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SEISMIC ANALYSIS OF A REINFORCED CONCRETE CONTAINMENT VESSEL MODEL

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ABSTRACT
Pre- and post-test analytical predictions of the dynamic behavior of a 1:10 scale model Reinforced Concrete Containment Vessel are presented. This model, designed and constructed by the Nuclear Power Engineering Corp., was subjected to seismic simulation tests using the high-performance shaking table at the Tadotsu Engineering Laboratory in Japan. A group of tests representing design-level and beyond-design-level ground motions were first conducted to verify design safety margins. These were followed by a series of tests in which progressively larger base motions were applied until structural failure was induced. The analysis was performed by ANATECH Corp. and Sandia National Laboratories for the United States Nuclear Regulatory Commission, employing state-of-the-art finite-element software specifically developed for concrete structures. Three-dimensional time-history analyses were performed, first as pre-test blind predictions to evaluate the general capabilities of the analytical methods, and second as post-test validation of the methods and interpretation of the test results. The input data consisted of acceleration time histories for the horizontal, vertical and rotational (rocking) components, as measured by accelerometers mounted on the structure's basemat. The response data consisted of acceleration and displacement records for various points on the structure, as well as time-history records of strain gages mounted on the reinforcement. This paper reports on work in progress and presents pre-test predictions and post-test comparisons to measured data for tests simulating maximum design basis and extreme design basis earthquakes. The pre-test analyses predict the failure earthquake of the test structure to have an energy level in the range of four to five times the energy level of the safe shutdown earthquake. The post-test calculations completed so far show good agreement with measured data.

INTRODUCTION
The seismic behavior of reactor containment structures has been under investigation during the past several years in a collaborative program between the United States Nuclear

DISCLAIMER

Regulatory Commission (USNRC) and the Nuclear Power Engineering Corporation (NUPEC) of Japan. NUPEC designed, constructed and tested two 1:10 scale models: a prestressed concrete containment vessel (PCCV), and a reinforced concrete containment vessel (RCCV). The models were tested using the high-performance shaking table at the Tadotsu Engineering Laboratory in Japan. The USNRC provided analytical evaluation, including pre- and post-test analyses, of the two models, with the objective of evaluating the predictive capabilities of currently available state-of-the-art software for the nonlinear time history analysis of concrete structures. The primary objective of the testing program is to demonstrate the capability of the PCCV and RCCV to withstand extreme design earthquakes with a significant margin of safety against major damage or structural failure. The present paper is restricted to the RCCV investigation; the PCCV analysis was reported on previously (NUREG/CR-6639, 1999).

The strategy followed in the analysis of the RCCV is to document and report the pre-test blind predictions, including the prediction of failure earthquake, prior to the start of the testing program. Post-test calculations are used to evaluate the effects of assumptions used in the pre-test calculations relative to actual conditions experienced in the tests. Because of the nonlinear behavior of RCCV structures, even for design-level input motions, the analyses must follow the same sequence as the test series in order to properly account for the accumulation of damage in the structure. However, the large number of tests performed made such an endeavor very expensive, and it was necessary to be selective in the number of analyses to be performed. Moreover, the motions recorded at the basemat during the test differ significantly from the targeted pre-test input motions. Consequently, the pre-test analysis results could only be used to provide a general trend for the damage and failure regimes of the structure. The post-test analyses will be used to interpret the test results and evaluate the predictive capabilities of the analysis method. A description of the computational model and pre-test analysis results was presented at SMiRT-15 (James, 1997). This paper emphasizes the post-test analysis. However, because the post-test analysis is still ongoing, only partial results are available.

RCCV TEST MODEL

The actual containment structure is connected to the reactor building through intermediate floors that are constructed as integral parts of the RCCV walls and the reactor building. The scaled test model is designed to be representative of the actual containment structure while meeting the limitations of the test equipment and the requirements needed for fabrication. A section containing the RCCV and intermediate floors must be isolated to form the RCCV test model. The isolated structure was scaled to fit within the capacity of the shake table while retaining the size and fabrication characteristics so that the structural behavior of the model remains compatible with the full-scale structure. The RCCV test model is depicted in Fig. 1 in the form of a finite element grid. The test model consists of a 0.2-m thick by 2.41-m tall reinforced concrete cylinder with an inside diameter of 3.625 m and a 1-m thick concrete basemat. The basemat is 9 m square and is bolted to the shake table. The access tunnel is represented as a 0.538-m diameter penetration through the RCCV wall along a diameter that is 90 degrees from the axis of the horizontal shaking direction. Two intermediate concrete floors, each 130 mm thick, frame into the cylindrical wall creating three nearly equal axial segments. The floors extend horizontally for a distance of 1 m from the wall so that the outer diameter of the floors is 6.025 m. The top of the RCCV test model consists of a 0.5-m thick by 6.025-m diameter section with a 0.8-m diameter penetration at the centerline. A 0.4-m thick by 1.250-m high circular wall is built into the top of this section to support added weights. The weights are added to achieve a consistently scaled fundamental frequency. The basemat weighs 224 metric tons. The cylindrical section, including the intermediate floors and top section, weighs 76 metric tons, and the attached mass weighs 274 metric tons. A steel liner, 1.6-mm thick, is anchored to the inside of the RCCV cylinder with longitudinal T stiffeners embedded in the concrete. The overall geometry of the test model is 1:10 scale, while the concrete wall thickness is scaled at 1:8 and the liner thickness is scaled at 1:4 for constructability purposes. The design operating pressure for the RCCV is 45 psi (3.16 kg/cm^2).

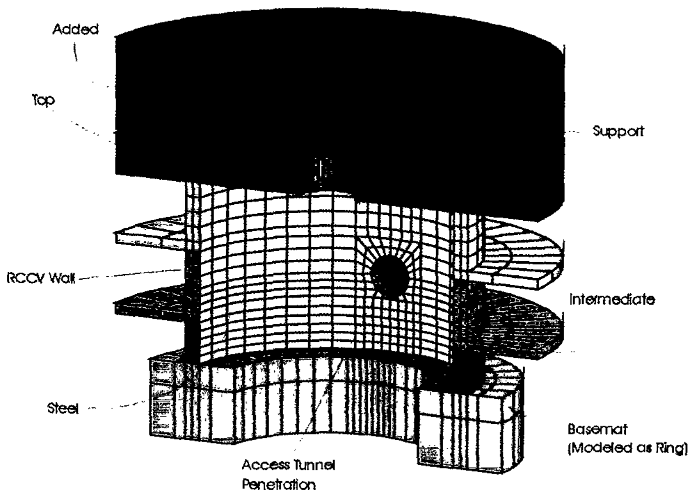

Figure 1. RCCV Finite Element Model

FINITE ELEMENT MODEL

Because the basemat is securely bolted to the shake table, only a ring section of the basemat is modeled, as shown in Fig. 1, and the prescribed target acceleration histories are imposed on the nodes along the cut section. Because of the large mass of the test specimen, the shake table was not able to prevent the model from rocking, which resulted in large vertical accelerations. This effect was observed to be stronger in the failure-level tests for the PCCV in which large vertical accelerations were measured in tests where horizontal accelerations only were applied (NUREG/CR-6639, 1999). The pre-test calculations applied the target motions to the basemat. However, the failure-level tests had a significant rocking component while the target motions were only horizontal. The differences between the assumed input accelerations at the basemat, and the actual response at these input points, cause the pretest calculations to vary from the measured response of the test vessel.

The geometry of the RCCV test model is assumed to be symmetric about a plane through the two buttresses, and the horizontal input acceleration history is applied in the direction parallel to this symmetry plane. Thus, a half-symmetry three-dimensional model is used, as illustrated in Fig. 1. Displacement constraints normal to the symmetry plane prevent any twisting or rocking of the model about this plane. However, it is anticipated that this effect should have little impact on the failure mode or failure-level earthquake.

The steel liner in the RCCV test specimen is modeled as fully bonded to the concrete through displacement compatibility at the common nodes. For solution economy, the bending stiffness of the liner and the stiffness of the T anchors embedded in the concrete are ignored. The model is not intended to capture local tearing at stiffener discontinuities or anchorage failure. However, the modeling of plastic straining in the liner is included so that damage in the liner and the potential for buckling due to deterioration of the concrete wall can be evaluated.

All reinforcing steel is explicitly modeled using truss-type subelements embedded in the concrete continuum elements at the appropriate locations. Fig. 2 illustrates the explicit modeling of the reinforcing bars in the model. The uniaxial stiffness and strength of these members are superimposed on that for the concrete elements through the shape functions of the continuum element. It is assumed that the reinforcing bars are fully bonded to the concrete so that the strains in the steel bars and the concrete are identical, and no dowel action is considered for the shear reinforcement. These assumptions could cause the analysis model to be too stiff late in the analyses after large loads begin to damage the rebar bonds in the test specimen.

All bolted connections are assumed not to slip. The true geometry of the added weights is modeled to account for the spatial distribution of the mass. The prestressing effect of the bolts on the concrete is modeled by including pretensioned truss-type subelements for the bolts.

The finite element grid shown in Fig. 1 was developed based on geometric requirements and past experience while considering the computer resources available. Twenty-node brick elements with quadratic displacement interpolation and 2 x 2 x 2 reduced gaussian quadrature integration are used to model the concrete and attached weights. The steel liner is

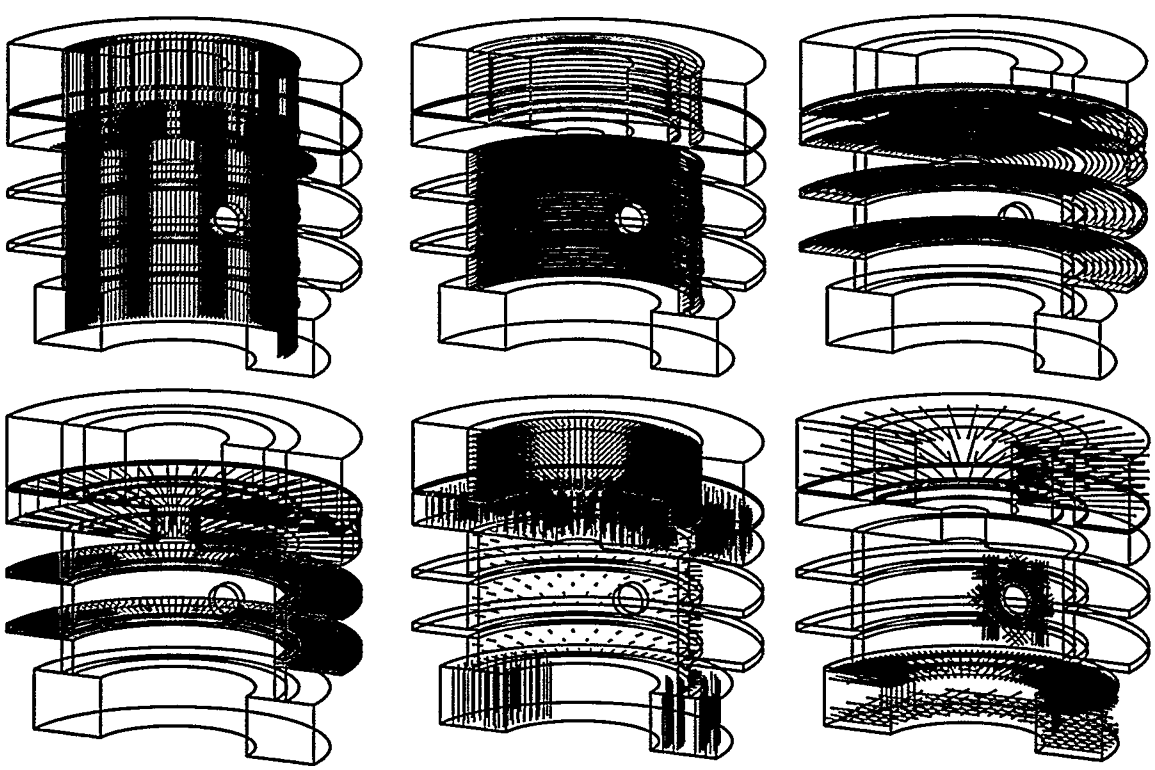

Figure 2. RCCV Finite Element Representation of Reinforcement

modeled with 8-node quadratic displacement interpolated plane stress elements using 2 x 2 reduced gaussian quadrature integration. The penetration liner and cover plates are similarly modeled. The model has 2140 elements, 9033 nodes, and 27,099 degrees of freedom.

MATERIAL MODELS

The ANATECH concrete material model (Rashid, 1996), using the smeared cracking methodology, is used to characterize the behavior of the reinforced concrete for the RCCV and basemat. A special submodel for viscous damping as a function of cracking is incorporated in the concrete constitutive model for consistency with the evolution of cracking damage in the structure. Hysteretic damping is already accounted for through cracking and compressive plasticity of the concrete material. The RCCV test specimen is constructed in stages, and compressive test data are available for each construction stage. The elastic modulus is calculated from the given compressive strength using the standard ACI formula, $E=57{,}000\sqrt{f'_c}$, where the units of compressive strength and modulus are in psi. The uniaxial fracture strain is based on Raphael's formula (1984) for tensile strength, namely, $1.7(f'_c)^{2/3}$ for the static strength and $2.6(f'_c)^{2/3}$ for the dynamic strength. The dynamic compressive strength is 15% higher than the static strength, Poisson's ratio is 0.19, and the weight density is 2400 kg/m^3 (150 lb/ft^3). The steel material models for reinforcing bars and liner plates are based on classical von Mises plasticity.

TIME-DEPENDENT LOADING

The model is first subjected to a series of low-amplitude sinusoidal motions to determine fundamental frequencies and the characteristics of the coupled test model and shake table. The response of the model to the design basis earthquake is obtained by first conducting separate tests using the individual horizontal and vertical components followed by tests using the combined horizontal and vertical components. The maximum design earthquake is referred to as the S1-level input, and the extreme design earthquake is called the S2-level input. The response of the model to a Loss of Coolant Accident (LOCA) in combination with the S1-level earthquake is obtained by dynamically testing the pressurized model. The margin of safety for the scaled test model is then determined by subjecting the test model to progressively larger amplitudes of the S2 motion (horizontal component only) until structural failure occurs. The test program also includes low amplitude tests to measure fundamental frequency changes as qualitative indicators of the damage sustained by the model after each test.

The testing sequence for the S1-level is as follows: design pressure, S1(H), S1(V), S1(H+V), and a leak test. A loss-of-coolant-accident pressure in conjunction with the S1-level event, S1(H+V)+LOCA, followed by a leak test are the next tests in the planned sequence. For S2 design-level testing, the planned test sequence is S2(H), S2(V), S2(H+V), followed by a leak test. Following these tests, a series of S2(H+V) tests are planned for public demonstrations. Finally, the RCCV test model is subjected to increasing levels of shaking designated as 2S2(H), 3S2(H), etc., until failure of the model occurs. The failure-level tests involve only the horizontal component with a leak test conducted after each test. However, time and budget constraints do not allow all planned tests to be analyzed, and a subset of the test sequence is chosen as follows: S1(H+V), S2(H+V), 2S2(H), followed by the failure-level analyses. The analysis input acceleration histories for the horizontal and vertical components of S1(H+V) are shown in Fig. 3. These values are the acceleration histories that were recorded at the top of the basemat.

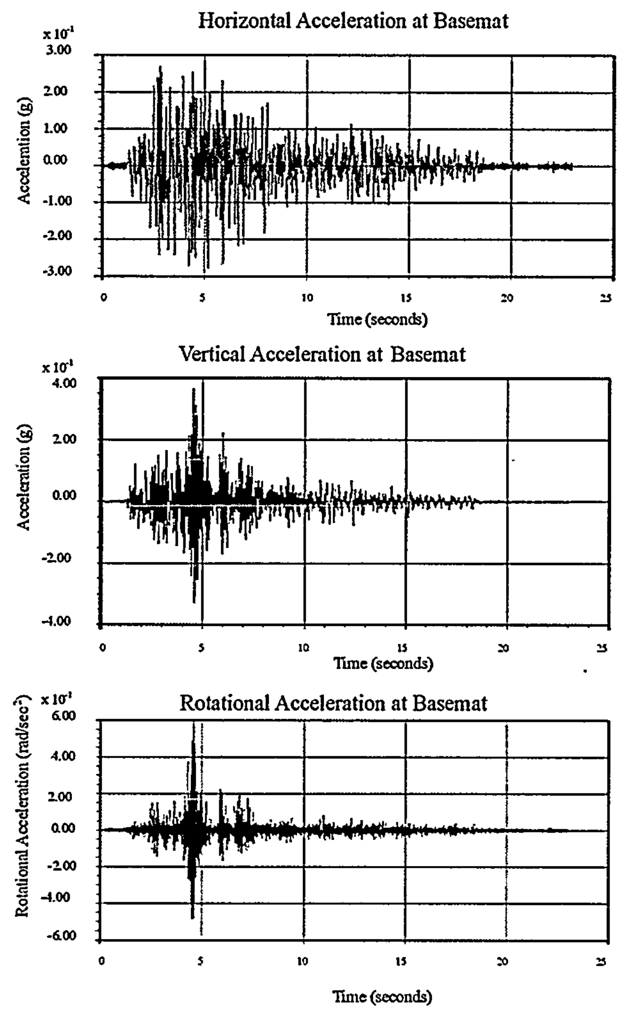

Figure 3. S1(H+V) Time History Input for Post-Test Analysis

PRE-TEST ANALYSIS RESULTS

Preliminary calculations are performed prior to the time history dynamic analyses to provide insight into the behavior of the RCCV model and identify important dynamic characteristics. These include developing the first few mode shapes and corresponding frequencies, generating the response spectra of the input accelerations, and performing static pushover analysis as a bound on the dynamic capacity of the model.

Mode Shapes, Frequencies, and Input Acceleration Spectra

Fig. 4 shows the first three fundamental mode shapes and frequencies. The first mode is dominated by the sliding shear-type deformation, although there does appear to be some rocking of the top mass which causes some bending deformations in the walls. The second mode is an axial extension mode inducing near uniform axial tension and compression in the RCCV wall and bending in the intermediate floors at the wall connections. The third mode is dominated by rocking of the top mass and bending in the upstream-downstream sections of the wall.

Fig. 5 shows, for illustration purposes, the acceleration spectrum for the horizontal component of the S1-level event. These data indicate that the first fundamental mode at 15.1 Hz coincides with the peak ground acceleration and, therefore, is

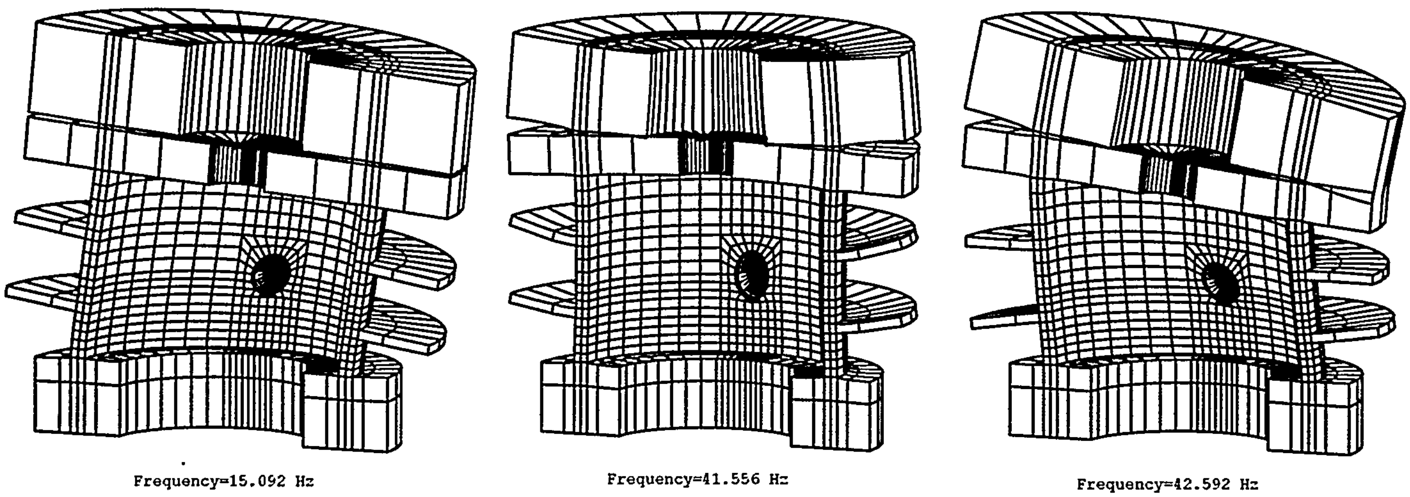

Figure 4. Lowest Three Modes and Frequencies

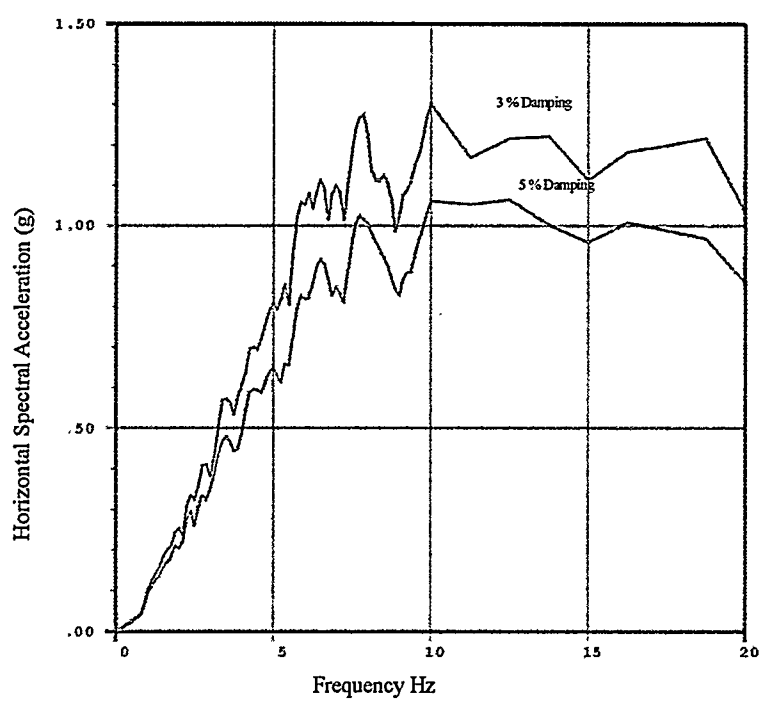

Figure 5. Response Spectrum for the S1(H) Motion

subject to large amplification. However, the higher modes are outside the frequency range of significant amplification. Thus, the dynamic response of the structure is primarily governed by the first shear-type mode. However, as damage accumulates in the structure, the fundamental modes will shift to lower frequencies.

Static Capacity Pushover

A static pushover analysis is conducted to verify the integrity of the finite element model and as a bounding estimation for the capacity of the test model. This analysis is conducted by fixing the basemat and incrementing the horizontal body force load of the top mass. This horizontal g force is incrementally increased until the computational model predicts failure. For this calculation, the horizontal force is applied monotonically in one direction. The top mass g load multiplier is plotted against the horizontal displacement of the top section. Fig. 6 shows the results of this static pushover calculation. An estimate of the dynamic capacity of the RCCV test model is obtained by applying a knockdown factor to the force-displacement response calculated by the static pushover. This curve is also plotted on Fig. 6 and is labeled as the dynamic capacity estimate. The knockdown factor was derived from the PCCV analysis (NUREG/CR-6639, 1999).

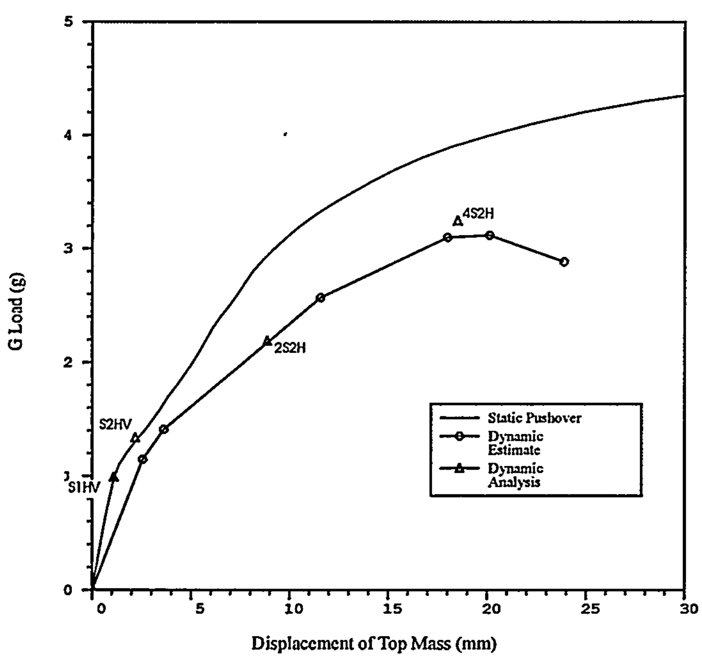

Figure 6. RCCV Static Dynamic Capacities

POST-TEST ANALYSIS AND COMPARISON WITH TEST DATA

As already noted, the dynamic analysis for the sequence of shake table tests is still in progress, and analytical results for the dynamic test simulating the S1(H+V)-level motion and S2(H+V) have been obtained so far. These results are discussed in this paper in relation to the measured data.

The RCCV model was first subjected to a static pressure test which, as expected, induced some cracking. The effect of the pressure-induced cracking is to reduce the experimentally determined fundamental frequency from 13.5 Hz to a value slightly above 12 Hz. It should be noted that the analytically determined initial frequency is 15.1 Hz, as shown in Fig. 4. Differences between the analytical and experimental frequencies may be due to the coupling between the structure and the shake table or the assumed modulus of elasticity of the concrete.

After the pressure test, the model was subjected to the S1(H) motion, followed by a frequency measurement. The measured response deviated from the target input, as discussed previously. The measured input is equivalent to 1.3 S1(H) accompanied by a strong vertical rocking component. The experimentally determined frequency dropped to 9.6 Hz, indicating significant additional cracking.

The test structure was further subjected to three more dynamic tests with target input motions simulating S1(H), S1(V) and S1(H+V). However, the measured base motions for these tests are, respectively, 1.1 S1(H), 1.1 S1(V), and 1.15 S1(H+V), in addition to rocking components. The frequency measurements showed continuous drop to slightly above 8 Hz after the S1(H+V) test.

The analysis plan calls for the S1(H) and S1(H+V) simulation tests as the first dynamic tests to be analyzed. However, the 1.3 S1(H) measured base motion was not available, and therefore the measured 1.15 S1(H+V) was the first dynamic test to be analyzed and compared to measurements. This meant that an analytical simulation of the cumulative damage that occurred in the preceding tests was necessary before attempting the 1.15 S1(H+V) test analysis. This analytical simulation was performed through a series of preconditioning analyses, with combined horizontal and vertical target input motions. The preconditioning analyses caused the fundamental frequency to decrease by amounts similar to those observed from the tests. The input motion used for the first preconditioning analysis consisted of a combined horizontal, vertical, and rotational motion obtained by multiplying the S1(H+V) test records by the factor 1.3. After the preconditioning analysis was applied, the RCCV model was analyzed using the S1(H+V) measured base motions, which consisted of horizontal, vertical and rotational (rocking) components, as shown in Fig. 3. The fundamental frequency, measured and calculated, at the end of the test is 8 Hz. The calculated response-time histories for the displacements and accelerations are shown respectively in Figs. 7 and 8, together with the measured response time histories. As can be observed

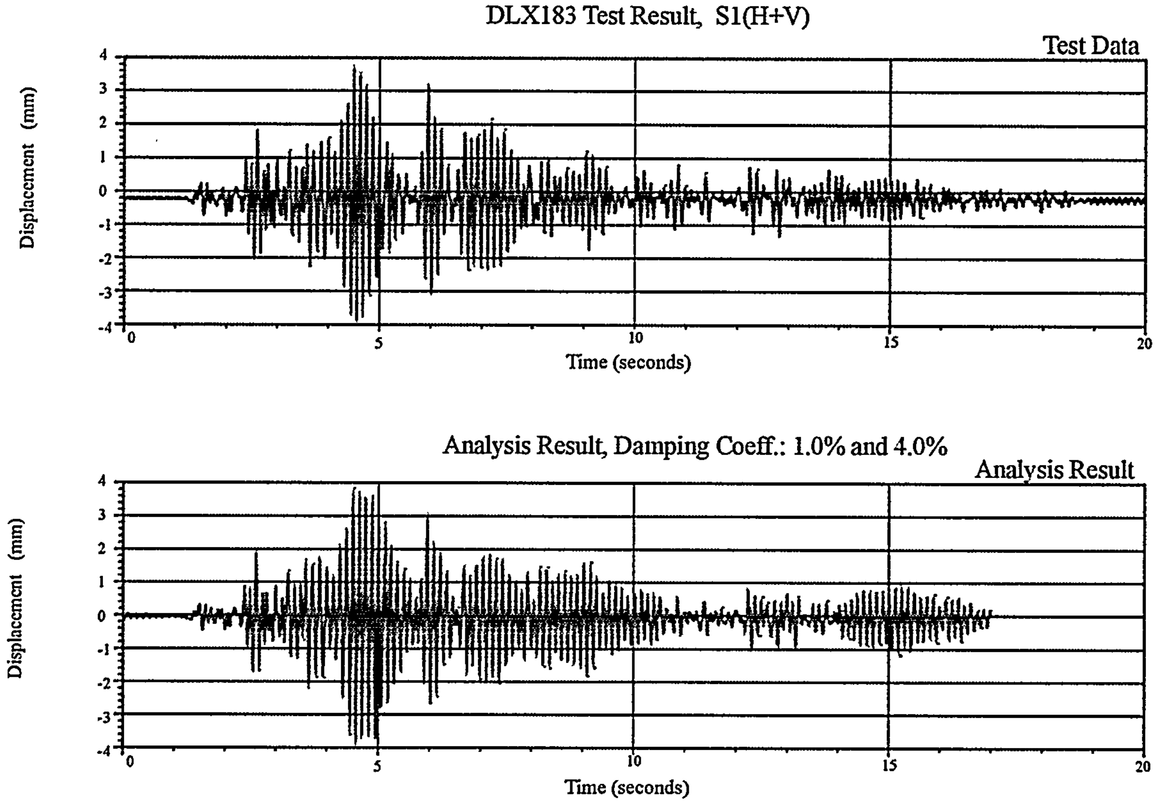

Figure 7. Comparison of Calculated and Measured Horizontal Displacement of Top Mass for S1(H+V) Test

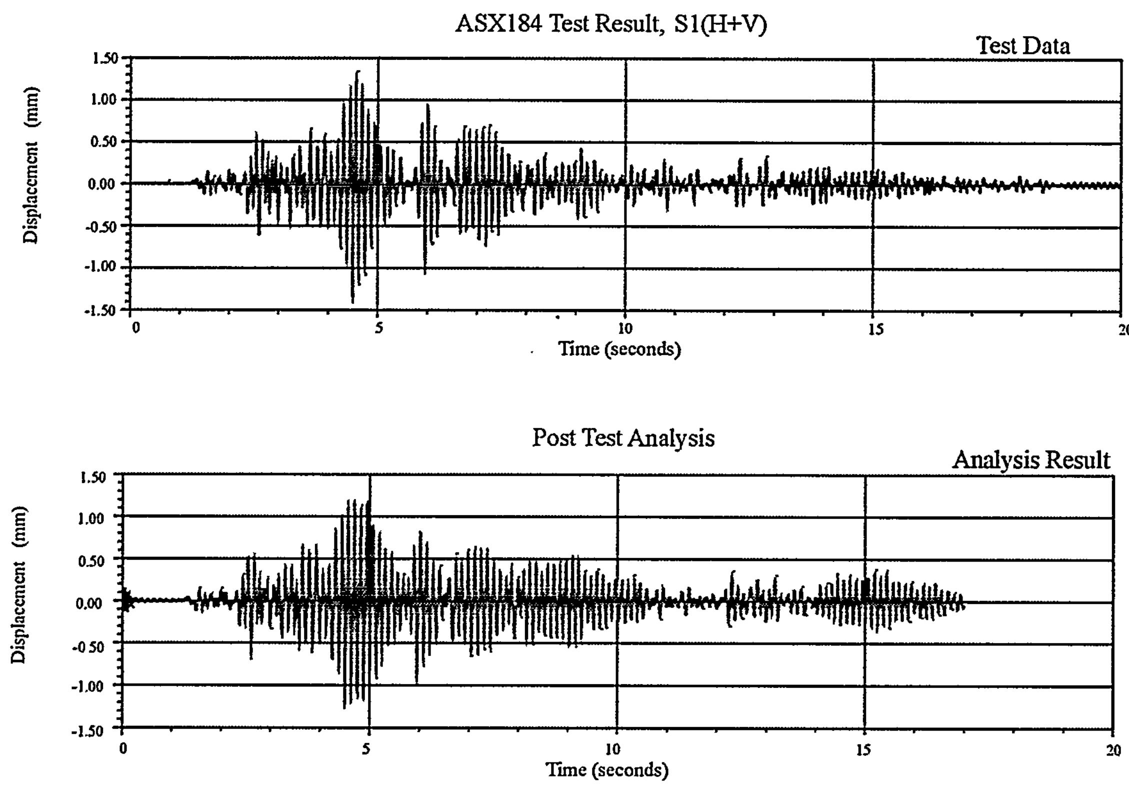

Figure 8. Comparison of Calculated and Measured Horizontal Acceleration of Top Mass for S1(H+V) Test

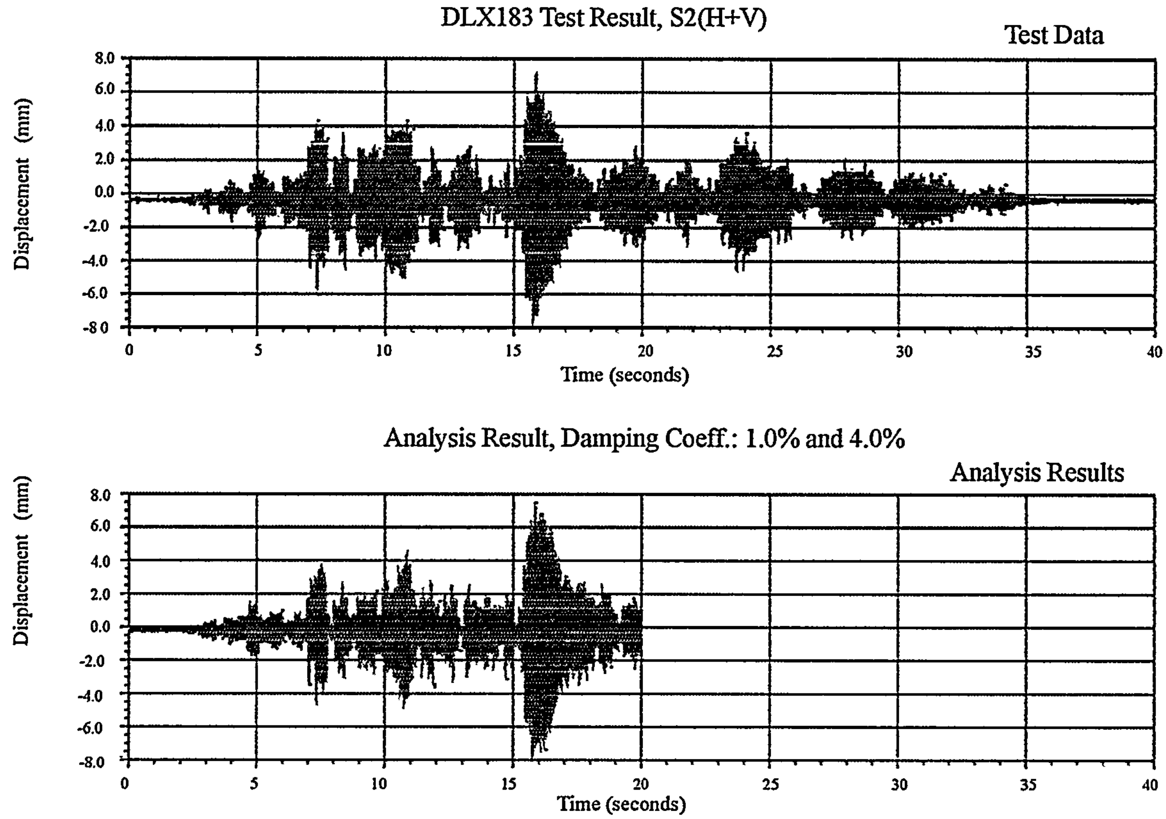

Figure 9. Comparison of Calculated and Measured Horizontal Displacement of Top Mass for S2(H+V) Test

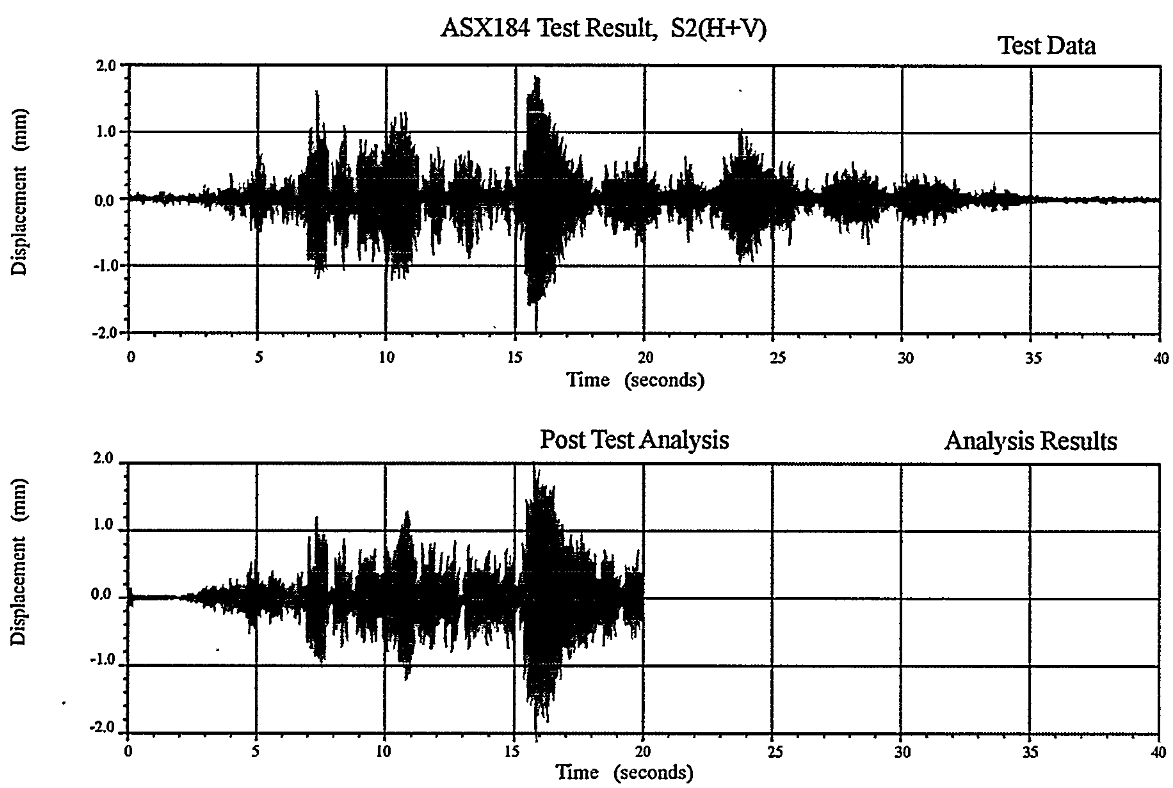

Figure 10. Comparison of Calculated and Measured Horizontal Acceleration of Top Mass for S2(H+V) Test

from these figures, the quality of the analytical results is quite good.

The analysis of the S2(H+V) simulation test followed the S1(H+V) analysis. Only part of the time history has been completed, and the results, compared with the measured response, are shown in Figs. 9 and 10. The frequency dropped to 6.8 Hz. Visual comparison of the measured and calculated time histories indicates reasonably close agreement.

CONCLUSIONS

The analysis results contained in this paper illustrate the progress made in coupling advanced analytical simulations with large-scale model testing to improve our understanding of the seismic capacities of RCCV structures. Similar analyses for PCCVs were previously reported (NUREG/CR-6639, 1999), which further demonstrated the validity of this computational methodology for the evaluation of the static and dynamic capacities of reinforced and prestressed containment structures. Safety concerns, which continue to increase the anticipated levels of seismic events, require validated analytical methods. Once the analytical methodology is validated for seismic loads using scaled models, capacities of actual RCCV structures can be predicted through numerical simulation.

ACKNOWLEDGMENTS

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REFERENCES

Nuclear Power Engineering Corporation of Japan, October 1997, "Summary Test Report for PCCV," (in Japanese), Report No. 34-2-2.

NUREG/CR-6639, SAND99-1464, ANA-98-0252, March 1999, "Seismic Analysis of a Prestressed Concrete Containment Vessel Model."

Raphael, J. M., March-April, 1984, "Tensile Strength of Concrete," *ACI Journal*, 81-17.

Rashid, Y. R. et al., September 1996, "Constitutive Modeling of Reinforced Concrete and Steel," ANATECH Corp., White Paper, ANA-96-0196.

Tsurumaki, S., March 1998, "NRC-NUPEC Collaboration Meetings on CCV Seismic Proving Tests; PCCV Test Results and Analysis," NUPEC Presentation.