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Analytical Study on Structural Failure Mode of 1/4 PCCV Test Model

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ABSTRACT

The Nuclear Power Engineering Corporation (NUPEC) and the US Nuclear Regulatory Commission (NRC) established a JAPAN-US cooperative research project on the ultimate capacity of nuclear reactor containment vessels subjected to internal pressure. As a part of this research, ultimate capacity pressure tests were carried out on a 1/4 scale prestressed concrete containment vessel (PCCV) model of existing PCCV for pressurized water reactors (PWR) in 2000 and 2001 at Sandia National Laboratories (SNL), USA. In addition, Round Robin pretest and posttest analysis meetings were held, with a view to improving the existing analysis methods for the nonlinear behavior of PCCVs. The Japan PCCV research group consisting of the above six companies participated in these meetings and presented pretest and posttest analysis results obtained in our research program. Many types of global and local analyses on a 1/4PCCV test model were conducted to establish an analysis methodology in our research program.

This paper describes the reliability of the analysis methods developed here for the nonlinear behaviors of PCCV up to ultimate internal pressure. In addition, the possible failure modes of PCCV were investigated by examining deformations and strain distributions of the model obtained from the analysis results.

Comparison of the analytical and test results confirmed that pretest and posttest analyses can predict and simulate not only the nonlinear behaviors but also the structural failure modes of the PCCV test model with good accuracy.

KEY WORDS: PCCV, Pressure test, FEM analysis, ultimate capacity, nonlinear behavior, structural failure, 1/4PCCV

1. INTRODUCTION

A JAPAN-US cooperative research project for containment vessels had been carried out since 1992 by NUPEC and NRC [1] and completed in October 2002 with great success. As a part of this research, Limit State Tests (LST) and Structural Failure Mode Test (SFMT) were carried out on a 1/4 scale PCCV model of existing PCCV in Japan, in 2000 and 2001, respectively, at SNL [2]. In addition, before and after the LST, Round Robin pretest and posttest analysis meetings were held, with a view to improving the existing analysis methods for the nonlinear behaviors of PCCVs. A Round Robin pretest analysis meeting was held with participants of 17 organizations from 9 countries in 1999 at SNL [3].

The Japan PCCV research group consisting of the above six companies participated in the Round Robin analyses. Many types of pretest and posttest analyses were conducted on the 1/4PCCV model to establish an analysis methodology that can predict the nonlinear behavior and failure mode of actual PCCVs subjected to increasing internal pressure.

This paper describes the reliability of the analysis methods developed by this group mainly for the structural nonlinear behavior and the structural failure modes of 1/4PCCV by comparing the test results with the pre-& post-test analysis results.

2. OUTLINE OF 1/4 PCCV TEST MODEL

The PCCV model is a uniform 1:4 scale model of actual PCCVs used in Japan. The model includes a steel liner and scaled representation of openings and penetrations, such as the equipment hatch (E/H), personal airlock (A/L), main steam (M/S), and feed water (F/W). In the region surrounding these openings, both the rebar ratios and the wall thickness are larger than those of general cylinder walls for reinforcement. The configuration and structure outline of the test model is shown in Fig.1 (a)(b).

The LST, in which nitrogen gas was adopted for internal pressurizing medium, was terminated by out of nitrogen gas due to leakage from liner tearing portions at the maximum pressure of 1.32MPa. Although the maximum rebar strain reached 1.68% and some steel yielding was observed at the maximum pressure, the structural failure was not observed [4].

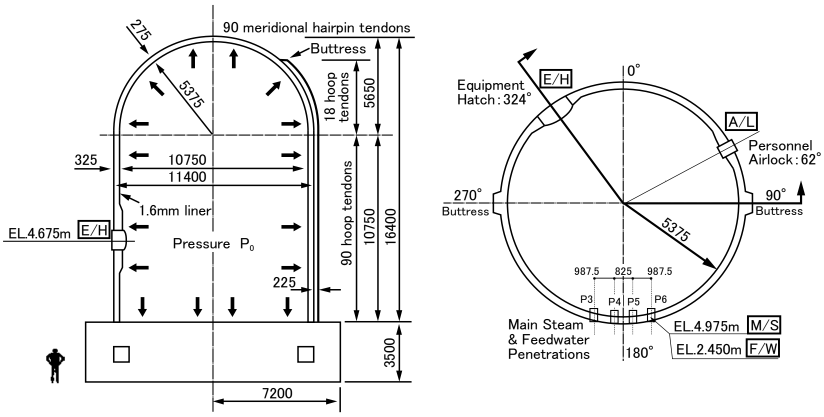


Fig.1(a) Configuration of 1/4PCCV test model

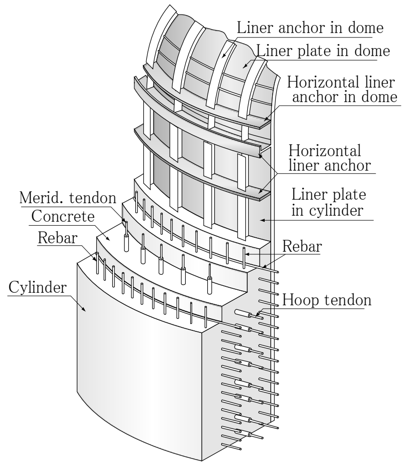


Fig.1(b) Wall structure of PCCV

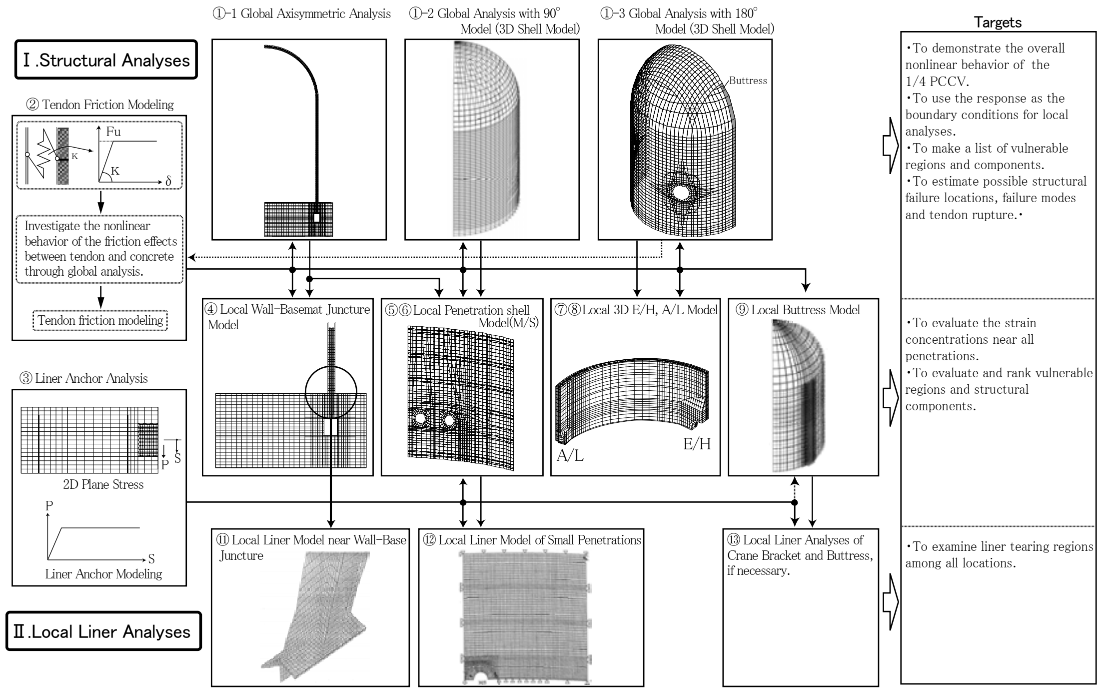


Fig.2 Structure of this research program

Therefore, after investigating the test data, a SFMT was conducted to evaluate the structural limit state behavior by re-pressuring the test model. In the SFMT, the liner tearing points that occurred during the LST were repaired, and hydraulic pressure was used as the pressurizing medium. The test resulted in a structural failure due to rupture of the hoop tendons and rebars at about the azimuth 6°at a height of about EL 7.0m at the maximum pressure of 1.42MPa[2].

3. OUTLINE OF PRETEST AND POSTTEST ANLYSES

The structure of this research is shown in Fig.2, which clearly shows analytical model images and the interrelation of global and local analyses. Three types of global and four types of structural local analysis models were conducted here. Fig.3, Fig.4, Fig.5 and Fig.6 show computational grids of an 180°model, a 90°model, an E/H model, a wall-basemat juncture model, and an M/S model, respectively.

The objective of the global analyses was to evaluate the global non-linear behavior of the test model, and to obtain the boundary conditions of local analyses. An axi-symmetric model idealizes a general portion at 135°azimuth. The 3D90°model shown in Fig.3 idealizes a one quarter section (azimuth 180° - 270°) of the test model. The 3D180°model shown in Fig.3 idealizes a half section (azimuth 90° - 270°) of the test model. In the global analyses, The friction model with μ=0.21[3] proposed in SMiRT15[5] is used to model the friction characteristics between concrete and tendons. It has been confirmed that this friction model can evaluate the slip amount and tendon strain distribution up to ultimate capacity [5]. Analytical modeling methods for these global analysis models are shown in detail in reference [6][7].

The objective of the local analyses was to evaluate the potential failure regions. Fine mesh models are used in the local analyses. The area near the wall-basemat juncture, E/H opening, A/L opening, M/S opening, and buttress are adopted as potential failure regions based on the results of global analyses. The displacements obtained from the global analyses are applied on the surrounding surfaces of the models as boundary conditions. In the E/H model, A/L model, and buttress model, the RC wall is modeled by 8-node solid elements. The multi-layered shell elements are used to model the RC wall in the M/S model. Axi-symmetric analysis is adopted in the wall - basemat juncture model.

The computer codes used here were FINAL, DIANA and MARK. FINAL [8] developed by Obayashi Corporation for nonlinear analysis of concrete structures was used for all global analysis models, E/H model, A/L model, M/S model and wall - basemat juncture model. The general purpose codes DIANA and MARK were used for the buttress model and Liner analyses, respectively.

For the concrete constitutive model, the authors adopted the equivalent uniaxial strain model proposed by Darwin et al. and developed for three-dimensional analysis by Murray. A smeared crack model is assumed for the crack model. The loading - unloading - reloading rules for principal stress-equivalent uniaxial strain relationships are schematically plotted in Fig.7 [9]. The Al-Mahaidi equation [10] is used to model shear retention in a cracked plane. The constitutive laws mentioned above are shown in detail in ref. [11].

For the steel materials (rebars, tendons and liner), the elasto-plastic theory based on von Mises yield criterion was used to represent nonlinear behavior.

The material properties of the concrete and the steel used in the analyses were the same as those given in [3].

Posttest analyses were performed with the 3D90°model, 3D180°model, and E/H model selected in consideration of the test results. In the posttest analyses, the tensile strength of concrete was reduced to zero in consideration of the effects of dying shrinkage of concrete as investigated in ref. [12]. The internal pressure was increased to 1.32MPa, and then decreased to 0.0MPa to simulate the pressure of the LST.

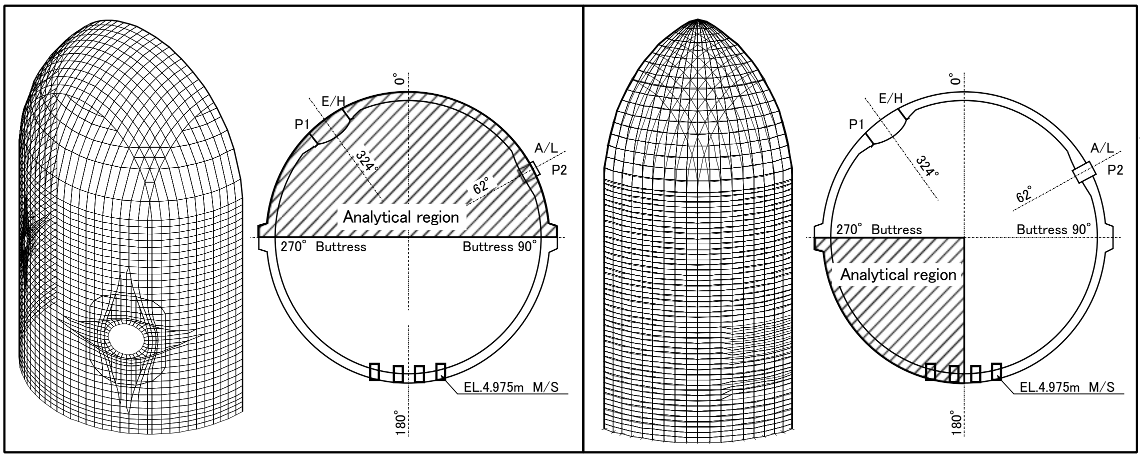


Fig.3 Computational grids and analytical regions of 180°model, 90°model

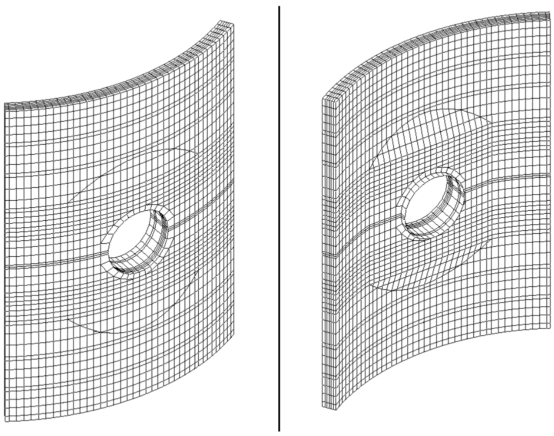


Fig.4 Computational grids of E/H model

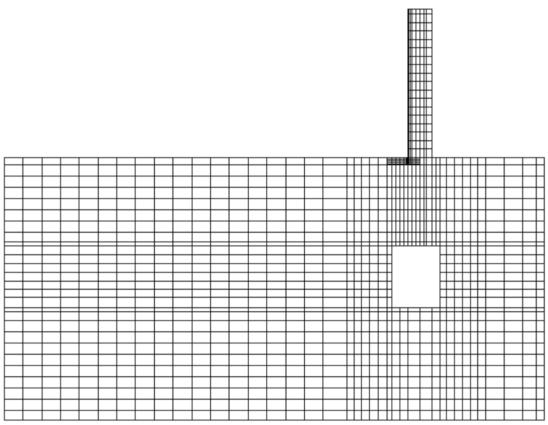
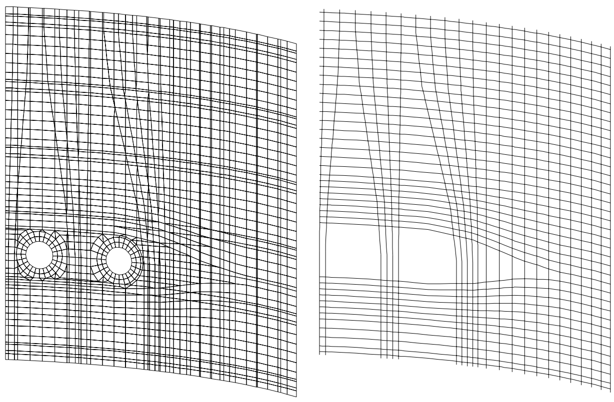


Fig.5 Computational grids of wall - basemat juncture model



(a) RC Wall (b) Tendon

Fig.6 Computational grids of M/S model

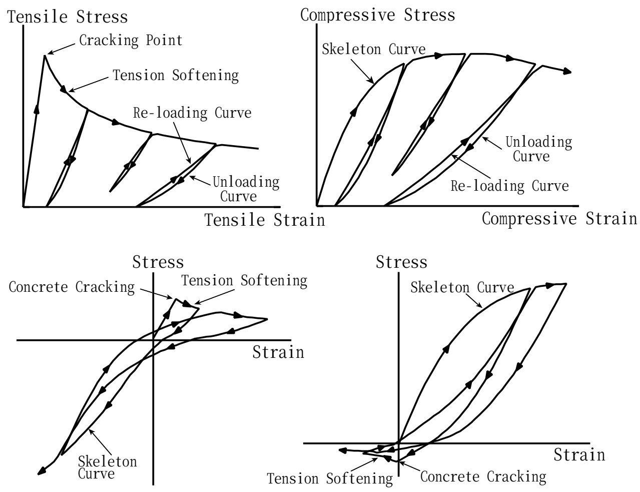


Fig.7 General rules of principal stress-equivalent uniaxial strain relationship of concrete

4. PREDICTION OF POTENTIAL FAILURE BY PRETEST ANALYSES

The possible structural failure regions of the test model by the pretest analyses are shown in Fig.8. Fig.9 shows the deformation contours obtained from the global analyses at pressure of 1.2MPa and 1.45MPa. Fig.10 shows the tendon strain distribution obtained from the global analyses. The results of the pretest analyses are summarized as follows.

1) As can be seen from Fig.9, the maximum displacement appears around the E/H at a pressure of 1.2MPa. However, when the pressure increases to 1.45MPa, the displacement of a general portion becomes largest. It was firstly found that the maximum displacement of the PCCV appears at a different location in each pressure level.
2) Tendon rupture is likely to occur in the general portion at a height of about EL.7.0m, as shown in Fig.10.
3) There is no possibility of bending/shear failure at the wall - basemat juncture prior to tendon rupture in the general portion according to the local analyses. As a reference, the minimum principal stress contour of concrete obtained from the wall - basemat juncture model is shown in Fig.11. Concrete crushing due to flexure occurs at the end of the wall at a pressure of 1.5MPa. However, the principal stresses elsewhere do not exceed the uniaxial compressive strength of concrete, and the shear ties in the wall - basemat juncture remain elastic up to 1.6MPa.
4) The strain concentrations of the rebars and liner appear in several portions where stiffness changes, such as rebar cut off sections and buttress as shown in Fig.12.
5) There is no possibility of structural failure at the portion near the E/H and A/L openings according to the local analyses. Because the vicinities of the openings are reinforced by increasing the rebar ration to more than that in the general portion.

From the above summaries, the structural failure modes and their associated pressure of the 1/4PCCV model can be inferred as follows:

a) The results obtained from pretest analyses for the 1/4PCCV are almost the same as those from the pressure tests of 1/6RCCV [13]. That is, the ultimate pressure was reached due to local ruptures of structural components at the mid-height of the cylinder wall, where the maximum radial displacements were obtained or observed. Thus, radial displacements in the vicinity of E/H, A/L, and M/S of the PCCV were determined to be relatively small compared to the maximum, resulting in no main rupture around these areas. Therefore, it can be said that the ultimate state of concrete containment structures like PCCV and RCCV can be induced by local ruptures of structural components due to local strain or stress concentrations at the mid-height of the cylinder wall, where the maximum radial displacements appear.
b) During the high pressure stage near the ultimate state, strains in the tendons and rebars, which bear approximately 91% of the structural capacity, are very close to their critical states. Thus, if one of them initially ruptures, catastrophic failure of the PCCV will occurs. It is very difficult to define which one fails first. However, if liner tearing occurs first, a slow depressurization of the PCCV will occur.

If the test model had been constructed accurately, and if any initial imperfections of the materials and structural components were also negligible at the time of the tests, the ultimate state of 1/4PCCV is estimated as follows:

Ultimate pressure: 1.5MPa

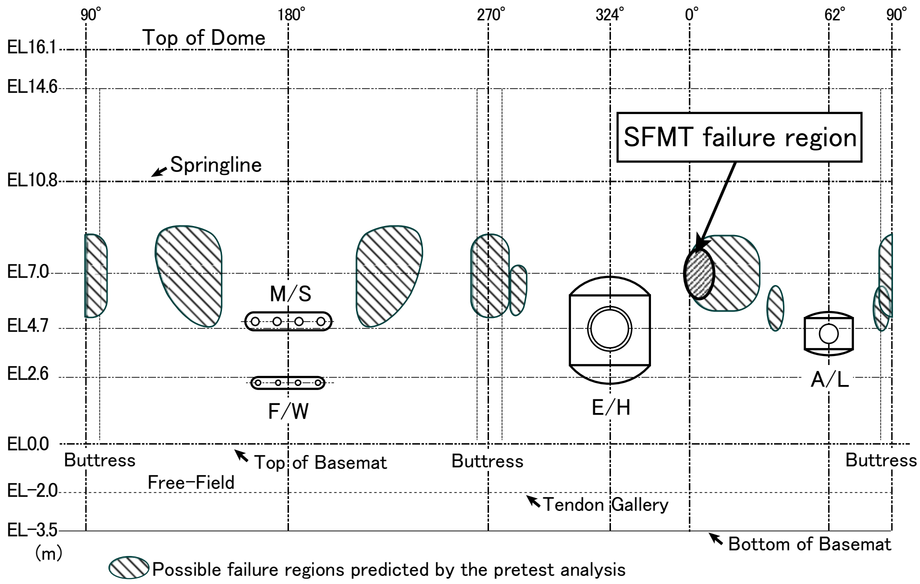
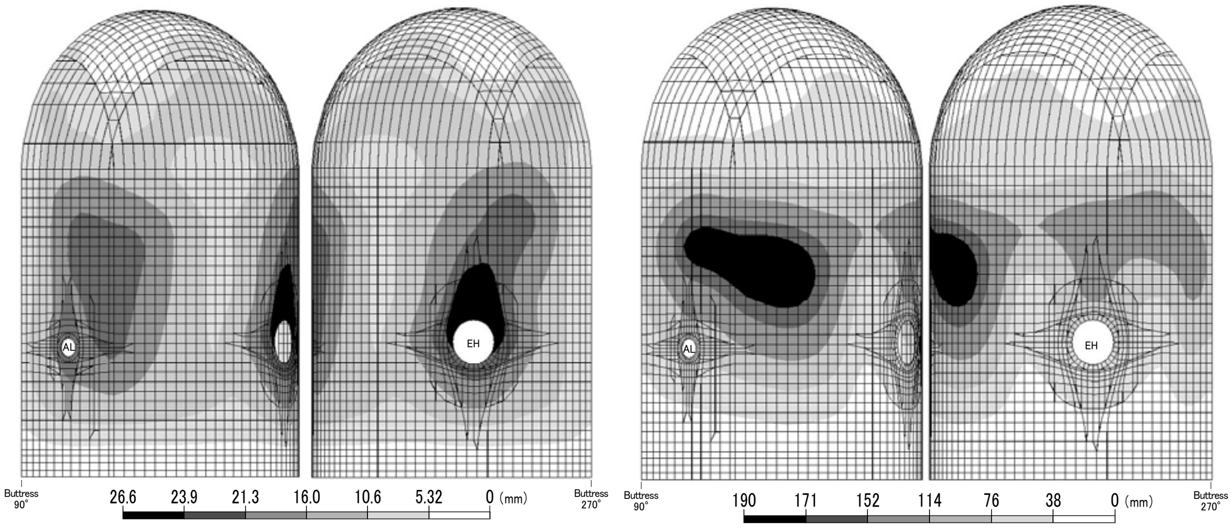


Fig.8 Comparison of the structural failure regions by analyses with the test(SFMT) result



Pressure=1.2MPa
Pressure=1.45MPa

Fig.9 Deformation contours by pretest analysis with 180°model

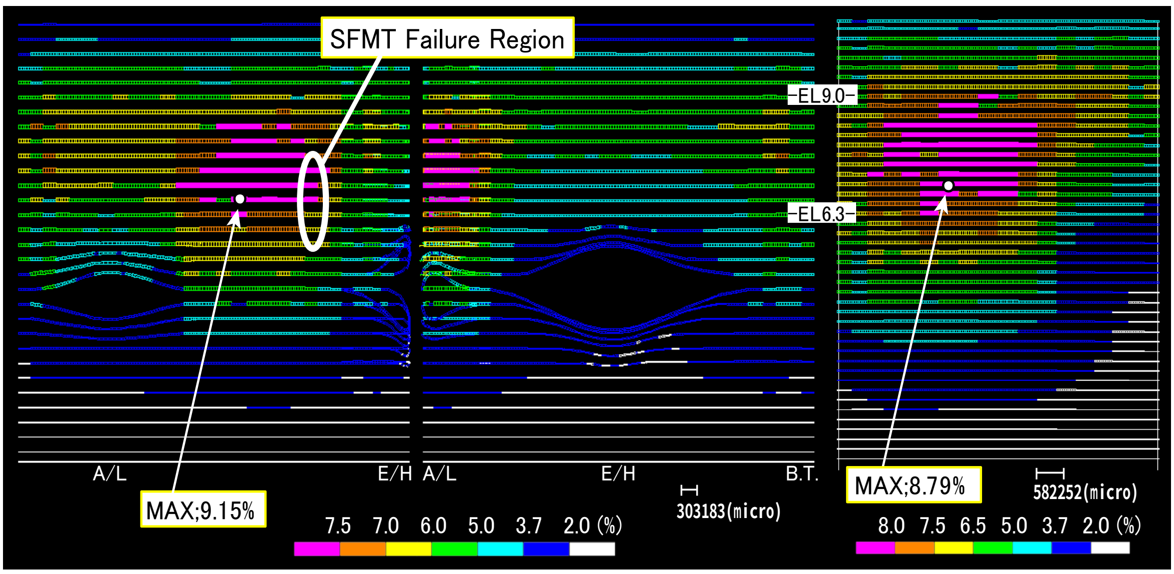


Fig.10 Tendon strain distribution contours by pretest analyses at 1.5MPa

Failure mode: Rupture of structural elements (tendon, rebar or liner) placed in the hoop direction at a cylinder wall at a height of about EL.7m.

5. COMPARISON OF PRE- & POST-TEST ANALYSIS RESULTS WITH TEST RESULTS

Comparisons of the pre- & post- test analyses with test results are shown in Fig.13 and Fig.14. Fig.13 compares the pressure - radial and vertical displacement relationships at the measuring points, and Fig.14 compares the pressure - rebar strain relationships. The test results shown here are of the LST.

It can be seen from these figures that pretest analyses can predict the nonlinear behaviors of the test results except the non-linearity of the first turning points. This is also found at the other measuring locations. This difference on the first turning points is also observed at most measuring points. That is, nonlinear responses of the test results behave as if the tensile strength of concrete is very small. In general, the first turning point in reinforced concrete structures has been recognized to be caused by concrete cracking. By studying the reason, it was found that the tensile strains of concrete in the test model due to drying shrinkage might be as large as the cracking strain in the tests. This is because the cracking point was not clearly observed in the test results.

However, in the posttest analyses, which takes account of drying shrinkage of concrete, the tendencies to nonlinear behaviors of the posttest analysis results and the test results are almost the same. An important point to emphasize is that the posttest analysis results were improved in accuracy near the first turning points. For the behaviors at discontinuous portions such as the E/H opening, A/L opening, buttresses, and apex, it is also shown that the analysis results agree well with the test results for both displacements and rebar strains.

For the characteristics of the unloading loops on the pressure - displacement and rebar strain relationships, the posttest analyses can also simulate the behavior of the test results with good accuracy.

6. STRUCTURAL FAILURE MODE OF THE TEST MODEL

A structural failure portion at the SFMT is plotted in the Fig.8 and Fig.10, which also show the possible failure portions by the pretest analyses. It can be seen from Fig.10 that the maximum tendon strain appears in the general portion(about azimuth 0° - 10°) at a height of EL.7.0m at a pressure of 1.5MPa, and the portion of the maximum tendon strain by pretest analyses almost agrees with the failure portion in the SFMT. The tendon strain distribution can not be obtained if the friction effects between tendon and concrete are not considered in the analysis. Therefore, it was confirmed that the friction model used here worked reasonably well. As the results, structural failure in the SFMT occurred in the one of the locations predicted from pretest analyses, as shown in Fig.8.

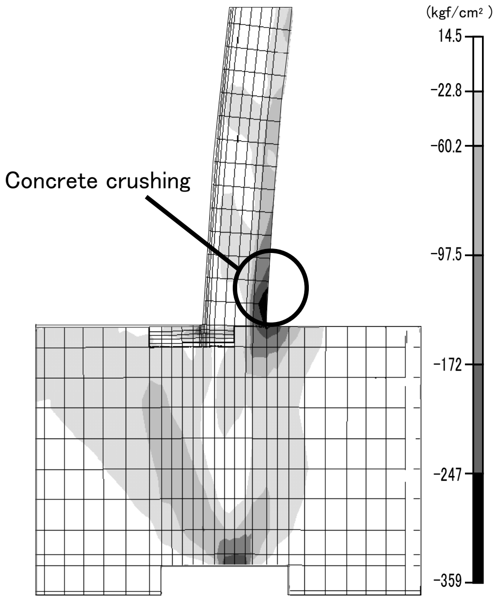


Fig.11 Minimum concrete principal stress contour by pretest analysis with wall - basemat juncture model

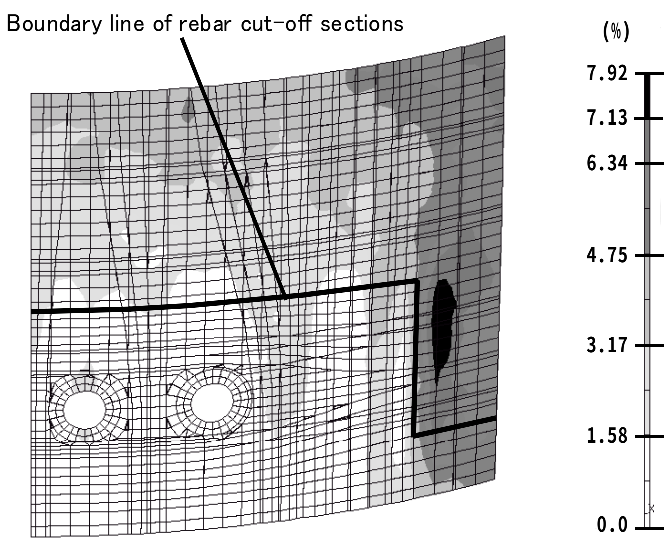


Fig.12 Inner hoop rebar strain contour by pretest analysis with M/S model

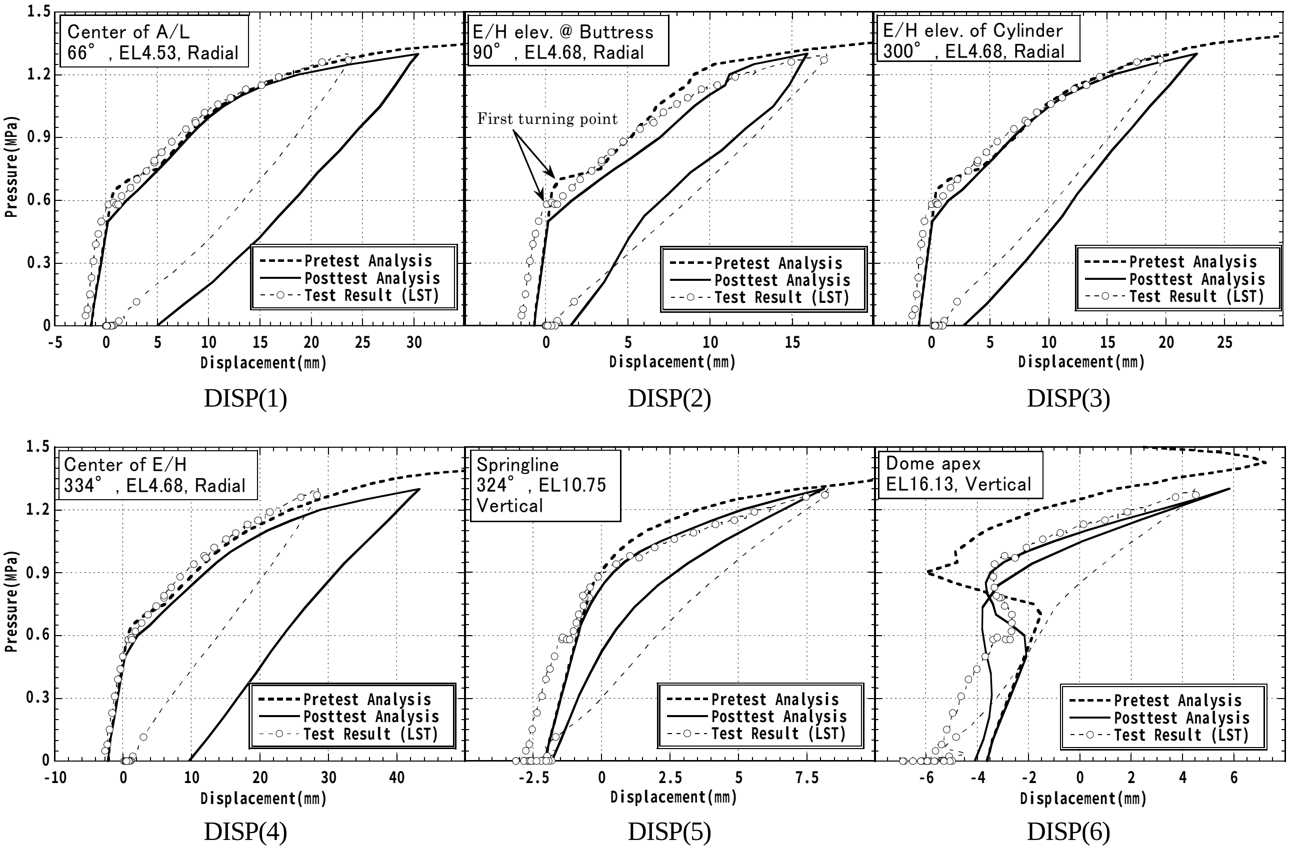


Fig.13 Comparisons of the pressure - displacement relationships

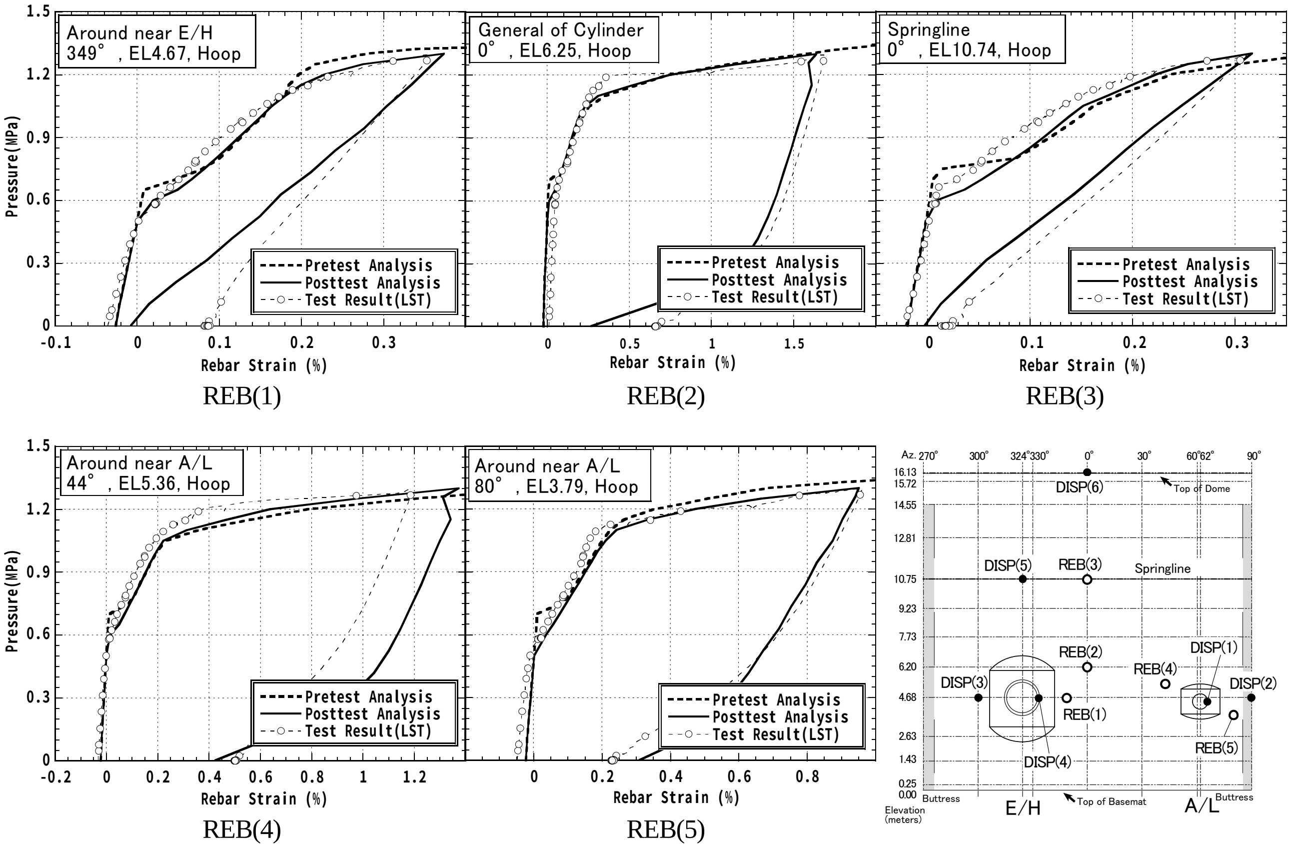


Fig.14 Comparisons of the pressure - inner hoop rebar strain relationships

7. CONCLUSION

The conclusions obtained from this analytical study are as follows:

1) Pretest analyses

(1) It was found that there is no possibility of causing a structural failure in the vicinity of openings, penetrations and the wall-basemat juncture, where discussions have been made for long time.

(2) Rupture of structural elements (tendon, rebar or liner) placed in the hoop direction at a height of about EL.7m at the pressure of 1.5MPa had been estimated to occur, while the ultimate pressure was about 1.42MPa in the SFMT.

(3) Pretest analysis predicted the nonlinear behaviors of the test results up to the ultimate state with good accuracy except a little difference at the first turning points at which cracking of concrete initiates.

2) Posttest analyses

(1) The posttest analysis, which took into account drying shrinkage of concrete, could improve the behavior by the pretest analysis at the first turning points and it's results moved to close to the test results fairly well.

(2) The effects on the dying shrinkage of concrete is preferability considered for evaluation of the nonlinear behavior of very complicated RC structures like PCCV. Those are affected by the ratio of reinforcement, concrete mix proportion, wall thickness and curing condition of concrete in the RC structures.

The structural nonlinear behaviors and structural failure mode of PCCV can be predicted with good accuracy by the analytical method developed here. However, performing this kind of analytical research, in general, requires enormous effort and labor. It was fortunately found from the SFMT and this analytical study that the ultimate pressure of PCCV can be simply approximated by the total sum of strength of steel materials in the hoop direction at the general portion.
Note: The test data is quoted from [2].

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