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ULTIMATE PRESSURE CAPACITY OF THE ACR™ CONTAINMENT STRUCTURE

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ABSTRACT

The Advanced CANDU Reactor or the ACR is developed by Atomic Energy of Canada Limited (AECL) to be the next step in the evolution of the CANDU© product line. It is based on the proven CANDU technology and incorporates advanced design technologies. Two standard designs of the ACR are developed: ACR-1000 and ACR-700. The ACR containment structure is an essential element of the overall defence in depth approach to reactor safety, and is a physical barrier against the release of radioactive material to the environment. Therefore, it is important to provide a robust design with an adequate margin of safety. One of the key design requirements of the ACR containment structure is to have an ultimate pressure capacity that is at least twice the design pressure

Using standard design codes, the containment structure is expected to behave elastically at least up to 1.5 times the design pressure. Beyond this pressure level, the concrete containment structure with reinforcements and post-tension tendons behaves in a highly non-linear manner and exhibits a complex response when cracks initiate and propagate. To predict the structural non-linear responses, at least two critical features are involved. These are: the structural idealization by the geometry and material property models, and the adopted solution algorithm. Therefore, detailed idealization of the concrete structure is needed in order to accurately predict its ultimate pressure capacity.

This paper summarizes the analysis carried out to establish the ultimate pressure capacity of the ACR containment structure and to confirm that the structure meets the specified design requirements.

Keywords: AECL, CANDU, ACR, containment, pressure

1. INTRODUCTION

The ACR-700 containment structure, Figure 1, consists of reinforced concrete basemat and post-tensioned concrete cylindrical wall and dome. The wall and the dome are reinforced with steel reinforcing bars as well. A steel liner is placed on the inside surface of the basemat, the wall and the dome and is physically connected to these

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structural elements. The containment structure is conservatively designed for an internal pressure of 450 kPa. This design pressure envelops the postulated accident pressure following any design basis accident including loss of coolant accidents and main steam line breaks. One of the key design requirements of the containment structure is to have an ultimate pressure capacity that is at least twice the design pressure; i.e. 900 kPa.

The ultimate pressure capacity of the containment structure is determined using finite element analyses. A three-dimensional finite element model of the containment structure is developed such that its structural characteristics are idealized. In developing the finite element model, considerations for the details of the containment structure and the joint to the basemat, in addition to location and weight of major equipment such as the reserve water tank, are made. A plasticity concrete material model is adopted for the concrete elements of the model. Elasto-plastic plasticity models are used for the reinforcing bars and post-tension tendons. The structural response of the containment structure at different stages of a monotonically applied internal pressure is determined. The failure criteria for different materials of the containment structure are applied and the ultimate pressure capacity of the containment is evaluated.

2. FINITE ELEMENT MODEL

The analysis of the ultimate pressure capacity for the containment structure is carried out using a 5°-sector finite element model. The effects due to the presence of the openings and the buttresses in the containment structure are not considered in this model. Figure 2 shows a 3D view of the complete finite element sector model. The model consists of the containment shell, the internal structure, the basemat, the reinforcement layers, the pre-stressing tendons, and the steel liner. Generally, ten elements are defined across the thickness of containment shell. Figure 3 shows the locations of the reinforcement layers and pre-stressing tendons within the containment wall thickness.

2.1 Geometry Model

A detailed description of the geometry of the finite element model is provided below.

2.1.1 Containment Shell

SOLID65 of the ANSYS element library is used to model the concrete containment structure. SOLID65 is an 8-node 3D structural solid with three degrees of freedom at each node. This solid element is capable of cracking in tension and crushing in compression. Five elements are used along the circumferential (hoop) direction of the dome, wall and base slab. Ten elements are used across the thickness of the dome, wall and base slab. Finer mesh sizes are specified at the wall/base slab joint where stress and strain concentrations are anticipated.

The inner radius of the containment wall is 19.75 m. The thickness of the dome is 1.00 m and that of the perimeter wall is 1.20 m. A transition region of 7.5° is assumed between the perimeter wall spring line and the spherical dome. A 2.50 m thick circular disc with 25.00 m radius represents the base slab.

2.1.2 Internal Structure

SOLID45 of the ANSYS element library is used to model the concrete internal structure. SOLID45 is an 8-node 3D structural solid with three degrees of freedom at each node. The internal structure is modeled as a sector of a cylinder of equivalent mass and stiffness. The cylinder is 38.6 m high and its radius is 17.0 m. The mass of the internal structure is used to determine the density of the material used for the idealized cylinder.

2.1.3 Steel Reinforcement

SHELL43 of the ANSYS element library is used to model the reinforcement layers. SHELL43 is 4-node plastic large strain shell element with six degrees of freedom at each node. The shell elements used for the reinforcement layers and the solid elements used for the concrete share the same nodes.

Hoop and meridian reinforcements are not identical in the containment wall. Therefore, double shell elements overlying each other are modeled in the wall to represent the steel reinforcements in both directions. Both shell elements are sharing the same nodes with the concrete elements. The shell elements identified for the wall hoop reinforcement are assigned a very small value for the modulus of elasticity in the meridian direction and the shell elements identified for the meridian reinforcement are assigned a very small value for the modulus of elasticity in the hoop direction.

2.1.4 Prestressing Tendons

Two sets of pre-stressing tendons are used for the containment shell. The first set is vertical inverted U-shape tendons that are equally spaced around the circumference of the containment wall. The second set is hoop tendons that start at 2.40m above the base and end at approximately half the height of the dome. Generally, the containment shell is covered with two perpendicular layers of pre-stressing tendons except at four regions in the dome where there are three layers of pre-stressing tendons; two layers of vertical tendons and one layer of hoop tendons.

LINK8 of the ANSYS element library is used to model the hoop tendons. LINK8 is a uni-axial tension-compression 3D spar element with three degrees of freedom at each node. SHELL43 of the ANSYS element library is used to model the vertical tendons. However, an orthotropic material with very small modulus in the hoop direction is specified for the shell elements modelling the vertical tendons in the wall. In the dome, an isotropic material is defined to force the shell element modelling the vertical tendons to act in the hoop and vertical (meridian) directions.

The shell elements modeling the vertical tendons do not share the same nodes with the concrete elements but they are coincident. Coupling between the tendons nodes and the corresponding coincident concrete nodes are established in such a way that the tendons are allowed to slide in the meridian direction for both the dome and the wall. The wall hoop tendons link elements share the same nodes with the concrete elements.

2.1.5 Steel Liner

SHELL43 of the ANSYS element library is used to model the steel liner. The steel liner is 6 mm plate and is stiffened by meridian and hoop stiffeners. To account for the effect of the stiffeners, an assumed equivalent thickness of 12 mm is assigned to the liner elements. The liner element is assumed to share the same nodes with concrete.

2.2 Boundary Conditions

The boundary conditions for the finite element model are consistent with the symmetry assumption of the loads to be applied to the model. Gravity load, pre-stressing load, and the internal pressure load are considered axi-symmetric with respect to the model geometry. Therefore, symmetric boundary conditions are used at all nodes located on the two radial planes. All the nodes on the bottom surface of the base slab are restrained in the three global directions simulating a fixed condition.

2.3 Material Properties

The material models for concrete, steel reinforcements (rebars), post-tensioned pre-stressing tendons and steel liner are defined using different material models in ANSYS.

2.3.1 Concrete Material Model

The concrete model is intended for concrete behaviour under relatively monotonic loading with fairly low confining pressures. Three models represent the behaviour of the concrete: linear elastic, compressive stress-strain model, and tensile stress-strain model.

Linear Elastic Model

Whether under compression or tension, the stress-strain relationship is assumed to be linear when the concrete stress is within the elastic range. The mechanical properties for the concrete material are provided Table 1. The material properties are defined by:

- Modulus of elasticity (E),
- Poisson's ratio (ν),
- Compressive yield strength (f_y), and
- Tensile strength (f_t).

Compressive Stress-Strain Model

When concrete is loaded in compression, it initially exhibits elastic response. As the stress increases, some inelastic hardening occurs and the response of the material softens. When the principal stress components are dominantly compressive, the response of the concrete is modelled by an elastic-plastic theory using a simple form of yield surface in terms of the equivalent pressure stress and the von Mises equivalent deviatoric stress. Associated flow and isotropic hardening are used.

A uni-axial stress-strain relationship outside elastic range is assumed. In this part, the stress-strain behaviour of plain concrete in uni-axial compression outside the elastic range is specified by stress as a function of plastic strain.

In addition, a failure surface for multi-axial stresses is assumed. This surface predicts the response to occasional strain reversals and strain trajectory direction changes by the isotropic hardening of the compressive yield surface when the principal stresses are dominantly compressive. ANSYS uses the five-point William and Wranke failure criterion, Reference 1. Figure 4 shows the biaxial concrete failure criterion. The parameters defining the failure criterion are presented in Table 2. The two parameters for the compressive strengths f1 and f2 are consistent with the condition for the hydrostatic stress all nodes.

Tensile Stress-Strain Model

When a uni-axial concrete specimen is loaded in tension, it responds elastically until cracks form at the tensile strength. For multi-axial behaviour, an independent “crack detection surface” that determines if a point fails by cracking is established by the defined failure criteria, Figure 4. This model uses oriented damaged elasticity concepts to describe the reversible part of the material response after cracking failure.

2.3.2 Reinforcement Material Model

Metal elasticity and plasticity models are used to describe the behaviour of steel reinforcements. The reinforcing steel is assumed to be elasto-plastic material. Bilinear Kinematic Hardening (BKIN) material model is used in the analysis. The values used for the steel reinforcement modelling are presented in Table 3.

2.3.3 Post-Tension Tendon Material Model

The pre-stressing tendon is assumed to be elasto-plastic material. Table 3 gives the parameter values for the elasto-plastic behaviour of post-tension tendon material model.

2.3.4 Steel Liner Material Model

The liner steel is assumed to be elastic-plastic material. Bilinear Kinematic Hardening (BKIN) material model was used to model the liner steel. Values used for modelling the liner steel material are shown in Table 3. Two analyses are carried out for two different values of the steel liner yield strength: 260 and 400 kPa.

3. LOADING

The finite element model of the containment structure is subjected to three loading conditions. The loading conditions include the effects of both the weight of the reactor building, the pre-stressing load, and the internal pressure. Each of the dead load and the pre-stressing load is applied in one load step. The internal pressure load is applied to the inside surface of the model in many load increments.

3.1 Dead Load

The gravitational acceleration is applied to the whole model in the axial direction.

3.2 Pre-stressing Load

By applying a differential temperature on the elements representing the vertical and hoop tendons, the forces in the tendons are developed, and consequently, compressive stresses in the containment shell are developed.

3.3 Internal Pressure

The inside faces of the elements representing the steel liner are loaded with a uniform pressure. The internal pressure load is applied incrementally up to the maximum pressure, which is set at more than three times the design pressure. The design pressure is 450 kPa.

Since the pressure is applied in a monotonic manner in one direction only, the direct non-linear solution technique is chosen. The automatic time-stepping feature of ANSYS is invoked to march to a solution at each load increment. The convergence criteria are selected to meet the concrete cracking model requirements and to allow for its discontinuous numerical behaviour. The radial degree of freedom at a node located at half the height of the containment wall is selected to monitor the solution progress. The selection is made based on numerous test runs indicating the location of maximum deformation.

4. RESULTS

4.1 General

Figure 5 shows the deformed shape of the containment structure at different loading increments of the analysis. Figure 6 shows the principle plastic strain of the concrete elements at different loading increments. The plastic strain indicates the zones where concrete cracks have occurred.

A load-displacement curve is presented in Figure 7. The load variable represents the internal pressure and the displacement variable represents monitored radial displacement at ¼, ½, full wall height, and at approximately the dome half-height. The load-displacement curve indicates a linear response up to a pressure of 796 kPa. This pressure level is about 1.8 times the design pressure. Beyond this pressure level, the deformation abruptly increases with a marginal pressure increase indicating the structure is softening; i.e. cracking. In fact, major degradation of the structure stiffness due to major hoop cracking has already taken place. At pressure level of about 800 kPa, a very small increase in the model stiffness is observed. The analysis is stopped at a pressure level of 1300 kPa where cracking has spread over almost the whole structure.

The analysis proceeded to a pressure level that is high enough to cause yielding in the steel liner and rebar and also in the wall hoop tendons in addition to the full structural degradation of the concrete containment. Table 4 summarizes the results of the ultimate pressure capacity analysis. It lists the pressure levels at which important stages in the containment structure behaviour occur during the internal pressure load history, such as the first cracking of pre-stressed concrete containment, first yielding of rebar, pre-stressing tendons, and/or steel liner. The ratios of these pressure levels to the containment design pressure are provided as well.

The containment structure behaves generally within the elastic range for a pressure exceeding 1.5 times the design pressure. No yielding in the steel liner or in the steel reinforcement is expected before the internal pressure load exceeds twice the design pressure. The ultimate pressure capacity is expected to exceed 1300 kPa, which is about 2.89 times the design pressure.

4.2 Non-linear Behaviour of Structural Elements

The first crack in the model occurs at a pressure level of 635 kPa and is located at the inside surface of the wall at the wall/basemat joint. Only one element is cracked at this pressure level and this crack takes place in the radial, meridian and hoop directions.

At a pressure level of 796 kPa, cracking starts at the wall mid-height. The cracking in this region occurs across the whole thickness of wall and takes place mostly in the hoop direction. The first crack in the dome takes place at pressure level of 796 kPa, as well, and is located at the inside surface at spring line. Few dome elements are cracked at this location and this cracking takes place in the radial, meridian and hoop directions.

At pressure level of 1260 kPa, all elements of the wall are cracked in the radial, meridian and hoop directions. At the same pressure level, the cracking in the dome extends beyond the spring line location towards the dome apex.

By the end of the analysis, at pressure level of 1300 kPa, in addition to fully cracked elements of the wall, all dome elements are cracked in the radial, meridian and hoop directions.

The steel liner started to yield in the hoop direction at pressure of 980 kPa at a point 10.60m above the base slab. The stress-strain relation history for the hoop stresses in the wall steel liner at that location is shown in Figure 8.

The wall hoop tendons started to yield at a pressure of 1275 kPa. At a pressure of 1300 kPa, the maximum elastic and plastic strain equal to 0.86% and 0.139%, respectively. The stress-strain relation for the wall hoop tendon at the wall mid-height throughout the loading history is shown in Figure 9. The vertical tendons and the dome hoop tendons did not reach yielding by the end of the analysis. At a pressure of 1300 kPa, the maximum elastic strain in the vertical tendons is still at 0.593%.

The inside face layer of the wall hoop steel reinforcement started to yield at pressure level of 1122 kPa at a point 10.60m above the base slab. The outside face layer started to yield at a pressure level of 1138 kPa. The inside layer of the wall meridian steel reinforcement started to yield at a pressure level of 1218 kPa at very localized region at the wall/basemat joint.

4.3 Discussion

The failure of the containment structure is defined and monitored when strain responses of individual structural element (i.e. liner, reinforcement and tendons) exceed specified strain limits. These strain limits are less than the nominal and test ultimate strain for the structural elements of the containment. Table 5 summarizes the specified strain limits, the attained strain responses at maximum pressure load, and the calculated capacity factor (i.e., strain limit/strain response due to pressure).

The yielding of the hoop tendons in the containment wall governs the ultimate pressure capacity analysis for the containment. This failure mode is common and is reported as the governing failure mode for past analyses performed for pre-stressed concrete containments structures, References 2 and 3.

In 2000, a ¼ scale PCCV model was tested in Sandia National Laboratories to assess the over-pressurization of pre-stressed concrete containments. Pre- and post-test analyses were conducted to predict and investigate the containment behaviour during the test, References 4, 5 and 6. A comparison between the reported containment behaviour of the PCCV model and the results of the ACR-700 ultimate pressure capacity analysis is made. Figure 10 illustrates the steel liner strain at the containment wall mid height during the internal pressure load history normalized to the design pressure for both the ACR-700 and the ¼ scale PCCV. The ACR-700 steel liner behaviour is reported, in Figure 10, for the two analyses carried out based on the two values of steel liner yield; i.e. 260 and 400 kPa. It should be mentioned that the yield strength of the steel liner for the PCCV model and analysis is 400 kPa.

A very close agreement is found between the predicted behaviour of the ACR-700 containment and that established for the PCCV model under internal pressure. The close agreement is found primarily in the plastic behaviour of the steel liner, steel reinforcement and the hoop tendons. A sudden degradation of the ACR-700 containment structure due to the concrete hoop cracking is observed while a rather smoother transition is observed in the PCCV case. This discrepancy is attributed to the different concrete material constitutive models used in the two cases. Despite the different concrete models, the overall behaviour of the ACR-700 containment closely matches that of the PCCV.

The ACR-1000 containment structure is similar to the ACR-700 containment structure in concept. The same structural design requirements and standards are applied to both standard designs. Complying with the same requirements in the containment structure design and detailing, the structural response of the ACR-1000 containment to internal pressure is expected to be similar to that of the ACR-700; i.e. the ultimate pressure capacity would be about 3 times the design pressure.

5. SUMMARY AND CONCLUSIONS

In this study, a detailed non-linear analysis of the ACR-700 containment structure is performed to evaluate its ultimate pressure capacity. A sector finite element model of the containment structure is developed including steel reinforcement, pre-stressing steel, and the steel liner. Provisions are made to account for the interaction with the concrete internal structures. Non-linear static analyses are carried out with applicable loadings taking into account the sequence of load application. Analyses are performed for fixed base condition. The containment is loaded with internal pressure in steps to capture the significant changes of the containment stiffness due to concrete cracking, yielding of reinforcement, pre-stressing tendons, and the steel liner.

The analyses conclude that the containment can sustain an internal pressure 1300 kPa, which is about 2.89 times the design pressure. Therefore, the design of the ACR-700 containment structure provides for a significant margin of safety against failure due to any potential accident within the containment. This margin provides for significant protection level to the public. The structural behavior of the ACR-1000 containment to internal pressure is expected to be similar to that of the ACR-700.

6. REFERENCES

1 Willam, K. and Warnke, E.P. "Constitutive Model for the Triaxial Behavior of Concrete", Colloq. Concrete Structures Subjected to Triaxial Stresses, ISMES, Bergamo, May 17-18, 1974, publ. in Proc. IABSE Report 19, Zurich, 1975, III-I, pp. 1-30.

2 Krishnaswamy C.N., Namperumal R., Al-Dabbagh A., 1983, Ultimate Internal Pressure Capacity of Concrete Containment structures, 7th Int. Conf. on Structural Mechanics in Reactor Technology (SMiRT-7), Chicago, vol. J3/6, p. 139-146.

3 Amin, M., Eberhardt, A.C., Erler, B.A., 1993, Design Considerations For Concrete Containments Under Severe Accident Loads, Nuclear Engineering and Design, vol. 145(3), p. 331-338.

4 Luk, V.K., 2000, Pretest Round Robin Analysis of a Pre-stressed Concrete Containment Vessel Model, NUREG/CR-6678.

5 Dameron, R.A. et al., 2003, Posttest Analysis of the NUPEC/NRC 1:4 Scale Prestressed Concrete Containment Vessel Model, NUREG/CR-6809.

6 Hessheimer, M.F. et al., 2003, Overpressurization Test of a 1:4-Scale Prestressed Concrete Containment Vessel Model, NUREG/CR-6810.

Appendix of Tables

Table 1: Concrete Material Properties

Property	Value
Mass Density (ton/m^3)	2.4
Modulus of Elasticity (MPa)	28000
Poisson Ratio	0.15
Ultimate compressive strength (MPa)	35
Tensile strength (MPa)	3.55
Shear Retention/Crack Opened	0.1
Shear Retention/Crack Closed	0.16

Table 2: Concrete Material Model and Failure Criterion

Parameter	Description	Value
f_c	Ultimate uni-axial compressive strength	35 MPa
f_t	Ultimate uni-axial tensile strength	$0.6\sqrt{f_c}$
f_{cb}	Ultimate biaxial compressive strength	1.20 f_c
f_1	Ultimate compressive strength (biaxial + ambient hydrostatic stress state)	1.45 f_c
f_2	Ultimate compressive strength (uni-axial + ambient hydrostatic stress state)	1.725 f_c

Table 3: Material Properties of Reinforcing Steel, Tendons and Steel Liner

Property	Steel Rebars	Tendons	Steel Liner
Mass Density (ton/m^3)	7.85	7.85	7.85
Modulus of Elasticity (MPa)	200000	195000	200000
Tangent Modulus (MPa)	2000		2000
Poisson Ratio	0.3	0.3	0.3
Yield strength (MPa)	400	1670	260 and 400

Table 4: Pressure Levels (kPa) at Important Stages

Event	Pressure	Pressure/Design Pressure
1st base slab radial cracking	635	1.41
1st wall meridian cracking	645	1.43
1st wall hoop cracking	796	1.77
1st yield of hoop rebar in wall	1122	2.49
1st yield of meridian rebar at wall/base slab joint	1218	2.71
1st yielding in liner	980	2.18
1st yielding of hoop tendons in wall	1275	2.83
Last converged solution pressure	1302	2.89

Table 5: Capacity Factors for Major Elements in Containment Structure

Parameter	Acceptance Criteria	Attained Strain	Capacity Factor
Hoop Tendons	Yield strain = 0.01	0.0099	1.01
Vertical Tendons	Yield strain = 0.01	0.0065	1.54
Steel Liner	12 × yield strain = 0.016	0.0048	3.33
Hoop Reinforcement	5 × yield strain = 0.01	0.0047	2.13
Meridian Reinforcement	5 × yield strain = 0.01	0.0038	2.63

Appendix of Figures

Figure 1: ACR-700 Containment Structure

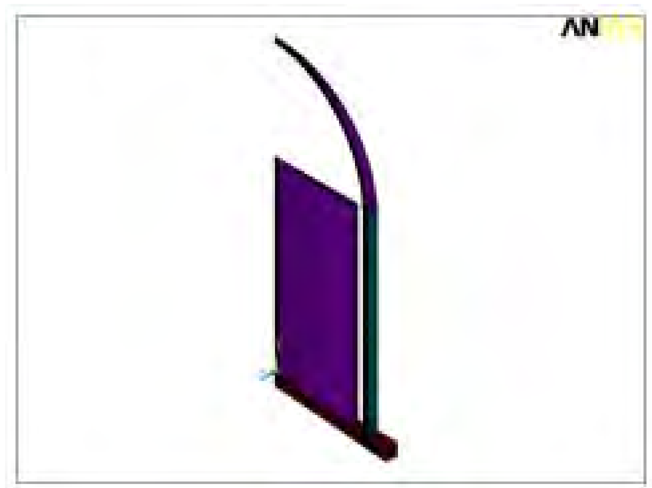


Figure 2: Reactor Building Sector Model

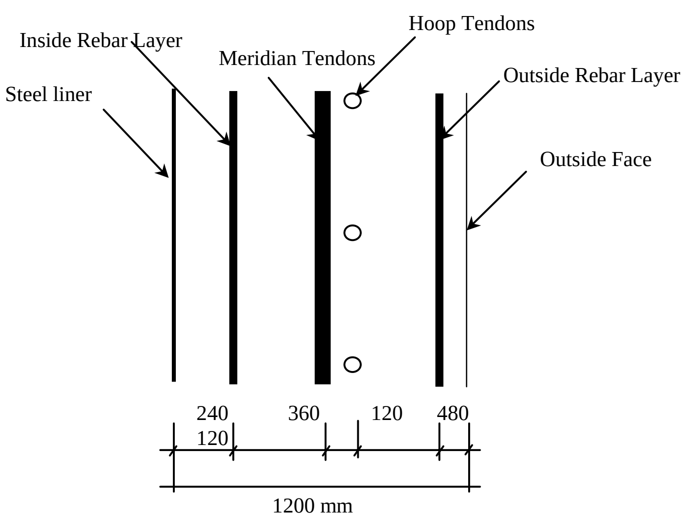


Figure 3: Layers Across Thickness

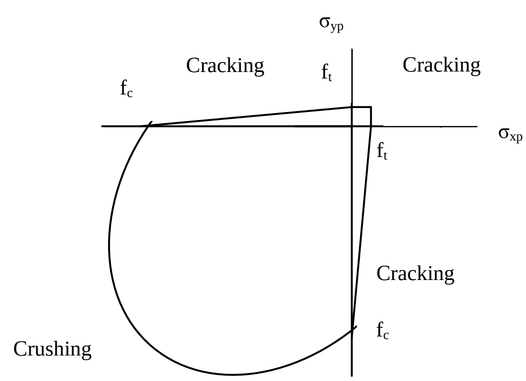
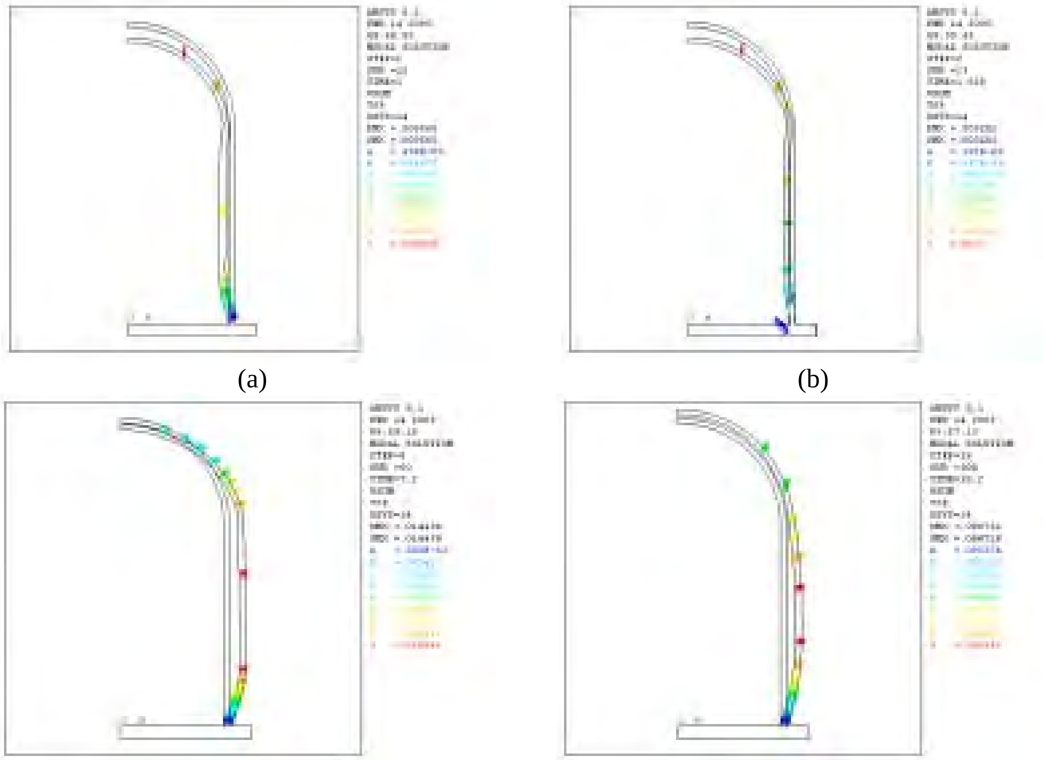
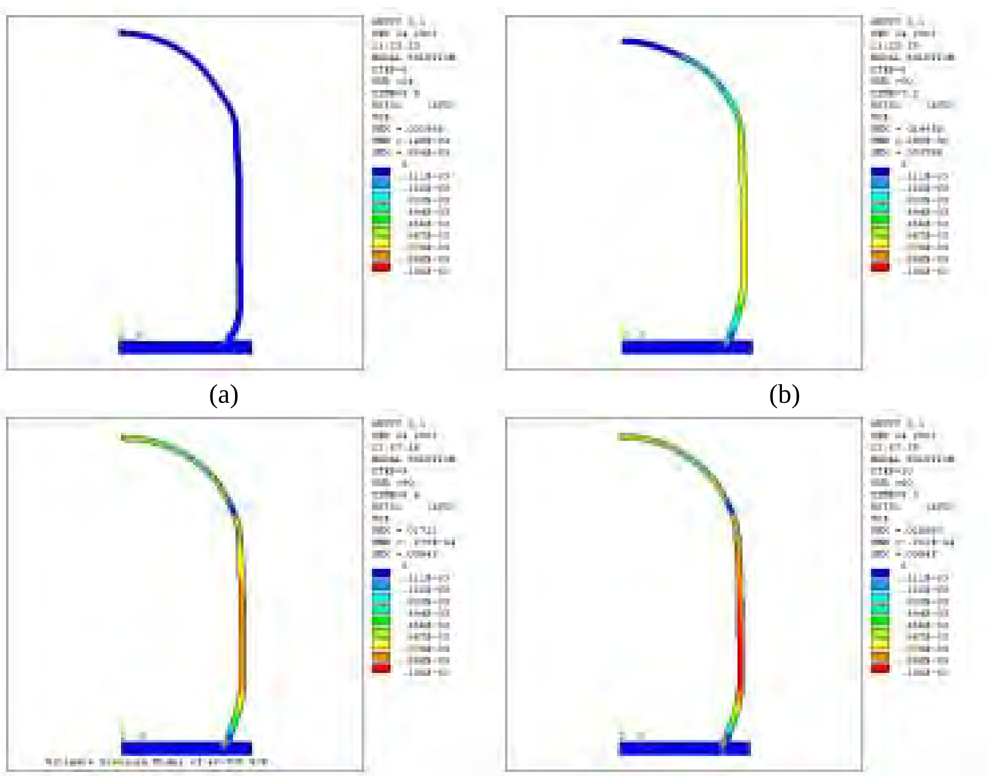


Figure 4: Biaxial Concrete Failure Criterion



(a) (b) (c) (d)

Figure 5: Deformation at pressures (a) 0 kPa, (b) 450 kPa, (c) 796 kPa, and (d) 1300 kPa



(a) (b) (c) (d)

Figure 6: Plastic strain at (a) first cracking, (b) hoop cracking, (c) 840 kPa, and (d) 865 kPa

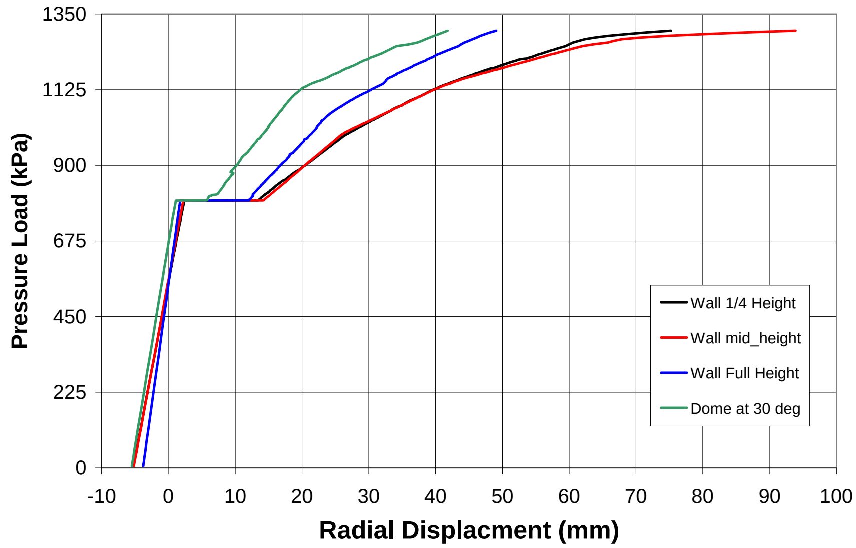


Figure 7: Load-Displacement Histories at Different Locations

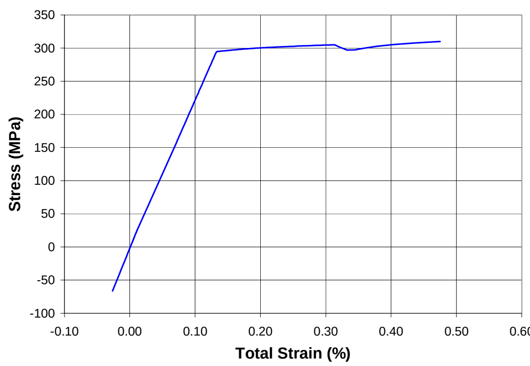


Figure 8: Steel Liner Hoop Response

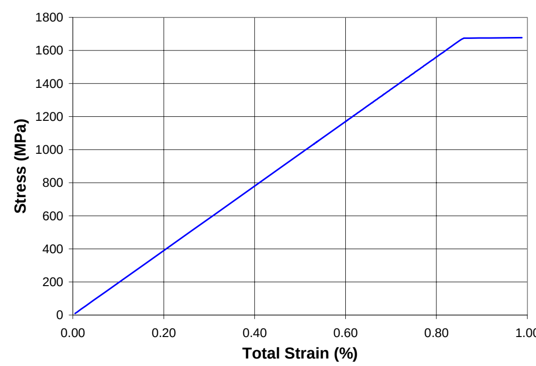


Figure 9: Hoop-Tendons Meridian Response