ABAQUS Users' Conference
Chester, United Kingdom, May 1999, PP. 425－438

Axisymmetric Global Structural Analysis of Prestressed Concrete Containment Vessel above Design Pressure

Takako KASHIWASE
Systems Safety Department
Nuclear Power Engineering Corporation
Tokyo, Japan

Masahito SATO
Lattice Co. Ltd.
Tokyo, Japan

ABSTRACT

NUPEC has been planning the ultimate strength test of Prestressed Concrete Containment Vessel (PCCV) imaging severe accident, in cooperation with USNRC. The test model is 1/4 uniform scale model of Japan actual PCCV in which design pressure is 0.393MPa. It involves an equipment hatch, several penetrations and liner with T-anchors. The pressurization process is achieved by supplying nitrogen to the test model up to containment failure.

This paper describes several techniques to obtain physically reasonable analysis results with better convergence. The main techniques are consideration of soil spring, treatment of gravity and treatment of dome part meridional tendon.

1. INTRODUCTION

Many techniques of treating post-cracking concrete behavior have been proposed for concrete structure analysis and the analysis modeling itself sometimes depends on the experiences. Although many seismic tests and analyses on concrete reactor containment vessels have been conducted, few tests and analyses, focused on static internal pressure loading beyond design basis condition, have been conducted. As for reactor concrete containment, Sizewell B in U.K.(Twidale, 1991) and 1/6 scale reinforced concrete containment vessel (RCCV) test in U.S(Horschel, 1992) were only available.

Sizewell B was internal pressure loading test using 1/10 scale PCCV model, which was terminated because of basemat uplift and could not reach ultimate structural condition. 1/6 RCCV test was conducted up to liner tearing and provided valuable data. However, it did not include the effect of prestressing of tendon.

Considering the above situation, NUPEC has been planning the ultimate strength test of Prestressed Concrete Containment Vessel (PCCV) imaging severe accident, in cooperation with USNRC. The test model is 1/4 uniform scale model of Japan actual PCCV in which design pressure is 0.393MPa. It involves an equipment hatch, several penetrations and liner with T-anchors. The pressurization process is achieved by supplying nitrogen to the test model up to containment failure. The pre-test analysis will be compared with measured data and the post-test analysis using this test data will improve analysis model.

This paper describes several techniques to obtain physically reasonable analysis results with better convergence using axisymmetric global modeling.

2. COMPUTATIONAL MODELS

2.1 Material properties

The measured average data of concrete Young's modulus, Poisson's ratio, compressive strength, and tensile strength were used based on the trial mix concrete tests after field curing. As for rebars the average stress-strain curves of each rebar material test were used except dumbbells. The average stress-strain curves obtained from the liner material tests were also used, assuming isotropy of the material. Two dimensionality of linear material was considered by Mises yield function. The measured stress-strain curve of a tendon material test was used. The summary of material properties is shown in Table 1.

2.2 Material models

A finite element program ABAQUS/Standard Ver. 5.8 was used to predict the global behavior of 1/4 PCCV test. The stress-strain curve of post-cracking concrete was determined through the following sensitivity study, referring to the displacement data at the cylinder mid point of the 1/6 RCCV test results conducted at SNL.(Horschel, 1992) That is, various tension-stiffening curves were compared under full shear retention model. The tension-stiffening was assumed to be linearly decreased and was reduced to zero at strain values of 10, 20, 30, 40 and 50 times of the concrete crack strain of 1.28×10^{-4}, considering the simplicity of global analysis. The 10 times model was the best agreement with 1/6 RCCV data, as shown in Fig. 1. However this 10 times model induced unstable behavior above 3.0 times design pressure (Pd) condition so that 1 % crack stress was retained even after 10 times of the crack strain.

The reduction of the shear modulus after concrete cracking was determined as follows. The full shear retention model was compared with the model in which shear stiffness under cracking condition was reduced linearly to zero at 10 times of crack strain. Both models were calculated up to 3.4 Pd where concrete cracking in hoop direction extended to the whole region. The analysis results were almost same so that the full shear retention model was adopted because of the better convergence of calculation.

Non-linear behavior in multi-axial stress field was traced by crack detection surface and compressive surface incorporated in ABAQUS code. Defaults values of the surface parameters in ABAQUS were used, except the ratio of the tensile strength to the compressive strength because of no biaxial test data.

2.3 Modeling of 1/4 PCCV test facility

The axisymmetric global analysis was conducted as the first step pre-test analysis. The analytical model included general parts of a dome, a cylinder and a basemat, as shown in Fig. 2. The wall of dome part and cylinder part was divided into 6 elements. The dome part was divided into 45 elements and the cylinder part was divided into 60 elements. The basemat had 10 vertical elements and 42 radial elements. However, the juncture region between basemat and cylinder had finer elements. The model included the total numbers of 1963 nodes and 1279 elements.

In order to select an element type for the concrete, three solid element types were compared (4-node bilinear (ABAQUS:CAX4), 8-node biquadratic reduced integration (ABAQUS:CAX8R) and 4-node bilinear incompatible mode (ABAQUS:CAX4I)). When 8-node biquadratic reduced integration element and the 4-node bilinear incompatible mode element were used, very high stress occurred in the loading end at about 3.0 Pd and the calculation became divergent. It was supposed that the force directions of some nodes in a element was inconsistent with the direction of element stress in these cases. Since the materials exhibited strong non-linearity and the contact between tendon and concrete was important under high internal pressure condition, 4-node bilinear element was adopted. 6 elements in wall thickness direction was adopted to evaluate bending behavior.

Rebar was defined as reinforcement (ABAQUS:REBAR) in solid element. Liner was modeled by shell element (ABAQUS:SAX1) and the liner node was shared with inside node of concrete.

2.4 Analytical modeling

Hoop tendon was modeled by rebar bonded to the concrete. Meridional tendon was modeled by shell element and the hoop direction stiffness was made zero in the cylinder part. The hoop direction stiffness was considered only above 45^0 dome angle, because the meridional tendons in the dome part are arranged as like mesh. Friction was specified between the concrete and the meridional tendon. The meridional tendon was allowed to slide relative to the concrete. Friction coefficient at dome part was 0.2156 which consisted of μ=0.21, average value of the measured friction coefficient, and friction coefficient considering tendon length effect (λ=0.001 per unit length). For cylinder part, μ=0 and λ=0.001 were used to stabilize the analysis. Although an empirical friction correlation between tendon and sheath was proposed as a function of loading end load and circumferencial angle,(Kashiwase, 1997) it was not used for the present global axisymmetric analysis. Because hoop tendons don't have a tensile force distribution and meridional tendons are arranged as like mesh.

As a boundary condition, a horizontal direction was fixed along axisymmetric axis to realize axisymmetric condition. Non-linear soil springs were placed at the bottom of basemat to simulate the ground reaction force. The actual ground stiffness was used against compression force and it was zero against tension force. However, the soil springs in non-uplift part had stiffness against tension force, because the vertical displacement became too large when all tensile stiffness of soil springs in non-uplift part were zero. The gravity was considered only at the bottom of basemat as a concentrated force.

3. SENSITIVITY STUDIES FOR THE DETERMINATION OF ANALYTICAL MODELINGS

This section describes the analytical techniques to obtain appropriate analysis results. The main techniques are treatment of dome part meridional tendon, consideration of soil spring and treatment of gravity.

In order to simulate a setting loss condition of tendon, meridional tendon was prestressed with the prescribed value of 503 kN and then loosed to the prescribed value of 470 kN. As for hoop tendon initial stress of 991 MPa, corresponding to the average value of hoop tendon stress, was imposed.

(1) Consideration of soil spring

Ground stiffness affects the global deformation behavior via ground reaction force under high internal static pressure condition. Therefore, the reaction force of the ground was taken into account by placing soil springs at every node of basemat. The uniform soil spring stiffness values of 1×10^{12} N/mm, 1×10^{6} N/mm, 1×10^{4} N/mm and 1×10^{2} N/mm were used for sensitivity analysis. As a result, it was found that soil spring stiffness affected not only deformation behavior but also the convergence under high internal pressure condition. The difference of analysis results under high pressure condition was governed by the uplift condition of the basemat. The stiffness values of 1×10^{6} N/mm provided the best convergence which was consistent with the ground stiffness of actual test site of 30 MPa(Miller, 1992). Therefore, the value of soil spring stiffness was finally distributed in proportion to the node area on the bottom of basemat using 30 Mpa ground stiffness.

(2) Treatment of gravity

Gravitational force should be considered correctly to evaluate uplift behavior of the model, since actual soil spring of 30 MPa was adopted. Two cases were compared. In one case gravity was considered as concentrated mass at every Gaussian point as usual and in the other case gravity was considered only at the bottom of the basemat as concentrated force.

Both analysis results were almost same, because vertical force by gravity was negligible compared with tendon tension force. For example, force balance of two elements at the apex of dome was evaluated as follows. Vertical component of the tensile force of meridional tendon was 25 kN. On the other hand, the vertical force by gravity was 0.49 kN, which was only 2 % compared with meridional tendon vertical force. Therefore, the gravity was considered only at the bottom of basemat as concentrated force.

In the ABAQUS model gravity is usually considered as concentrated mass at every Gaussian point and the displacement is calculated by solving equation of motion. Hence, calculation time was longer and the convergence of calculation became worse under large displacement condition.

(3) Treatment of dome part meridional tendon

Figure 3 shows the analytical results of vertical displacement of the test model for two modelings of meridional tendon in dome part. The one model simulated hoop stiffness in the meridional tendon above 45^0 dome angle (model (a)), and the other model simulated hoop stiffness in the meridional tendon above spring line (model (b)). The difference of two models was observed from 2.3 Pd. The apex displacement (1) of model (a) increased in downward direction, while in model (b) it increased in upward direction. This was more clearly demonstrated in Fig. 4 showing the deformation of test vessel under 3.0 Pd condition.

The difference of concrete cracking condition caused the above different behavior. Model (a) caused vertical cracking, while model (b) exhibited as if spring line were fixed. Fig. 5 shows cracking condition in hoop direction at 2.4 Pd where the difference of cracking condition for the two models became remarkable. In model (a) cracking extended to dome part, while in model (b) it occurred in the cylinder part only.

As mentioned above, the vertical displacement depended on tendon modeling in the dome part. However, the test model failure will occur near penetrations in cylinder part and the strain distribution of the cylinder part for both models were same. Therefore, the model (a) was selected in the present analysis because of the better convergence under higher internal pressure.

4. CONCLUSION

An axisymmetric analysis was conducted to predict the global behavior of 1/4 PCCV test. The analytical techniques to obtain appropriate analysis results were summarized as follows.

The stress-strain curve of post-cracking concrete was determined through sensitivity study, referring to 1/6 RCCV test results. That is, post-cracking stress was assumed to be linearly decreased to 1 % of crack stress at 10 times of the concrete crack strain (1.28×10^{-4}) and 1 % of crack stress was retained even after 10 times of the crack strain because of obtaining better convergence. For shear stiffness the full retention model was used because of the better convergence of calculation.

4-node bilinear element was adopted for concrete modeling, considering strong non-linearity of the materials and the importance of contact between tendon and concrete.

Non-linear soil springs were placed at the bottom of basemat to simulate the ground reaction force which affected the overall deformation of the test facility. The value of ground stiffness was the actual one for compression and was zero value for tension.

The gravity was considered only at the bottom of basemat as a concentrated force due to very small gravitational force compared with forces acting on an element. Since this modeling did not require the calculation of mass matrix composed of equation of motion, calculation time became shorter and the convergence of calculation became better.

Regarding to the treatment of dome part meridional tendons arranged like mesh, the hoop direction stiffness was considered only above 45^0 dome angle because of better convergence under higher internal pressure. Because the test model failure will occur near penetrations in cylinder part and the strain distribution of the cylinder part was not affected by this modeling.

5. REFERENCES

Horschel, D. S., 1992, "Experimental Results From Pressure Testing A 1:6-Scale Nuclear Power Plant Containment," NUREG/CR-5121, SAND88-0906

Kashiwase, T., and Nagasaka, H., 1997, " Analysis Study on Change of Tendon Tension Force Distribution during the Pressurization Process of Pre-stressed Concrete Containment Vessel, " Proceedings of ICONE-5, ICONE5-2389

Miller, C. M., 1992, "Geotechnical Investigation Job No. 1-20904 Containment Technology test Facility - West Albuquerque, New Mexico," GEO-TEST

Twidale, D., and Crowder R., 1991, "Sizewell 'B' - a One Tenth Scale Containment Model Test for the UK Programme," Nuclear Engineering and Design 125, pp.85-93

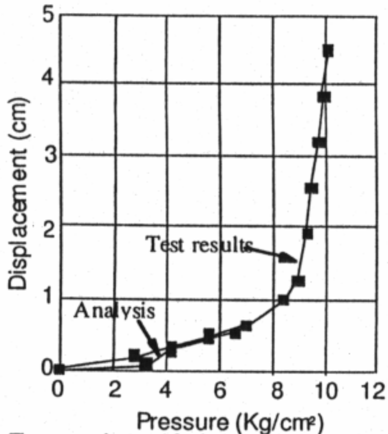
6. ACKNOWLEDGMENT

This work is sponsored under the contract by the Ministry of International Trade and Industry, Japan.

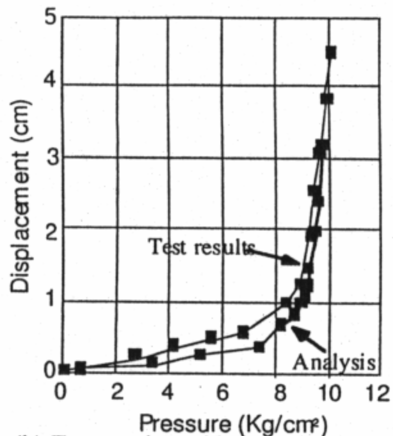
Table 1 Summary of material property

Material	Young's modulus (MPa)	Poisson's ratio	Yielding stress (MPa)
Concrete	2.70×10^{4}	0.18	3.59
Tendon	1.94×10^{5}	0.3	1.47×10^{3}
Rebar	1.85×10^{5}	0.3	459.
Linear	2.19×10^{5}	0.3	377.
Sheath	1.92×10^{5}	0.3	-

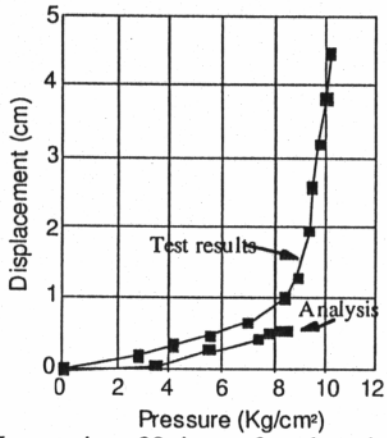
* Tensile strength



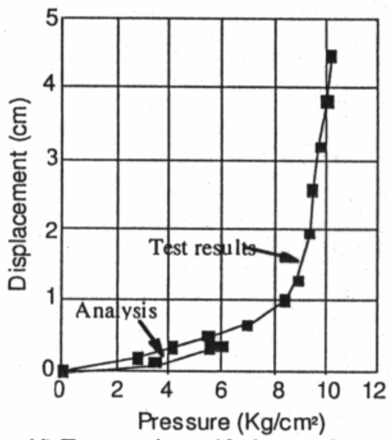
(a) Zero strain at 10 times of crack strain



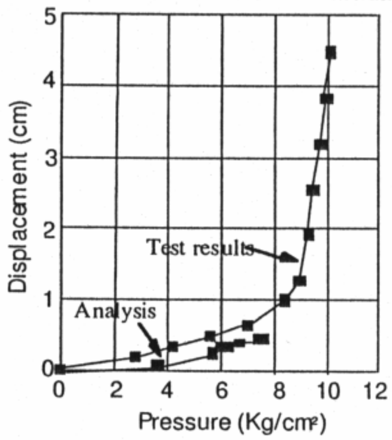
(b) Zero strain at 20 times of crack strain



(c) Zero strain at 30 times of crack strain



(d) Zero strain at 40 times of crack strain



(e) Zero strain at 50 times of crack strain

Fig. 1 Sensitivity analysis for determining tension-stiffening characteristics

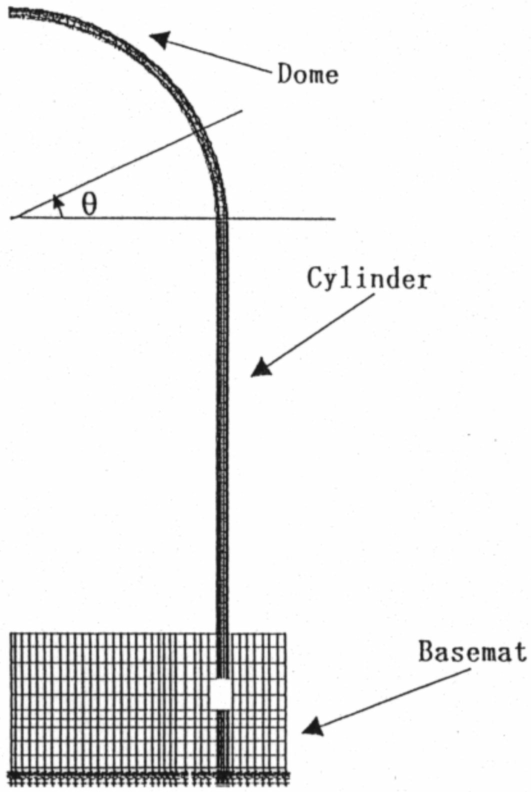


Fig. 2 Analysis modeling

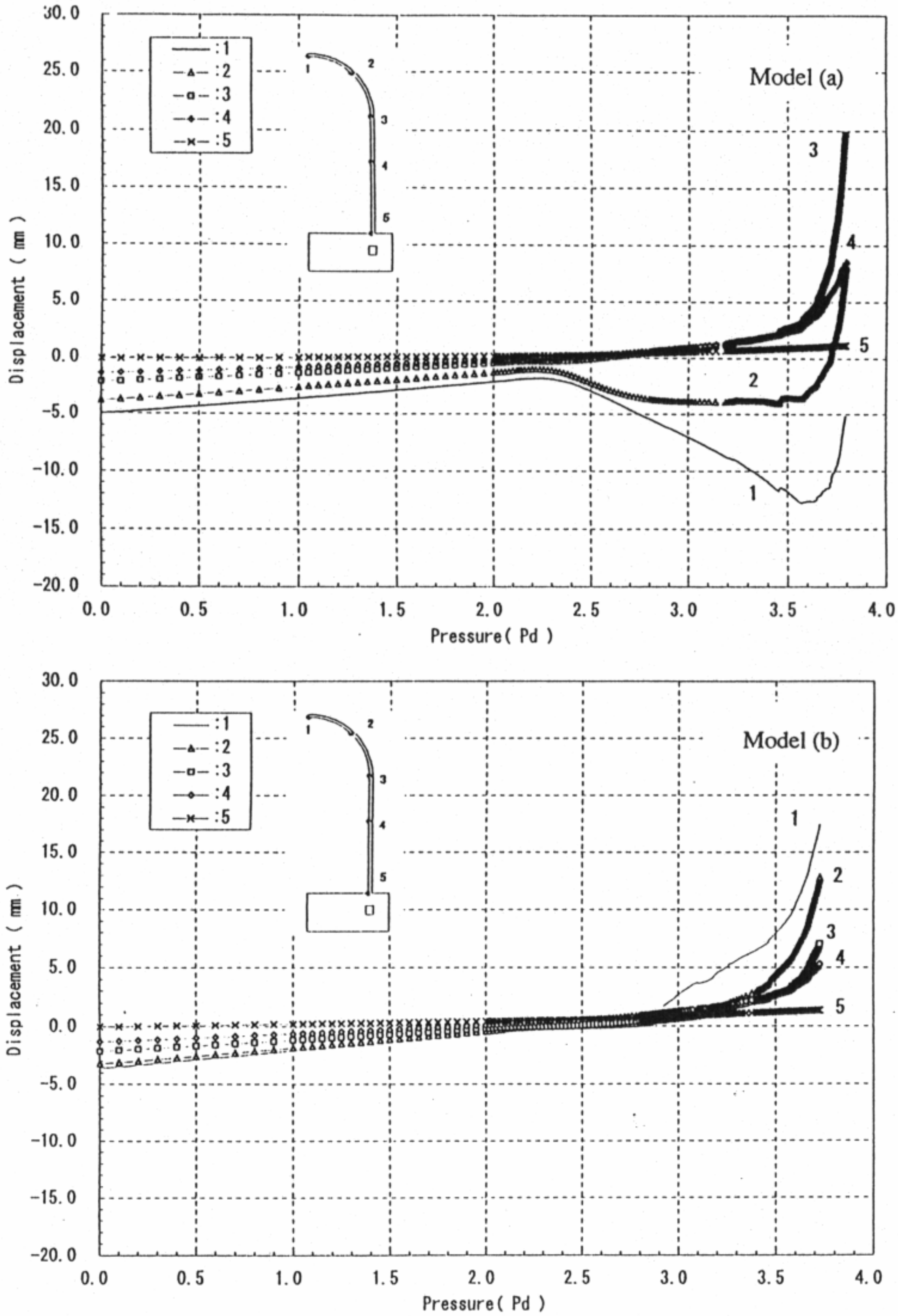


Fig. 3 Effect of meridional tendon modeling on vertical displacement

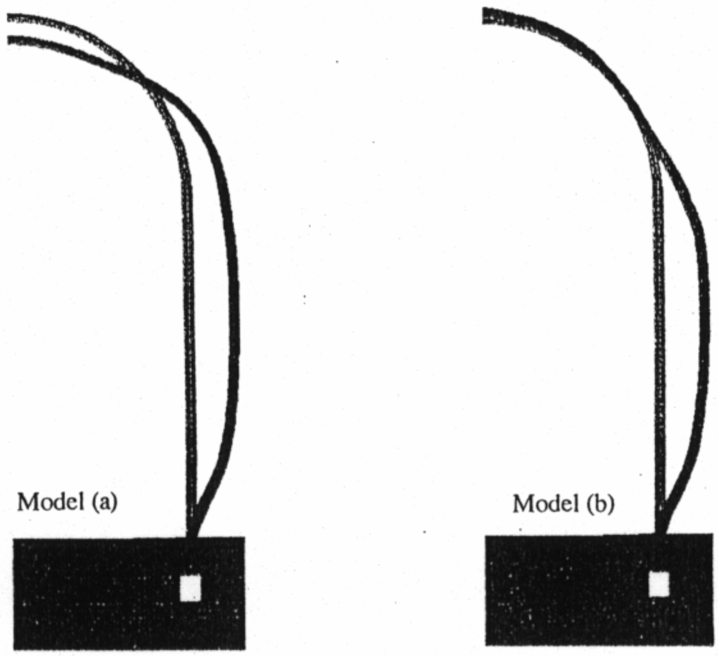


Fig. 4 Effect of meridional tendon modeling on overall deformation behavior under 3.0 Pd

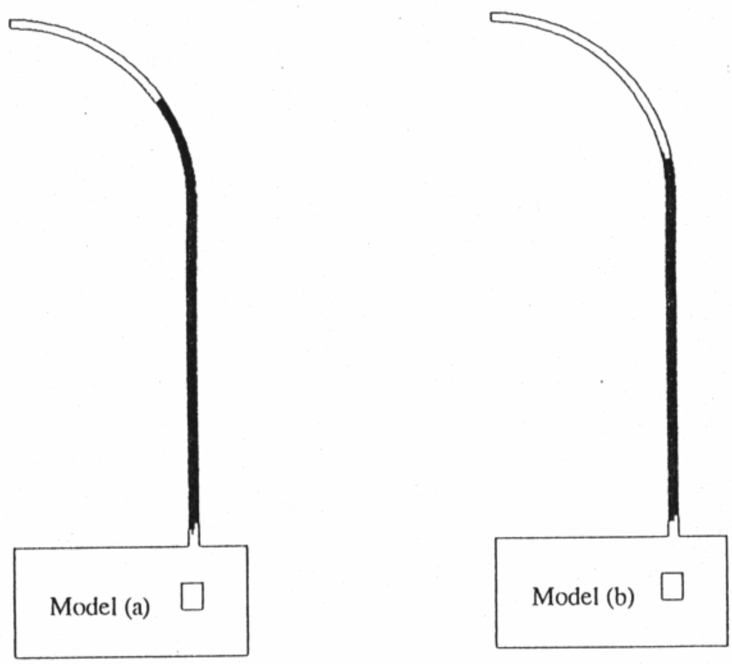


Fig. 5 Effect of meridional tendon modeling on hoop crack behavior under 2.4 Pd