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EARTHQUAKE SIMULATION TESTS OF BRIDGE COLUMN MODELS DAMAGED DURING 1995 KOBE EARTHQUAKE

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ABSTRACT :

A series of shake table tests of two reinforced concrete bridge column models was conducted. One model is designed to be failed in flexural at the bottom of the column (flexural failure model), and the other is designed to be failed in shear at the cut-off point of the longitudinal reinforcement (shear failure model); both are models of bridge columns that were severely damaged during the 1995 Hyogo-ken Nanbu (Kobe), Japan, earthquake. The models are one-third scaled ones of full-scaled reinforced concrete bridge column models that are planed to be tested on the E-Defense in 2007 and 2008. Failure mechanisms under strong ground excitation and three dimensional dynamic behaviors of these columns were investigated. The flexural failure column was failed around the bottom of the column. Spalling of cover concrete, buckling of longitudinal reinforcement, and crush of core concrete were observed after the test. The shear failure column was eventually failed in shear. The flexural cracks were first observed around the cut-off point of the longitudinal reinforcement, and then shear diagonal cracks developed, resulting in destructive shear failure. Both final failure modes are typical ones observed during the 1995 Kobe earthquake.

KEYWORDS: bridge, reinforced concrete column, shake table test, flexural failure, shear failure

1. INTRODUCTION

The 1995 Hyogo-ken Nanbu, Japan, earthquake (Kobe earthquake) caused destructive damage to bridges. A research program on bridge structures using the world largest shake table, E-Defense, has been conducted since 2005 (Kawashima et al. 2008), which includes a series of shake table tests of full-scaled reinforced concrete bridge columns. The program are expected to provide valuable information on the failure mechanisms of reinforced concrete bridge columns that were severely damaged during the 1995 Kobe earthquake as well as on the effect of specimen size on the evaluation of the seismic performance and dynamic failure mechanisms, and on the development of advanced analytical models.

As a preliminary study of the full-scaled shake table tests of reinforced bridge columns, a series of shake table tests of two small-scaled reinforced concrete bridge column models was conducted (1) to provide experimental data of small-scaled models to investigate the effect of specimen size, and (2) to conduct preliminary research on the dynamic failure mechanism of reinforced concrete bridge columns that were damaged during the 1995 Kobe earthquake.

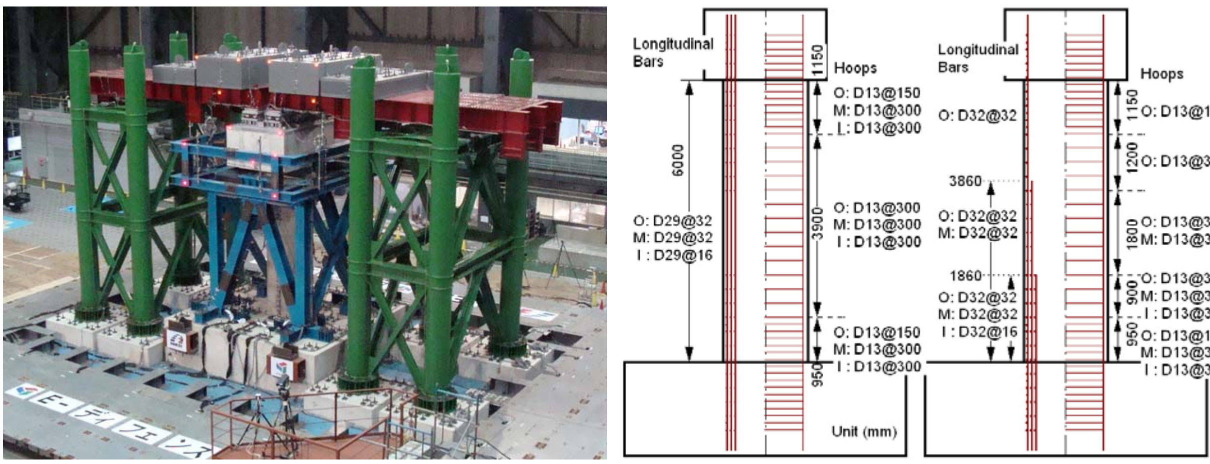
2. TEST SETUP AND SPECIMENS

2.1. Overview of Shake Table Test of Full-Scaled Reinforced Concrete Bridge Columns

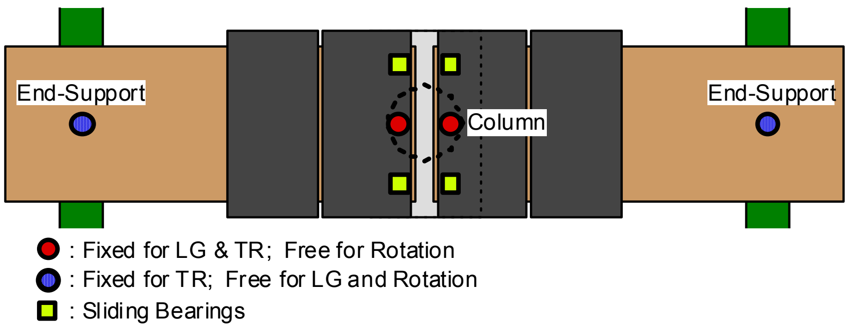
Seven specimens are planed to be shaken on the E-Defense in the research program for bridge structures (Kawashima et al. 2007). Among those, two specimens are the models that were severely damaged during the 1995 Kobe earthquake, and these two are the specimens that this project focuses on. One is designed to be failed in flexural at the bottom of the column (flexural failure model) and the test has been conducted in 2007 (Kawashima et al. 2008), and the other is designed to be failed in shear at the cut-off point of the longitudinal reinforcement (shear failure model), which is to be tested in 2008.

Figure 1 shows the test setup, reinforcement details of the specimens and bearing conditions. Two girders, which steel blocks as weights of a superstructure are fixed to, are supported by fixed bearings on the top of the specimen and a longitudinal-movable/ transverse-fixed bearing at each end. Two sliding bearing are placed on each side of the fixed bearing on the specimen to prevent overturning of the decks. Due to this setup, the inertia force of the girder-block assembly is applied to the top of the specimen (7.5 m from the bottom of the column) in the longitudinal direction, and to the center of the gravity of the girder-block assembly (9.14 m from the bottom) in the transverse direction. The inertia mass of each girder-block assembly excluding mass of the load cells and bearings on the specimen is 145 ton. Since all the inertia force is applied to the specimen due to the boundary condition, the inertia mass in the longitudinal direction is 289 ton. In the transverse direction, on the other hand, not only the specimen but also the end supports carry the inertia force. Thus, the inertia mass in the transverse direction is 199 ton.

The diameter of the column is 1.8 m, and the height of the column is 7.5 m. 80 of SD345, 29-mm-diameter deformed bars are arranged in the flexural failure model as the longitudinal reinforcement without cut-off. On the other hand, two cut-off points are designed for the shear failure model. The inner rebar is anchored at 1.95 m from the bottom of the column, and the middle rebar is anchored at 3.95 m. At the bottom, 80 of SD345, 32-mm-diameter deformed bar are arranged. As transverse reinforcement, SD345, 13-mm-diameter deformed bars are provided for both the columns. The pitch of the transverse hoops is set at 300 mm excluding the both ends of the column. In the region of about 1 m from the bottom and the top of the column, the outer transverse reinforcement is arranged at 150-mm-pitch. The transverse reinforcement is anchored with lap splices, and the anchorage length is 300 mm.



(a) Test setup (b) Flexural failure model (c) Shear failure model



(d) Boundary conditions of girder-block assembly

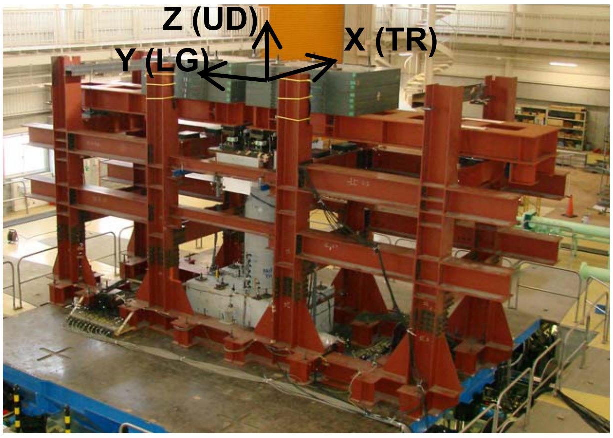
Figure 1 Test setup and specimens of full-scaled bridge column models

The design concrete strength is set to be 27 N/mm^2. The design yield strength of SD345 bars is 345 N/mm^2.

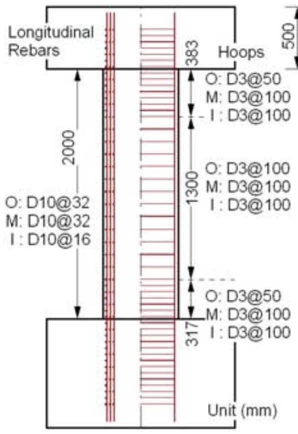
2.2. Test Setup and Specimens for 1/3-Scaled Models

Figure 2 shows the test setup and specimens of one-third scaled models. The specimens are designed considering the similitude requirements (Krawinkler and Moncarz, 1982). Because the plan, specimen details, setup, etc, of the full-scaled models had been minor changed since the test plan of the small-scaled test had been decided, however, some conditions such as the design material strength, the transverse reinforcement ratio, etc, do not satisfy the similitude requirements.

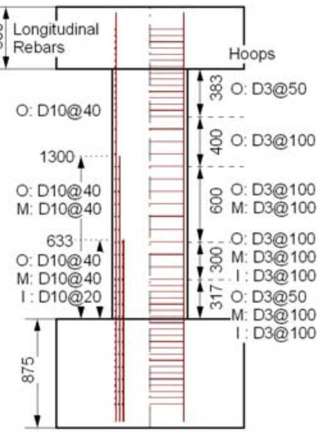
A similar setup and the same bearing conditions with the full-scaled model test were used. In this test program, the longitudinal and transverse directions are defined as the Y and X directions, and the sides face the X positive and the X negative directions are defined as the Xp and Xn faces, respectively. Likewise, the Yp and Yn faces are defined. Due to the setup, the points inertia force applied to are 2.5 m and 3.65 m from the bottom of the column for the longitudinal (Y) and transverse (X) directions, respectively. Inertia mass in the longitudinal and transverse directions are 37.8 ton and 26.6 ton, respectively.



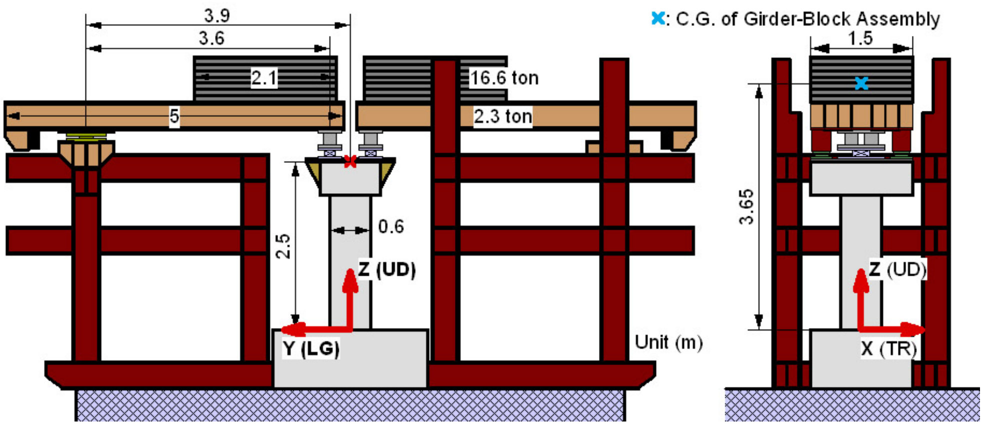
(a) Test setup



(b) Flexural failure specimen



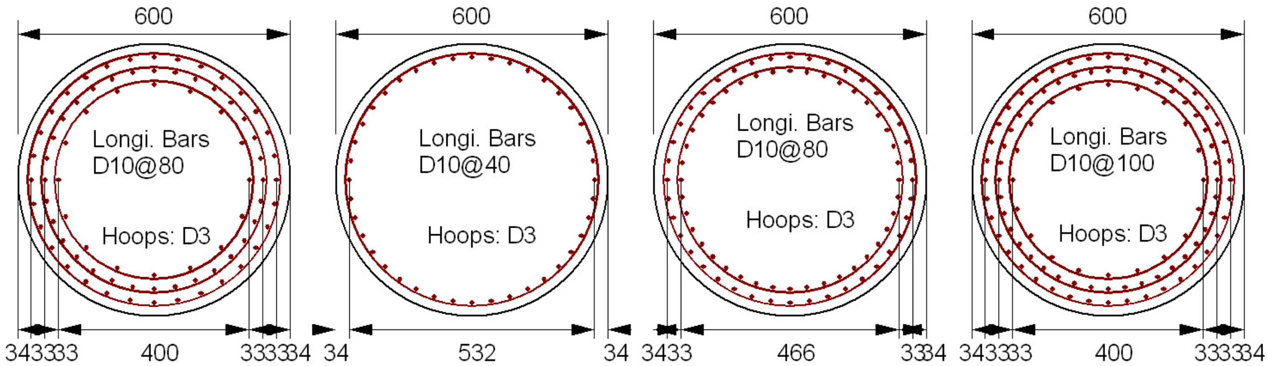
(c) Shear failure specimen



(d) Test setup and dimensions

Figure 2 Test setup and specimens of 1/3-scaled bridge column models

Figure 3 shows the cross sections. The diameter of the column is 0.6 m, and the height of the column is 2.5 m. 80 of SD295A, 10-mm-diameter deformed bars are arranged for the flexural failure specimen as the longitudinal reinforcement without cut-off. The longitudinal reinforcement ratio is 2.02%. On the other hand, two cut-off points are designed for the shear failure specimen. The inner rebar is anchored at 0.63 m from the bottom of the column, and the middle rebar is anchored at 1.3 m. At the bottom, 100 of SD345, 10-mm-diameter deformed bar are provided. The longitudinal reinforcement ratios at the bottom, lower cut-off point, and the upper cut-off point are 2.52%, 2.02% and 1.01%, respectively. As transverse reinforcement, SD295, 3-mm-diameter deformed bars are provided for both the specimens. The pitch of the transverse hoops is set at 100 mm excluding the both ends of the column. In the region of about 0.3 m from the bottom and the top of the column, the outer transverse reinforcement is arranged at 50-mm-pitch. The transverse reinforcement ratio at the bottom of the column is 0.1%. The transverse reinforcement is anchored with lap splices, and the anchorage length is 100 mm. The design concrete strength is 27 N/mm^2. Table 1 shows the material properties, which were obtained from the material tests.



(a) Upper cut-off point (b) Lower cut-off point (c) bottom

(1) Flexural failure specimen (2) Shear failure specimen

Figure 3 Cross sections of 1/3-scaled bridge column models

Table 1 Material Properties

(a) Concrete

	f_{c0} (N/mm^2)	E_c (kN/mm^2)
Flexural Failure Specimen	27.9	28.8
Shear Failure Specimen	28.8	26.5

(b) Reinforcing steel

	f_{sy} (N/mm^2)	E_s (kN/mm^2)
Longitudinal bars (SD295A)	351.4	178.3
Longitudinal bars (SD345)	374.2	179.8
Hoops	280.4	212.9

The ductility performance, flexural strength and shear strength were evaluated based on the JRA specifications (JRA, 2002) using the actual material properties. The crack, yield and ultimate displacements at the top of the column of the flexural failure specimen are 0.7 mm, 10 mm, and 28 mm in the longitudinal direction. The flexural strengths in the longitudinal and transverse directions are 188 kN and 129 kN, respectively.

Figure 4 compares the flexural strength and shear strength along the column height of the shear failure specimen. Flexural strength at each section was computed based on the moment-curvature analysis with consideration of cut-off of the longitudinal reinforcement. The anchorage length of the longitudinal bars is assumed to be about 0.21 m according to the JSCE standard specification (JSCE 2002).

The shear strength P_s was computed based on the following equation according to the JRA specifications:

$$P_s = S_c + S_s \tag{2.1}$$

where

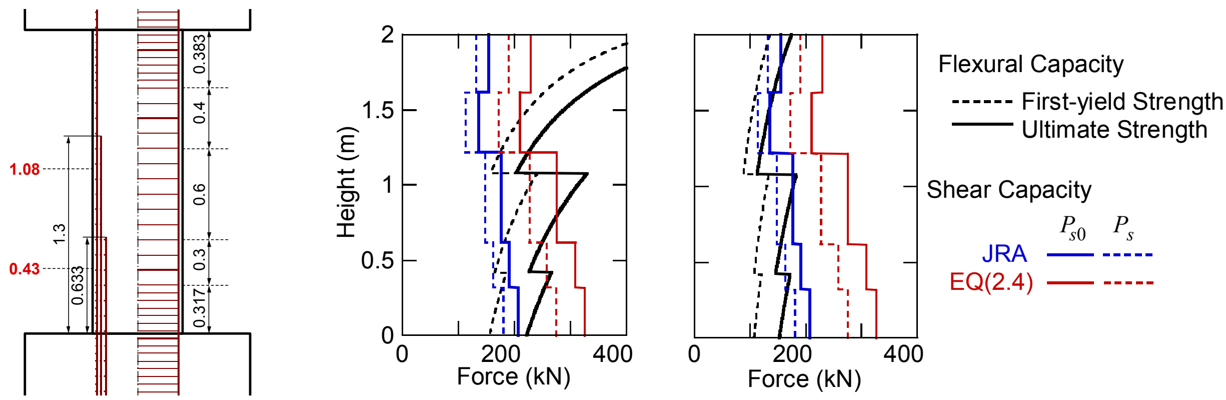
$$S_c = c_c c_e c_{pt} \tau_c b d \quad ; \quad S_s = \frac{A_w \sigma_{sy} d(\sin\theta + \cos\theta)}{1.15a} \qquad (2.2); (2.3)$$

where S_c is the concrete shear capacity, S_s is the shear strength provided by reinforcing steel, τ_c is the averaged shear stress, coefficients, c_c, c_e, c_{pt} are the modification factors on the effects of cyclic loading, the effective height and the axial tensile reinforcement ratio, respectively, b is the width of cross section, d is the effective height, A_w is the sectional area of hoop ties, σ_{sy} is the yield strength of hoop ties, θ is the angle between hoop ties and the vertical axis, and a is the spacing of hoop ties. P_{s0} is the shear strength without consideration of shear strength deterioration due to the cyclic loading effect. To evaluate the actual shear strengths of the specimen, τ_c was determined from the following equation (Kawano et al. 1996), which was proposed as mean value of experimental data.

$$\tau_c = 0.72 \times d^{-0.33} \times \left(\frac{24}{f_{c0}}\right)^{-1/3} \times \left(\frac{1.2}{\rho_l}\right)^{-1/3} \tag{2.4}$$

where d is the effective depth (m), f_{c0} is the concrete strength (N/mm^2), and ρ_l is the longitudinal reinforcement ratio (%). τ_c is determined to be 0.731 N/mm^2 for the critical section, where is the region of 1.2 m to 1.6 m from the bottom. In Figure 4, the shear strength of the specimen obtained based on the averaged shear stress (= 0.366 N/mm^2; $c_e c_{pt} \tau_c$= 0.446 N/mm^2) according to the JRA specification is also shown. The averaged shear stress is determined as the values that is equal to the mean minus twice of the standard deviation of Eq. (2.4) because of safety consideration for the design.

To predict the actual failure mode of the shear failure column, the shear strength of the column based on Eq. (2.4) was considered. According to Figure 4, flexural yielding first occurs at the bottom and the upper cut-off point because the flexural first-yield strengths are the smallest. The smallest shear strength along the column is 210 kN as P_{s0} and 172 kN as P_s in the region of 1.2 m to 1.6 m from the bottom. This P_{s0} is 35% larger than the flexural first-yield strength at the bottom and the upper cut-off point and 3% larger than the flexural ultimate strength of these points. If the deterioration of the shear strength due to cyclic flexural damage is considered, the shear strength decreases and becomes smaller than the flexural ultimate strength. Thus, the shear failure specimen was expected to be damaged in flexure around the upper cut-off point, and then shear failure to occur around the upper cut-off point.



(a) Reinforcement details (b) Strength in LG direction (c) Strength in TR direction

Figure 4 Evaluated failure mode of shear failure column

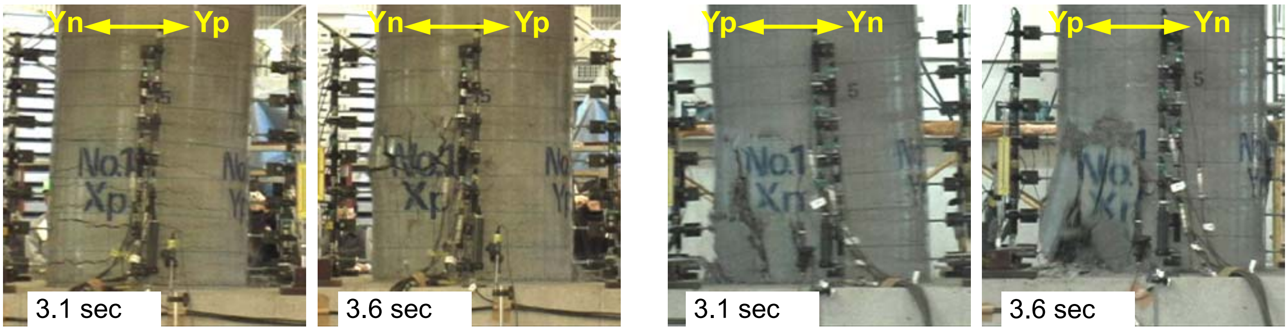
2.3. Input Ground Motions

The specimens were tested under three dimensional ground motions. Ground accelerations measured at JR Takatori station during the 1995 Kobe earthquake (Nakamura, 1995) were selected for this study and the NS, EW and UD components were inputted in the longitudinal (Y), transverse (X), and vertical (Z) directions, respectively. The tests had two phases; one is for dynamic response in elastic range, and the other is for that in nonlinear range. The amplitude of the ground motions were scaled by 10% and 80% for the tests, respectively, based on the preliminary analytical studies. The time of the ground motions was scaled using a time scale factor equal to 0.6 ($\approx \sqrt{3}$) considering the similitude requirements.

3. FAILURE AND RESPONSE OF FLEXURAL FAILURE SPECIMEN

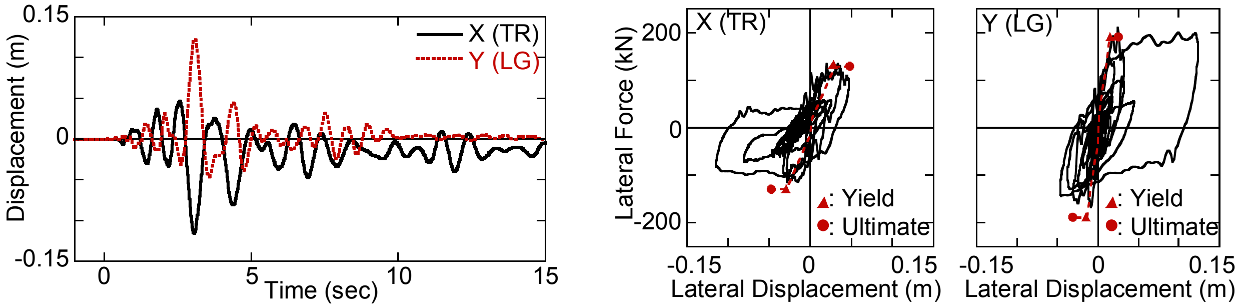
The fundamental natural periods prior to the test were 0.25 seconds and 0.38 seconds for the longitudinal and transverse directions, respectively. The maximum response displacements in the longitudinal and transverse directions were 2.6 mm and 1.6 mm. No flexural cracks were observed after the elastic level test.

Figure 5 shows the damage progress of the specimen during the nonlinear level test, and Figure 6 shows response displacement and lateral force versus lateral displacement hystereses during the nonlinear level test. Several flexural cracks were observed up to 3 seconds around the bottom of the column, when the response displacement was about the computed ultimate displacement. The maximum response displacement occurred at 3.1 seconds, which were 123 mm and 115 mm in the longitudinal and transverse directions. As a distance from the origin, the maximum response was 168 mm, which was 6 times larger than the ultimate displacement of the specimen. Spalling of cover concrete and buckling of longitudinal reinforcing bars were observed. The lateral forces observed were smaller to the values that the current design specification predicts, indicating the bilateral flexural loading effects.



(a) Xp face (b) Xn face

Figure 5 Damage progress of flexural failure specimen



(a) Response displacement (b) Lateral force-displacement hystereses

Figure 6 Response of flexural failure specimen at top of column

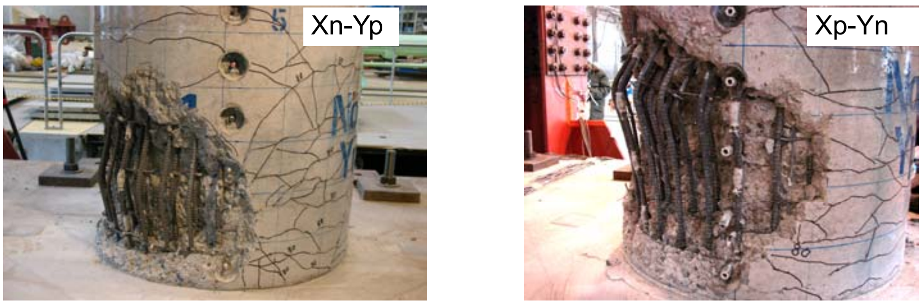


Figure 7 Failure mode of flexural failure specimen

As shown in Figure 7, flexural damage occurred in the region of 0.2 m ~ 0.4 m from the bottom of the column, and longitudinal reinforcing bars in the middle and outer layers were buckled at the Xn-Yp face where was the compression region when the maximum displacement occurred at 3.1 seconds. On the other hand, damage occurred in the region of 0.2 m ~ 0.45 m from the bottom at the Xp-Yn face. Reinforcing bars in all three layers were buckled. The pitch of the transverse reinforcement changes at 0.317 m from the bottom, resulting in the rebar buckling in the Xp-Yn face. The failure mode is similar to one of the typical failure modes observed during the 1995 Kobe earthquake.

4. FAILURE AND RESPONSE OF SHEAR FAILURE SPECIMEN

The fundamental natural periods prior to the test were 0.27 seconds and 0.38 seconds for the longitudinal and transverse directions, respectively. The maximum response displacements in the longitudinal and transverse directions were 2.6 mm and 1.6 mm. Minor flexural cracks were observed after the elastic level test around the lower cut-off point.

Figures 8 and 9 show damage progress, response displacement and lateral force versus lateral displacement hystereses of the shear failure specimen during the nonlinear level test. In Figure 9, the response after 3.1 seconds shows dotted lines because the top slab of the column contacted to the safety frame and thus the results included this effect after 3.1 seconds. At about 2 seconds of the nonlinear level test, flexural cracks were observed at 1.2 m from the bottom, and then slight shear cracks initiated around the same location. The lateral displacement at this point was about 40 mm. When the lateral displacement increased up to 175 mm from 2.8 to 3.1 seconds, first flexural damage occurred around 1.4 m from the bottom, which was around the upper cut-off point, and then the shear cracks extended from 1.4 m to 0.6 m, causing the destructive shear failure, which was one of the major causes of the destructive damage during the 1995 Kobe earthquake.

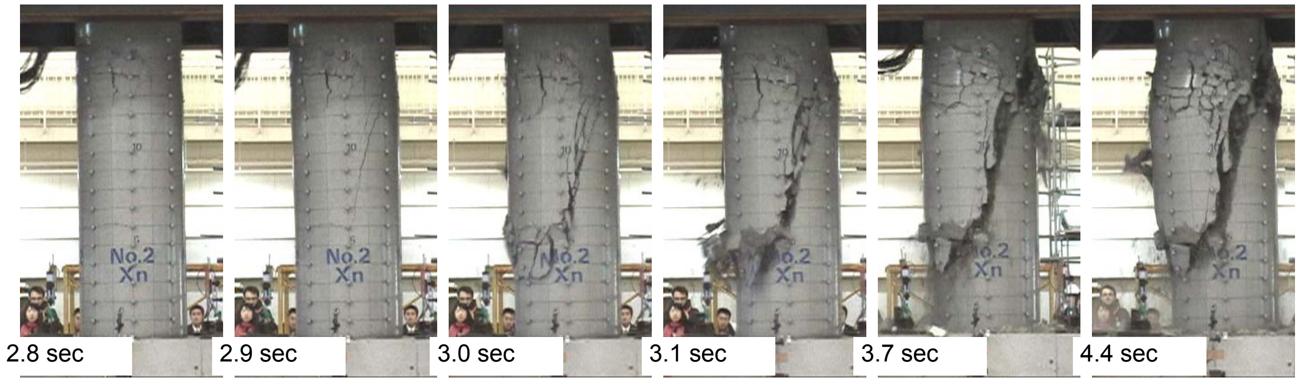
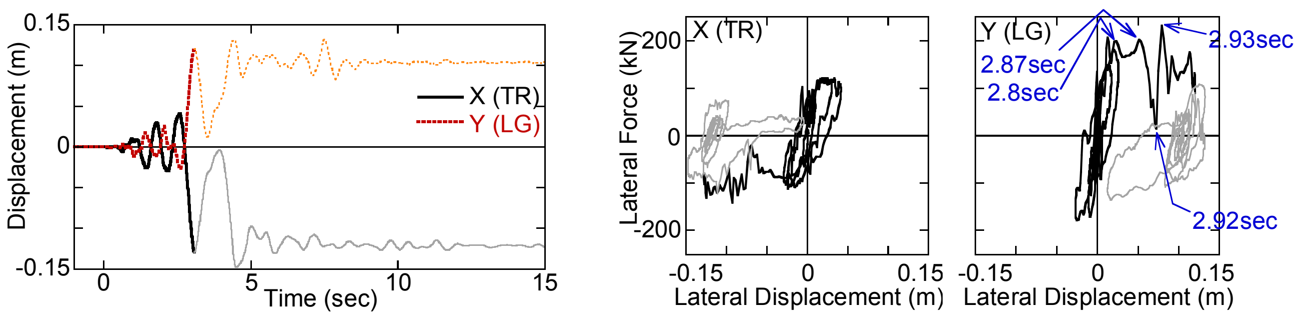


Figure 8 Damage progress of shear failure specimen



(a) Response displacement (b) Lateral force-displacement hystereses

Figure 9 Response of shear failure specimen at top of column

The lateral force in the longitudinal direction reached 200 kN at 2.87 seconds, and then the force decreased to 16 kN due to the shear failure. Thus, the shear capacity of the specimen was estimated to be 200 kN. Because the P_{s0} computed using Eq (2.4) for the averaged stress of concrete was 221 kN, the shear capacity obtained from the test was about 90% of the computed shear capacity.

5. CONCLUSIONS

A series of shake table tests was conducted for one-third scaled models of the full-scaled reinforced concrete bridge column models to provide experimental data of small-scaled specimens to investigate the effect of specimen size, and to conduct preliminary research on the dynamic failure mechanism and response of reinforced concrete bridge columns that were damaged during the 1995 Kobe earthquake. Below are the conclusions determined from the study:

1. The damage progress of the typical failure modes, which are the flexure failure at the bottom of the column and shear failure after flexural damage around the cut-off point of longitudinal reinforcement, were simulated on a shake table.
2. The current design specification provides a good estimation of the flexural capacity of the flexural failure specimen. Only flexural cracks were observed at the computed ultimate displacement, and spalling of cover concrete, buckling of longitudinal reinforcement were observed when the lateral displacement exceeded six times of the computed ultimate displacement.
3. The current design specification estimated about 60% of the observed shear capacity because of the safety consideration in the specification. If the averaged shear stress is obtained from Eq.(2.4), the shear capacity was approximately predicted. The failure mode was also predicted by comparison of the flexural capacity and the shear capacity; the flexural damage around the upper cut-off point triggered the shear failure.
4. Experimental data of small scaled specimens is provided to investigate the specimen size effect on the dynamic failure mechanism and nonlinear response.

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