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Evaluation of Shear Wall Capacity Using Nonlinear Finite Element Analysis

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1. Introduction

As can be seen from the Fukushima nuclear power plant accident, a containment building is the final protecting shield to prevent radiation leakage. Thus, the assessment of seismic performance of a containment building against seismic loads is very important. Since a containment building is a complex structure, its seismic performance is analyzed through finite element analysis. Because of the high nonlinearity of concrete, an appropriate material model should be established when used in finite element analysis.

In this study, a nonlinear concrete material model is calibrated with the numerical simulation of the CASH benchmark test performed in the OECD/NEA [1]. CASH (capacity of reinforced concrete shear walls) is an international benchmarking program organized under an initiative of the OEDC/NEA (Nuclear Energy Agency). The series of reinforced concrete shear wall panels were tested at the European Laboratory for Structural Assessment (ELSA, Joint Research Centre) between 1997 and 1998[2].

2. Finite Element Model of the Shear Wall

In this section the finite element model is described.

2.1 Benchmark Model

The core of benchmark model is a shear wall of length L=2.68 m and height H=1.2 m. The numerical model consists of 2008 solid elements (C3D8) for concrete and 7750 truss elements (T3D2) for reinforcement. Figure 1 shows the whole finite element modeling of the benchmark model. The material properties of the benchmark model are summarized in Table 1.

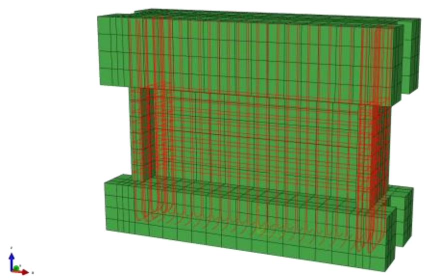


Fig. 1. The whole finite element modeling of the benchmark model

Table I: The material properties of the benchmark model

		σ_e (MPa)	σ_u (MPa)	ε_u
Concrete	Compressive	-	39.9	0.0022
	Tensile	-	3.9	0.000112
Reinforcement	@6	568.5	663.7	0.21
	@8	594.4	672.0	0.23
	@10	572.8	651.0	0.25
	@14	592.1	518.7	0.24

2.2 Concrete Damaged Plasticity Model

The concrete damaged plasticity model [3, 4] was used for concrete material behavior. The concrete damaged plasticity model is selected in the present study because the model has potential to represent complete inelastic behavior in both tension and compression. In addition, the material model was appropriately modified to ensure the numerical stability.

2.2.1. Tension stiffening model

In order to simulate the complete tensile behavior of reinforced concrete in ABAQUS, a post failure stress-strain relationship (Fig. 2 (a)) is used, which provides a general capability for modeling tension stiffening, strain-softening, and reinforcement interaction with concrete. To define this model, user should input young's modulus (E_0), stress (σ_t), cracking strain ($\tilde{\varepsilon}_t^{ck}$) values, and the damage parameter values (d_t) for the relevant grade of concrete. The cracking strain should be calculated from the total strain using the equation (1) below:

$$\tilde{\varepsilon}_t^{ck} = \varepsilon_r - \varepsilon_{ot}^{el} \qquad (1)$$

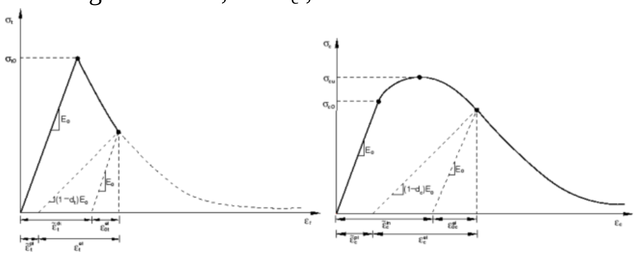
Where, $\varepsilon_{ot}^{el} = \sigma_t / E_0$, elastic strain corresponding to undamaged material; and ε_t, total tensile strain.

2.2.1. Compressive Stress-Strain Model

To define the compressive stress-strain relation of concrete (Fig. 2 (b)), user needs to enter the stresses (σ_c), inelastic strains ($\tilde{\varepsilon}_c^{in}$) corresponding to stress values, and damage properties (d_c) in relation with inelastic strains. Therefore, total strain values should be converted to the inelastic strains using the equation (2):

$$\tilde{\varepsilon}_c^{in} = \varepsilon_c - \varepsilon_{oc}^{el} \qquad (2)$$

Where, $\varepsilon_{oc}^{in} = \sigma_c / E_0$; ε^{el}_{oc}, elastic strain corresponding to undamaged material; and ε_c, total tensile strain.



(a) Tension stiffening model (b) Compressive stress-strain model
Fig. 2. Concrete damaged plasticity model

2.3 Reinforcement Model

The rebar reinforcement was modeled using the classical meal plasticity model where piecewise stress-strain data were used for hardening (Fig. 4).

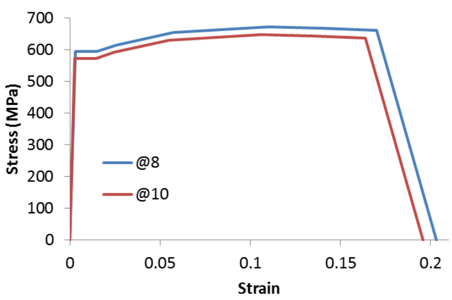


Fig. 4. stress-strain model of reinforcements

2.4 Boundary Condition & Load

The boundary conditions were constructed considering the experimental conditions. The bottom surface of the base guard was modeled as fixed. In order to control end lifting, the displacement of the upper guard in the Y and Z directions was constrained. The boundary conditions were modeled as closely as possible to the experiment. A pushover analysis was performed in the positive X- direction.

3. Result of Numerical Simulation

As shown in Fig. 4, the results of the numerical simulation and benchmark test are compared.

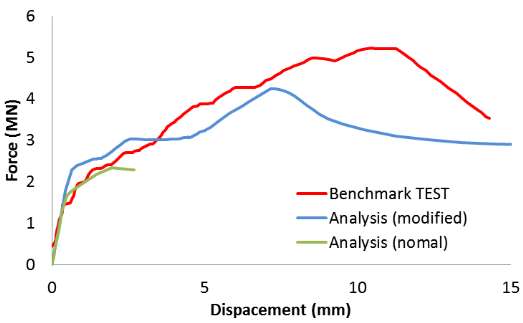


Fig. 4. Comparison between numerical simulation and Benchmark test

The analytical results show an overall similar response to the experimental results. The maximum load was 17% smaller than the experimental results (analysis: 4.2MN, test: 5.2MN). The error of the analytical result is judged to be because the boundary condition of the analytical model is not perfectly matched with the experiment. The analytical stability was secured through the modified material model. The numerical stability of the concrete damaged plasticity model was also confirmed.

4. Conclusions

In this study, the concrete material model for nonlinear finite element analysis is investigated. The concrete damaged plasticity model was selected to consider the nonlinearity of concrete behavior. Then, the calibrated concrete model can be used on the future of seismic simulation of a containment building.

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REFERENCES

[1] N001_A469_2014_EDF_A, PRESENTATION OF THE CASH BENCHMARK PHASE 1, OECD/NEA, 2014
[2] Pierre Labbé, Pierre Pégon, Javier Molina, Christian Gallois, Danièle Chauvel, THE SAFE EXPERIMENTAL RESEARCH ON THE FREQUENCY DEPENDENCE OF SHEAR WALL SEISMIC DESIGN MARGINS, 11th Canadian Conference on Earthquake Engineering, At Victoria, 2015.
[3] Lubliner, J., Oliver, J., Oller, S. and Onate, E., A plastic-damage model for concrete, International Journal of Solid and Structures, 25:299-329, 1989.
[4] Lee, J. and Fenves, G.L., Plastic damage model for cyclic loading of concrete structures, Journal of Engineering Mechanics, 124(8):892-900, 1998.