Method to simulate ultimate dynamic response of a reinforced concrete shear wall under seismic loading

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ABSTRACT
A spring-mass model was studied to simulate ultimate seismic behavior of reinforced concrete shear walls. The conventional hysteretic rules used in Japan were applied to the model. The skeleton curve was determined from the force-displacement relationship calculated by the two dimensional finite element static analysis using the concrete cracking and crushing material model. The method presented here was able to trace the vibration test results with good accuracy in the ultimate range as well as in the highly plastic range.

1. INTRODUCTION

In the seismic design of nuclear reactor buildings, the evaluation of their response characteristics and ultimate strength is very important when assessing the seismic reliability and safety factor of those buildings. Reinforced concrete (RC) shear walls of reactor buildings play a major role as earthquake-resisting elements. The Nuclear Power Engineering Corporation (NUPEC) performed a vibration test of RC shear walls to study their ultimate dynamic response characteristics and proposed the utilization of the test as an International Standard Problem (ISP) [1] to study safety analysis codes and their analytical/modeling methods for earthquake response.

We studied the ISP from the viewpoint of whether the conventional Japanese design analysis models are applicable to ultimate dynamic response of RC shear walls. We employed the spring-mass dynamic analysis which includes two force-displacement hysteretic rules recommended by the Japan Electricity Association [2] for designing nuclear reactor buildings in Japan. The skeleton curve of the force-displacement relationship used in the dynamic analysis was determined from the force-displacement relationship calculated in the finite element static analysis. The computer codes employed were our original RESP-E for the dynamic analysis and ADINA [3] for the static analysis.

In the following sections, first, the vibration test is summarized (section 2); then the static analysis is presented (section 3); finally the dynamic analysis is discussed (section 4).
2. VIBRATION TEST [1]

Fig. 1 shows a RC test specimen which has a web shear wall connected with two flange walls. The test specimen was subjected to artificial input acceleration, as shown in Fig. 2, five times with different input levels: elastic level (called RUN-1), shear crack initiation level (RUN-2), moderately plastic level (RUN-3), highly plastic level (RUN-4, maximum input acceleration = 5770mm/s²), and ultimate failure level (RUN-5, maximum input acceleration = 12300mm/s²). The test results are presented with the calculated ones in section 4. It should be noted that, in RUN-5, horizontal shear sliding failure occurred around the bottom of the web wall and the web concrete fell out. In this paper, we discuss only RUN-4 and RUN-5.

3. FINITE ELEMENT STATIC ANALYSIS

3.1 Finite element model

To determine the skeleton curves for the dynamic analysis, the finite element static analysis was performed.

We employed the two-dimensional model shown in Figs. 3(a), (b). The web wall was modeled using four-node isoparametric plane stress elements 75 mm thick; for the top slab, plane strain elements 4000 mm thick; for the flanges, plane strain elements 1000 mm thick (the same as the total anchor length of the web horizontal rebars) for RUN-4 and 3000 mm thick (the same as the full flange length) for RUN-5. The reason why we employed different element thickness for the flanges for RUN-4 and RUN-5 is that, in RUN-5, in which the web wall failure occurred, the flanges must support much of the shear force instead of the failed web wall; this resulted in larger effective flange length (i.e., element thickness) in RUN-5 than in RUN-4.

The rebars were modeled using two-node truss elements, as shown in Fig. 3(b); the section area of a truss element was the total rebar section area within the element’s governing volume; every truss element shared common nodes with the adjacent plane elements at the same coordinates; this means that the bond-slip and dowel-action effects were not considered.
The bottom nodes of the whole finite element model were completely fixed. The horizontal centerline of the top slab was subjected to monotonic horizontal prescribed displacement. The nonlinear static equation was solved using the simple incremental method with a constant displacement increment of 0.01 mm per solution step.

3.2 Material model [3]

Fig. 4(a) shows the uniaxial compressive stress-strain relation of the concrete model which is scaled up or down depending on triaxial stress conditions, using the compressive failure envelope shown in Fig. 4(b). The tensile stress-strain relation for the concrete model includes the linear tension stiffening effect, as shown in Fig. 5(a), which is employed after tensile failure (cracking) when triaxial stress conditions reach the tensile failure envelope shown in Fig 5(b).

The crack model includes smeared cracking, orthotropic stress-strain relation, reduced normal/shear stiffness of the cracked surface, fixed initial crack direction, and crack closing depending on strain normal to the cracked surface.

For the stress-strain relation of the rebar, the bilinear elastic-plastic model under the von Mises yield condition is assumed. The yield stress is 39.1 kgf/mm².
3.3 Static analysis results for RUN-4

The calculated force-displacement relationship for RUN-4 is shown as the solid line in Fig. 6 with the experimental hysteresis loops which were drawn such that all the positive/negative hysteresis loops of RUN-1 through RUN-4 are overlapped in the same frame. The calculated relationship is approximately bilinear. The elastic limit of force and displacement is in good agreement with the experimental one, and the plastic part of the relationship is mainly caused by shear cracking, the development of which is shown in Figs. 9(a)-(c).

Fig. 7 shows the magnified deformation corresponding to the top slab displacement of 4 mm; at this deformation level, the crack distribution spreads all over the web wall, as shown in Fig. 9(c); this crack distribution is similar to the experimental one shown in Fig. 8.

Using the calculated force-displacement relationship, we determined the trilinear skeleton curve for the dynamic analysis for RUN-4.

![Graph showing force-displacement relationship](image)

**Fig. 6 Comparison of experimental and calculated force-displacement relationship in RUN-4**

![Magnified deformation in static analysis, RUN-4 (Displacement of 4 mm)](image)

**Fig. 7 Magnified deformation in static analysis, RUN-4 (Displacement of 4 mm)**

![Visual observation of cracks in experiment (Final state of RUN-4)](image)

**Fig. 8 Visual observation of cracks in experiment (Final state of RUN-4)**

![Calculated crack/crush distribution of web in static analysis, RUN-4](image)

(a) Displacement of 1 mm  
(b) Displacement of 2 mm  
(c) Displacement of 4 mm

**Fig. 9 Calculated crack/crush distribution of web in static analysis, RUN-4**

(slashes: single crack,  box: two orthogonal cracks)

3.4 Static analysis results for RUN-5

Figs. 10, 11, 12, and 13 show the force-displacement relationship, the calculated magnified deformation corresponding to the top slab displacement of 30 mm, the experimental final crack distribution, and the calculated crack development, respectively. Fig. 10 was drawn as described in section 3.3, using all the hysteresis loops of RUN-1 through RUN-5 (note that no data were recorded over 20 mm displacement in the experiment).

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In these figures, some interesting behaviors are observed between the calculated and experimental results. Stiffness softening of the force-displacement relationship was caused by shear sliding failure of the web wall arising from concrete crushing. The shear failure occurred around the bottom of the web wall apart from the fixed base, as “→” marks the failure band in Figs.11, 12, and 13(c). The calculated force-displacement relationship approximately traces the profile of the experimental one, as shown in Fig.10, except for a "transitional" loop which appears between the prefailure and postfailure loops.

Using the calculated force-displacement relationship, we determined the multilinear skeleton curve for the dynamic analysis for RUN-5. The curve included the negative stiffness between 8 and 30 mm in displacement and was assumed to keep the constant force of 40 tonf over 30 mm in displacement.

![Fig.10 Comparison of experimental and calculated force-displacement relationship in RUN-5](image)

![Fig.11 Magnified deformation in static analysis, RUN-5 (Displacement of 30mm)](image)

![Fig.12 Visual observation of cracks in experiment (Final state of RUN-5)](image)

![Fig.13 Calculated crack/crush distribution of web in static analysis, RUN-5](image) (\: single crack, \: two orthogonal cracks, \: crush)

4. SPRING-MASS DYNAMIC ANALYSIS

4.1 Spring-mass model
For the spring-mass dynamic analysis, we employed the one-degree-of-freedom model, as shown in Fig.14, which consists of a mass and a shear spring. The top slab weight, the added weight to the top slab, and the upper half weight of the web and flanges were took into account for the mass. The base of the model was excited by the acceleration history obtained in the test at the upper surface of the base slab.
The skeleton curves of the force-displacement relationships for the shear spring were determined as described in sections 3.3 and 3.4 by the finite element static analyses.

To the shear spring, we applied two force-displacement hysteretic rules shown in Fig.15: the maximum-displacement-oriented rule (Figs.15(a)-(c)) and the origin-oriented rule (Fig.15(d)). The former was used before web wall failure and the latter was used after web wall failure. Therefore, force-displacement paths are as follows. When a path is on or reaches the skeleton curve, it traces the curve (Figs.15(a),(b),(d)). When the path goes back below the curve, we switch the hysteretic rules as: (i) before web failure, the path is oriented toward the opposite absolute-maximum displacement point ever attained (Figs.15(b),(c)) and (ii) after web failure, it is oriented toward the origin of the force-displacement relationship (Fig.15(d)). For switching the hysteretic rules, we used the criterion that they were switched at the moment when the web wall deformation angle reached the assumed web failure angle. In RUN-5, we defined the failure angle as 17.5mm/2400mm based on the test data (Fig.19).

Dynamic viscous damping was assumed to be proportional to the initial stiffness of each RUN. We switched the equivalent damping ratio at the same time as the hysteretic rules described above: 4% before web failure and 2% after it. The former was obtained from the test and the latter was empirically defined. For the time integration of equation of motion, the central difference method with a constant time step of 0.0001 second was employed.

4.2 Dynamic analysis results for RUN-4

Figs.16 and 17 show the inertia force-displacement relationships obtained from the experiment and the calculation, respectively. The max./min. inertia force and displacement of the calculated hysteresis loops are in good agreement with those of the experiment.
The calculated time histories of the displacement and acceleration are shown in Figs. 18(a) and (b), respectively, compared with the experimental ones. Both amplitude and time period of the calculated waves are in good agreement with those of the experiment.

![Fig.16 Experimental inertia force-displacement relationship in RUN-4](image1)

![Fig.17 Calculated inertia force-displacement relationship in RUN-4](image2)

(a) Displacement (mm) vs. time (s)

(b) Acceleration (mm/s²) vs. time (s)

![Fig.18 Comparison of displacement and acceleration time histories in RUN-4](image3)

4.3 Dynamic analysis results for RUN-5

Figs. 19 and 20 show the force-displacement relationships obtained from the experiment and the calculation, respectively. In both relationships, three kinds of hysteresis loops are similarly observed: the prefailure ones with high stiffness and forces, the postfailure (ultimate) ones with low stiffness and forces, and a transitional one which connects the pre/post failure ones.

![Fig.19 Experimental inertia force-displacement relationship in RUN-5](image4)

![Fig.20 Calculated inertia force-displacement relationship in RUN-5](image5)
The calculated time histories of the displacement and acceleration are shown in Figs.21(a) and (b), respectively, compared with the experimental ones. The characteristics of the calculated waveforms, except for the transitional response between 3.7 and 4.1 seconds, closely resemble those obtained in the experiment even in the ultimate range from 4.1 seconds onward.

Fig.21 Comparison of displacement and acceleration time histories in RUN-5

5. CONCLUSIONS

We presented a method of simulating the ultimate seismic response of RC shear walls. The calculated results, such as acceleration, displacement, and the force-displacement relationship, were in good agreement with the vibration test results in the ultimate range as well as in the highly plastic range. Therefore, it was found that the spring-mass model, which employed switching the maximum-displacement-oriented rule and the origin-oriented rule, was applicable to simulating the ultimate seismic response of the RC shear wall.

However, some of the modeling key parameters, such as the effective flange length, the switching criteria for the force-displacement hysteretic rules and damping ratio, must be further researched for establishing more general modeling methods.

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