NFAP Calculation of the Response of a 1/6 Scale Reinforced Concrete Containment Model

C. J. Costantino
City University of New York, New York, NY USA
S. Pepper, M. Reich
Brookhaven National Laboratory, Upton, NY USA

ABSTRACT

The details associated with the NFAP calculation of the pressure response of the 1/6th scale model containment structure are discussed in this paper. Comparisons are presented of some of the primary items of interest with those determined from the experiment. It was found from this comparison that the hoop response of the containment wall was adequately predicted by the NFAP finite element calculation, including the response in the high pressure, high strain range at which cracking of the concrete and yielding of the hoop reinforcement occurred. In the vertical or meridional direction, it was found that the model was significantly softer than predicted by the finite element calculation; that is, the vertical strains in the test were three to four times larger than computed in the NFAP calculation. These differences were noted even at low strain levels at which the concrete would not be expected to be cracked under tensile loadings. Simplified calculations for the containment indicate that the vertical stiffness of the wall is similar to that which would be determined by assuming the concrete fully cracked. Thus, the experiment indicates an anomalous behavior in the vertical direction.

INTRODUCTION

Brookhaven National Laboratory (BNL), along with a number of other organizations, undertook in the Fall of 1986 to perform a detailed response analysis of the 1/6th Scale Model containment structure constructed and tested at The Sandia National Laboratory (SNL). This reinforced concrete facility, (FIG. 1) designed as a model of a containment structure for a nuclear power plant, was pressurized to failure in July, 1987. Each of ten organizations involved undertook the task of predicting the response of the pressure vessel well into the nonlinear regime and potential mode of failure of the vessel. One goal of this study has been to improve the analysis capability available to the engineering community, particularly with regard to the ability to forecast behavior at and near failure of the various structural materials involved.

BNL PREDICTION MODEL - THE NFAP CODE

BNL has helped develop and exercised a large scale computer program to study the nonlinear behavior of reinforced concrete structures near failure. This effort has been aimed at developing complete analytic material models which have applicability to all sorts of concrete structures, including containment facilities. The analysis tool bears the acronym NFAP, standing for the Nonlinear Finite Element Analysis Program. For this calculation, the containment structure is simulated as a two-dimensional axisymmetric finite element model, subjected to the effects of both gravity loadings as well as statically applied internal pressures. Effects of penetrations have been neglected as having small influence on the global pressure response of the structure. Large displacement effects have been included in this calculation. As can be expected for this highly nonlinear calculation, the numerical solution proceeds in a standard incremental fashion.

MODELING OF MATERIALS

The specific details of the material models used in the NFAP Code are presented elsewhere (Sharma et al, 1983, 1985, Costantino et al 1987, 1988). For any simulation of reinforced concrete structures, the interaction between steel and concrete is typically handled in one of two ways. First, some of the steel reinforcement is modeled explicitly by means of axial (truss) elements connected to specific node points of the mesh. In this containment calculation, the stirrups located in the lower section of the cylinder wall are modeled directly via steel truss elements. The vast majority of steel reinforcement is modeled in the analysis using the distributed or smeared concept for reinforced concrete. In this approach, the steel is smeared into the concrete element, resulting in a locally homogeneous material making up the element. The separate constitutive properties of each material component are tracked throughout the calculation to ensure that individual stress paths are suitable.
modeled. Specific smearing and desmearing procedures are then used to ensure that proper node point loads are determined which satisfy the required equilibrium equations.

Uniaxial Steel Rebar

The stress-strain data provided for the rebar essentially consists of three sections, namely, an initial elastic zone, a short perfectly plastic zone, and a hardening section which is approximately parabolic in shape. For this calculation, however, the stress-strain data for the steel reinforcement above the yield point has been represented as a simple linear hardening material. A linear regression analysis was used to best fit the stress-strain data above the yield point.

Multiaxial Steel Models

The constitutive model for the multiaxial steel, such as the liner materials, is based on the standard incremental theory of plasticity, using the von Mises yield criterion and an associated flow rule. For this calculation, the steel was assumed to behave as an isotropically hardening material, with no translation of the yield surface. Such a model is generally adequate for those cases involving monotonically increasing loading with no significant unloading, as applies for most of this problem. Again, regression analyses were performed to best fit the data provided to describe the hardening behavior of the liner steel.

Concrete Stress-Strain Model

The concrete elastic-plastic material model (Chen, 1975, 1980) is used in NFAP to predict the nonlinear behavior of the concrete. This model has been found to yield reasonable agreement with experimental data and has been employed for other failure evaluations of concrete containments. The model is based on separate assumed failure surfaces in the tension and compression zones in stress space. In the plastic flow regime, defined by stress states on the current yield surface and incremental stresses indicating loading, the plastic strain increments are determined by the usual normality rule.

The ability of the concrete to fracture in both tension (cracking) and compression (crushing) has been included in the analysis. A dual stress and strain criterion is used to define the initiation of cracking, which are based on the stress and strain invariants for multiaxial states, and are correlated with available experimental data for cracking strains in uniaxial compression and tension. When the concrete crushes under compressive loading, the stiffness of the concrete element is assumed to be zero for additional loading of the material. For the case of tension cracking, however, the stiffness and stress terms associated with strains normal to the crack direction are reduced to a small fraction of their initial value. Due to difficulties with excessive grid redefinition, the modeling of discrete cracks within the finite element mesh is impractical, except for those predefined cracks which are modeled along interelement boundaries before the start of computations. Consequently, an approximate procedure is used within NFAP in which the cracks are assumed to develop over finite areas within the element, using specified cracking criteria. When cracks are computed to occur at a specific point within an element, the material constitutive matrix and internal stress condition is suitably modified in accordance with the computed crack orientation. The concrete is still, however, considered to be a homogeneous though nonisotropic material. Thus, rather than representing the physical material separations developed by the crack, the gross effect of the crack on the structural stiffness of the material is considered directly.

Soil Foundation Model

It has been found that relative basemat movements can have an important effect on the calculation of stresses at the basemat-cylinder wall intersection, a potential failure point in the structure. Therefore, in our NFAP calculation, it was felt important to include the soil flexibility in the model. The soil is represented as a bilinear elastic Winkler foundation, that is, a one dimensional soil element with a relatively high stiffness under compressive loading and a zero stiffness under tensile loading.

FINITE ELEMENT MODELING

The general configuration of the finite element model used in this calculation is shown in Fig. 1. The model is an axisymmetric one, and includes both the mudmat and foundation soil in the structural model. The complete finite element model consists of 1549 node points and 539 elements. Of the 539 elements, 459 are two dimensional, 8-noded isoparametric elements, while 80 are one-dimensional axisymmetric truss elements. The truss elements are used to represent the Winkler soil springs and shear steel (stirrups) in the lower cylindrical wall. Generally, the mudmat is simulated by means of three elements through its thickness, the basemat by seven elements through the thickness, and the cylindrical wall and dome by seven elements through the thickness. In addition, separate elements are used to represent the steel liner plate in the various sections, with the liner considered attached to the concrete at its node points. Thus, a total of eight elements are used to represent the through-thickness properties of the structural wall, dome and basemat.
RESULTS OF CALCULATIONS

As mentioned previously, this axisymmetric model was subjected to gravity and internal pressure loadings. The large
displacement formulation was included in the calculation and a modified Newton-Raphson incremental solution scheme was
used. Gravity loads were first turned on, and the solution obtained. Following this, a total of 89 load steps were applied to the
containment model by incremental pressure loadings. Initially, 5 psi pressure increments were used until a pressure of 40 psi
was reached. Beyond this pressure, the incremental loadings were reduced to eliminate convergence problems due to nonlinear
material effects, as well as improving the tracking of concrete cracking. From 130 psi to the end of the calculations, stopped
at 142 psi, pressure increments of 1/2 psi were used as extensive yielding occurs in this pressure range.

Structural Deformations

A chronology of the response of the containment to the applied loads indicates the primary characteristics of its behavior.
The effect of the dead load initially applied is to essentially develop a vertical rigid body movement of the structure into the
foundation soil of about 0.17 inches. As internal pressure is applied, the containment vessel responds primarily elastically
until the pressure reaches a value of about 30 psi. At this pressure, some nonlinear concrete strains develop in the outer
elements of the cylindrical wall due to shell bending. Of more importance, however, concrete flexural cracking begins at the
inside face of the cylinder at the intersection with the basement. The dome remains essentially elastic. As the applied pressure
reaches 40 psi, hoop cracking begins in the outer elements of the cylinder wall near mid-height. The deformations now begin
to significantly increase, as the cracking at the basement/cylinder wall junction extends into the wall. At about 70 psi, bending
cracks begin to form at the center of the basement, as the basement curls up due to the pressure loads. Shear cracks now develop
in the cylinder wall above elevation 6 6", above the zone with shear steel (stirrups). This in turn leads to an increase in the
zone of hoop cracking as the shear stiffness near the bottom of the cylinder decreases. In addition, shear cracks develop in the
dome, immediately above the top of the seismic steel area.

At about 80 psi, multiple cracking develops in the wall/basemat junction as the cracked zone in this area increases. The
stresses in the meridional steel dowels embedded into the basemat become larger as the zone of cracked concrete in this area
increases. At about 90 psi, the cracked zone at the wall/basemat junction extends through the entire wall thickness, and hoop
cracking extends throughout most of the superstructure. The area of bending cracks in the basemat extends over a larger depth,
but the stresses in the steel in this area are still small. The steel liner is still elastic, but peak stresses are of the order of 50 ksi
in the center of the cylinder wall. At about 104 psi, yielding begins to develop in the steel rebar dowels which extend into the
basemat below the cylinder/wall junction. In addition, some yielding of the liner occurs at midheight of the cylinder. The
stresses in the dome reinforcing are still reasonably small, reaching a value of about 25 to 30 ksi. At about 124 psi, yielding
of the hoop steel begins in the cylinder wall in the area above the stirrups, and yielding of the dowel steel becomes more
extensive. Strains in the basemat dowels reach a value of about 0.9%, or about four times yield. The plastically deformed area
of the steel liner also spreads upwards towards the springline, while liner stresses in the dome still remain below yield. Total
yielding of the hoop steel occurs at about 128 psi pressure, above which the calculations indicate a rather general ballooning
of the superstructure. The hoop steel continues to deform plastically, although hoop strains are still only about two times the
yield strain for the rebar. The calculations were cutoff at a pressure of 142 psi.

Strain Comparisons

In addition to the general response of the containment profile discussed above, some comparisons of strains computed at
various sections of the wall with measured data are presented. The hoop strain developed in some rebar at midheight of the wall
is presented in Fig. 2, and is compared with measured data. In this figure, the computed strains in the element are uniform, as
can be expected in the hoop direction and compare favorably with the measured data. Small differences at pressures around and
above 130 psi occur, when steel yielding develops, which can generally be attributed to the small differences in modeling of
the yield point of the rebar material. Similar comparisons of computed hoop strains with measured data from other locations
in the wall also indicate good comparisons. From these comparisons, it is concluded that computation of hoop behavior of
the cylindrical wall is excellent, this agreement extending well into the region where significant concrete tensile cracking and
yielding of the hoop reinforcement occurs. This lends confidence in both the adequacy of the computer formulation as well as
the details of the computer material models to simulate failure of reinforced concrete.

In the vertical direction, meridional strains at various elevations above the base were compared to corresponding measured
data. In Fig. 3, the comparison of vertical strains near the spring line show that the computed strains are significantly less
than those measured in the experiment. Similar comparisons at other locations along the vertical wall indicate the same
behavior. In all cases, the effective stiffness (slope of the pressure-strain curves) of the actual containment is about 4 to 5
times stiffer than that indicated by the analytic solution. The measured strains, in fact, agree approximately with the strains
that would develop in the axial direction if the concrete were fully cracked. Thus, the assumption of the effective development
of bond between the vertical steel and concrete, assumed to be perfect even at low stress levels in the analytic model, is an area
of concern requiring further evaluation. At the bottom of the containment where the vertical steel turns into the slab, the development of the bar is in fact not so dependent on bond. The comparison of predicted to measured vertical strains for this case was found to be good. Some discrepancies appear across the thickness of the computational element, indicating that in this region where bending is significant, the element mesh required to adequately predict strains must be finer than was actually used in the calculation. Other comparisons at similar locations low in the wall again compare favorably with those predicted, with variations over the finite element indicating the impact of bending variation in these strains.

CONCLUSIONS

On the basis of this complex calculation, several conclusions can be reached, these having to do with (a) the behavior of the containment vis-a-vis the NFAP calculation and (b) the development of potential modes of failure that may develop as a generic containment is pressurized. With respect to the first issue, namely, the comparison of the prediction with the experiment, two primary aspects can be mentioned. The prediction of the large strain hoop response of the cylinder wall was found to be in good agreement with the experimental results, leading to a conclusion that the concrete models used in the calculation reasonably predict behavior of reinforced concrete even into the large strain range of interest at failure. However, this conclusion should be tempered with the fact that in the hoop direction, the strain-displacement relations are known, and the axisymmetric behavior is essentially one-dimensional in the hoop direction. Thus the intuitive nature of the stress-strain relations should be expected to yield good results for the hoop response.

In the vertical direction, agreement between measured and predicted strains was not found. Discrepancies in vertical stiffness of the order of three to four times were noted, with the experimental displacements being much larger than computed throughout the pressure history of the containment. Such discrepancies were also found by other agencies using different calculational models. Apparently, the containment wall behaves as if the concrete provided no stiffness and its stiffness were derived solely from the vertical steel. Such behavior may be due to various factors, such as improper bond development of the relatively large bar sizes used in the model as compared to the wall thickness. Of more importance to the design of actual containments, it is recommended that vertical and diagonal steel should be designed so that they may be developed without reliance on bond (positive tiedown), particularly in regions where extensive cracking will be expected to develop.

As concerns the development of potential failure conditions in the calculational model, two primary modes of potential failure of the containment develop as internal pressure is applied. First, total yielding of the hoop steel occurs at some pressure which extends well into the dome area. At this pressure, the calculations indicate a general ballooning of the containment structure as additional pressure is applied. Secondly, the calculations also indicate a second potential failure mode developing at the base of the cylinder wall. At some pressure, the cracked zone of concrete extends through the thickness of the cylinder wall. Additional pressure then causes this zone to extend into the basemat and at the same time cause yielding of the dowel steel in both the inner and outer faces of the containment wall. The calculated strains in the dowels are about 4 times the yield strain at these pressures. Continued pressure can then be applied to the structure until either pullout of the dowels occurs or the cracking of the concrete reaches the outer edge of the basemat, which is unreinforced. Such behavior could lead to a localized failure and therefore an inability to support additional pressures.

In the actual containment model, failure developed not by yield of the basemat/wall junction or the hoop steel, but by tearing of the steel liner and eventual loss of pressure retaining capacity. The indications from the post-test discussion are that the liner tearing strain is a function of not only the global stretching of the liner as determined from an axisymmetric calculation as described herein, but is also a function of three dimensional effects introduced at or near penetrations as well as the specific mechanism used to attach the liner to the concrete wall (in this case, studs). Such aspects can not specifically be included in this global approach, but must be empirically related to the global strain fields.

REFERENCES

Costantino, C.J., Pires, J., Pepper, S., "BNL Pretest Prediction of Pressure Response of 1/6 Scale Model Containment Structure", Document No. 01-1703, Brookhaven National Laboratory, for Sandia National Laboratory, January 1987
Costantino, C.J., Pepper, S., Reich, M., "NFAP Calculations of the Pressure Response of the 1/6th Scale Model Containment Structure", Brookhaven National Laboratory, for the US Nuclear Regulatory Commission, February, 1988
Sharma, S., Reich, M., Chang, T.Y., "Review of Current Analysis Methodology for Reinforced Concrete Structural Evaluations", NUREG/CR-3284, April, 1983
FIGURE 1 CONFIGURATION OF FINITE ELEMENT MODEL FOR CONTAINMENT
FIGURE 2  STRAIN IN LAYER 6 HOOP STEEL AT ELEVATION 6' 9"

FIGURE 3  AXIAL STRAIN IN LAYER 2 MERIDIONAL STEEL