13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2972

SEISMIC DAMAGE ASSESSMENT OF NUCLEAR POWER PLANT CONTAINMENT STRUCTURES

HoHyun Cho[1], Hyun-Moo Koh[2], Chang-Hun Hyun[3], Hyun Mock Shin[4]

SUMMARY

This paper presents a procedure for assessing seismic damage of steel and RC containment structures using the nonlinear time-history numerical analysis. Two kinds of damage index are introduced at finite element and structural levels. Nonlinear finite element analysis for the containment structure applies RC and steel shell elements using a layered approach leading to damage indices at finite element and structural levels, which are then used to assess the seismic damage of the containment structure. An example of such seismic damage assessment is seismic damages of the containment structure of KORI I nuclear power plant in Korea, which is a double structure combined RC shielding building and steel containment vessel, were evaluated against artificial earthquakes generated with a wide range of PGA according to US NRC regulatory guide 1.60. Structural responses and corresponding damage index according to the level of PGA are investigated. Through this assessment, It was shown that the steel containment vessel behaved elastically for earthquakes corresponding to seismic design criteria similar to PSC containment structure.

INTRODUCTION

Satisfactory and safe performance of containment structures under seismic occurrence is necessary in nuclear power plants to avoid completely radioactive leakage from nuclear reactor and power supply equipment. In connection to this, the Korea Institute of Nuclear Safety (KINS) is currently operating a Earthquake Monitoring Center, that keeps track of seismic ground motions at nuclear power plant sites, evaluates Korean earthquake characteristics and assesses seismic damage in nuclear structures. The center has conducted a multi-year research project along with Seoul National University to establish a seismic damage assessment system which estimates probable seismic damages of concrete containment structures by performing inelastic time-history analysis [1,2]. Last year, we used this system to assess seismic damage for PSC containment structure, where most of the containment structures in Korea is made of PSC [3]. In the assessment procedure, two kinds of damage index were introduced at finite element and

[1] Ph. D. Student, Seoul National University, Seoul, Korea, E-mail : chh@sel.snu.ac.kr
[2] Professor, Seoul National University, Seoul, Korea, Email : hmkoh@snu.ac.kr
[3] Ph. D, Korea Institute of Nuclear Safety, Daejeon, Korea, Email : k203hch@kins.re.kr
[4] Professor, Sungkyunkwan University, Suwon, Korea, Email : hmshin@yurim.skku.ac.kr

structural levels. At the time of the seismic damage assessment of RC and steel containment structure, nonlinear finite element analysis for the containment structure applies RC and steel shell elements using a layered approach leading to damage indices at finite element and structural levels, which were then used to assess the seismic damage of the containment structure. An example of such seismic damage assessment is the seismic damages of the containment structure of KORI I nuclear power plant in Korea, which is a double structure combined RC shielding building and steel containment vessel, were evaluated against artificial earthquakes generated with a wide range of PGA according to US NRC regulatory guide 1.60.

FINITE ELEMENT MODELING OF A CONTAINMENT STRUCTURE

Since seismic damages in structures are caused by inelastic deformations, the proposed seismic damage assessment makes use of nonlinear time history analysis to account for material nonlinearities and earthquake characteristics. An improved Newmark method which is called the HHT-α method, was chosen as the direct integration method because of its stability and accuracy.

Though most of the containment structures in Korea are made of PSC, we perform the seismic damage assessment for RC and steel containment structures by adopting a special nonlinear RC and steel shell finite element using layered approach and appropriate inelastic material models. Through this, the accuracy of the system for correctly describing the behavior of RC and containment structures will be validated by means of comparative study of the analytical and experimental results at element and structural levels. Introducing appropriate inelastic material models and including also a nonlinear reinforced steel model, the reinforced concrete shell element was developed using a four-node quadrilateral thin flat shell finite element with 6 DOFs per node proposed by Kim et al. (2001) [4].

Finite Element Formulation for RC and steel Shell Element

The developed shell element is a four-node quadrilateral thin flat shell finite element with 6-DOFs per node. The sixth DOF is obtained by combining a membrane element with a normal rotation(θ_z), the so-called drilling degree of freedom, and a discrete Kirchhoff plate element [5, 6, 7]. In order to analyse RC shells with nonlinear behavior, the layer method is used, assuming that several thin-plane stress-elements are layered in the direction of thickness. In the layered element formulation, the shell is divided into several paneled layers and two-dimensional constitutive models were applied to take into account material nonlinearities. The constitutive matrix can be rotated from the local axes to the global axes and added to the constitutive matrix for the concrete, or it can be used to define the properties as an overlaying element, adding stiffness to the connected nodes. This technique allows for the addition of any number of additional layers of steel with different orientations to be added to the finite element model. More detailed development is presented in [4].

Nonlinear Material Modeling of Reinforced Concrete

Models for concrete are divided into models for uncracked concrete and cracked concrete. For uncracked concrete, the elasto-plastic fracture model for the biaxial stress state proposed by Maekawa and Okamura (1983) is used [8]. As concrete shows high nonlinearity after cracking, the three following models are applied: the tension stiffness model for the tension stress of concrete in the direction normal to the crack, the compressive stiffness model for the degradation of compressive stiffness in the direction parallel to the crack and the shear transfer model in the shear direction at crack plane.

The basic model adopted to represent the crack is a non-orthogonal fixed-crack model for the smeared crack, which is widely known to be robust for crack representation. The initiation of a crack is assumed to start when the tensile stress reaches the fracture envelope. After the initiation of a crack in concrete, anisotropy becomes significant, and the stress-strain relationship is expressed in orthogonal anisotropy in the directions normal and parallel to the crack.

The behavior of the reinforcing bar in the concrete after yielding must consider simultaneously the characteristics of the steel and the bond effect between steel and concrete. To account for such characteristics of reinforced concrete, a tri-linear model is therefore adopted here.

DAMAGE INDEX

Damage index is introduced for the seismic damage assessment of the containment structure in order to numerically quantify the degree of damage. The concept of damage index can provide the means to quantify damage and relate it to costs and other consequences such as potential risk after earthquake. In this regard, damage index can play an important role in retrofit decision-making and disaster planning in earthquake region. Two kinds of damage index are introduced at finite element and structural levels. A simple lumped mass model is used to represent global damage level of the containment structure as presented in [2]. The damage index at finite element level is explained in the following section.

Damage Index at Structural Level

A simple lumped mass model in Fig. 1 is used to represent global damage level of the containment structure. It has been founded from recent researches that damage sustained in a structure is primarily dependent on the deformation level, with the number of cycles of loading having only a small effect on seismic damage [9, 10, 11]. Here the following damage index is adopted.

$$D.I._s = \frac{\delta_{\max} - \delta_y}{\delta_u - \delta_y} \qquad (1)$$

This index was proposed by Roufaiel and Meyer[12] and uses yielding displacement(δ_y), ultimate displacement at failure(δ_u) and the maximum displacement(δ_{max}) under seismic event. If the maximum displacement remains below the yielding displacement, the value of damage index is negative. That means the seismic behavior of containment structure remains within the elastic bounds and the structure did not suffer any seismic damage.

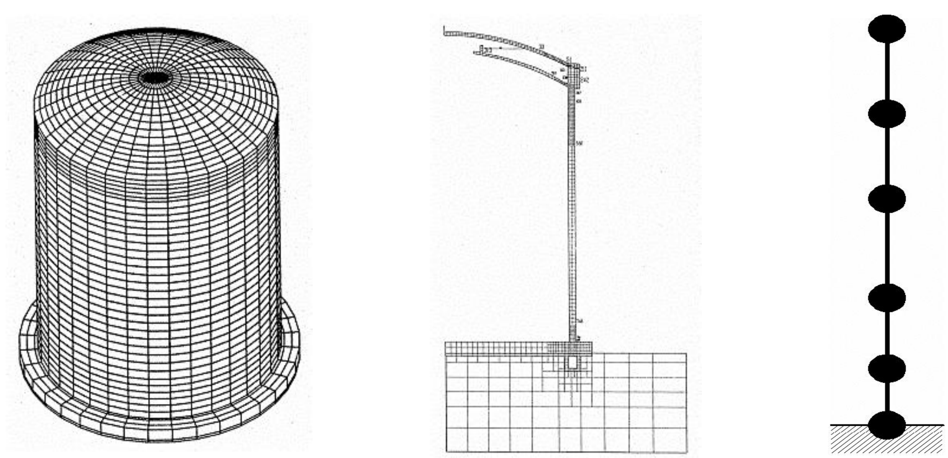

Fig. 1 Lumped mass model for damage index at structural level

Compressive Damage Index for Concrete at Finite Element Level

Compressive damage index is defined as 0.75 at "failure time" corresponding to a situation where the principle compression strain of concrete reaches the ultimate strain of concrete [13], and as 0.40 when the principle compressive strain of concrete reaches the strain at peak stress, corresponding to irreparable damage state (Table 1).

$$D.I._{compressive} = 1 - ftg_c\left(\frac{2\varepsilon_{cu} - \varepsilon_{cs}}{2\varepsilon_{cu}}\right)^2 \tag{2}$$

where, $ftg_c = 1.0 - 0.3AD_c$ and $AD_c = \sum \frac{1}{N_{2fc}}$ is the parameter of cumulative damage caused by cyclic loading. N_{2fc} represents the number of loading cycles until concrete reaches the fatigue failure, ε_{cu} is the ultimate strain of concrete and ε_{cs} is the principle compression strain of concrete at each analysis step.

Tensile Damage Index for Steel at Finite Element Level

Tensile damage index is defined as 0.75 at "failure time" corresponding to a situation where the tension strain of steel reaches the ultimate strain of steel, and as 0.40 once the strain of steel reaches the yielding plateau, corresponding to irreparable damage state. And, the damage index is defined as 0.10 when the strain reaches 75% of yielding strain of steel corresponding to the starting time of light flexural cracks (Table 1).

$$D.I._{tensile} = 1.20\left(\frac{\varepsilon_{ts}}{2\,ftg_r\varepsilon_{tu}}\right)^{0.67} \tag{3}$$

where, $ftg_r = 1.0 - 0.3AD_r$ and $AD_r = \sum \frac{1}{N_{2fr}}$ is the parameter of cumulative damage caused by cyclic loading based on the fatigue model of steel proposed Miner[14]. N_{2fr} represents the number of loading cycles until steel reaches the fatigue failure, ε_{tu} is the ultimate strain of steel and ε_{ts} is the principle tension strain of steel at each analysis step.

Table 1. Damage Index at Finite Element Level

Item		Ultimate strain (ε_{cu} or ε_{tu})	Damage Index ($D.I._{element}$)
Concrete	Compression · Shear	$0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{sm}}{f'_{cc}}$	$1 - ftg_c\left(\frac{2\varepsilon_{cu} - \varepsilon_{cs}}{2\varepsilon_{cu}}\right)^2$
Steel	Tension	0.10	$1.20\left(\frac{\varepsilon_{ts}}{2\,ftg_r\varepsilon_{tu}}\right)^{0.67}$
ρ_s = transverse confining steel ratio, f_{yh} = yield stress of the confining steel ε_{sm} = steel strain at maximum tensile stress, f'_{cc} = confined concrete compression strength			

EXAMPLE OF SEISMIC DAMAGE ASSESSMENT: KORI I NUCLEAR POWER PLANT

The structure selected for seismic damage assessment is the steel containment structure of KORI I nuclear power plant(Fig. 2a). The structure, built in 1977, is a double structure combined RC shield building and steel containment vessel. KORI I NPP has been designed as shown in Table 2. The typical cross section and material property of KORI I are given in Fig. 2b and Table 3. The equipment opening (with 3m diameter) is located on the north-east 52.5° from east direction.

(a) (b)

Fig. 2 (a) KORI I containment structure, (b) Typical vertical section of the structure

Table 2. Seismic Design Criteria for KORI I

Earthquake Case	Horizontal Acceleration	Vertical Acceleration
No loss of function	0.2g	0.14g
Design	0.1g	0.07g

Table 3. Material Property for KORI I

Concrete	Compressive strength(f_c')	212.67
	Poisson's ratio(ν)	0.16
	Specific weight(γ)	2.4 $tonf / m^3$
Steel	Yield strength(f_{py})	4,221 kgf / cm^2
	Elastic modulus(E)	2,100,000 kgf / cm^2

Input Ground Motion

Artificial ground accelerations have been generated with respect to the US NRC regulatory guide 1.60[15]. Each of the artificial earthquakes were generated for three directions (horizontal NS and EW, vertical) corresponding to US NRC regulatory guide 1.60 response spectra with PGA of 0.2g, and each of horizontal artificial earthquakes were then scaled to adjust PGAs of 0.1g, 0.2g, 0.4g, 0.6g, 0.8g, 1.0g, 1.2g, 1.4g and 1.6g. In addition, each of the vertical artificial earthquakes were scaled to adjust PGAs of 0.07g, 0.14g, 0.28g, 0.42g, 0.56g, 0.70g, 0.84g, 0.98g and 1.12g for the correspondence with seismic design criteria of Table 2.

Finite Element Modelling

All the structural components of the walls and domes have been modelled using the shell elements, as described above through various material models selected according to the density of reinforcement bars. Openings in structures being subject to largest stress concentration, which is the weakest point of the structure subjected to seismic event, can be foreseen to be the equipment opening. Following, the location and dimension of the equipment opening must be exactly introduced in the finite element model.

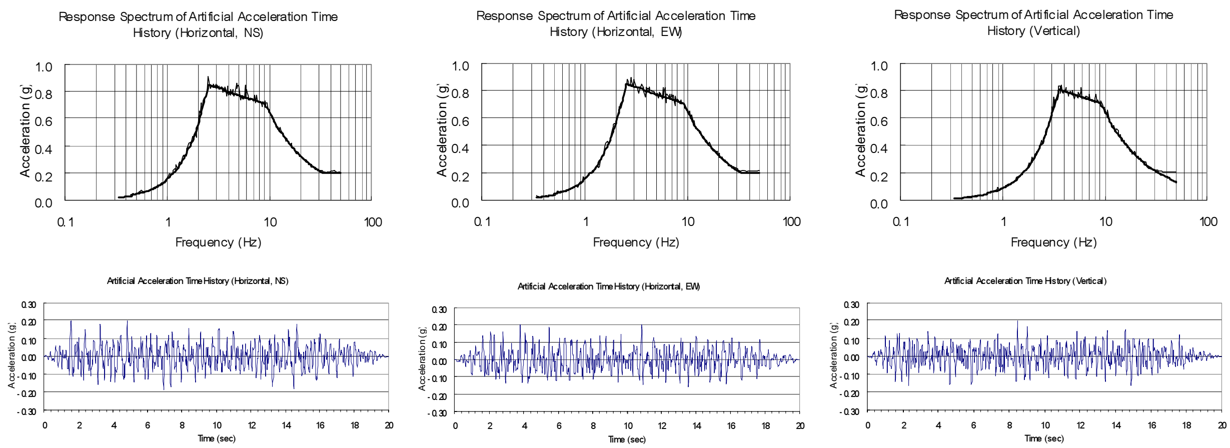

Fig. 3 Response spectra and acceleration time histories for PGA=0.2g

Determination of the Parameters for the Structural Damage Index

The parameters (δ_y, δ_u) for the computation of damage index at structural level [3] can be determined through the push-over analysis. Since the weakest point is the equipment opening located on the north-east of the structure, the push-over analysis is performed by applying a vertically distributed loading in the NE-SW direction (Fig. 3a).

In the analysis, the yield displacement (δ_y) is defined as the displacement which deviates from the elastic range. The elastic range is assumed to be the range of displacements lying in the domain located around 5% of the initial tangent stiffness. The ultimate displacement (δ_u) is assumed to correspond to the situation where any of the elements reaches the ultimate strain defined in Table 1. Analysis is interrupted once the ultimate displacement occurred. Push-over analysis results are given in Table 4.

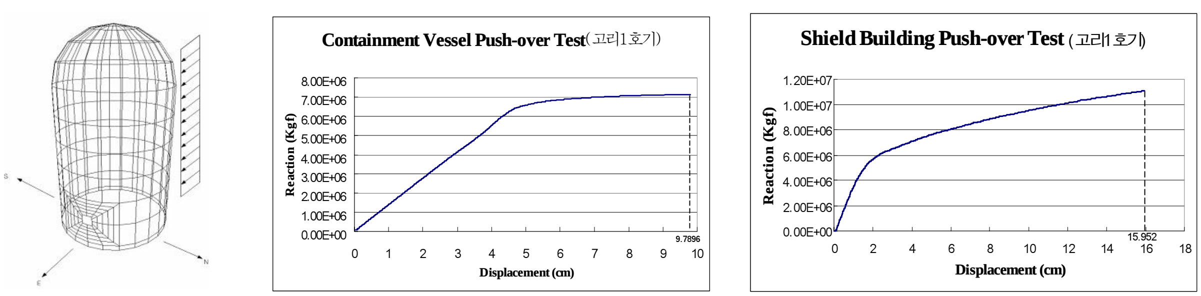

Fig. 4 Push-over Analysis

Table 4. Parameters for Structural Damage Index of KORI I

	δ_y (cm)	δ_u (cm)
Containment vessel	4.49	9.79
Shielding building	1.27	15.95

Seismic Damage Evaluation using Damage Index

Tables 5 shows the structural level damage index of steel containment vessel and RC shield building for the earthquake corresponding to no loss of function and design. This table suggests that the containment structure behaves elastically for earthquakes corresponding to seismic criteria that are earthquakes with PGA of 0.1g and 0.2g. Negative values of the structural damage index mean that the behavior of the structure remains with the elastic range.

Also, Fig. 5 presents the result of the seismic damage assessment for the nonlinear seismic analyses for each PGA ranging from 0.1g to 1.6g. From Fig.5, we can see that the structural damage index increase with the PGA. Especially, unlike the result of seismic damage assessment for PSC containment structure[3], the structural level damage index of steel containment vessel have negative value for all ranges of earthquake from 0.1g to 1.6g. This is considered to be because the total mass of steel containment vessel is less than that of concrete structures. And the structural level damage indices of RC shield building have similar value to PSC containment structure. On the other hands, there is a likelihood that failure would occur in RC shielding building at PGA of 1.2g level, most of PSC containment structures have failed at more than PGA of 1.4g. However, this does not matter as the structure merely has a function to shield the steel containment vessel from the environmental attack.

Table 5. Seismic Analyses Results for Seismic Design earthquakes

Earthquake Case	Steel Containment Vessel (D.I.s)	RC Shielding Building (D.I.s)
No loss of function	-0.78 (0.33cm)	0.01 (1.45cm)
Design	-0.81 (0.17cm)	-0.03 (0.76cm)

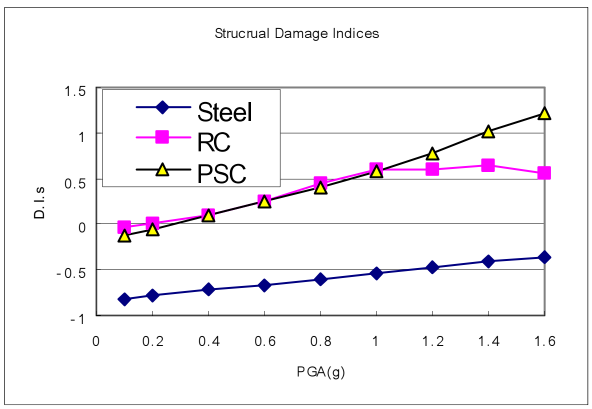

Fig. 5 Damage index at structural level according to PGA

CONCLUSIONS

In this paper, a seismic damage assessment procedure that allows the actual seismic resistance capacity and the damage level of containment structures to be assessed was presented. Damage indices at finite element and structural levels were computed to provide quantitative assessment of damages that structures may suffer under seismic event. The damage index at element level was used to detect the heaviest local damage and damage index at structural level was used to provide global damage state of the structure.

From such assessment of RC shielding building and steel containment vessel, it appeared that the structural damage index and the maximum damage index at finite element level increased with the PGA similar to the result of the seismic damage assessment of PSC containment structure. But compared to the result of the PSC containment structure, steel containment vessel behaves for all ranges of earthquake from 0.1g to 1.6g. Thus, it can be said that steel containment vessel is superior to the PSC containment structure in terms of seismic performance. RC shielding building is more vulnerable than the PSC containment structure because it has larger value of damage index and was failed to earthquake with the lower PGA.

As it is important for containment structures in nuclear power plants to show satisfactory safe performance under seismic occurrence so that they completely avoid radioactive leakage from nuclear reactor and power supply equipment, results obtained from such seismic damage assessment procedure are believed to be valuable and reliable data not only with respect to seismic risk assessment but also for predicting seismic damage in nuclear containment structures.

ACKNOWLEDGEMENT

The authors wish to thank the Korea Institute of Nuclear Safety (KINS) for their financial support in connection with research. The authors are also grateful to the Korea Earthquake Engineering Research Center (KEERC) at Seoul National University and the Korea Science and Engineering Foundation (KOSEF) for their grant.

REFERENCES

1 Seoul National University, Development of a Seismic Damage Assessment Program for Nuclear Power Plant Structures, Korea Institute of Nuclear Safety, 2002

2 Hyun, C. H., Koh, H. M., Cho, H. H., Lee, S. K., and Choi, K. R., Development of Seismic Damage Assessment System for Nuclear Power Plant Structures in Korea, Proc. 17th Structural Mechanics in Reactor Technology, No. K564, Prague, Chech Republic., 2003

3 Cho, H. H., Koh, H. M., Hyun, C. H., Shin, M. S. and Kim, M. S. Seismic Damage Assessment of Reinforced Concrete Containment Structures, Proc. 17th Structural Mechanics in Reactor Technology, No. K565, Prague, Chech Republic., 2003

4 Kim, T. H., Lee, K. M., and Shin, H. M., Nonlinear Analysis of Reinforced Concrete Shells Using Layered Elements with Drilling Degree of Freedom. In: ACI Structural Journal. V. 99, No. 4, 2002, pp.418-426.

5 Zienkiewicz, O.C. and Taylor, R.L., The Finite Element Method, Vol. 2-Solid and Fluid Mechanics, Dynamics and Non-linearity. McGraw Hill Book Co., 4th ed., 2001

6 Lanheng Jin, Analysis and Evaluation of a Shell Finite Element with Drilling Degree of Freedom. Masters thesis, University of Maryland at College Park, 1994

7 Hilton, E. and Owen, D.R.J., Finite Element Software for Plates and Shells. Pineridge Press Ltd., Swansea, UK., 1984

8 Maekawa, K., Irawan, P., and Okamura, H., Three-Dimensional Constitutive Laws of Reinforced Concrete. In: International Conference on Applied Concrete Mechanics APCOM, Seoul, 1996

9 Mwafy, A. M.,¡¡Seismic Performance of Code-designed RC Buildings. Ph.D. thesis, Imperial College, London, (2001)

10 Williams, M. S., Villemure, I. and Sexsmith, R. G., Evaluation of Seismic Damage Indices for Concrete Elements Loaded in Combined Shear and Flexure. In: ACI Structural Journal. 94-3 (1997), pp.315-322.

11 Ellingwood, B. R., Earthquake Risk Assessment of Building Structure. In: Reliability Engineering and System Safety, V.74 (2001), pp. 251-262.

12 Roufaiel M.S.L. and Meyer C., Analytical modelling of hysteretic behavior of R/C frames. In: Journal of Structural Engineering, ASCE, V.113, No.3 (1987), pp.429-444.

13 Priestley, M.J.N., Seible, F. and Calvi, G.M., Seismic Design and Retrofit of Bridges. John Wiley & Sons, Inc., 1996

14 Miner, M.A., Cumulative Damage in Fatigue. In: Journal of Applied Mechanics. V.67, No.3, 1945, pp.A159-A164.

15 United State Nuclear Regulatory Commition, Design Response Spectra for Seismic Design of Nuclear Power Plants, *US NRC Reg. Guide, 1.60*, 1973