Application of Numerical Simulation and Vibration Measurements for Seismic Damage Assessment of Railway Structures*

Fumiaki UEHAN** and Kimiro MEGURO***

In this study, the authors discuss methods to assess the future/actual damage to RC structures by using numerical simulations and vibration measurements. First, the applicability of the Applied Element Method (AEM) is examined as an assessment tool for the seismic performance of RC structures with/without retrofit. Cyclic loading tests and seismic response of RC structures are simulated. Next, a method to improve the accuracy of vibration diagnoses of earthquake damaged RC structures is discussed by using damage assessment criteria calculated with the AEM. The AEM could simulate the damage behavior of RC columns, jacketed RC columns and an actual railway viaduct. The change of natural frequencies due to damage to RC columns and an actual railway viaduct with steel jacket were also correctly estimated. Seismic performance check of structures and development of assessment criteria for damage inspection can be effectively done by the AEM.

Key Words: Structural Analysis, Damage Mechanics, Nondestructive Inspection, Railway Structure, Numerical Simulation, Seismic Damage, Applied Element Method, Vibration Measurement

1. Introduction

A tool for assessing seismic performance (*assessment of future damage*) of RC structures is indispensable to mitigate the earthquake damage to a railway system. A tool for the inspection of the damage levels (*assessment of actual damage*) of RC structures is also necessary for promptly restoring an ill-functioned railway system due to an earthquake. In this study, we discuss methods to assess the future/actual damage to RC structures by using numerical simulations and vibration measurements. In the future, it is indispensable to correctly detect weak points of RC structures and provide effective and economical reinforcement for the earthquake disaster mitigation of railways. The authors study the applicability of *Applied Element Method* (AEM)⁽¹⁾ as an evaluation tool for the seismic performance of RC structures with and without seismic strengthening.

Numerical simulations of the failure mechanism of RC columns and railway viaduct are carried out in order to check whether the weak point of RC structures can be correctly detected by the AEM or not. For promptly restoring an earthquake-damaged railway, a tool for the inspection of the actual damage levels of RC structures is crucial. In order to assess the degree of damage to a RC structure using the inspection techniques based on vibration measurements, it is necessary to identify the changes of the dynamic characteristics due to structural damage in advance. To this end, the accuracy of the AEM on the simulation of natural frequency of damaged structure is investigated through the comparison of the results of experiments and numerical simulations.

2. Numerical Simulation Technique

Figure 1 illustrates the modeling of a RC structure in the AEM⁽¹⁾. It is assumed that the structure is modeled on an assembly of small rectangular elements obtained by virtually dividing the structure. Each element is connected by pairs of normal and shear springs located at contact points, which are distributed along the element edges. In the case of two-dimensional analysis, each element has three degree of freedoms. A concrete material model is applied to

* Received 13th January, 2004 (No. 04-4011)

** Structural Mechanics Laboratory, Railway Technical Research Institute, 2-8-38 Hikari-cho, Kokubunji-shi, Tokyo 185-8540, Japan. E-mail: uehan@rtri.or.jp

*** Institute of Industrial Science, The University of Tokyo, 4-6-1 Komaba, Meguro-ku, Tokyo 153-8505, Japan. E-mail: meguro@iis.u-tokyo.ac.jp

Fig. 2 Material model of concrete and reinforcement

Fig. 1 Modeling of RC structure in the Applied Element Method (AEM)

each spring. At the rebar locations, two sets of springs are used, one for the concrete and the other for the reinforcing bar. Nonlinear material models of steel and concrete as shown in Fig. 2 are given to the springs. If the spring stress exceeds its resistance, it may yield and eventually fail. In this way, the AEM can follow the structural behavior from the elastic range to the total collapse.

The two-dimensional model of a jacketed column⁽²⁾ is composed of three different types of elements as shown in Fig. 3. The first one is the element for the RC inside jacket (E_C), the second one is the element for the steel jacket on the sides of the RC column (E_{JS}) and the third one is the element for the steel front and rear jackets (E_{Jb}). The first element type has RC material properties whereas the remaining have steel material properties. There is no connection between the E_{Jb} and E_C elements as shown in Fig. 3. Because the edge elements of both E_C and E_{Jb} are connected with E_{JS} , the steel jacket can restrain the inner concrete core. The concrete core may crack and the reinforcement may yield and cut.

Fig. 3 Modeling of RC column with steel jacket

3. Numerical Simulation of the Failure Mechanism of RC Structures

3.1 Numerical simulation for earthquake disaster mitigation of RC structures

It is indispensable to correctly detect weak points of RC structures and provide effective and economical reinforcement for earthquake disaster mitigation of railways in the future. In this section, we study the applicability of the AEM as an evaluation tool for the seismic performance of RC structures with and without seismic strengthening. First, the main reinforcement cutoff is considered as the weak point of the RC column. For this, the change in the failure mode of RC columns due to the change in the anchorage length of the main reinforcement is simulated. Then, simulations to verify the effect of the seismic strengthening of the RC columns with steel jacket are carried out. Finally, in order to check whether the weak point of structure can be detected correctly by the AEM or not, the seismic response simulation of two-storied RC rigid frame viaduct which collapsed during the 1995 Kobe earthquake is performed.

3.2 Experiment and numerical simulation of vulnerable RC column

3.2.1 Outline of the experiment and numerical simulation Kawashima et al.⁽³⁾ conducted the loading test of specimens of RC columns with main reinforcements having inadequate anchorage length. The details of the specimens are shown in Table 1. The difference among the four specimens is the reinforcement cutoff. From the design viewpoint, the reinforcement development length — according to the design standard — is 116 cm. The

Fig. 4 Specimen layout (upper) and failure mode (lower: experiment (left), simulation (right))

Table 1 Details of specimens

Specimen No.	n-1	n-2	n-3	n-4
Dimensions of cross section (cm)	50 × 50			
Effective height (cm)	250			
Shear span ratio	5.4			
Cutoff length from base (cm)	-	110	135	160
Yield stress: main reinforcement (MPa)	308			
Yield stress: hoop reinforcement (MPa)	272			
Young's modulus: reinforcement (GPa)	200			
Compressive strength: concrete (MPa)	31.3	32.0	32.5	31.9
Young's modulus: concrete (GPa)	28.0			

main reinforcement in Specimen $n-1$ is not cut, i.e. it extends along the whole column height. Half of the main reinforcement of Specimens $n-2$, $n-3$, and $n-4$ is cut at the column mid-height. In Specimen $n-2$, the main reinforcement length is 110 cm or 6 cm shorter than the recommended by the design standard whereas in Specimens $n-3$ and $n-4$, the lengths are 135 cm and 160 cm, respectively, longer than the design standard recommendation.

The specimen footing is fixed to the reaction floor and the cyclic load is applied to the top end by a dynamic actuator. In this case, no axial force is applied. The yield displacement δ_0 ($= 1.3$ cm) of the Specimen $n-1$ is defined as the standard displacement. The displacement amplitudes are gradually increased, $n \cdot \delta_0$ ($n = 1, 2, 3, \dots$). The number of cycles for one loading amplitude is ten.

The specimens are modeled in 2-D using 620 5 cm-side square elements. Plane stress condition is assumed. The strength of concrete and steel bars in the numerical model is the same as the strength of real specimens. Al-

Table 2 Comparison of maximum strength

Specimen No.	n-1	n-2	n-3	n-4
Experiment (kN)	167	148	161	166
Simulation (kN)	154	143	145	160

though experimental loading conditions are adopted in the numerical analysis, the number of loading cycles per one loading step is one.

3.2.2 Results and considerations Table 2 shows the maximum load for the experiment and numerical simulation. The results of numerical simulation are 90 to 97% of the experimental results. The maximum strength of the main reinforcement in the numerical simulation was assumed 1.75 times of the yield strength of the real reinforcements. It is considered that the maximum strength of reinforcement used in the simulation was smaller than the actual value.

The results of the experiment and numerical simulation are compared. Figure 4 shows the specimen failure modes and Fig. 5 shows the distribution of the main reinforcement axial strain.

Specimen $n-1$ (No cutoff): In both the experiment and simulation, the damage to the RC column concentrates at the column bottom. The column reinforcement fractures and the column reaches its ultimate state at displacements equal to $8 \delta_0$ and $9 \delta_0$ in the experiment and simulation, respectively.

Specimen $n-2$ (Cutoff length: 110 cm): In both the cases, the damage concentrates at the center of the column where the main reinforcement is cut. The axial strain of

Fig. 5 Main reinforcement axial strain (upper: experiment, lower: simulation)

the main reinforcement without cutoff concentrates at the cutoff height. The column reaches its ultimate state for an input displacement of $6.5 \delta_0$ in the experiment and $6 \delta_0$ in the simulation.

Specimen *n*-3 (Cutoff length: 135 cm): In both cases, the damage to the RC column concentrates at the column bottom but the reinforcement cutoff region is also damaged. In the experiment, damage begins to concentrate at the bottom after a displacement of $6 \delta_0$ and when it equals $8.5 \delta_0$, the specimen reaches its ultimate state at the bottom. In the numerical model, the hoop reinforcement breaks when the input displacement is $6 \delta_0$ after which the damage concentrates at the bottom.

Specimen *n*-4 (Cutoff length: 160 cm): In both cases, the RC column damage concentrates at the column bottom. When the input displacement is $8 \delta_0$, the column reaches its ultimate state due to the main reinforcement fracture.

In the experiments, Specimen *n*-2, which has the shortest development length, is damaged at the column center where the main reinforcements are cut. As the development length increases, the damaged region shifts to the column bottom and the damage state approaches Specimen *n*-1, which has continuous main reinforcement. The AEM numerical simulation results matched well the phenomenon that occurred in the experiments. The maximum strength and ultimate displacement obtained with the simulation coincided with the experimental results. The simulated axial strain of the main reinforcement fairly agrees with the experiment results, and the simulated results followed the tendency of the strain distribution changes that accompany the changes of the development lengths. It

is well known that the measurement of the reinforcement strain is very difficult in case of damaged RC structures. Kawashima et al.⁽³⁾ pointed out that a certain error might be included in the axial strain experiment results. Therefore an extensive discussion of the accuracy of the simulated strain is not considered necessary.

3.3 Experiment and numerical simulation of RC column reinforced by steel jacket

3.3.1 Outline of experiment and numerical simulation In order to check the effect of earthquake strengthening of RC columns by steel jacketing, Kawashima et al.⁽³⁾ conducted loading tests of RC specimens with steel jackets. Table 3 shows the details of the specimens used in the experiment. The details of those specimens are almost the same being the difference among them the steel jacket length. The reinforcements of all specimens are cut 90 cm above the base, which is 22 cm shorter than the design standard recommendation. Specimen *j*-1 is not strengthened. The steel jacket of Specimen *j*-2 is 50 cm long and the center of the steel jacket coincides with the main reinforcement cutoff level. A 75 cm-long steel jacket is used for Specimen *j*-3 and the main reinforcement cutoff level is 25 cm above the jacket lower edge. The tensile strength of steel jacket is 274 MPa and the space between the RC column and the steel jacket is filled with a 3 mm-thick layer of epoxy resin.

The boundary and loading conditions of this set of experiments are similar to those of the experiments described in the previous section. However, in this case 282 kN of axial force is applied and the yield displacement $\delta_0 (= 1.5 \text{ cm})$ of the Specimen *j*-1 is defined as the standard displacement.

Fig. 6 Specimen layout (upper) and failure mode (lower: experiment (left), simulation (right))

Table 3 Details of specimens

Specimen No.	j-1	j-2	j-3	(j-4)
Dimensions of cross section (cm)	50 × 50			
Effective height (cm)	260			
Shear span ratio	5.6			
Cutoff length from base (cm)	90			
Yield stress: main reinforcement (MPa)	409	357		
Yield stress: hoop reinforcement (MPa)	433	245		
Thickness: steel jacket (cm)	-	1.0		
Length: steel jacket (cm)	-	50	75	30
Young's modulus: reinforcement (GPa)	200			
Compressive strength: concrete (MPa)	42.0	43.8	35.6	35.6
Young's modulus: concrete (GPa)	28.0			

Table 4 Comparison of maximum strength

Specimen No.	j-1	j-2	j-3
Experiment (kN)	124	128	126
Simulation (kN)	127	129	133

Each specimen is modeled in 2-D with 640 5 cm-side square elements. Plane stress condition is assumed. The strength of concrete and steel bars of the numerical models is same as the strength of real specimens. Although experimental loading conditions are adopted in the numerical analysis, the number of loading cycles per one loading step is one.

3.3.2 Results and considerations Table 4 shows the maximum load obtained experimentally and numerically. The numerical simulation results are 100 to 105% of the experiment results. Figure 6 shows the

Fig. 7 Main reinforcement axial strain (upper: experiment, lower: simulation)

observed failure modes and Fig. 7 shows the distribution of the main reinforcement axial strain obtained by experiment and simulation.

Specimen j-1 (Without jacketing): In the both experiment and simulation, the damage to the RC column concentrates at the cutoff height. The RC column reached its ultimate state for a displacement equal to $7 \delta_0$ in the experiment and $6 \delta_0$ in the simulation.

Fig. 8 Damage condition of JR Hansui viaduct

Specimen j-2 (Steel jacket length: 50 cm): The column center near the upper end of the steel jacket and the column bottom were damaged in both experiment and simulation. Finally, the bottom end damage became severe.

Specimen j-3 (Steel jacket length: 75 cm): In both experiment and simulation, the RC column damage concentrates at the column bottom. The RC column reached its ultimate state when the input displacement was $7\delta_0$ in the experiment and $6\delta_0$ in the simulation.

Specimen j-4 (Steel jacket length: 30 cm – numerical simulation only): A RC column strengthened with a 30 cm-long steel jacket was simulated in order to check the failure mode of a vulnerable column with inappropriate jacketing. In this case, the damage concentrated at the column section near the upper end of the steel jacket.

In the experiment, we confirmed that the damage to the Specimen j-1, which is not strengthened, concentrates at the column center where the main reinforcement is cut and the region where damage concentrates shifts to the lower part of the column as the steel jacket length increases.

The results of the numerical simulation by AEM showed the same phenomenon that occurred in the experiments. The maximum strength and ultimate displacement were simulated with sufficient accuracy. Moreover, the simulated failure mode results of the Specimen j-4 suggest that numerical simulation by AEM may be effectively utilizable as the detection method of inadequate earthquake strengthening design.

3.4 Simulation of collapse behavior of real viaduct

Numerical simulation of the seismic response of the Japan Railways (JR) Hansui viaduct, which collapsed during to the 1995 Kobe Earthquake, is performed. The damage condition⁽⁴⁾ is shown in Fig. 8. The structure is a 3-span double-decked viaduct with a typical section as shown in Fig. 9. 18 cm-side square elements are used to model the viaduct. The number of distributed springs between each two adjacent sides is 10. The material properties of concrete and steel bars are defined by considering the actual strength as shown in Table 5. The arrangement

Fig. 9 Viaduct layout

Table 5 Material properties of viaduct model

Concrete		Reinforcing bar	
Young's modulus (GPa)	28.0	Young's modulus (GPa)	210
Compressive strength (MPa)	32.0	Yield stress (MPa)	350
Tensile strength (MPa)	2.40		

of reinforcing bars is the same as the real arrangement shown in Fig. 9. The natural frequency of the numerical model considering the effect of the soil-pile system is almost the same as the real natural frequency of the undamaged JR Hansui viaduct. The collapse behavior of the model due to the NS component of the JR Takatori ground motion⁽⁵⁾ is shown in Fig. 10. The damage condition of the numerical model is almost the same as that observed.

4. Simulation of the Change of Natural Frequency Due to Damage

4.1 Applicability of vibration measurements for damage inspection

In the field of health monitoring of railway structures, inspection techniques using the dynamic characteristics of structures have been developed. Such methods make use of the natural frequency of structures as the index of damage and/or deterioration levels. The vibrations induced by sources such as a passing trains or cars, impact on the structure and microtremors, is used in order to get the natural frequency of structure. The degree of damage to a RC structure, especially a jacketed structure, cannot be fully ascertained by visual inspection. In order to assess the

Fig. 10 Collapse behavior of numerical model due to ground motion of JR takatori site

Fig. 11 Specimen used in the experiment

degree of damage to a RC structure using the inspection techniques based on vibration measurements, it is necessary to identify the changes of the dynamic characteristics associated with structural damage in advance. Therefore, the accuracy of the AEM to simulate the natural frequency of damaged structures is investigated through the comparison of the experimental results and numerical simulations.

4.2 Simulation of natural frequency of a damaged RC column

Experiments to ascertain the change of natural frequencies of RC columns due to damage were carried out⁽⁶⁾. The specimen used in the experiment, which is a scale model of railway viaduct column, is shown in Fig. 11. Cyclic loading was applied gradually to cause damage to the specimen. Impact vibration tests, in order to obtain a specimen natural frequency, were performed at each damage level. The specimen material properties that were obtained through material tests are shown in Table 6. The specimen was modeled with 5 cm-side square elements. The number of distributed springs between two adjacent elements was 10. In case of the experiment, 1 loading step was composed of 3 cycles, and the impact vibration test was carried out after each step. The upper graph of Fig. 12 depicts the inputted displacement and the lower graph shows the natural frequency changes for both experiment and simulation.

In case of the numerical simulation, the results of the case in which 1cycle/step is considered are shown in

Table 6 Material properties of specimen

Concrete	Column	Footing
Young's modulus (GPa)	21.9	22.1
Compressive strength (MPa)	28.5	28.7
Tensile strength (MPa)	2.25	2.77
Reinforcing bar	D22	D10
Young's modulus (GPa)	193	181
Yield stress (MPa)	421	385

Fig. 12 Input displacement (upper) and change of natural frequency due to damage (lower)

Fig. 13 Viaducts and loading equipments

Fig. 12 because there are not major differences between 1cycle/step and 3cycles/step cases. The simulated natural frequencies of 1/4, 1/2 and 3/4 cycles are also shown in Fig. 12. The simulated results agree well with the experimental results.

4.3 Simulation of the natural frequency of a damaged RC viaduct reinforced by steel jacket

4.3.1 Outline of experiment and numerical simulation Naganawa et al.⁽⁷⁾ conducted the loading experiment of an actual viaduct reinforced by steel jacket.

Table 7 Combination of material properties

Case	①	②	③	④	⑤	⑥
Concrete	C1	C1	C2	C2	C3	C3
Longitudinal bars	S1	S2	S1	S2	S1	S2

Fig. 14 Arrangement of reinforcing bars and cross section of the jacketed column

Fig. 15 Jacketed column numerical model

Daiichi Shinagawa viaduct R13, which was removed due to the opening of Shin-Shinagawa station, was used in the experiment. The viaduct structure is a 3-span, single lane, RC rigid frame supported by 2 columns. The typical section of the viaduct and the loading equipment are shown in Fig. 13. Each column was reinforced with a 6-mm thick steel jacket as shown in Fig. 14. The gap between the column and the steel jacket was 30 mm, and it was filled with shrinkage-compensating mortar. Walls were installed in the frames adjacent to R13, namely R12 and R14, to act as reaction walls. The slabs between viaducts were cut and the loading jacks were installed there. The cyclic loading test along the direction parallel to the track was carried out by displacement control. After applying cyclic loadings with maximum displacements of ± 15 mm, ± 30 mm, ± 60 mm, ± 90 mm, ± 120 mm, ± 150 mm, ± 180 mm, ± 210 mm, and ± 240 mm, a monotonic loading of $+350$ mm was imposed as the last step. The impact vibration test along the direction parallel to

Table 8 Material properties of numerical model

	Compressive strength	Young's modulus		Yield stress	Young's modulus
C1	23.5 MPa	24.5 GPa	S1	490 MPa	200 GPa
C2	35.0 MPa	28.0 GPa	S2	558 MPa	200 GPa
C3	17.6 MPa	21.6 GPa	Stirrup	400 MPa	200 GPa

Fig. 16 Change of natural frequency of the real viaduct and the numerical models

the track was carried out after each loading step in order to investigate the natural frequency changes due to the structural damage.

The viaduct column was modeled using AEM considering 8.25 cm-side square elements as shown in Fig. 15. It was assumed that the top of the column was restrained against rotation because the viaduct beam was very rigid. The behavior of the whole viaduct was represented with one column carrying one eighth of the total mass of the slab and beams. The design compressive strength of the concrete of the viaduct was 23.5 MPa, and the compression test results, which were obtained at the time of construction, gave a compressive strength of 32.9 MPa. SD49 and SS41 were used for the longitudinal reinforcing bars and stirrups, respectively.

At the first stage of the analysis, the properties of the material of the viaduct were not fixed. Six models with different material properties, as shown in Table 7, were created by combining the material properties of three types of concrete and two types of longitudinal reinforcing bar as shown in Table 8. In addition, by arranging a soil-foundation spring at the bottom of the model, the natural frequency of the column was adjusted so that it corresponded to the measured natural frequency of the ac-

tual viaduct. As a result, the natural frequencies of all the models were equal in the initial state. The changes of the natural frequency of each model due to the structural damage were analyzed and compared with the experimental results.

4.3.2 Results and considerations The experimental and numerical results are shown in Fig. 16. According to the experimental results, the natural frequency reduced to 85% of the original value after the ± 30 mm loading. After the ± 120 mm loading, the natural frequency reduced to 50% and remained almost constant after that. In the simulation result by AEM, the natural frequency reduced to 79–83% after the ± 30 mm loading, to 55–61% after the ± 120 mm loading, to 50–54% after the ± 240 mm loading, and to 47–51% after the ± 350 mm loading. In the numerical simulation results, the difference in the material properties of each model did not have much influence on the natural-frequency change.

5. Concluding Remarks

The AEM could simulate the damage behavior of the RC column, the jacketed RC column and the real railway viaduct. The changes in natural frequencies due to damage to the RC column and the real railway viaduct with steel jacket were also simulated correctly. Through the above-mentioned studies, we confirm that the AEM can function as a high-performance seismic damage simulator for RC structures and retrofitted RC structures. Seismic performance check of existing pre code-revision structures, selection of the most effective retrofitting method, and development of assessment criteria for damage inspection can be effectively done by AEM. These results are very useful for the assessment and mitigation of railway seismic damage.

References

- (1) Meguro, K. and Tagel-Din, H., A New Efficient Technique for Fracture Analysis of Structures, Bulletin of Earthquake Resistant Structure Research Center, IIS, No.30, (1997), pp.103–116, Univ. of Tokyo.
- (2) Uehara, F. and Meguro, K., Vulnerability Assessment of Jacketed Viaduct Using Microtremor Measurement & Numerical Simulation, Proceedings of 12th World Conference of Earthquake Engineering (CD-ROM), (2000).
- (3) Kawashima, K., Unjoh, A. and Iida, H., Seismic Inspection and Seismic Strengthening Methods of Reinforced Concrete Bridge Piers at Mid-Height Where Main Reinforcements Are Terminated with Inadequate Anchorage Length, Report of Public Works Research Institute Ministry of Construction, (in Japanese), Vol.189 (1993).
- (4) Editorial Committee for the Report on the Hanshin-Awaji Earthquake Disaster, Report on the Hanshin-Awaji Earthquake Disaster, Damage to Civil Engineering Structures, Bridge Structure, (1996), Maruzen.
- (5) Nakamura, Y., Uehara, F. and Inoue, H., Waveform and Its Analysis of the 1995 Hyogo-Ken-Nanbu Earthquake II, JR Earthquake Information, No.23d, RTRI, (1996), pp.9–16.
- (6) Shimono, H., Watanabe, T. and Satou, T., A Consideration on the Damage Evaluation of RC Members, Proceedings of the 52th Annual Conference of the Japan Society of Civil Engineering I-A, (in Japanese), (1997), pp.402–403.
- (7) Naganawa, T., Tanma, Y., Yoshida, K., Kaji, K. and Nakano, S., Experimental Study on Real RC Frame Viaduct Reinforced by Steel (Vol.2)—Study on Soundness Diagnosis of RC Frame Viaduct Reinforced by Steel after Earthquake—, Proceedings of 55th Annual Conference of JSCE, (in Japanese), V-501, (2000), pp.1004–1005.