

REINFORCED CONCRETE CONTAINMENT STRUCTURES IN HIGH SEISMIC ZONES

T. S. AZIZ

*Acres Consulting Services Limited,
5259 Dorchester Road, Niagara Falls, Ontario L2E 6W1, Canada*

C. F. REEVES

*Engineering Mechanics, Stone & Webster Engineering Corporation,
P.O. Box 2325, Boston, Massachusetts 02107, U.S.A.*

SUMMARY

Reinforced concrete containments of the deformed bars as well as the prestressed concrete containment variety have been used in many countries. Because the reinforced concrete cylinder is in a biaxial tension field under design basis accident pressures and thereby cracked in two planes, its direct capacity to resist forces generated by an earthquake is small. Some conceptual designs are needed to make a reinforced concrete containment or a prestressed concrete containment feasible in a high seismic zone.

A new structural concept for reinforced concrete containment structures at sites where earthquake ground motions in terms of the Safe Shutdown Earthquake (SSE) exceeds 0.3 g is presented. The Structural concept is based on: (1) an inner steel-lined concrete shell which houses the reactor and provides shielding and containment in the event of loss of coolant accident; (2) an outer annular concrete shell structure which houses auxiliary reactor equipment and safeguards systems. These shell structures are supported on a common foundation mat which is embedded in the subgrade. Under stipulated earthquake conditions the two shell structures interact to resist lateral inertia forces. Thus the annular structure which is not a pressure boundary acts as a lateral support for the inner containment shell. The concept is practical, economically feasible and new to practice.

An integrated configuration which includes the interior shell, the annular structure and the subgrade is analyzed for several static and dynamic loading conditions. The analysis is done using a finite difference solution scheme for the static loading conditions. A semi-analytical three-dimensional finite element scheme combined with a Fast Fourier Transform (FFT) algorithm is used for the dynamic loading conditions.

The effects of cracking of the containment structure due to pressurization in conjunction with earthquake loading are discussed. Analytical results for both the finite difference and the finite element schemes are presented and the sensitivity of the results to changes in the input parameters is studied. General recommendations are given for plant configurations where high seismic loading is a major design consideration.

1. Introduction

A nuclear power plant sited in a high seismic zone must be designed and constructed so that it can be shut down safely in the event of a large earthquake.

During the middle sixties the power industry began to increase orders for nuclear power plants for high seismic regions. Issues concerning public safety became very important and the earthquake design requirements for nuclear power plants, especially those built in areas of high seismicity, were substantially upgraded. Typically, plants are now being designed for earthquake ground motions of 0.5g and higher in active seismic zones. Thus, the structural engineers engaged in the nuclear power industry had to develop concepts and sophisticated analytical techniques in order to provide satisfactory designs.

The primary function of a containment structure is to limit the possible release of fission products to the environment in the event of any postulated accident to the reactor or associated systems and to provide biological shielding.

While containment structures analysis and design have grown many times more detailed and complex and the specific loads considered in the design process more numerous, the concrete containments being constructed nowadays are still very similar to the containments which were designed and built many years ago. The main reason for such a slow development is the large constraint placed on any inovated changes in the design by the different governmental regulatory agencies. Thus, it is expected that changes to current containment design concepts will develop very slowly.

2. Design Considerations for Nuclear Containment Structures

Several loading conditions must be considered in the design and analysis of containment structures. It has been demonstrated that many of these mandatory loading conditions do not produce governing conditions of design. For simplicity this paper is limited to considerations of two major loading conditions which have a significant effect on a design for high earthquake forces. The two loading considerations are: (1) The design basis accident (DBA); and (2) The safe shutdown earthquake (SSE).

Several reinforced concrete containment structures have been designed and constructed for design basis accident (DBA) pressures of 45 psi coincident with SSE accelerations up to 0.2g. These designs meet the requirements of the Nuclear Regulatory Commission (NRC) and appropriate building codes [1,2]. In many of these cases, the design of the reinforcing steel (with the exception of earthquake reinforcement) is governed by the ultimate or factored DBA pressure (P) and temperature (T) loads, where the load factors are 1.5

and 1.0 respectively^[1]. There are no earthquake forces stipulated in this loading combination. Other factored loading combinations,^[1] which include pressure, temperature and earthquake forces, establish the requirements for the earthquake steel. Though earthquake forces induce stresses in some of the pressure boundary reinforcement, earthquake loading does not generally control the pressure boundary design when the SSE acceleration is equal to or less than 0.2g. Further, from experience it is known that the magnitudes of amplified accelerations and displacements in the containment structure do not present any difficulty in the design of Nuclear Steam systems (NSSS) for ground motion of this intensity.

For reinforced concrete containments, earthquake forces would control the design in many areas of the pressure boundary when the SSE ground acceleration is equal to 0.3g. Further, earthquake forces become increasingly important in the boundary design when the SSE acceleration exceeds 0.3g.

With reference to Fig. 1, there are several generalized forces at any location in the containment cylinder wall which results from the application of the various factored loading combinations^[1]. Some of these combinations include the effects of three orthogonal and simultaneous earthquake motions^[3]. Only those forces that are important to earthquake design are shown in Fig. 1 and discussed in this paper. These forces are defined as follows:

- Nm - Net meridional tension force resulting from DBA pressure (P) and liner temperature (T), dead load (DL), and the effects of the three orthogonal earthquake motions.
- Nh - Hoop tension force due to DBA pressure (P) and liner temperature (T).
- Ve - Tangential shear force due to the combined effects of two orthogonal horizontal earthquake motions. This force is a maximum at the base of the containment and decreased with height.

Table 1, Case A (1.0DL+1.5P+1.0T), shows typical values of the Nm, Nh, and Ve forces in the pressure boundary for a 3,400 Mwt Pressurized Water Reactor containment structure where the SSE acceleration is 0.17g. For this factored load combination, the value of Nm is 291 kips/ft at the base of the wall. This force increases with height as the effect of the dead load decreases. The value of Nh is a maximum of 667 k/ft in the "free membrane zone" of the cylinder; and it decreases to a low value where the cylinder is constrained at the foundation mat. As previously stated, there are no earthquake forces involved in this loading combination.

Under the 1.0DL + 1.0P + 1.0T + 1.0SSE loading combination, (Table 1, Case A') the tension force (Nm) at the base of the cylinder wall is 336 k/ft. This value reflects the tension which results from the combined effects of DBA and the SSE forces. The two horizontal earthquake motions cause tension because of the overturning inertia effects on the containment. The vertical earthquake motion further reduces the effect of the dead load, thus increasing the tension. The tensile effects of the earthquake loading on the Nm force decrease with height, while the tensile effects of pressure and temperature increase as in Case A. The Nh force, (Case A'), is not affected by

earthquake ground motion and it varies as in Case A. The Case A' tangential shear force, V_e , in the containment wall is a maximum of 98 k/ft, and it decreases with height.

It is noted that the forces from the three statistically independent simultaneous earthquakes are combined, at any given point in the structure, as the square-root-of-the-sum of the square (SRSS) to obtain the probable earthquake response^[4].

The design of the primary, cadwelded^[5] meridional and hoop reinforcement in the pressure boundary is based upon the N_m and N_h forces from Case A. The design of the tangential-diagonal reinforcement is based upon the N_m , N_h , and V_e forces from Case A', or possibly one of the other factored loading combinations which include earthquake forces^[1]. It is assumed in the design that the strain of tangential-diagonal reinforcement is compatible with the primary reinforcement under DBA loading. The arrangement of the earthquake reinforcement in the containment is shown in Fig. 1.

For comparison other representative design values for the in-plane forces of the pressure boundary for a 3,800 MWT PWR reference plant^[6] are shown in Table 1, as Cases B and B'. The design pressure and temperature conditions are somewhat different than for the 3,400 MWT plant and the SSE is 0.3g.

When earthquake forces are combined with DBA forces,^[1] the pressure boundary at any point must be designed for the maximum tension force, N_m , (including tension due to earthquake forces), applied simultaneously with the maximum tangential shear force, V_e . This is a direct result of the application of the SRSS of the earthquake forces^[4].

It is a requirement that a containment structure be subjected to a structural acceptance test (SAT) where the containment is pressurized to 115 percent of the DBA pressure which causes concrete cracks in the vertical and horizontal directions that close when the containment is depressurized since the internal forces are contained by the continuous hoop and meridional reinforcement. These cracks would open up during a DBA and the tangential shear force, V_e , produced by an earthquake loading would be resisted by aggregate interlock along the cracked surfaces, as well as the tangential-diagonal earthquake reinforcing steel.

In order to establish allowable tangential shear stresses for the resistance of cracked reinforced concrete due to surface roughness along the cracks (aggregate interlock) tests were conducted at Cornell University^[7] on 3,000 psi ultimate design strength concrete specimens which were clamped with pre-set cracks. On the basis of these tests it was recommended that orthogonally reinforced concrete could resist 160 psi in tangential shear without excessive strain. This results in a 104 k/ft capacity in a 4 ft - 6 in thick wall. Shear greater than 160 psi must then be carried by a system of tangential diagonal reinforcement in the containment wall, Fig. 1. However, the NRC has not fully accepted the results of these tests and has established currently allowable shear stresses of 60 psi and 40 psi for the SSE and 1/2 SEE conditions, respectively.

3. Design for High Seismic Forces

For sites where SSE ground motion substantially exceeds 0.3g, a containment structure must be capable of withstanding the combined effects of DBA pressure and high seismic forces; and be designed in a way such that the earthquake response of the reactor equipment is minimized. A prestressed concrete reactor containment structure is a natural solution to the problem of adequate strength. However, the prestressing concept does not have the advantage of reserve ductility under dynamic loading.

Preliminary designs of reinforced concrete containment structures for light water and gas cooled reactor systems now being manufactured in the United States have been evaluated for response to SSE ground accelerations of 0.5g. This work leads to structural concepts which are common to all of these systems:

(1) The overall height of the containment and the elevation of the reactor equipment should be held to a practical minimum in order to moderate seismic response spectra of reactor equipment.

(2) The ratio of overall height of the plant to the minimum foundation plan dimension should be kept as small as possible. This will tend to reduce amplification of ground motion up through the structure, and thus reduce the structural forces, subgrade bearing pressures, and magnitudes of amplified response spectra (ARS) used for equipment design.

(3) Substantial embedment of the plant into the subgrade lowers the center of gravity, reducing amplification of ground motion. Deep embedment also improves the margin of safety for stability to resist overturning and sliding.

(4) A large common foundation mat should be employed to the degree feasible to support the containment structure, internal structure and nuclear steam supply system (NSSS), auxiliary reactor equipment, safeguards, and fueling systems. This will produce low acceleration profiles, increase stability, and reduce subgrade bearing pressures which result from three components of earthquake motion.

Interstructure relative displacements will be minimized and forces in piping, ducts and electrical cables that pass from structure to structure will be much lower than they would be if the structures were on separate foundations. Furthermore, a common foundation mat will eliminate the potential problem of structure-soil-structure interaction. Additionally, amplified response spectra for the design of safety related equipment and piping will be lower than they would be for a nonunitized configuration. Differential settlement problems under operating conditions are also avoided.

(5) Symmetry should be considered to the degree possible in the structural layout. By maintaining centers of mass reasonably close to centers of rigidity, undesirable torsional response can be held to a minimum.

(6) The meridional forces (Nm) and tangential shear forces (Ve) in the containment structure due to the earthquake motion require special consideration in order to provide an economical design.

(7) The internal structure which supports the NSSS should be designed so that amplification of ground motion up through the structure is moderated. Structural complexities to support the equipment should be avoided.

The plant shown schematically in Fig. 2 meets the conceptual requirements outlined in the preceding text. The section depicts a layout for a High Temperature Gas Cooled Reactor (HTGR). However, this conceptual design is adaptable to a Pressurized Water Reactor (PWR) and to a Mark III Boiling Water Reactor with a reinforced concrete containment and integral steel liner.

The design consists of a large circular foundation mat which supports concentrically: the Prestressed Concrete Reactor Vessel (PCRv); the containment; and an annular building which houses the reactor auxiliary equipment, and safeguards. Symmetry has been maintained to the extent that equipment layout and plant function allow. The depth of embedment and the foundation mat thickness are variables which must be determined and optimized when the properties of the subgrade, the DBA conditions and the earthquake intensity are established.

Figure 3 shows the results of seismic analyses in the form of plots of horizontal acceleration profiles for the containment of the HTGR Plant founded on a soil site. The analysis was done in accordance with NRC guidelines [3,4, 8] for a 0.5g SSE ground acceleration. The profiles are plots of peak response motion obtained by a semi-analytical three dimensional finite element analyses of subgrade and the structure.

In these studies the containment was modeled first as a free standing structure without any structural connection to the remainder of the annular building except at the mat. Rattle space was provided between the containment and the annular building roof and floors so that there would be no impact between the structures. The results of this analysis are denoted as Case C' in Figure 3.

A summary of the forces derived from the earthquake analysis of Case C' are shown in Fig. 4 and the combined forces are given in Table 1. The tangential shear force, V_e , calculated from a shell analysis of the containment using the inertia forces obtained from the dynamic analysis is 386 k/ft at the base and decreases with height. The force which develops in the meridional (Nm) direction, in response to the three earthquake motions, is 638 k/ft tension, also decreasing with height.

When the earthquake forces are combined with the force effects of the DBA, as in Table 1, large quantities of reinforcing steel are required in the vertical and tangential-diagonal directions, and the design is prohibitive.

To avoid this problem a structural tie is introduced between the roof of the annular building and the containment shell. The tie is designed to transmit tangential shear forces from the containment structure to the annular building and thus provide a lateral support for the containment under horizontal earthquake loading. This will reduce the overturning and shear forces in the containment wall.

During seismic response, relative horizontal displacements between the containment and the roof of the annular building would be restricted in the tangential direction and only tangential shear forces will be transmitted. The detail should be designed to allow the containment strain freely in the radial and vertical directions under SAT and DBA conditions and not pinch or constrain the pressure boundary. The structural tie should work such that if the combined inertia loading from two horizontal earthquakes is, say, in the 12 o'clock direction, the forces in the tie will increase sinusoidally from zero at 12 o'clock to a maximum at 3 o'clock and 9 o'clock, and decay to zero at 6 o'clock.

The forces in the tie during earthquake response are transmitted by diaphragm action through the roof of the annular building to the external wall, and thence down to the foundation mat and the subgrade. Loading which would otherwise build up in the containment wall is delivered to a part of the structure (the annular building) which is not a pressure boundary. Simply stated, the structural tie divides the earthquake loading between the containment and the annular building.

The effects of the tie system on structural response of the containment are shown in Figs. 3,4, denoted as Case D'. Figure 3 shows a reduction in the acceleration profile of the containment. In Fig. 4 it is shown that the V_e forces for Case D' are much less than for Case C'. The forces in the N_m direction are also significantly reduced. Under these circumstances a design of the pressure boundary can be achieved because the combined effects of shear-tension forces (N_m and V_e) are manageable. Moreover, since the forces in the N_m direction at the base of the containment are reduced, the shear forces and the bending moments in the foundation mat are also significantly reduced. The structural tie must be carefully designed to minimize radial shear and out of plane bending in the containment wall, and the pressure boundary must be reinforced for the interaction loading. It is emphasized that maximum loading at any given instant during the response, will occur only at two diametrically opposite locations in the wall.

4. Seismic Analytical Models

In the seismic analyses for Cases C' and D', horizontal and vertical ground response spectra were used as recommended in NRC Regulatory Guide 1.6^[3]. These spectra were normalized to peak acceleration values of 0.5g for natural periods approaching zero. Artificial earthquake time histories consistent with the frequency content displayed in the site design response spectra were generated through a multi-phase stochastic model^[9]. Fig. 5 shows the original design spectrum and the artificial time history spectrum for 1 percent equipment damping. These spectra do not include the effects of soil-structure interaction.

Damping values used in the structure for the different analyses are consistent with Regulatory Guide 1.61^[8]. Damping values in the subgrade are considered strain dependent and were derived from a one dimensional wave

propagation analysis of the subgrade [10] and were kept constant throughout the parametric studies.

To determine the soil-structure interaction effects, the structural model is coupled to the surrounding medium using a finite element domain. The analysis thus accounts for structural embedment as well as variation of the soil properties with depth.

Finite element and finite difference solutions for the seismic analysis of nuclear containments are very attractive. Some precautions must be taken, however, to guarantee the accuracy of the results. The size and shape of the elements, the modeling of the internal damping, and the reproduction of a semi-infinite continuum through appropriate boundary conditions or a finite domain are significant factors which must be considered in the analysis. In order to reproduce adequately the propagation of waves through a continuum, the size of the elements or alternately the finite difference mesh should not be longer than 1/6 to 1/8 of the wave length corresponding to the highest frequency of interest. This wavelength is given by the relation:

$$l_{\min} = \frac{C_s}{f_{\max}} = C_s \cdot T_{\min} \quad \dots(1)$$

where f_{\max} and T_{\min} represent the largest frequency and the smallest period of interest respectively, and C_s is the shear wave velocity of the soil

Since this structure is axisymmetric and is subjected to an unsymmetric loading the semi-analytical finite element process described by Wilson (1965) [14] is used. In this process the axisymmetric structure is represented as an assemblage of quadrilateral toroidal elements interconnected along nodal circles and material properties are defined individually for each element in the assemblage to take into consideration soil stratification. The loads as well as the displacements are expanded into fourier series in the θ -direction. Thus the radial, vertical and tangential components of displacements (Fig. 7) are expressed as:

$$\begin{aligned} U^r(r, z, \theta) &= \sum_{n=1}^{\infty} U_n^r(r, z) \cdot \text{Cos}n\theta & \dots \\ U^z(r, z, \theta) &= \sum_{n=1}^{\infty} U_n^z(r, z) \cdot \text{Cos}n\theta & \dots(2) \\ U^\theta(r, z, \theta) &= \sum_{n=1}^{\infty} U_n^\theta(r, z) \cdot \text{Sinn}\theta & \dots \end{aligned}$$

For uniform density and elastic properties in the circumferential direction, orthogonality of the trigonometric functions can be exploited to represent the general three-dimensional problem of an axisymmetric structure subjected to arbitrary loading as a series of uncoupled two-dimensional problems. The complete solution then is the superposition of all the two-dimensional finite element solutions. However, the effective external loads associated with horizontal earthquake ground motion $\ddot{U}_G(t)$ along $\theta = 0$ are

symmetrical about the $\theta = 0$ plane and are given by:

$$F^x(r, z, \theta) = -\rho(r, z) \cdot \cos\theta \cdot \ddot{U}_G(t) \quad \dots$$

$$F^z(r, z, \theta) = 0 \quad \dots (3)$$

$$F^\theta(r, z, \theta) = \rho(r, z) \cdot \sin\theta \cdot \ddot{U}_G(t) \quad \dots$$

where $\rho(r, z)$ is the mass density of the structure. Only the $n=1$ term in equations (2) is required in the analysis of earthquake response. The stiffness and mass matrices for the $n=1$ case are calculated by known procedures. The stiffness and mass matrices of the complete structural assemblage are obtained from the corresponding element matrices by direct stiffness methods. To account for the radiation of waves from the core region into the far field a consistent energy transmitting boundary is used and its contribution to the total stiffness matrix is added. This concept of an energy transmitting boundary has been reported by Waas and Lysmer (1970) [11] and elaborated by Roesset, Kausel, and Chiang-Liang (1974) and is presented elsewhere. [12,13] The whole analysis is done in the frequency domain where the transfer functions of the structural system at different nodal points are obtained. The time histories of accelerations, displacements, stresses...etc are obtained by using fast-fourier transformation techniques.

Fig. 7 shows the soil-structure finite element model used to perform the analysis and the location of the transmitting boundary. The bedrock at a depth of 1000' is assumed rigid in the analysis.

Fig. 8 shows the acceleration profiles obtained for the different structural components when the control ground motion is stipulated at the foundation level.

Fig. 9 shows the acceleration profiles obtained when the control ground motion is stipulated at the surface.

Fig. 10 shows typical Amplified Response Spectra (ARS) for Case D' (tied configuration) at the top of the PCRV. The comparison shows that there is deamplification of response intensity and a shifting of the resonant peaks when the ground motion is applied at the surface. This indicates a filtering of ground motion due to embedment.

5. Conclusions

From the previous studies presented, the following conclusions can be made:

(1) The design of the primary reinforcing steel in reinforced concrete containment structures is usually governed by the effects of the DBA pressure and the associated temperature for sites with SSE ground accelerations up to 0.2g. When the SSE is equal to or greater than 0.3g, earthquake forces become very important in the design of the pressure boundary and govern the design in many areas.

(2) For intense ground motions, (SSE in the range of 0.5g and greater

special conceptual design techniques must be employed to control the shear and tension forces in the pressure boundary.

(3) Sophisticated analytical techniques are indispensable when plant design for high seismic ground motion is contemplated.

(4) The way the ground motion is specified (at the surface or the foundation level) is an important factor to be considered in the analysis. For the cases studied, it was demonstrated that pressure boundary acceleration profiles obtained when the control ground motion was stipulated directly at the foundation level is 20 - 25 percent higher than those obtained when the motion is stipulated at the surface. The Amplified Response Spectra (ARS) showed somewhat similar trends but they showed a change in their frequency content.

(5) For the cases studied, increasing the shear modulus of the underlying subgrade resulted in higher acceleration profiles and higher forces in the containment. This phenomenon suggests that care should be exercised when the input parameters for seismic models are chosen.

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TABLE I
CONTAINMENT STRUCTURE IN-PLANE FORCES

<u>FACTORED COMBINATION LOAD</u>	<u>1.ODL + 1.5P + T</u>			
CASE	A	B	C	D
MERIDIONAL FORCE, Nm, AT BASE OF CYLINDER k/ft.	291	278	251	251
HOOP FORCE, Nh, 40 Ft. ABOVE BASE k/ft.	667	820	790	790
TANGENTIAL SHEAR FORCE, Ve, k/ft.	0	0	0	0
<u>FACTORED COMBINATION LOAD</u>	<u>1.ODL + 1.OP + 1.OT + 1.OSSE</u>			
CASE	A'	B'	C'	D'
MERIDIONAL FORCE, Nm, AT BASE OF CYLINDER k/ft.	336	560	768	315
HOOP FORCE, Nh, 40 Ft. ABOVE BASE k/ft.	453	540	540	540
TANGENTIAL SHEAR FORCE, Ve, k/ft.	98	240	386	193* 132**

* Ve force in containment wall at structural tie.

** Ve force in containment wall at base.

- CASE A, A' - 3400 MWt PWR, 140 ft. ID, P = 45 psi
T = 150°, SSE = 0.17g, rock subgrade
- CASE B, B' - 3800 MWt PWR, 150 ft. ID, Reference Plant,
P = 48 psi, T = 150°, SSE = 0.3g, soil subgrade
- CASE C, C' - HTGR Plant, 150 ft. ID, P = 45 psi
T = 120°, SSE = 0.5g, soil subgrade, containment and annular
building untied
- CASE D, D' - Same as CASE C, but containment is tied to annulus building.

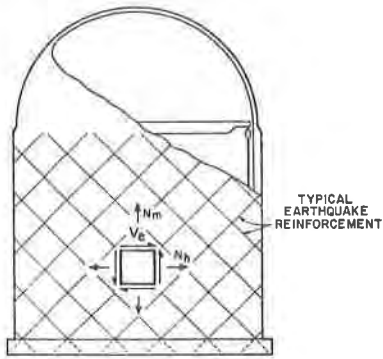


Figure 1
Pressure Boundary in-plane Forces

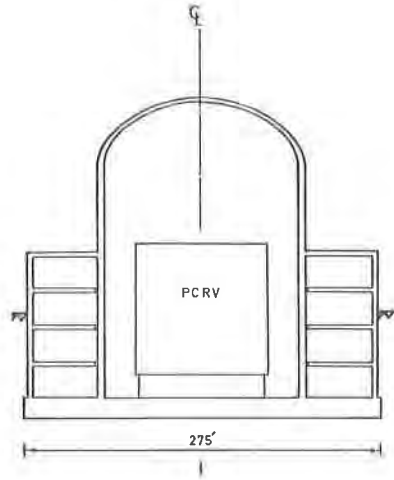


Figure 2
Cross Section of HTGR Plant for a High Seismic Region

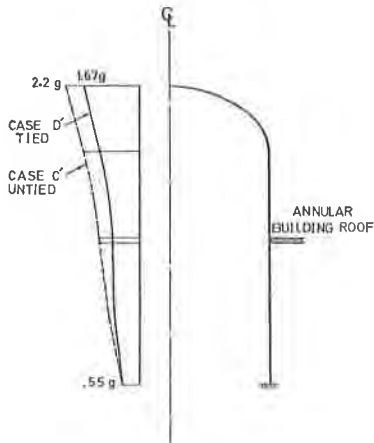


Figure 3
Pressure Boundary Acceleration Profiles
(SSE = 0.5g)

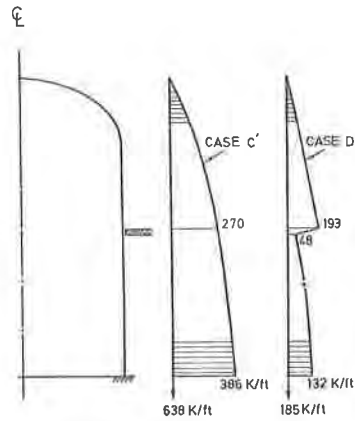


Figure 4
Pressure Boundary in-plane Forces due to Earthquake Loading only (SSE = 0.5g)

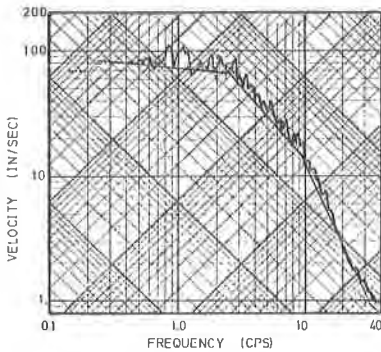


Figure 5
Ground Design Spectrum and Artificial Time History Spectrum Used in the Analysis

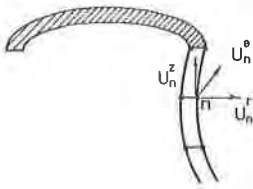


Figure 6
Semi-Analytical Finite Element Solution

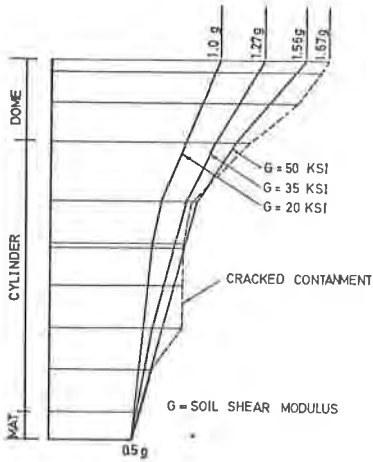


Figure 8
Acceleration Profiles - Foundation Level Motion

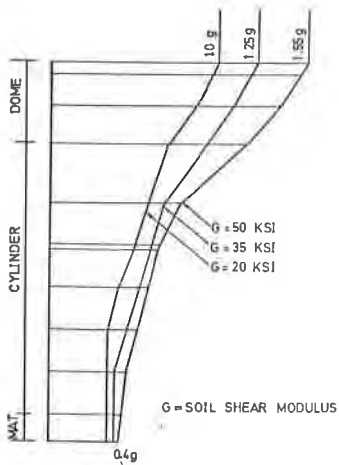


Figure 9
Acceleration Profiles - Surface Motion

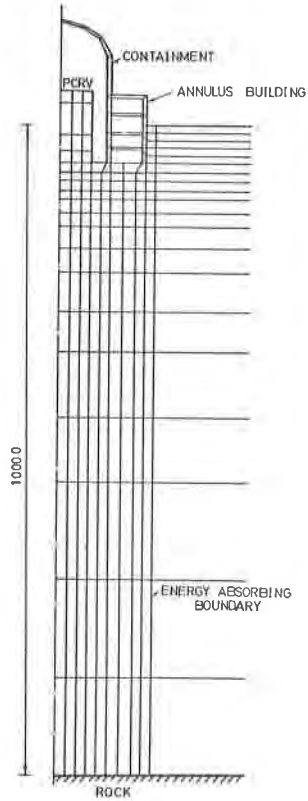


Figure 7
Soil-Structure Finite Element Model

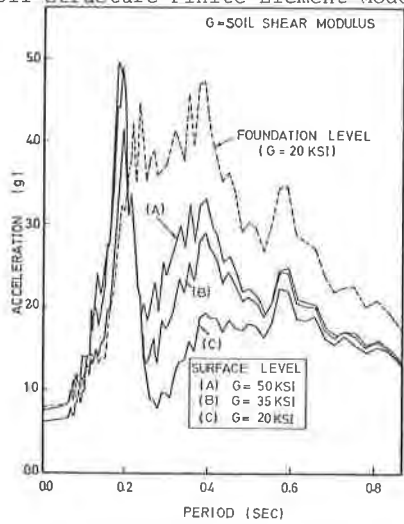


Figure 10
ARS at Top of the PCRV