Performance of Steel Containment Anchorage System During a Severe Accident

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INTRODUCTION

During the past few years, several researchers have increased efforts to investigate the performance of nuclear power containment buildings during hypothetical severe accidents. Failure of a containment building to perform its function is considered to take place when a leak path is formed. After performing scaled model tests, the containment integrity division of Sandia National Laboratories (SNL) has suggested that steel containment buildings will fail by rupturing when resulting plastic strains reach the ultimate strain of the containment material (Clauss, 1987). The researchers at SNL have concluded that the actual geometry must be analyzed when the suggested strain criterion is applied. Among the several leakage paths is the containment building anchorage system. Structural deformation of the anchorage system could produce leakage paths from the containment pressure boundaries that could result in the release of radioactive material. This paper summarizes some of the results of an ongoing research project sponsored by Sandia National Laboratories. The objective is to predict the internal pressure at which leakage induced by deformation of the anchorage system occurs. A typical anchorage system of an ice condenser containment building was selected for investigation by the ABAQUS finite element computer program.

<u>Failure Modes of a Nuclear Containment Anchorage System Nuclear</u>: Power plant steel containments are free-standing structures often held down by a number of pretensioned, high-strength bolts (see Fig. 1). These anchors are distributed around the containment building base and are embedded in a concrete floor. The failure modes of headed bolts are classified in (ACI 349-80) as: (1) failure of the anchor itself (ductile failure), and (2) failure in the concrete section (brittle failure). Brittle failure of a concrete connection is catastrophic and must be avoided. On the other hand, ductile failure allows for movement and redistribution of forces. It provides some warning of impending collapse if the movement is being monitored. In any case, the failure of the containment anchorage system must be considered as a potential failure mode when the containment performance is predicted.

<u>Ductile Failure</u>: Ductile failure results in the elongation of the bolts themselves. Typically, the bolts are made of ductile steel and; therefore, it is reasonable to assume that the failure of these bolts occurs when the tensile stress exceeds the bolt material tensile strength, F_u. This assumption is consistent with the criterion proposed by Sandia. Summing vertical forces at a pressure p_u, all bolts reach their maximum specified tensile strength F_u when: $P_u = (n\ A_b\ F_u/p\pi r^2)$; where n is the number of bolts, A_b is the bolt cross-sectional area and r is the radius of the containment vessel.

Brittle Failure: Brittle failure occurs when the embedded steel anchor does not fail in tension but rather pulls out a section of the concrete. The pulled-out section of concrete is approximately conical, and the failure is caused by diagonal shear forces and tension on the conical surface (ACI 349-80). In this case, the pressure which causes failure is calculated as (ACI 349-80): $p_u = (4 \sqrt{f'_c}\, A_n/p\pi r^2)$; where $\sqrt{f'_c}$ is the concrete compressive strength and A_n is the effective stress area, defined by the projected area of stress cones. The procedure outlined above can be used to predict approximate pressures at which failure may occur in an anchorage system of a nuclear containment. Considering the importance of this problem, a more reliable and comprehensive approach to investigate the behavior of the anchorage system is desirable. A three-dimensional finite element analysis of the anchorage system of a typical steel containment is summarized below.

SENSITIVITY STUDY OF MODELING PARAMETERS

Problems associated with the idealization of reinforced concrete structures for finite element analysis were investigated. These include bond behavior, dowel action, shear retention stiffness in cracked concrete, and rebar idealization. Simple models with relatively few elements were analyzed by the ABAQUS (1985) finite element program to address these issues.

Bond Idealization: Bond behavior can be accounted for by using three methods. In the first approach, the reinforcing bar elements are connected to the concrete elements with link elements consisting of orthogonal springs. Various formulations for the bond stress-slip relationship which may be used to define the spring behavior are summarized by Gerstle et al. (1982). Another alternative is to allow for the concrete to carry some fictitious tensile stress even after reaching its tensile strength. This method, referred to as "tension stiffening," (Scanlon, 1971). and ABAQUS (1985). The third method to account for bond behavior involves the use of a modified stress-strain relationship for the reinforcing steel, as introduced by Gilbert and Warner (1977).

The tension stiffening concept described herein was selected for its ease of application and aid to solution convergence. Tension stiffening can be incorporated into ABAQUS (1985) by expressing the retained tensile stress (beyond concrete tensile strength) as a linearly decreasing function of strain normal to the cracks. Schnobrich (1986) suggested different values for the maximum strain of, ε_y, $10\varepsilon_c$ and $3\varepsilon_y$ (see curves A, B and C in Fig. 2); where ϵ_y is the rebar material yield strain and ϵ_c is the concrete cracking strain. These were used to investigate the behavior of the tension member shown in Fig. 3. A quarter-symmetry model consisting of solid elements (C3D8 in ABAQUS) was used for idealizing the tension member. An axial load was applied in increments at the end face of the rebar, and a sufficient number of iterations within each load increment were allowed to ensure convergence.

Figure 4 illustrates steel stress versus displacement at the end face of the rebar. The analysis with no tension stiffening resulted in a cracking load lower than the experimental and analytical results obtained by Houde (1973) and Khouzam (1977). On the other hand, the use of tension stiffening as represented by curves A and B in Fig. 2 yielded results bounded by those obtained with curve C and no tension stiffening. Although curve C gives results closer to the experimental results, it cannot be used in analysis. A value of strain greater than the yield strain of the reinforcement would artificially increase the total stress in the direction of any yielded reinforcement (Schnobrich, 1986).

Dowel Action and Aggregate Interlocking: To study dowel action and aggregate interlocking, the shear wall tested and analyzed by Al-Mahaiidi (1979) (Fig. 5 was investigated). A comprehensive analytical expression for convenient adaptation of dowel stiffness in finite element analysis is still not available (Cedolin et al., 1982). Dowel action and aggregate interlocking were modeled by other investigators using a reduced shear modulus G, in the material matrix. Cedolin and Dei Poli (1977) observed that a linearly decreasing shear modulus as a function of the strain ϵ_1 normal to the crack yields a better representation of shear behavior. A similar hyperbolic expression developed by Al-Mahaiidi (1979) was used in this work. This is defined as:

$$G = 0.4G_1/(\varepsilon_1/\varepsilon_o), \text{ for } \varepsilon_1 > \varepsilon_o \quad [1] \; ; \quad G = G_1, \quad \text{for } \varepsilon_1 < \varepsilon_o \qquad [2]$$

where ε_o is the cracking tensile strain and G_1 is the elastic shear modulus of uncracked concrete. In modeling the shear wall problem for analysis, solid elements (C3D8 in ABAQUS) were used to idealize the concrete while truss elements (C1D2 in ABAQUS) were used to idealize the reinforcing bars. Reinforcement at each end of the panel were represented by two layers as shown by solid lines in Fig. 5. Bond behavior was incorporated by the tension stiffening approach outlined earlier.

Figure 6 illustrates the load-displacement relationship obtained from two analyses by using different tension stiffening curves (curves A and B in Fig. 2). Also shown are the experimental and analytical results obtained by Al-Mahaiidi (1979). The experimental result was adjusted by Al Mahaiidi to reflect the effects of true fixity at the base, which cannot be duplicated in the experiment. As can be seen, the results obtained by using curve B for tension stiffening yielded results closer to the adjusted experimental results.

Based on the studies summarized above, the tension stiffening (curve B in Fig. 2) was chosen to model bond behavior and the shear retention given by Eqs. 1 and 2 were chosen for the analysis of the containment anchorage system. Discrete rebar idealization was selected over smeared reinforcement after the arrangement of rebars within the containment base was considered.

ANALYSIS OF CONTAINMENT ANCHORAGE SYSTEM

An anchorage system of an ice condenser steel containment building was selected for investigation under static internal pressure and its own weight. The containment building is a cylindrical steel structure covered by a hemispherical dome and fabricated with A516 Gr.60 steel. This is enclosed by a 3-foot-thick reinforced concrete shield wall. A 1/4-inch thick steel-liner plate forms the base of the containment vessel. This plate is embedded in the concrete mat and covered by 2 feet of reinforced concrete. Anchorage is provided by 180 high-strength preloaded anchor bolts, 2.5 inches in diameter and 8 feet long. The bolts are embedded in a 12-foot-thick concrete mat, which in turn is anchored to the rock beneath with tie rods grouted into 10- to 15-foot deep holes. The model idealizes portions of the containment vessel and shield building, the anchor bolt, the anchor plate, the stiffener plates, the reinforced concrete mat, and the soil foundation material (see Fig. 7). The heights of the portions of the shield and containment building in the model were limited to 3 rt, where r and t are the mean radius and thickness of each, respectively. The finite element mesh was generated with PATRAN (1987, release 2.2) on Microvax II. Initially, the concrete mat was modeled using 20-node solid elements (C3D20R in ABAQUS), the containment vessel with 8-node shell elements (S8R), and the anchor bolt with 3-node truss elements (C1D3). This model contained 2904 elements with nearly 15,000 nodes. Loads were applied at the nodes at the top of the shell model to simulate forces induced in the meridional direction by an applied internal pressure.The weight of the structural components inside the containment was approximated to be 500 psf. The bolt preload was incorporated by using the initial conditions parameter available in ABAQUS. The concrete and rock interface at the bottom was idealized with linear springs the carry compression only. The tie rods between the rock and the concrete mat were idealized by nonlinear springs. Appropriate boundary conditions were imposed on the symmetry planes of the model.

The analysis was begun on Cray XMP-416 with an initial internal pressure of 20 psi. However, the execution was terminated after 1 hour of CPU time because of time limitations. Therefore, the model was rebuilt with 8-node solid (C3D8), 4-node shell(S4R) and 2-node truss (C1D2) elements. Mesh size was also increased slightly to reduce the total number of elements. The resulting model has 1934 elements having approximately 4000 nodes.

SUMMARY

A nonlinear analysis is being conducted to evaluate the performance of an anchorage system of a nuclear power plant steel containment under hypothetical accident conditions. Sensitivity studies demonstrated the bond and shear behavior in reinforced concrete can be adequately represented by the tension stiffening and shear retention, respectively . A three-dimensional analytical model for the analysis of the anchorage system was developed and is currently being analyzed using the ABAQUS finite element program. The critical regions for failure will be investigated to identify the most likely mode of failure. Conclusions relating the performance of the anchorage system during a severe accident to the overall containment strength will be drawn.

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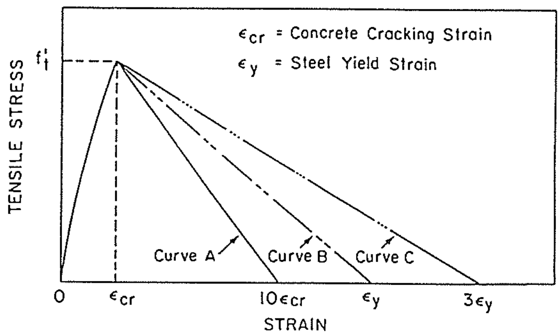

Fig. 2. Models to account for tension stiffening in cracked concrete

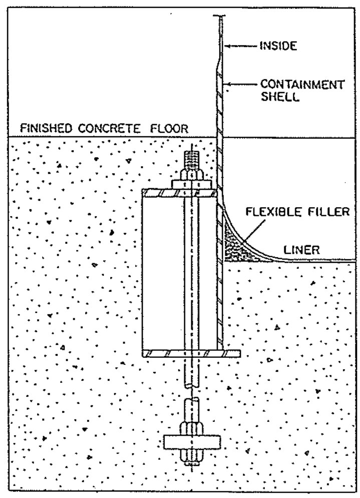

Fig. 1. Containment building base connection

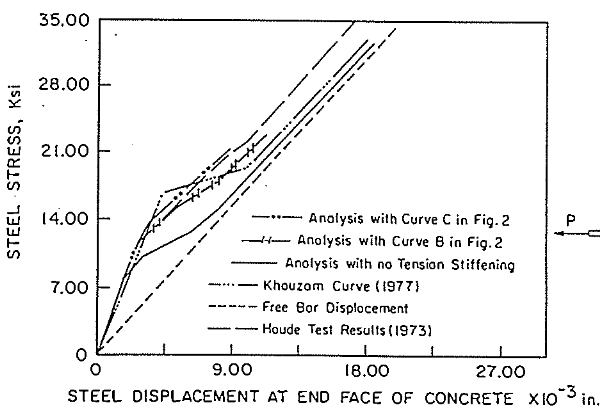

Fig. 4. Stress-displacement relationships for the tension member in Figure 3

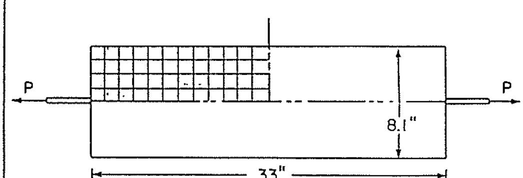

Fig. 3. Tension member used in analysis

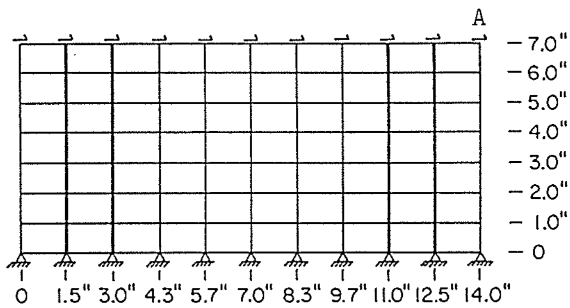

Fig. 5. Finite element idealization of shear wall

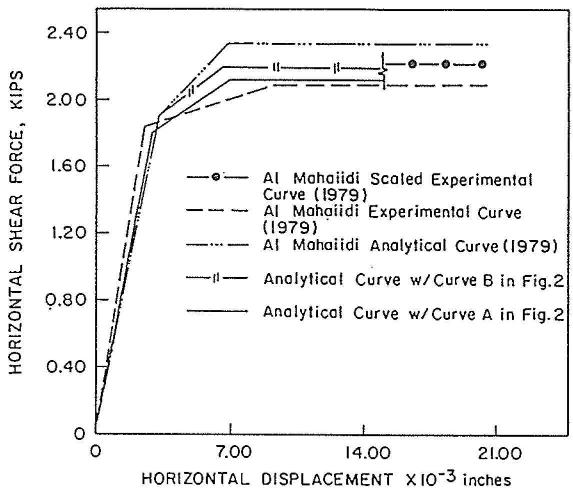

Fig. 6. Load-displacement at Point A of shear wall in Figure 5

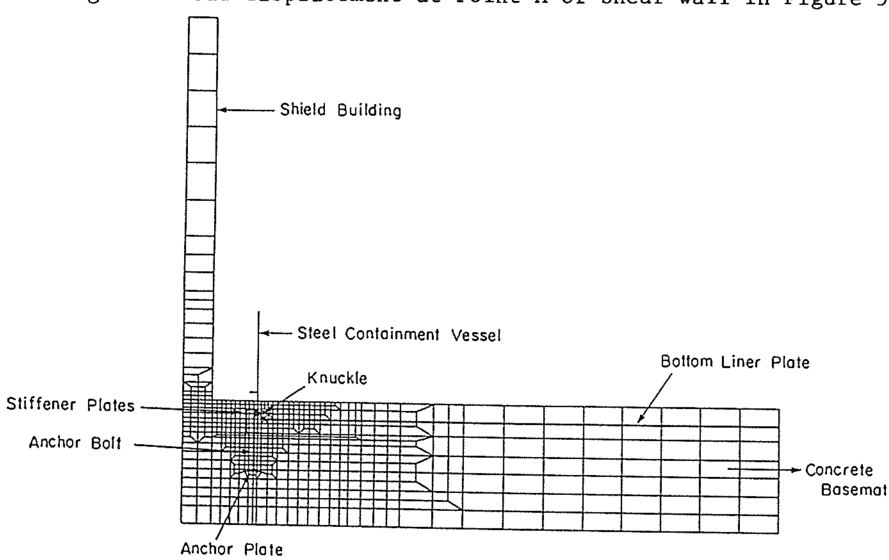

Fig. 7. Finite element model for analysis of containment anchorage system