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THE SEISMIC RELIABILITY OF VVER-1000 NPP PRESTRESSED CONTAINMENT BUILDING

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ABSTRACT

The failure probability assessment of the containment building is an essential feature of the Level 2 PSA–studies of nuclear power plants. The geometry of the containment was determined by the preliminary design. The seismic hazard of the plant site was assessed during Level 1 PSA of Loviisa plant. The initial information for seismic fragility analysis of the containment is the seismic response of the structure. The structural model for response analysis was the stick model. The stress analysis of the containment was carried out using the shell element model. The fragility evaluation of the containment was performed with the PROBAN–program. The structure was modeled as a parallel system consisting of the most heavility stressed elements. The resulting fragility curve gives the conditional probability of failure as a function of peak ground acceleration. The seismic hazard and the fragility were convolved to obtain the annual nonexceedance probability distribution for the collapse frequency of the structure. The dominating range of this distribution was of 1E–12 to 1E–8.

1. INTRODUCTION

The reliability of an entire structure including the nuclear power plant structures has to be determined as a reliability of the structural system. This means that the study cannot be limited to any individual component but the structure must be treated as a whole. The redundancy in structural systems is always active by the nature i.e. the structure acts as a load distributing system so that the alternative loading routes actively carry the total load, whereas the passive redundancy requires only the capability to function on demand. The reliability of structure or a structural component can be modeled with sufficient accuracy with the aid of brittle failure mode i.e. in the case of a failure the load carrying capacity of the structure drops instantly to zero. The failure of the NPP containment structure can be described as a failure of the brittle system. This system can be modeled with the aid of the method of finite elements. Thus, every element acts as a brittle component.

2. THE CONTAINMENT GEOMETRY AND STRUCTURAL RESPONSE

The inner radius of the containment cylinder and the dome is 22 m. The height of the cylinder is 41.2 m and the wall thickness for cylinder and dome are 1.2 m and 1.0 m, respectively. The material properties which were chosen to be random variables were the compressive strength of concrete, the wall thickness, the yield strength of reinforcement steel, the cross-sectional area of the reinforcement and the moment arm of the reinforcement. The distributions of the above variables were assumed to be normal and the mean values and coefficients of variation were 46.3 MN/m^2 and 0.1 for concrete strength, 1.2 m and 0.1 for wall thickness, 530.5 MN/m^2 and 0.036 for steel yield limit, 70 cm^2 and 0.05 for steel cross-sectional area and 0.825 m and 0.1 for moment arm of reinforcement. The containment building was modeled with the aid of the stick model for the seismic response analysis. The seismic response of the model was computed for the horizontal response spectrum of the bedrock given in Fig. 1 /1/ and for the corresponding acceleration time-history. As a result of the response analysis the maximum horizontal acceleration is obtained in the nodal points of the model. The maximum value of 0.317 g was obtained on the top of the model. The acceleration profile was applied as a load to the shell model of the containment building. The shell model is described in Fig. 2. The ZPA acceleration in the bedrock level of the shell model was scaled from 0.1 g to 1.5 g and for every ZPA level the stresses in the shell model were computed. The collapse of the containment occurs when the elements in the bottom of the model lose their load carrying capacity.

3. THE COMPUTATION OF THE CONTAINMENT FRAGILITY

As a result of extensive experiments that have been carried out for the reinforced concrete cylinders loaded with cyclic load, Akino, 1982 /2/ and Tanaka, 1988 /2/, it has been observed that the collapse is caused because of the concrete break in the bottom of the wall after the yield of reinforcement. On the basis of above results the collapse of the reinforced containment building can be defined as a compressive break of the concrete in the bottom of the containment wall. Thus the limit state equation for the unit length in the circumferential direction of the containment wall can be formulated as follows

$$g(X) = f'_c - \left(\frac{N_Y}{t} + \frac{M_Y f'_c}{A_s f_Y\, jt}\right) \tag{1}$$

where

f'_c = the compressive strength of concrete
f_Y = the yield limit of reinforcement
A_s = the cross-section area of reinforcement
N_Y = the vertical normal force
M_Y = the bending moment about the horizontal axis
t = the thickness of the shell
jt = the inner moment arm of reinforcement.

Assuming that after the compressive failure of the concrete the strength of the element drops immediately to zero and assuming that the failure of the containment is caused by the failure of the elements in the bottom of the containment the collapse probability of the containment can be determined by investigating the structure as a parallel system /4/. It is further assumed that the number of parallel elements on one side of the cylinder is six which corresponds to the length of one quarter of the cylinder circumference. The parallel structural system was analysed for each ZPA level from 0.1 g to 1.5 g and the corresponding failure probability was assessed with the aid of PROBAN-reliability analysis program. The resulting fragility curve is given in Fig. 3.

4. THE CONVOLUTION OF THE SITE HAZARD AND THE STRUCTURAL FRAGILITY

The seismic hazard of the site is given in the form of hazard curves which have been given in reference /5/. The total number of hazard curves were eighteen and each of them had a specific weight. The convolution of the hazard curves and the fragility curve was performed with the aid of numerical Monte Carlo integration. The result of the convolution integral is the cumulative probability distribution of the annual failure frequency of the containment building. The distribution curve is depicted in Fig. 4 and the median value of the distribution is 2.0E−12. Further, in order to assess the upper bound value for the annual failure frequency the convolution of the fragility curve was performed using the seismic hazard curve which gave largest value for seismic hazard. The result of this computation gave the median value of 7.5E−8 for the annual failure frequency of the containment.

5. CONCLUSION

The collapse probability of the reinforced concrete containment building was evaluated. The level of seismic hazard of the site was very moderate corresponding to the seismicity level of southern Finland. The seismicity of the site was presented in the form of a family of hazard curves. The fragility curve of the structure was evaluated using conventional methods and treating the containment as a parallel structure. For the median value of the fragility the value of 1.5 g was obtained. As a final phase of the study the convolution of the hazard and the fragility was performed. Depending on the hazard assumption used the median value of the annual containment failure frequency varies between the values of 2.0E−12 and 7.5E−8.

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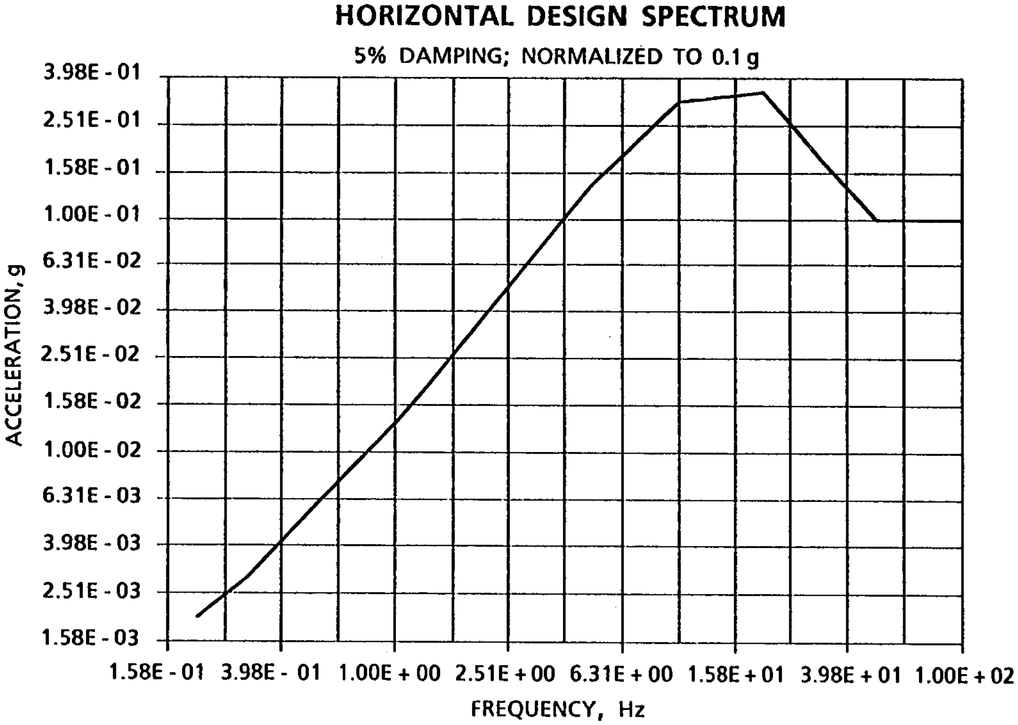

Fig. 1 The horizontal response spectrum of the bedrock.

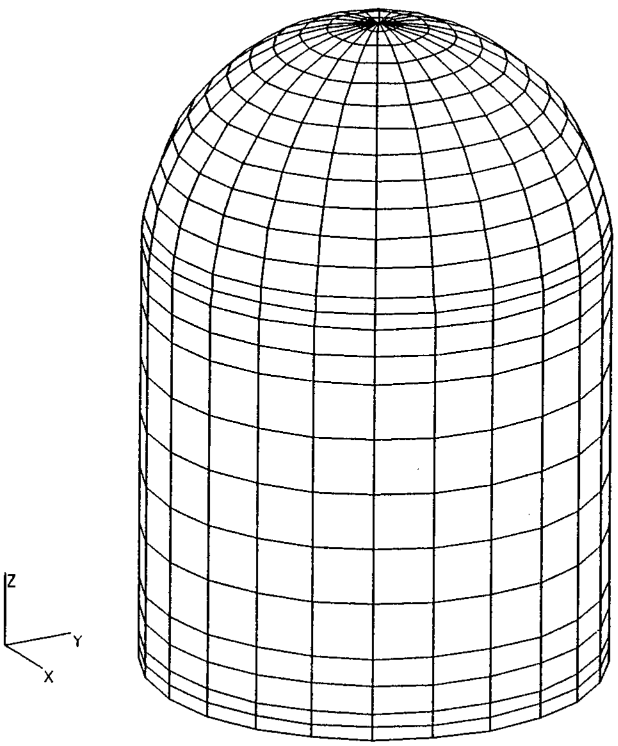

Fig. 2 The shell model of the containment building.

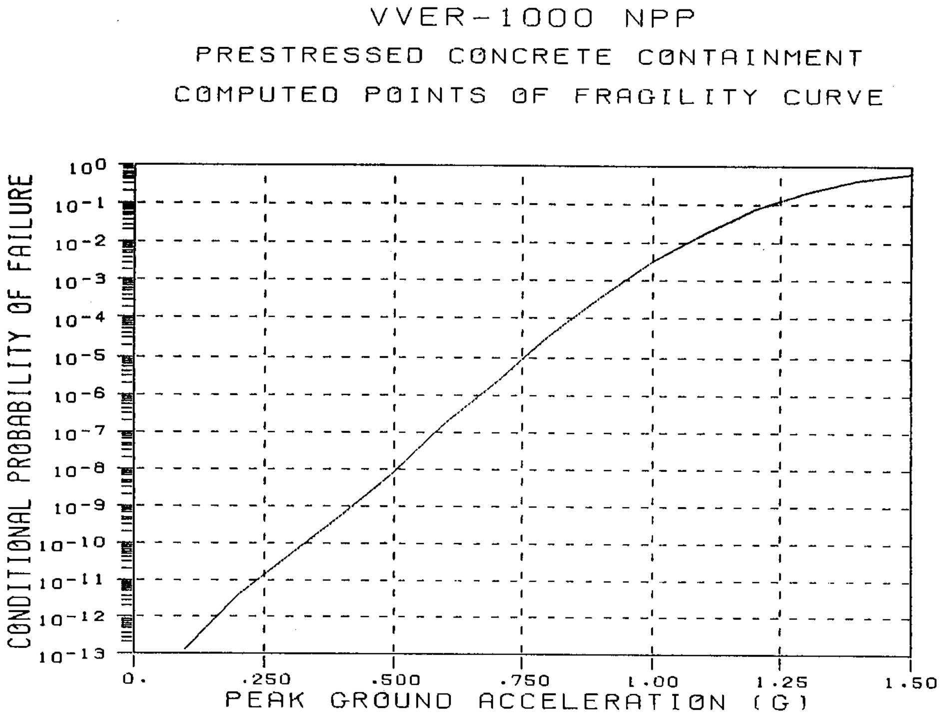

Fig. 3 The fragility curve of the containment building.

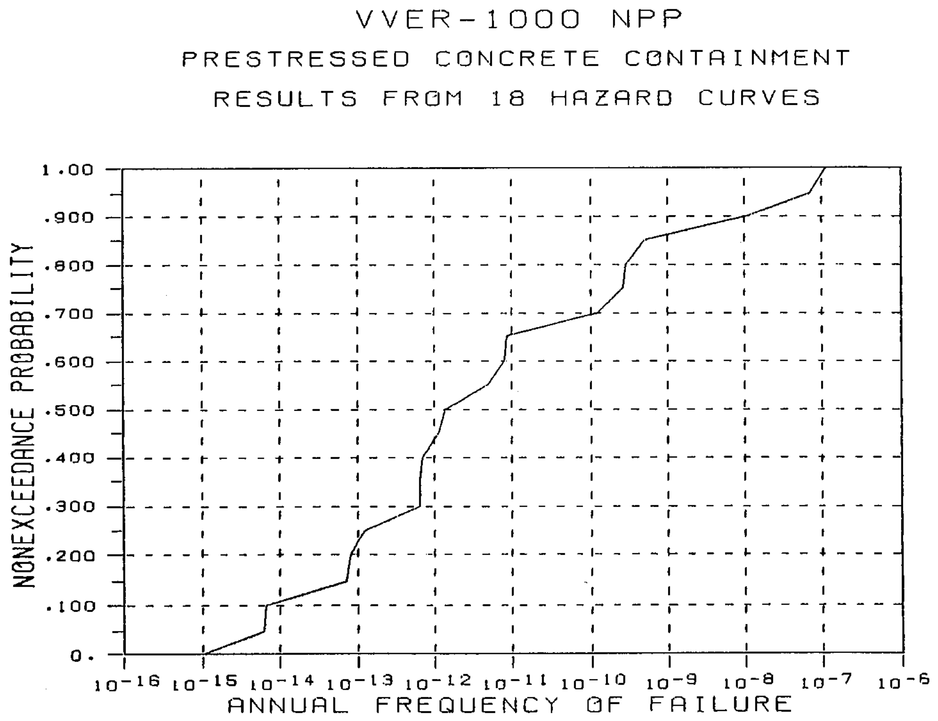

Fig. 4 Nonexceedance probability distribution of the annual frequency of containment failure.