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Nonlinear analysis on RC shear wall shaking table test

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ABSTRACT

The seismic shear wall which is proposed for the International Standard Problem (ISP) is analyzed using an FEM code. The dynamic behavior of reinforced concrete shear wall up to failure was simulated using a model consisting of shell and solid elements. The reliability of the code is confirmed by comparing simulated and test results.

1 INTRODUCTION

In order to evaluate the seismic safety during large earthquakes, estimation of the ultimate strength of structures is necesary. By comparing the intensity of earthquake corresponding to ultimate strength and design seismic load, the safety margin can be calculated. However, estimating ultimate strength require many large-scale tests. 3-D FEM analysis is an alternative for testing and is promising for seismic design.

The authors developed the 3-D non-linear analysis code, NAPISOS, which can analyze the structural response against earthquakes in cooperation with the Central Research Institute of Electric Power Industry. This code can handle the base isolation system of reactor buildings which might be realized in future.

Many organizations participated the Seismic Shear Wall ISP, which was proposed by Nuclear Power Engineering Corporation (NUPEC) and organized by OECD Nuclear Energy Agency. This analysis is one of twelve FEM dynamic analyses submitted to the ISP. In this paper, we focused on the description of a model which consists of shell and solid elements, and constitutive laws of cracked concrete.

2 MODELING AND CONSTITUTIVE LAW OF MATERIALS

In order to analyze seismic response of the reactor buildings in nuclear power plants, the modeling of reinforced concrete shear walls is important. Layered shell elements are applied to simulate the walls' non-linear behavior up to failure.

2.1 Test specimen and analytical model

The shape of specimen for the " Seismic Ultimate Dynamic Response Test"[1] is shown in Fig. 1. NUPEC performed vibration test on a shaking table at Tadotsu in Japan. The final analytical 3-D model is composed of layered shell elements and solid elements as shown in Fig. 2. Taking advantage of symmetrical shape, a half of the specimen is analyzed. The flange walls have five concrete layers and two reinforcement layers. However, the web wall has a single layer each of concrete and reinforcement. The base slab, top slab and pendulum are modeled with solid elements. Where the walls, and the top and base slabs join, the shell elements are mounted inside the solid elements and make transmission of bending moment from the wall to the solid element more accurately possible.

2.2 Layered shell element

Fig. 3 shows the shell element, which is composed of concrete plates and reinforcement planes. Concrete elements are divided into several layers of the same thickness, and the non-linear behavior varies in each layer. The non-linear properties of material are considered only in plane direction.

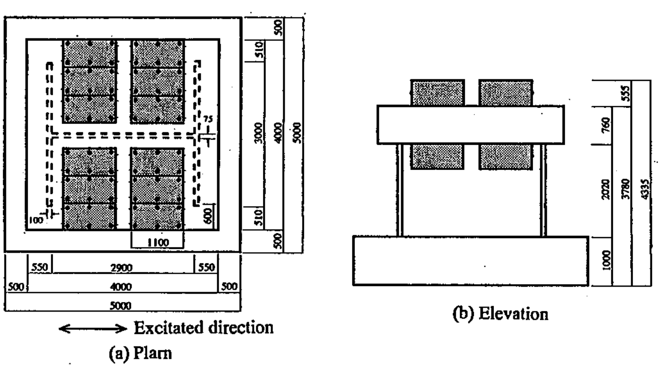


Fig. 1 A test specimen of a RC shear wall

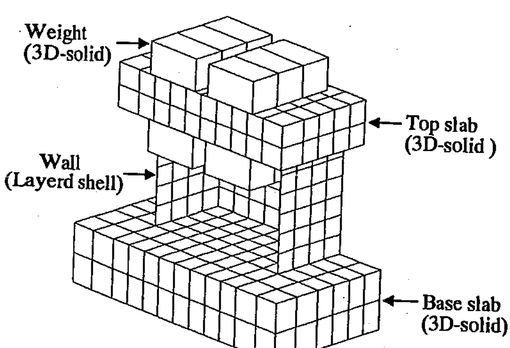


Fig. 2 FEM model of the test specimen

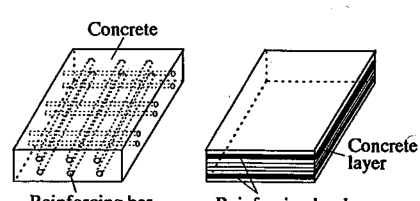


Fig.3 Outline of layerd shell element

2.3 Non-linear properties of concrete

Under compressive stress field, the Drucker-Prager's law is assumed as elasto-plastic material properties. Under tensile axial stress, the smeared crack initiates when the principal tensile stress exceeds criteria. The secondary crack caused by cyclic loading, has a direction perpendicular to the first crack in each layer of element (Fig.5). Fig. 6 shows the relationship between axial stress and strain in the cracking direction. The maximum strength parallel to cracking direction decrease by a half compared to the strength before cracking[2]. Rules for hysteresis loops of cracked concrete are shown in Fig. 7[3]. The hysteresis loop is composed of a series of connected lines. The hysteresis rule in compression stress is expressed by the plastic strain proposed by Karsan[4]. Degradation of compressive strength under cyclic loading is based on the Yamada's rule[5]. Fig. 8 shows the outline of experiments performed by Yamada et al., and their proposed experimental equations. Shear stiffness decreases as a function of strain perpendicular to the cracking direction. The degradation relations refer to the Yamada and Aoyagi rule as shown in Fig. 9[6].

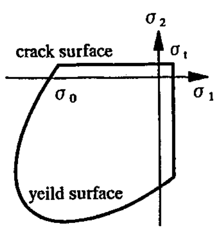


Fig. 4 crack and yeild surface (Drucker-Prager)

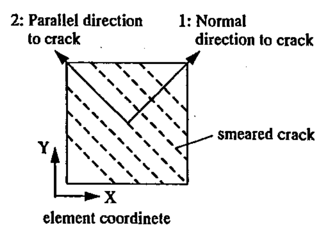


Fig. 5 Coordinete system after cracking

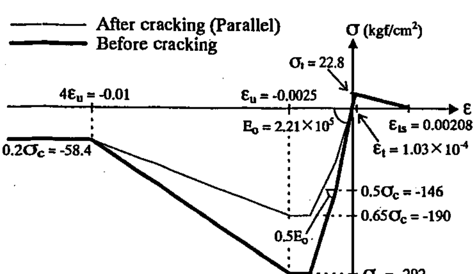


Fig. 6 Envelop curve of stress-strain relationship of concrete

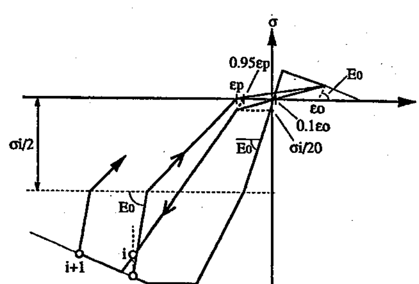


Fig. 7 Stress-strain relationship of concrete under cyclic loadings

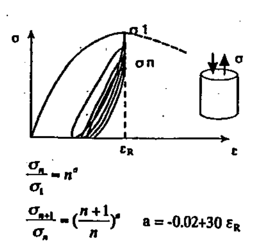


Fig. 8 Degradation of compressive strength under cyclic loading

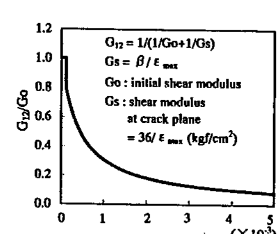


Fig. 9 Shear rigidity after cracking

2.4 No-linear properties of reinforcement

Reinforcements are replaced by anisotropic steel planes having an equivalent sectional area. The stress-strain relationship in both tensile and compressive stress is assumed to be bilinear as shown in Fig. 10. It is also assumed that the reinforcements and concrete are perfectly bonded.

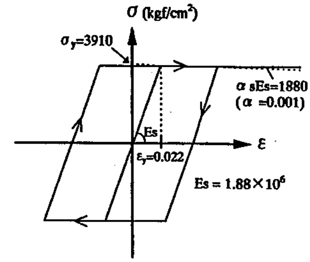


Fig. 10 Stress - strain relationship of reinforcement

2.5 Analytical method and conditions

Newmark's beta method is used for numerical integration. Convergence criteria are evaluated by Mewton-Raphson method. Damping is proportional to stiffness and has a constant value of 1.1 percent for initial natural frequency. In analyzing six successive runs in the test, damage conditions, such as cracking and residual stress before each run, were maintained. Table 1 shows the numerical data of material properties used in this analysis.

3 NATURAL PERIOD OF EACH MODEL

Natural period varied slightly with different models as shown in Fig. 11. The final model which have solid elements for the top and base slab, is able to take rotational inertia and local deformation where the slabs and walls join into account. The computation time for slabs of solid elements was not much longer than the time for slabs of shell elements.

4 COMPARISON OF TEST AND ANALYTICAL RESULTS

The maximum values of response accelerations of the test and analysis are compared in Table 2. The analysis and test results agree well through all runs except for Run 2, where the maximum value of test is larger than that of the analysis. Time histories of acceleration and displacement for Runs 4 and 5 are shown in Fig. 12-1 and -12-2. There is a slight difference near the time of 1.5 seconds in Run 4. However, after this and up to the ultimate state (3.5 seconds in Run 5), analytical results clearly simulate the decrease in stiffness

Transfer functions between the base slab and the top slab are shown in Fig. 13 and 14 for Runs 4 and 5 . Not only frequency values at the maximum peaks, but also those values for other peaks have good coincidence. The inertia at the top slab versus relative displacement between the base and top slab is compared in Fig. 15 and 16 for Runs 4 and 5. The analytical results indicate that unloading gradients at the peak are less than those in the test for Run 4, but loop width is wider near the origin than in the test. Therefore the difference in the total area enclosed by the loops is not very great between the test and analysis. On the whole, the stiffness of uploading and peak values are almost the same in both results. However, after the ultimate load, where displacement is more than 0.8cm, the analytical results for Run 5 did not agree with test results.

Table 1 Material properties

Concrete	
Compressive strength (kgf/cm^2)	292.0
Young's modulus (kgf/cm^2)	2.21×10^5
Poisson ratio	0.155
Cracking strength (kgf/cm^2)	22.8
Reduction factor of shear modulus	36.0
Reduction factor of compressive strength	0.65
Reinforcement	
Young's modulus (kgf/cm^2)	1.88×10^6
Yield strength (kgf/cm^2)	3910.0

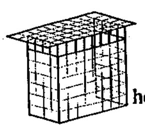
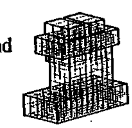
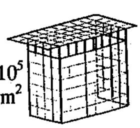
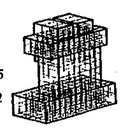
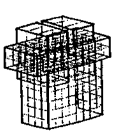
Model	Over view	Natural frequency(Hz)			Model	Over view	Natural frequency(Hz)		
Shell model		14.0 horizontal	39.0 vertical	68.9 banding	Top slab and base slab solid		13.5	37.0	59.3
Shell model Ec=2.06×10^5 kgf/cm^2 (0.88%)		13.2	37.3	66.3	Top slab and base slab solid Ec=2.21×10^5 kgf/cm^2 (0.95%)		13.2	36.7	58.4
Top slab solid		13.8	38.9	61.0	Test		13.2 horizontal		

Fig. 11 Natural frequency

Table2 Maximum response of Top slab (Horizontal)

	Displacement (mm)		Acceleration (mm/s^2)	
	Test	Analysis	Test	Analysis
RUN-1	0.29	0.29	2080	1960
RUN-2	0.58	0.74	3980	4870
RUN-2'	1.05	1.08	6070	5620
RUN-3	1.63	1.56	7040	6260
RUN-4	3.72	3.51	8820	8660
RUN-5	> 20.0	23.8	13410	14530

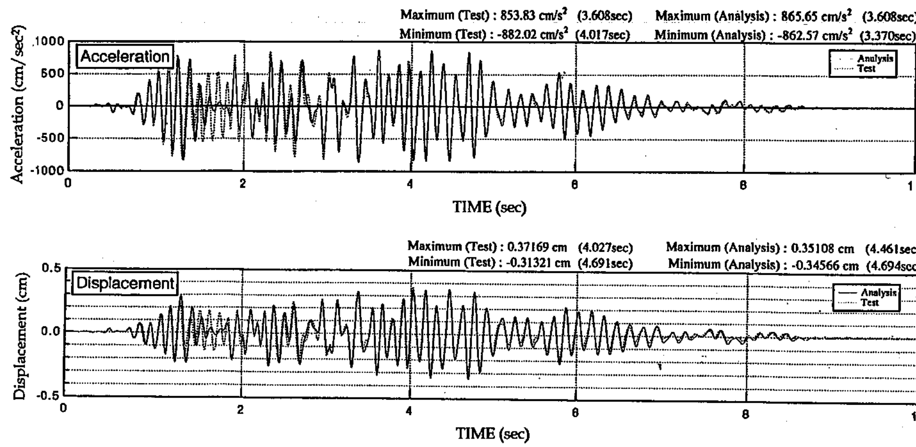


Fig. 12-1 Comparison of response at the top slab in the tests with analytical results (RUN-4)

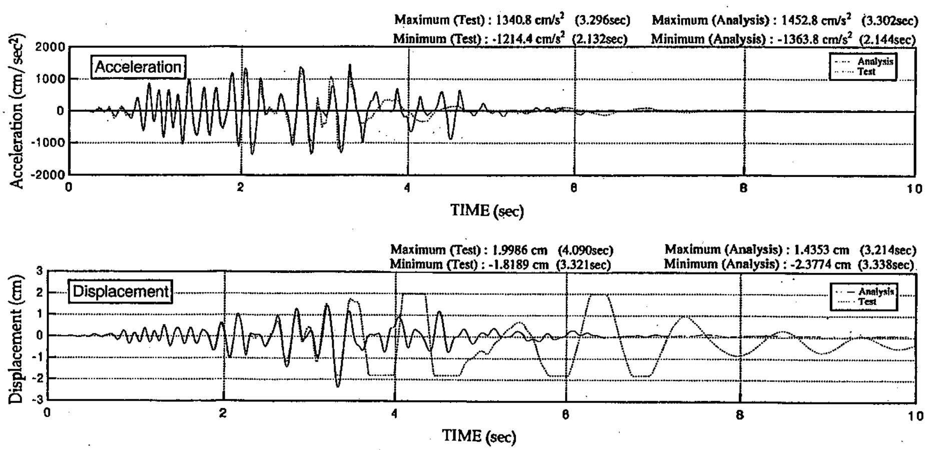


Fig. 12-2 Comparison of response at the top slab in the tests with analytical results (RUN-5)

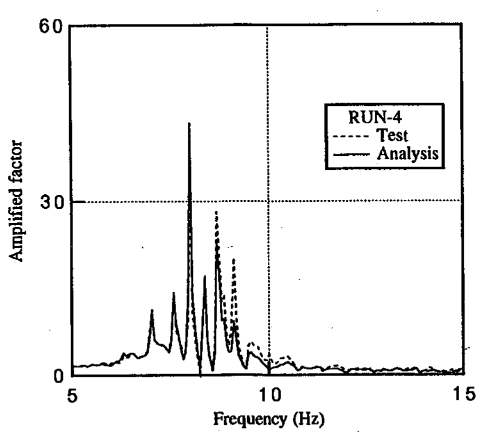


Fig. 13 Transfer functions in RUN-4

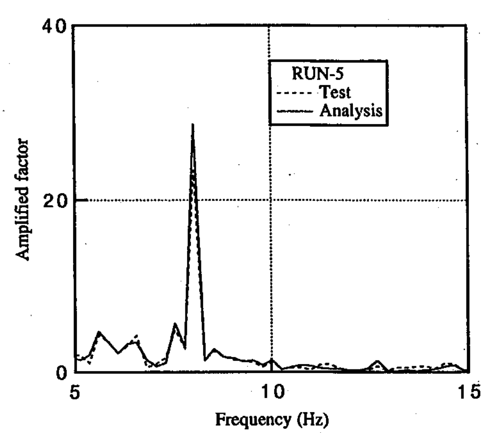


Fig. 14 Transfer functions in RUN-5

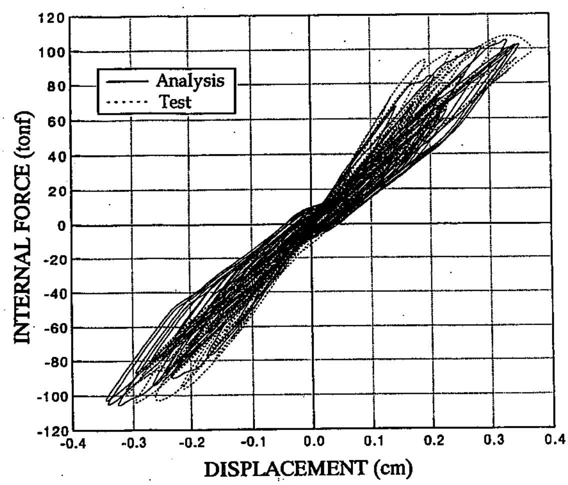


Fig. 15 Comparison of hysteresis loops in the tests with analytical results (RUN-4)

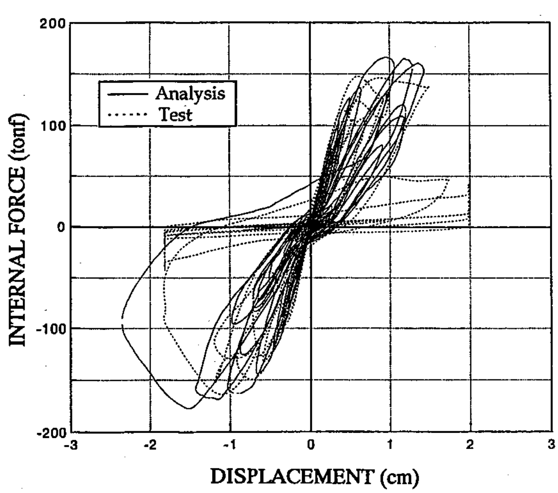


Fig. 16 Comparison of hysteresis loops in the tests with analytical results (RUN-5)

5 CONCLUSIONS

By using 3-D non-linear dynamic response analysis program, NAPISOS, the seismic shear wall for the ISP is analyzed.

A model which consists of solid elements for the base and top slab, and shell elements for the walls is analyzed. This model makes it possible to properly consider the rotational inertia of the top slab. Moreover, the solid elements are able to take local deformation near the joining of the slabs and walls. This model is applicable because computation time was not much longer than for a model using shell elements for the top slab.

Analytical results, such as the time histories of acceleration and displacement coincided well with the test results until the ultimate state. It is confirmed that the developed code can simulate the dynamic response of reinforced concrete shear wall under seismic load.

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