Evaluation of Ultimate Load Bearing Capacity of the Primary Containment of a typical 220MWe Indian PHWR

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ABSTRACT

This paper presents the analysis of the Inner Containment structure of a typical 220 MWe Indian PHWR for the purpose of evaluating its ultimate load bearing capacity under beyond design accident scenario. The methodology adopted for the non-linear analysis of the prestressed concrete inner containment structure including the various issues, viz. behaviour of concrete under compression and tension, tension stiffening, cracked shear modulus etc. have also been discussed in this paper. Based on the detailed analysis, the safety margin of the inner containment structure under beyond design accident scenario has been evaluated.

INTRODUCTION

The typical Indian PHWRs of 220MWe series are having a double containment system in which the inner containment (IC) is in prestressed concrete and the outer containment (OC) is in reinforced concrete. The IC is a cylindrical structure of 48.435m height, capped with a prestressed concrete segmental dome with four large openings to facilitate the replacement of steam generators should it be needed at later date. The OC is a reinforced concrete structure having similar shape of that of IC and envelops the IC (Fig.1). The IC is designed to resist the design basis accident pressure and temperature due the various postulated accident scenario viz. LOCA, MSLB etc.

In order to study the margin available for the IC beyond the design basis accident scenario, a non-linear analysis of the IC has been carried out considering the reinforcements & prestressing steel embedded in the concrete. The analysis model has been developed using a degenerated layered shell element in which the reinforcements/prestressing steels are modeled using a smeared approach. Various aspects of behaviour of reinforced/prestressed concrete under tension and compression viz. tension stiffening, compression behaviour of the concrete etc., have been accounted for using the state of the art knowledge available in the literature.

This paper presents a comprehensive description of the analytical model used in the evaluation of the ultimate load bearing capacity of the IC under the beyond design accident scenario with due consideration to the aspect of analytical modeling of the behaviour of the reinforced concrete shell under tension & compression. The behaviour of the IC has been presented in this paper at different stages of loading beyond the design pressure. The additional factor of safety of the design of the inner containment beyond the design basis accident has been discussed in this paper.

ANALYSIS METHODOLOGY

Geometrical Idealisation

The containment is a thin shell structure in which the stresses developed due to different loads are mainly membrane in nature and the normal stresses in thickness direction are negligible. Radial (transverse) stresses develop in the containment structure by means of embedded curved cables (curvature effect). Radial tensile stress also occurs due to sudden transition in thickness of the shell (transition effect) as well as due to presence of voids due to cable sheath (stress concentration effect). The radial tensile stresses are more during the construction time and reduce under the internal pressure due to accident condition. Since the magnitude of the radial stresses both under compression and tension is negligible, a shell model using 8 noded degenerate quadratic shell element has been adopted for evaluating the non-linear response of the structure. Transverse shear deformations are considered in the element formulation by assuming that the normal to the mid-surface of the element remains straight but not necessarily normal during deformation. Three translational and two rotational degrees of freedom are considered per node. The layering system helps in introducing the reinforcements in relevant layers and also in tracing the progress of cracking through the depth of the shell.

Material Modelling

The materials used are concrete and reinforcing / prestressing steel. The constitutive behaviour of all the above materials are described below. Properties of concrete in tension and compression are modelled separately.

Concrete in Tension

Concrete behaves linearly upto its tensile strength. Then it cracks and the tensile stress reduces gradually to zero with increase in strain. The most common procedure to consider this phenomenon is through a stress based cracking criterion in conjunction with a smeared crack model where the cracks in concrete are assumed smeared over the element[1]. Cracks are assumed to form in planes perpendicular to the direction of maximum principal tensile stress as soon as this stress reaches the specified concrete tensile strength f_t. Taking 1 and 2 as the two principal directions in the plane of the structure, the stress - strain relationship for concrete cracked in the direction 2 is

$$\begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \\ \tau_{13} \\ \tau_{23} \end{Bmatrix} = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 \\ 0 & E & 0 & 0 & 0 \\ 0 & 0 & G_{12}^C & 0 & 0 \\ 0 & 0 & 0 & G_{13}^C & 0 \\ 0 & 0 & 0 & 0 & \frac{5G}{6} \end{bmatrix} \begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ v_{12} \\ v_{13} \\ v_{23} \end{Bmatrix} \quad \text{....(1)}$$

When the tensile stress in the direction 2 reaches the value f'_t a second cracked plane perpendicular to the first one is assumed to form, and the stress strain relationship becomes

$$\begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \\ \tau_{13} \\ \tau_{23} \end{Bmatrix} = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & G_{12}^C/2 & 0 & 0 \\ 0 & 0 & 0 & G_{13}^C & 0 \\ 0 & 0 & 0 & 0 & G_{23}^C \end{bmatrix} \begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ v_{12} \\ v_{13} \\ v_{23} \end{Bmatrix} \quad \text{....(2)}$$

The cracked concrete is anisotropic and these relations must be transformed to the global x-y axis.

Tension Stiffening

Due to bond effects, even after formation cracks, concrete carries between cracks a certain amount of tensile force normal to the cracked plane. The concrete adheres to the reinforcing bars and contributes to the overall stiffness of the structure. Several approaches based on experimental results have been employed to simulate this tension stiffening behaviour. A gradual release of the concrete stress component normal to the cracked plane is adopted in this work. The process of loading and unloading of cracked concrete is also taken care of. Unloading and reloading of cracked concrete is assumed to follow the linear behaviour with a fictitious elasticity modulus E_i given by[1]

$$E_i = \frac{\alpha f_t'\left(1 - \frac{\varepsilon_i}{\varepsilon_m}\right)}{\varepsilon_i} \qquad \varepsilon_t \le \varepsilon_i \le \varepsilon_m \quad \text{....(3)}$$

Where α, ε_m are tensions stiffening parameters, ε_i is the maximum value reached by the tensile strain at the point considered and ε_t is the tensile strain at maximum tensile stress (Fig 2). If the crack closes, i.e. if the strain component normal to the crack plane becomes negative, the concrete acquires the non-cracked behaviour in the corresponding direction, but the crack direction and the maximum tensile strain continues to be stored. The value ε_i can be readily modified to simulate bond deterioration during reloading. The normal stress (and /or σ_2) is obtained by the following expression (Fig.2)

$$\sigma_1 = \alpha f_t'\left(1 - \frac{\varepsilon_1}{\varepsilon_m}\right), \quad \varepsilon_t \le \varepsilon_1 \le \varepsilon_m$$

or by,

$$\sigma_1 = \sigma_i \frac{\varepsilon_1}{\varepsilon_i}, \quad if \quad \varepsilon_1 \le \varepsilon_i \quad \text{....(4)}$$

where ε_1 is the current tensile strain in material direction 1.

The experimental tests have shown large variations in the above coefficients. For the tension-stiffening constant, ε_m a fixed value of 0.002 is employed whereas α has been assumed to be 0.7 for all the layers [1]. The value of f'_t has been taken as the design tensile strength of concrete.

Cracked Shear Modulus

Experimental results indicate that a considerable amount of shear stress can be transferred across the rough surface of cracked concrete. Also, the dowel action of steel bars contributes to the shear stiffness across cracks. Tests have shown that the primary variable in the shear transfer mechanism is the crack width, although aggregate size, reinforcement ratio and bar size also have an influence. A common procedure to account for aggregate interlock and dowel action in a smeared cracking model is to attribute an appropriate value to the cracked shear modulus G^c. In the present work a similar approach is adopted, where the cracked shear modulus is assumed to be a function of the current tensile strain [1].
For concrete cracked in the 1 direction,

$$G_{12}^C = 0.25G(1-\varepsilon_1/0.004):G_{12}^C = 0 \; if \; \varepsilon_1 \geq 0.004$$

$$G_{13}^C = G_{12}^C$$

$$G_{23}^C = \frac{5G}{6} \qquad \text{....(5)}$$

Where G is the uncracked concrete shear modulus and ε_1 is the tensile strain in the direction 1. For concrete cracked in both directions,

$$G_{12}^C = 0.25 * G(1-\varepsilon_1/0.004); G_{12}^C = 0 \; if \; \varepsilon_1 \geq 0.004$$

$$G_{23}^C = 0.25 * G(1-\varepsilon_2/0.004); G_{23}^C = 0 \; if \; \varepsilon_2 \geq 0.004$$

$$G_{12}^C = 0.5 * G_{13}^c; \; G_{12}^C = 0.5 * G_{23}^C \; if \; G_{23}^C < G_{13}^C \qquad \text{....(6)}$$

If the crack closes the un-cracked shear modulus G again assumed in the corresponding direction.

Compressive Behaviour of Concrete

The most common approach to model the compressive behaviour of concrete is through a plasticity-based model. This requires a yield criterion and a flow criterion for obtaining the response beyond the yield.

A linear elastic, plastic hardening model is generally employed. In the present application a perfectly plastic model is considered for σ_0 (σ_0 = the ultimate compressive strength under uni-axial compression) since the shell is stressed predominantly in tension and additional computations required when using a strain hardening model can be avoided. Concrete is assumed to be elastic till the equivalent stress reaches the uni-axial compressive strength of the material. From then on, perfect plasticity is assumed.

Yield Criteria

In the present analysis using thick plate and shell elements, transverse shear effects are taken into account and therefore, a tri-axial yield criterion needs to be employed. This criterion is formulated in terms of the first two stress invariants and only two material parameters are involved in its definition.

$$f(I_1, J_2) = [\beta(3J_2) + \alpha \; I_1]^{0.5} = \sigma_O \qquad \text{....(7)}$$

Where α, β are material parameters and σo is the equivalent effective stresses taken as the compressive stress from the uni-axial test. In term of principal stresses the expression for the yielding can be written as,
The stress state in thick plates and shells is usually not far from the bi-axial one, since the stress normal to the

$$\beta[(\sigma_1^2+\sigma_2^2+\sigma_3^2)-(\sigma_1\sigma_2+\sigma_2\sigma_3+\sigma_1\sigma_3)]+\sigma(\sigma_1+\sigma_2+\sigma_3)=\sigma_O^2 \qquad \text{....(8)}$$

middle plane (σ_z) is negligible. Therefore it is reasonable to obtain the material parameters by fitting bi-axial test results. The uni-axial compression test and the biaxial test under equal compressive stresses ($\sigma_1 = \sigma_2$) are used to define the constants. For practical purposes a relation can be assumed between the equal bi-axial yield stress f_{cp} and the uni-axial yield stress $f^1{}_c$.

$$f_{cp} = 1.16 \text{ (to } 1.20\text{)} f^1{}_c$$

In such cases the yield condition is a function only of one material parameter ($\sigma_0 = f^1{}_c$) which is the most reliable constant to characterise the concrete behaviour and is easily obtainable from the experimental tests. If Kupfer's[3] results are employed, $f_{cp} = 1.16 f^1{}_c$, then the use of Eq. (8) gives

$$\alpha = 0.335\ \sigma_0\ ,\ \beta = 1.335$$

and Eq. (7) can be written in terms of the stress components as

$$f(\sigma) = \{1.355[(\sigma_x^2 + \sigma_y^2 + \sigma_x\sigma_y) + 3(\tau_{xy}^2 + \tau_{xz}^2 + \tau_{yz}^2)] + 0.355\sigma_0(\sigma_x + \sigma_y)\}^{0.5} = \sigma_o \qquad \text{....(9)}$$

The analytical result obtained using the above expression is compared with the experimental results of Kupfer in bi-axial stress space and is shown in Fig. 3. In the perfectly plastic model, σ_0 is taken as the ultimate stress $f^1{}_c$ obtained from a uni-axial compression test.

Flow Rule

Plastic flow of yielded concrete is assumed to follow the associated flow rule. The plastic strain increment is defined as

$$d\varepsilon^p{}_{ij} = d\lambda\ (\ \partial f(\sigma)\ /\ \partial\sigma_{ij}) \qquad \text{....(10)}$$

Where $d\lambda$ is a proportionality constant, which determines the magnitude of the plastic strain increments and the gradient defines its direction perpendicular to the yield surface. The yield function is dependent on the instantaneous stress values. The current stress-function $f(\sigma)$ is the yield condition. Detailed formulation of the flow rule as well as hardening rule is given in reference [1].

Crushing Condition

Crushing type of concrete fracture is a strain-controlled phenomenon. Crushing of concrete occurs when the compressive strains (in terms of effective strains) exceed the limiting strain obtained as,

$$\beta(3J_2^{'}) + \alpha I_1^{'} = \varepsilon_u^2 \qquad \text{....(11)}$$

where $I_1^{'}$ and $J_2^{'}$ are strain invariant and ε_u is the ultimate total strain extrapolated from uni-axial test results. Using the material parameters α and β determined from Kupfer's results, the crushing condition is expressed in terms of total strain components as

$$1.355[(\varepsilon_x^2 + \varepsilon_y^2 + \varepsilon_x\varepsilon_y) + 0.75(\gamma_{xy}^2 + \gamma_{xz}^2 + \gamma_{yz}^2)] + 0.335\,\varepsilon_u\,(\varepsilon_x + \varepsilon_y) = \varepsilon_u^2 \qquad \text{....(12)}$$

When ε_u the specified ultimate strain value (0.0035) is reached, the material is assumed to lose its entire characteristic strength and rigidity.

Reinforcing and Prestressing Steel

The reinforcing bars/prestressing steel tendons are considered as steel layers of equivalent thickness in the present model. Each steel layer has a uni-axial behaviour resisting only the axial force in the bar direction. A linear elastic and plastic hardening behaviour is assumed for the reinforcing/prestressing steel.

THE NON-LINEAR ANALYSIS

One half of the containment structure has been modelled on account of symmetry. Complete fixity has been assumed at the base where the containment wall is connected to the raft. The inner containment structure is subjected to two types of loading i.e., constant load consisting of prestress and self weight of the structure and variable load consisting of internal pressure. The temperature load corresponding to the accident condition has not been considered in the present study. The internal pressure, representing the accident pressure needs to be applied gradually. Hence, special provision is made in the finite element program to increment only the internal pressure.

Initial elastic analysis is performed applying an initial load increment. Strains and stresses are computed. Separate routines are employed to compute the current material properties of the uncracked, cracked concrete layers and elastic or plastic steel layers. After the evaluation of the current properties, stresses are computed and integrated to obtain residual forces. The current material state is stored.

The residual forces thus obtained represent a deviation from equilibrium. Convergence of the iteration is checked with respect to norm of displacements and permissible tolerance. If the convergence is not satisfied, the residual forces are applied back on the structure and the above solution process is repeated. If the convergence is achieved, then the next load increment is applied. Stiffness matrices are recomputed and the process is repeated. It may be noted that the stiffness matrix is formulated at the beginning of each increment. In addition, it is reformulated after specified number of iterations to accelerate the convergence.

The problem is highly non-linear. To have a control on the load increment size, the modified Newton - Raphson procedure is employed. Load increments have been varied as the internal pressure increases. Initailly, a load increment of 0.25 times the design pressure has been considered upto the design pressure. Subsequently, the load increments have been varied from 0.1-0.01 times the design pressure till the failure load is reached. A tolerance of 5 % of displacement norm is found to be satisfactory. [Ref.1]. However, to avoid any divergence of results in the initial portion of the load deformation curve, a stringent tolerance of 0.5% of displacement norms is applied throughout the analysis. Since it is difficult to achieve this tolerance near failure load, a higher tolerance of 5% has been used.

ANALYSIS RESULTS

Response of Structure under Prestressing Forces

The non-linear analysis for dead load and prestress load (Constant load) has been carried out considering a single load increment. Cracking is observed mainly in the outer layer of the ring beam under prestress loading. The zones showing the extent of cracking in the outer layer are presented in Fig.4.

Response of Structure under Internal Pressure

The response of the structure upto an internal pressure of around 1.5 times the design pressure is linear. The cracks that appear on the outer layer of IC around the ring beam region under prestress slowly get closed with the increase of internal pressure. A few cracks appear in the inner face of the dome near the steam generator opening as may be seen in Fig.5.

Response of Structure at the time of Initiation of Reinforcement Yielding

The initiation of yielding of reinforcement steel is observed at an internal pressure value of 1.75 times the design pressure on the outer face of the Dome near the SG opening.

Response of Structure at the Initiation of through-and-through Cracking

The initiation of the through-and-through cracking has started at an internal pressure value of 1.65 times the design pressure. However, the cracking has been detected in the zone very close to the SG opening. The zones of significant through-and-through cracks develop at a load factor of 1.8 as shown in Fig.6.

Response of Structure prior to Failure

The response beyond the load factor of 1.85 is highly non-linear. With the increase of internal pressure, the cracking in the dome region increases drastically beyond the load factor of 1.85.

The analysis failed to converge beyond a load factor of 1.97 due to excessive yielding of reinforcements in the dome region mainly in the thickened portion of the dome region around the SG opening. The extent of through-and-through cracks at the time of failure is shown in Fig.7. The deformed shape of the structure prior to failure is shown in Fig.8.

DISCUSSION OF ANALYSIS RESULTS

Fig.9 shows the plot of non-linear response of structure under internal pressure at the location of maximum deformation. It is observed from the load-deflection curve that the response of the Inner Containment Structure is linear between points 1 and 2. Initiation of reinforcement yielding starts at point 3 at load factor 1.75. Through the thickness cracking is observed at point 4 at load factor 1.80). The structural failure takes place at point 5 at a load factor of 1.97. Fig.10 shows the corresponding load-deformation (vertical) curve at the crown of the dome.

CONCLUSION

Based on the present study, it is concluded that the primary containment of a typical 220Mwe Indian PHWR can withstand an internal pressure of 1.97 times its design pressure before structural failure. The mode of failure in the structure is due to excessive yielding of reinforcements.

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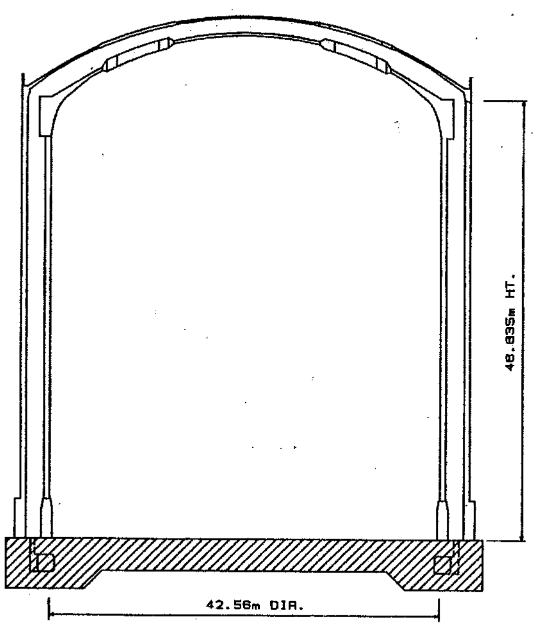

FIG. 1: INDIAN PHWR CONTAINMENT SYSTEM

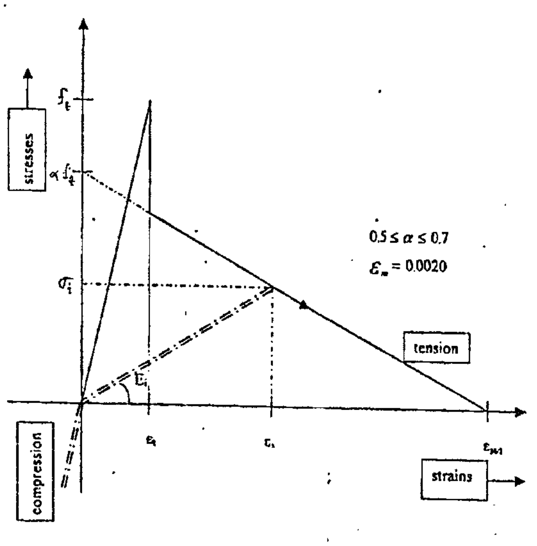

Fig. 2 : Loading and Unloading Behaviour of Concrete Illustrating Tension Stiffening Behaviour

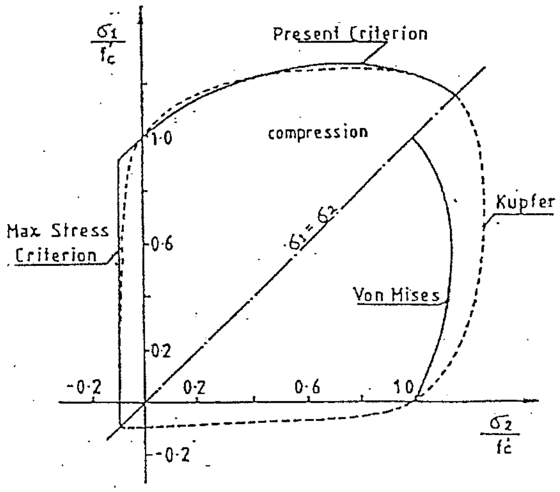

Fig:3: Comparison of the Theoretical Model with that of the Experimental Results of Kupfer in Bi-directional stress space

Fig: 4 : Extent of Cracking in the Outer Layer of Containment under prestress loading

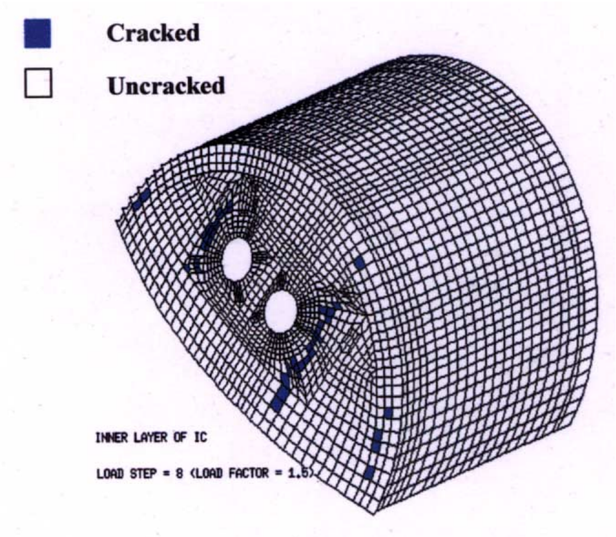

Fig: 5: Extent of Cracking in the Inner Layer of Containment

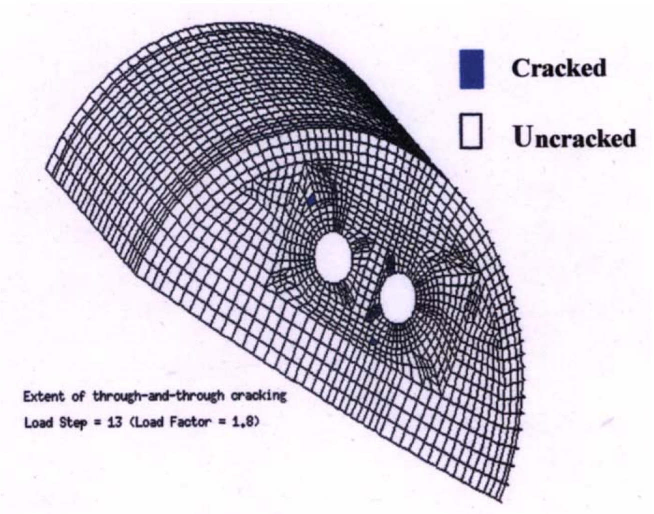

Fig: 6 : Extent of thru'-n-thru' Cracking in the Containment

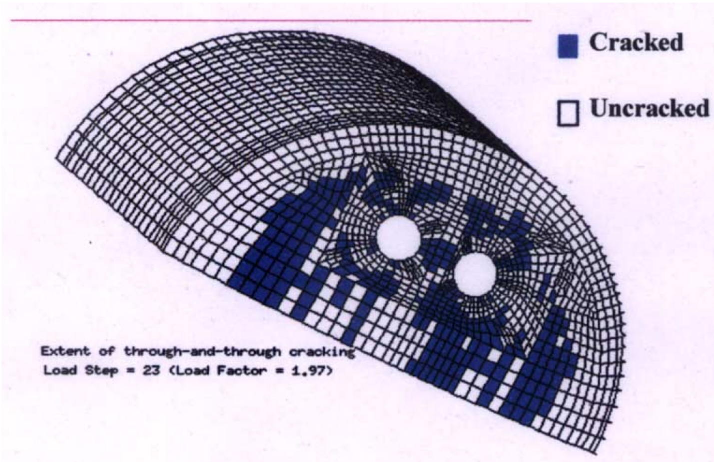

Fig: 7 : Extent of thru'-n-thru' Cracking in the Containment

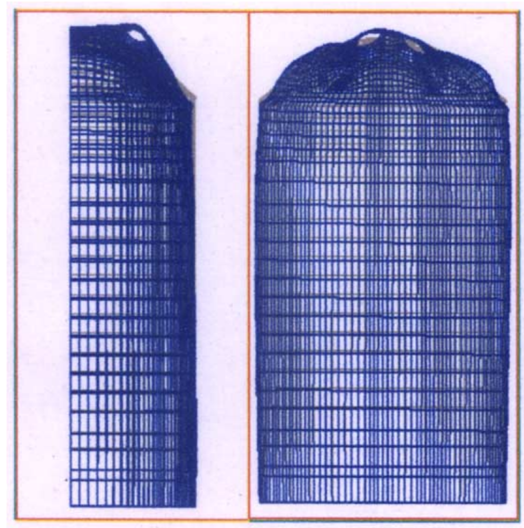

Fig. 8: Deformed Shape of the Containment at Failure Load

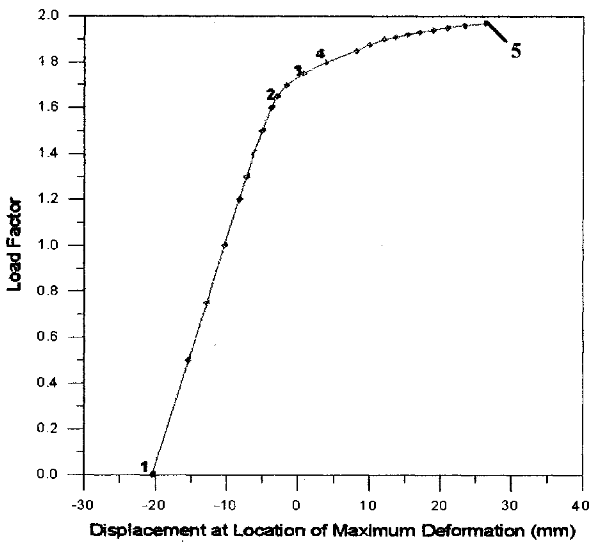

Fig.9: Load-Deformation Curve at the Location of Maximum Deformation

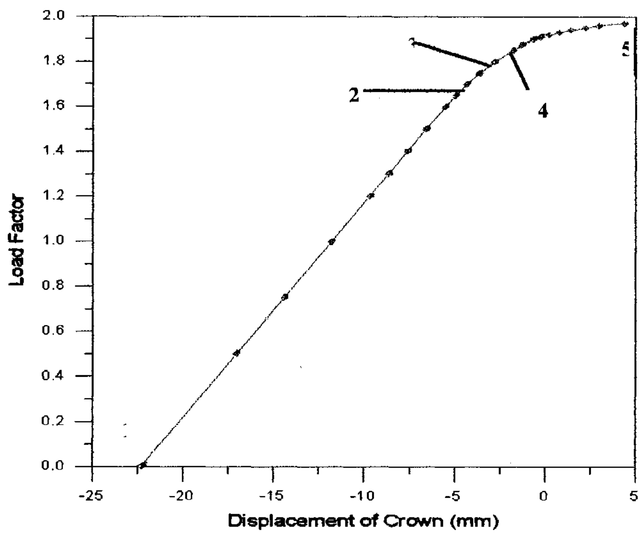

Fig. 10: Load-Deformation curve at Crown