DYNAMIC FEM ANALYSIS OF REINFORCED CONCRETE SHEAR WALLS

Dachang ZHANG[*1], Takashi KASHIWAZAKI[*2] and Hiroshi NOGUCHI[*3]

ABSTRACT: Considering the nonlinear characteristics of reinforced concrete (RC), nonlinear dynamic FEM analysis has been carried out for RC shear wall specimens subjected to dynamic loads with a general model and a simplified model in DIANA. The time histories of acceleration and displacement responses with the general model were simulated reasonably well by the analyses, and showed a good agreement with the test results. The differences between the results with two models were investigated, and the reasons of them were discussed. The acceleration response spectra and cracking condition and damping effect were also investigated.
KEYWORDS: Seismic load, nonlinear dynamic FEM analysis, acceleration and displacement response, response spectra, cracking condition, damping effect

1. INTRODUCTION

In recent years, in order to investigate the dynamic performance of reinforced concrete structures subjected to seismic loads, a great many of tests and analytical studies have been carried out. The NUPEC had carried out static reversed cyclic loading tests of seismic shear walls and scaled models of reactor buildings to study their restoring characteristics. Focusing on loading rate effect, damping characteristics and dynamic response of seismic shear walls at the ultimate loading conditions by applying dynamic and pseudo-dynamic loads, the shaking table tests of RC seismic shear walls were carried out. In the paper, the three-dimensional dynamic FEM analyses of the RC shear walls were made with general purpose computer program DIANA, which were carried out in the test. The objective of this paper is to verify the FEM analytical models for dynamic response analysis and study the seismic response and the dynamic performance of the RC shear wall subjected to earthquake loads. The results of dynamic FEM analysis have a good agreement with the test results, and the dynamic performance of RC shear walls is discussed.

2. SUMMARY OF THE TESTS

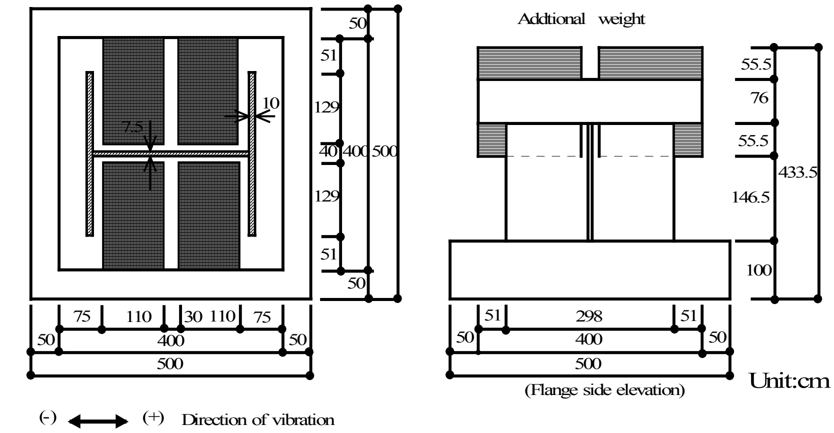
The tests were carried out by the Nuclear Power Engineering Corporation NUPEC to evaluate the seismic behavior of reactor buildings entrusted by the Ministry of International Trade and Industry (MITI). The data of the test results were provided as a Seismic Shear Wall ISP sponsored by OECD/NEA/CSNI. (1)

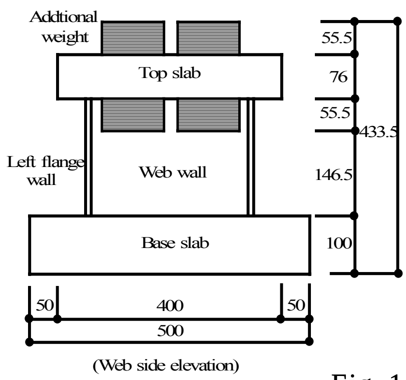
Fig.1 shows the specimens tested. The web wall was 75 mm thick, 2900 mm long clear span and 2020 mm clear height with a shear span ratio of 0.8. The flange walls were 100 mm thick and 2980 mm long. Deformed bar of nominal diameter 6.35 mm with spacing pitch 70 mm in a double layer was used for the vertical and horizontal reinforcement of the web wall. Total mass including top slab was 122,000 kg. Vertical compressive stress in the wall was 1.5 MPa. The specimens were subjected to excitation in one direction. The vibration test steps were set corresponding to the five target response levels. Each level uses the same input acceleration waveforms with varying amplitude. **Fig. 2** shows the input acceleration waveform for Run-4.

*1 Graduate School of Science and Technology, Ph.D. Student, Member of JCI

*2 Faculty of Engineering, Chiba University, Research Assistant, Member of JCI

*3 Faculty of Engineering, Chiba University, Professor, Member of JCI





Weight

weight of top slab: 291000 kg

additional weight: 929000 kg

total of weight: 122000 kg

Thick

web wall: 75 mm

flange wall: 100 mm

Cross sectional area: 813500 mm²

Vertical stress: 1.5 N/mm²

Fig. 1 Specimen

In this test, the following five steps were investigated:

i) Small amplitude level in elastic range (Run-1)

ii) Shear deformation angle of about 1/2000 rad (Run-2)

iii) Shear deformation angle of about 1/1000 rad (Run-3)

iv) Shear deformation angle of about 1/500 rad (Run-4)

v) Shear deformation angle of about 1/250 rad(Run-5)

Table 1 and **Table 2** show the modulus of elasticity and the mean strength test results of concrete and steel.

Before each test, small amplitude vibration tests were carried out to obtain the dynamic characteristics of the specimen. The natural frequency and damping factor obtained by these tests were summarized.

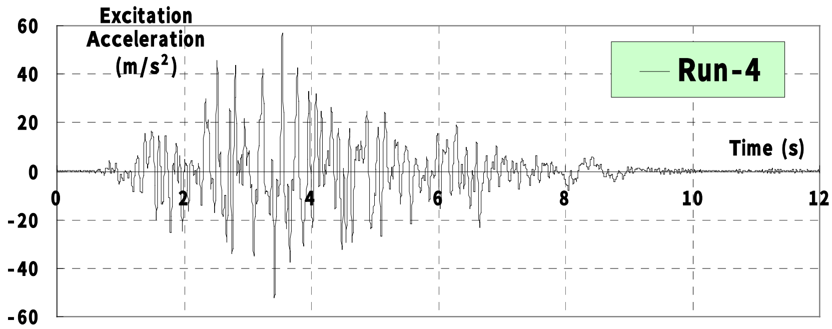


Fig. 2 Input acceleration time history for Run-4

Table 1 Tensile strength test of deformed bar, **D6**

Yield Strength (N/mm²)	Tensile Strength (N/mm²)	Modulus of Elasticity (× 10⁵N/mm²)
391	495	18.8

Table 2 Strength test results of concrete of web and flange walls

Compressive Strength (N/mm²)	Tensile Strength (N/mm²)	Modulus of Elasticity (× 10⁵ N/mm²)	Poisson's ratio
23.4	2.92	2.28	0.16

3. FINITE ELEMENT MODELS

3.1 Governing equations

The system governing equations for a nonlinear transient dynamic problem at time t can be given as [(2)]

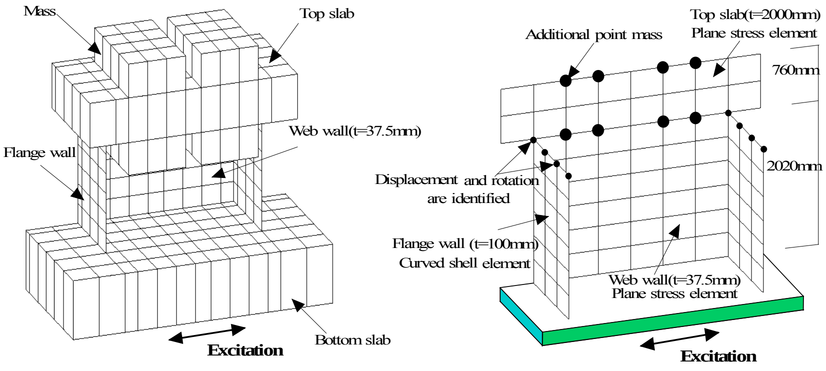
$$M\ddot{u}_{t+\Delta t} + C\dot{u}_{t+\Delta t} + Ku_{t+\Delta t} = f_{ef}(t+\Delta t) = -M\ddot{u}_{su}(t+\Delta t) \qquad (1)$$

where, u is the relative displacement and external loading $f(t+\Delta t)$ has been replaced by an effective loading due to the base excitation $f_{ef}(t+\Delta t)$ in which $\ddot{u}_{su}(t+\Delta t)$ represents the applied base acceleration vector. The symbols M, C and K are the mass matrix, the damping matrix and the stiffness matrix, respectively.

In practice the presence of damping reduces the steady state response and damps out the transient response. Applying a non-modal solution technique, it is necessary to evaluate the damping matrix C explicitly and usually viscous damping effects can be included by assumption of Rayleigh damping which is the form

$$C = aM + bK \qquad (2)$$

where a and b are constants to be determined from given damping ratios.



Model 1 **Model 2**

Fig. 3 Finite Element Models

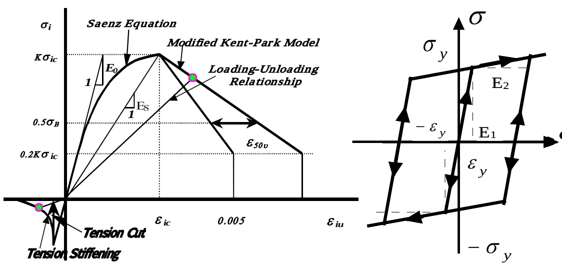
3.2 Finite element models

The finite element models are shown in Figure 3. **Model-1** is a general model that is similar to the specimens, and **Model-2** is a simplified model that is idealized from the specimens. The web wall is discretized with four-node quadratic plane-stress elements and the flange walls are discretized with four-node quadratic curved shell elements with three points in the out-of-plane direction.

3.3 Material models

(1) Material model of concrete:

Concrete was modeled as an isoparametric plane-stress element with four nodes. For its constitutive model, the theory of crack models based on total strain originally proposed by Vecchio and Collins [3]. The DIANA subroutine is developed to model the concrete model that is shown in **Fig.4** (a), but the DIANA original unloading-reloading model is used for the unloading-reloading model in the analysis. The loading-unloading-reloading condition was monitored with the additional unloading constrains r_k which were determined for both tension and compression to model the stiffness degradation in tension and compression separately.



(a) Concrete (b) Steel

Fig. 4 Constitutive models for materials

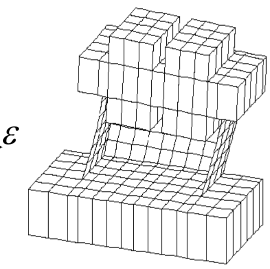


Fig. 5 Eigenmode 1 (f_1=13.2Hz)

(2) Material model of steel bars:

Steel bars were modeled as layered element, and the constitutive model was indicated by von Mises failure criteria. The stress-strain relationship was defined as shown in **Fig.4** (b). The bond-slip between concrete and steel is not considered.

4. DYNAMIC ANALYSIS

4.1 Eigenvalue analysis

The eigenvalue analysis was performed for a twofold purpose, firstly to check the finite element model, and secondly to determine the parameters a, b of the Rayleigh matrix. The parameters a and b can be calculated according to the equations $a=2\omega_1\omega_2\beta$, $b=2\beta$, where $\alpha=\omega_1/\omega_2$ and $\beta=(\zeta-\alpha\zeta_2)/(\omega_1-\alpha\omega_2)$, and ω_1, ω_2 are the two lowest frequency of the beginning of each Run.

The first five eigenmodes of Run-1 were determined which were expected to be representative for the structure. These are close to the analytical and tested results [1][4]. The first two eigenvalues from **Model-1** are f_1=13.2[Hz], f_2=37.7[Hz], and the first two eigenvalues from **Model-2** are f_1=13.7[Hz], f_2=45.0[Hz]. The stiffness of **Model-2** is greater than that of **Model-1**. Therefore, the parameter a is 1.074 and b is 0.00005 for Run-1.

4.2 Transient nonlinear analysis

The experimental program consisted of five levels, Run-1, Run-2, Run-3, Run-4 and Run-5, in which the amplitude was increased in each sequential run. The target levels at each run corresponded to a specific type of damage of the structure. During the nonlinear dynamic analysis, the Newmark method was used, which β was 0.25. The time interval was set at 0.004 second.

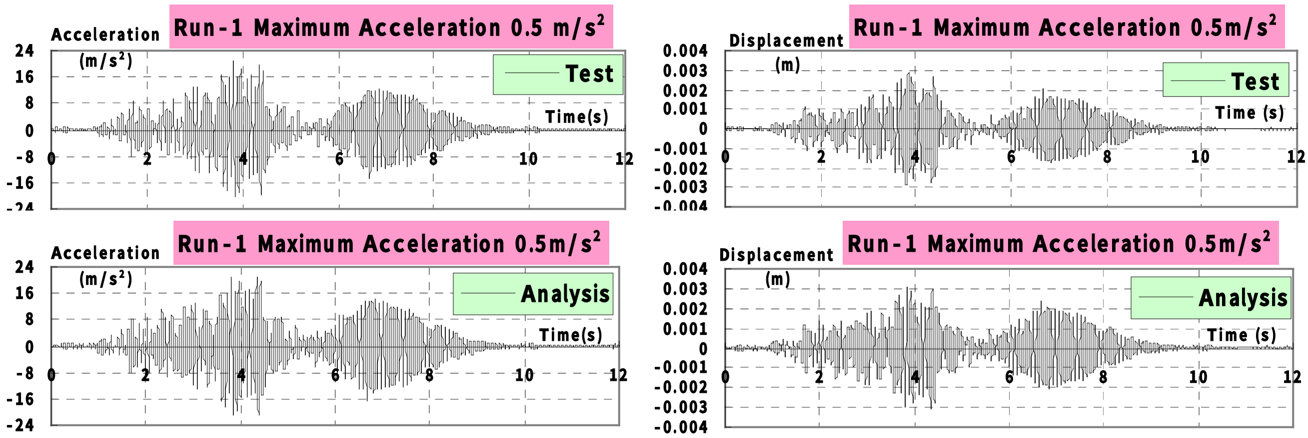


Fig. 6 Time history of acceleration and displacement response for Run-1 with Model 1

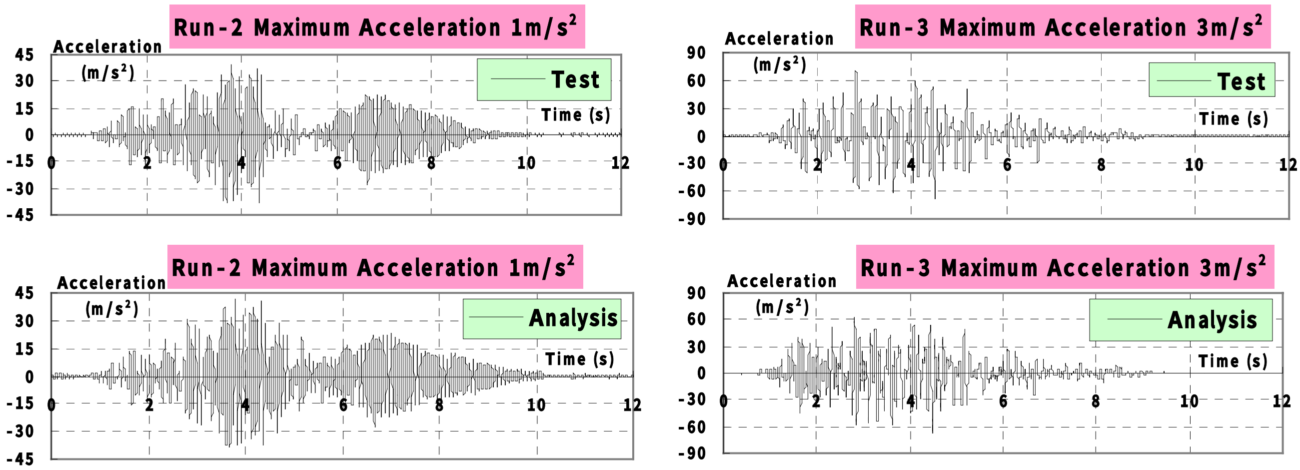


Fig. 7 Time history of acceleration response for Run-2 and Run-3 with Model 1

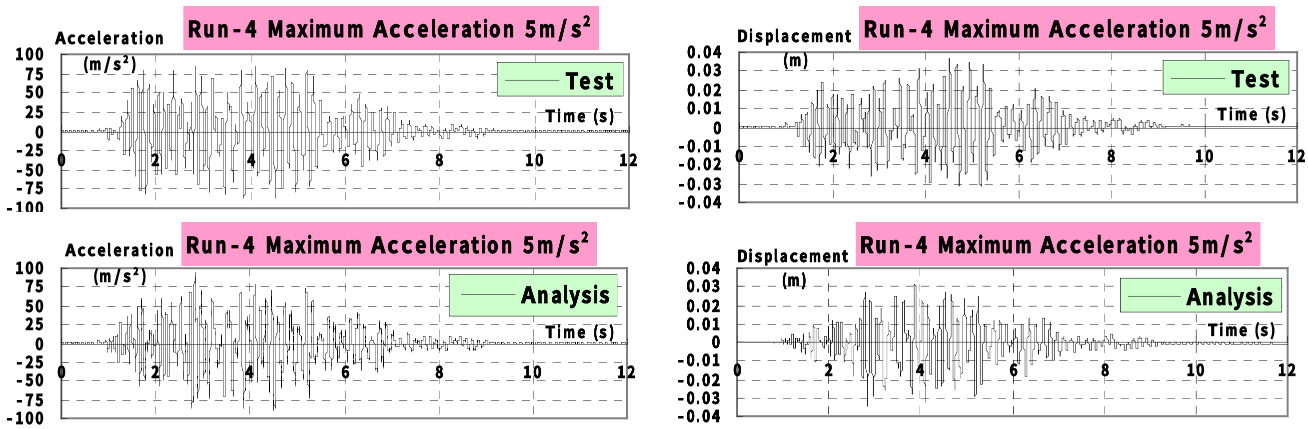


Fig. 8 Time history of acceleration and displacement response for Run-4 with Model 1

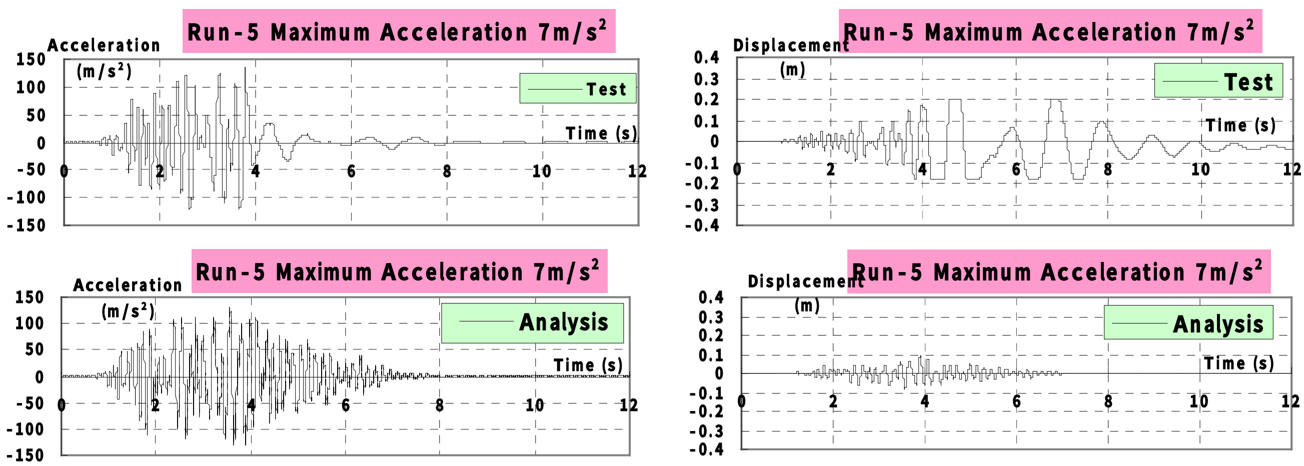


Fig. 9 Time history of acceleration and displacement response for Run-5 with Model 1

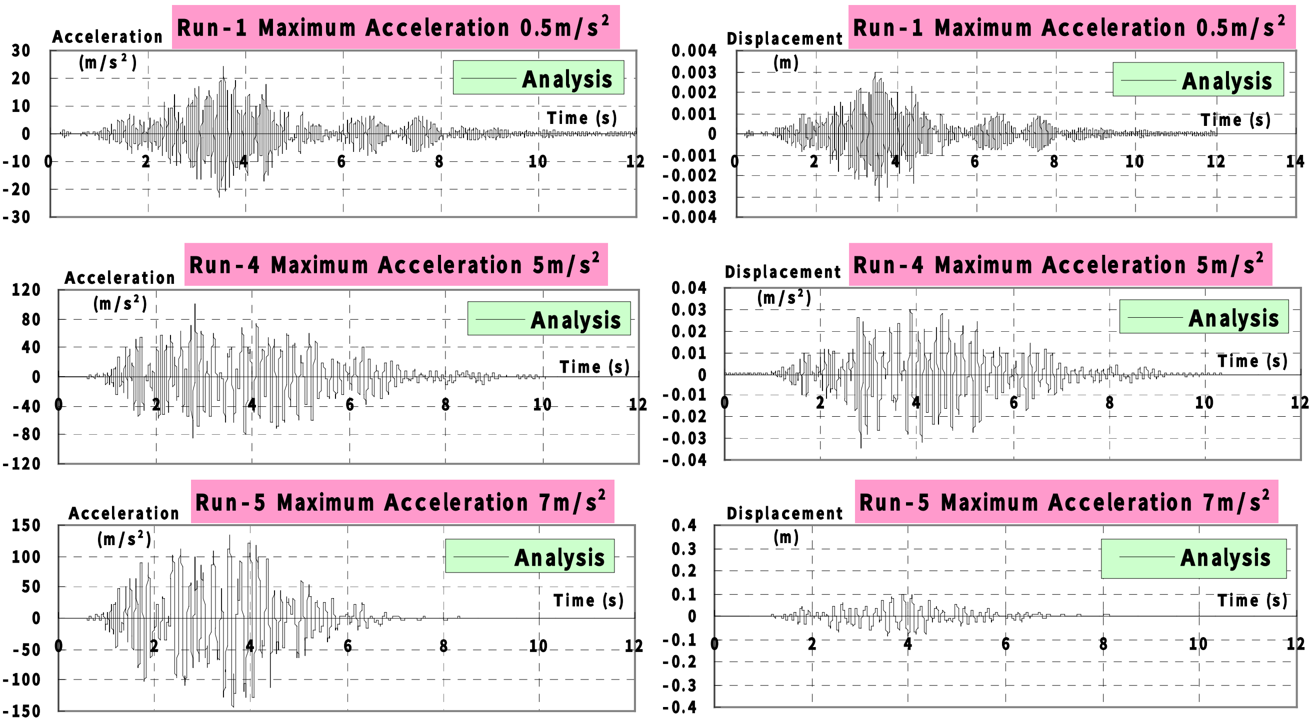
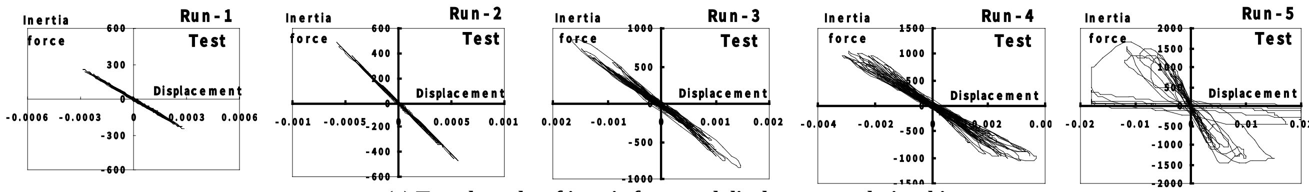
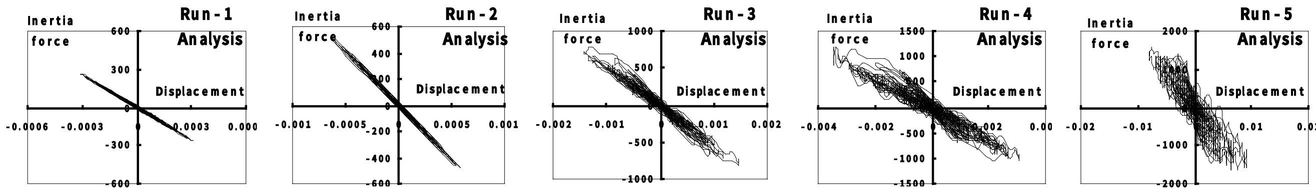


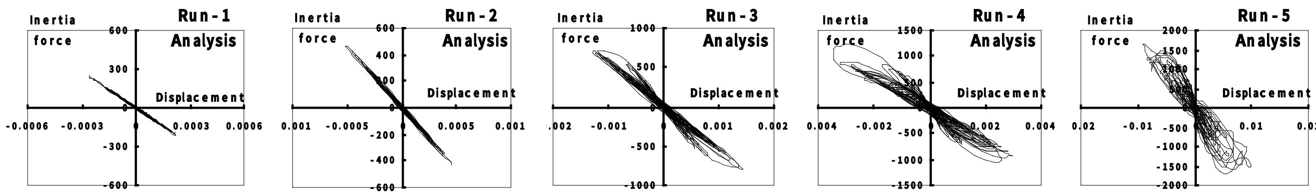
Fig. 10 Time history of acceleration response for Run-1, Run-4, Run-5 with Model 2



(a) Tested results of inertia force and displacement relationship



(b) Analytical results of inertia force and displacement relationship with Model 1



(c). Analytical results of inertia force and displacement relationship with Model 2

Fig. 11 Comparison of inertia force and displacement relationship

(1) Time History of Response

Run-1: The displacement and acceleration responses of time history of analytical results with general **Model-1** and test results are compared in **Fig. 6**, and the analytical results can simulate the test results in whole history very well. In the same way, seen from **Fig. 7**, the authors can know that the acceleration responses of time history for **Run-2** and **Run-3** by the analysis with general **Model-1** can simulate the test results very well too.

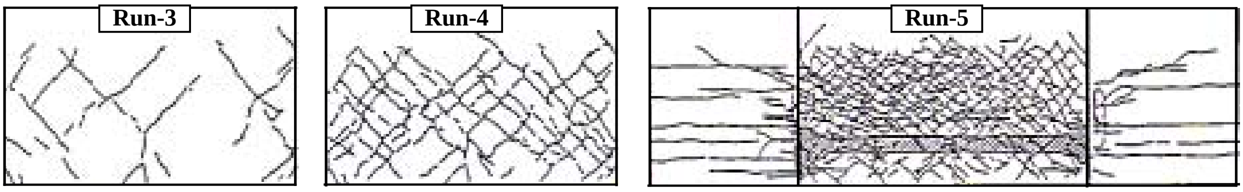
Run-4: The specimens' steel started to yield and concrete entered the plastic condition. **Fig. 8** shows that the displacement and acceleration responses of time history by analysis with general **Model-1** can simulate the test results nearly. There is a little difference between them because of the limits of loading-unloading material model.

Run-5: Seen from **Fig. 9**, the acceleration responses of time history by analysis with general **Model-1** can simulate the test results before four seconds when the shear sliding failure occurred, but it can't simulate the test results after the failure. The shear sliding failure can't be simulated as the failure of the whole RC shear wall system. The displacement responses of time history can't simulate the test results well from the beginning of **Run-5**. The main reasons are difference between the assumed and factual loading-unloading concrete model, and difference between static and dynamic loading-unloading failure criteria.

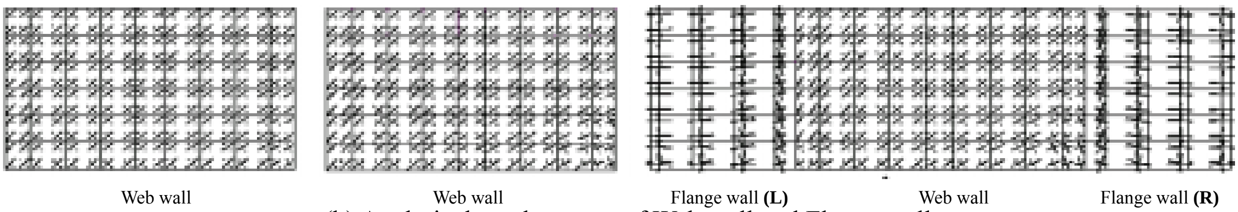
The results with simplified **Model-2** are shown in **Fig. 10**. The displacements and rotations of flange walls' points of **Model-2** are identified, which are different from the factual condition. So, the stiffness of **Model-2** is greater than that of specimen, and the analytical results of **Run-1** can't simulate the test results so well. The results of **Run-4** and **Run-5** with **Model-2** are similar to that of **Model-1**, because the stiffness softening happened and the stiffness of the **Model-1** is nearly equal to that of **Moeld-2**, after entered the material plasticity.

(2) Inertia Force Relationship and Displacement

Fig. 11 shows the inertia force and displacement relationship. The inertia force was calculated based on the acceleration and the mass of top slab. The analytical and tested results of Run-1 and Run-2 are linear because the specimen is in elastic. From Run-3, the specimens entered the inelastic condition, but the analytical results of Run-3 and Run-4 can nearly simulate the test results until the ultimate strength. The analysis of the Run-5 can't simulate the test results after the peak, because of the discrepancy of loading-unloading material model and failure criteria. **Model-2** didn't behave the response of head-vibration as **Model-1**.



(a) Tested Crack pattern of Web wall and Flange wall



(b) Analytical crack pattern of Web wall and Flange wall

Fig. 12 Crack pattern

(3) Crack Pattern

Fig. 12 shows the comparison of the analytical and tested crack patterns. It shows that the analytical cracks can simulate the process of crack propagation of the specimen. But there are some differences in the flange walls' cracks pattern between analytical results and test results, which the flange walls' vertical cracks were hardly occurred in the test.

5. CONCLUSIONS

(1) The dynamic FEM analyses can simulate the shear wall tests about the time history of acceleration and displacement responses before the ultimate strength.
(2) The analytical inertia force and displacement relationship can show the energy dissipation as that of the test.
(3) Even the simplified model can simulate the process of response well except elastic condition.
(4) The process of crack propagation can be simulated by dynamic FEM analysis.
(5) There are some differences between the analytical and tested results after ultimate strength. This indicates problems to be solved by loading-unloading material model and failure criteria.

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