AMERICAN PRACTICES IN THE DESIGN OF PRESTRESSED CONCRETE CONTAINMENT STRUCTURES

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In the last two years the nuclear power industry has been successful in capturing approximately 50 percent of new electrical generating capacity. Financial and business newspapers and periodicals have provided extensive coverage of the tremendous upswing of power plant orders going to producers of uranium-fueled, water-cooled and moderated nuclear reactors.

All predictions by both private and government sources indicate an even larger share for nuclear power plants of the rapidly growing electric market. One of the major factors in making nuclear plants competitive with oil-, gas- and coal-fired plants is the capability of large plants (500-1100 MWe*) using a single reactor. The larger single reactors lead to significantly reduced unit capital cost (dollar/killowatt) when compared to plants in the range of 200-300 MWe. The civil and structural engineering features of these plants have presented some of the greatest engineering and construction challenges of recent times.

*Megawatts electrical

CONTAINMENT STRUCTURES

One of the prominent features of pressurized water, nuclear plants is the containment. Several different concepts are in use today and new ones are being evolved to meet the demands of larger plant size, metropolitan siting, and economics. The containment concept discussed herein is that of enclosing a pressurized water, nuclear reactor steam supply system within a leak-tight structure. The structure must contain the energy stored in the reactor, steam generators and associated piping and equipment (see plant cross section, Fig. 1). The structure serves as the final safeguard or barrier against release of any radioactive fission products to the plant site or environs.

Consideration of public safety is the primary criterion in providing such a barrier. Elaboration of this basic criterion results in various assumptions of improbable accidents of a very severe nature. Such design accidents assume rapid release (6-10 sec.) of the energy stored in the reactor, steam generators, and associated piping. Various pipe-size breaks are assumed and the most

Nuclear reactor power plants provide more than 50 percent of recent new electrical generating capacity. Their design and construction have presented some of the greatest challenges in engineering. Prestressed concrete provides a unique solution to the requirements for containment structures. Current practices in the United States are thoroughly detailed.

severe case of peak pressure build-up determines the controlling design criteria for the containment structure.

Review of the following typical parameters illustrate the magnitude of the problem in providing containment for an 800 MWe reactor:

Inside diameter—116 ft.
Inside height—171 ft.
Gross internal volume—1,750,000 cu. ft.
Design pressure—59 psi (8500 psf)
Design temperature—287°F

In addition to resisting internal pressure and temperature loads, the structure must also:

1. Provide protection of the reactor system and the leak-tight membrane against external missiles due to turbine generator failure or tornadoes.
2. Provide radiation shielding of the contained fission products following an accident.
3. Resist external loads due to earthquakes, hurricanes and tornadoes.

After reviewing various structural systems to provide containment structures of such a magnitude, the use of post-tensioned concrete appeared to offer the best solution because of the following advantages:

1. The membrane strength of the structure is provided by a large number of redundant biaxial ligaments (tendons) which precludes the possibility of catastrophic failure due to an initial local failure.
2. The tendons are unbonded and are therefore isolated from local stress concentrations or secondary moments. Thus, the basic membrane strength is not affected by local peak strains in the concrete.
3. During installation of the tendons, each tendon is overstressed by jacking up to 80 percent of its ultimate strength to compensate for friction losses. It is then seated at 70 percent of ultimate and during the life of the structure relaxes to 58 to 60 percent of ultimate strength. The stress change due to pressurization of the structure is in the range of 2 to 3 percent (i.e., an increase from 58 percent to 60 or 61 percent) of its ultimate strength. Since the tendons have been initially overstressed, each tendon has been proof-tested to a load approximately 33 percent higher

than it will ever see during normal service or during a hypothetical accident.

4. The concrete is maintained under compression which results in superior crack control and affords better corrosion protection for both the prestressing system and the bonded reinforcing steel used for secondary stresses, penetration reinforcement, and temperature steel.
5. Sufficient inherent strength is available in the walls to permit anchoring of the high pressure pipe systems directly to the concrete through the penetration closure sleeve or cone that forms the leak-tight barrier with the liner plate (see Fig. 6). This type of detail eliminates all expansion joints and their associated maintenance problems.
6. The use of prestressing appears to offer the greatest possibilities for providing the containment structures for the larger plants required in the near future.
7. In addition, its estimated cost is comparable to that of a reinforced concrete structure, and appreciably less than that of a welded steel vessel with a separate radiation shield.

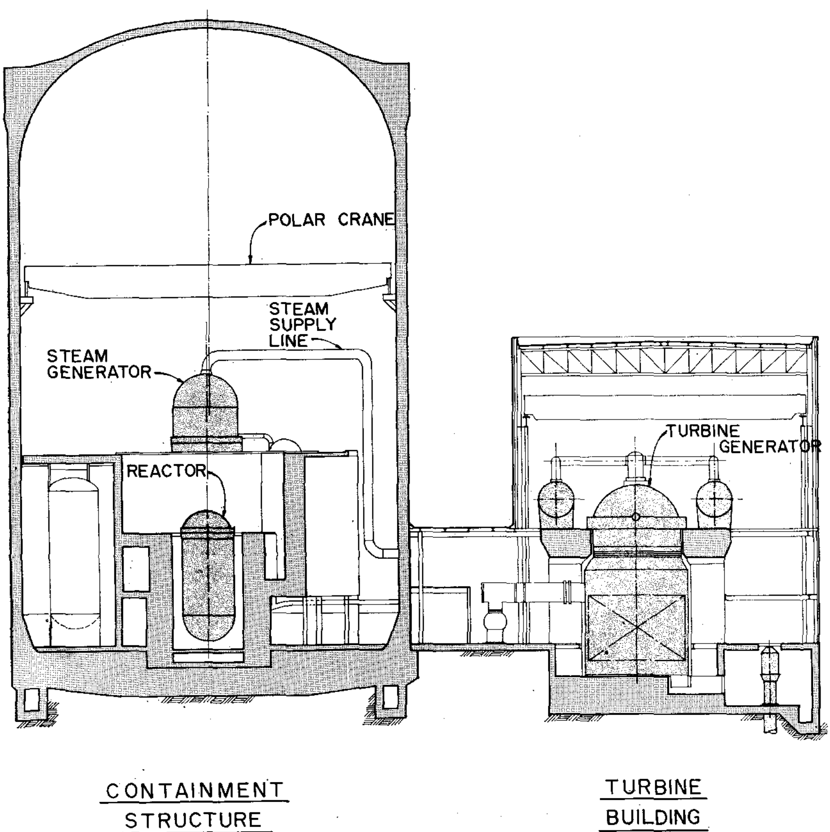

Fig. 1. Typical cross-section of pressurized water, nuclear power plant

TYPES OF STRUCTURES

A number of different types of post-tensioned containment structures are in the process of being designed and constructed in the United States. The different types are:

1. Fully post-tensioned using vertical tendons, hoop tendons anchored at vertical buttresses, and a triangular arrangement of dome tendons.
2. Fully post-tensioned, using opposing helical tendons anchored at the top and bottom of the cylinder, and a triangular arrangement of dome tendons.
3. Post-tensioned only vertically, with mechanically spliced reinforcing steel in the hoop direction and in the dome.

TYPICAL TYPE 1 CONTAINMENT STRUCTURE

The authors are involved in the design of only Type 1, and therefore this paper will be limited to that type. A typical configuration is shown in Fig. 2. Table 1 shows a listing of the various structures under design and/or construction by Bechtel Corporation. Owners and major suppliers are also shown. All field work is done by Bechtel Corporation with the exception of the erection of the liner plate. Table 1 also shows the quantities of materials required for the various structures.

Typical details of the structure are shown in Figs. 3, 4 and 5. Fig 3 shows the details of the dome and its intersection with the ring girder. The outside dimensions of the ring girder are determined primarily by providing sufficient space and edge distance for the anchorage of the vertical and dome tendons. The inside curvature of the ring girder provides a smooth transition for membrane stress flowing from the dome to the cylinder. It also results in an end closure of the structure that approximates an elliptical head with a major to minor axis ratio of $\sqrt{2}:1$.

The tendon system for the dome consists of three equal groups oriented at 120 deg. to each other as shown in Fig. 2. The tendons in each group run across the dome in parallel vertical planes. The center group is in a single layer. The other two groups are equally split into two layers each, one above and one below the center single layer group. This results in five layers of tendons and uniform eccentricity of the prestressing force at the intersection of the dome and the ring girder.

As shown in Figs. 2 and 4, the cylinder is prestressed horizontally by hoop tendons, each covering 120 deg. of circumference and anchored at the reinforced concrete vertical buttresses. Six buttresses are provided, with half of the tendons at each buttress being anchored at that buttress.

Fig. 5 shows the intersection of the cylinder and the foundation slab, and the access gallery provided at the anchorage of the vertical tendons. To date, none of the post-tensioned containments has been provided with a post-tensioned foundation slab. All foundation slabs have been provided with ASTM A 432 high strength (60,000 psi yield strength) reinforcing steel. Earlier studies indicated that the large moments and shears in the slab caused by dead load were reversed by the internal pressure load and that the membrane forces in the slab were not significant compared to the moments and shears. Thus the tendon

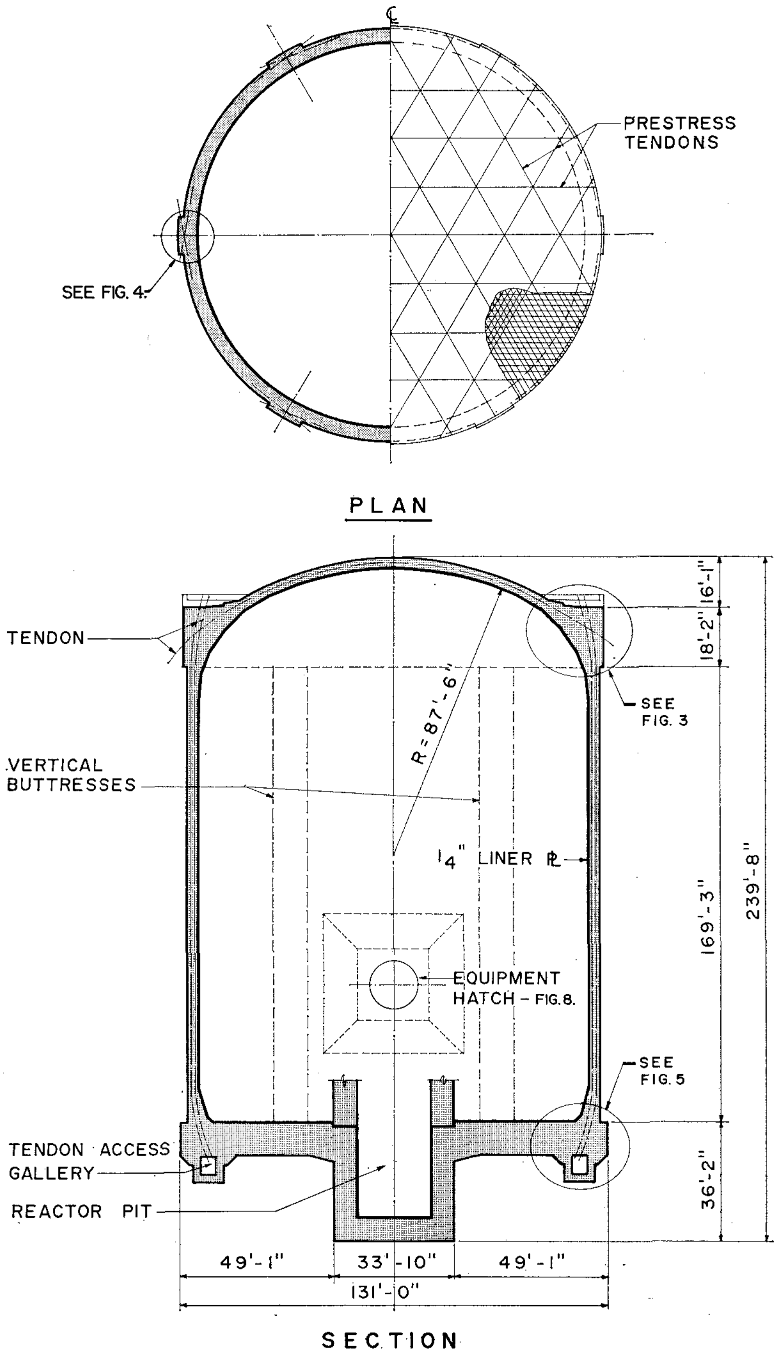

Fig. 2. Plan and section of a typical containment structure

Table 1. Current nuclear projects utilizing post-tensioned concrete containment structures

Plant	Rating MWe	Owner	Dimensions, ft.		Design Pressure psig	Tendons		Concrete cu. yd.	Liner Plate Supplier
			Inside Dia.	Height		Supplier	Tonnage		
Turkey Point No. 3	728	Fla. Power & Light Co.	116	170.5	59	Prescon	895	15,600	GATX[2]
Turkey Point No. 4	728	Fla. Power & Light Co.	116	170.5	59	Prescon	895	15,600	GATX[2]
Palisades	785	Cons. Power Co.	116	190	55	Ryerson	1050	17,000	GATX[2]
Point Beach No. 1	474	Wisconsin Mich. Power Co.	105	147	60	Ryerson	800	13,500	Graver
Point Beach No. 2	474	Wisconsin Mich. Power Co.	105	147	60	Ryerson	800	13,500	Graver
Oconee No. 1	822	Duke Power Co.	116	210	59	Prescon	1150	18,800	So. Boiler
Oconee No. 2	822	Duke Power Co.	116	210	59	Prescon	1150	18,800	So. Boiler
Oconee No. 3	822	Duke Power Co.	116	210	59	Prescon	1150	18,800	So. Boiler
1972 Nuclear Unit	800	Arkansas Power & Light Co.	(1)	(1)	(1)				
Rancho Seco Unit #1	800	Sacramento Municipal Utility Dist.	(1)	(1)	(1)				
Calvert Cliffs Nuclear Plant		Baltimore Gas & Electric Co.							
Unit #1	800		(1)	(1)	(1)				
Unit #2	800		(1)	(1)	(1)				

1. Under Preliminary Design
2. General American Transport Erectors

system could not be deflected to gain a significant lever arm to resist moments as in a typical beam where moments do not reverse. The use of post-tensioning also meant that the stressing would have to be done prior to any significant construction of the walls. These reasons, plus the higher cost of a post-tensioned slab, led to the use of a reinforced foundation slab.

PRESTRESSING TENDONS

The unbonded tendons used throughout all the structures consist of 90–¼-in. wires (ASTM A 421) anchored at both ends by BBRV button-headed type anchorages. Each 90-wire tendon is housed in a 3½ to 3¾ in. inside diameter sheathing of 24 ga., spiral wound, corrugated steel. The tendons are shop fabricated to length with one shop button-headed anchorage in place. They will be shipped to the field coiled to facilitate handling and will be pulled into position and field button-headed just prior to post-

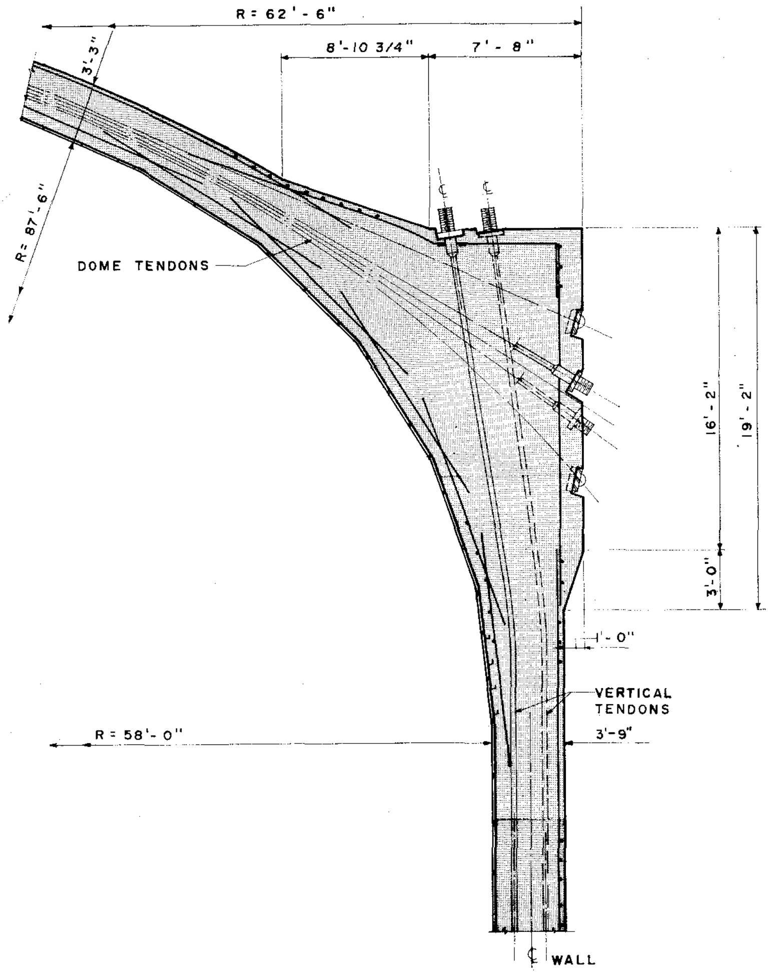

Fig. 3. Detail of dome, ring girder and cylinder transition

tensioning of the structure. All tendons will have one end field button-headed to permit the use of smaller sheathing by pulling the tendon without a stressing washer on the lead end. Shop button-heading of both ends would require a 6-in. minimum inside diameter sheathing to pass the stressing washer. The 6-in. diameter sheathing greatly increases the quantity of corrosion protection wax over that required to fill the 3¾-in diameter sheathing. The cost differential of the larger sheathing and wax, plus increased congestion with the larger sheathing, dictated the field button-heading of one end of the tendon.

Every reasonable effort will be made to protect the prestressing wire against atmospheric corrosion during its shipment and installation, and during the life of the structure. Prior to shipment the wire will be coated with a thin film of petrolatum containing rust inhibitors, such as Dearborn Chemical Company 490 R. The interior surface of the sheathing is also coated with the same material during manufacture to protect against rusting during shipment, storage, and prior to filling with a wax-like material. The sheathing filler material used for permanent corrosion protection is a modified, thixotropic, refined petroleum oil base product such as Dearborn Chemical Company No-Ox-Id CM Casing Filler. It has a proven history of serving as a filler for cased pipelines under railroads and highways. The material will be introduced into the sheathing after stressing by pumping at ambient temperature at a pressure of about 100 psi.

During the 40-year life of the structure a surveillance program will monitor the tendons for loss of prestress and corrosion. Three extra tendons are provided in the dome, hoop, and vertical groups for a total of 9 additional tendons. Lift off readings will be taken on these tendons at approximately 4-year intervals to determine loss of prestress compared to that assumed. At the same time, sample wires will be removed from 3 of the tendons, to inspect for corrosion.

LINERS

An important feature of the structure is the ¼-in. thick welded A 36

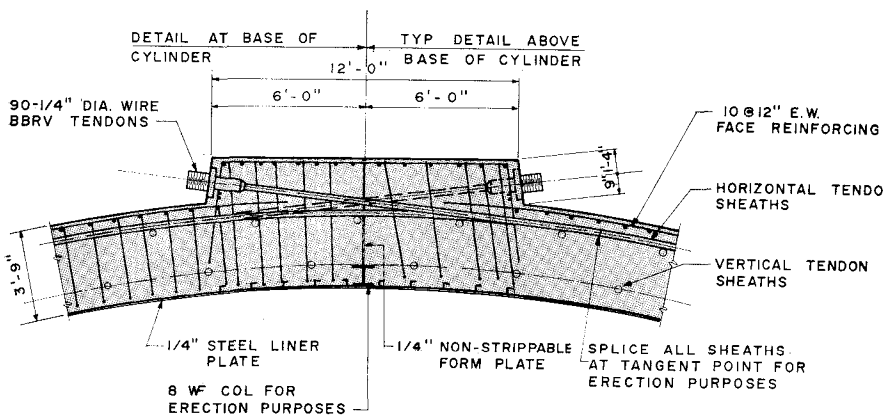

Fig. 4. Detail of vertical buttress

or A 442 steel liner plate used as a leak-tight membrane. The liner plate serves no structural purpose in the present designs. It functions strictly as a ductile liner material.

The liner is continuous and is located as shown in Figs. 2, 3, 4 and 5. The liner for the foundation slab is welded to inserts embedded in the top of the slab. It is then covered with an 18-in. thick concrete slab that provides both a wearing surface and depth for floor drains and other utilities without penetrating the liner plate. The liner plate for the cylinder and dome portion of the structure is attached to the concrete with continuous angle stiffeners. This plate also serves as the form for the cast-in-place concrete.

The basic design requirement of the liner plate is that it be restrained against significant distortion under accident conditions of high temperature. The thermal expansion of the plate, relative to the slower reacting concrete, creates compressive strains beyond yield during the design acci-

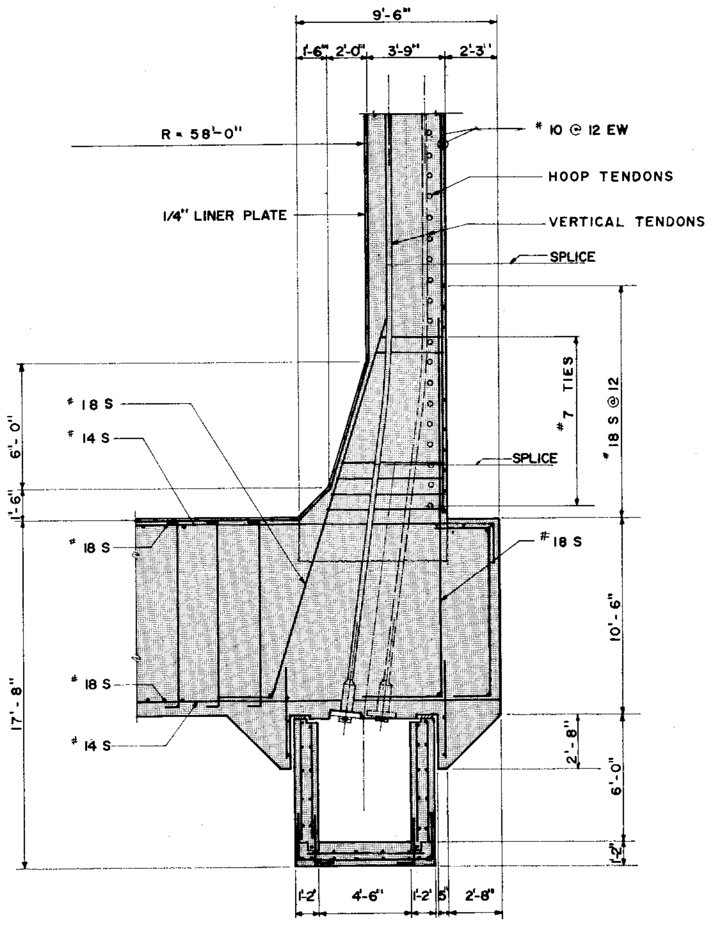

Fig. 5. Detail of cylinder and foundation slab intersection

dent. This strain is in addition to that existing due to post-tensioning plus creep. For this reason the angle attachment system is used to prevent any local buckling from developing a progressive failure of the anchorages and large uncontrolled buckling. The angles also serve as form stiffeners during concrete placement.

Quality control for the liner plate consists of the standard mill tests required by the ASTM specification, user tests similar to the mill tests, welder qualification tests, 100 percent visual inspection of the welding, 2 to 10 percent radiographic inspection of each welder's work and 100 percent vacuum box soap bubble tests of all welds. In addition, the welds in the floor plate are covered with leak chases to pressure test the welds prior to placement of the concrete floor topping. (The Point Beach structures have this detail for all liner plate weld seams).

There are numerous piping and electrical penetrations through the cylinder portion of the vessel. Typical details of each type are shown in Figs. 6 and 7. The reinforcement of the liner plate at the penetration and the pressure carrying sleeve attaching to the pipe or electrical canister are designed, fabricated and inspected in accordance with the ASME Code, Section III, Nuclear Vessels.

A significant advantage of a massive containment such as this is that all high pressure, high temperature lines may be anchored to the structure at the point of penetration. There is sufficient inherent strength to resist even large accident loads due to complete pipe rupture. The direct anchoring of the pipe to the wall by a steel plate cone and anchor bolts eliminates the use of expansion joints and potential leaks.

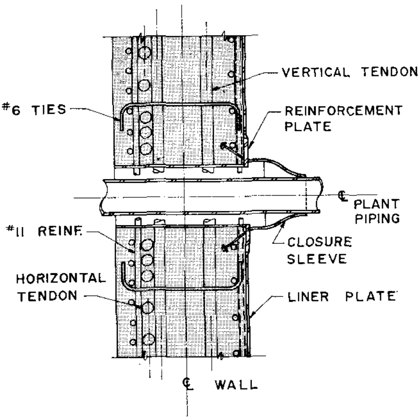

Fig. 6. Typical pipe penetration

Penetrations range in size from 2 to 3 in. in diameter up to a 14-ft. diameter opening for equipment access, Fig. 8. Personnel access is provided by an 8-ft. diameter lock designed for the accident pressure on either door. The equipment hatch is a single head and is used only during the time the reactor is shut down. The concrete around the equipment hatch is thickened to provide for the increased membrane stress, bending stresses, and the congestion of the deflected horizontal tendons. As shown in Fig. 8, all tendons are deflected around the opening in a smooth pattern to approximate the stress flow. Additional bonded reinforcing steel is used to accommodate the discontinuity stresses due to the temperature and pressure. A similar treatment is given to other smaller penetrations of the structure. Fig. 9 shows a group of penetrations and the deflected tendon pattern.

DESIGN

The significant loads that must be considered in the design of a containment structure are:

1. Dead load (D)
2. Live load, such as snow, floor loads from adjacent structures, etc., usually combined with dead load due to insignificance (L)
3. Pressure—internal positive pressure due to accident conditions, or negative pressure due to barometric pressure differential (P)
4. Temperature gradients due to normal operation and due to accident (T_O and T_A)
5. Seismic—design earthquake (E) and maximum credible earthquate (E′)
6. Tornado or hurricane winds (W)
7. Prestress load (F)

The design criteria are elaborated in more detail later in the article. In general, the design requires two approaches:

1. A "working stress design" for the conditions of the prestress load plus dead load and then that combination plus accident pressure and temperature.
2. A "yield strength design" for the above conditions when combined with earthquake or winds with all loads having the appropriate load factors as prescribed in Prestressed Design Criteria.

Both the "working stress design" method (WSD) and the "yield strength design" (YSD) assume elastic response of the structure. The major difference in the designs is that for the WSD some residual prestress remains, and stresses are limited to some fraction of yield such as ½ f_y for the reinforcing steel; for the YSD no prestress remains in the concrete, net membrane tension may exist due to thermal effects, the stresses are permitted to reach yield (reduced by the ϕ-factor similar to Chapter 15, ACI Building Code), and strains are held to essentially the proportional limit with the exception of the liner plate which is permitted to yield in both design approaches.

The significant areas of the structure for design analysis are:

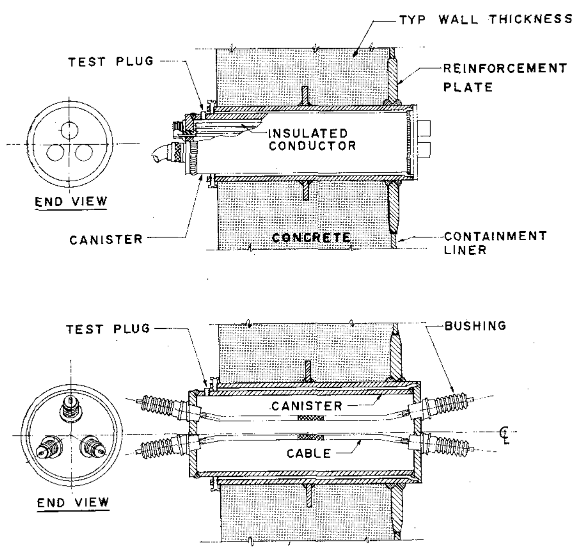

Fig. 7. Typical electrical penetrations

1. The foundation slab—this slab is prestressed against the internal pressure load by the foundation pressure. It is a slab on an elastic foundation and the interaction can significantly affect the magnitude of the dead load moments for the case of internal pressure plus the dead load.
2. The junction of the cylinder with the foundation slab.
3. The junction of the ring girder with the cylinder and the dome.
4. Items 2 and 3 are further complicated by having the stress concentrations due to the tendon anchorages superimposed upon the discontinuity stress.
5. Steady state and transient thermal gradients.
6. Vertical buttresses.
7. Penetrations.
8. Concentrated loads due to piping and equipment support.
9. Liner plate.

COMPUTER PROGRAM

The axi-symmetric loads such as dead load, pressure, temperature and prestress (1 to 5 above), are superimposed on the structure by the use of a finite element computer program. The program is based on simulating the structure and its foundation material by a series of contiguous axi-symmetric ring type elements, connected along circumferential joints (see Fig. 10). Based on energy principles, force equilibrium equations are formed in which the radial and axial displacements at the circumferential joints are the unknowns of the system. A solution of this set of equations is inherent

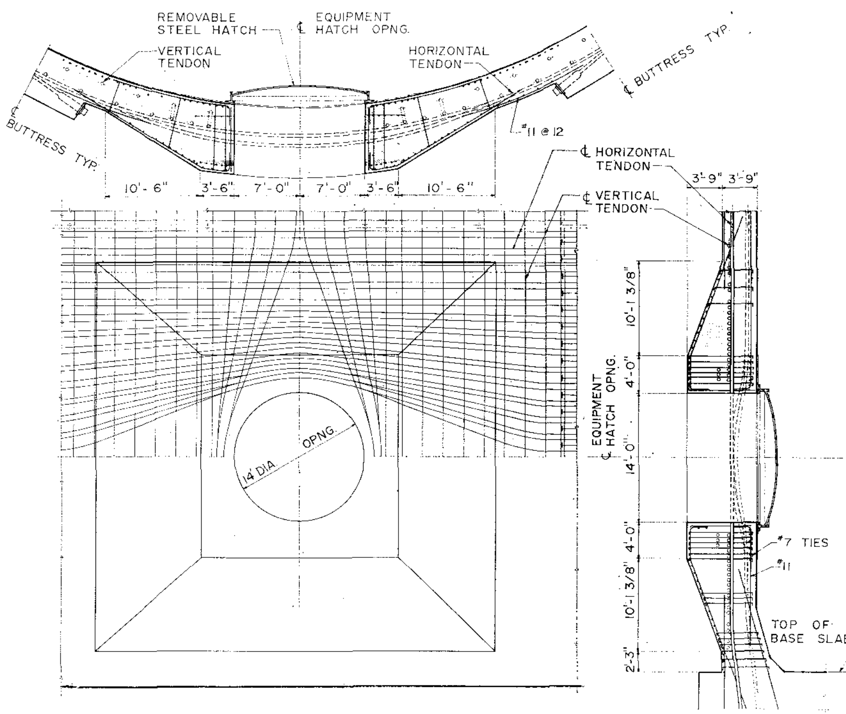

Fig. 8. Equipment hatch penetration

in the solution of the finite element system. The initial development of the computer program used in the analysis of the containment structures was conducted at the University of California at Berkeley in 1962, under a National Science Foundation grant. Since that time the program was further modified and refined by Dr. Edward L. Wilson. The validity of the specific program used in the containment structure analysis has been established by the analysis of axi-symmetric solids with known exact linear solutions.

The advantages of the finite element method, as compared to other numerical approaches, are numerous:

1. The method is completely general with respect to geometry and material properties.
2. Complex bodies composed of up to seven different materials may be represented.
3. Complex transient and steady state gradients can be analyzed by inputting the temperature of each separate element.
4. Each material may have a nonlinear stress-strain curve, that is, a yield point may be considered by the program.
5. The program will output direct stresses, principal stresses and deflection for each element of the structure.
6. Material properties may be easily varied to obtain parametric studies of the importance of such factors as modulus of elasticity of the soil.

The above advantages permit the analysis to consider items such as yielding of the liner plate, the interaction between the foundation slab and the foundation material, creep of the concrete, and geometric irregularities such as the ring girder.

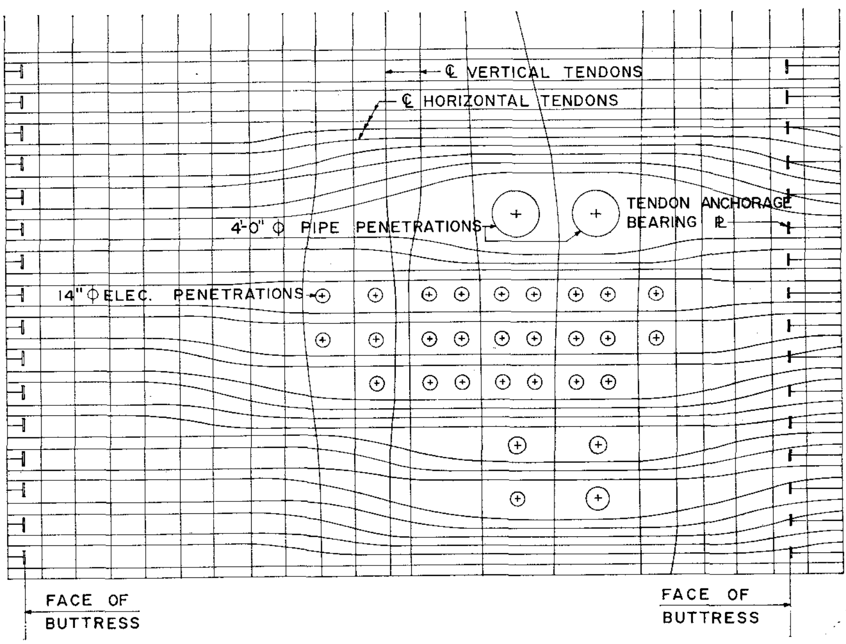

Fig. 9. Deflected tendons at penetration group

It is necessary to provide the computer with the coordinates of all nodal points of the elements describing the structural configuration. A mesh generation routine was developed to have the computer do the bulk of this work, 70 to 80 percent. A cathode ray plotting routine has been developed that prints out pictures of the element mesh or iso-stress contour lines for critical areas of the structure. Sample print outs are shown in Figs. 11 and 12. Such methods save considerable time and also serve to spot errors quickly in mesh generation and stress output.

SPECIAL DESIGN CONSIDERATIONS

For determining the anchorage zone stresses, as well as the stresses caused by hoop prestressing forces, a plane strain analysis has been carried out using the finite element method for the buttress. Because the problem is three-dimensional, plane strain analysis is a better approximation than plane stress analysis. For the analysis, a cylindrical quadrant with unit thickness and with one buttress at the center of this quadrant has been considered.

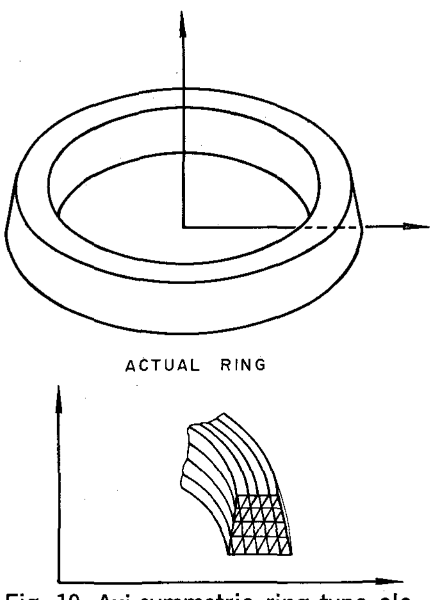

Fig. 10. Axi-symmetric ring type elements

Since the anchorages are quite close, and one above the other, they can be approximated by a vertical line load. The effects of prestress were applied as concentrated forces at the anchorages and uniformly distributed load according to the curvature of the tendons. The resulting iso-stress plots, Fig. 13, show very little tensile stress created in the buttress area and a relatively uniform compressive stress in the cylinder and the thickened buttress except immediately under the anchorage bearing plate.

Design analyses for lateral loads, such as wind or seismic, are by conventional methods once the loads are established. The loads on the containment structure caused by earthquake are determined as a result of a dynamic analysis of the structure. The dynamic analysis is made on an idealized structure of lumped masses and weightless elastic columns acting as spring restraints. The analysis is performed in two stages: the determination of the undamped natural frequencies of the structure and its mode shapes; and the modal response of these modes to the earthquake as represented by a response spectrum curve. Since the spectral response technique yields the maximum value of response for each mode, and these maxima do not occur at the same time, the response of the modes of vibration are combined on a root-mean-square basis to obtain the most probable value of maximum response.

Present designs have considered ground accelerations of from 5 to 10 percent gravity for the design earthquake and 12 to 20 percent gravity for the maximum credible earthquake. The overturning moments associated with these ground accelerations lead to the soil pressure being concentrated on as little as one half of the foundation slab. Such a soil pressure distribution becomes a major design consideration for the slab under either dead load or accident pressure conditions.

Analysis of penetrations falls into two classes—large and small. Small penetrations are considered to be those with a diameter less than about 2½ times the wall thickness. They are strengthened with additional reinforcing and no additional concrete thickness is used unless required to gain space to deflect tendons around the hole.

Larger penetrations, required for the equipment hatches, are treated somewhat differently. Analytical solutions for the determination of the state of stress in the vicinity of equipment openings are obtained from Reference 1.

The analysis of the containment vessel as a whole is first carried out without considering any openings, using the axi-symmetric finite element computer program. The containment vessel with the opening in it is then analyzed in the following steps:

1. Formulation of the boundary conditions based on the stresses obtained from the vessel analysis without the hole.
2. Evaluation of constants in the

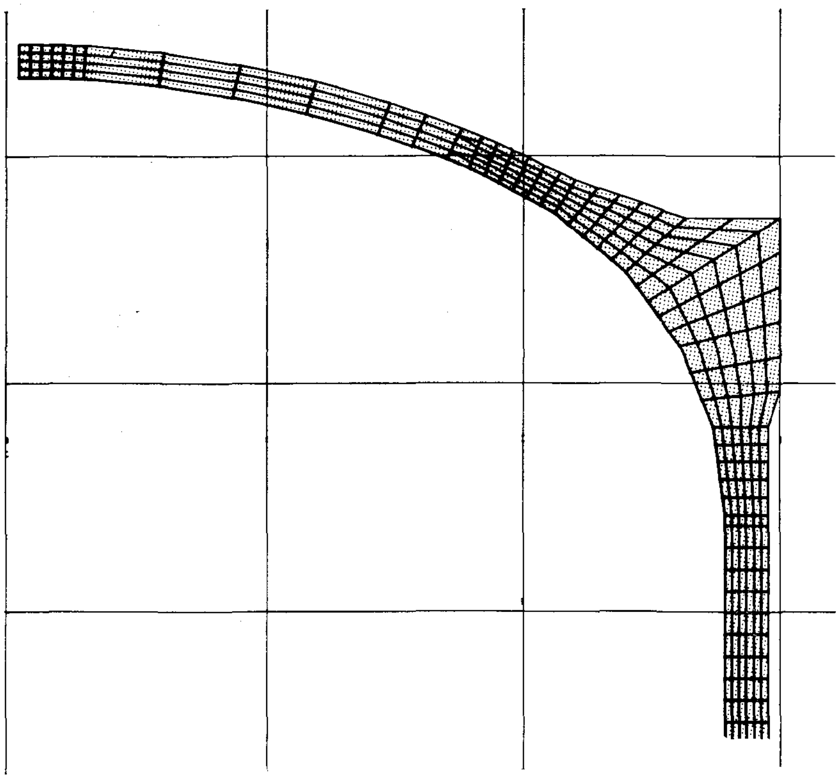

Fig. 11. Cathode ray print-out of element mesh

solutions given in Reference 1.

3. Calculation of membrane forces, moments and shears around and at the edge of the opening based on the boundary conditions from step 1.
4. The wall thickness around the opening will then be increased and reinforced to carry the higher forces, moments and shears. When increasing the thickness, cognizance is taken of the stress concentration factor due to thickening in accordance with Reference 2.
5. Finally, the design will be checked to ensure that the strength of the reinforcement provided replaces the strength removed by the opening. This check is to maintain a good degree of compatability between the general vessel shell and the area around the opening.

The effect of concentrated loads and moments are analyzed using the procedures given in Reference 3. If these loads and moments exceed the capacity of the prestressed shell, local bonded reinforcement is added to the typical pattern of temperature reinforcement. In general the analyses have shown that quite large loads due to crane brackets and thermal pipe expansion anchors can be carried with a nominal

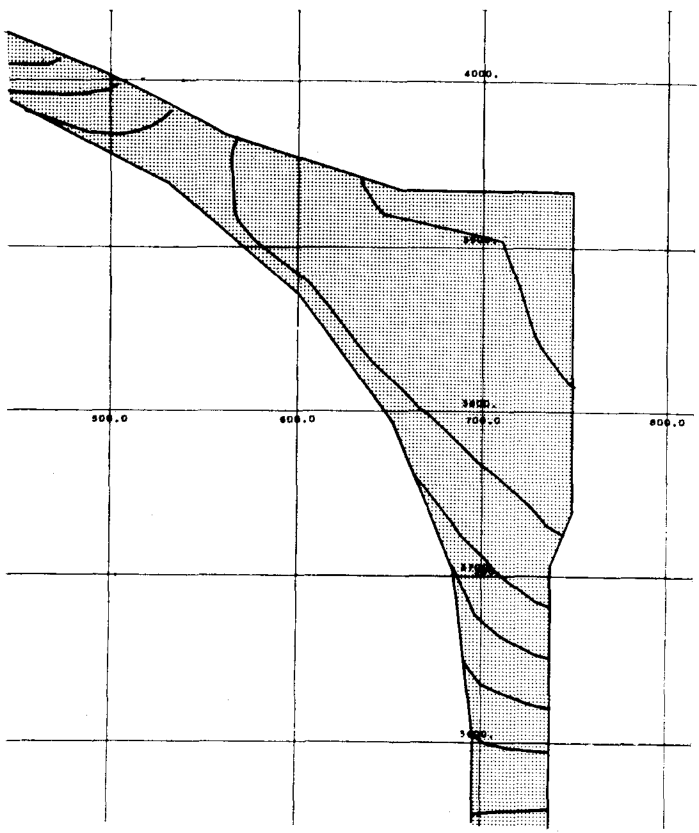

Fig. 12. Cathode ray print-out of hoop iso-stresses due to internal pressure

amount of extra local reinforcing.

STRAIN INSTRUMENTATION PROGRAM

Three of the initial structures to be constructed, Turkey Point Unit No. 3, Palisades and Oconee Unit No. 1, will be instrumented with various strain gages to monitor the performance of the structure under the following conditions:

1. Dead load effect on the foundation slab and the base of the cylinder.
2. Application of the prestressing load.
3. Overload pressure at 115 percent of design pressure.

Approximately 450 gages are affixed to the liner plate, embedded in the concrete, and attached to the reinforcement. The intent is to obtain the actual moment curves for the foundation slab, base of the cylinder, ring girder area, buttress area, equipment hatch, and areas of the dome and cylinder away from discontinuities. Structure deflection and crack propagation measurements will also be recorded. Measured values will be compared with design values for evaluation of analytical techniques and as substantiation of structural performance. A number of tendons will be furnished with load cells to measure load change during the pressure test.

All instrumentation will be abandoned following this initial structural test. The gages are not considered either necessary or reliable for long-term surveillance.

PRESTRESSED DESIGN CRITERIA

The main basis for the design of these structures (and this applies to containment structures as well as primary vessels) is the safety of the structure under extraordinary circumstances. This, of course, stands to reason by itself and has been agreed by both the ACI Committee on Reactor Vessels and the ASME Committee on Pressure Vessels.

In addition to providing the necessary safety, the proper performance of the vessel must be investigated at various stages of loading to insure satisfactory behavior. This concept of proper behavior does not imply mere adherence to an arbitrary set of allowable stresses, as is frequently done for conventional structures.

To insure safety and proper performance two criteria to be considered are:

1. The integrity of the liner plate to avoid leakage.
2. The provision of a low-strain elastic response for the structure so that its behavior may be completely predictable under the various design loadings.

In order to provide a factor of safety above and beyond the maximum credible incident, the design loads are increased by suitable factors and then equated to the yield strength of the vessel. It is noted that the yield strength rather than the ultimate strength of the vessel is used, since this is the point where the integrity of the liner may be affected. The ultimate strength in itself is of much less importance. If the strains resulting from the various factored loads meet the requirements of yield strength, the vessel is considered safe. Admittedly, yield strength is difficult to define insofar as concrete is concerned.

For the performance of the vessel under various loading conditions, the stresses and particularly the strains must be computed by refined methods, almost universally with the help of electronic computers, so that the analyses do accurately describe the actual behavior of the vessel. While it is desired to keep the various parts of the structure within working stresses, it is not imperative that these be the same allowable stresses as specified in the ACI Building Code, or some other code.

Although the ACI Code serves as a useful guide, it should be noted that the allowable stresses are based on experience and actual behavior of conventional structures, coupled with conventional methods of analyses, which may be somewhat crude. The values computed by these conventional methods generally do not represent the true stresses, but in connection with the codified allowable stresses, they form a set of fairly well defined criteria upon which economical and safe designs of conventional structures can be made.

Unconventional and sophisticated structures, like containment structures, are subjected to much more refined analyses. Such analyses include effects of temperature, shrinkage, creep, deformations and secondary stresses of all sorts, hence the stresses arrived at are much

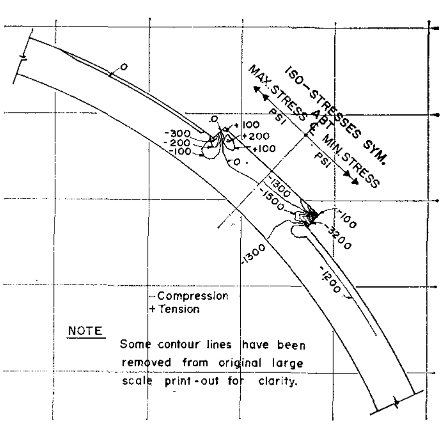

Fig. 13. Cathode ray print-out of iso-stress plot of principal stresses due to prestress

more exact and do not require the latitude of factors such as are used for conventional structures and reflected in code values.

None of this is to say that in all cases the ACI Building Code allowable stresses are not applicable, nor that they are always too low. In some cases the stresses specified for containment structures are lower than those in the ACI Code, in some higher. But in any case, the proper behavior, particularly with respect to strain, must be the guiding criterion.

When dealing with stresses in containment structures, an important distinction must be made between flexural stresses and membrane stresses. Generally speaking, membrane stresses are primary and perhaps more significant. For example, membrane tension may cause serious cracking and, under design loads, it must be limited or not permitted at all. On the other hand, flexural tensile stresses and flexural cracking may not be of great significance, especially if the tensile stress block or the amount of cracking is limited.

A more serious problem is the combination of membrane tension together with shearing stresses. This happens at points of discontinuity, or under heavy earthquake loads together with high internal pressure. The shearing stresses may be due to flexural bending, which creates a radial shear, or due to membrane shear.

A containment structure is usually designed for the following loading conditions:

1. Prior to prestressing
2. At transfer of prestress
3. Under sustained prestress
4. At design loads
5. At yield loads

For the present, not having any

code to follow, the following criteria have been applied to containment structures:

Prior to Prestressing. Under this condition the structure is simply designed as a conventionally reinforced structure. The only time the vessel is in this condition is during construction, hence allowable stresses can be higher than usually permitted by the ACI Building Code. Care should be taken, however, that the vessel does not develop large cracks which may not be closed up with prestressing.

At Transfer of Prestress. This is a condition of no internal pressure. The concrete will be under high precompression produced by the tendons. The ACI Building Code (ACI 318-63) allows a compressive stress of 0.60 f'_c for prestressed structures. In containment vessels, however, the membrane compressive stress is often limited to a smaller value, such as 0.30 f'_c, in order to limit creep and shrinkage deformations. However, compressive stresses can be increased to 0.60 f'_c when membrane and flexural stresses are combined.

For local stress concentrations in limited areas, such as around openings, occasional elastic stresses up to 0.75 f'_c may be permitted, providing reinforcement is designed for the localized conditions to insure safety under ultimate load. Membrane tension and flexural tension will be permitted provided they do not jeopardize the integrity of the liner plate. Membrane tension will be limited to $1.0\sqrt{f'_c}$. When membrane tension exists, the stress in the liner plate will be limited to 0.5 f_y. When there is flexural tension, but no membrane tension, the section will be designed in accordance with Sect. 2605 (a) of the ACI Code.

Shear criteria will be in accordance with Chapter 26 of the ACI Code as modified by the equations shown under "Yield Load", using a load factor of 1.5 for shear loads.

Under Sustained Pressure. The conditions for design and the allowable stresses are much the same as for the Transfer of Prestress conditions.

The ACI Code limits concrete compression to 0.45 f'_c under sustained prestress load. For reactor vessels, the membrane compression is usually limited to 0.30 f'_c, while compressive stress can be up to 0.60 f'_c when combined with flexural compression, thus bracketing the ACI values. With these same limits used for the transfer conditions, the stresses under sustained prestress will be reduced due to creep and are not likely to be controlling.

Design Loads. Under design loading, the structure will experience, in addition to dead load stresses, stresses and strains due to the maximum credible incident pressure in combination with lateral loads and temperature effects on liner and structure. Under the action of wind and earthquake, horizontal forces producing shear in the plane of the shell will result in diagonal or principal tensile stresses (membrane principal tension). At design loads the membrane principal tension is often controlled by limiting the allowable stress to 1.5 f'_c. For shear stress exceeding 1.5 f'_c non-prestressed reinforcement shall be provided so that the component in the direction of principal tension will carry the excess tensile force within the allowable stress of the reinforcement.

Flexural principal tension is the tension associated with secondary bending in planes perpendicular to the surface of the shell and shear stresses normal to the shell. At design loads the section will be under slight membrane compression stress. ACI Code provisions for shear as a measure of diagonal or principal tension are adequate for either of these cases. For the case of zero membrane stress up to 100 psi membrane compressive stress the provisions of ACI Building Code, Chapter 12, for reinforced concrete are generally applied. For the case of membrane compressive stress above 100 psi the provisions of Chapter 26 for prestressed concrete are applied.

Yield Load. The yielding of the structure is defined as sufficient deformation either in elongation or in shortening or bending, so as to affect the integrity of the steel liner. The yielding of non-prestressed steel in *either tension* or *compression* may not affect the integrity of the liner. On the other hand, the yielding of the prestressed tendons could be of major concern because it may mean large accompanying cracks which would impair the liner integrity. Concrete under high stress may not affect the liner integrity unless its ultimate strain is exceeded and crushing occurs.

The structure shall be checked for the factored loads and load combinations given below:

(a) $C = 1/\phi\,(1.05D+1.5P+1.0T_A+1.0F)$

(b) $C = 1/\phi\,(1.05D+1.25P+1.0T_A+1.25E+1.0F)$

(c) $C = 1/\phi\,(1.0D+1.0P+1.0T_A+1.0E'+1.0F)$

C = required capacity of the structure to resist factored loads.

ϕ = capacity reduction factor (similar to that required by ACI 318)

For other nomenclature see "Design".

The capacity of the structure (c) is defined as the upper limit of elastic behavior of the effective load-carrying structural materials. For steels (both prestressed and non-prestressed) the elastic limit is considered to be the guaranteed minimum yield given in the appropriate ASTM specifications. For concrete it is the ultimate value of shear (as a measure of diagonal tension) and bond per ACI 318-63, and the 28-day ultimate compressive strength for concrete in flexure, f'_c.

The *maximum compressive strain* in concrete should be held to that corresponding to the ultimate stress, f'_c, and a straight-line distribution from there to the neutral axis assumed. For concrete membrane compression, the yield strength can be assumed as 0.85 f'_c to allow for local irregularities. The reinforcing steel forming part of the load carrying system should be allowed to go only to yield, but not beyond.

A further definition of "yielding" in the case where mild steel is used for the liner plate, is that deformation of the structures shall not cause strains in the steel liner plate to exceed 0.005 in/in. The yielding of non-prestressed reinforcing steel will be allowed, either in tension or compression, if the above restrictions are not violated. Yielding of the prestressing tendons is not allowed under any circumstances.

Membrane tension of $3\sqrt{f'_c}$ and combined flexural tension (membrane plus flexural) of $6\sqrt{f'_c}$ is allowed in checking the yield strength of the structure. When principal flexural tension exceeds $6\sqrt{f'_c}$ due to thermal gradients through the wall, non-prestressed reinforcement should be added to resist the thermal stresses based on cracked section theory similar to that contained in ACI 505-54*. These values are chosen with a conservative margin against cracking (modulus of rupture) because of the possible reduction in tensile strength at construction joints. However, the construction joints shall be treated so that either sufficient natural bond is obtained or a suitable bonding agent shall be used to gain the required continuity.

Shear stress limits and shear reinforcement for radial shear will be in accordance with Chapter 26 of the ACI Code with the following exceptions:

1. Formula 26-12 of the code shall be replaced by

$$V_{ci} = k\, b'd\sqrt{f'_c} + M_{cr}\frac{V}{M'} + V_i$$

where

$$k = \left[1.75 - \frac{0.036}{np'} + 4.0\, np'\right]$$

but not less than 0.6 for $p' \geqq 0.003$.
for $p' < 0.003$, the value of k shall be zero.

$$M_{cr} = \frac{I}{Y}\left[6\sqrt{f'_c} + f_{pe} + f_n + f_i\right]$$

f_{pe} = compressive stress in concrete due to prestress, after all losses, applied normal to the cross-section (including the stress due to any secondary moment) at the extreme fiber of the section at which tension stresses are caused by live loads.

f_n = stress due to axial applied loads (negative for tension stress and positive for compression stress).

f_i = stress due to initial loads (including the stress due to any secondary moment) at the extreme fiber of a section at which tension stresses are caused by applied loads (negative for tension stress and positive for compression stress).

$$n = \frac{505}{\sqrt{f'_c}}$$

$$p' = \frac{A'_s}{bd}$$

V = shear at the section under consideration due to the applied loads.

M' = moment at a distance $d/2$ from the section under consideration, measured in the direction of decreasing moment, due to applied loads.

V_i = shear due to initial loads (positive when initial shear is in the same direction as the shear due to applied loads).

2. Lower limit placed by ACI Code on V_{ci} as $1.7\, b'd\sqrt{f'_c}$ will not be applied.
3. Formula 26-13 of the code shall be replaced by

$$V_{cw} = 3.5\, b'd\sqrt{f'_c}\left(\sqrt{\frac{1 + f_{pc} + f_n}{3.5\sqrt{f'_c}}}\right)$$

The term f_n is as defined above. All other notations are in accordance with Chapter 26. This formula is based on the recent tests and work done by Dr. A. H. Mattock of the University of Washington.

When the above equations show that allowable shear in concrete is zero, radial horizontal shear ties will be provided to resist all the calculated shear.

**Specifications for the Design and Construction of Reinforced Concrete Chimneys, ACI Journal, Sept. 1954.*

POTENTIAL IMPROVEMENTS

As stated previously in the article, one of the advantages of using post-tensioned concrete for containment structures was that it appeared to offer greater potential for future design improvements and cost reductions than other concepts. Some of the developments presently being considered or underway are:

1. Replacement of the steel liner plate by a plastic coating. It has been noted with great interest that the Canadians are proceeding with a vinyl-lined, post-tensioned containment structure. Although not presently scheduled for any immediate structure, Bechtel Corporation along with Duke Power Company and possibly other utilities are planning post-tensioned test facilities to subject coatings to accident pressures and temperatures. The test periods will be long enough to substantiate material properties and leak rates under actual conditions.
2. Reduction of friction to reduce the quantity of anchorages. DuPont has recently developed a plastic coating and lubricating technique for wire and strand that results in a friction factor of less than 0.01. This means that a tendon could make three full turns around the structure and have a stress loss due to friction of only 10%. The plastic coating and contained corrosion inhibitor also permit direct embedment in the concrete and should provide excellent corrosion protection.
3. The concept of partial prestressing would permit a reduction of prestressing steel and would make greater use of the temperature steel in carrying structure overloads. Sufficient prestressing would be used to maintain a slight residual membrane compression under the design accident condition. Considerable overload capacity exists beyond this point due to the combined steel area of the liner plate, the bonded reinforcing steel required for temperature gradients and secondary moments, and the prestressing tendons. This overload capacity would be used to satisfy the factored load criteria.

There are other post-tensioned concepts being applied by other designers, such as the helical-wrapped structure. Certainly other variations and refinements will appear as experience is gained both in design techniques and construction methods. It is anticipated that this experience, and developments such as those above, will permit increasingly better solutions to the structural challenges offered by modern nuclear power plant containment requirements.

REFERENCES

1. Erigen, A. C., Naghdi, A. K. and Thiel, C. C., "State of Stress in a Circular Cylindrical Shell with a Circular Hole," *Welding Research Council Bulletin*, No. 102, January 1965.
2. Levy, Samuel, McPherson, A. E. and Smith, F. C., "Reinforcement of a Small Circular Hole in a Plane Sheet Under Tension," *Journal of Applied Mechanics*, June 1948.
3. Wichman, K. R., Hopper, A. G. and Mershon, J. L., "Local Stresses in Spherical and Cylinder Shells Due to External Loadings," *Welding Research Council Bulletin*, No. 107, August 1965.

Discussion of this paper is invited. Please forward your discussion to PCI Headquarters by September 1 to permit publication in the December 1968 issue of the PCI JOURNAL.