Post Test Evaluation of Pre-Stressed Concrete Containment Vessel Model with BARC Finite Element Code ULCA

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ABSTRACT

Containment safety assessment for beyond the design basis events has been a thrust research area for Indian Pressurized Heavy Water Reactor (PHWR) power programme. The paper presents the highlights of the post-test evaluation of Pre-stressed Concrete Containment Vessel (PCCV) model tested at Sandia National Labs, USA reported in NUREG 6810/6809 [1-2] in a Round Robin analysis activity in 2000-01. The analysis results obtained with BARC in-house finite element code ULCA are evaluated with the above published test results. In the pre-test analysis phase, the first liner tearing and three values of failure pressure predictions namely the upper bound, the most probable and the lower bound could successfully be made. This was an important requirement of the round robin analysis activity and these criteria can be further used to identify all the various failure modes, its location with the help of the observed sensor responses. The present post-test evaluation of the numerical results with the observed first liner tearing at the pressure of 2.5 Pd (model design pressure, Pd=0.39 MPa), Limit State Test (LST) pressure of 3.3 Pd and Structural Failure Mode Test (SFMT) pressure of 3.65 Pd shows that along with the PCCV global response, the local responses at the standard output locations are well correlated with these three limiting bounds of the estimates.

INTRODUCTION

A Pre-stressed Concrete Containment Vessel (PCCV) model (Fig 1) was constructed and tested at Sandia National Laboratory, USA under a collaborative research programme jointly sponsored by Nuclear Power Engineering Corporation (NUPEC) of Japan and the U.S Nuclear Regulatory Commission (NRC) as reported in NUREG 6810/6809 [1,2]. The 1:4 size scaled PCCV model with design pressure (Pd) of 0.39 MPa represents the pre-stressed concrete containment of 1100 MWe PWR plant located at OHI-3 Japanese station. The objective of the program was to study the model performance under the design basis accident (DBA) loads and further due to its over pressurization in case of beyond the design basis accident (BDBA) scenarios. An international round robin analysis was coordinated and the participants from different countries were asked to predict the ultimate load capacity of the PCCV model and structural response at the 55 specific standard output locations. Such studies help to validate the design and analysis procedures for the containment structure under the postulated DBA and further the assessment of the containment performance can be made for BDBA. In addition, it is important to study the various failure modes of the containment for BDBA, which would help to evaluate the containment suitability as a shielding cover and the risk assessment can be made. Such severe accident scenarios are though extremely low probable but its effect needs to be quantified for risk assessment and improving the public acceptance of nuclear power plants.

BARC, Trombay participated during the initial stage of this round robin analysis programme through invitation from Sandia National Laboratory, USA [3]. We submitted the mid term analysis report, Basha et al [4] and the pre-test predictions have been presented in SMiRT16 workshop and SMiRT17 conference proceedings, Basha et al [5,6] in addition the detail pre-test predictions are available in Basha et al [7]. Meanwhile the test results of the PCCV model have been released by USNRC in NUREG CR/6810 [1] and are available for the post-test evaluation of the earlier predictions. We present in this paper some salient observations on the PCCV model behavior with regard to our pre-test predictions and the subsequent post-test evaluations reported in Basha et al [8] with BARC in-house finite element code ULCA results. As per the original requirement of the round robin analysis activity [3] the following three sets of predictions were required to be made available by the participants before the test.

- Best estimate of the ultimate pressure for the model with 90% confidence.
- Lower bound conservative estimate of the minimum pressure, which the test model would reach during the test with 90% confidence.

- Upper bound estimate of the maximum pressure beyond which the test model is unlikely to remain in pressurized condition with 90% confidence.

To the best of our knowledge, BARC code ULCA could only successfully meet the most important requirement of the round robin analysis activity to identify the three levels of failure pressure, namely; the best estimate, the lower bound and the upper bound in the blind predictions during the pre-test phase. In addition, the failure sequence of PCCV model was required to be traced with regard to first cracking of concrete in cylinder due to hoop and meridional stress, first yield of hoop rebar in cylinder, first yield of meridional rebar in wall base junction, first concrete cracking of dome above and below 45 degree dome angle. Our pre-test results included all the above landmarks and we could also predict the sequential yielding of the liner and other steel members at different locations. A total 55 number of standard output locations (SOLs) were identified by the organizers [3] to evaluate the PCCV model inelastic structural performance with the help of recorded strain and displacement data. We had been earlier able to predict the PCCV model ultimate load in good agreement with the experiment and the response prediction are now shown to be in reasonable agreement with the test results at 40 SOLs. The numerical results of the PCCV structural response at the specific SOLs for the above mentioned three estimates of the failure pressures have further helped us to study the various failure modes of the PCCV model in a more rational manner.

The PCCV model test was conducted initially in 2000 where in first localized liner failure was observed at 2.50 Pd near E/H location. After the repair of the liner the test was further continued till further pressurization of the model was difficult due to large number of liner tearings resulting into high leakage. This was identified as limit state test (LST), which was observed at 3.30 Pd in October 2001. The PCCV test was subsequently continued after repairing the liner cracks and sealing the concrete cracks with polymer to obtain the ultimate structural collapse load, which was termed as structural failure mode test (SFMT) and the test was finally concluded in November 2001. The test medium during SFMT was water with limited nitrogen [2]. In this paper the response of PCCV model is described with respect to individual sensor response at the SOLs and the LST and SFMT pressures are well correlated with our best and upper bound estimates.

BARC Finite-Element Code ULCA

The in-house finite element code ULCA (Ultimate Load Capacity Assessment) has earlier been used for evaluating the ultimate load capacity of the reinforced and pre-stressed concrete containment of PHWR for beyond design basis accidents, Singh et al [9], Gupta et al [10,11]. It is based on the layered shell element for the composite structures similar to that developed by Figueiras and Owen [12], Mang [13-14] and Onate [15]. This code adopts the degenerate concept of formulating general isoparametric shell elements using selective integration scheme. In this code a layered approach with single mid point integration scheme is adopted for each layer which takes into account the non-linear stress profile in the thickness direction and material properties can be modeled as a discontinuous function of thickness. This layered approach has been used to represent the different concrete layers and thus it is possible to simulate progressive cracking through the shell thickness. Ultimate tensile strength and suitable tension stiffening parameters represent the concrete cracking behavior. The reinforcing steel is represented as a smeared layer of equivalent thickness with uni-axial strength and rigidity, thus global anisotropic behavior of shell and material non-linearity in steel rebars are accounted for. Various constitutive material models are employed to represent compressive behavior of concrete. Dual criteria for yielding and crushing in terms of stresses and strains are considered, which are complemented with a tension cut-off representation. Significant improvements were made in the code ULCA to make it suitable for the analysis of steel lined containments. The liners are modeled with steel layers on the inside containment wall with the assumption of no slip condition. The hardening parameters for steel members were made strain dependent to fit the specific stress-strain data for PCCV model. For concrete no data was available for strain hardening parameters from the organizers of the Round Robin exercise. However, provision exists in the code to fit the actual non-linear stress-strain data for concrete for refined analysis.

Finite Element Models for PCCV

In the present study two finite elements models have been used for the analysis of PCCV test model. Model-1 with most of the SOLs simulates the free field response in the quadrant from 90 degrees near buttress to 180 degrees near the steam and feed water line penetrations. Model-2 (Fig 2) simulates the quadrant from 270 degrees near buttress to 360 degrees with largest opening due to equipment hatch (E/H) around which the standard output locations are placed. This model with the largest penetration predicts the local behavior of the test model. The two finite element models intuitively evolved with engineering judgment to adequately represent most of the details of the PCCV test model with buttresses, large opening and discontinuity regions could successfully predict the expected behavior of the structure in the linear and non-linear regimes up to the ultimate (limit state) and the collapse (structural failure mode) pressures. The finite element mesh for Model-1 consists of 405 numbers of degenerate shell elements, with each element having eight concrete layers and nine smeared steel layers including inner liner across the thickness. The different steel layers consist of innermost layer of liner, two layers of inner and outer vertical bars, two layers of inner and outer hoop bars and two layers of tendons. In addition, the trim bars near the buttress were modeled as the ninth layer. This has resulted in 48 number of reinforcement layer patterns with 148 material properties depending on the orientation, size and position of the rebars in the containment model. Fixed boundary conditions were applied at the base of the structure and symmetry conditions are applied at the buttress end and the other end located 90 degrees away from the buttress. The finite element mesh for Model-2 covers the quadrant from 270 degrees to 360 degrees. The largest opening E/H has a diameter of 0.773m and is located at an azimuth of 324 degrees. This mathematical model consists of 508 layered shell elements with each element having 8 concrete layers and seven smeared steel layers including the liner across the PCCV model wall section. This discretization leads to 120 reinforcement layer patterns with 238 material properties depending on the orientation, position and size of rebars in the PCCV test model.

PCCV global performance during LST and SFMT

In the PCCV model test the first liner failure was noticed at 2.5 Pd prior to LST, which was near E/H opening due to high strain concentration. It may be noted that in our mid term analysis report submitted to Sandia National Laboratory by Basha et al [4], the first appearance of liner tearing predicted by BARC code ULCA was at 2.315 Pd for analytical Model-1. This preliminary analysis did not account for the stiffening effects due to pre-stressing tendons to compute the appearance of the first linear tearing. During this stage, one of the objectives was to obtain the peak strain/displacement regions for locating the transducers at the optimum locations in the PCCV test model on a simpler numerical model before the experiment. This premature localized liner failure observed in the test model could be due to the presence of liner studs and possibly due to inherent flaws and such failures have been reported for earlier Sandia containment model test by Dameron et. al. [16]. The lower bound pre-test predictions with due consideration to the stiffening effects of tendons with the analytical Model-2 was 2.75 Pd. As described earlier the lower bound pre-test predictions were made with the conservative assumption of ideal elasto-plastic behaviour of the steel members. Hence our conservative preliminary predictions for the PCCV inelastic analysis and the Model-2 lower bound estimate (range 2.315Pd - 2.75Pd) could envelop the first localized liner failure. The lower bound value for the liner failure in case of Model-1 was predicted as 3.15 Pd in which the localized strain gradients due to openings were not present.

The PCCV test was further continued after repairing the teared liner portion and the test model reached to a LST of 3.30 Pd in October 2000 as reported in [1]. During our pre-test analysis, Basha et. al. [5-7] all the subsequent predictions for the best estimate and the upper bound ultimate load were made with Model-1 and Model-2, where in the stiffening effects of tendons were accounted. For the best estimate predictions Model-1 reached ultimate load of 3.30 Pd, while the Model-2 predicted the ultimate load of 3.15 Pd. As mentioned earlier, these limit loads for the best estimates were obtained during the pre-test phase with the elastic-strain hardening plasticity model of steel members. It is noteworthy that the ULCA predictions for the best estimate of the ultimate load are in excellent agreement with LST pressure of 3.30 Pd at which large liner tearings have been reported at multiple number of locations [1,2] resulting into the observed leakage of ~ 1000% mass per day.

In SFMT the test model failed at 3.65 Pd with large vertical crack originating at cylinder mid-height at 6^o azimuth and propagated upwards and downwards in meridional directions, finally leading to a large circumferential crack at the base wall junction. This resulted into secondary shear failure of the test model. Our upper bound prediction accounts for the tendon and concrete interaction and predicts the collapse pressure of 3.65 Pd with Model-1 and 3.45 Pd with Model-2. Again it is illustrated that our upper bound predictions with the influence of additional pre-stress introduced by the dilating tendons are in very good agreement with the SFMT pressure of 3.65 Pd.

Based on the available test result of first localized liner tearing at 2.5 Pd, limit state test (LST) pressure of 3.30 Pd and ultimate collapse pressure or structural failure mode test (SFMT) pressure of 3.65 Pd, BARC code ULCA predictions are shown to be in very good agreement with the PCCV experimental results [1,2] during all the phases of tests. The lower bound, the best estimate and the upper bound limit loads predicted during the pre-test phase have very good correlation with the test observations namely the localized liner tearing, large scale liner tearings at multiple locations and structural collapse respectively as summarized in Table 1. Although the global behaviour of the PCCV model were shown to be consistent with the test observations during the pre-test stage; Basha et al [5-7], it is further desirable to evaluate the predicted response of PCCV at different locations with the observed test data of strains and displacements. It is shown that the PCCV model response is influenced with regard to specific location, relative stiffness of the concrete and steel members, load transfer mechanisms, inherent material and construction variability. At times it is noted that our best estimate and upper bound models influence the local inelastic response of the PCCV model.

Comparison of PCCV local behaviour

As mentioned earlier, there were 55 standard output locations (SOLs) [4] for displacement and strain predictions during pre-test stage and most of the important locations were included by us; Basha et. al. [5-7]. Some of the output locations could not be covered in the present analysis. Output location 1 where the vertical displacement was requested could not be provided since it is located at the top of basement. In our finite element models the basement has not been modeled and the cylinder-raft junction is treated as fixed. The responses at output locations 14 and 15 on the center of E/H and air lock (A/L) have not been predicted as the openings have been modeled without any cover. The tendons have been modeled as smeared layers in the present analysis; hence only the tendon peak strain values near the equipment hatch at the ultimate pressure are given and the detailed strain history at the standard output locations 48-55 for the tendons are omitted. The predicted responses obtained during pre-test phase with best estimate and upper bound analyses are compared with the responses of the sensors at 40 standard output locations.

Sensors response near base wall junction

For SOL#3 (Fig.14) at 1.43m from base wall junction the comparison of radial displacement between most probable prediction up to LST pressures is in very good agreement. However, the agreement is poor at lower pressures due to probable instrument drift errors. Similar observation is noted for SOL#5 (Fig. 15) at 4.68 m elevation. The strain response in the inner rebar at 1.43 m from base wall junction at SOL#20 (Fig.17) is in good agreement with the upper bound prediction. This is due to the fact that the strain response is strongly influenced by the tendon dilation for the relatively flexible rebar members while the overall displacement across the wall section is an average representation of the overall section. For outer rebar meridional strain at 1.43 m elevation SOL #21 (Fig.18), the experimental data shows an initial drift which after applied correction (with an uniform shift in the LST response equivalent to initial drift) is closer to the upper bound prediction while the most probable strain prediction is higher.

Sensors response near mid height of cylinder

Radial displacement response at SOL#6 (Fig.19) is shown to be bounded by the most probable and the upper bound responses. However, the agreement with the upper bound model is closer up to the LST pressure. For SOL #23 (Fig.22) the strain in rebar at mid height of PCCV is closely predicted with our upper bound model. The rebar strain response at mid height buttress location SOL#33 (Fig24) is also predicted with upper bound model in close agreement with response up to LST pressure. The free field liner strains at SOL #43 (Fig. 27) are again shown to be in excellent agreement with upper bound prediction. It may be noted that in the initial elastic portion (< 2.0Pd), the difference between the most probable and upper bound predictions is marginal. Due to tendon dilation in the inelastic regime the strain predictions are higher than the upper bound predictions for the most probable model. However, at SOL # 44 (Fig.29) sensor is located near buttress and hence the most probable prediction is in close agreement with the experimental results up to the LST pressure.

Sensors response at the spring line and dome

Radial displacement and vertical displacement responses near spring line at SOL #7 and #8 (Fig. 31-32) are observed to be in good agreement with the experimental test data up to LST pressure for most probable model. Near the spring line buttress location SOL #13(Fig. 34) the comparison of radial displacement due to most probable model is in good agreement with the experimental data.

The hoop strain response at SOL# 24(Fig. 36) near spring line is shown to be in good agreement with our upper bound predictions. At this elevation the strain in liner in hoop direction SOL# 41(Fig. 38) is again observed to be in reasonably good agreement with the upper bound prediction model. Similar observation is noted for hoop strain at SOL #27(Fig. 40) and meridional strain at SOL #29(Fig. 42), which are located in dome at 45^o from the spring line. Thus again the upper bound model shows very good agreement of strains with the experimental results up to the LST pressure, which highlights the influence of tendon dilation on the rebar members near the ultimate state and confirms our inference as described in the earlier section.

CONCLUSIONS

Our three estimates of the failure pressure predictions for the PCCV due to the over-pressurization could successfully meet the round robin analysis requirements and the structural behaviour could be reasonably simulated in agreement with the test-results. For the global structural failure the best estimate model has been shown to yield the failure pressure in agreement with the PCCV test results. However, the influence of tendon dilation and the resulting concrete steel member interaction is important for predicting the local structural response at the specific sensor locations (SOL) in agreement with the experimental observations. The present approach evolved with our in-house FE code ULCA can be used for the containment performance evaluation for DBA/BDBA scenarios.

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Table 1: Summary of pressure predictions by ULCA code and test results for the PCCV model as function of design pressure (Pd=0.39 MPa)

Test Sequence	Expt. Observation	Numerical Predictions		
		Model 1	Model2	Remarks
First Liner tearing pressure	2.50 Pd	3.15Pd 2.315Pd*	2.75Pd	Lower Bound
LST pressure	3.30 Pd	3.30Pd	3.15Pd	Most Probable
SFMT pressure	3.65Pd	3.65Pd	3.45Pd	Upper Bound

* During mid term analysis feed back phase a value of 2.315 Pd was predicted without accounting tendon stiffness

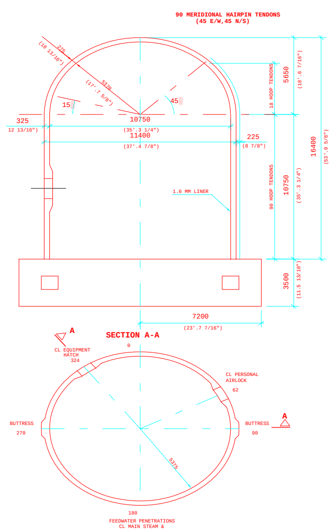

Fig.1. Outline Sketch of PCCV Model

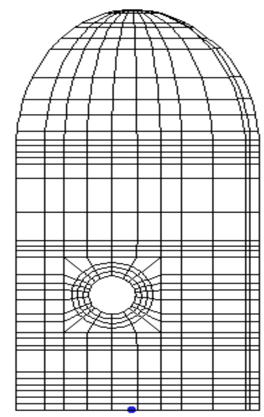

Fig.2. FEM Shell Model of PCCV

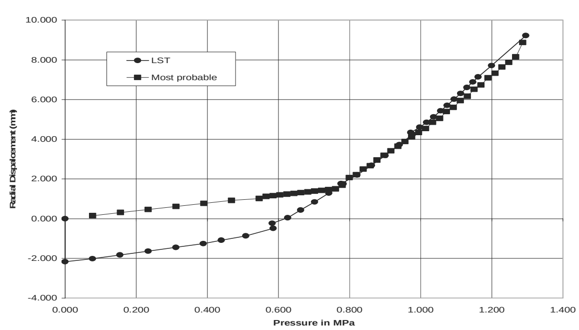

Fig. 3 Pressure vs Displacement at SOL#3

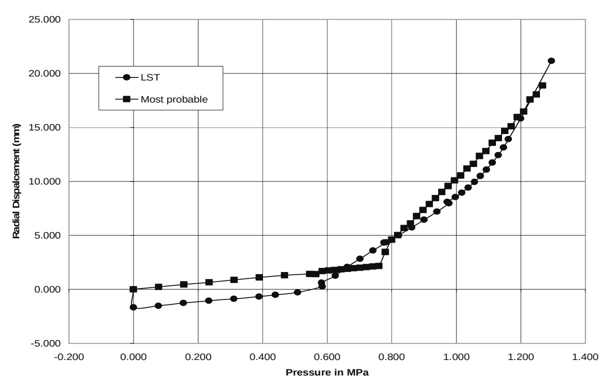

Fig. 4 Pressure vs Displacement at SOL#

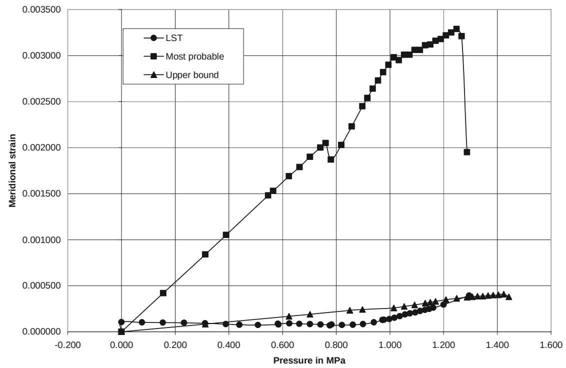

Fig. 5 Pressure vs Strain at SOL#20

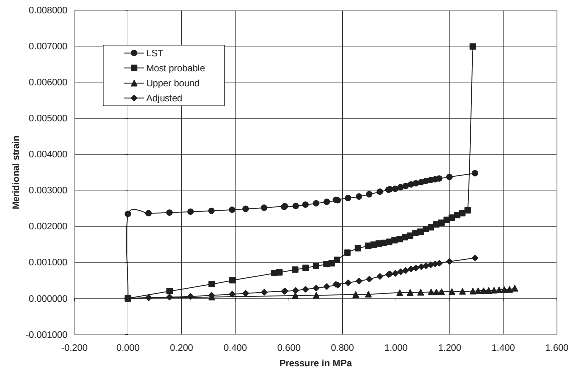

Fig. 6 Pressure vs Strain at SOL#21

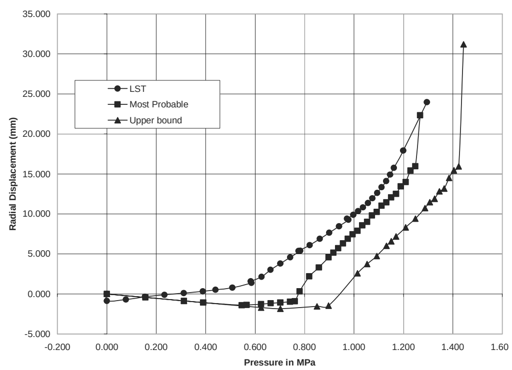

Fig. 7 Pressure vs Displacement at SOL#6

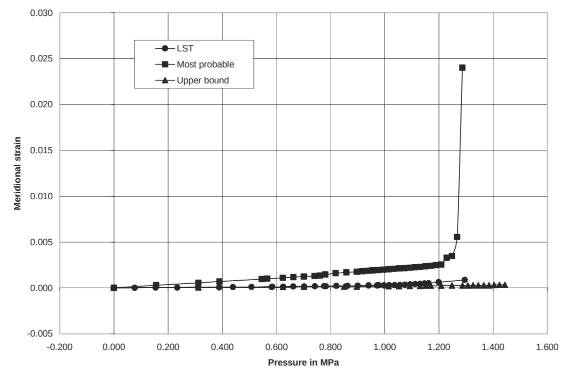

Fig. 8 Pressure vs Strain at SOL#23

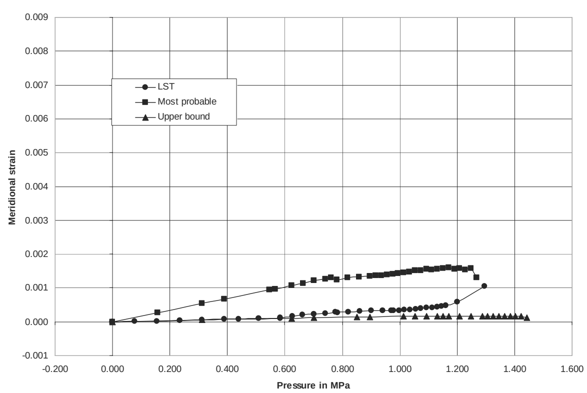

Fig. 9 Pressure vs Strain at SOL#33

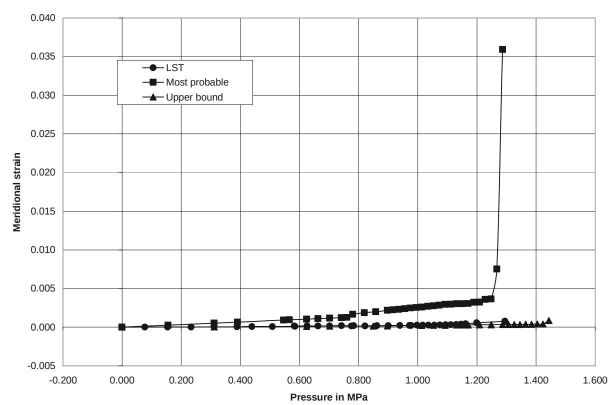

Fig. 10 Pressure vs Strain at SOL#43

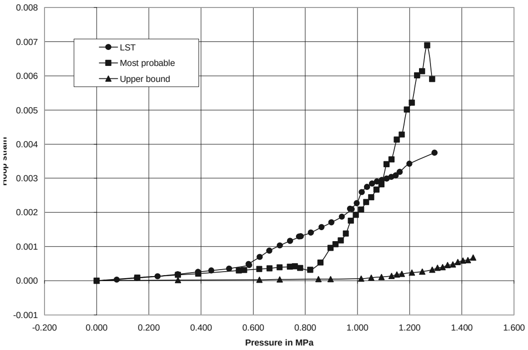

Fig. 11 Pressure vs Strain at SOL#44

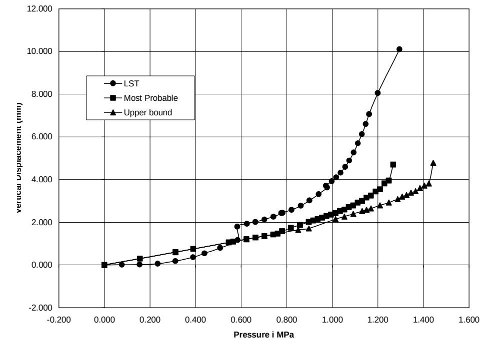

Fig.12 Pressure vs Displacement at SOL#8

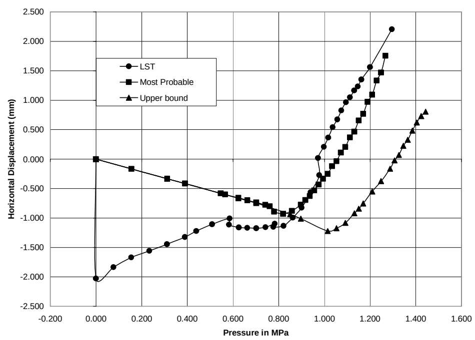

Fig. 13 Pressure vs Displacement at SOL#9

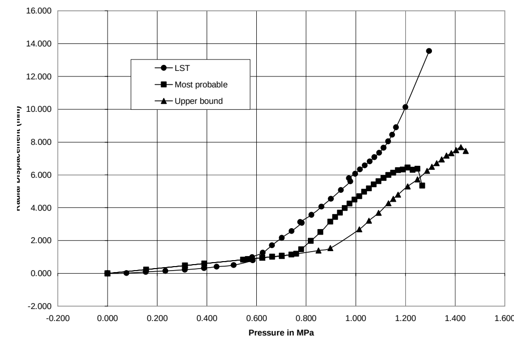

Fig. 14 Pressure vs Displacement at SOL#13

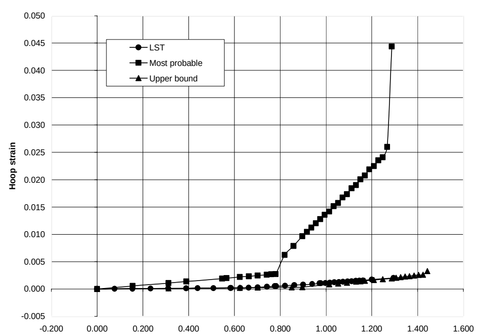

Fig. 15 Pressure vs Strain at SOL#24

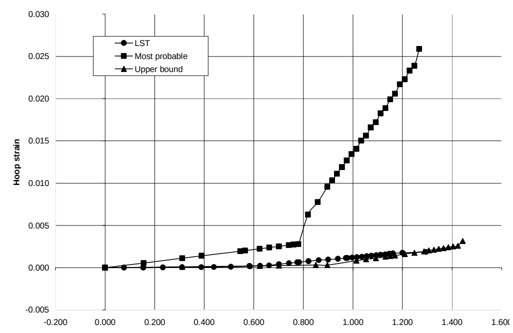

Fig. 16 Pressure vs Strain at SOL# 41

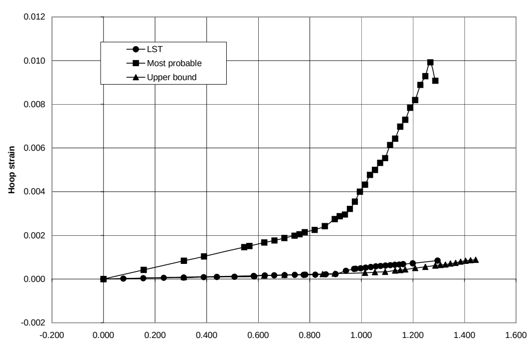

Fig. 17 Pressure vs Strain at SOL#27

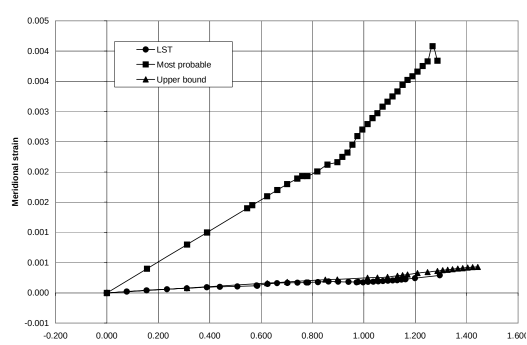

Fig.18 Pressure vs Strain at SOL#29