Response Simulation of 1/4 Scale Prestressed Concrete Containment Vessel Model

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ABSTRACT

Under the ultimate internal pressure the nuclear reactor containment exhibits non-symmetrical and non-linear behavior. This is due to the structure having non-symmetric factors such as buttresses, penetrations and tendon layout in the dome. Therefore, the behavior of the structure under ultimate pressure should be assessed with a three dimensional model which considers the non-symmetry. In this study, an assessment method using the 3D global model is proposed. In order to simulate the real behavior of the prestressed concrete containment vessel, two numerical models - an axisymmetric model and a three-dimension model - are refined by comparison of the analysis results and with existing research results. In the analysis of prestressed concrete containment, exhibiting highly nonlinear behavior, both Menetrey-Willam' s concrete failure criteria and modified Drucker-Prager' s concrete failure criteria are introduced to the finite element models. In accordance with Smith' s research results, non-associated plastic flow rules are applied for the concrete material model.

INTRODUCTION

Prediction of the response of the prestressed concrete containment vessel (PCCV) is very important for the safety of nuclear power plant. Also, there are growing discussions on the performance of the containment under severe accident pressure. Hence, a more realistic evaluation of the PCCV behavior due to postulated overpressure is necessary to maintain containment integrity. In this paper, a prediction of the response of the the Sandia National Laboratories 1:4 scale PCCV[1] mode was made through various type of numerical modeling taking in account the appropriate non-linearity for each material. For the nonlinear finite element analysis, the PCCV is idealized as an axisymmetric model and a three dimensional global model. In order to simulate the real behavior of the PCCV, both numerical models are refined by comparison of the results of the two analyses and with the existing research results. Also, more recently developed material models for concrete are introduced to the finite elements models. One is Menetrey-Willam' s concrete failure criterion which is used for axisymmetric global model, and the other is Modified Drucker-Prager' s model which is used for three-dimensional global model.

The computer program ABAQUS[2] is used to analyze the axisymmetric finite element model and the three-dimensional model of containment and nonlinear material properties of concrete, liner plate, reinforcing steel, and prestressing tendon with increasing the internal pressure to failure. Thereby, the final results which include the failure mode and the corresponding internal pressure level are determined. Also, in order to obtain better understanding of the numerical modeling, comparisons and discussions addressing the consistency or inconsistency of both results from the two finite element modeling are made.

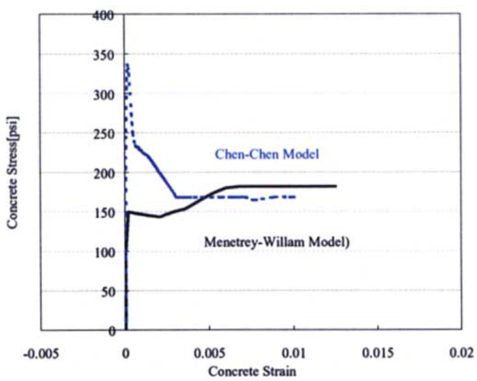
MATERIAL MODELS

The components of the PCCV including concrete, reinforcing steel, prestressing tendon and liner plates are modeled as nonlinear materials. When the finite element method with non-linear as analysis technique is used, the constitutive model of the materials is very important.

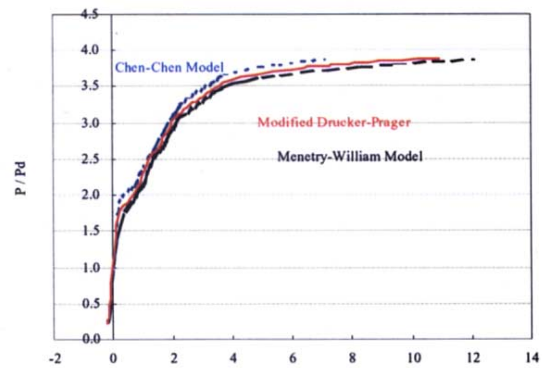
Concrete Failure Criterion

A The Chen-Chen' s model provided in the ABAQUS program correctly represents the concrete tension cracking and compression plasticity, but this model has corners in the stress space. These corners cause difficulties and complications in obtaining numerical solutions. The ultimate pressure capacity (UPC) assessment of the PCCV using Chen-Chen' s model, therefore, is hard to obtain because the concrete material behavior is highly nonlinear.

The material non-linearity for concrete is defined here as tensile type failure, such as the Menetrey-Willam' s model[3] and modified Drucker-Prager' s model with non-associated plastic flow[4, 5]. This failure criterion is defined by the formation of major cracks and the loss of the tensile strength normal to the crack direction. Menetrey-Willam' s model and modified Drucker-Prager' s model have the shortcomings that cannot represent the concrete compressive failure and estimates lower value of the concrete tensile cracking as shown in Figure 1, but their numerical solution is very stable under highly nonlinear status. Therefore, both models are effective for the UPC of the PCCV where the concrete tensile behavior is dominant and the material behavior is highly nonlinear.



(a) Concrete tensile behavior



(b) Internal pressure-displacement relation

Figure 1 Comparison results of the concrete models by sample analysis

The Menetrey-Willam' s model is used for the concrete constitutive model in the axisymmetric model, and the modified Drucker-Prager' s model is introduced for the concrete model in the three-dimensional model. The concrete failure criteria in the Menetrey-Willam' s model depend on a set of three independent scalar invariants. For geometric interpolation, the Haigh-Westergaard' s coordinates are used in the following equation.

$$\zeta = \frac{1}{\sqrt{3}} I_1 \tag{1}$$

$$\rho = \sqrt{2J_2} \tag{2}$$

$$\cos 3\theta = \frac{3\sqrt{3}}{2}\frac{J_3}{\sqrt{J_2{}^3}} \tag{3}$$

where ζ is the hydrostatic stress invariant, ρ is the deviatoric stress invariant, and θ is the deviatoric polar angle. The circular trace of the deviatoric polar radius $\rho(\theta)$ is transformed into a triple symmetric ellipse through the elliptic function $r(\theta, e)$ as follows, developed by Klisinski based on the five-parameter model by Willam-Warnke[6].

$$r(\theta,e) = \frac{4(1-e^2)\cos^2\theta + (2e-1)^2}{2(1-e^2)\cos\theta + (2e-1)\sqrt{4(1-e^2)\cos^2\theta + 5e^2 - 4e}} \tag{4}$$

The polar radius $r(\theta,e)$ extends to all polar directions $0 \le \theta \le 2\pi$ using the three-fold symmetric. Convexity and smoothness of the elliptic function require that $0.5 \le e \le 1.0$. This model is characterized by hyperbolic yield surface in the meridian plane $p - R_{mw}q$, where p is a pressure and $R_{mw}q$ is a Menetrey-Willam equivalent stress. From the relationships between the Willam-Warnke model, the Mohr-Coulomb model, and the Menetrey-Willam model, equation (4) can be translated into equation (5).

$$R_{mw}(\theta,e) = \frac{4(1-e^2)\cos^2\theta + (2e-1)^2}{2(1-e^2)\cos\theta + (2e-1)\sqrt{4(1-e^2)\cos^2\theta + 5e^2 - 4e}} R_{mc}(\frac{\pi}{3},\phi) \tag{5}$$

where $R_{mc}(\frac{\pi}{3},\phi) = \frac{(3-\sin\phi)}{6\cos\phi}$ is a polar radius of the ϕ-plane in the Mohr-Coulomb model. When the eccentricity value is taken as $e = 0.52$, the resulting failure trace of this model provides close agreement with the experimental data by Kupfer, Hilsdrof, and Rush [3]. When the eccentricity value is 0.52, the friction angle and the dilation angle, which can be induced from the relationships between the Mohr-Coulomb model, the Drucker-Prager model, and the Menetrey-Willam model, are 71.56 degrees and 56.97 degrees, respectively.

In the modified Drucker-Prager failure model, the yield surface and flow potential parameters for elasto-plastic material are defined by setting of the model parameters - the *K*-factor, the friction angle β, and the dilation angle ψ. This failure model provides approximate global solutions since the orthotropic nature of the post-cracking is not captured. The Drucker-Prager yield function is written as equation (6)[2].

$$F = t - p\tan\beta - d = 0 \tag{6}$$

where $t = (q/2)[1+(1/K)-(1-1/K)(r/q)^3]$, with the stress invariants, p, q, r, defined in stress and strain measurements. The shear strength of the material, d, is related to the uniaxial tension or compression yield stress. In the case of tension yield stress, the shear strength of the material is defined by equation (7).

$$d = (\frac{1}{K} + \frac{1}{3}\tan\beta)\sigma_t \tag{7}$$

The material parameter $K(\theta, f_\alpha)$ controls the shape of the yield surface in the deviatoric plane. To ensure convexity of the yield surface, $K(\theta, f_\alpha)$ must use between 0.778 and 1.0. The friction angle β is the angle between the yield surface and the pressure stress axis in the meridian plane. The plastic flow potential for this model is written as equation (8),

$$G = t - p\tan\psi \tag{8}$$

where the dilation angle ψ is the angle between the flow potential and the p-axis in the meridian plane. Three parameters including the K-factor, the firction angle, and the dilation angle are estimated as 0.778, 71.56 ,and 56.97 respectively.

Reinforcing Steel (Rebar), Prestressing Tendon, and Liner Plate

Rebar materials are generally incompressible when they deform plastically and yielding is independent of the pressure stress. The Von-Mises failure criterion is, therefore, used for this steel material. According to Hsu' s study result [7], the stress-strain curves of a bare steel bar and of a steel bar embedded in concrete are quite different. The stress-strain relationship of rebars embedded in concrete are introduced in the analysis instead of those for bare rebars. The shape of the stress-strain curve of the rebar resembles two straight lines with the slope of E_s before yielding and the slope of $E_p{}'$ after yielding as shown in the Figure 2.

The stress-strain curve of tendon consists of two straight lines jointed by a curve as shown in Figure 3. The first part is a straight-line up to $0.7f_{pu}$, where f_{pu} is the ultimate strength of the tendon. The curved part is expressed by the Ramberg-Osgood equation that meets the first part at a stress level of $0.7f_{pu}$. The stress-strain behavior of the liner plate steel is modeled using an elasto-plastic model. The von Mises failure surface with kinematic hardening is used to represent the nonlinear behavior of the material.

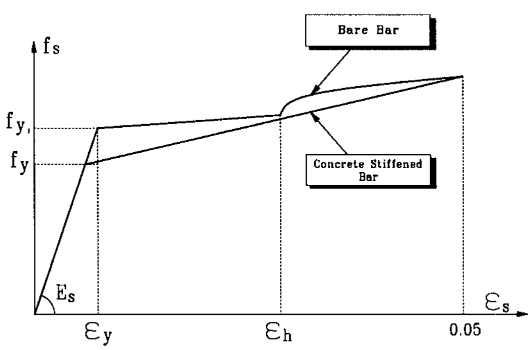


Figure 2 Stress-strain relationship of rebar using bilinear model

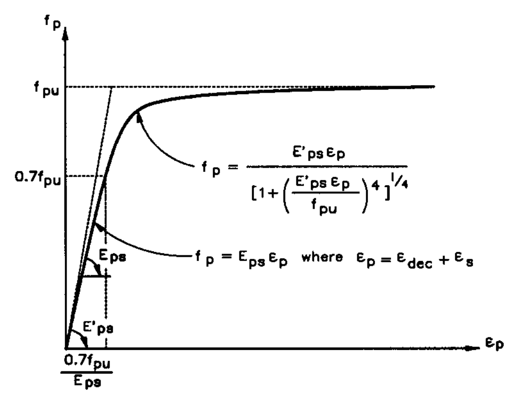


Figure 3 Stress-strain relationship of prestressing tendon

FINITE ELEMENT ANALYSIS

Axisymmetric Finite Element Model

The axisymmetric finite element model consists only of the axisymmetric cylindrical vessel, a spherical dome and the concrete base slab. This model is intended to provide the general global behavior of the PCCV model considering uplift of base slab. This model consists of 809 eight-node axisymmetric solid elements, nonlinear soil spring elements and 4698 nodal points. The concrete structure is modeled with eight-node axisymmetric solid elements. The liner steel on the inside surface of the PCCV is made up of three-node shell elements. The liner elements, which are offset from the prestressed concrete elements, are connected to the concrete solid elements by rigid link elements. All rebars and tendons are assumed to remain rigidly bonded to the concrete. Vertical liner anchors are modeled as a beam of rectangular cross-section dimension. The tendon gallery is considered in the model. The floor liner plate is assumed rigidly connected to the eight-node concrete solid elements since the effect of steel and concrete interaction during the flexural deformation of the slab is not significant for the thick base slab of the PCCV. The bottom of the slab rests on a soil foundation, which is modeled by nonlinear soil springs with tension cut-off.

Three-dimensional Finite Element Model

The three-dimensional finite element model includes large penetrations, such as equipment hatch and air-lock, and two buttresses which will cause deviation from an axisymmetric response and may decrease the capability of the PCCV. In order to simulate more realistic behavior near these regions, a more refined mesh is developed. Also, a rigid interconnection between shell element in the base slab and shell elements in the wall is introduced to properly simulate the shell/slab junction. The model consists of 1720 four-node shell elements, nonlinear soil spring elements and 1425 nodal points.

The concrete structure is modeled with composite shell elements consisting of a thin inner layer of steel representing the liner and much thicker outer concrete multi-layers. As mentioned in the axisymmetric model, the tendons and rebars are similarly assumed to remain rigidly bonded to the concrete. The bottom of the slab rests on a soil foundation, which is modeled by the nonlinear soil spring with tension cut-off.

Analysis Results

The pressure levels corresponding to the event milestones are summarized in Table 1. The deformation shapes of PCCV under ultimate pressure load are shown in Figure 4 and Figure 5. The pressure-radial displacement relationships by both FE analyses are compared in Figure 6. The pressure-vertical displacement relationships by FE analyses are shown in Figure 7 and Figure 8, respectively. And the pressure-hoop tendon strain relationship is shown in Figure 9.

Table 1 Pressure levels (MPa) corresponding to the events milestones

Milestones	Simple Model	2D Model	3-D Model
First cracking of concrete in cylinder due to hoop stresses	0.64	0.540	0.537
First cracking of concrete in cylinder due to meridional stresses	-	0.602	0.606
First yielding of liner due to hoop stresses	1.005	1.051	1.083
First yield of rebar in cylinder	1.005	1.051	1.083
First yield of meridional rebar in wall-base slab juncture	-	-	1.469
First cracking of dome concrete above 45° dome angle	-	0.602	0.606
First cracking of dome concrete below 45° dome angle	-	0.602	0.606
Hoop tendons in cylinder reaching 1% strain	1.323	1.435	1.407
Hoop tendons in cylinder reaching 2% strain	1.374	1.474	1.453
Hoop tendons in cylinder reaching 3% strain	1.412	1.514	1.491

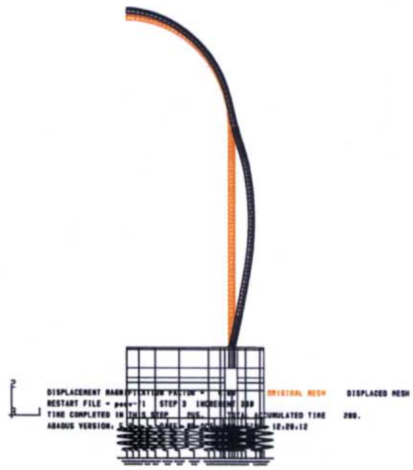


Figure 4 Configuration of axisymmtric model

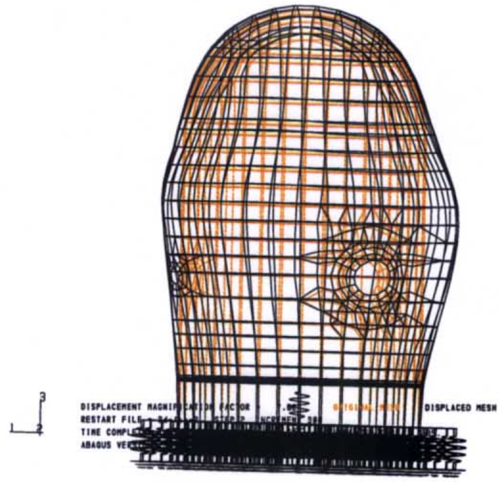


Figure 5 Configuration of 3D model

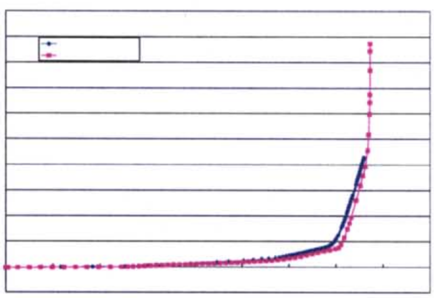


Figure 6 Radial displacement at the mid-height of cylinder

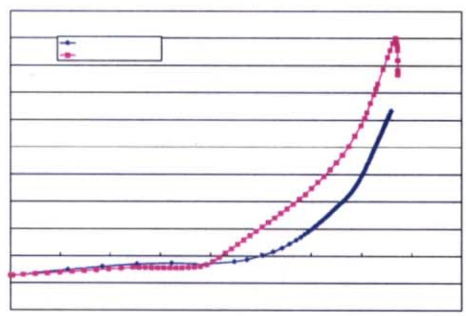


Figure 7 Vertical displacement at dome apex

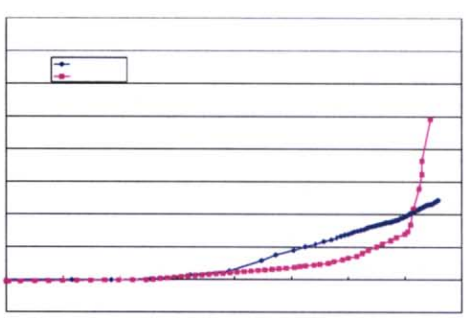


Figure 8 Vertical displacement at base slab

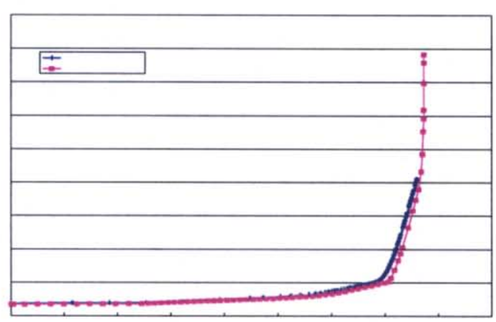


Figure 9 Strain in the hoop tendon at the mid-height of cylinder

CONCLUSIONS

1) The nonlinear behavior of prestressed concrete containment vessel under internal pressure loading is strongly influenced by the non-symmetrical characteristics of the structure. Therefore the axisymmetrical analysis is not appropriate to exactly assess the ultimate pressure capacity of prestressed concrete containment vessel.
2) The constitutive models of the material introduced here are confirmed to be able to avoid numerical instability up to ultimate pressure capacity of prestressed concrete containment vessel.
3) The three dimensional model proposed here is confirmed to be able to evaluate the exactly nonlinear behavior of prestressed concrete containment vessel, and the model is able to obtain the analysis results of the various locations.
4) The uplift of a base slab affects the global behavior of prestressed concrete containment vessel. The effect of this should, therefore, be considered in analysis.

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