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## ULTIMATE LOAD CAPACITY ASSESSMENT OF INDIAN PHWRs - SOME PRE TEST RESULTS

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### INTRODUCTION

The 235 MWe and 500 MWe Pressurized Heavy Water Reactors (PHWRs) being constructed in India have prestressed/reinforced concrete double containment design to withstand the pressure and thermal load in the event of loss of coolant accident and main steam line break and thus control the release of radioactivity. The containment performance is ensured for the above design basis accidents through suitable analytical methods followed by reactor building leak test to establish the design and construction. However, in view of the growing concern in light of Three Mile Island and Chernobyl accidents assurance to public and environmentalists must be given even for severe beyond the design basis accidents. Moreover it is necessary to know the reserve strength and failure mode of containments to assess the extent of damage for these extremely low probable accidents. The present paper highlights some of the studies carried out on concrete structures to predict the ultimate load capacity. Various constitutive models such as crushing, tensile cracking and shear stiffening are studied and effect of these parameters on the reserve strength of structures is brought out. Further a global approach is used to analyse the outer containment wall model of Narora 235 MWe PHWR with a suitable development of layered degenerate shell finite element where the stiffeners and cables are suitably represented and a smeared cracking model is applied. The results of this study are presented in detail which forms the basis for further analysis to study the local failure modes of the containment models. The emphasis in this program is on experimental validation hence the computer code is checked for various benchmark experimental and analytical results.

### LAYERED SHELL FINITE ELEMENT FOR REINFORCED CONCRETE

In order to build up capability to predict the ultimate load of complex structures such as containment it is necessary to develop capability for preliminary prediction of limit loads for various reinforced and prestressed concrete structures. The global limit load analysis helps to predict the local failures near openings such as main air lock and local regions of high stress concentration on a local model. Thus it is possible to demonstrate that failure is not catastrophic. In view of this, it was decided to adopt all the suitable constitutive models such as concrete cracking and crushing in a three dimensional layered degenerate shell element formulation. Accordingly a smeared plasticity model based on stress/strain failure [1,2] criteria was used in the

formulation. In this case the yield function depends on mean normal stress and shear stress invariants. For tensile failure mode a tension cut off model was used with tension stiffening parameters to account for load bearing capacity of cracked concrete section based on reported experimental results. In the present problem smeared layered approach helps to model the tendons at appropriate locations.

## CASE STUDIES

### (1) Parabolic Shell With Uniformly Distributed Load

A parabolic shell with variable thickness (fig.1) subjected to uniformly distributed pressure is analysed with the present finite element code with 36 elements and longitudinal, transverse and shear tendons with 17 smeared layer patterns and 8 equal concrete layers. This shell has been analysed by various investigators [1,3] and results have been validated against experiment conducted by Hedgren et al [4]. Normal deflections of crown and edge are shown in fig.2 for geometric linear as well as geometric nonlinear case. The crown deflection vary linearly up to a load factor of  $F=1.0$ . Above  $F=2.8$  the crown deflection increases more rapidly until failure occurs. As shown in fig.1 for load factor of  $F=0.4$ , the structure behaves in linear elastic manner with no permanent deflection. At higher loads crown deflection is significant. This is due to flexural cracking and formation of longitudinal yield line. The ultimate failure load factor predicted by present code is  $F=4.33$ , which compares well with the experimental load factor of  $F=4.4$  [4]. The ultimate load capacity of the shell is predicted accurately if the increase in internal moment on account of vertical uplift of shell crown is taken into account in geometrical and material nonlinear analysis. For geometrical linear and material nonlinear analysis the ultimate load factor is only 3.2 which is not true representation of structure's reserve strength as noted in the experiment.

### (2) Reinforced Concrete Nuclear Containment

A nonlinear analysis of a reinforced concrete nuclear containment subjected to internal pressure was carried out with the present finite element code with 24 axial divisions to account for variation in reinforcement. The circumferential and hoop reinforcements were modelled into four steel layers across the thickness. The present results compare well with the analytical results (table 1). Up to about 2.1 kg/sq cm (30 psi) of internal pressure, there is no cracking and the structure behaves elastically. It enters into non linear range beyond this pressure and the ultimate capacity is reached at 2.31 kg/sq cm (33 psi) of internal pressure. Fig.3 shows the effective stresses in concrete at ultimate pressure of 2.31 kg/sq cm (33 psi). The results of the present finite element code compare well with the results of Akbar and Gupta [5]. In this case the shell fails in membrane region due to concrete cracking while the steel reinforcement has still some reserve capacity.

### (3) Narora 235 MWe PHWR Outer Containment

The containment for Indian PHWRs have a primary inner containment wall (ICW) which is made of prestressed concrete construction. The secondary outer containment wall (OCW) is made up of reinforced concrete construction (fig 4). For Indian PHWRs the inner primary containment on its own is ensured to provide leaktightness of the order of 0.3% volume of enclosed air in one hour under

NPT. During reactor operation, annular space is under small negative pressure. In the event of failure of vacuum creating pumps of the purging system due to non-availability of power, a positive pressure will develop on OCW which is the design pressure of 0.07 kg/sq cm. This event occurring subsequent to LOCA is of extremely low probability. The present analysis predicts the ultimate pressure for OCW which is the final barrier between the radioactivity and plant personnel. This structure was modelled with 371 nodes and 78 elements, each of which is divided into six concrete layers into a quarter shell model. The openings in the OCW were not modelled. The reinforcement was modelled with four smeared steel layers. The base of the OCW consists of high reinforcement at 91.2 m elevation due to vertical dowel rebar providing fixity with the base raft. The inner and outer hoop rebars are continuous up to 97.2 m elevation. Above this elevation the hoop and meridional rebars are uniform. The containment is discretised along the height so as to simulate change in hoop and meridional rebars at any elevation. The structure becomes very rigid at 126.23 m elevation due to the inner containment roof slab connected to outer containment. This cellular roof slab has not been modelled in the present analysis hence at this elevation the nodes have been restrained for radial deflection. The annular space below the slab is only pressurised in the event of LOCA.

Good comparison is noted in linear elastic range (table 2). Figs. 5-6 show the effective stress in concrete for top and bottom layer respectively. In the dome region the stresses are very low. The effective stresses are high near the roof slab and at the bottom of containment. Significant bending stresses are developed at the base of the containment. The effective stresses in hoop reinforcement are well below the yield stress of steel [Fig 7], there is no yielding of steel rebars. Fig 8 shows the deformation mode of OCW at different load factors compared to the design pressure. The concrete fails at the mid of the cylindrical wall well below roof slab, which is in the membrane region. The ultimate load at which failure occurs is 0.456 kg/sq cm, about 6.8 times the design pressure. This high margin of ultimate load ensures the inherent safety of the outer containment wall structure which is the final barrier between the radioactivity and the plant personnel.

## CONCLUSIONS

The aim of this endeavour is to regain the faith of public and ensure that even in case of severe accident the PHWR containments have enough reserve strength and exposure to public and plant personnel is the minimum. The studies carried out so far demonstrate that containment failure is not catastrophic in extremely low probable hypothetical severe accident cases and the leak at ground level will give indication for emergency planning. The ultimate load on similar global model is being further used to study the behaviour of ICW structure for such accidents. The overall picture emerging from these studies will help to arrive at the ultimate capacity of the containment structures.

## REFERENCES

1. Hinton and Owen; Finite Element Software for Plate & Shells; Pineridge Press Limited. (1985).
2. Chen W.F; Plasticity in Reinforced Concrete; McGraw Hill; New York. (1982).
3. Mang H A and Meschke G, "Nonlinear Finite Element Analysis of Reinforced Concrete Structures" Engg. Structure. Volume 3. April (1991).
4. Arthur W.Hedgren. Jr and David. P. Billington; Mortar Model Test on

Cylindrical Shell of varying Curvature and Thickness ACI Journal. Feb(1967).  
 5. Habibollah Akbar and Ajay.k.Gupta; Ultimate Behaviour of an R.C. Nuclear  
 ar Containment Subjected to Internal Pressure and Earthquake; Journal of  
 structural Engineering, Vol 112, No. 6, June(1986).

Table 1 COMPARISON OF RESULTS IN ELASTIC RANGE AT 20 PSI INTERNAL PRESSURE

Material Properties kg/sq cm		Present Code	Theoretical
$E_c = 282100.0$ (4030,000 psi)	Radial Deflection	0.154 cm (0.0605in)	0.156 cm (.0612 in)
$\mu = 0.15$	Hoop stress	21.92 kg/sq cm (313.1 psi)	23.31kg/sq cm (333.3 psi)
$f_c = 350.0$ (5000 psi)	Meridional Bending	7679.41 kg	7697.79 kg
$f_t = 37.1$ (530 psi)	Moment	(16915.0 lb)	(16955.0lb)
$E_s = 4200.0$ (60,000 psi)			

Table 2 COMPARISON OF RESULTS FOR NARORA OCW (0.07 KG/SQ CM)

Material (kg/sq cm)		Present code	Theoretical
$E_c = 255000.0$	Radial deflection	0.0215 cm	0.0209 cm
$f_c = 200.0$	Hoop stress	2.64 kg/sq cm	2.607 kg/sqcm
$\mu = 0.2$	Meridional Bending		
$f_y = 4250.0$	Moment	1026.3 kg	1277.3 kg
$E_s = 2100000$			

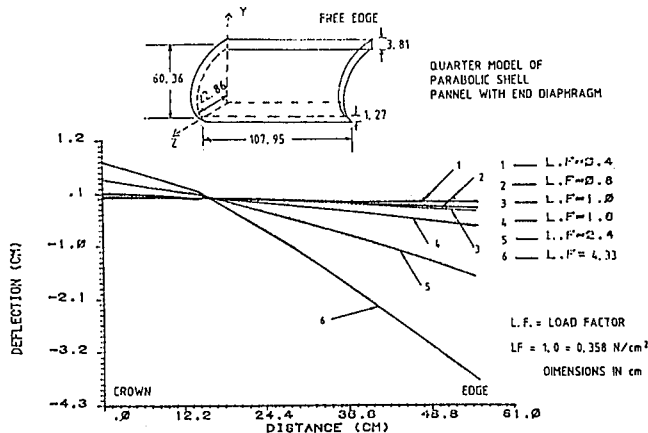


FIG.1 TRANSVERSE VARIATION OF NORMAL DEFLECTION AT MIDSPAN OF PARABOLIC SHELL PANEL

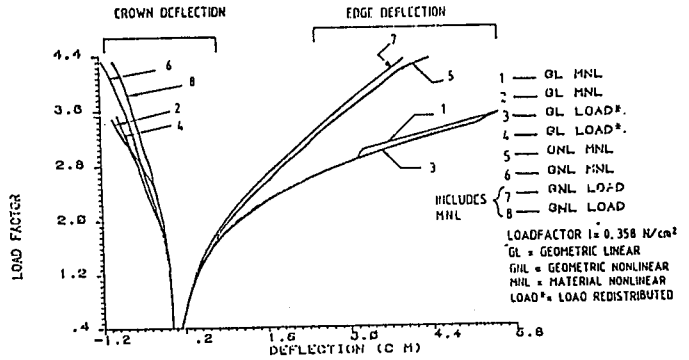


FIG. 2. PARABOLIC SHELL EDGE & CROWN NORMAL DEFLECTION

MAX : 697.00

MIN : 146.10

LEGEND

- ▨ 530.50 : 697.00
- ▨ 450.00 : 530.50
- ▨ 350.00 : 450.00
- ▨ 250.00 : 350.00
- ▨ 146.10 : 250.00

PSI

ROTX = 180.0  
 X ROTY = 0.0  
 ROTZ = 0.0

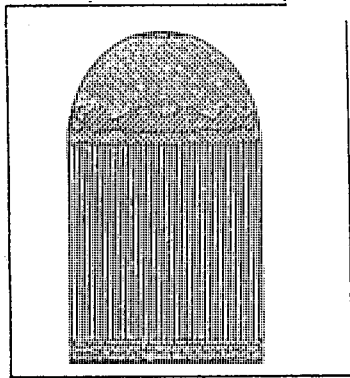


FIG. 3 REINFORCED CONCRETE CONTAINMENT EFFECTIVE STRESS IN CONCRETE; LOAD = 33 PSI

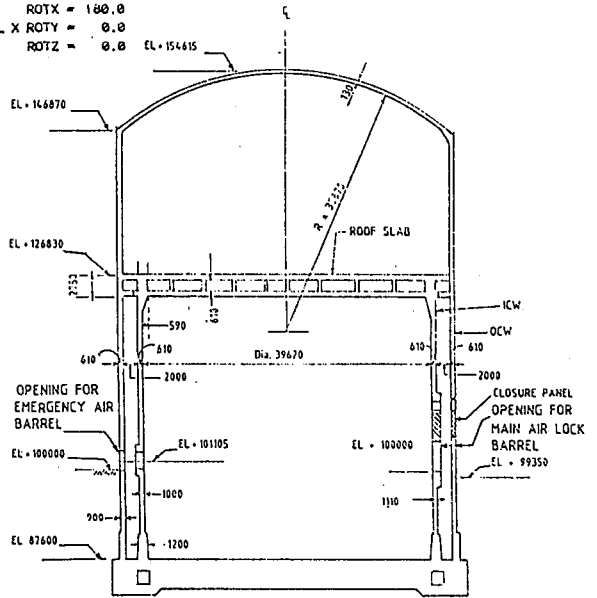


FIG. 4. 235 MWe INDIAN PHWR CONTAINMENT SCHEMATIC NARORA STATION

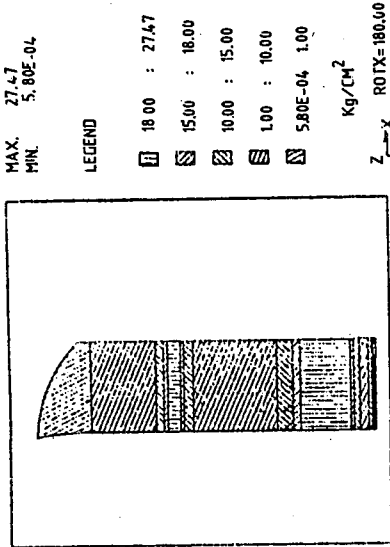


FIG. 6. 235 MWe PHWR CONTAINMENT EFFECTIVE STRESS IN CONCRETE (BOTTOM LAYER)  
LOAD= 476 Kg/CM<sup>2</sup>

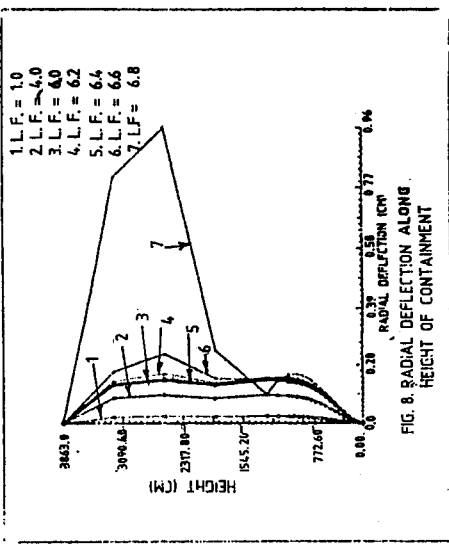


FIG. 8. RADIAL DEFLECTION ALONG HEIGHT OF CONTAINMENT

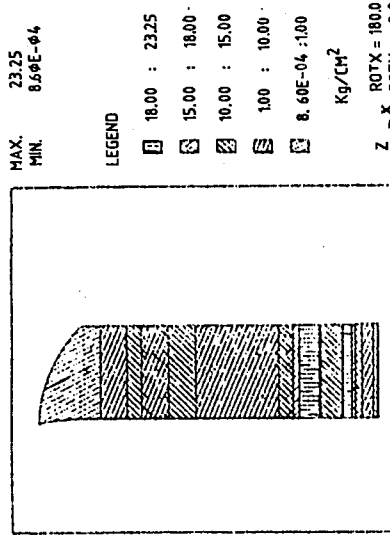


FIG. 5. 235 MWe PHWR CONTAINMENT EFFECTIVE STRESS IN CONCRETE (TOP=LAYER)  
LOAD= 476 Kg/CM<sup>2</sup>

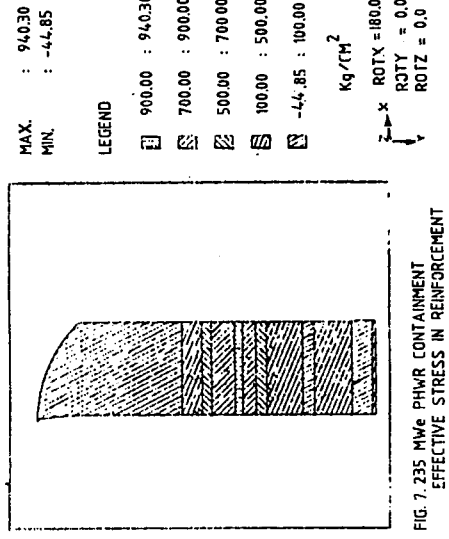


FIG. 7. 235 MWe PHWR CONTAINMENT EFFECTIVE STRESS IN REINFORCEMENT  
LOAD = 476 Kg/CM<sup>2</sup>