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## Analysis of the spent fuel storage bay (500 MWe Indian PHWR)

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**ABSTRACT:** The stainless steel liner, the backing channels and the concrete storage bay were analysed for all possible loads. A global analysis of the concrete pool and a local analysis of a substructured portion with an embedded channel showed that stresses in concrete, channel and its struts were within limits specified by the ASME code. It was seen that the liner plates would never buckle because of the stabilising effect of water pressure.

### 1 INTRODUCTION

The Spent Fuel Storage Bay (SFSB) is a rectangular R.C.C. tank lined inside with SS 304L liner plates (2m x 1.25m) welded to channels embedded in concrete. The channel is a flat SS 304L plate welded to a carbon steel (IS 2062) ISMC75 channel. Anchoring struts are welded on either side of the channel, staggered fashion at 450 mm 'centre to centre'. Pool dimensions are 50m x 12m x 9.3m, filled with demineralised water upto a height of 8.5m. The pool walls are 700 mm thick with counterforts on the outside (fig.1). Liner plate thickness for the walls and floor are 3.15 and 6.3mm respectively. The spent fuel trays stand on the floor liner and impose a pressure of 8.5kg/cm<sup>2</sup>. A maximum gap of 3mm between concrete and liner is postulated. Temperature of pool water may rise to 50 °C on outage of water cooling. This is an increase of 20 °C over ambient temperature.

The purpose of this study was to predict the structural behaviour of the liner and the embedded channels under the postulated loads.

The analysis was done in two stages. First, the concrete pool and embedded channels were analysed. Loading on the pool is a combination of water pressure, soil pressure, temperature gradient through walls, shrinkage, gravity and earthquake loads (S.S.E). In this analysis, only the severest load combination has been taken.

In the second stage, only the liner plates were analysed. Thermal expansion along with the applied compressive strain from backing concrete will give rise to a possibility of buckling of the liner.

### 2 ANALYSIS OF THE STORAGE BAY AND EMBEDDED CHANNELS

#### 2.1 Method of analysis

Channel dimensions are very small compared to the pool walls. Therefore local interaction between channel and the surrounding concrete dies out within a short distance. Therefore,

the problem may be split in two parts :

a) *Global analysis* – R.C.C. pool is modelled with 3–D finite elements. Channel and liner are not modelled. This is because stresses around the channel are local in character, and the liner just follows the displacement imposed on it by the backing concrete.

Stiffness of concrete is much larger than that of channel, hence loading on channel is as an applied displacement from surrounding concrete. Thus a simple way to calculate the gross stresses in channel is to multiply the stress in concrete by the ratio of the young's moduli of steel and concrete ( $E_{ratio}$  method) [1].

b) *Local analysis* – Local stresses due to deformation of the faces of the channel however cannot be obtained by the  $E_{ratio}$  method. A detailed local analysis is done for this. Maximum stressed region of the global model is treated as a substructure in which the channel is modelled.

### 2.1.1 Global analysis

One quarter of the SFSB has been modelled (fig.2). Quarter symmetry has been assumed although true symmetry is absent. Raft is very rigid and has therefore not been modelled. The worst load combination comprising of internal water pressure, temperature gradient through the wall, shrinkage, S.S.E. and gravity has been applied. Earthquake load has been taken by lumping the entire water mass on the walls, with a spectral acceleration of 0.2g at the fundamental frequency (20.5 Hz) applied statically. This is a conservative approach.

#### 2.1.1.1 Results

A table of maximum horizontal stresses ( $N / mm^2$ ) on the inside surface of the walls is given below :

CASE	LONG WALL	SHORT WALL	JUNCTION
Temp + Shrinkage	-3.676	-3.494	+11.93
Internal press.	+0.464/-0.331	+0.47/-0.453	+0.941
Gravity	-0.05	+0.02/-0.06	-0.09
Earth Q. (Y dir)	-0.01/+0.236	+0.04/-0.07	+0.582
Earth Q. (X dir)	+0.63/-0.511	+0.68/-0.587	+3.472
Combined	-5.462	-6.536	+12.68

Maximum magnitude of stress in the vertical direction is at the junction and is equal to  $-6.62 N/mm^2$ . A plot of the  $\sigma_{xx}$  stress is shown in fig. 2.

#### 2.1.1.2 Discussion

Maximum stress occurs at the junction and is tensile. Gross stress in the channel is obtained by multiplying this maximum tensile stress ( $12.68 N/mm^2$ ) by the  $E_{ratio}$  (7.37). This value of  $94 N/mm^2$  is smaller than the yield stress of steel. Maximum compressive stress which causes the local deformation mentioned in para 2.1(b) occurs in the vertical direction at the junction. This is 1.2 times the maximum stress ( $\sigma_{xx}$ ) in the long wall away from junction. Maximum contribution to stress is from temperature gradient and shrinkage. An estimate of the maximum compressive stress in channel is obtained by multiplying the maximum compressive stress in concrete ( $-6.62 N/mm^2$ ) by the  $E_{ratio}$  (7.37) to yield a

value of 48.8 N/mm<sup>2</sup> .

## 2.1.2 *Local analysis*

### 2.1.2.1 *Method of analysis*

Stresses between the counterforts repeat themselves. This situation lends itself to a plain strain analysis of a horizontal section between two counterforts. A representative section was selected which lies between two counterforts on the long wall. However maximum compressive stress lies in the vertical direction at the junction. This stress is greater than the maximum stress in the long wall by a factor 1.2. Hence all stresses obtained from the local analysis are multiplied by a factor 1.2 and reported.

### 2.1.2.2 *Analysis*

The substructure was modelled with plain strain elements (fig. 3&4). At the edge of the substructure, displacements as obtained in the global model were applied as displacement boundary conditions. All loads that act on this part of the global model were also applied in the submodel.

### 2.1.2.3 *Results*

Fig. 3. shows the  $\sigma_{xx}$  stress in the entire substructure. These stresses are same as those observed in the global model. Fig. 4. shows the  $\sigma_{xx}$  stresses in the channel and surrounding concrete. Maximum stress in the steel channel is  $-78.36$  N/mm<sup>2</sup> . Maximum stress in the bottom of the channel is  $-44$  N/mm<sup>2</sup> . Maximum local concrete stress is  $-16.9$  and  $+3.4$  N/mm<sup>2</sup>. These stresses are well below the respective yield strengths.

### 2.1.2.4 *Discussion*

Top and sides of the channel are in bending, while bottom section is in membrane compression. Maximum stress of 78.36 N/mm<sup>2</sup> is highly localised at the slot. In the bottom plate of the channel, stress is 44.28 N/mm<sup>2</sup> which compares well with the predicted value of 48.8 N/mm<sup>2</sup> . Thus the  $E_{ratio}$  method can be used to predict the gross stresses in the channel.

### 2.1.2.5 *Code check*

The liner anchor allowable loads are given in the ASME Boiler and Pressure Vessel code [2]. The code limits the mechanical load under extreme environmental conditions to 0.67 times the liner anchor yield force capacity. Maximum applied force is on the bottom plate of the channel, and its ratio to the yield force is 0.185. This is much smaller than 0.67.

## 3 ANALYSIS OF LINER PLATE

### 3.1 Assumptions

Liner plate being very thin, will follow the displacements of concrete. Maximum compressive strain applied by the pool wall is obtained from the global analysis of the pool ( $6.62 \text{ N/mm}^2 / E_{\text{concrete}} = 2.32\text{E-}4$ ). On this will be superimposed the thermal strain due to the  $20^\circ\text{C}$  temperature rise ( $3.4\text{E-}4$ ). Thus the total applied compressive strain on the liner is  $5.72\text{E-}4$ . The liner plates are subjected to compressive strain only in the presence of water in the pool. So if buckling of the plates is to be considered, it must be in conjunction with the pressure of water acting against the liner. But buckling is due to an imposed strain, and hence will not lead to catastrophic failure. The liner plate will begin to lift from the concrete wall when the upward deflection due to applied strain of  $5.72\text{E-}4$  exceeds the downward deflection due to water pressure.

### 3.2 Analysis

The details of the analysis for the wall liner are discussed, and the results for the floor liner are presented later.

i) 2m x 1.25m x 3.15mm – wall liner plate :

The critical buckling stress for this plate is found as follows [3] :

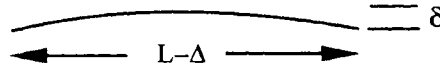
$$\sigma_x' + (a^2/b^2) \sigma_y' = 1.1 (E t^2 a^2)/(1-\nu^2) [3/a^4 + 3/b^4 + 2/(ab)^2]$$

$$\sigma_{\text{crit}} = 4.56 \text{ N/mm}^2; \epsilon_{\text{crit}} = \sigma_{\text{crit}} / E_s = 4.56 / 1.95 \text{E}05 = 2.34 \text{E-}05$$

The applied strain of  $5.72\text{E-}4$  is higher than the buckling strain of  $2.34\text{E-}5$ . Hence, as such the liner should buckle. But water pressure tends to flatten the liner against the wall. The following calculation is done to find the depth of water below which buckling will not occur.

A simplified equation to find the upward deflection of plate due to applied strain from concrete is given below [4] :

$$\Delta = \delta^2 \pi^2 / (4L)$$



where,  $\Delta$  = applied displacement;  $L$  = liner length;  $\delta$  = central deflection.

$$\Delta = 5.72\text{E-}4 \times L$$

Upward deflection ' $\delta$ ' was found to be 31 mm.

Elastic deformation of plate under water pressure 'q' is [3]

$$y_{\text{max}} = \alpha q b^4 / (E t^3)$$

where,  $E = 2.1\text{E}10 \text{ kg/m}^2$

$$t = 3.15 \text{ mm}$$

$$\alpha = 0.0251 \text{ for } a/b=1.6$$

$$b = 1.25 \text{ m}$$

Equating the elastic deflection under water pressure to the buckling deflection of 31 mm, we get the water pressure 'q' =  $326 \text{ kg/m}^2$ , ie. a height of 0.3m of water. This means that any liner plate immersed at a depth greater than 0.3m is not going to buckle. 0.3m is small compared to the vertical plate dimension of 2m. Thus even the top liner plates will not buckle. Only membrane stresses due to the applied strain will act on them.

ii) 2m x 1.25m x 6.3mm – floor liner plate :

A similar calculation for the floor liner plate shows that the downward deflection due to water pressure, 102mm, is much larger than the upward buckled deflection of 31mm, and hence no buckling of the floor liner will occur.

### 3.3 Stress limits

ASME [2] limits the liner strains under service conditions to less than 0.002. The applied strain of  $5.72E-4$  is below this limit. The primary stress due to the deflection of 3mm under water pressure is  $2.37 \text{ kg/mm}^2$  for the 3.15 mm plate, and  $4.74 \text{ kg/mm}^2$  for the 6.3mm plate. These are less than the  $S_y$  ( $21 \text{ kg/mm}^2$ ) limit on primary membrane plus bending.

### 3.4 Shear failure of liner weld

On buckling, the load carrying capacity of a liner plate decreases drastically. One could envisage a scenario where the adjacent plate has not buckled and therefore carries the full compressive load. For this case there will be shear forces acting on the liner channels. If the weld fails, it could lead to a propagation of failure. This is because once the weld shears, two adjacent liner plates form one large buckled plate. The post buckling load carrying capacity of this large plate is smaller than that of a single plate. This will lead to a larger force unbalance on the adjacent channel weld. This is checked below for the floor liner plate where maximum compressive force is developed :

$$\begin{aligned} \text{Total force} &= \sigma \times \text{area} = (\epsilon \times E) \times (L \times t) = (5.72E-4 \times 2.1E5) \times (2000 \times 6.3) \\ &= 1.5E6 \text{ N.} \end{aligned}$$

Weld perimeter (p) = 6500 mm ; Weld width (w) = 6.3 mm.  
Therefore, maximum load that can be resisted by the weld =

$$\begin{aligned} \tau_{\text{all}} \times w \times p &= 100.0 \times 6.3 \times 6500 \\ &= 4.1E6 \text{ N.} \end{aligned}$$

The total force is smaller than the maximum load that can be taken by the weld. Hence propagation of buckling failure will not occur.

## 4 Conclusion

A detailed analysis of the concrete pool and the embedded channels shows that the stresses are within the allowable values. It has also been shown that due to the stabilising influence of water pressure, the possibility of buckling of the liner plate due to imposed strain by concrete does not exist.

## 5 References

- [1] Strength of Materials – Timoshenko : Mc. Graw Hill.
- [2] ASME Boiler & Pressure Vessel Code – Section III Div 2 Sub. CC.
- [3] Roark's Formulas for Stress and Strain – Warren C. Young.
- [4] Note of Calculations of Tightness (Leak Tightness) Canning : Electricity of France, Clamart Equipment Region, Nuclear Power Station of Fessenheim, 1st Section, Batch 1. : N T 6722 Rev.A FM 23.

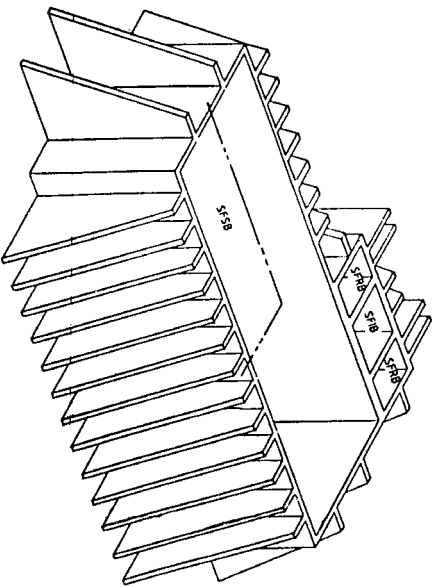


Fig. 1 General Layout of SFSB Pool

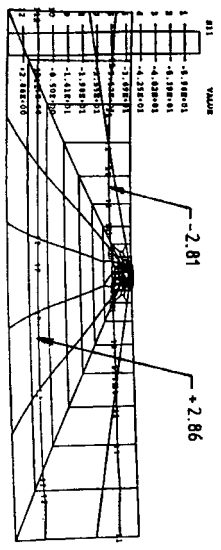


Fig.3 Full submodel with channel : Deformation &  $\sigma_{xx}$  (N/mm<sup>2</sup>)

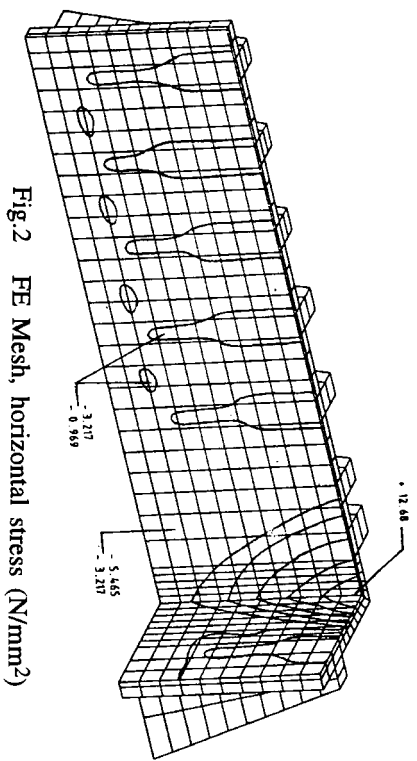


Fig.2 FE Mesh, horizontal stress (N/mm<sup>2</sup>)

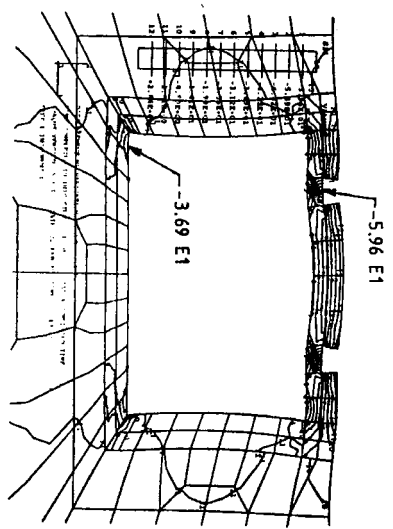


Fig.4 Channel with local stresses : Deformation &  $\sigma_{xx}$  (N/mm<sup>2</sup>)