expected that the mortar models would crack at a slightly higher value of internal pressure than the concrete prototype.

Table 5. Summary of Properties of Mortar Model Microconcrete

	28-Day-Cured Wet Concrete		90-Day-Cured Wet Concrete		90-Day-Cured Dry Concrete	
	Average	Range (% of mean)	Average	Range (% of mean)	Average	Range (% of mean)
Modulus of elasticity, psi	3.8×10^6		4.2×10^6		4.3×10^6	
Poisson's ratio					0.18 ^a	
Compressive strength, psi	6840	17.3	6960	23.6	7010	35.1
Splitting tensile strength, psi	761	9.9	743	18.9	762	12.1
Modulus of rupture, psi	803	23.2	1189	23.5	1014	31.4
Adjusted modulus of rupture, ^b (psi)	698		1035		882	

^aPoisson's ratio was obtained from dry specimens at an age of 270 days.

^bBecause of a significant size effect, modulus-of-rupture data for 3-in.-deep beams are generally about 15% above data for 6-in.-deep beams.³ The adjusted modulus values reflect this difference and provide a better basis for comparison with prototype concrete values.

4.2.2 Stressteel Bars

The certified load-strain curve for the heat of 1/2-in.-diam bars from which all the mortar model prestressing tendons were made is shown in Fig. 23. The actual ultimate strength of the 1/2-in. bars was 23% higher than the guaranteed minimum of 28,000 lb.

The properties of the model bars compare reasonably well with those of the prototype given in Fig. 21. The important properties are the tensile strength, the yield strength, the modulus of elasticity, and the elongation. All these properties for the model bars differed from those of the 1 3/8-in.-diam prototype head and circumferential bars by less than 10%. The tensile strength of the model bars was 10.5% higher than that of the 1 1/8-in.-diam prototype axial bars, the yield strength was 7.3% higher, the modulus of elasticity was 10.4% lower, and the elongation was 14% higher.

Fig. 23. Tensile Stress-Strain Curve for Stressteel Bars Used in Mortar Models.

4.3 Epoxy Models

The 1/5-scale prestressed epoxy model and the axisymmetric epoxy model were machined from solid castings of identical material. The epoxy resin consisted of

	Parts by Weight
Bakelite ERL-2774	100
Phthalic anhydride	42
Hexahydrophthalic anhydride	20

This material is referred to by Leven²⁹ as standard epoxy for photoelastic use. Although only the axisymmetric model was tested photoelastically, 1 ml of silica gel (trade name, Cab-O-Sil) was added per pound of mix to the material for both models. This additive improves the intensity of scattered light in scattered-light photoelastic models. The casting procedure and curing cycle recommended by Leven were used for the epoxy material.

The stress-strain properties of the epoxy were examined by testing eight 1/2-in.-diam tensile specimens taken from the same castings as were used for the models. Each specimen was instrumented with two diametrically opposed axial strain gages and two diametrically opposed transverse strain gages. The gages were wired in series so that bending effects were eliminated and average values of the axial and transverse strains were recorded. The curves shown in Fig. 24 are representative of the results obtained from the eight tests. The modulus of elasticity was 0.5×10^6 and Poisson's ratio was 0.38. These values are identical to those obtained for the same material, without the Cab-O-Sil, used in a previous epoxy strain-gage model.³⁰

Mechanical properties tests were not performed on the magnesium used for the base plate of the prestressed epoxy model nor on the aluminum used for the axisymmetric epoxy model base plate. Rather, standard handbook values of the elastic properties were used in the finite-element analyses. The modulus of elasticity was assumed to be 6.5×10^6 psi for the magnesium

Fig. 24. Tensile Stress-Strain Curve for Epoxy.

and 10.5×10^6 psi for the aluminum. Poisson's ratio was taken as 0.3 for both materials.

5. FINITE-ELEMENT ANALYSES

The technologies of prestressed-concrete reactor vessels and of finite-element analysis have, in a sense, developed together. The massive and heterogeneous PCRV's, with their thick walls, steel liners, prestressing tendons, and reinforcing bars, presented a formidable problem for conventional methods of stress analysis but one that was ideally suited to the newly emerging finite-element method. Consequently, finite-element analyses are now routinely employed in the design and analysis of PCRV's in the United States, France, Great Britain, and other countries.

These analyses have been predominantly elastic and two dimensional and are based on either axisymmetric or plane assumptions. Generally, an axisymmetric finite-element computer program is used for the bulk of the analyses to be performed, and a plane finite-element program is used to examine on an approximate basis the effects of nonaxisymmetric features, such as head penetrations, circumferential prestressing arrangements, and non-circular vessel cross sections.³¹ The results of these plane analyses are then used in conjunction with and to modify the axisymmetric analysis results to approximate the true vessel behavior.

Recently three-dimensional finite-element computer programs were developed and added to the methods available to the PCRV analyst.³² At present these three-dimensional programs require large amounts of input preparation and output interpretation. Consequently, although they have the capability of performing very detailed analyses of the nonaxisymmetric features of a PCRV, they are used only sparingly. The two-dimensional elastic programs remain the principal tools of the PCRV stress analyst and, as was stated in Chapter 1, one of the basic purposes of this investigation was to evaluate the adequacy and relative accuracy of both two- and three-dimensional finite-element analyses applied to PCRV's.

The two- and three-dimensional elastic finite-element programs that were used in the investigation were all developed by Gulf General Atomic (formerly General Atomic Division of General Dynamics) for the U.S. Atomic Energy Commission and, although readily applicable to other types of structures, they were written specifically for PCRV analyses. Two different axisymmetric programs, SAFE-AXISYM³³ and SAFE-PCRS,³⁴ were used. The basic

difference in the two programs is that in the former the equilibrium equations are solved by an indirect, or iterative, method, while in the latter a direct solution is utilized.³⁵ For plane analyses the program SAFE-PLANE³⁶ was used, in which, as in SAFE-AXISYM, an indirect solution method is used. Finally, the three-dimensional analyses were performed with SAFE-3D.^{37,38}

In the plane program the structure to be analyzed is divided into an assemblage of triangular elements connected at their corners. These points of connection are referred to as nodal points. Within each triangular element the displacement components u and v are assumed to vary linearly with the coordinates x and y . Thus the stresses and strains within each element are constant, and consequently this type of element is referred to as a constant-strain triangle.

On the basis of the assumed simplified form of the displacements, the element stiffnesses (relations between displacement components imposed on the nodes and the resulting nodal forces) are determined, and the equilibrium equations for the overall structure are formulated by the direct stiffness method. The equilibrium equations, which are a set of linear algebraic equations, are equivalent to specifying that the sum of the components of the forces acting on each nodal point in the structure be equal to zero or to the applied force on that nodal point. Thus there are two equations and two unknowns (the u and v nodal displacements) for each nodal point in the structure, except those with known specified displacement boundary conditions.

In the axisymmetric programs a meridional cross section of the structure to be analyzed is divided into an assemblage of triangles. Rather than being flat triangular plates as in the plane case the elements in the axisymmetric analysis are actually circular rings with triangular cross section. The displacements u and v correspond to the radial and axial coordinates and, as in the plane analysis, they are assumed to vary linearly within each element. However, because of the form of the strain-displacement relations for the axisymmetric case the circumferential strains and, consequently, the stresses are not constant within the elements.

The three-dimensional program is a direct extension of the plane analysis to a third dimension. In place of triangular elements, tetrahedral

elements are used, and the three displacement components u , v , and w are assumed to vary linearly with the three coordinates x , y , and z . Thus the stresses and strains are again constant within each element. The final set of equilibrium equations consists of three equations for each nodal point in the structure, and the three displacement components at each nodal point are the unknowns.

The remainder of this chapter is divided into three principal sections. In the first the two-dimensional elastic analyses, both axisymmetric and plane, are described; in the next the three-dimensional analysis is described; and the final section contains a discussion of an approximate two-dimensional cracking analysis that was performed by using one of the axisymmetric elastic programs.

5.1 Two-Dimensional Elastic Analyses

5.1.1 Axisymmetric Analyses

The element layout for the axisymmetric analyses performed on the concrete prototype vessel is shown in Fig. 25. There were 831 elements (808 triangles plus 23 bar elements representing the steel reinforcing) and 473 nodal points. The analyses of this layout were all made with the SAFE-AXISYM program.

In the axisymmetric idealization, the basic hexagonal outside shape of the vessel was approximated by an inscribed right circular cylinder having a 24-in. radius. The prestressing anchor loads were represented by simple ring-line loads, the penetrations in the top head were "smeared out" to give a homogeneous region of equivalent reduced stiffness, as described below, and of course all the other nonaxisymmetric features of the vessel were ignored. The reinforcing-bar cages in the base region were included.

The modulus of elasticity and Poisson's ratio for the concrete were taken as 3.5×10^6 psi and 0.15, respectively. The latter differs slightly from the 0.18 given in the previous chapter for the prototype concrete, and the bulk of the analyses was performed before the exact value was obtained. However, this small difference in the Poisson's ratio had a negligible

Fig. 25. Axisymmetric Finite-Element Layout.

effect on the results. The modulus of elasticity for the steel parts was taken to be 30×10^6 psi.

The array of penetrations in the top head had a twofold effect. First, the penetrations reduced the effective stiffness of that portion of the head, and this in turn affected the remaining portions of the vessel. Second, they produced local stress concentrations in the penetration region. By replacing the penetration region with a homogeneous material

of equivalent reduced stiffness, the first effect was approximately accounted for. This is commonly done in PCRV analyses.³¹ The effective elastic constants to be used in the region of reduced stiffness were initially determined from Ref. 39, which gives the effective constants for perforated plates with triangularly pitched circular holes. The values are for in-plane loading, but the authors show that the same constants apply to bending loadings if the ratio of plate thickness to hole pitch is sufficiently large.

The effective constants obtained from Ref. 39 were $E^* = 0.67E$ and $\nu^* = \nu$, where E and ν are the modulus of elasticity and Poisson's ratio, respectively, and the asterisks denote effective values. The vessel head thickness was 1.4 times the penetration pitch. Results given in Ref. 39 show that for head thickness-to-hole pitch ratios less than about 2.0 the effective constants for bending loads are higher than those for in-plane loads. Thus the above values were slightly low, since the loading on the vessel head was mostly bending.

The initial values were determined and used before the steel penetration liner thickness had been established, and thus the effect of the steel liners was not considered. The liners increased the stiffness of the penetration region, and in fact the liner thickness was chosen to approximately compensate for the concrete removed.

The plane finite-element analysis of the head penetration region, described below, took into account the steel liners and thus provided not only the stress concentration values in the penetration region but also information from which the effective modulus could be calculated. The modulus determined by this method was $E^* = 0.94E$. Even this value is somewhat low because it was determined for in-plane loading rather than for bending. In conclusion then the actual effective modulus for bending effects in the penetration region was estimated to be very nearly equal to the actual modulus of the concrete, so $E^* \sim E$.

Since the axisymmetric analyses had been carried out with the initial effective modulus estimate of $E^* = 0.67E$, a few axisymmetric cases were re-run with $E^* = E$ to determine the error involved in the original runs. The stresses and deformations outside the penetration region were affected

little by the change. Within the penetration region, differences up to 14 or 15% were observed. Since the procedure for treating the penetrations were initially only approximate, the entire set of axisymmetric analyses was not rerun, and the axisymmetric results presented here were all obtained with the original estimate of the effective modulus, $E^* = 0.67E$.

With regard to the penetrations the additional assumption was made that the pressure acting over the penetration areas was carried by the head, whereas it was actually transferred through the tie rods to the bottom steel plate.

Two types of axisymmetric analyses were run for the concrete prototype. In the first the design loading conditions were analyzed, and three specific cases were examined. First, the design pressure of 500 psi and the design level prestressing of $0.6f'_s$ were examined; second, the design pressure of 500 psi alone was examined; and finally, the initial design prestressing level of $0.7f'_s$ was treated. Plots of the stress trajectories and stress levels for each of these three cases are presented later.

The second type of axisymmetric analysis was referred to as a unit load analysis. The seven unit loadings separately analyzed consisted of a unit internal pressure, a unit axial prestressing load, a unit head prestressing load, and unit loads for the first, second, third, and fourth circumferential prestressing bands (see Fig. 25). The results for each case were then treated as a matrix, and a program was written to add these matrices in any specified linear combination. In this way several combinations of loadings were examined without performing a finite-element analysis of each case. All the various steps in the prestressing sequence and pressurization were examined in this manner.

The analyses described above and the results presented in the following section are for the concrete prototype. Finite-element analyses were performed separately for the prestressed epoxy model, since the difference in Poisson's ratio between concrete and epoxy (0.18 versus 0.38) was fairly substantial and would have some effect on the calculated stresses. Thus although the experimental results for the mortar models were compared with the analytical predictions for the concrete prototype, the experimental

results for the prestressed epoxy model were compared only with analytical predictions actually for that model.

Since the axisymmetric epoxy model was shaped differently in the base region, a completely different analysis from the above was performed on it. The program SAFE-PCRS was used. Essentially the same element layout as shown in Fig. 25 was used in the head and upper walls. A total of 790 elements and 471 nodes was used.

5.1.2 Axisymmetric Results for Design Loadings - Concrete Prototype

The results for the case of the design pressure of 500 psi plus the design prestressing of $0.6f'_s$ are presented in Figs. 26 through 32. This case represents the design operating state of the prototype vessel.

The principal stress trajectories, which are lines everywhere tangent to the directions of maximum and minimum principal stress, are shown in Fig. 26. These are the directions of principal stress in the meridional plane. Because of symmetry, the third principal stress always lies in the circumferential direction.

Figures 27 through 32 are contour plots of the radial, circumferential, axial, shear, maximum, and minimum stress levels, respectively. An examination of Figs. 28, 31, and 32, which are for the three principal stresses, shows that at the design pressure level the predicted vessel stresses are entirely compressive, except for a region at the haunch, where the maximum stress is just beginning to go into tension. Thus, as specified in Chapter 2, the average stress on any vessel cross section was compressive, and the maximum tensile stresses were less than 232 psi.

Figures 33 through 39 are analogous to Figs. 26 through 32, except that they are for an internal pressure of 500 psi only. By superposing percentages of the stress levels shown for pressure only to the operating stress levels, the effects of increasing pressure may be seen. Care must be taken of course to remember that the maximum and minimum stresses are directional and can be added to any point only if they have the same direction.

These considerations indicate that with increasing internal pressure above 500 psi the tensile strength of the concrete would soon be exceeded

Fig. 26. Principal Stress Trajectories for Prestressing Plus 500-psi Internal Pressure.

Fig. 27. Radial Stresses for Prestressing Plus 500-psi Internal Pressure.

Fig. 28. Circumferential Stresses for Prestressing Plus 500-psi Internal Pressure.

Fig. 29. Axial Stresses for Prestressing Plus 500-psi Internal Pressure.

Fig. 30. Shear Stresses for Prestressing Plus 500-psi Internal Pressure.

Fig. 31. Maximum Principal Stresses for Prestressing Plus 500-psi Internal Pressure.

ORNL-DWG 67-4352

Fig. 32. Minimum Principal Stresses for Pre-stressing Plus 500-psi Internal Pressure.

ORNL-DWG 67-4343

Fig. 33. Principal Stress Trajectories for 500-psi Internal Pressure.

ORNL-500 67-4383

Fig. 34. Radial Stresses for 500-psi Internal Pressure.

ORNL-500 67-4384

Fig. 35. Circumferential Stresses for 500-psi Internal Pressure.

Fig. 36. Axial Stresses for 500-psi Internal Pressure.

Fig. 37. Shear Stresses for 500-psi Internal Pressure.

ORNL-ORNL 67-4387

Fig. 38. Maximum Principal Stresses for 500-psi Internal Pressure.

ORNL-ORNL 67-4388

Fig. 39. Minimum Principal Stresses for 500-psi Internal Pressure.

by the maximum stress in the haunch region and cracking would occur. Likewise, examination of Figs. 28 and 35 for circumferential stresses shows that at 500 psi the predicted circumferential, or hoop, stress in the cylinder wall is about 500-psi compression, but at 1000 psi the circumferential stress level would be about 500-psi tension, which is near the tensile strength of the prototype concrete. Thus at about 1000-psi internal pressure, major vertical cracking would be expected in the walls.

The final set of figures of this series, Figs. 40 through 46, shows the predicted stress state immediately after the initial prestress of

Fig. 40. Principal Stress Trajectories for Initial Prestressing.

Fig. 41. Radial Stresses for Initial Prestressing.

Fig. 42. Circumferential Stresses for Initial Prestressing.

Fig. 43. Axial Stresses for Initial Pre-stressing.

Fig. 44. Shear Stresses for Initial Pre-stressing.

Fig. 45. Maximum Principal Stresses for Initial Prestressing.

Fig. 46. Minimum Principal Stresses for Initial Prestressing.

$0.7f'_s$ is applied. Figures 42 and 46, which show the circumferential and minimum principal stress levels in the vessel, respectively, indicate that the maximum compressive stresses were well below the 2700-psi limit specified in Chapter 2.

5.1.3 Plane-Stress Analysis of Head Penetration Region

The element layout for the plane-stress analysis of the top-head penetration region is shown in Fig. 47. Because of symmetry conditions, only a 30° symmetry sector was considered. The outer radius of the section analyzed was taken at the point where the head meets the haunch on the

Fig. 47. Plane Finite-Element Layout for Analysis of 30° Sector of Head Penetration Region.

vessel interior. There was a total of 945 elements and 537 nodes in the layout. The inner layer of elements at each penetration was considered to be steel and represented the 1/8-in.-thick steel liner. The analysis was carried out by using the SAFE-PLANE program.

The section was assumed to be acted upon by an in-plane traction of 1000 lb/in. applied in the positive radial direction along the outer boundary circle. Thus the slab would have exhibited in-plane normal stresses of 1000 psi everywhere if it were not for the effects of the penetrations. At any given point, any deviation from 1000 psi ratioed to 1000 psi gave a factor that could be used to approximately adjust the axisymmetric results to account for the penetrations.

The same material constants were used as for the axisymmetric analyses. The modulus of elasticity and Poisson's ratio for the concrete were taken to be 3.5×10^6 psi and 0.15, respectively, and for the steel the values were 30×10^6 psi and 0.3.

In addition the penetration region of the prestressed epoxy model was analyzed. The same element layout as above was used, but since the epoxy penetration had no liners, all elements were taken to be epoxy. The modulus of elasticity and Poisson's ratio used were 0.5×10^6 psi and 0.38, respectively. The significant results for both the concrete prototype and the prestressed epoxy model are given in the next section.

5.1.4 Head Penetration Analysis Results - Concrete Prototype and Prestressed Epoxy Model

The general results from the concrete prototype head penetration region analysis are given in Figs. 48, 49, and 50. The principal stress trajectories are shown in Fig. 48, while the radial and circumferential stress levels are shown in the contour plots of Figs. 49 and 50, respectively. The fact that these contour levels are everywhere relatively close to the applied stress level of 1000 psi indicates the effectiveness of the steel liners in compensating for the concrete removed.

The instrumentation layout for the models is described in Chapter 6. Basically, however, the instrumentation was located in three main meridional planes - one across the flats of the outside hexagonal shape and one across each of the adjoining vertices. The experimental results were

Fig. 48. Principal Stress Trajectories in Head Penetration Region.

Fig. 49. Radial Stresses in Head Penetration Region.

Fig. 50. Circumferential Stresses in Head Penetration Region.

compared with finite-element predictions in these planes. Thus what was needed from the penetration analyses were factors by which the radial and circumferential stresses predicted by the axisymmetric analyses could be multiplied to approximate the stresses on these planes in the penetration region. These factors can be obtained from Fig. 51 for the concrete prototype and Fig. 52 for the prestressed epoxy model. These figures are plots of the radial and circumferential stress distributions obtained from the penetration region analyses on a line across the flats of the outer hexagonal shape and on a line across the vertices of the outer hexagonal shape.

5.1.5 Plane-Strain Analysis of Vessel Wall Cross Section

To approximately determine the effects of the nonaxisymmetric features of the cylindrical wall of the vessel, a series of generalized (constant axial strain) plane-strain analyses of a section of wall taken through the heavy steel prestressing anchor blocks was carried out. The element layout for these analyses is shown in Fig. 53. Because of symmetry conditions, only a 30° symmetry sector was considered. The outline of one of the steel

Fig. 51. Radial and Circumferential Stress Distributions in Head Penetration Region of Concrete Prototype.

Fig. Penetrat

Fig. 52. Radial and Circumferential Stress Distributions in Head Penetration Region of Prestressed Epoxy Model.

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Fig. 53. Plane Finite-Element Layout for Analysis of 30° Sector of Vessel Side Wall.

anchor blocks is shown by heavy lines on the element layout. There was a total of 469 elements and 266 nodes. The analyses were carried out by using the SAFE-PLANE computer program with the elastic constants of the elements modified to convert from a plane-stress analysis to a generalized plane-strain analysis.

The materials constants used in the analyses for the concrete prototype were the same as for the axisymmetric analyses. The modulus of elasticity and Poisson's ratio for the concrete elements were 3.5×10^6 psi and 0.15, respectively, and the values for the steel elements were 30×10^6 psi and 0.3. The analyses were carried out for the same three design loading conditions as considered in the axisymmetric case, and the results are presented in the following section.

It should be emphasized that, like the axisymmetric analysis results, these results are only approximate. They would be strictly correct only for an infinitely long cylinder having the uniform cross section depicted in Fig. 53. Actually, the cylindrical wall is neither infinitely long nor of uniform cross section along its length. Thus the in-plane stresses shown in the following section should be viewed as an overestimate of the variation around the vessel wall. The true variation would be expected to lie between the zero variation predicted by the axisymmetric analyses and the variation predicted by the plane-strain analyses.

5.1.6 Wall Analysis Results for Design Loadings - Concrete Prototype

The results for the case of the design pressure of 500 psi plus the design prestressing of $0.6f'_g$ are presented in Figs. 54 through 59. This case represents the design operating state of the prototype vessel, and the results shown correspond to the axisymmetric analysis results shown in Figs. 26 through 32. An examination of Fig. 56, which is a plot of the circumferential stress levels, indicates that the predicted variation around the vessel wall is quite significant, especially on the inside surface. All the stresses in the concrete are compressive, except for a small region on the outer surface that is just beginning to change to tension.

Fig. 54. Principal Stress Trajectories for Prestressing Plus 500-psi Internal Pressure.

Fig. 55. Radial Stresses for Prestressing Plus 500-psi Internal Pressure.

Fig. 56. Circumferential Stresses for Prestressing Plus 500-psi Internal Pressure.

Fig. 57. Shear Stresses for Prestressing Plus 500-psi Internal Pressure.

ORNL-DWG 68-1403

Fig. 58. Maximum Principal Stresses for Prestressing Plus 500-psi Internal Pressure.

ORNL-DWG 68-1404

Fig. 59. Minimum Principal Stresses for Prestressing Plus 500-psi Internal Pressure.

Figures 60 through 65, analogous to Figs. 54 through 59, are for an internal pressure of 500 psi only. Figure 62, which shows the circumferential stress levels, again indicates a significant variation around the vessel wall. A comparison of Fig. 62 with Fig. 35, which showed the circumferential stress levels from the axisymmetric analysis for an internal pressure of 500 psi alone, indicates that the axisymmetric analysis considerably overestimated the circumferential stress level on the inside surface at the meridional plane taken across the flats of the outside hexagonal shape. This will be readily apparent in Chapter 8 when experimental stresses for pressure only are compared with axisymmetric predictions.

The final set of figures in this series, Figs. 66 through 71, show the stresses due to the initial prestress of $0.7f'_s$. The minimum principal stress levels are shown in Fig. 71. Predicted stresses in the concrete are below the 2700-psi compressive limit specified in Chapter 2, except in a small region immediately adjacent to the anchor block. This region would be classified as a point of obvious stress concentration and thus is not subject to the 2700-psi limit.

5.2 Three-Dimensional Elastic Analysis

The analyses described above were all two dimensional and, as such, treated the true vessel geometry only approximately. As may be seen in Chapter 8, a fairly good prediction of the stress variation throughout the models was obtained from these two-dimensional analyses. However, to correctly treat the nonaxisymmetric structure of the models, a three-dimensional analysis was required, and this was undertaken with the use of the three-dimensional program SAFE-3L.

The idealized structure analyzed is shown in Fig. 72a. It consisted of a 30° symmetry sector of the top half of the prototype vessel. The more important features of the vessel, including the outside hexagonal shape, the large steel prestressing anchor blocks for the circumferential tendons, and the steel anchor plates for the axial and head tendons, were retained in the model that was analyzed.

Even in the three-dimensional analysis some approximations were necessary. Only the top half of the prototype vessel was considered, even

ORNL-DWG 68-1411

Fig. 60. Principal Stress Trajectories for 500-psi Internal Pressure.

ORNL-DWG 68-1412

STRESS LEVELS (psi)

+1600	+200
+1400	+100
+1200	0
+1000	-100
+800	-200
+600	-300
+500	-400
+400	-500
+300

Fig. 61. Radial Stresses for 500-psi Internal Pressure.

ORNL-DWG 68-1415

Fig. 62. Circumferential Stresses for 500-psi Internal Pressure.

ORNL-DWG 68-1415

Fig. 63. Shear Stresses for 500-psi Internal Pressure.

ORNL-DWG 68-1414

Fig. 64. Maximum Principal Stresses for 500-psi Internal Pressure.

ORNL-DWG 68-1413

Fig. 65. Minimum Principal Stresses for 500-psi Internal Pressure.

Fig. 66. Principal Stress Trajectories for Initial Prestressing.

Fig. 67. Radial Stresses for Initial Prestressing.

Fig. 68. Circumferential Stresses for Initial Prestressing.

Fig. 69. Shear Stresses for Initial Prestressing.

ORNL-ORR 68-1407

Fig. 70. Maximum Principal Stresses for Initial Prestressing.

ORNL-ORR 68-1406

Fig. 71. Minimum Principal Stresses for Initial Prestressing.

Fig. 72. Finite-Element Idealization for Three-Dimensional Analysis.

though the head and base were not identical. They were assumed to be sufficiently close in behavior for the top half alone to be analyzed, with symmetry conditions imposed at the midplane. Also the three pairs of head prestressing tendons were actually at different levels within the head to permit their crossing at the center; the middle layer was chosen as representative for the idealization. The two meridional planes bounding the 30° sector analyzed were two of the three planes that were extensively

instrumented in the models. This was an added motivation for choosing this particular sector for analysis.

The head penetrations were not included in the analysis since, as has been shown, the steel liners compensated for the penetrations almost entirely. Stress concentrations do, however, occur around the penetrations, and the results of the plane penetration region analysis, as summarized in Fig. 51, were used with the three-dimensional results to predict the actual stresses in the penetration region.

The three-dimensional finite-element idealization used is shown in Fig. 72b. There were 2,678 nodes and 12,231 tetrahedral elements. The elements shown in Fig. 72b are mostly octahedrons made up of three-component tetrahedrons. The octahedral elements were the basic input elements to the program and were subdivided internally into tetrahedrons. The epoxy liner and the small amount of steel reinforcing under the anchor plates were ignored in the idealization.

The modulus of elasticity and Poisson's ratio for the concrete were taken as 3.5×10^6 psi and 0.15, respectively. For the steel parts the corresponding values were 30×10^6 and 0.3.

Two loading cases were considered that represented design conditions. First, the initial prestressing loads of $0.7f'_g$ alone were considered and, second, the internal design pressure of 500 psi alone was considered. The output for these two analyses was voluminous, and no attempt is made here to present typical or representative results. Rather, the predicted stresses and deflections in the two bounding meridional planes are given in Chapter 8 and compared with experimental results and with two-dimensional finite-element predictions.

5.3 Two-Dimensional Cracking Analysis

The axisymmetric finite-element analysis of the concrete prototype was also used to predict and examine the effects of incipient cracking during overpressurization of the vessel. The cracking analysis was intended to predict, insofar as possible within the axisymmetric limitations of the analysis, the pressure level at which cracking began, the location

of these cracks, their rate of growth with increasing pressure, and finally the deformation behavior of the vessel with increasing pressure.

The maximum tensile stress theory was assumed to govern cracking, and the failure stress was taken as 605 psi. This value is the average of the splitting tensile strengths of the 90-day wet specimens and the 90-day dry specimens given in Table 4. Whenever the maximum principal stress was predicted to exceed 605 psi at any location, a crack was assumed to form perpendicular to the direction of the maximum stress, the finite-element mesh was altered accordingly to simulate the presence of the crack, and the analysis was continued. This procedure works only so long as cracks are not initiated by circumferential stresses. When they are the axisymmetry of the model is destroyed, and the analysis must be terminated.

The basic finite-element layout that was shown in Fig. 25 was used for the cracking analysis. Based on the results previously presented for the cases of prestressing plus the 500-psi design pressure and of pressure alone, the first crack was predicted to occur at a pressure of 600 psi and was located in the haunch.

The finite-element mesh layout in the region of the haunch is shown redrawn in Fig. 73. The initial crack was assumed to extend through the nodal point on the surface of the haunch. To simulate the crack the first node was replaced by two nodes so that adjacent elements were disconnected. A crack extending to the second node was thus simulated. The pressure levels in Fig. 73 indicate the crack depth at the given level. Because of axisymmetric requirements the crack was necessarily assumed to extend completely around the vessel in the circumferential direction.

For the revised mesh with the node on the surface replaced by two nodes the 600-psi pressure level was reexamined to determine the vessel behavior at 600 psi with the crack and to see whether the crack would progress further at 600 psi. Examination of the stresses in the crack vicinity indicated that the crack had not progressed further, and no other crack had formed. The analysis was then repeated for an internal pressure of 700 psi, and on the basis of the results for 600 and 700 psi it was projected that further cracking at the tip of the existing crack would occur at 800 psi.

Fig. 73. Crack-Propagation Analysis in Haunch Region.

The analysis was continued in the manner described above, and it was found that another increment of crack growth occurred at 900 psi. The element mesh was altered for the third time, and the vessel was reexamined at 900 and 950 psi. At 950 psi it was found that the circumferential stresses in a large portion of the cylindrical wall of the vessel had either exceeded or were close to the tensile strength of 605 psi. Thus major longitudinal cracking was indicated in the wall of the vessel, the axisymmetry was destroyed, and the cracking analysis was terminated. The results obtained at 900 and 950 psi indicated that the vertical cracking actually would occur at an internal pressure of 940 psi.

An examination of the results obtained at 900 and 950 psi also showed that the radial and circumferential stresses on the outside surface of the head would produce shallow cracking near the center of the head at an internal pressure slightly above 900 psi. Also, it was estimated from the results that horizontal cracking would begin on the outer surface of the cylinder near midheight at an internal pressure of about 1175 psi.

The typical deformation behavior with increasing pressure predicted by the series of cracking analyses is shown in Fig. 74, where the center deflection of the head relative to the deflection at the outer edge of the head is plotted as a function of internal pressure. The light straight lines show the behavior predicted by the series of linear analyses. The smooth curve drawn through the predictions depicts the actual expected behavior with increasing overpressure.

This predicted head deflection is shown in Chapter 8 and compared with experimental measurements. The agreement is reasonably good, particularly for the pressure level at which the nonlinear response begins. There was no way of knowing exactly when the initiation of cracking actually occurred in the haunch, but indications are that it occurred at about the pressure level predicted by the analysis and that the longitudinal cracks in the walls also occurred at about the predicted level.

Other investigators^{40,41} used a similar cracking analysis procedure and tried both the maximum principal stress failure theory and the maximum principal strain failure theory. They found that the maximum principal stress theory predictions were considerably in error, while the maximum

Fig. 74. Predicted Deflection of Center of Prototype Vessel Head Versus Internal Pressure.

principal strain predictions were much closer to observed behavior. Since in the present investigation the maximum stress theory predictions agreed fairly well with observed behavior, predictions were not made with the maximum strain theory.

6. INSTRUMENTATION

Several types of instrumentation were used in the experimental portions of the model study program. These included bonded resistance strain gages on the outer and inner surfaces of the models, embedded strain gages in the concrete prototype and mortar models, bonded resistance strain gages on the reinforcing steel, bonded resistance strain gages on the prestressing tendons, load transducers at prestressing tendon anchorages, crack-detection gages or strips that were actually long single-element bonded resistance strain gages, dial gages for deflection measurements, pressure transducers for recording internal pressures, and photoelastic coatings on the concrete prototype and mortar models for indicating the location and growth of initial cracking.

The concrete prototype had a full complement of instrumentation consisting of 268 surface strain gages, 20 embedded strain gages, 49 reinforcing-bar strain gages, 11 strain-gaged tendons, 11 load transducers, 15 crack-detection strips, 1 pressure cell, and 19 dial gages, plus the photoelastic coating. There were thus 375 electrical instrumentation points plus the dial gages and photoelastic coating.

Of the models, only the second mortar model, which was tested hydraulically, had essentially the full complement of instrumentation matching that of the prototype. The first mortar model and the prestressed epoxy model had a reduced amount of instrumentation. The instrumentation that was used on these models did, however, have a direct correspondence to portions of the instrumentation used on the prototype vessel.

The instrumentation layout for the concrete prototype is presented in the following section of this chapter, and the instrumentation used for the models is discussed relative to that used for the prototype. Later sections describe the instrumentation in detail, including the types of gages, the methods of application, and the wiring and compensation schemes. Finally, the methods of data acquisition are discussed.

6.1 Instrumentation Layout

The instrumentation used for the concrete prototype, the mortar models, and the prestressed epoxy model was located principally in the three vertical planes labeled planes 1, 2, and 3 in Fig. 75. Plane 2 was considered to be the primary plane of interest, since the design of the vessel was based on an idealized axisymmetric structure having the meridional cross section cut by plane 2. The axisymmetric finite-element analyses were based on this cross section, and consequently most of the analytical predictions were comparable to the experimental data obtained in plane 2.

Fig. 75. Three Principal Planes of Instrumentation.

Planes 1 and 3, adjoining plane 2, were instrumented to provide data for studying the variation of strains and stresses around the vessel as affected by the hexagonal outer shape of the vessel and by the nonaxisymmetric features of the prestressing system arrangement. The photoelastic coating was applied to a 60° sector of the vessel's outer surface as shown in Fig. 75. The sector chosen was opposite the three main planes of instrumentation.

The locations of the surface, embedded, and reinforcing-bar strain gages are shown in Fig. 76. With only a few exceptions, the surface gages on the outer and inner surfaces were located in planes 1, 2, and 3. The exceptions are gages that were located adjacent to several of the prestressing-tendon anchor plates and adjacent to one of the top-head penetrations. The surface gages were generally arranged in two- or three-gage rosettes. With the two-gage rosettes, stresses could be calculated from strains only after the principal directions were assumed to coincide with the meridional and circumferential directions. With the three-gage rosettes the principal directions could be determined, so these results were used as a check of the validity of the two-gage rosette results. At the vertices of the outer hexagonal surface of the vessel, only single vertical gages were necessary because all stress components except the meridional stress were necessarily zero.

The surface gages were numbered according to the plane in which they were located. The 1XX series was in plane 1; the 2XX series, in plane 2; and the 3XX series, in plane 3. The prefix C denoted surface gages, and the suffixes 1, 2, and 3 designated circumferential, 45° , and meridional gages, respectively.

In addition to the concrete surface gages shown in Fig. 76, there were 14 bonded resistance strain gages on the bottom steel plate. Since these gages were not in an area of primary interest the results obtained from them are not presented here.

The embedded strain gages were grouped into five four-gage rosettes, and there was one rosette in each of five horizontal planes in the vessel. These planes are labeled in the elevation view in Fig. 76, and the location of the gages in these planes is shown in section B-B. Three of the four

gages in each rosette were located in a meridional plane, and the fourth gage was oriented in the circumferential direction. With the assumption of axisymmetry, these four gages were sufficient to determine the complete stress state at each rosette location.

The embedded gages were numbered similarly to the surface gages. The prefix E denoted an embedded gage, and the suffixes 1, 2, 3, and 4 designated radial, 45° , meridional, and circumferential gages, respectively. This numbering scheme is illustrated in Fig. 76.

All but three of the reinforcing-bar strain gages were located in planes 1 and 2 and are shown in the lower right-hand corner of Fig. 76. The prefix R on the gage number denoted that the gage was a reinforcing-bar gage, and the suffixes 1 and 3 designated gages located along the axis of a circumferential bar and along the axis of a meridional bar, respectively.

The prestressing-tendon strain gage and load transducer locations are shown in Fig. 77. Eleven tendons were chosen for strain gages and load transducers. The first four, numbered 1 through 4, were axial tendons. Those numbered 5, 6, and 7 were head tendons, with one of the two tendons across each pair of flats being instrumented. Finally, those numbered 8 through 11 were tendons in the first through fourth bands of circumferential prestressing, respectively. The tendons chosen for instrumentation were arranged so that there was at least one instrumented tendon in each group of tendons simultaneously tensioned in the prestressing sequence.

A typical instrumented tendon assembly is also depicted in Fig. 77. Two diametrically opposed axial strain gages were used on the tendon to eliminate bending effects. The tendon strain gages are denoted by the prefix B. Each tendon that was strain gaged also had a load transducer installed at the nonjacking end anchorage. The transducer gages were denoted by the prefix BT.

To obtain an indication of the locations and levels of cracking on the inner surface of the vessel, a network of 15 crack-detection gages or strips was applied to the inner surface as shown in Fig. 78. In the concrete prototype these strips consisted of either two or three 5-in.-long, single-element strain gages wired in series. For the mortar models a

Fig. 77. Locations of Tendon Strain Gages and Load Transducers.

Fig. 78. Locations of Crack-Detection Gages.

single 5-in.-long gage sufficed to span essentially the same regions of the vessel.

There were meridional strips in planes 1, 2, and 3 that spanned the upper cylindrical portion of the vessel wall, the haunch region, and the head. There were also circumferential strips near the vessel midplane, near the haunch region, and on the head. The prefix S denoted these crack-detection strips. The 1XX, 2XX, and 3XX series numbers denoted meridional strips in planes 1, 2, and 3, respectively, and the OXX series denoted circumferential strips.

In addition to providing a qualitative picture of the gross deformation behavior of the various vessel regions, the strips gave an indication of the presence of major cracking in each region. A major crack under one of the strips usually was indicated either by a complete loss of the circuit or by an off-scale output from the strip. Typical results from these crack-detection strips are presented in Chapter 8.

The arrangement of the dial gages used for obtaining deflection profiles on the concrete prototype is shown in Fig. 79. Gages D-0 through D-9 were located in plane 2. The remaining gages were located directly opposite plane 2.

The instrumentation that has been described was for the concrete prototype, which had the full complement of instrumentation. The second mortar model had nearly the full complement of instrumentation, but major portions were omitted for the first mortar model and the prestressed epoxy model. The first mortar model had 33 surface strain gages, 11 strain-gaged tendons, 11 load transducers, 15 crack-detection strips, and 1 pressure cell, plus the photoelastic coating. There were thus 71 electrical instrumentation points plus the photoelastic coating. The prestressed epoxy model had 61 three-gage rosettes, or a total of 183 separate surface gages, plus 11 strain-gaged tendons for a total of 194 electrical instrumentation points.

For reference the instrumentation used for the concrete prototype, the mortar models, and the prestressed epoxy model is summarized in Table 6.

Table 6. Instrumentation Used for Concrete Prototype,
Mortar Models, and Prestressed Epoxy Model

Refer to Figs. 75 through 79

Number	Concrete Prototype	First Mortar Model	Second Mortar Model	Prestressed Epoxy Model
Surface Strain Gages ^{a,b}				
C101-	1,3		1,3	
C102-	1,3		1,3	
C103-	1,3		1,3	
C104-	1,3		1,3	
C105-	1,3		1,3	
C106-	1,3		1,3	
C107-	1,3		1,3	
C108-	1,3		1,3	
C109-	1,2,3		1,2,3	X
C110-	1,3		1,3	
C111-	1,2,3		1,2,3	X
C112-	1,3		1,3	
C113-	1,2,3		1,2,3	X
C114-	1,2,3		1,2,3	X
C115-	1,2,3		1,2,3	X
C116-	1,3		1,3	X
C117-	1,2,3		1,2,3	
C118-	1,3		1,3	
C119-	1,2,3		1,2,3	X
C120-	1,3		1,3	
C151-	1,3		1,3	
C152-	1,3		1,3	
C153-	1,3		1,3	
C154-	3		3	
C155-	3		3	
C156-	3		3	
C157-	3		3	
C158-	3		3	
C159-	3		3	
C160-	3		3	
C161-	1,3		1,3	
C162-	1,2,3		1,2,3	
C163-	1,3		1,3	

^aIn the case of the surface strain gages on the concrete prototype and on the mortar models, 1, 2, and 3 refer to a circumferential gage, a 45° gage, and a meridional gage, respectively.

^bThe surface strain gages on the prestressed epoxy model were all three-gage rosettes.

Table 6 (continued)

Number	Concrete Prototype	First Mortar Model	Second Mortar Model	Prestressed Epoxy Model
Surface Strain Gages ^{a,b} (continued)				
C164-	1,2,3		1,2,3	
C165-	1,3		1,3	X
C166-	1,2,3		1,2,3	
C167-	1,3		1,3	X
C168-	1,2,3		1,2,3	
C169-	1,3		1,3	
C170-	1,2,3		1,2,3	X
C171-	1,3		1,3	
C201-	1,3		1,3	X
C202-	1,3		1,3	
C203-	1,3		1,3	X
C204-	1,3		1,3	X
C205-	1,3		1,3	X
C206-	1,3		1,3	X
C207-	1,3		1,3	X
C208-	1,3		1,3	X
C209-	1,2,3	1,2,3	1,2,3	X
C210-	1,3		1,3	X
C211-	1,2,3	1,2,3	1,2,3	X
C212-	1,3		1,3	X
C213-	1,2,3	1,2,3	1,2,3	X
C214-	1,2,3	1,2,3	1,2,3	X
C215-	1,2,3	1,2,3	1,2,3	X
C216-	1,2,3	1,2,3	1,2,3	X
C217-	1,2,3		1,2,3	X
C218-	1,2,3		1,2,3	X
C219-	1,2,3	1,2,3	1,2,3	X
C220-	1,3		1,3	X
C251-	1,3		1,3	X
C252-	1,3		1,3	
C253-	1,3		1,3	X
C254-	1,3		1,3	X
C255-	1,3		1,3	X
C256-	1,3		1,3	X
C257-	1,3		1,3	X
C258-	1,3		1,3	X
C259-	1,3		1,3	X
C260-	1,2,3		1,2,3	X
C261-	1,2,3		1,2,3	X
C262-	1,2,3		1,2,3	X
C263-	1,2,3			
C264-	1,2,3		1,2,3	X

Table 6 (continued)

Number	Concrete Prototype	First Mortar Model	Second Mortar Model	Prestressed Epoxy Model
Surface Strain Gages ^{a,b} (continued)				
C265-	1,2,3		1,2,3	X
C266-	1,3		1,3	
C267-	1,3		1,3	X
C268-	1,2,3		1,2,3	X
C269-	1,2,3		1,2,3	X
C270-	1,2,3		1,2,3	X
C271-	1,2,3	1,2,3	1,2,3	
C272-	1,2,3	1,2,3	1,2,3	X
C273-	1,2,3		1,2,3	X
C274-	1,2,3		1,2,3	X
C275-	1,2,3	1,2,3	1,2,3	X
C276-	1,3	1,2,3	1,3	X
C301-	1,2,3		1,2,3	X
C302-	1,3		1,3	
C303-	1,2,3		1,2,3	X
C304-	1,3		1,3	
C305-	1,2,3		1,2,3	X
C306-	1,2,3		1,2,3	X
C307-	1,2,3		1,2,3	X
C308-	1,3		1,3	X
C309-	1,2,3		1,2,3	
C310-	1,3		1,3	
C311-	1,2,3		1,2,3	X
C351-	3		3	
C352-	3		3	
C353-	3		3	
C354-	3		3	
C355-	3		3	
C356-	1,3		1,3	
C357-	1,2,3		1,2,3	
C358-	1,3		1,3	X
C359-	1,2,3		1,2,3	
C360-	1,3		1,3	X
C361-	1,2,3		1,2,3	
C362-	1,2,3		1,2,3	X
Embedded Strain Gages ^c				
E1-	1,2,3,4			
E2-	1,2,3,4		1,2,3,4	

^cIn the case of embedded strain gages, 1, 2, 3, and 4 refer to a radial gage, a 45° gage in the meridional plane, a meridional gage, and a circumferential gage, respectively.

Table 6 (continued)

Number	Concrete Prototype	First Mortar Model	Second Mortar Model	Prestressed Epoxy Model
Embedded Strain Gages ^c (continued)				
E3-	1,2,3,4		1,2,3,4	
E4-	1,2,3,4		1,2,3,4	
E5-	1,2,3,4		1,2,3,4	
Reinforcing-Bar Strain Gages ^d				
R101-	3		3	
R102-	1		1	
R103-	3			
R104-	1			
R105-	3		3	
R106-	1		1	
R107-	3		3	
R108-	1		1	
R109-	3		3	
R110-	1		1	
R111-	3			
R112-	1			
R113-	3		3	
R114-	1		1	
R115-	3			
R116-	1		1	
R117-	3		3	
R118-	1		1	
R119-	3			
R120-	1			
R121-	3		3	
R122-	1		1	
R123-	3		3	
R201-	3		3	
R202-	1		1	
R203-	3			
R204-	1			
R205-	3		3	
R206-	1		1	
R207-	3		3	
R208-	1		1	
R209-	3		3	
R210-	1		1	

^dFor the reinforcing-bar gages, 1 refers to a gage on a circumferential bar and 3 to a gage on a meridional bar.

Table 6 (continued)

Number	Concrete Prototype	First Mortar Model	Second Mortar Model	Prestressed Epoxy Model
Reinforcing-Bar Strain Gages ^d (continued)				
R211-	3			
R212-	1			
R213-	3		3	
R214-	1		1	
R215-	3			
R216-	1		1	
R217-	3		3	
R218-	1		1	
R219-	3			
R220-	1			
R221-	3		3	
R222-	1		1	
R223-	3		3	
Prestressing-Tendon Strain Gages				
B1	X	X	X	X
B2	X	X	X	X
B3	X	X	X	X
B4	X	X	X	X
B5	X	X	X	X
B6	X	X	X	X
B7	X	X	X	X
B8	X	X	X	X
B9	X	X	X	X
B10	X	X	X	X
B11	X	X	X	X
Prestressing-Tendon Load Transducers				
BT1	X	X	X	
BT2	X	X	X	
BT3	X	X	X	
BT4	X	X	X	
BT5	X	X	X	
BT6	X	X	X	
BT7	X	X	X	
BT8	X	X	X	
BT9	X	X	X	
BT10	X	X	X	
BT11	X	X	X	

Table 6 (continued)

Number	Concrete Prototype	First Mortar Model	Second Mortar Model	Prestressed Epoxy Model
Crack-Detection Gages ^a				
S001	1	1	1	
S002	1	1	1	
S003	1	1	1	
S004	1	1	1	
S005	1	1	1	
S006	1	1	1	
S007	1	1	1	
S101	3	3	3	
S102	3	3	3	
S103	3	3	3	
S201	3	3	3	
S202	3	3	3	
S301	3	3	3	
S302	3	3	3	
S303	3	3	3	
Deflection Gages				
D0	X			
D1	X			
D2	X			
D3	X			
D4	X			
D5	X			
D6	X			
D7	X			
D8	X			
D9	X			
D10	X			
D11	X			
D12	X			
D13	X			
D14	X			
D15	X			
D16	X			
D17	X			
D18	X			
Photoelastic Coating				
	X	X	X	X

Fig. 79. Locations of Deflection Gages

6.2 Description of Instrumentation for Concrete Prototype and Mortar Models

6.2.1 Surface Strain Gages

A photograph of the strain gages on the top head of the concrete vessel is shown in Fig. 80, and Fig. 81 shows the gages on the inner surface of

Fig. 80. Strain Gages on Outer Surface on Top Head of Concrete Prototype.

Fig. 81. Strain Gages on Inner Surfaces of Concrete Prototype.

the concrete vessel. In both cases the gages are located primarily in the three meridional planes (1, 2, and 3) described above. The two- and three-gage rosettes consisted of individual gages placed one on top of the other.

For the concrete prototype the individual gages were Micro-Measurements type EA-06-19CDK-120, which is an epoxy-encapsulated foil gage with a resistance of 120 ohms and an active gage length of 1.9 in. For the mortar models, the individual gages were Micro-Measurements type SA-06-750DT-120, which is an encapsulated foil gage with a resistance of 120 ohms and an active gage length of 0.75 in. The gage length for the mortar

models was thus related to the gage length for the prototype vessel by approximately the model scale factor ($1.9/0.75 = 2.53$ versus the model scale factor of 2.75). Consequently from the standpoint of gage length the gage readings from models and prototype should be comparable.

Surface preparation and gage application procedures were essentially the same for models and prototype. Surfaces where strain gages were to be applied were first covered and sealed with a mixture of epoxy and fine sand or grit that had passed through a standard No. 100 sieve. The epoxy used was Epon 815 with CIBA 910 hardener. After the epoxy mortar had set the surface was sanded and cleaned with acetone, and the gages were applied at premarked locations by using an epoxy that cured at room temperature and a minimum of 3-psi clamping pressure. Following the initial set of the epoxy adhesive, the gage installations were postcured with heat lamps for approximately 2 hr at a temperature between 150 and 175°F.

Uninsulated 4-mil-diam wire was used to connect the gages to terminal tabs (see Fig. 80). The terminal tabs were connected in turn to terminal pins with larger wire. On the inner surfaces of the vessels the Fiberglass-reinforced epoxy liner was applied at this point. Only the terminal pins protruded through the liner, and lead wires were connected later to these pins. On the outer surfaces the completed gage assemblies were coated with an additional layer of epoxy for protection. In addition the gages on the outer surfaces of the mortar models were coated with a layer of Bean Gagekote No. 1.

Bridge completion and dummy compensation were provided by similar gages mounted on blocks of the same material that were cast at the same time as the vessel being tested. One dummy gage was used for approximately 23 to 24 active gages.

The bulk of the surface strain-gage data was recorded by the Beckman-Dextir data-acquisition system described later. Data from a few gages were recorded on portable digital strain-indicating equipment, which will also be described.

6.2.2 Embedded Strain Gages

The embedded strain-gage rosettes that were cast into the wall of the concrete prototype are shown in Fig. 82. The rosettes were supported

Fig. 82. Embedded Strain-Gage Rosettes Attached to Vertical Prestressing Tendon Duct.

by one of the metal sheaths for the vertical prestressing tendons. The inset in Fig. 82 gives a closer view of one of the rosette assemblies.

Each leg of a rosette assembly was basically an epoxy dowel in which four ordinary foil strain gages were embedded. For the concrete prototype the foil gages used as the basic sensing element were Budd type C6-141-B, which is a 120-ohm gage with an active gage length of $1/4$ in. The four foil gages were mounted on an epoxy strip $1/4$ in. wide by approximately $1/32$ in. thick, with two gages mounted end-to-end on each side of the strip. An epoxy adhesive that cured at room temperature was used. After it had been wired the epoxy strip assembly was centered in the $1/4$ -in.-diam opening of a two-part Teflon mold and subsequently encapsulated in the epoxy dowel. The dowels resembled machine screw studs $1/4$ in. in diameter by $1\ 1/2$ in. long. The overall gage length of $1\ 1/2$ in. was based on the generally accepted rule that for valid average strain readings in concrete the gage length should be at least three times the maximum aggregate dimension.⁴² During the process of casting a dowel a brass screw was embedded in one end to use for assembling the dowel into the rosette configurations.

The two gages on each side of the epoxy strips were wired in series, and then the two pairs of gages in each dowel were wired together in parallel. The resulting circuit thus had a total resistance of 120 ohms. Compensation was provided by similar dowels embedded in concrete blocks that were cast along with the vessel. The resulting half-bridge circuits were then wired to the Beckman-Dextir data-acquisition system.

The embedded gages for the second mortar model were simply smaller versions of those described above. They were made the same way, but the overall dimensions of the epoxy dowel were $1/4$ in. in diameter by 1 in. long. The foil gages used were Budd type C6-121, which has an active gage length of $1/8$ in.

Although the embedded gages provided desirable and useful information, part of the reason for including them in the instrumentation for the models was to investigate their reliability and accuracy. Commercial strain gages for embedment in very small-scale models are conspicuously lacking, particularly for long-term measurements. Gages for these very small models

should ideally have maximum dimensions less than about 2 in., but the two most commonly used types - the unbonded wire resistance gage and the acoustic, or vibrating-wire, gage - are significantly larger than this.

The most obvious and economical solution to the problem is to encapsulate a regular bonded resistance strain gage of adequate length in some waterproof plastic material, as was done in constructing the embedded gages used in the model study program. This approach has been used by several investigators; however, problems have often arisen, and the results and conclusions have been rather different. Plastic-encapsulated resistance gages and the experience gained in their use are discussed in detail in Ref. 42.

Briefly, problems have arisen from three sources. First, most plastics are not completely effective moisture barriers, and over long periods of time some moisture may penetrate to the gage and cause drift. Furthermore, many plastics absorb small amounts of water that cause volume changes which could produce apparent strains. Second, the mismatch of thermal expansion between plastic and concrete and the gage self-heating effect often produce problems. These effects were not, however, a factor in the present investigation because elevated temperatures were not involved and the rapid scanning rate of the Beckman-Dextir data-logging system precluded any significant self-heating of the gages. Finally, creep and relaxation of the plastic relative to the surrounding concrete could result in apparent strain indications.

Plastic-encapsulated resistance gages are therefore generally considered to have good short-term stability but probably poor long-term stability. Thus the embedded gages in the concrete prototype and the second mortar model were expected to give good strain readings during the short-term prestressing and pressure tests, but their strain indications during the time-dependent test period were expected to be of dubious value. However, as is illustrated by the results presented in Chapter 8, the embedded gages appeared to perform reasonably well throughout the test sequence, including the time-dependent period. The time-dependent period in these tests was very short, so the fact that the gages performed well does not imply that they would be satisfactory over a relatively long period.

6.2.3 Reinforcing-Bar Strain Gages

The techniques used for preparing the reinforcing bars for strain-gage application and for sealing the gage assemblies followed, in general, the recommendations of the Portland Cement Association,⁴³ although specific details were altered. For the concrete prototype the gages used were Budd type C6-141-B, which is a foil gage with an active gage length of 1/4 in. and a width of 1/8 in. The reinforcing bars were ground and filed smooth in the regions where the gages were to be located, cleaned with acetone, and dressed with a metal conditioner and neutralizer. The gages were then applied by using an epoxy adhesive that cured at room temperature. Terminal tabs were applied at the same time, and the gages were connected to the tabs with uninsulated 4-mil-diam wire.

After the lead cables were attached to the tabs, the gage assemblies were waterproofed with Bean Gagekote No. 5, which is a flexible epoxy that cures at room temperature. The assemblies were then covered with a thick layer of Mobile Wax 2300 for additional sealing and for physical protection from the wet concrete and the vibration equipment during casting.

The reinforcing-bar cage for the prototype is shown in Fig. 83 with the gages in place, sealed, and wired. Figure 84 shows the reinforcing-bar cage for the second mortar model, again with gages in place, sealed, and wired. The embedded strain-gage rosettes for the second mortar model are also shown in Fig. 84. The gages used on the reinforcing bar in the mortar model were Budd type C6-111, which is a foil gage with an active gage length of 0.063 in. and a width of 0.067 in. The reinforcing-bar preparation, the gage application procedures, and the sealing techniques were the same for the mortar model as for the concrete prototype.

In both cases the gages were self-compensating when mounted on carbon steel; however, one dummy gage for approximately ten active gages was used to complete the half-bridge. For both model and prototype the reinforcing-bar gage readings were recorded on portable digital strain-indicating equipment.

Fig. 83. Strain-Gaged Reinforcing-Bar Cage for Concrete Prototype.

Fig. 84. Strain-Gaged Reinforcing-Bar Cage and Embedded Gages for Second Mortar Model.

6.2.4 Prestressing-Tendon Strain Gages

Strain gages on the prestressing-tendon bars were mounted in pairs diametrically opposed. The two gages on each bar were wired in opposite legs of a full bridge, with the remaining legs being similar dummy gages on identical unstressed bars. The full bridge was connected to the Beckman-Dextir data-acquisition system in the case of the prototype and the second mortar model and to portable digital strain-indicating equipment in the case of the first mortar model.

For the concrete prototype, the individual gages were Micro-Measurements type EA-06-19CDK-120, which is the same gage as was used on the concrete surfaces. For the mortar models, the individual gages were Micro-Measurements type EA-06-750DT-120, which is an encapsulated gage similar to the type used on the surfaces of the mortar models. The gages were applied to the bars by using Bean BR-610 adhesive and a 250°F temperature cure. This adhesive is highly recommended for time-dependent tests because of its low creep characteristics. The gage assemblies were protected with Bean Gagekote No. 1, which is a general-purpose coating for laboratory use where water immersion is not expected.

6.2.5 Prestressing-Tendon Load Transducers

The positions of the load transducers in the tendon assemblies were shown in Fig. 77. The transducers were simply thick-walled steel cylinders with strain gages attached for measuring strain in the cylinders. Each transducer was precalibrated to relate the strain reading to force.

In the case of the concrete prototype the transducers were 2 in. long, with an outside diameter of 3 in. and an inside diameter of 1 7/16 in. For the mortar models the transducers were 1 5/16 in. long, 1 1/4 in. in outside diameter, and 3/4 in. in inside diameter.

The gages used on both sets of transducers were Budd type C6-141-B, which as previously described is a foil gage with a 1/4-in. active gage length and a width of 1/8 in. The gages were applied with Bean BR-610 adhesive cured at a temperature of 250°F. They were connected to terminal tabs with uninsulated 4-mil-diam wire, and the assemblies were coated with Bean Gagekote No. 1.

Four gages were installed on each transducer. Two diametrically opposed gages were mounted axially (parallel to the direction of load application), and two diametrically opposed gages were mounted circumferentially. The full bridge mounted in this manner was self-compensating. The Beckman-Dextir data-acquisition system was used for readout in the case of the prototype and second mortar model, while portable digital strain-indicating equipment was used for the first mortar model.

6.2.6 Crack-Detection Strips

The individual gages used for the crack-detection strips in both the prototype and the mortar models were 5-in.-long single-element foil strain gages (Budd type C6-1X1-L5). The gages had a basic resistance of 120 ohms, but in the prototype either two or three were wired in series to span the same vessel regions as spanned by single gages on the models. The gages wired in series were compensated by gages of the same resistance mounted on concrete blocks.

The gages were applied with an epoxy adhesive that cured at room temperature. No special surface preparation was used, and no clamping pressure was used because of the irregular contours covered by the gages. The gages were wired to terminal tabs and then to terminal pins in the same manner as the other inner surface gages.

The crack-detection strips were used only during the pressure tests. They were connected to continuously recording strip-chart recorders. Preliminary tests showed that the strips were adequate as strain gages for concrete applications. Until nonlinearities due to cracking occurred the indicated strains were an adequate measure of the actual concrete strains integrated over the length of the strips.

6.2.7 Deflection Gages

The dial indicators used for measuring the deflection profile of the outer surface of the prototype vessel were mounted on a steel frame that was attached to the floor and encompassed the model in a vertical plane. The frame was isolated from any movement of the vessel itself.

The indicators had a resolution of 0.0001 in., and deflections were read and recorded at each pressure level. For safety the faces of the indicators were scanned by remotely controlled television cameras at the higher pressures, and the readings were recorded.

6.2.8 Photoelastic Coatings

The general method of applying birefringent coatings for the determination of surface strain distributions has been widely used for metal structures, particularly since epoxies with good strain-optical sensitivities

have become available. These better materials, however, are still not sufficiently sensitive to determine elastic strain distributions in concrete structures, except at locations where very high compressive strains exist. Nevertheless, the method has been found useful for concrete structures because it can give a qualitative indication of the formation and growth of cracks before they become visible to the naked eye.⁴⁴

The basic mechanism of failure of mortars and concretes under load is the initiation and propagation of small cracks that extend and interconnect to form major structural cracks.⁴⁵ As these microscopic cracks, or "microcracks," form, strain concentrations are induced that are sufficient to produce fringe patterns in photoelastic coatings applied to the surface. As the cracks extend and interconnect the concentration of fringes increases and clearly indicates the path of major cracking.

To observe the initiation and propagation of cracks on the outer surfaces of the concrete prototype and the two mortar models, one 60° sector of the outer surface of each was covered with a photoelastic coating. The coatings were applied after the structures were prestressed.

The coating used was birefringent plastic sheet, type PS-1A, supplied by Photoelastic, Inc. The sheets were 0.130 in. thick and were cemented to the concrete surfaces with an epoxy adhesive which, for reflective purposes, had an aluminum powder suspended in the mixture. Actually, the aluminum powder was not necessary because a reflective backing was applied to the sheet at the factory. In preliminary tests on small concrete beams loaded in flexure this coating exhibited extremely sharp fringe patterns produced by small cracks at the surface of the concrete under the coating.

To observe the fringe patterns a simple reflective polariscope was used with a white-light source. Quarter-wave plates were used to eliminate isoclinic fringes, which were of no interest. The reflective patterns were photographed through one of the Polaroid filters. A Nikon 35-mm camera and Ektachrome color film were used.

The development of crack patterns in the cylindrical wall of the second mortar model, as indicated by the photoelastic coating, is illustrated in the next chapter by a series of photographs made of the reflective patterns at progressively increasing pressure levels.

6.3 Description of Instrumentation for Prestressed Epoxy Model

6.3.1 Surface Strain Gages

The strain gages on the outer surfaces of the prestressed epoxy model are shown in Figs. 16 and 17. The gages used were three-gage rosettes, Micro-Measurements type EA-06-062RB-120. This is a small 45° rectangular rosette foil gage with an individual active gage length of 0.062 in. and a resistance of 120 ohms. As may be seen in Fig. 17 the gages were oriented in a symmetrical manner with the 45° gage always in the meridional direction.

Very little surface preparation was required for the epoxy models. The 32- μ satin finish was ideal for mounting strain gages. Ethyl alcohol was used to clean any oil contaminants remaining from the machining operation, and the gages were applied with an epoxy that cured at room temperature. Uninsulated 4-mil-diam wire was used to connect the gages to terminal tabs, and leads were connected from the tabs to the acquisition equipment by using twisted pairs of vinyl-coated wire.

Compensation for the gages was provided by dummy gages of the same type mounted on epoxy blocks. The Beckman-Dextir data-acquisition system was used to collect the data.

Gage drift due to localized heating caused by the poor heat-dissipating properties of epoxy has often been a source of trouble in epoxy strain-gage models. It was not a problem here because of the relatively fast scanning rate (eight channels per second) of the Beckman-Dextir system. Excitation of each gage was required only for a few milliseconds prior to taking a reading. In addition the excitation voltage was reduced for the epoxy model tests. Consequently effects due to self-heating of the gages were negligible.

6.3.2 Prestressing-Tendon Strain Gages

The strain gages mounted on the small drill-rod bars that served as the prestressing tendons for the epoxy model were Budd type C6-121-A, which is a 120-ohm foil gage with an active gage length of 0.125 in. Like the

gages on the larger prestressing bars of the prototype and mortar models, the gages were mounted in diametrically opposed pairs.

The rods were polished with a 400-grit sandpaper and then cleaned with acetone, metal conditioner, and neutralizer. An epoxy adhesive that cured at room temperature was used to apply the gages. After the initial cure, the gages were postcured at a temperature of approximately 200°F.

No terminal tabs were used because of space restrictions between the bars and the ducts through the model. Instead, magnet wire approximately 6 mils in diameter was attached to the gages and brought outside the model. At this point the leads were connected to 32-gage stranded-copper vinyl-coated phonograph wire.

The Beckman-Dextir data system was used for recording data from the prestressing-tendon gages.

6.4 Description of Instrumentation for Axisymmetric Epoxy Model

As mentioned in Chapter 1 the axisymmetric epoxy model was tested both photoelastically by using the scattered-light technique and with strain gages. The strain-gage locations and the lines along which stresses were determined photoelastically are shown in Fig. 85.

Because the model was axisymmetric, a two-gage rosette was sufficient to completely determine the stresses at any point. The gage used was Micro-Measurements type EA-06-062TT-120, which is a small, compact 90° rosette with an individual active gage length of 0.0625 in. and a resistance of 120 ohms. The surface preparation and the gage installation methods used were the same as those used for the prestressed epoxy model.

6.5 Data Acquisition

Most of the data taken in the model study program was recorded by a Beckman-Dextir automatic data-acquisition system. This system was augmented by portable strain-indicating equipment and by strip-chart recorders. Each of these types of data-acquisition equipment is described briefly in the following paragraphs.

Fig. 85. Cross Section of Axisymmetric Model Showing Photoelastic Data Lines and Strain-Gage Rosette Locations.

6.5.1 Beckman-Dextir Automatic Data-Acquisition System

The Beckman-Dextir system consists of a data-acquisition unit composed of a number of remotely located data-gathering boxes assigned to different experiments, a central data-processing unit, and a central data-recording unit. Each remotely located data-gathering box is capable of accepting 25 analog inputs and 25 user-related digital inputs from decimal switches or other contact-closure devices. The individual boxes are connected to the central processing and recording units by a party-line arrangement. This party line carries the control sequence signals to the data-gathering boxes and also carries the analog voltages and digital constants back to the data-processing unit.

Eleven data-gathering boxes or 275 analog channels were available for the model study program. These boxes were modified to provide bridge excitation and completion for the strain-gage circuits by the use of internal switching and external power supplies.

The low-level analog signals from the data-gathering boxes of the Beckman-Dextir system are transmitted to the central data processor where they are amplified to ± 5 v full scale. These amplified voltages are then digitized and passed on to the data-recording unit where they are recorded on IBM-compatible magnetic tape and if desired printed on paper tape. The user-related digital information from each data-gathering box is also recorded in the same format as the digitized analog information.

The central data processor receives the time of day to the nearest minute in four digits from a time-of-day clock and the day of the year from a day-of-year calendar. These data are recorded along with box number and channel number data.

Each data-gathering box may be scanned automatically at 5-, 15-, 30-, or 60-min intervals or on a continuous basis. Also a push-button switch on each box allows for one scan of the data entered through that box. The scanning rate within each box is eight channels per second. The system accuracy is stated to be $\pm 0.07\%$ of full scale, and the system repeatability, 0.04% of full scale.

All data from all experiments for a 24-hr period were recorded on a single magnetic tape. At the end of the 24-hr period the tape was sent to

an IBM 360/75 computer where the millivolt readings for each experiment were converted to engineering units (strains or tendon forces in the case of the model study program). All data for that 24-hr period were then printed out for each experiment and returned to the experimenter. In addition the data were recorded on a master tape for storage. The master tapes were later used to obtain various computer plots of the data, including strain versus time for every gage during the time-dependent test period and pressure versus strain for every gage during the pressure-testing period.

6.5.2 Portable Strain-Indicating Equipment

Portable digital strain-indicating equipment was used for recording all data from the first mortar model test and to augment the 275 available Beckman-Dextir channels in the case of the second mortar model test and the concrete prototype test. The portable equipment included a W. T. Bean Model BEC-205 digital strain indicator with an associated automatic 30-channel switching and balancing unit and two Budd Model TC-22 digital strain indicators with associated automatic 20-channel switching and balancing units.

Paper-tape printers were used in conjunction with the Bean indicator and one of the Budd indicators. Data from the second Budd indicator were recorded manually. Both the Bean indicator and the Budd indicator have a stated accuracy of $\pm 0.1\%$ on any calibrated span setting.

6.5.3 Strip-Chart Recorders

A Sanborn model 350 eight-channel strip-chart oscillograph and a Brush model BL-350 eight-channel oscillograph were used for recording output from the crack-detection strips during the pressure tests of the prototype and mortar models. The overall accuracy of both these systems, including linearity and readability, is estimated at approximately 4%. In addition to the crack-detection strips, the internal pressure indicated by a strain-gage pressure transducer was also continuously recorded on a strip chart.

7. TEST DESCRIPTIONS AND OBSERVATIONS

7.1 Concrete Prototype and Mortar Models

The concrete prototype and the two mortar models were subjected to essentially the same sequence of testing. All three were prestressed with the same basic sequence of prestressing steps. Each then underwent a period of time-dependent testing during which creep, shrinkage, and relaxation effects were monitored. On the thirty-third day after prestressing, each vessel was subjected to a series of pressure tests at increasing pressures that culminated in failure due to gross uncontrollable leakage. Two days after the pressure tests the tendons were detensioned, and the time-dependent behavior was monitored for an additional three days to observe any initial recovery effects.

The time schedule for testing the concrete prototype and the two mortar models is summarized in Table 7, where each significant step in curing and testing the vessels is shown in terms of the number of days after the vessel was cast. For reference the concrete properties tests at 28 and 90 days (described in Chapter 4) are also shown.

Although the curing times, the concrete specimen testing times, and the time schedules for the actual testing sequences, beginning with prestressing, were identical for the three vessels, the elapsed times between casting and prestressing differed. The concrete prototype was prestressed at an age of 193 days; the first mortar model, at 68 days; and the second mortar model, at 158 days. Thus the first mortar model was considerably younger at the time it was tested than were the concrete prototype and the second mortar model. This difference was evidenced by the relatively large shrinkage exhibited by the first mortar model control specimens during the time-dependent test period and the negligible shrinkage exhibited by the older control specimens of both the second mortar model and the concrete prototype.

Each of the three major tests phases - prestressing, time-dependent testing, and pressure testing - is described in the following sections.

Table 7. Time Schedule for Concrete Prototype and Mortar Model Tests^a

^aThe numbers above bars representing time periods denote the length of the period, while the numbers above individual events denote the number of days since casting.

7.1.1 Prestressing

The three models were each prestressed in 12 major steps. The following tendons were tensioned to one-half their full initial level of $0.7f'_g$ in sequence: (1) axial tendons, (2) head tendons, (3) first band of circumferential tendons, (4) second band of circumferential tendons, (5) third band of circumferential tendons, and (6) fourth band of circumferential tendons; and then to their full initial level of $0.7f'_g$ in the same sequence. Each of these steps was analyzed by using the axisymmetric finite-element method to verify that no objectionable tensile stresses were induced. During the actual prestressing of each vessel a complete set of experimental data was recorded after each step, and these experimental results were compared with the finite-element predictions.

Standard Stressteel tensioning equipment was used for the prestressing operations. Figure 86 shows the concrete prototype during the step in the prestressing sequence in which the first circumferential band of tendons was tensioned. In Fig. 87 the third circumferential band of tendons on the first mortar model is shown being tensioned. In each case six hydraulic rams were used with a single pump so that six bars could be simultaneously tensioned in a single step. Thus two steps were required to tension the 12 axial tendons to each specified level, while the six head tendons or the six tendons in each circumferential band could be tensioned to each specified level in a single step.

The initial prestressing level immediately after completion of the prestressing operation was specified to be that corresponding to a tension in the Stressteel bars of $0.7f'_g$. To achieve this initial level different temporary jacking forces were required in the concrete prototype and in the mortar models. Although Stressteel bars were used in both cases the method of anchoring was different. A nut and thread anchorage was used for the mortar models, while a Howlett Grip Nut anchorage was used for the prototype. For the nut-and-thread anchorage it was found that seating losses could be completely eliminated by tightening the nuts snugly with a small wrench while the temporary jacking force was being applied. Thus a temporary jacking force of $0.7f'_g$ resulted in an initial prestressing force of about the same value.

Fig. 86. Concrete Prototype Vessel During Prestressing Operation.

Fig. 87. First Mortar Model During Prestressing Operation.

The Howlett Grip Nuts of the prototype prestressing system, on the other hand, were tightened only hard-tight. There is an inherent sealing loss in the Grip Nut arrangement that is substantial for short bars in which the overall elongation is initially small, and thus it was necessary to apply temporary jacking forces equal to $0.8f'_s$ in order to obtain an initial tendon force of about $0.7f'_s$ immediately after transfer of the load from the jacks.

The actual initial tendon loads immediately after completion of the prestressing operation were determined both by strain gages on the bars and by load transducers at the anchorages. The average level for each group of tendons of each vessel is given in Table 8 as a percentage of f'_s , the guaranteed minimum ultimate strength of the tendons. These actual values are generally lower than the specified initial level of $0.7f'_s$, but the largest deviation is less than 10%, and at the time the pressure tests were carried out the prestressing levels were still above the final design prestressing level of $0.6f'_s$ in every case.

Table 8. Measured Initial Prestressing Loads

Tendons	Prestressing Load (% f'_s)		
	Concrete Prototype	First Mortar Model	Second Mortar Model
Axial	74.9	67.3	65.8
Head	64.5	63.8	68.2
First circumferential band	65.8	66.9	72.1
Second circumferential band	64.4	64.5	67.9
Third circumferential band	69.3	64.1	68.1
Fourth circumferential band	70.0	66.6	69.3

7.1.2 Time-Dependent Tests

The period of time-dependent testing of each model began immediately after completion of the last step in the prestressing sequence and continued for 38 days. Except for a period of a few hours on the thirty-third day when the pressure tests were being conducted and on the thirty-

fifth day when the tendons were being detensioned, data from the instrumentation connected to the data-logging system were generally recorded automatically every hour. At the end of the test, computer plots of the readout of each gage or transducer versus time were obtained. Data from the instrumentation connected to manual readout equipment were generally recorded only once each day, and plots of the readout versus time were prepared by hand.

In addition, strains on unloaded control specimens were recorded throughout the time-dependent test periods. The control specimens were extra flexural test beams that were cast along with the respective models and subjected to exactly the same environment. Strain gages on the control specimens were identical to those on the surfaces of the respective models. There were three separate control specimens for the concrete prototype, four for the second mortar model, and two for the first mortar model.

7.1.3 Pressure Tests

The planned pressure-testing sequence was the same for each of the three models and consisted of the following pressure cycles:

1. 0 to 300 psi in steps of 100 psi,
2. 0 to 300 psi in a single step,
3. 0 to 600 psi in steps of 100 psi,
4. 0 to 900 psi in steps of 200 psi up to 600 psi and in steps of 100 psi from 600 psi to 900 psi,
5. 0 to the maximum attainable test pressure in steps of 200 psi up to 600 psi and in steps of 100 psi from 600 psi to the maximum pressure.

The choice of this particular sequence was based on the desire to have at least two cycles of loading that would serve as "shakedown" cycles to a pressure somewhat less than the design pressure. The two cycles to 300 psi served that purpose. The third cycle to 600 psi corresponded to what might be an initial proof test.* The responses to these first three

*The 600-psi pressure level was 1.2 times the design pressure. Actual proof-test pressures in vessels built to date have generally been 1.15 times the design pressure.

cycles remained essentially in the elastic region and demonstrated the repeatability of the responses to elastic loadings. The fourth cycle to 900 psi entered the region of inelastic behavior, and the fifth to the maximum attainable pressure demonstrated behavior upon reloading a vessel previously subjected to an overpressure into the region of inelastic response.

The actual pressure test history of the three models is shown by the bar charts in Fig. 88. The concrete prototype and the second mortar model, both of which were tested hydraulically, followed the planned pressure testing sequence exactly as described above. The maximum pressure reached in the prototype was 1130 psi, and in the second mortar model the maximum pressure was 1390 psi. In both cases these maximum pressures were limited by gross leakage and the capacities of the hydraulic pumps used.

Fig. 88. Pressure Test Histories of Concrete Prototype, First Mortar Model, and Second Mortar Model.

The planned sequence was interrupted in the first mortar model at 850 psi by a premature leak in the epoxy liner near the vessel base. The leak was repaired by pouring a 2-in. layer of epoxy into the bottom of the vessel through one of the top-head penetrations. Major structural cracking and leakage then occurred at 965 psi upon reloading pneumatically. A final pressurization to 1040 psi was carried out hydraulically.

The pressure testing and the observed vessel response are described in detail in the following sections for each of the three models.

7.1.3.1 Concrete Prototype. The hydraulic pressurization system for the prototype included a fixed-volume pump capable of delivering oil at 60 gpm and pressures up to 2500 psi. The pump was mounted on a 150-gal reservoir with an adjustable relief valve. The pressurizing oil was a 20-weight petroleum-base hydraulic fluid.

The piping for the pressurization system consisted of a loop for circulating flow and a branch line that dead-ended into the vessel. The circulated flow was throttled to create a back pressure equivalent to the pressure desired in the vessel. This was found to be an accurate and easy method for pressurization.

No sign of leakage or distress in the vessel was observed until a pressure of approximately 1000 psi was reached in the final pressure cycle. The subsequent behavior can best be described by referring to Fig. 89, which shows selected frames taken from a movie made of the vessel during the test. Frame *a* shows the vessel just after the first sign of leakage, which occurred at approximately 1000 psi. The initial leakage came from the vicinity of the anchor plates for the head-prestressing tendons. Oil may be seen dripping down the sides of the vessel from these leaks. As was determined later, these initial leaks were produced by cracks extending from the haunch region where the head joined the cylindrical portion of the vessel.

Frame *b* shows the vessel at the maximum pressure of 1130 psi. Leakage is still confined to the leaks in the head tendon anchor region. Immediately after this frame was taken, major longitudinal cracks appeared in the cylindrical wall and released bursts of oil from the vessel. One such burst may be seen at the left of the vessel in frame *c*, which shows

(a) ~1000 psi

(b) ~1130 psi

(c) ~900 psi

(d) ~820 psi

Fig. 89. Progressive Cracking and Leakage of Concrete Prototype During Pressure Test to Failure. Photographs taken (a) at first sign of leakage; (b) at maximum pressure just prior to major structural cracking; (c) just after major structural cracking; and (d) 1 min later, just before test was terminated.

the prototype immediately after the formation of these major cracks. The pressure had dropped to about 900 psi in this frame. Subsequently the pressure in the vessel was limited by the 60-gpm capacity of the pump. The test was continued until the 150-gal oil reservoir was emptied. The final frame in Fig. 89 shows the vessel 1 min after frame c was taken, at which time the pressure had dropped to 820 psi. Several sprays of oil may be seen coming from the cylindrical portion of the vessel.

The structural response of the prototype vessel is illustrated by the plots in Figs. 90 and 91, which show pressure versus deflection data for the middle of the top head and for a point near the midpoint of the vessel wall. Data for the two pressure cycles to 300 psi, the cycles to 600 and 900 psi, and the pressurization to the final maximum pressure are shown. These curves are indicative of the behavior not only of the prototype but, as will be seen in the next chapter, of the two mortar models as well. At about 600 psi the overall behavior became nonlinear; this indicated the initiation of cracking.

Fig. 90. Pressure Versus Measured Vertical Deflection at Center of Head of Concrete Prototype Vessel.

Fig. 91. Pressure Versus Measured Radial Deflection Near Midpoint of Cylindrical Wall of Concrete Prototype Vessel.

7.1.3.2 First Mortar Model. The pneumatic pressurization system for the first mortar model utilized bottled nitrogen gas at 1700 psi. Six bottles were connected to a direct line to the vessel through a manifold and central regulator-controller arrangement.

Following the initial leak at 850 psi, which was repaired, new nitrogen bottles were installed in the pressurization system and testing was resumed. Upon repressurization, some leakage occurred again at 850 psi, but major structural cracking and gross leakage occurred fairly abruptly at 965 psi. The failure was evidenced only by a drop in pressure and the loud hissing of escaping gas; it was not at all catastrophic. When the pressure had dropped to 750 psi the inlet gas line was closed, and after about 10 min the pressure had dropped through leakage to approximately 100 psi.

The pressure steps are depicted in Fig. 92, which is a plot of the internal pressure of the model as a function of time. The plateaus at 200, 400, and 600 psi represent pauses to record data. The effect of the leak at 850 psi may be seen, as well as the abrupt gross leakage at 965 psi. The relatively slow leakage rate following the closing off of the

Fig. 92. Pressure-Time History for Tests to Failure of First Mortar Model.

inlet gas is significant. It demonstrates the "self-healing" capabilities of a PCRV structure following major structural cracking.

One of the observations made following the pneumatic test is worth describing because of its relevance to the leakage of gas through the walls of a PCRV with a liner leak. The floodlights used to illuminate the model for the movies that were made warmed the shellac coating on the outer surface of the model. The coating was thus relatively soft and pliable during the pneumatic test. Following the test many small blisters were observed in the shellac coating on the cylindrical portion of the model in the general vicinity of the structural cracks. These blisters indicated that once the liner began to leak the gas permeated the concrete of the model and formed the blisters in the shellac on the outer surface. This helps explain why, in this particular model, gas suddenly released into the cracks in the vessel walls had no apparent effect on the behavior of the structure. The gas added no significant additional loading on the tendons.

After the pneumatic test was completed it was decided to pressurize the model hydraulically in an attempt to subject it to a higher pressure.

The model was therefore filled with oil and repressurized, with bottled gas at 1500 psi again being used as the pressure source. In a period of about 6 sec the pressure reached a peak of 1040 psi. During the few seconds that the pressure was maintained, first oil and then an emulsified mixture of oil and gas sprayed from visible structural cracks running vertically in the cylindrical portion of the vessel. These cracks had been visible as fine hairline cracks after the pneumatic test. They were considerably more obvious after the hydraulic test, and it was found that in some places small chips of concrete had been broken out of the cracks by the escaping oil.

Three selected frames from movies taken during the tests of the first mortar model are shown in Fig. 93. The first two frames made during the pneumatic test show the model at the maximum pressure of 965 psi and 1 min later at 750 psi. These two frames show simply that the gas escaping at a relatively high pressure produced no visible distress in the vessel. The third frame shows the vessel at the maximum hydraulic pressure of 1040 psi. Several streams of oil may be seen coming from the vertical cracks in the wall of the vessel, and an emulsion was beginning to escape from the upper portion of the vessel where the inlet gas entered.

7.1.3.3 Second Mortar Model. The hydraulic pressurization system for the second mortar model was essentially the same as for the prototype, except for the pump used. Since the maximum pressure reached in these tests was limited by the flow capacity of the pump, it was felt that there would be some logic to using a pump for the second mortar model tests with a pumping capacity related to that of the prototype vessel pump by approximately the volumetric scale factor of the model ($2.75^3 = 20.8$).

The pump used in this case was a variable-volume pump with a volume control from 0 to 3 gpm. The volume was fixed for this test at the maximum flow rate of 3 gpm, which approximately satisfied the specified relationship to the flow rate of the prototype vessel pump ($60/3 = 20$).

No sign of leakage or distress was observed until a pressure of about 1100 psi was reached in the final pressure cycle. By 1200 psi, gross leakage was occurring at vertical cracks in the wall. The behavior is illustrated in Fig. 94, which shows three selected frames from the movie

PHOTO 98332

(a) ~965 psi

(b) ~750 psi

(c) ~1045 psi

Fig. 93. Progressive Cracking and Leakage of First Mortar Model During Pressure Tests to Failure. Photographs taken (a) at maximum pneumatic pressure; (b) 1 min later when inlet gas line was closed; and (c) at maximum pressure in subsequent hydraulic test.

(a) ~1180 psi

(b) ~1390 psi

(c) ~1200 psi

Fig. 94. Progressive Cracking and Leakage of Second Mortar Model During Pressure Test to Failure. Photographs taken (a) soon after first sign of leakage; (b) at maximum pressure; and (c) 1 min later, just before test was terminated.

made during the test. Frame *a* shows the model at about 1180 psi, which was after the first signs of leakage had occurred. As in the case of the prototype vessel, the initial leakage came from the vicinity of the anchor plates for the head prestressing tendons. The first signs of oil dripping from the anchor plates are visible in the first frame.

Frame *b* shows the vessel at the maximum pressure of 1390 psi. In contrast to the concrete prototype in which the maximum pressure was reached before any gross leakage from vertical cracks occurred, the maximum pressure in the second mortar model occurred after gross leakage had begun. Several streams of oil from vertical cracks may be seen in frame *b*. Most of the same streams are still visible in frame *c*, which shows the model 1 min later at a pressure of about 1200 psi. The pressure level was dictated by the 3-gpm capacity of the pump. It is significant that the leakage patterns here are essentially the same as those illustrated for the concrete prototype in Fig. 89.

The photographs in Figs. 95, 96, and 97 further illustrate how cracking progressed with increasing pressure in the second mortar model. These are photographs of the fringe patterns in the photoelastic coating that was applied to one of the hexagonal faces of the model. The head tendon anchor plate at the top of each picture and the circumferential tendon anchor blocks can be used as an aid to becoming properly oriented. There is some distortion in the pictures, because each is actually a composite of two or three separate photographs.

The first photograph, taken through a Polaroid filter, shows the reflective patterns in the coating at 0 psi. There are no fringes, since the coating was applied after the model was prestressed. The second photograph, taken at 700 psi, shows no distinct fringes of the type that would be caused by microcracks. The widespread darkening is due to the general increase in positive strain level with increasing internal pressure.

The first photograph in Fig. 96 shows a fringe pattern in the lower center that was just beginning to appear at 800 psi. This indicates the presence of a vertical microcrack in the cylindrical wall near the vessel base. In the second photograph of Fig. 96, which was taken at 900 psi, the initial crack had grown part way up the vessel, and a second vertical crack was beginning to appear at the edge of the anchor blocks on the

0 psi

700 psi

Fig. 95. Photostress Patterns on Second Mortar Model at 0 and 700 psi.

right. Finally, in the first photograph in Fig. 97, taken at 1000 psi, the second crack had grown almost the full height of the vessel wall, and in the second photograph, taken at 1100 psi, the second vertical crack had grown a little further and had widened significantly, as indicated by the increased number of fringes. The development of vertical cracking indicated by these photographs is consistent with the observation that

800 psi

900 psi

Fig. 95. Photostress Patterns on Second Mortar Model at 800 and 900 psi.

gross oil leakage from the vertical cracks occurred at 1130 psi in the concrete prototype and at 1200 psi in the second mortar model.

7.1.4 Observed Crack Patterns

The observed cracking patterns for the concrete prototype and for the two mortar models are depicted schematically in Fig. 98. The vessel deformation, shown grossly exaggerated in the drawing, was scaled from the dial-gage measurements made at 500 psi on the prototype.

1000 psi

1100 psi

Fig. 97. Photostress Patterns on Second Mortar Model at 1000 and 1100 psi.

Three families of cracks occurred in each model. First, there were circumferential cracks at the haunch. These might be referred to as secondary cracks in that a small extension of the crack relieved, at least in part, the stresses causing the extension. Thus these cracks propagated slowly with increasing pressure. Generally they did not extend completely across the wall, but they did often reach the anchor plates for the head tendons and cause the early observed leakage in that region.

Fig. 98. Schematic Drawing of Cracking and Deformation Patterns.

The second family of cracks consisted of circumferential cracks in the cylinder wall that extended radially inward at vessel midheight. Like the cracks in the haunch region, these were secondary cracks, and they extended only partly through the vessel wall.

The final family of cracks consisted of the vertical cracks distributed around the wall. These were primary structural cracks, in contrast to the secondary cracks described above. A crack extension in the radial direction did not appreciably reduce the hoop stress that caused it. Thus these cracks formed and propagated fairly rapidly, at least in the middle portion of the vessel away from head and base effects. No cracks were observed in the head region of any of the models.

From the descriptions of vessel behavior and leakage given in the previous sections and from postfailure examinations of the cracked vessels, the probable sequence of cracking and the corresponding pressure

levels can be postulated. The haunch cracking apparently began at a pressure of about 600 psi, since the vessel response became nonlinear at that pressure. The haunch cracks grew slowly outward with increasing pressure. In the range 900 to 1000 psi the vertical cracks in the walls began to form, and by the time pressures of 1100 to 1200 psi were reached, these cracks were completely developed and distributed around the vessel. The circumferential cracks at midheight apparently formed only after the vertical cracking was well developed. These circumferential cracks were barely visible in all three models, and they did not appear on all six faces of the models.

These observations are in reasonable agreement with the analytical predictions, discussed in Chapters 2 and 5, that haunch cracking would begin at 600 psi, which agrees exactly with observations; that major vertical cracking would occur at 940 psi, which is within the range observed; and that horizontal or circumferential cracking at vessel midheight would occur at about 1175 psi, which is slightly higher than the actual values, at least in the prototype and the first mortar model. Figures 99 through 104 are photographs of the cracks in the prototype vessel and mortar models, and they illustrate how the families of cracks shown schematically in Fig. 98 looked in the actual vessels. The vertical cracks on the inner surface of the cylindrical portion of the prototype and on the inner surface of the cylindrical portion of the second mortar model are shown in Figs. 99 and 100, respectively. The vertical cracks on the outer surface of the cylindrical portions of the prototype and the second mortar model are shown in Figs. 101 and 102. Fine circumferential cracks are also evident on the outer surface of the prototype at midplane.

There is a close similarity between the cracks in the prototype and those in the mortar model. However, there is an observable difference in the photographs between the crack patterns on the inner surface of the prototype and on the inner surface of the second mortar model. There appear to be fewer vertical cracks in the mortar model than in the concrete prototype. There are at least two factors that probably contributed to this. First, many of the cracks in the prototype (Fig. 99) were barely visible hairline cracks that were traced over with a wax pencil. Similar

Fig. 99. Vertical Crack Pattern on Inner Surface of Cylindrical Wall of Concrete Prototype Vessel.

Fig. 100. Vertical Crack Pattern on Inner Surface of Cylindrical Wall of Second Mortar Model.

Fig. 101. Crack Pattern on Outer Surface of Cylindrical Wall of Concrete Prototype Vessel.

Fig. 102. Crack Pattern on Outer Surface of Cylindrical Wall of Second Mortar Model.

Fig. 103. Cross Section of First Mortar Model at Midheight.

Fig. 104. Vertical Cross Section of First Mortar Model.

cracks probably were not visible in the mortar model. Thus the number of large significant cracks was more nearly equal in model and prototype than the photographs indicate. Second, the cracking of mortar generally takes place with a smaller number of cracks than the cracking of concrete, which usually has microfissures.⁴⁶ The model microconcrete was not a true mortar, but it was closer to it than the concrete of the prototype and so would be expected to exhibit fewer cracks.

The first mortar model was sectioned at the midplane of the cylinder and also vertically across a diameter for inspection of interior cracking. The top half of the vessel is shown in Fig. 103 before being sectioned vertically. The oil leakage through the wall at vertical cracks stained the surrounding concrete and vividly showed up the symmetrical crack pattern that appeared in the cylindrical wall of the vessel. Since there are cracks through all but one of the ducts for the vertical tendons, these ducts obviously acted as stress raisers and determined the locations of the main cracks. It is believed, however, that the presence of the ducts had little effect on the pressure levels at which the complete cracks developed.

The vertical section through the first mortar model is shown in Fig. 104. Of particular note are the cracks extending out from each haunch region and the cracks extending inward on each side of the cylindrical wall. There is also a crack evident under the anchor of the vertical tendon on the left. Although this may have been an original fault, it has nonetheless the orientation usually observed for cracks in anchor zones of prestressed members.⁴⁷

7.2 Prestressed Epoxy Model

Testing of the prestressed epoxy model was limited to a strain-gage analysis for two elastic loadings. The first was the initial prestressing corresponding to $0.7f'_g$, and the second was the design prestressing corresponding to $0.6f'_g$ plus the design pressure of 50 psi. The resulting strain-gage data were compared with finite-element predictions for the two loading cases.

Hydraulic hand pumps were used both to tension the tendons and to apply the internal hydraulic pressure. For the initial prestressing test, strain-gage data were recorded at zero load, one-half load, full load, and at zero load again. Linearity of the strain-gage readings with load could thus be checked. The same loading levels were used for the internal pressure in the second loading case. In that case, however, the prestressing loads were held constant at $0.6f'_s$ throughout the test.

7.3 Axisymmetric Epoxy Model

The axisymmetric epoxy model was tested with an internal pressure of 50 psi, which was the pressure at which the prestressed epoxy model was tested. Both a scattered-light photoelasticity analysis and a strain-gage analysis were performed. The experimental data from the two types of tests were compared with each other and with finite-element predictions.

7.3.1 Photoelastic Test

Since the details of the photoelastic study are the subject of Ref. 17, the test is described only briefly here. When high-intensity, monochromatic, polarized light is beamed through a stressed model, scattering of the light produces alternate light and dark bands that can be analyzed to determine the stress state in the model. A laser is an ideal light source for scattered-light photoelasticity, and recent advances in photoelastic test methods have stemmed primarily from the advent of the use of lasers as a light source.^{48,49} The scattered-light test has the advantages of being three dimensional and nondestructive. It is not necessary to freeze in the stresses or to slice the model for examination.

The scattered-light polariscope used in the present study is shown in Fig. 105. For photoelastic analysis, the epoxy model was immersed in the tank of oil in the foreground. The laser beam from the continuous gas laser shown in the top of the photograph was then directed into the tank from above by means of a prism. Complete stress distributions were obtained photoelastically along the broken lines labeled 1 through 7 in Fig. 85.

Fig. 105. Scattered-Light Polariscope Used in Test of Axisymmetric Epoxy Model.

7.3.2 Strain-Gage Test

The axisymmetric model was strain gaged after the completion of the photoelastic study and was tested again with 50-psi internal pressure loading. The strain-gage test followed the same procedures as were used for testing the prestressed epoxy model. The pressure was applied with a hydraulic hand pump, and strain-gage data were recorded at zero load, one-half load, full load, and at zero load again.

8. EXPERIMENTAL RESULTS AND COMPARISONS WITH ANALYTICAL PREDICTIONS

In the introductory chapter of this report, two basic purposes of the model study program were stated. The first was to investigate the suitability of small-scale PCRV models, and the second was to evaluate the adequacy and relative accuracy of two- and three-dimensional elastic finite-element analyses. The results and comparisons presented in this chapter were thus selected to give a representative and realistic picture of the behavior of the models relative to the behavior of the prototype and of the accuracy and adequacy of the various analytical predictions.

Since it was not feasible to present all experimental results for all vessels and for all steps and phases of the testing sequence, representative gages in the regions of primary interest were chosen as "featured" gages, and results from these so-called featured gages are, as far as possible, presented for the prototype and for all models for all test phases. However, to insure that a complete appraisal of the tests can be made, essentially all the experimental data are presented for the design prestressing loadings and for the internal design pressure loadings.

As discussed previously, plane 2 (see Fig. 75) was the primary plane of interest in the models. Consequently the featured gages were chosen in that plane, as shown in Fig. 106, which depicts the "across-the-flats" section of the prototype structure cut by plane 2 and shows the locations of the surface strain-gage rosettes in plane 2. The embedded gage locations, although not actually in plane 2, are also shown. The rosetter chosen as featured gages are identified by the boxes around the numbers. These featured gages provide relatively good coverage of the upper half of the structure.

The remaining portion of this chapter is divided into three principal sections. The first deals solely with the elastic response of the models during prestressing and during pressure cycling in the elastic range. Extensive comparisons are presented between the experimental results and the elastic predictions from the two- and three-dimensional finite-element analyses.

Fig. 106. Strain-Gage Locations in Plane 2. Numbers of gages chosen for detailed presentation of results are shown in rectangular boxes.

The second section deals with the time-dependent behavior of the concrete prototype and mortar models during the time-dependent test phase. Plots of strain versus time are presented for the featured gages and for companion unloaded control specimens. In addition plots of tendon load versus time are presented.

The final section presents the experimental results obtained during the overpressure tests. Plots of pressure versus strain are presented for the featured gages. Additional plots of pressure versus vessel deflection, pressure versus tendon loads, and pressure versus crack-detection strip strains are also presented.

8.1 Elastic Behavior

8.1.1 Measured and Predicted Stress Distributions for Internal Pressure Only

In the elastic domain the stresses due both to the initial prestressing loads and to the internal pressure loading are of interest. Here the stresses due to the two types of loading have been separated, and the measured and predicted stresses due only to the internal design pressure are presented. The stresses due to the initial prestressing alone are presented in the following section.

The predicted stress distributions were obtained directly from the various finite-element analyses described in Chapter 5. The experimentally determined stresses were obtained directly from the strain-gage rosette readings by the use of Hooke's law in its general three-dimensional form. Although all the analytical predictions were obtained by using modulus of elasticity and Poisson's ratio values for the concrete and microconcrete of 3.5×10^6 psi and 0.15, respectively, the actual values obtained from test specimens for each structure, and given in Chapter 4, were used to convert the experimental strains to stresses.

8.1.1.1 Predictions Based on Two-Dimensional Analyses. Predicted and measured meridional and circumferential surface stresses for the concrete prototype are shown in Fig. 107 for an internal pressure loading of 500 psi. The curves show the finite-element predictions for each surface, and the points show values measured with strain-gage rosettes.

Fig. 107. Measured and Predicted Meridional and Circumferential Surface Stresses for Concrete Prototype Vessel Subjected to Internal Pressure of 500 psi. Predictions based on two-dimensional analyses.

The finite-element results are primarily from the axisymmetric analysis, although the results of the plane analysis of the head penetration region have been used to approximately account for the effects of the penetrations in both plane 2 and planes 1 and 3.

The cross section of the prototype that is shown in Fig. 107 corresponds to that cut by plane 2, and it may be recalled from Chapter 5 that the axisymmetric finite-element analysis was based on this cross section. Nevertheless, the experimental data shown in the figure were obtained not only from gages in plane 2 but also from the gages in planes 1 and 3. In addition, results from the few gages that were located off the three main planes near penetrations and anchor plates (see Fig. 76) are also plotted. Thus, not all the experimental results are directly comparable with the finite-element predictions.

For the plane 2 data, in particular, it can be seen that for the meridional stress the agreement between theory and experiment is reasonably good. For the circumferential stress, agreement is reasonably good except on the inner surface of the cylinder. The disagreement there can be attributed to the nonaxisymmetric cross section of the vessel wall. Both the plane analysis of a section of the vessel wall and the three-dimensional analysis indicated that the effects of the hexagonal outer shape of the wall and of the heavy steel anchor blocks for the circumferential prestressing tendons had a significant influence on the circumferential stress distribution around the vessel wall. The difference between the plane 2 experimental data and the planes 1 and 3 experimental data on the inner wall illustrates this variation.

Figures 108 and 109 for the first and second mortar models are analogous to Fig. 107 for the prototype. The analytical predictions shown in the three figures are exactly the same. Since the first mortar model was not fully instrumented, only a limited number of experimental points are available for comparison with predictions. The available data do, however, agree with the analytical predictions to about the same extent as the prototype data. The second mortar model had essentially the same instrumentation as the prototype, and therefore Fig. 109 offers a better comparison of model data with predictions. As in the case of the prototype, the agreement between theory and experiment is reasonably good, except in the case of the circumferential stress on the inner surface of the cylinder.

To compare the mortar model data directly with the prototype data, the plane 2 experimental data for all three vessels were replotted in Fig. 110.

Again the solid lines show the axisymmetric finite-element predictions. This figure illustrates the close agreement between mortar models and concrete prototype for the design pressure loading.

Fig. 108. Measured and Predicted Meridional and Circumferential Surface Stresses for First Mortar Model Subjected to Internal Pressure of 500 psi. Predictions based on two-dimensional analyses.

Fig. 109. Measured and Predicted Meridional and Circumferential Surface Stresses for Second Mortar Model Subjected to Internal Pressure of 500 psi. Predictions based on two-dimensional analyses.

Fig. 110. Comparison of Meridional and Circumferential Surface Stresses in Plane 2 for Concrete Prototype and Mortar Models Subjected to Internal Pressure of 500 psi. Predictions based on two-dimensional analyses.

The predicted and measured meridional and circumferential stresses for the prestressed epoxy model are shown in Fig. 111 for an internal pressure of 50 psi. It may be recalled from the discussion of the prestressed epoxy model in Chapter 3 that the design prestress and pressure

Fig. 111. Measured and Predicted Meridional and Circumferential Surface Stresses for Prestressed Epoxy Model Subjected to Internal Pressure of 50 psi. Predictions based on two-dimensional analyses.

loadings were chosen to produce stresses one-tenth those in the prototype. Thus Fig. 111 is analogous to the previous figures showing the prototype and mortar model results except that the stresses are reduced by a factor of 10. To compensate, the plotting scale was increased by the same factor.

The curves in Fig. 111 show the finite-element predictions which, it may be recalled from the discussions in Chapter 5, were obtained from analyses made specifically for the prestressed epoxy model. Because of the differences in Poisson's ratios (0.15 for the concrete versus 0.38 for the epoxy) and because the top-head penetrations in the epoxy model were unlined, unlike the prototype penetrations, the predicted stresses in the epoxy model are not quite the same as those in the concrete prototype. By comparing Figs. 107 and 111 it may be seen that aside from the penetration region the predicted meridional stresses are essentially the same for the epoxy model and the concrete prototype. However, in regions of high stress gradients, notably near the haunch, which joins the head to the cylindrical portion of the vessel, the predicted circumferential stresses are significantly different in magnitude. This is understandable since the steep circumferential gradients are in large part related to the meridional gradients or discontinuities through Poisson's ratio. Thus the circumferential stresses in regions of large gradients are larger in magnitude than the corresponding stresses in the prototype.

The agreement between the measured and predicted meridional stresses is reasonably good in Fig. 111. For the circumferential stresses the agreement is also reasonable except on the inner surface of the cylinder, where the measured values are considerably below the predicted values. Just as in the case of the concrete prototype and the mortar models, this disagreement is due to the hexagonal outer shape of the wall and to the large anchor blocks for the circumferential tendons, both of which were neglected by the axisymmetric analysis.

Generally the experimental data for the prestressed epoxy model agreed at any given location with analytical predictions to about the same degree that the concrete prototype and mortar model experimental data agreed with analytical predictions. In fact, except for the penetration regions and

the limited regions of high stress gradient mentioned above, the experimental data from the epoxy model and the concrete prototype and mortar models agreed reasonably well.

The final model tested was the axisymmetric epoxy model. One of the purposes of testing that model was to demonstrate the reliability and accuracy of finite-element techniques applied to an idealized, truly axisymmetric model of a thick-walled vessel. The model was tested both photo-elastically and with strain gages at an internal pressure of 50 psi, and in Fig. 112 both sets of experimental results are compared with axisymmetric finite-element predictions of the meridional and circumferential surface stresses. The agreement between the three sets of data is generally very good.

Comparison of Fig. 112 for the axisymmetric epoxy model with Fig. 111 for the prestressed epoxy model shows the degree to which the nonaxisymmetric features of the prestressed model affected the stress distributions. Likewise, comparison of Fig. 111 for the prestressed epoxy model with Figs. 107 through 110 for the concrete prototype and mortar models shows the degree to which the epoxy model stresses deviated from the concrete and mortar model stresses.

8.1.1.2 Predictions Based on Three-Dimensional Analysis. The three-dimensional elastic finite-element analysis of the prototype vessel was briefly described in Chapter 5. Here the resulting predictions for the internal pressure loading are compared with both experimental results and two-dimensional predictions. Since experimental results from all the models were presented and compared in the previous section, only the prototype data are generally considered here.

The predicted and measured meridional and circumferential surface stresses for plane 2 are shown in Fig. 113 and for planes 1 and 3, in Fig. 114. The agreement between theory and experiment is much better than that between the experimental data and the axisymmetric predictions, particularly in the case of the circumferential stresses on the inner surface. This improved agreement is emphasized in Figs. 115 and 116, where both the three- and two-dimensional finite-element predictions are compared with experimental data. An examination of these figures indicates that the

Fig. 112. Measured and Predicted Meridional and Circumferential Surface Stresses for Axisymmetric Epoxy Model Subjected to Internal Pressure of 50 psi.

axisymmetric analysis has generally overestimated the stresses compared with both the three-dimensional analysis and the experimental results.

Additional comparisons between the experimental results and the two- and three-dimensional finite-element predictions are shown in Figs. 117 and 118. Figure 117 shows the predicted deflection profile of the outer surface of the prototype at a section across the flats of the outside hexagonal shape (plane 2) and at a section across the vertices (planes 1

Fig. 113. Measured and Predicted Meridional and Circumferential Surface Stresses in Plane 2 for Concrete Prototype Vessel Subjected to Internal Pressure of 500 psi. Predictions based on three-dimensional analysis.

Fig. 114. Measured and Predicted Meridional and Circumferential Surface Stresses in Planes 1 and 3 for Concrete Prototype Vessel Subjected to Internal Pressure of 500 psi. Predictions based on three-dimensional analysis.

- ▲ EXPERIMENTAL RESULTS, PLANE 1
- EXPERIMENTAL RESULTS, PLANE 3
- EXPERIMENTAL RESULTS, PLANE 2
- THREE-DIMENSIONAL FINITE ELEMENT PREDICTIONS
- - - TWO-DIMENSIONAL AXISYMMETRIC PREDICTIONS

Fig. 115. Comparison of Meridional Stresses Predicted by Two- and Three-Dimensional Analyses with Test Results for Concrete Prototype Vessel Subjected to Internal Pressure of 500 psi.

- ▲ EXPERIMENTAL RESULTS, PLANE 1
- EXPERIMENTAL RESULTS, PLANE 3
- EXPERIMENTAL RESULTS, PLANE 2
- THREE-DIMENSIONAL FINITE ELEMENT PREDICTIONS
- - - TWO-DIMENSIONAL AXISYMMETRIC PREDICTIONS
- TWO-DIMENSIONAL PLANE-STRAIN PREDICTIONS

Fig. 116. Comparison of Circumferential Stresses Predicted by Two- and Three-Dimensional Analyses with Test Results for Concrete Prototype Vessel Subjected to Internal Pressure of 500 psi.

Fig. 117. Measured and Predicted Deflections for Concrete Prototype Vessel Subjected to Internal Pressure of 500 psi.

Fig. 118. Measured and Predicted Circumferential Stress Distributions Around Cylindrical Wall of Concrete Prototype and Second Mortar Model Subjected to Internal Pressure of 500 psi.

and 3). The solid lines represent deflections predicted by the three-dimensional analysis, and the dashed lines on the section taken across the flats represent deflections predicted by the axisymmetric analysis. Finally the points are experimental measurements made on the concrete prototype at an internal pressure of 500 psi. The agreement between the three-dimensional analysis predictions and the experimental results is reasonably good, while the axisymmetric analysis overestimated the deflections. This result was expected, since the relatively large steel components, as well as the outside hexagonal shape, were not accounted for in the axisymmetric analysis. The idealized axisymmetric model analyzed was thus less stiff than either the actual structure or the more realistic model treated in the three-dimensional analysis.

The variation of circumferential stress around the wall of the vessel is further illustrated in Fig. 118. Here the measured and predicted circumferential surface stresses are plotted on a cross-sectional drawing of a 60° sector of the vessel wall. Experimental data from both the concrete prototype and the second mortar model are shown. Results from strain-gage rosettes at the levels of the second and third circumferential bands of prestressing are shown, as well as results from rosettes located at a level midway between the two bands (see Fig. 76).

The solid curves are based on the three-dimensional analysis and show the predicted variation at the level of the second circumferential band of prestressing. The dashed curves show the results from the plane-strain two-dimensional analysis of the vessel wall that was described in Chapter 5. Finally, the shaded bands show the stress ranges predicted between the second and third bands by the axisymmetric analysis.

The axisymmetric analysis predicted, of course, no variation of stress around the wall. The plane analysis, on the other hand, overestimated the circumferential variation because it was assumed that the cross section of Fig. 118 extended infinitely in the axial direction, while actually the cross section between bands of circumferential prestressing differed significantly. The axisymmetric and plane analyses together, however, give a reasonable idea of the stress variation.

8.1.2 Measured and Predicted Stress Distributions for Prestressing Loads Only

In past tests on models of prestressed concrete reactor vessels, it has generally been found that the correlation between theory and experiment is clearly better for pressure loadings than for prestressing loadings.¹⁴ The results presented here are no exception.

Several plausible reasons for this relatively poor correlation can be advanced. First, the prestressing loads are applied at discrete points through steel anchor plates. This naturally produces a much more distorted or irregular stress distribution than that produced by the uniform internal pressure loading. This is particularly true in most models where the prestressing forces are applied by a relatively small number of tendons. In this respect the model prestressing loadings are more grossly nonuniform than are those in actual vessels where hundreds of prestressing tendons are used. Further, most of the specific features of the prestressing system affecting the stress distribution are neglected in the usual axisymmetric analysis.

Second, the prestressing loading is the first loading to which a PCRV is subjected. There are no previous "shake-down" loadings. In almost any structure the first loading cycle produces more erratic strains than subsequent loadings.

Third, gaps and voids probably exist between the concrete and embedded steel parts, particularly the prestressing anchor plates. The compression produced by the prestressing forces tends to close these gaps. Thus they may have a significant effect on local strain distributions during prestressing, but no appreciable effect during normal pressure loadings.

Finally, prestressing usually takes place over a sufficiently long period for transient creep effects to be significant. In the case of the models discussed here, the entire prestressing operation for each model was completed in a single day. Yet, initial creep strains accumulated in just a few hours can be quite significant. In some model tests, fairly elaborate correction schemes have been employed to factor creep strains out of the prestressing data.⁴⁶ This was not done here.

8.1.2.1 Predictions Based on Two-Dimensional Analyses. Predicted and measured meridional and circumferential surface stresses produced by the initial prestressing loads are shown in Figs. 119 through 122 for the concrete prototype, the first mortar model, the second mortar model, and the prestressed epoxy model. Unlike the pressure case, the analytical predictions for the concrete prototype and the mortar models are not identical because separate analyses were performed for each by using the actual (measured) prestressing loads that existed at the end of the prestressing sequence (see Table 8). The experimental points in each figure show the stresses that were calculated on an elastic basis from the strains existing at the end of the prestressing sequence.

The irregular stress distributions are readily apparent in Figs. 119 through 122. The large peaks in the distributions occurred in the regions where the prestressing loads were applied, and since the axisymmetric analysis did not take account of the steel anchor plates and blocks nor of the discrete spacing of the loads circumferentially around the structures, these peaks are unrealistic. Away from these regions the predicted stresses are fairly uniform and agree reasonably well with experimental results.

8.1.2.2 Predictions Based on Three-Dimensional Analysis. The comparisons of the three-dimensional finite-element predictions with experimental results for the initial prestressing loading on the concrete prototype are shown in Fig. 123 for plane 2 and in Fig. 124 for planes 1 and 3. The agreement between theory and experiment is not so good here as it was in the case of the internal pressure loading.

The disagreement in the regions of anchor plates and blocks is of particular interest. The meridional stresses in the head near the anchor plates for the axial tendons may be considered as an example. The compressive stresses normal to the concrete-steel interfaces are not nearly so large as predicted. The difference is probably due to the presence of small gaps between steel and concrete, as previously discussed. Until these gaps were closed, normal stresses could not be transmitted between the steel and the concrete, and consequently they were zero at the interface.

Fig. 119. Measured and Predicted Meridional and Circumferential Surface Stresses for Concrete Prototype Vessel Subjected to Initial Prestressing Loads Only. Predictions based on two-dimensional analyses.

Fig. 120. Measured and Predicted Meridional and Circumferential Surface Stresses for First Mortar Model Subjected to Initial Prestressing Loads Only. Predictions based on two-dimensional analyses.

Fig. 121. Measured and Predicted Meridional and Circumferential Surface Stresses for Second Mortar Model Subjected to Initial Prestressing Loads Only. Predictions based on two-dimensional analyses.

Fig. 122. Measured and Predicted Meridional and Circumferential Surface Stresses for Prestressed Epoxy Model Subjected to Initial Prestressing Loads Only. Predictions based on two-dimensional analyses.

Fig. 123. Measured and Predicted Meridional and Circumferential Surface Stresses in Plane 2 for Concrete Prototype Vessel Subjected to Initial Prestressing Loads Only. Predictions based on three-dimensional analysis.

Fig. 124. Measured and Predicted Meridional and Circumferential Surface Stresses in Planes 1 and 3 for Concrete Prototype Vessel. Subjected to Initial Prestressing Loads Only. Predictions based on three-dimensional analysis.

8.1.3 Measured and Predicted Stress Variations at Selected Points During Prestressing and Pressure Testing

The comparisons between theory and experiment presented in the previous two sections were confined to just two specific loadings — the initial prestressing and internal design pressure. The concrete prototype and the two mortar models were actually subjected to many loading combinations during prestressing and later during pressure cycling. Data were recorded at each step in the prestressing sequence and the pressure cycling, and these results for each gage rosette location were compared on a step-by-step basis with predictions. These comparisons are perhaps more informative than those presented in the previous sections, and consequently the measured and predicted stresses for the entire elastic test history are presented here for each of the featured surface gages and for all three vessels.

The measured and predicted meridional and circumferential stresses for gage rosette C-209, which was located near midheight on the inner surface of the wall of the concrete prototype, are shown in Fig. 125 for the

Fig. 125. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-209 on Concrete Prototype Vessel.

12 steps in the prestressing sequence and for pressure cycles from 0 to 300 psi twice, 0 to 600 psi, and 0 to 900 psi. The solid steps are axisymmetric finite-element predictions, while the points are experimental results. The agreement is reasonable except for the circumferential stresses during the pressure testing. This disagreement caused by the nonaxisymmetric cross section of the wall was discussed previously.

One important feature of Fig. 125 and subsequent similar plots should be mentioned. Between the prestressing sequence and the pressure cycling there was the period of time-dependent testing during which creep and shrinkage strains occurred. Thus at the beginning of the pressure cycling the total strains referenced to the original zeros at the beginning of prestressing could not be used to calculate stresses because they contained an inelastic component. For purposes of preparing the plots a somewhat artificial procedure was adopted.

An examination of the tendon loads during the time-dependent test phase indicated that on the average the loads dropped about 5% for the prototype and the mortar models. A rough approximation then was that the stresses in the vessels at the end of the time-dependent test period were 95% of what they were immediately after prestressing. This of course ignored the stress redistribution that takes place with creep and shrinkage. Nonetheless the predicted stresses shown in Fig. 125 for the beginning of the pressure cycling are those existing at the end of prestressing reduced by 5%. The predictions for the pressure cycles were plotted from this base, as were the experimental results for pressure cycling.

Figures 126 and 127 for the first and second mortar models are analogous to Fig. 125 for the prototype. In both cases the agreement between theory and experiment is again reasonable, except for the circumferential stresses produced by internal pressure. Also, by comparing Figs. 126 and 127 with Fig. 125, it may be seen that the experimental results from the two models agree well with those from the prototype. Two points should be made, however, in comparing the plots for the three different structures. First, the measured prestressing loads acting on each structure at each step were used to obtain the theoretical prestressing predictions. Hence the predictions for the prototype and the two mortar models are

Fig. 126. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-209 on First Mortar Model.

Fig. 127. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-209 on Second Mortar Model.

not quite the same. Second, in the case of the first mortar model the prestressing forces existing after the 12 steps in the prestressing sequence were slightly adjusted by an additional step. Thus there were 13 steps, and in Fig. 126, as in most other figures presented on the following pages for the first mortar model, 13 prestressing steps rather than 12 are shown.

In the remainder of this section, sets of figures analogous to Figs. 125, 126, and 127 are presented for each of the featured surface gage rosettes shown in Fig. 106. Figures 128, 129, and 130 are for rosette C-211, and Figs. 131, 132, and 133 are for rosette C-213. Like rosette C-209, these rosettes were located on the inner surface of the cylinder. The degree of agreement between theory and experiment and between experimental points for models and prototype is about the same as for rosette C-209.

The haunch area is a more difficult region in which to predict behavior. Figures 134, 135, and 136 show the results for rosette C-214, which was located on the haunch of the concrete prototype, the first mortar model, and the second mortar model. Here the agreement between theory and experiment is not very good for the prototype and the second mortar model, although the agreement between experimental points for the two is good. The disagreement between theory and experiment at this location of high stress gradient is probably worse than for any other significant area in the upper half of the vessels.

The next region considered is the inner surface of the top head. Figures 137 and 138 show results for rosette C-215, and Figs. 139, 140, and 141 show results for rosette C-220 (actually, rosette C-219, which was centered between penetrations, was substituted for rosette C-220 in the case of the first mortar model). In the case of rosette C-215, one of the gages on the second mortar model failed during the test. Consequently, no results are shown for that model.

For rosette C-215, the agreement between theory and experiment is reasonably good, and the agreement between model and prototype is also good. For rosette C-220, which was located between penetrations in the head, the theoretical predictions consist of axisymmetric finite-element results modified by the results of the plane-stress finite-element analysis of the penetration region. The agreement between theory and experiment for this location is very good for the prototype and for both mortar models.

Fig. 128. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-211 on Concrete Prototype Vessel.

Fig. 129. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-211 on First Mortar Model.

Fig. 130. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-211 on Second Mortar Model.

Fig. 131. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-213 on Concrete Prototype Vessel.

Fig. 132. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-213 on First Mortar Model.

Fig. 133. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-213 on Second Mortar Model.

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Fig. 134. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-214 on Concrete Prototype Vessel.

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Fig. 135. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-214 on First Mortar Model.

Fig. 136. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-214 on Second Mortar Model.

Fig. 137. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-215 on Concrete Prototype Vessel.

Fig. 138. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-215 on First Mortar Model.

Fig. 139. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-220 on Concrete Prototype Vessel.

Fig. 140. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-219 on First Mortar Model.

Fig. 141. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-220 on Second Mortar Model.

The remaining comparisons to be made are for rosettes on the outer surfaces of the structures. Figures 142 and 143 are for rosette C-258, which was located on the outer surface of the cylinder opposite rosette C-209, and Figs. 144 and 145 are for rosette C-260, which was located on the outer surface of the cylinder opposite rosette C-211. There were no rosettes on the first mortar model at these locations. The agreement between theory and experiment is very good at these two locations.

The final comparisons are shown in Figs. 146, 147, and 148 for rosette C-272 and in Figs. 149, 150, and 151 for rosette C-276. Both these rosettes were located on the outer surface of the head. Rosette C-276 was located between penetrations in the head, and the theoretical predictions for its location are, as in the case of rosette C-220, a combination of axisymmetric finite-element results and results from the plane-stress finite-element analysis of the penetration region. The agreement between theory and experiment for these outside head gages is reasonable.

8.2 Time-Dependent Behavior

The time-dependent test period for the concrete prototype and the two mortar models began immediately after completion of the prestressing operation and was continued for 38 days. On the thirty-third day the pressure tests were conducted, and on the thirty-fifth day the tendons were detensioned. Except for the periods of pressure testing and detensioning, data were recorded at regular and frequent intervals throughout the time-dependent test period. In this section the time-dependent strains measured by the featured gages on the prototype and the models are compared. Also the time-dependent strains measured simultaneously on unloaded control specimens for each structure are compared. Finally, the average tendon loads measured with time for the prototype and for each model are presented.

8.2.1 Time-Dependent Concrete Strains in Prototype and Models

Time-dependent strain plots are presented in this section with little comment or explanation. Many factors affect the creep strains at a given location, including the initial elastic stress distribution, the basic

Fig. 142. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-258 on Concrete Prototype Vessel.

Fig. 143. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-258 on Second Mortar Model.

Fig. 144. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-260 on Concrete Prototype Vessel.

Fig. 145. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-260 on Second Mortar Model.

Fig. 148. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-272 on Second Mortar Model.

Fig. 149. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-276 on Concrete Prototype Vessel.

Fig. 150. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-276 on First Mortar Model.

Fig. 151. Measured and Predicted Meridional and Circumferential Stresses for Gage Rosette C-276 on Second Mortar Model.

creep response characteristics of the material, the age and moisture condition of the material, and the moisture migration within the structure. Obviously no attempt was made to achieve similitude between models and prototype for all these factors.

Even between the two mortar models there was an expected difference in time-dependent behavior. The first mortar model was prestressed at 68 days, while the second mortar model was prestressed at 158 days. Consequently significant drying shrinkage occurred in the first mortar model during the time-dependent test period, while only negligible shrinkage occurred in the second mortar model during the time-dependent test period.

Apparently creep may be considered as the sum of two components; namely, basic creep and drying creep.⁵⁰ Basic creep occurs independently of moisture movement, while the drying creep component has characteristics similar to those of free drying shrinkage and is in fact dependent on the free shrinkage. Consequently, other things being equal, larger creep strains would be expected in the first mortar model than in the second.

Also the age of a structure at loading has a large effect on creep,⁵¹ with younger specimens exhibiting considerably more creep than older ones. This would again indicate larger creep strains in the first mortar model than in the second.

An attempt was made at Gulf General Atomic, Inc., to predict the creep behavior of the models of this study by using the linear viscoelastic finite-element creep analysis procedure described in Ref. 32, and the predicted creep strains at various locations were compared with measured values from the first mortar model.⁵² The agreement was better than expected and was probably fortuitous, at least in part, since the creep response used by the analysis was for a completely unrelated concrete.

The meridional and circumferential time-dependent surface strains measured on the prototype and mortar models are shown in Figs. 152 through 155 for the inside rosettes C-209, C-211, C-214, and C-220, and in Figs. 156 through 159 for outside rosettes C-258, C-260, C-272, and C-276. The strain levels are measured from those existing at the beginning of the prestressing sequence, and the time-dependent changes are shown as a function of the number of days since the completion of prestressing.

Fig. 152. Comparison of Measured Meridional and Circumferential Strains Versus Time in Concrete Prototype and Mortar Models for Gage Rosette C-209.

The measured meridional and circumferential strains indicated by the embedded gage rosette E-4 are shown as a function of time in Fig. 160 for the concrete prototype and the second mortar model. As was discussed in Chapter 6, which described the instrumentation, these embedded gages were expected to exhibit poor long-term stability due to the ingress of water into the epoxy encapsulating material. However, no apparent erratic behavior is evident from Fig. 160. Also, a comparison of Fig. 160 with

Fig. 153. Comparison of Measured Meridional and Circumferential Strains Versus Time in Concrete Prototype and Mortar Models for Gage Rosette C-211.

Figs. 153 and 157, which showed the results for rosette C-211, the inside surface rosette at the same level as E-4, and rosette C-260, the outside rosette at the same level as E-4, indicates that the embedded gages exhibited behavior close to that of the corresponding surface gages.

Shrinkage strains that occurred in unloaded control specimens corresponding to the prototype and the two mortar models are shown in Fig. 161. The control specimens (three for the prototype, two for the first

Fig. 154. Comparison of Measured Meridional and Circumferential Strains Versus Time in Concrete Prototype and Mortar Models for Gage Rosette C-214.

Fig. 155. Comparison of Measured Meridional and Circumferential Strains Versus Time in Concrete Prototype and Mortar Models for Gage Rosette C-220.

mortar model, and four for the second mortar model) were extra flexural test beams that were cast along with the respective vessels and subjected to exactly the same environment as the vessels. It may be seen that the specimens for the prototype and second mortar model, which were relatively old, exhibited very small amounts of shrinkage, while the specimens for the first mortar model, which was relatively young, exhibited quite

Fig. 156. Comparison of Measured Meridional and Circumferential Strains Versus Time in Concrete Prototype Vessel and Second Mortar Model for Gage Rosette C-258.

significant shrinkage. This dependence of free shrinkage on age is in general agreement with data obtained by others.⁵³

8.2.2 Time-Dependent Tendon Load Losses and Relaxation

The average tendon loads as a function of time after prestressing are shown in Figs. 162, 163, and 164 for the prototype and the first and second mortar models. Generally, the curves shown for the axial tendons

Fig. 157. Comparison of Measured Meridional and Circumferential Strains Versus Time in Concrete Prototype Vessel and Second Mortar Model for Gage Rosette C-260.

are the average results from the four instrumented axial tendons in each vessel, and the curves shown for the head tendons are the average results from the three instrumented head tendons. The solid lines show the actual tendon loads as indicated by the load transducers, while the dashed lines show the apparent tendon loads determined on an elastic basis from the tendon strains. It was assumed that at day zero the actual tendon strains gave a correct indication of the loads. Thus in those cases where the

Fig. 158. Comparison of Measured Meridional and Circumferential Strains Versus Time in Concrete Prototype and Mortar Models for Gage Rosette C-272.

indicated transducer loads did not quite agree with the loads determined from the tendon strains, the transducer calibration factors were altered slightly to bring the two into agreement. This was equivalent to using the initial prestressing loadings to calibrate the transducers.

As would be expected the actual tendon loads indicated by the transducers at the anchorages decreased with time. This decrease was due to creep and shrinkage of the concrete and possibly to tendon relaxation.

Fig. 159. Comparison of Measured Meridional and Circumferential Strains Versus Time in Concrete Prototype Vessel and Second Mortar Model for Gage Rosette C-276.

Fig. 160. Comparison of Measured Meridional and Circumferential Strains Versus Time in Concrete Prototype Vessel and Second Mortar Model for Gage Rosette E-4.

That part of the load loss due to tendon relaxation can be determined by comparing the transducer loads with the apparent loads determined from the tendon strains. Since relaxation occurs at constant strain the loads calculated from the tendon strains show no decrease due to relaxation. Consequently the difference between the dashed curves and the solid curves represents the load loss due to relaxation of the tendons.

Fig. 161. Shrinkage Strains Versus Time in Unloaded Control Specimens.

In the concrete prototype the tendon relaxation was small but apparent. For the first mortar model the relaxation appeared to be more significant. Finally, in the case of the second mortar model, some of the dashed curves are below the solid curves. No explanation can be offered for this other than that it may simply be due to inaccuracies of the load transducers.

Fig. 162. Average Tendon Forces Versus Time for Concrete Prototype Vessel.

Fig. 163. Average Tendon Forces Versus Time for First Mortar Model.

Fig. 164. Average Tendon Forces Versus Time for Second Mortar Model.

Actually the load transducers used were not sufficiently precise to give the kind of measurements needed for an accurate determination of relaxation. Their main drawback was their relatively short length and the resulting large unpredictable effect of end conditions. Transducers of the design used should be considerably longer. Nonetheless the results do seem to indicate that in addition to the concrete creep and shrinkage, some small measurable amount of tendon relaxation did occur.

8.3 Behavior of Prototype and Models at Overpressures

The general short-time response of a correctly designed PCRV to an increasing internal pressure loading can best be explained by the idealized general load-deformation curve shown in Fig. 165 (Ref. 13). The figure shows three distinct regimes of structural behavior that have been repeatedly observed in tests of PCRV's.

Fig. 165. Idealized General Load-Deformation Behavior of a PCRV.

In the first regime, which should extend to a pressure level slightly above the design pressure, the overall vessel response is linearly elastic. In this regime the compressive strains produced in the concrete by the prestressing loads are being reduced, and at the upper end of the regime localized strains may go into the tensile range. With a further increase in pressure the general tensile strains in the concrete reach the point where significant cracking occurs, and the response curve begins to bend over markedly. This is the second regime. As the pressure continues to increase, additional cracking occurs, but since the steel prestressing tendons and reinforcing remain elastic the overall vessel

response remains elastic (but not linearly so), and the vessel deformations remain more or less reversible. In the third regime the crack pattern becomes fully developed, and for the most part the steel carries all the load. In this regime steel reinforcing begins to deform plastically and, later, just before the ultimate load is reached, the prestressing tendons also may begin to deform plastically.

It is on the basis of the progressive behavior illustrated in Fig. 165 that sudden and unexpected structural failure of a PCRV is considered highly unlikely. The overall load-deformation characteristics of the concrete prototype and the two mortar models were generally similar to those shown in Fig. 165. The overall vessel deflections and the tendon strains in particular exhibited the same behavior. Not all the concrete strain gages, however, indicated the typical pressure-strain response. This is understandable because the strains at any one location in a PCRV are dependent on the overall vessel behavior. Cracking in one location can very easily result in strain relief in another location. The gross behavior, however, would remain the same as that exhibited in Fig. 165.

In the remainder of this section, representative load-deformation results for prototype and models are presented to demonstrate the response of the structures to overpressure loadings and to show the similar behavior of models and prototype. The results include pressure versus strain curves for featured gages on the prototype and second mortar model, measured and predicted deflection versus pressure curves for the top head of the prototype, pressure versus tendon load curves for the prototype, and finally, pressure versus strain curves for representative crack-detection strips on the prototype and the models.

8.3.1 Concrete Strains Versus Pressure

The meridional and circumferential strains measured as a function of pressure on the concrete prototype and the second mortar model are shown in Figs. 166 through 169 for inside rosettes C-209, C-211, C-214, and C-220. The data are for the two pressure cycles to 300 psi, one to 600 psi, one to 900 psi, and the final cycle to the maximum pressure, which was 1130 psi for the prototype and 1390 psi for the second mortar model.

Fig. 166. Measured Meridional and Circumferential Strains with Cycles to Increasing Pressures for Gage Rosette C-209 on Concrete Prototype Vessel and Second Mortar Model.

Fig. 167. Measured Meridional and Circumferential Strains with Cycles to Increasing Pressures for Gage Rosette C-211 on Concrete Prototype Vessel and Second Mortar Model.

Fig. 168. Measured Meridional and Circumferential Strains with Cycles to Increasing Pressures for Gage Rosette C-214 on Concrete Prototype Vessel and Second Mortar Model.

The last data points in the final cycle do not correspond to the highest pressures reached but to the highest pressures for which complete data sets were recorded.

Figures 170 through 173 show corresponding results for outside rosettes C-258, C-260, C-272, and J-276. Finally, Fig. 174 shows similar results for embedded gage rosette E-4. The prototype and model behavior characteristics are quite similar in almost every case.

Fig. 169. Measured Meridional and Circumferential Strains with Cycles to Increasing Pressures for Gage Rosette C-220 on Concrete Prototype Vessel and Second Mortar Model.

Fig. 170. Measured Meridional and Circumferential Strains with Cycles to Increasing Pressures for Gage Rosette C-258 on Concrete Prototype Vessel and Second Mortar Model.

Fig. 171. Measured Meridional and Circumferential Strains with Cycles to Increasing Pressures for Gage Rosette C-260 on Concrete Prototype Vessel and Second Mortar Model.

Fig. 172. Measured Meridional and Circumferential Strains with Cycles to Increasing Pressures for Gage Rosette C-272 on Concrete Prototype Vessel and Second Mortar Model.

In most cases the local strains began to decrease as the vessel test entered the second and third regimes. This is understandable because as the concrete crack patterns developed and the cracks opened up, the stresses in the concrete were relieved and the strains decreased. There were exceptions to this where cracks in one area caused increased stresses in another area or where microcracking actually occurred under a gage. This

Fig. 173. Measured Meridional and Circumferential Strains with Cycles to Increasing Pressures for Gage Rosette C-276 on Concrete Prototype Vessel and Second Mortar Vessel.

Fig. 174. Measured Meridional and Circumferential Strains with Cycles to Increasing Pressures for Gage Rosette E-4 in Concrete Prototype Vessel and Second Mortar Model.

latter effect is much more noticeable in the results from the long crack-detection gages that will be shown later.

8.3.2 Vessel Head Deflection Versus Pressure

An approximate axisymmetric elastic cracking analysis of the prototype was described in Chapter 5, and a plot of the predicted center-head

deflection versus pressure was given. In Chapter 7 the measured response of the prototype head was shown as a function of pressure. In Fig. 175 the measured and predicted behavior characteristics of the top head are compared.

The obvious disagreement occurred largely in the elastic response and was discussed in connection with Fig. 117. It was concluded that due to the simplifying assumptions made the structure actually analyzed by the axisymmetric finite-element analysis was much less stiff than the actual structure with its hexagonal outer shape and embedded steel parts. Other than this basic shortcoming of the axisymmetric analysis in predicting deflections, the results of the cracking analysis seem reasonable. The predicted pressure at which cracking first occurred (the beginning of the second regime) agrees well with the measured response. Also the degree of nonlinearity is in reasonable agreement.

Fig. 175. Comparison of Measured and Predicted Top-Head Deflection for Concrete Prototype Vessel.

8.3.3 Tendon Loads Versus Pressure

The changes in average tendon forces with increasing pressure are shown in Fig. 176 for the concrete prototype. The data points for the axial tendons and the head tendons are the averages of all tendons of each type that were instrumented. For the circumferential tendons there was only one instrumented tendon per band.

The tendon forces at zero pressure were given in Fig. 162, which showed the time-dependent behavior of the prototype tendons during the period between prestressing and pressure testing. For the axial tendons the average tendon force at the beginning of the pressure tests was 106.5 kips; for the head tendons, 135.0 kips; and for the circumferential tendons the average for all four bands was 143.9 kips. On this basis the average axial tendon force increased by approximately 1.5% due to the application of the internal design pressure of 500 psi; the average head tendon force increased by 0.9%; and the average circumferential force in the second and third bands, which are the middle two bands, increased by approximately 2.2%. These small increases illustrate one of the desirable characteristics of a concrete vessel: under the normal range of loading, the loads in the actual load-carrying members (the tendons) change very little. Fatigue problems due to cyclic loadings are thus virtually nonexistent.

The response of both the axial and circumferential tendons followed the three-regime concept illustrated in Fig. 165. Since the head tendons went directly across the head near its midplane and since no cracking apparently occurred in the head, these tendons showed very little nonlinear response. For the axial tendons and the circumferential tendons, the tendon loads began to increase rapidly with increasing pressure as cracking occurred.

It is interesting to examine the response of the tendons in the third regime. The data indicate that the behavior in that regime was very nearly linear. Thus the concrete crack pattern had developed, at least in part, and the tendons were directly carrying a portion of the internal pressure loading.

The sloping line segments shown on the axial and circumferential tendon plots serve to indicate how much of the load was actually being

Fig. 176. Average Tendon Force Increase Versus Internal Pressure for Concrete Prototype Vessel.

directly carried by the tendons. These slopes give the calculated rate of load increase on the basis of the tendons carrying all the axial and hoop loads. For the circumferential tendons the final slopes of the measured response curves for the second and third band tendons closely match the calculated slope. This illustrates that the pattern of vertical cracks had completely developed and the cracks had opened up all the way across the thickness of the wall. The slopes for the first and fourth band tendons are greater; this is due to the restraining effects of the heads.

In the case of the axial tendons the measured final slope is also greater than that predicted on the basis of the axial tendons carrying all the axial load. This is reasonable since none of the circumferential cracks extended radially all the way across the vessel wall. Thus the concrete continued to carry some axial load.

The final point to be made regarding Fig. 176 is that when the response curves for the second and third band tendons are replotted on a graph showing pressure versus total tendon force, a straight line tangent to the curves in the third regime passes very nearly through the origin for zero tendon force and through a point corresponding to the predicted failure pressure of 1560 psi for rupture of the circumferential tendons at their ultimate strength of 261.5 kips. This further illustrates that the circumferential tendons were carrying the full hoop load in the mid-portion of the vessel.

8.3.4 Crack-Detection Strip Results

The final three figures of this report, Figs. 177, 178, and 179, show comparisons of crack-detection strip results for the prototype and two mortar models. Strips located in three significant regions were chosen for presentation. The results shown in Fig. 177 are for vertical, or meridional, strips on the cylindrical portion of the vessels, as shown. Since there were no circumferential cracks observed in this region, no extremely large indicated strains that would lead to failure of the gages would be expected.

Fig. 177. Comparison of Indicated Behavior for Meridional Crack-Detection Strip on Cylindrical Wall of Concrete Prototype and Mortar Models.

Fig. 178. Comparison of Indicated Behavior for Circumferential Crack-Detection Strip on Cylindrical Wall of Concrete Prototype and Mortar Models.

Fig. 179. Comparison of Indicated Behavior for Meridional Crack-Detection Strip Across Haunch of Concrete Prototype and Mortar Models.

The results in Fig. 178 are for circumferential strips on the cylindrical portion of the vessels at midheight. Here the major vertical cracks passed perpendicularly under the gages. Thus large strains and even failure of the gages were expected and were obtained. The cross mark on the final curve for the second mortar model denotes complete failure of the circuit at that point. The double arrows on the final curves for the prototype and first mortar model indicate that the outputs from these strips went off-scale and the circuits exhibited erratic behavior on the subsequent and final loading.

Finally the results shown in Fig. 179 are for meridional strips spanning the haunch region where circumferential cracking occurred. Again, large strains would be expected in this region. Failure of the circuit occurred in the case of the second mortar model, and off-scale readings and subsequent erratic behavior were obtained for the prototype.

9. SUMMARY AND CONCLUSIONS

Increased consideration is being given to the use of small-scale ($\leq 1/20$) models in the design and development of prestressed-concrete reactor vessels for nuclear power plants, and reliance is being placed on more detailed finite-element analyses. To add to the understanding of small-scale PCRV models and confidence in them and to demonstrate the relative accuracy of two- and three-dimensional finite-element analyses, the Oak Ridge National Laboratory carried out a program of PCRV model design, analysis, and testing as a part of a basic program sponsored by the U.S. Atomic Energy Commission. The program had two stated purposes: (1) to investigate the suitability and accuracy of small-scale microconcrete models (or mortar models, as they are commonly called) and epoxy models for determining elastic stress distributions and cracking and failure modes of concrete vessels, and (2) to evaluate the adequacy and relative accuracy of two- and three-dimensional elastic finite-element analyses when applied to relatively complex vessel shapes requiring simplifications in arriving at the idealized structure actually analyzed.

Five models were designed, analyzed, built, and tested. The first of these was a small concrete vessel that served as the prototype for the other four. The prototype was approximately 1/10-scale compared with an actual PCRV. This is near the accepted scale-range for large models. The four models of the prototype consisted of two 1/2.75-scale microconcrete, or mortar, models; a 1/5-scale prestressed epoxy model; and a simplified, unprestressed, axisymmetric epoxy model.

The tests of the prototype vessel, as well as those of the two mortar models, began with the prestressing operation, which was followed by a 4 1/2-week time-dependent test period during which creep, shrinkage, and relaxation effects were monitored. The prototype and the mortar models were then pressurized internally to increasing pressures up to failure, which was defined by excessive and uncontrollable leakage through major structural cracks.

The highest pressure reached in the prototype, which was pressurized hydraulically, was 1130 psi. This pressure was more than twice the design pressure of 500 psi. For one of the mortar models, which was also

pressurized hydraulically, the highest pressure reached was 1390 psi, and for the other mortar model, which was pressurized pneumatically, the highest pressure reached was 965 psi. These maximum pressure levels were primarily indicative of the quality of the respective Fiberglas-reinforced epoxy liners of the models and of the pressurization systems for each. The pressures at which major cracking occurred in the walls were essentially the same for all three.

The short-term behavior of the prototype and mortar models remained essentially elastic throughout the prestressing phase and during pressure testing to levels below approximately 600 psi. As the internal pressure was increased above 600 psi, cracking apparently began in the haunch region on the inner surface, and these haunch cracks grew slowly outward with increasing pressure. This early cracking was evidenced by an increasing nonlinearity in the load-deformation response curves for the prototype and models. In the range 900 to 1000 psi, major vertical cracking began in the walls, and by the time 1100 to 1200 psi was reached these cracks were completely developed and well distributed around the vessel walls. At these higher pressures circumferential cracks also began to form in the vessel walls at midheight. When the pressure was released the cracks closed with relatively little visible indication of structural damage. There was no indication that cracking had occurred in any of the flat heads.

The short-term behavior of the two mortar models closely duplicated that of the concrete prototype. In the elastic regions the stress levels and the stress distributions were very similar. The load-deformation responses at higher pressures, the modes of cracking and the pressure levels at which they occurred, and even the number and location of the major cracks were essentially the same in prototype and models. Also there was no discernible difference in behavior of the model tested hydraulically and the one tested pneumatically.

The response to overpressures of both prototype and models was typical of that exhibited by PCRV's. Major cracking occurred gradually and was not catastrophic in any sense. It is on the basis of a progressive

type of failure behavior, such as was exhibited by these models, that sudden and unexpected failures of PCRV's are considered to be incredible.

The prestressed epoxy model and the unprestressed axisymmetric model were elastic models, and as such they were tested only within the elastic range. The prestressed model was tested with prestressing loads only and then with an internal pressure added. The axisymmetric model was subjected to an internal pressure only.

In general, the stresses determined from the prestressed epoxy model test agreed acceptably well with the elastic stresses determined from the concrete prototype test. In regions of high stress gradient, notably near the haunch, the circumferential stresses were slightly overestimated by the epoxy model. This was explained by the fact that the steep circumferential gradients were, in large part, related to the meridional gradients or discontinuities through Poisson's ratio, which was considerably larger for the epoxy than for the concrete (0.38 compared with 0.18). This is a limitation for epoxy models, but it is not considered to be a serious one.

The axisymmetric epoxy model was not truly a model of the prototype except that it did have the same principal geometrical proportions. One of the purposes of the axisymmetric model was to demonstrate the reliability and accuracy of finite-element techniques applied to an idealized, truly axisymmetric model of a thick-walled vessel. Stresses in the model were determined experimentally both by strain gages and by the scattered-light photoelasticity method. Both sets of data agreed very closely with axisymmetric finite-element predictions and thus demonstrated the accuracy of the axisymmetric finite-element approach when applied to a truly axisymmetric thick-walled structure and the potential of the scattered-light photoelasticity method of analysis.

Finite-element analyses of PCRV's have been predominantly elastic and two dimensional, and computer programs based on either axisymmetric or plane assumptions have been used. Generally an axisymmetric analysis is used for the bulk of the investigation, and a plane analysis is used to examine approximately the effects of nonaxisymmetric features such as head penetrations, circumferential prestressing arrangements, and noncircular vessel cross sections. The two-dimensional analyses performed on the

concrete prototype and mortar models and on the prestressed epoxy model followed this approach.

The application of two-dimensional analyses requires approximations and simplifications in the geometry and in the loadings. Consequently the resulting predictions are only approximations from which, it is hoped, the true behavior can be deduced with sufficient accuracy. A truly three-dimensional analysis avoids most of the approximations and simplifications necessary for two-dimensional analyses, and consequently the prototype was also analyzed by using a three-dimensional program. The resulting predictions were compared with the test results and the two-dimensional predictions.

The comparisons of the three-dimensional predictions with test results showed that the elastic stresses and deflections in the heterogeneous vessel were predicted quite well, particularly for the pressure loading. The two-dimensional axisymmetric analysis consistently overestimated both the stresses and displacements. This was easily explained in terms of the idealized structure analyzed. The axisymmetric analysis failed to predict, of course, the significant stress variation around the wall of the vessel. This variation was correctly predicted by the three-dimensional analysis. The two-dimensional plane-strain analysis of the wall cross section also predicted the variation with acceptable accuracy.

Even though the prototype vessel was more grossly nonaxisymmetric than most actual PCRV's, the axisymmetric predictions coupled with the plane predictions were probably sufficiently close to the test results for most purposes. This, together with the relative ease with which two-dimensional analyses can be applied, accounts for their widespread application to PCRV's. Nonetheless there are regions of PCRV's, such as around large penetrations and in complex anchor zones, where three-dimensional analyses are absolutely necessary for a realistic prediction of elastic behavior.

In summary, the principal observations and conclusions drawn from the model study program are that

1. The behavior of the concrete prototype was typical of that exhibited by PCRV's. The response to loading was elastic throughout the

design range. When the vessel was subjected to increasing internal pressures above the design level, cracking occurred in a gradual and progressive manner. When the internal pressure was released, the cracks closed with relatively little visible indication of structural damage.

2. The behavior of the small-scale mortar models closely duplicated that of the concrete prototype and thus pointed up their potential as models of PCRV's.

3. There was no discernible difference in behavior between the mortar model tested pneumatically and the mortar model tested hydraulically.

4. Although the prestressed epoxy model predicted the elastic stresses in the concrete prototype reasonably well, a strain-gaged epoxy model does not appear to provide any information that could not as easily be obtained from a mortar model, which is simpler and cheaper to build. An epoxy model could, however, offer advantages when used photoelastically for studies in which internal stresses are desired.

5. The acceptable agreement, in most areas, between the two-dimensional finite-element analyses and the experimental results, and the very good agreement between the more involved three-dimensional analysis and the experimental results demonstrates the effectiveness and reliability of elastic finite-element computer analyses currently used for PCRV's.

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