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論 文

[2215] Identification of Stiffness Deterioration of Concrete Structures

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1. INTRODUCTION

Experiences from past strong earthquakes have shown the vulnerability of civil structures to strong ground shakings. Even in the case of mild earthquakes or under normal loading conditions, some level of damage is expected in structures, especially in concrete structures which deteriorate due to many factors such as creep, corrosion of reinforcements and microcracking. In this regard, in order to assess the soundness of these structures, both qualitative and quantitative analyses must be done. Recently, the authors developed the local identification for framed structures using Kalman filter and showed its numerical feasibility in the identification of the stiffness characteristics of individual members of plane frames [1]. Local identification has practical importance since damage is local in nature and damage in a few critical members significantly affects the deterioration level of the complete structure. The applicability and limitations of local identification when real structures are analyzed have still to be verified though. Therefore, an experimental investigation is conducted and presented in this paper.

The authors performed the shaking table test of a reinforced concrete space frame at the vibration laboratory of the Institute of Technology of Tokyu Construction Co. The accelerations and strains at critical locations of the specimen were measured and used as input and observations for the purpose of structural identification by Kalman filter. To observe the deteriorating condition of the specimen, the dynamic properties such as modal parameters and the stiffnesses of the members were identified at different stages of the structure. Results showed that system identification can be used in order investigate the deteriorating behavior of the structure as it accumulates damage due to intense ground oscillations.

2. LOCAL IDENTIFICATION

In local identification, an arbitrary frame member with end nodes h and j is isolated from the structure and an internal node i must be introduced. Only the responses at the nodes h, i and j are computed or measured and used in the identification of the structural characteristics of the member which include the axial rigidity (EA), flexural rigidity (EI) and

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damping parameters. The general procedure of the local identification is shown by the flowchart in Fig. 1 and the formulations of the equations necessary for the identification of the structural parameters by Kalman filter are given in Oreta and Tanabe [1] and Oreta [2].

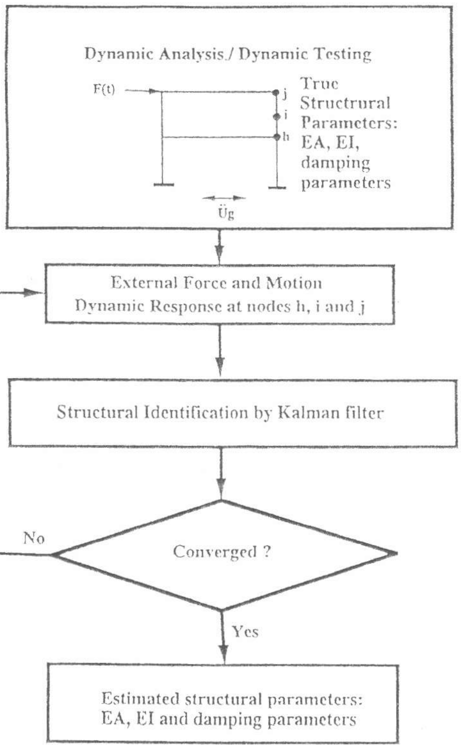

Fig. 1: Local Identification of Plane Frames

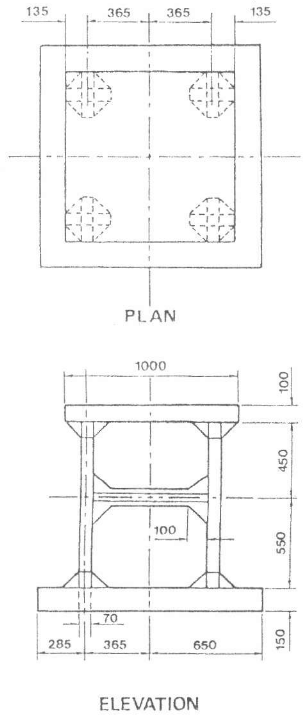

Fig. 2: Test Specimen

Table 1 Member Properties of Specimen

	Column	Beam
Cross-sectional area (cm^2)	49.00	17.36
Steel Reinforcement	SD35, D10	SD35, D10
Steel Ratio (%)	5.82	8.22
E_c (kg/cm^2)	2.55×10^5	2.55×10^5
E_s (kg/cm^2)	2.10×10^6	2.10×10^6

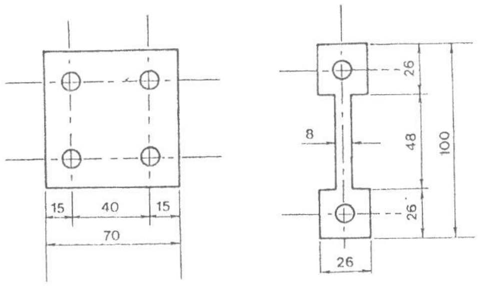

Fig. 3: Detail of Column and Beam Members

3. DESCRIPTION OF EXPERIMENT

3.1 TEST SPECIMEN

The test specimen used for the vibration test is a rigid space frame consisting of reinforced concrete elements (Fig. 2). It has a weight of 945 kg, height of 110 cm, width of 100 cm and depth of 100 cm. The top of the slab is supported by four columns with 7.0 cm square cross-sections. At the midheight of the columns are stiffening beams with I-shape cross-sections. Details of the column and beam members are shown in Fig. 3. Table 1 shows the properties of these members. An additional steel mass weighing 935 kg was symmetrically installed on the top slab.The specimen was bolted on top of the shaking table. Steel channels were installed at the four sides of the bottom slab to rigidly fix the specimen on the table.

3.2 INSTRUMENTATION

To systematically monitor the response of the structure during the experiment, critical points of the members of the specimen were selected. Since the motion of the shaking table was conducted along the $N-S$ direction or $X-$axis, then the two opposite frames of the structure at the east and west sides were observed. The east frame consists of columns A and B, while the west frame consists of columns C and D. For each column and beam of the two frames, internal and boundary nodes were selected and observed. The notation used for the nodes are shown in Fig. 4. Different measuring devices or sensors were attached to the specimen to monitor accelerations, displacements and strains at the critical locations. Strain-gauge type accelerometers were installed at the top and bottom slabs to measure the horizontal motion. To obtain the vertical and horizontal responses at specified nodes of the column and beam members, accelerometers were installed. Linear-variable displacement transducers (LVDT) were also installed at the bottom ends of the columns to measure the bottom rotation. 100 mm pi-gauges ($\pi-$gauges) were fixed at the opposite sides of the columns and beams at the specified nodes to measure the strains. These strains were used to derive indirectly the rotations at the nodes. The locations of these sensors for a typical frame are shown in Fig. 5.

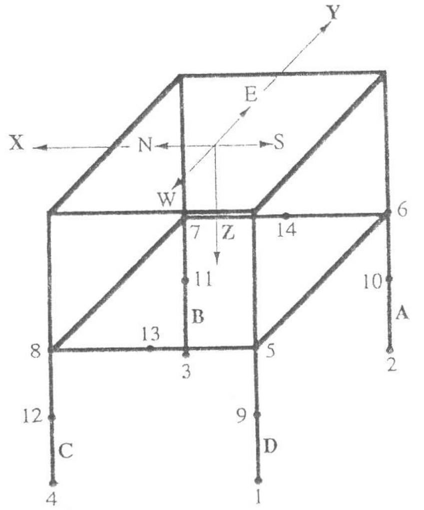

Fig. 4: Notation of Nodes and Columns

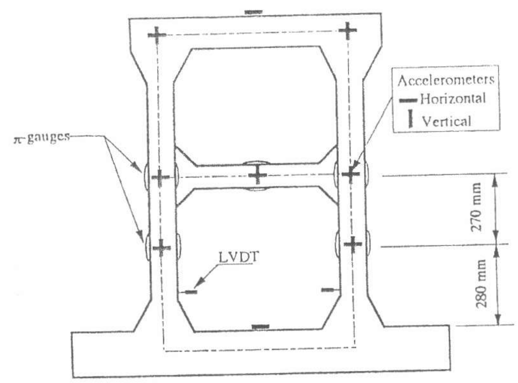

Fig. 5: Location of Sensors

3.3 VIBRATION TESTS

Vibration tests were conducted using the shaking table facility of the Institute of Technology of Tokyu Construction Co. in Kanagawa, Japan. Free vibration tests were performed by hitting the top slab of the specimen by a hammer. These tests were performed at the beginning before any actual shaking table test and after every shaking table test. Low amplitude tests using a sinusoidal function as an input excitation were also performed. The sinusoidal function had a low and constant amplitude of 20 gal and forcing frequency of 7.0 Hz. The low amplitude was intended such that no apparent damage was introduced to the structure. These tests were also performed at the beginning and after every main shaking table test.

The main shaking table tests were performed using a sinusoidal function as an input excitation. The amplitude of the sinusoidal function was increased from 50 gal gradually until the structure failed. The frequency of the sinusoidal function was set to a value close to the fundamental frequency of the structure for resonance to occur. The purpose of the main shaking table test was to introduce damage to the structure gradually. As the intensity of the shaking increased, the structure accumulated damage in the members reducing the overall stiffness.

Before the main test, cracks already existed in some parts of the structure, particularly at the beams where hairline cracks were observed. These cracks may have been due to shrinkage and accidental shocks during handling and transporting. The columns, however, were intact and no major crack existed. In the beginning, a forcing frequency of 7.0 Hz was used in the shaking table tests. By applying the shaking table tests with increasing amplitude of 50, 75, 100, 150, 200, 250, 300, 400 and 500 gal, diagonal cracks at the beams were observed. The beam connecting columns C and D lost its rigidity quickly. With increase in intensity of the oscillation, several cracks also started to form at the ends of the columns. After applying the 600 gal shaking, the columns C and D started to fail at the bottom portions. At this stage, the fundamental frequency of the structure decreased and was estimated to be 4.5 Hz. To destroy the structure, the forcing frequency was changed to 4.5 Hz and continuous shaking was performed until the structure failed. After 90 cycles of shaking, vertical cracks parallel to the reinforcements were formed at columns C and D and the concrete cover spalled. The measuring devices such as the accelerometers and π−gauges on the faces of the columns were removed and it was impossible to make more measurements. The shaking table test was terminated at this stage. It must be noted that at this stage, the columns are relatively still stiff because the reinforcements and core concrete were still intact. The failure of the structure was more of spalling of the concrete cover in the columns and shear failure in the beams. Fig. 6 shows the specimen at failure.

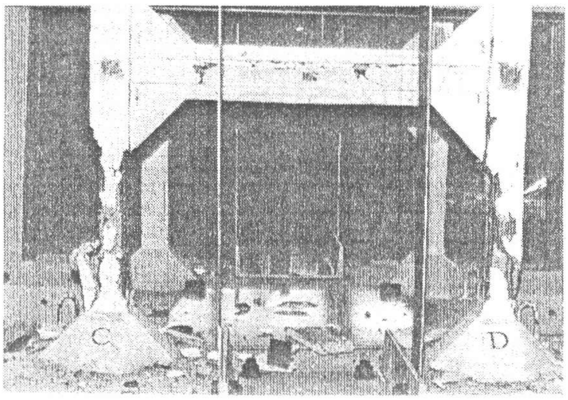

Fig. 6: Specimen at Failure

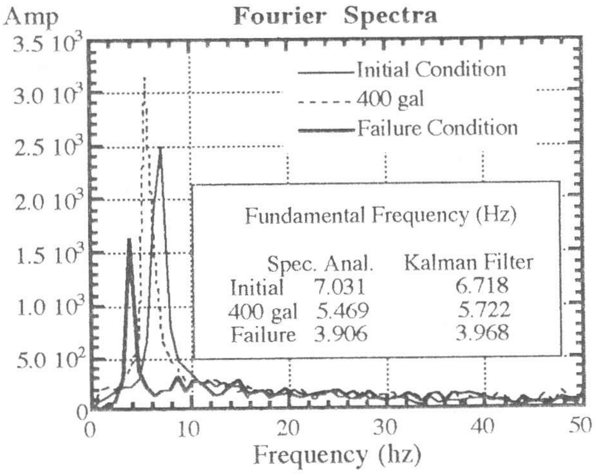

Fig. 7: Fourier Spectra (Free Vibration Tests)

4. INVESTIGATION OF STRUCTURAL DETERIORATION

4.1 GLOBAL CHANGES IN DYNAMIC CHARACTERISTICS

To observe the changes of the dynamic properties of the overall structure, the measured data from the free vibration or impact tests were analyzed. Using the modal parameter identification by Kalman filter for free vibration case [3] and the measured acceleration data of the top slab as observed data, the dynamic properties (damping ratio and frequency) of the structure modeled as a single-degree-of-freedom system at each stage were identified. Fast Fourier transforms (FFT) were also computed for 1024 points of each acceleration and the corresponding Fourier spectra were obtained. By peak-picking, the fundamental frequency at each stage was obtained. Fig. 7 shows the Fourier spectra for three stages of the specimen. The decrease of the fundamental frequency from the initial condition to the failure condition was clearly demonstrated by the shifting of the peak amplitude from a higher to a lower frequency. The estimated frequencies of the structure at different stages using spectra analysis and Kalman filter are also shown in Fig. 7. Initially, the specimen had a fundamental frequency of about 6.7 to 7.0 Hz. As the specimen was subjected to sinusoidal shaking with increasing amplitudes of the sinusoidal function, the members of the frame, particularly, the stiffening beams at midheight were damaged. With more intense shaking, cracks started to form in the columns and the concrete cover started to spall. As a result, the global stiffness and frequency of the structure decreased. At the failure condition, the frequency of the specimen decreased to about 4.0 Hz.

4.2 COLUMN STIFFNESS IDENTIFICATION

The local identification method developed for plane frames was applied to the estimation of the stiffness properties of the column members of the specimen using the data obtained from the sinusoidal shaking tests. The measured and derived response quantities at the boundary nodes were used as input data, while the internal response quantities such as horizontal acceleration or rotational displacement were used as observations. To neglect the table-specimen interaction, the measured acceleration at the base or bottom slab was used as the input motion.

Four stages of the structure which were denoted as s1t2c, s1t5a, s1t8a and s1t10 were considered. At s1t2c, the response quantities obtained from a low amplitude test of 20 gal amplitude which was performed before any main shaking table test were used. Hence, this stage represented the initial or intact state of the structure. The stage, s1t5a, was based on a main shaking table test of 200 gal amplitude. During this stage, the cracks which initially existed in the beams aggravated and started to extend diagonally to the ends. The stage, s1t8a, was that of a main shaking table test with 400 gal amplitude. At this stage, the shear cracks formed at the ends of the beams particularly for the beam connecting columns C and D. Some cracks also started to form at the top and bottom of columns C and D. Stage, s1t10, was a low amplitude test of 20 gal amplitude performed after the main 600 gal amplitude sinusoidal test. After the 600 gal amplitude sinusoidal oscillation, vertical cracks near the bottom of columns C and D were observed. Failure of the west frame started to occur. This was the stage just before the structure failed.

Table 2. Identified Stiffness Parameters of the Columns

	Column A	Column B	Column C	Column D
s1t2c (Initial)				
EA	1.843×10^{10}	1.456×10^{10}	1.059×10^{10}	1.423×10^{10}
EI	7.901×10^{10}	8.500×10^{10}	8.432×10^{10}	8.000×10^{10}
s1t5a (200 gal)				
EA	8.110×10^{9}	4.805×10^{9}	2.716×10^{9}	1.328×10^{9}
EI	7.510×10^{10}	7.465×10^{10}	5.347×10^{10}	7.863×10^{10}
s1t8a (400 gal)				
EA	2.254×10^{9}	1.525×10^{9}	8.000×10^{9}	4.514×10^{9}
EI	7.500×10^{10}	7.235×10^{10}	5.166×10^{10}	7.669×10^{10}
s1t10 (600 gal)				
EA	2.952×10^{9}	2.911×10^{9}	8.676×10^{9}	1.247×10^{9}
EI	7.806×10^{10}	6.790×10^{10}	4.512×10^{10}	5.880×10^{10}

Table 2 shows the results of the identification for each stage. From Table 2, the decreasing value of the flexural rigidity for each column was observed except for column A. Using s1t2c as the reference, the percentage reduction of the flexural rigidities EI at the three stages for the three columns are as follows: (1) Column B: 12.17% at s1t5a, 14.88% at s1t8a and 20.11% at s1t10; (2) Column C: 36.59% at s1t5a, 38.73% at s1t8a and 46.99% at s1t10; and (3) Column D: 1.71% at s1t5a, 4.13% at s1t8a and 26.5% at s1t10. It is noted that column C has the largest stiffness degradation followed by column D. This result was observed in the experiment where the damage in columns C and D was worst especially at the failure stage. The failure of the columns, however, was simply spalling of concrete cover and the decrease of the flexural stiffness was due to the reduction of the area of the cross-section. It is interesting to note that the theoretical values of the stiffness parameters based on the actual dimensions of the column members were close to the estimates. Using the transformed section method where the ratio of the steel and concrete elastic moduli, $n = 8.24$, transformed area, $A = 72$ cm^2, and transformed moment of inertia, $I = 283$ cm^4, the theoretical values of the stiffness parameters were calculated; i.e. for axial rigidity, $EA = 1.799 \times 10^{10}$ N and the flexural rigidity, $EI = 7.064 \times 10^{10}$ N-cm^2. The estimates of the flexural rigidities (EI) converged and followed a decreasing trend as the data for different shaking stages were used. On the contrary, the estimation of the axial rigidity (EA) did not follow this continuous decreasing trend. In some stages it decreased and in others it increased. The main reason for this is that the measured vertical motion at the nodes were too small to make the identification. In general, the estimated values showed the deteriorating trend of the stiffness characteristics of the structure. However, it is noted that an improvement of the modeling of the members by considering the effects of the rigid zones at the ends, and a more accurate method of measuring the responses particularly the rotations would yield better estimates of the stiffness parameters.

4. CONCLUSION

The results of the present study showed that the deterioration of the structure can be observed by the system identification approach. Modal identification showed the deterioration of the structure as reflected by the decrease of its fundamental frequency with the accumulation of damage. Local identification, on the other hand, illustrated the apparent reduction of the stiffness of column members with damage and the nonlinear behavior caused by damage due to the formation of cracks, spalling of concrete, and shear cracks in the beams. In general, the identified parameters showed the deteriorating trend of stiffness characteristics of the structure.

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