ULTIMATE PRESSURE CAPACITY OF CANDU 6 CONTAINMENT STRUCTURES

J.P. Radulescu, L. Pradolin and J.C. Mamet
Canatom Inc.
2020 Université, Bureau 2200, Montréal, Canada H3A 2A5

1. INTRODUCTION

This paper summarizes the analytical work carried out and the results obtained when determining the ultimate pressure capacity (UPC) of the containment structures of CANDU 6 Nuclear power Plants. The purpose of the analysis work was to demonstrate that such containment structures are capable of meeting design requirements under the most severe accident conditions. For this concrete vessel subjected to internal pressure, the UPC was defined as the pressure causing through cracking in the concrete.

The present paper deals with the overall behaviour of the containment. The presence of openings, penetrations and the ultimate pressure of the airlocks were considered separately.

2. DESCRIPTION

The containment structures of CANDU 6 nuclear power plants consist mainly of a cylindrical wall supporting a ring beam, a lower dome and an upper dome. The wall is supported by a circular base slab and sometimes by a sub-base. The dimensions of the containment structures are given in Figure 1.

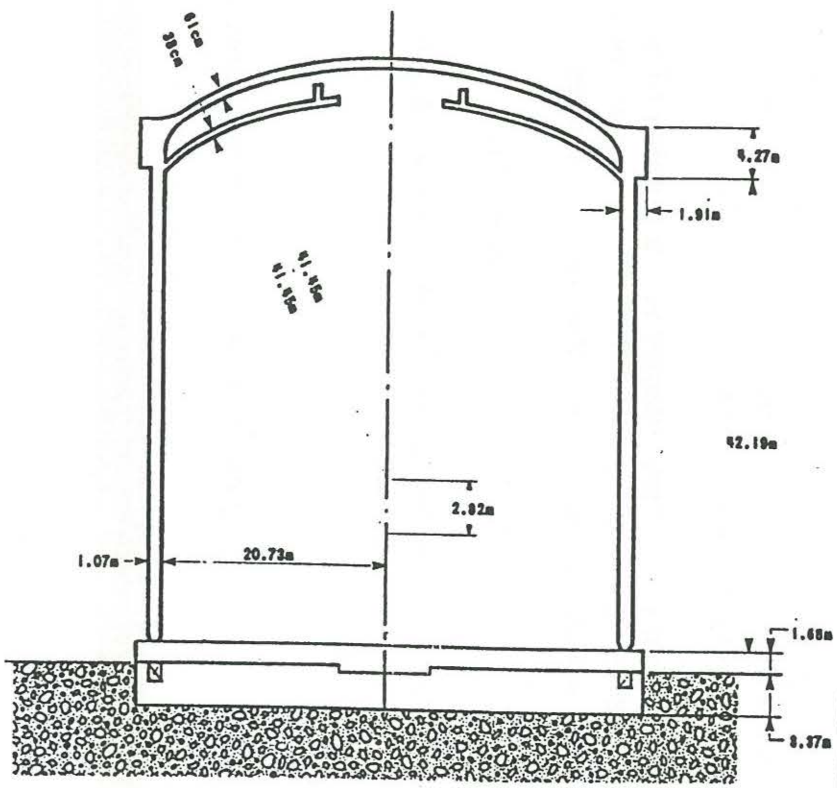
Inside the containment are located the nuclear reactor and a number of power and safety related systems as well as heavy concrete walls and slabs that are also attached to the base slab but are independent from the wall and domes.

The space between the two domes is used to store dousing water.

The containment structure, except the lower dome, is entirely prestressed. Figure 2 shows the layout of the prestressing cables.

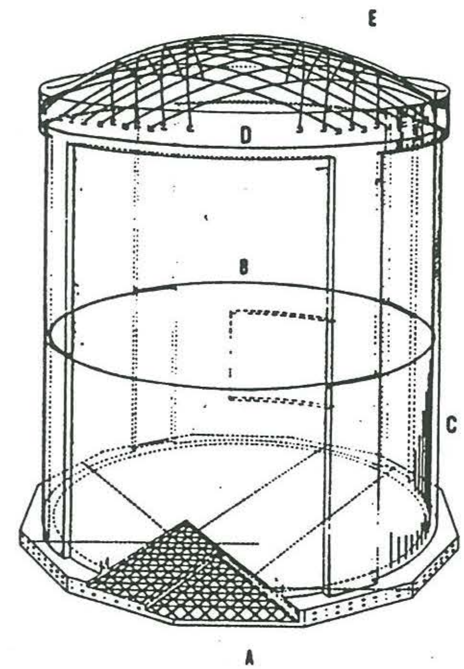
3. DESIGN CRITERIA

The Design Criteria of the Reactor building define three accident pressures as follows (1):



CONTAINMENT STRUCTURE

FIGURE 1



GENERAL ARRANGEMENT OF PRESTRESSING CABLES

FIGURE 2

- For a design accident pressure of 124 kPa (LOCA with total loss of dousing) maximum permissible stresses and strains in the containment are specified with particular attention to maximum tensile stress requirements at the surface of concrete.

- For the steam line break accident (200 kPa), it is specified that no damage to the containment shall occur, i.e. no through-wall cracking is permitted.

- During a steam line break associated with total loss of dousing (400 kPa), limited damage to the containment is allowed such that no consequential damage to reactor systems could occur.

4. ANALYSIS OF CONTAINMENT STRUCTURE

4.1 Mathematical Model

The computer program ANSYS was used to generate a three-dimensional model for the non-linear analysis of the whole reinforced concrete containment. The model represents a slice of the containment, of the sub-base and of the supporting rock limited by two vertical meridional planes forming an angle of 6 degrees between themselves (Figure 3). The model consists of three elements throughout in the circumferential direction. Symmetry boundary conditions were assumed at all the nodes in the two meridional planes. The sliding membrane between the base slab and the sub-base slab is represented in the model by allowing the bottom of the base slab to move radially, but not vertically, with respect to the sub-base slab. The internal structure is not represented. Also, as for the general linear elastic analysis, the effect of the buttresses is neglected and the openings are not included in the model. These openings have been the object of a separate detailed non-linear analysis.

The model consists of isoparametric concrete solid elements (Solid 65) defined by eight corner nodes, each node having three translational degrees of freedom.

For each part of the structure, different numbers of layers are input across the thickness in order to introduce the actual reinforcing and prestressing and, at the same time, to allow a sufficiently detailed vision of the cracking propagation to emerge from the results. In general, 5 elements are provided for the base slab, 7 elements for the perimeter wall, 4 elements for the lower dome and 5 elements for the upper dome. Figure 4 shows the mesh in the upper part of the containment. The ring beam and the hinge at the base of the perimeter wall were more refined in view of the complexity of the reinforcing and prestressing in the two joint regions.

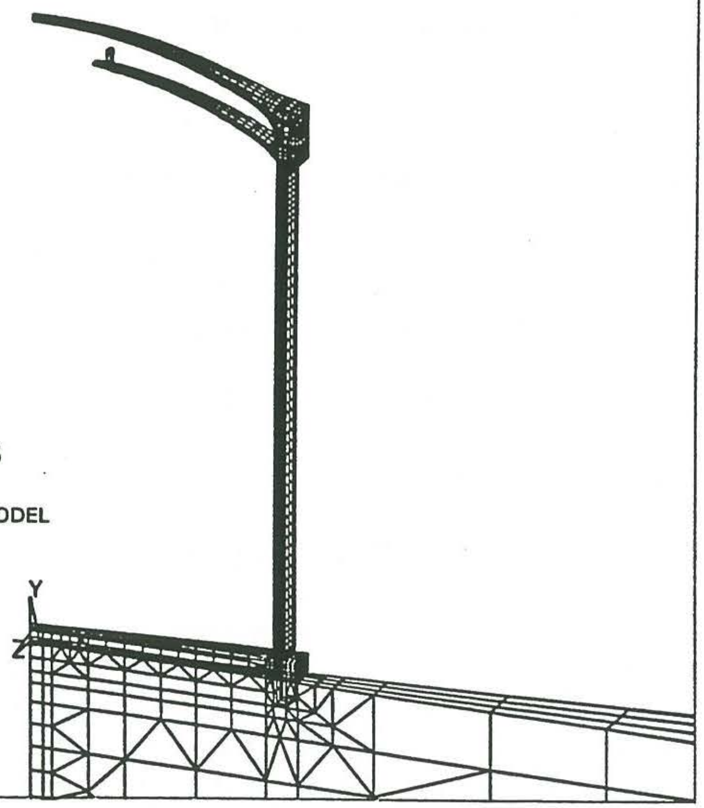


FIGURE 3

FINITE ELEMENT MODEL

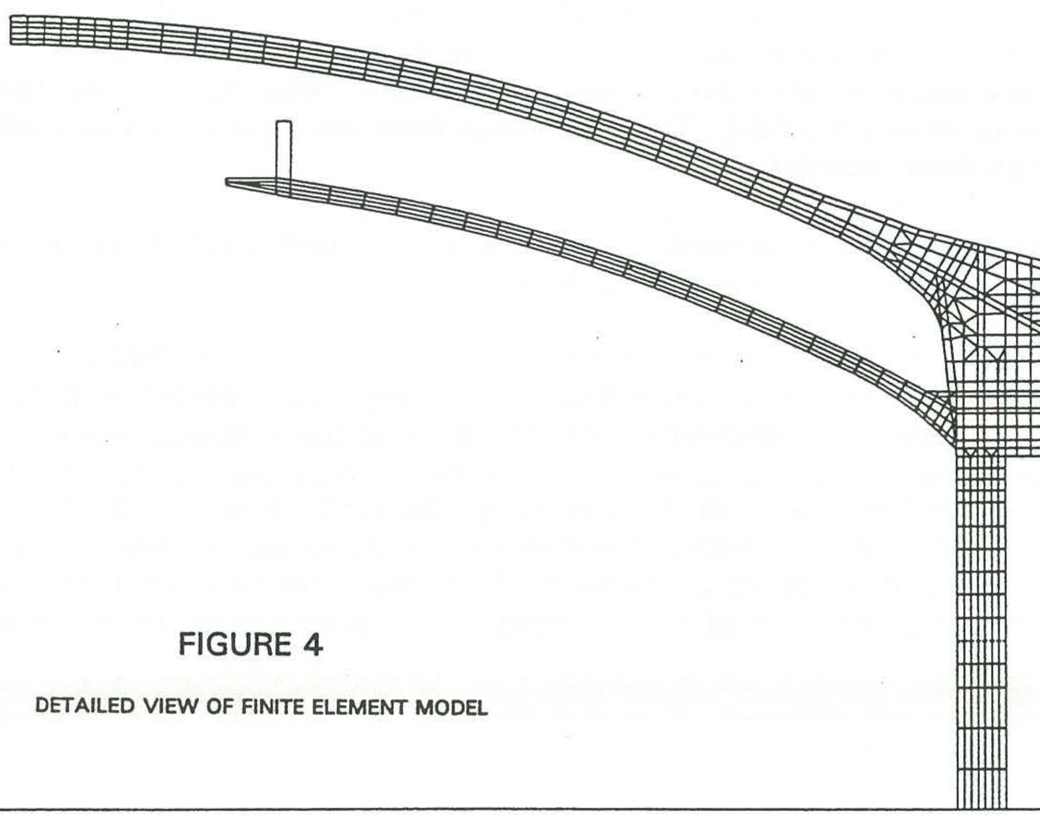


FIGURE 4

DETAILED VIEW OF FINITE ELEMENT MODEL

For each reinforced concrete element, the rebar modeling capability of ANSYS is available to describe reinforcement characteristics, including rebar material specifications, a volume ratio defined as the rebar volume divided by the total element volume, and the orientation angles. Up to three different rebars specifications can be assumed to exist in a concrete element.

In order to introduce the reinforcement present in the containment, generally, the first and the last element of the cross section of each structural component are reinforced in the meridional and hoop directions; additional elements were reinforced in the upper and lower part of the perimeter wall and in the ring beam region. Transverse reinforcement was also introduced in the lower part of the perimeter wall as well as in the ring beam region.

The upper dome prestressing system consisting of a net of 3 layers of cables running along great circles approximately 120° apart, has been converted into equivalent meridional and circumferential steel and added to the layer of elements located at mid-depth. For the base slab, the two equivalent nets of cables were introduced in the second and fourth layers of elements.

Vertical steel corresponding to the perimeter wall meridional cables is added to the central layer of elements; circumferential steel for the hoop prestressing is present in the second layer of elements from the outside face. Also, hoop steel was provided in the ring beam to take into account the upper and lower circumferential prestressing of this region.

4.2 Analytical Procedure

Failure Criterion

One of the most important characteristics of concrete is its low tensile strength which results in tensile cracking at stresses that are very low compared to the compressive strength. The tensile cracking reduces the stiffness of the concrete and is usually the major contributor to the non-linear behaviour of reinforced concrete structures. This abrupt strain softening characteristic behaviour of concrete causes sudden changes in local stress levels. The crack development and subsequent stress redistribution having such a major influence on the basic behaviour of reinforced concrete structures, an accurate modeling of the cracking behaviour of concrete is undoubtedly an important factor.

Various mathematical models based on test results have been developed for the description of the failure of three-dimensional concrete structures. As failure criterion of concrete under a multi-axial stress state, the ANSYS program uses the five points Willam-Warnke criterion associated with elasticity-based constitutive relations (2).

The shape of the five points Willam-Warnke failure surface for concrete is a function of the first (hydrostatic) invariant of the stress tensor and of the second and third invariants of the deviatoric stress tensor (3) to (5). It is described in the three-dimensional stress space by the shape of its cross-sections in the deviatoric planes and its meridians in the meridian planes.

The five parameters used to define the Willam Warnke criterion include three simple test data, the uniaxial compressive strength (f'c = 35.0 MPa), the uniaxial tensile strength (f'_t = 3.55 MPa), the equal-biaxial compressive strength (f'cc = 1.2 * f'c), and two strength under high compression, functions of f'c. The modulus of elasticity of the reinforced concrete used is 29,600 MPa and Poisson's ratio 0.15.

The reinforcing steel is assumed to follow an elastic-plastic stress-strain relationship, the Bilinear Kinematic Hardening (BKIN) option, which uses the Von Mises criterion with associated flow rule and kinematic hardening. The following reference values were used: yield strength 400.0 MPa, elastic modulus 200,00 MPa and tangent modulus 20,000 MPa.

In the hinge the tangent modulus of steel was further reduced to 2,000 MPa.

The prestressing steel is assumed to be elastic on account of the high value of its yield point, 1,678 MPa. The following properties were used: ultime strength 1,860 MPa, modulus of elasticity 193,000 MPa.

Element Formulation

Among the different approaches for crack modeling, the ANSYS program uses the smeared-cracking model. In this approach, the cracked concrete is assumed to remain a continuum, i.e. the cracks are smeared out in a continuous fashion. It is assumed that the concrete becomes orthotropic or transversely isotropic after the first cracking has occurred. Thus, after cracking has occurred, modified new tangent-material-stiffness matrices are established for the resulting orthotropic concrete material. Also, it is assumed that a crack closes when the direct stress across the crack becomes compressive.

In the tension-tension-compression state, if the failure criterion is reached for σ_1 and σ_2, cracking occurs in the planes perpendicular to those principal stresses. Further, if the failure criterion is reached for all three principal directions in a tensile-tensile-tensile state, cracks are developed in the planes perpendicular to σ_1, σ_2 and σ_3.

Shear strength reserves due to aggregate interlocking will be accounted for by retaining a positive shear modulus. Also, the tensile stress relaxation will model post-cracking behaviour of concrete in order to support tensile loads as cracking progresses.

If the material at an integration point fails in uniaxial, biaxial or triaxial compression, the material is assumed to crush at the point, crushing being defined as the complete deterioration of the structural integrity of the concrete, such that the contribution to the stiffness of an element at the integration point in question can be ignored.

Up to four material properties can be defined in the concrete element formulation used (solid element 65 in ANSYS program), one matrix material (e.g. concrete) and a maximum of three independent reinforcings in three directions. Each reinforcement, which has elastic-plastic properties has a uniaxial stiffness and is assumed to be smeared throughout the element.

4.3 Loading

The following loadings were considered major: self-weight of containment, weight of internal structure, piping loads and internal pressure loads. The containment was completely loaded before applying the internal pressure, by taking into account the construction phases and sequential build-up of dead loads and prestressing loads.

The non-linear analysis was conducted with successive internal pressure load increments. The full Newton-Raphson process was used, with an update of the global stiffness matrix at every iteration and an out-of-balance force and displacement convergence check.

First, the internal pressure loads were increased from 0.0 to 142.7 kPa (20.7 psi) to obtain a non-linear response of the containment equivalent to a pressure test. From this point on, care had to be taken to apply the load slowly so as to reproduce the correct sequence of crack formation and to avoid numerical problems which could result from non-converged load steps. Therefore, small pressure increments of only 7.0 kPa (1.0 psi) were used once a significant cracking process began. Special case was taken between 338 and 352 kPa (40 and 51 psi) and between 414 and 427 kPa (60 and 62 psi) where the changes in pressure were as small as 3.4 kPa (0.5 psi). These two pressure intervals correspond to two important changes in structural stiffness. The analysis was discontinued at a pressure of 585 kPa (82.0 psi) because of imminent convergence difficulties due to generalized cracking, significant yielding of the reinforcing steel and large displacements.

It is to be noted that the pressure rise has been estimated by AECL to take approximately 45 seconds to reach 180 kPa and 120 seconds to reach 380 kPa. Given a fundamental frequency of about 3.5 Hz for the Reactor Building, no dynamic amplification of the pressures was considered.

Temperatures were not included in the analysis. This was considered conservative because operating design temperatures cause thermal gradients, the effect of which is to induce compressive stresses in one part of the cross sections. Such stresses would increase the pressure at which through cracking would appear. Also, as cracking develops in concrete and as the structure deforms, thermal stresses tend to be released. Accidents cause temperatures to rise quickly in the Reactor Building but because of the thermal inertia of concrete, their effects will not be felt by the containment structure as quickly as that of pressure.

Long term shrinkage and creep induce low levels of stresses in the containment except at the springing of the upper dome and in the perimeter wall above the hinge. However, preliminary studies have indicated that the hoop direction in these two areas was not critical for the ultimate pressure capacity. Therefore, for the sake of clarity, these load cases were omitted.

5. FINDINGS

This section contains the salient results of the non-linear containment analysis and their interpretation.

The following definitions were used to define the cracking directions: a meridional (horizontal) crack is caused principally by meridional stresses and a hoop (vertical) crack by hoop stresses.

a. At 142.7 kPa, stresses were found to be within allowable limits both in tension and compression.

b. Except locally at the inside face of the lower dome springing and at the hinge, there are no cracks in the containment at a pressure of 200.0 kPa.

   At 241.3 kPa a meridional stress cracking process is initiated at the inside face at the springing of the upper dome, stretching rapidly over a 5° along this face. While these cracks are propagating into the thickness, they never reach the outside face where a narrow compressive region persists all along the pressurization history of the containment. Maximum compressive stresses remain acceptable.

c. There are no cracks in the wall up to a pressure of 324.0 kPa. From this level, the first meridional cracks begin to develop at the outside face below the ring beam, and up to 427.4 kPa spread over a height of about 2.0 m and penetrate up to a maximum of 43% of the thickness.

Meridional cracks, which were initiated at the inside face of the hinge at a pressure just below 206.8 kPa, penetrate over 60% of the hinge thickness at 427.4 kPa. There are no hoop cracks in the hinge region during the pressurizing history, up to 448.1 kPa. Also a superficial layer of the base slab below the hinge cracks as the hinge cracks propagate.

d. The first through-crack, initiated from the outside face and caused by the meridional stresses, occurs in the upper dome at 365.4 kPa, in a region located between 16-17° and 19° with respect to the vertical axis of symmetry. There are no hoop stress cracks in the upper dome at this stage.

Between 365.4 kPa and 413.7 kPa only an extension of the previous upper dome distribution of cracking is observed. The initial inside face meridional cracks continue also to penetrate up to 70-75% through the thickness. Also, there are no upper dome hoop cracks up to 413.7 kPa, even if the corresponding hoop stresses are becoming tensile from the vertical axis of symmetry to the cross section located at 16°.

e. By increasing the pressure from 413.7 kPa to 427.4 kPa a second critical point in the global behaviour of the containment is reached.

The upper dome cracks first due to the hoop stresses over 17° from the central axis, and a secondary quasi-generalized meridional cracking pattern is developing over the entire upper dome, excepting at the outside face of the springing.

At the same pressures, vertical through cracking is developing in the perimeter wall approximately over the entire height. Simultaneously, the outside face meridional wall cracks that appeared first are closing, but a new set will appear on the outside face between 4.5 m and 10.3 m below the ring beam.

f. From 427.4 kPa up to 441.3 kPa, as mentioned before a nearly generalized hoop cracking pattern is developing throughout the perimeter wall from 3.5 m above the base slab. This process is continuing towards the hinge region, but without interference with the cracking pattern of the hinge.

At 441.3 kPa, stresses in the outside element of the hinge reach the failure criterion in compression combined with a doubly cracked remaining section. It is to be noted that there is no yielding in the reinforcing steel in the hinge region.

g. The reinforcing steel first yields at a pressure of 448.1 kPa in the meridional direction at the inside face of the springing of the upper dome and then in the hoop direction on both sides of the perimeter wall. Further, at 482.6 kPa the reinforcing steel yields in both directions at the crown of the upper dome and in a region of the perimeter wall in the meridional direction.

h. Near 565.3 kPa, the upper dome becomes simply a tensile membrane near final failure. The reinforcing steel has yielded and only the prestressing cables contribute to the ultimate strength. More precisely a double plastic mechanism is formed with two inflection points situated approximately at 7.0° and 22.0° from the central axis.

The attached figures are showing the variation of strains (Figures 5 to 8) and displacements (Figures 9 and 10) versus the height of the perimeter wall or the position angle between a given section in the upper dome and the axis of symmetry for various increasing pressure levels.

6. COMPARISON WITH UNIVERSITY OF ALBERTA STUDY

Between 1976 and 1980, the Atomic Energy Control Board contracted the University of Alberta to conduct a similar study of the ultimate capacity of the Gentilly-2 containment.

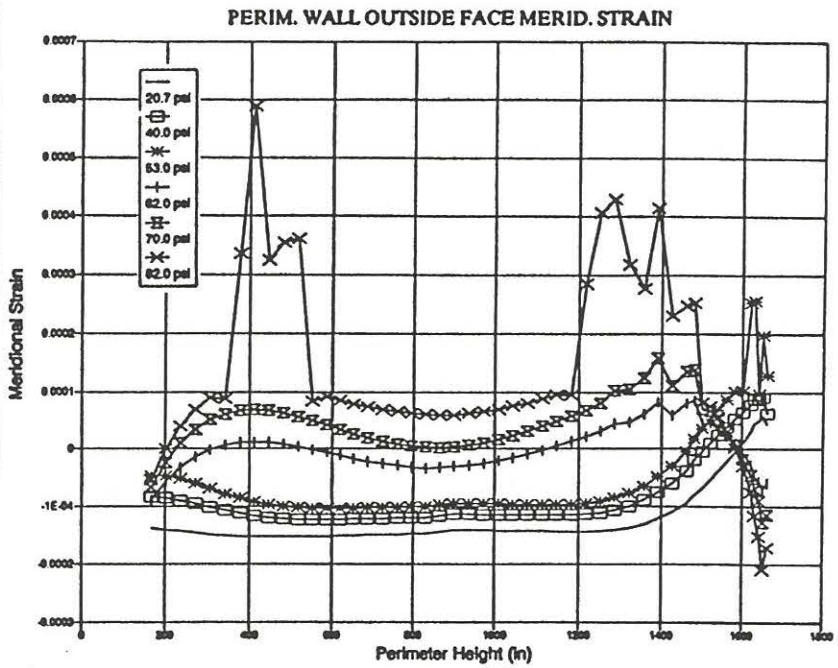
That study was both analytical and experimented and included several pioneering aspects that contributed to significant advances in the field of non-linear analysis of concrete shells (6).

An extensive review of this work was undertaken.

It is to be noted that there are significant differences between the Gentilly-2 containment and the structures built at other plants which in general have been built in regions of higher seismicity.

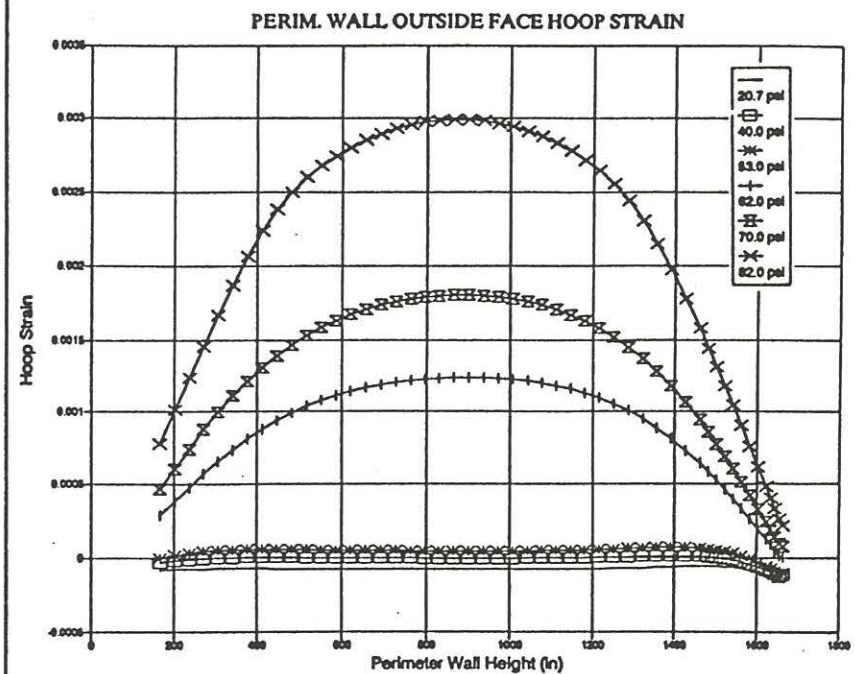
When these differences are taken into account, it was found that the results obtained for both structures compared well.

The general behaviour of the two buildings under increased internal pressure was almost identical, including the distribution and progression of cracked areas.



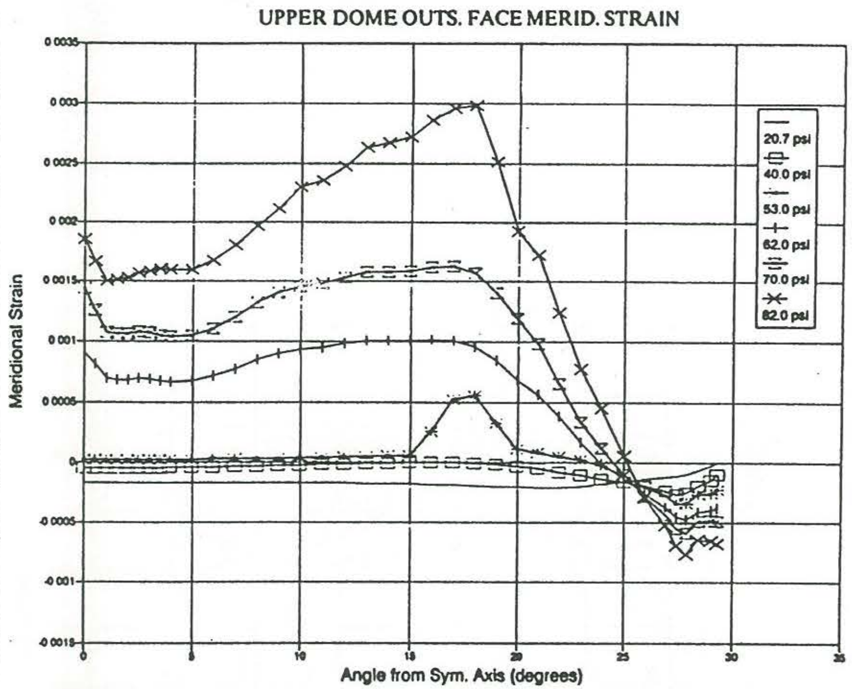
PERIMETER WALL OUTSIDE FACE MERIDIONAL STRAIN

FIGURE 5



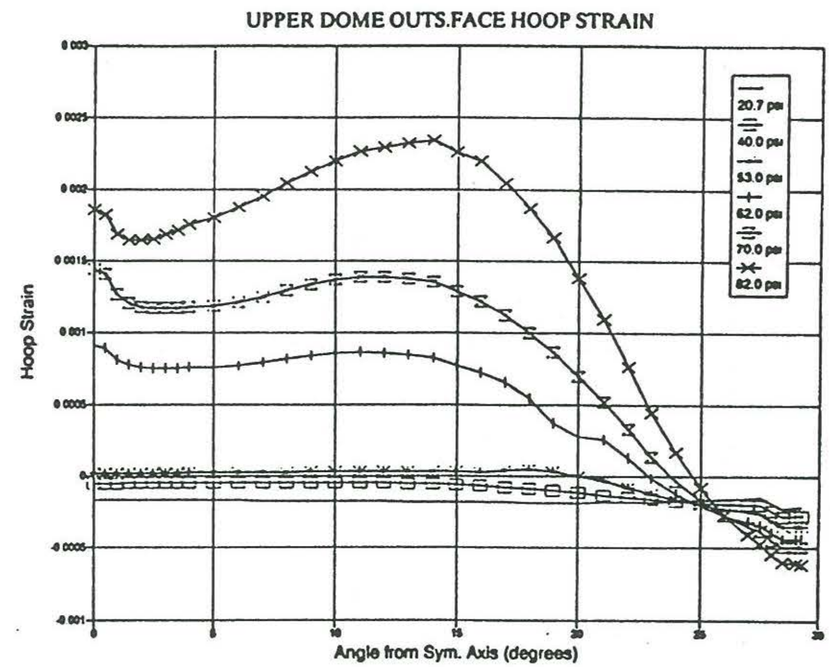
PERIMETER WALL OUTSIDE FACE HOOP STRAIN

FIGURE 6



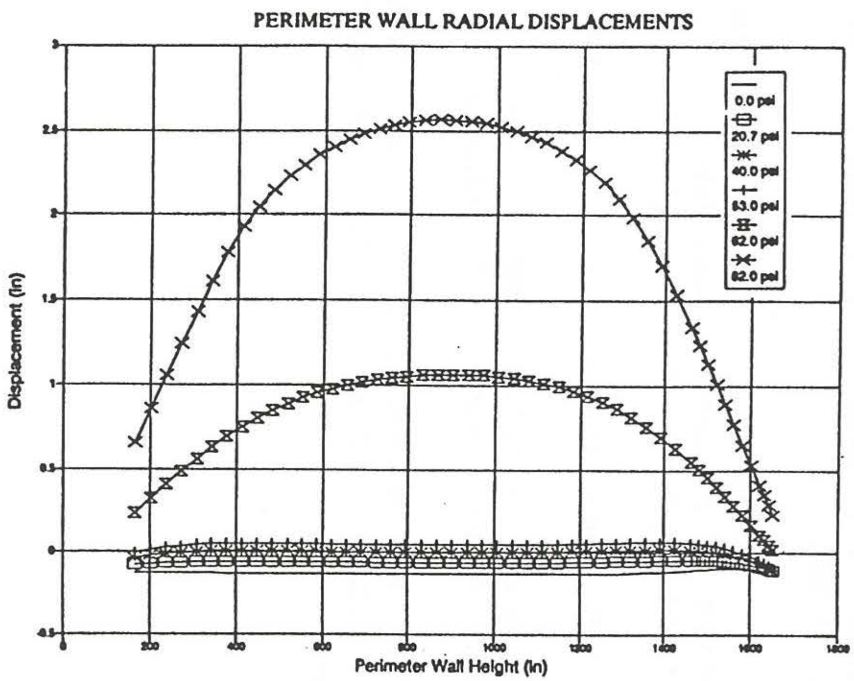
UPPER DOME OUTSIDE FACE, MERIDIONAL STRAIN

FIGURE 7



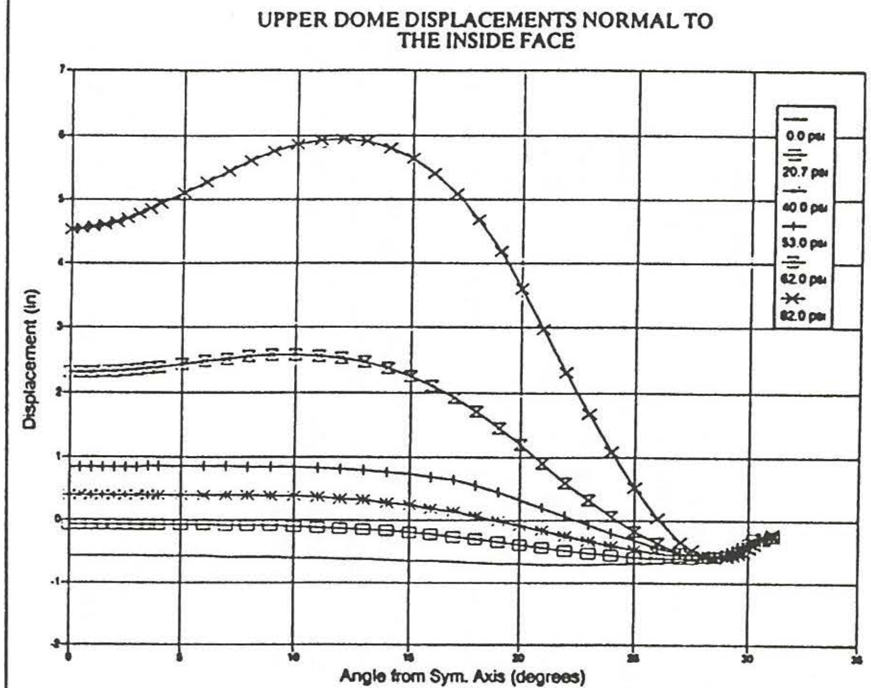
UPPER DOME OUTSIDE FACE, HOOP STRAIN

FIGURE 8



PERIMETER WALL RADIAL DISPLACEMENTS

FIGURE 9



UPPER DOME DISPLACEMENTS NORMAL TO THE INSIDE FACE

FIGURE 10

7. SUMMARY OF THE RESULTS

On the basis of the above findings, it may be concluded that the design criteria for the Reactor Building Containment are fully satisfied. At 143 kPa, stresses were found to be within allowable values both in tension and compression. This confirms the conclusions reached during the design process using linear elastic analysis procedures. Also, no through crack was observed at 200 kPa. The first through crack developed in the upper dome at a pressure of 365 kPa. This gives a factor of safety against through cracking of 365/200 = 1.82. The lower and upper domes were subjected to a pressure of 565 kPa without collapse.

There is no damage to be expected to the Reactor systems up to a pressure well above 400 kPa. Also, there is no through crack in the containment wall or in the hinge region for pressures up to the same value. No yielding of the steel rebars was detected in the hinge up to 441.3 kPa.

8. REFERENCES

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