SEISMIC PERFORMANCE OF RC BRIDGE COLUMNS WITH TERMINATION OF MAIN REINFORCEMENT WITH INADEQUATE DEVELOPMENT

T. Sasaki[1] , H. Kurita[1] and K. Kawashima[2]

[1] *Graduate Student, Dept. of Civil Engineering, Tokyo Institute of Technology, Japan*
[2] *Professor, Dept. of Civil Engineering, Tokyo Institute of Technology, Japan*
Email: sasaki.t.aa@m.titech.ac.jp, kurita.h.aa@m.titech.ac.jp, kawashima.k.ae@m.titech.ac.jp

ABSTRACT:

Shear failure of RC columns due to insufficient termination of main reinforcements resulted in extensive damage in bridges. Failure mechanism of a column with termination at two heights is clarified to identify the loading protocol dependence of the shear failure mode.

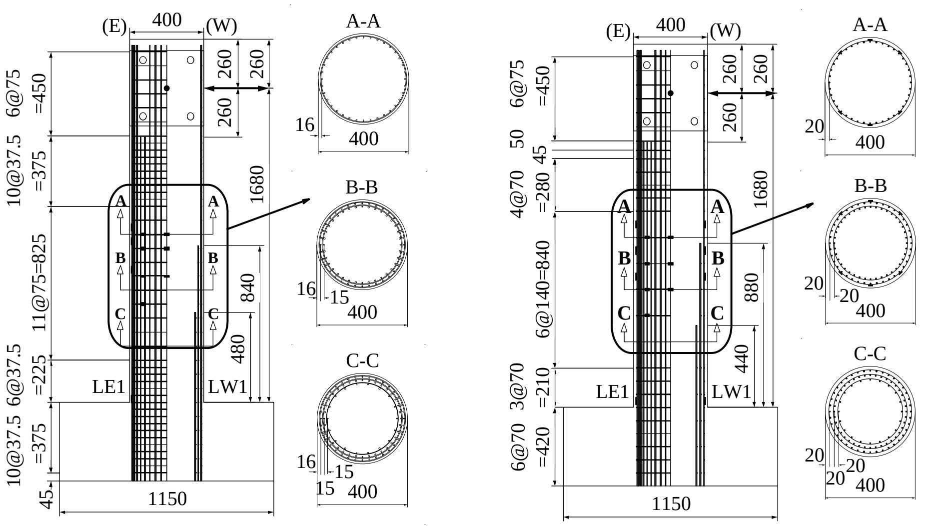
KEYWORDS: bridge, seismic damage, shear failure, loading experiment

1. INTRODUCTION

Shear failure of RC columns which were designed based on the pre-1980s design codes resulted in extensive damage in bridges. One of the most destructive failures occurred in 1995 Kobe, Japan earthquake when a number of bridges collapsed. This type of damage still continues to occur. It was a common practice prior to the 1980s to reduce number of longitudinal reinforcements as sectional force demand decreases by simply anchoring longitudinal reinforcements in a tension zone. Development was insufficient at the terminations of longitudinal reinforcement (cut-off). There were other deficiencies in design including overestimation of the shear capacity and lack of consideration on the lateral confinement.

Extensive research has been directed toward the mechanism and evaluation of the shear failure of RC columns associated with bar termination at one or two heights with insufficient development. For example, Ishibashi et al developed an evaluation on the shear failure based on the shear and moment capacity ratio at the termination zone [Yamamoto, Ishibashi, Otsubo and Kobayashi 1984]. Kawashima et al proposed to evaluate shear failure at the termination zone based on the shear demand and shear capacity [Kawashima, Unjoh and Iida 1993]. Ikeda et al identified why failure did/ did not occur during the Kobe earthquake in several prototype columns in which longitudinal reinforcements were terminated at two locations [Ikehata, Adachi, Yamahuchi and Ikeda 2001]. Because small diameter bars were used in the early days, it was often that longitudinal reinforcements were set in three layers at the plastic hinge by terminating the inner layer bars first and then the center layer bars next. Thus longitudinal reinforcements are terminated at two locations (heights). Ikeda et al found that the shear and flexure strength ratio should not be too small for developing a shear failure extending from the upper termination zone to the lower termination zone; otherwise a shear failure occurs only near the upper termination zone.

The authors further extended the study of Ikeda et al and found that shear failure mode changes depending on whether the column response is cyclic with the response amplitudes almost the same in the positive and negative directions or predominant in one dimension. This paper presents a series of experiments conducted for clarifying the shear failure mechanism of RC columns with termination at two locations.



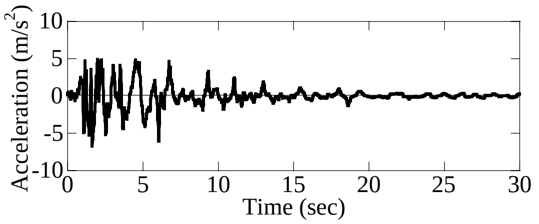
(a) Series A (b) Series B

Figure 1 Test Specimens

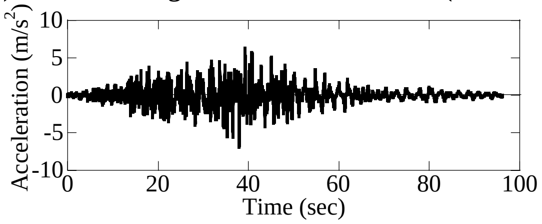
2. MODELS AND LOADINGS

Five circular column models with a shear and flexure strength ratio at the upper termination of 0.84 and three circular column models with a shear and flexure strength ratio at the upper termination of 0.78 were constructed as shown in Fig. 1. They are denoted here as A series and B series, respectively. All the eight models are 1.68 m tall with a 400 mm diameter. Five models in A series have essentially the same properties with the models used by Ikeda et al [Ikehata, Adachi, Yamahuchi and Ikeda 2001]. They are 1/7 scaled models of a prototype column which failed in shear during 1995 Kobe earthquake. Longitudinal bars with a 6 mm diameter were set in three layers: 36 bars in the outer and center layers and 18 bars in the inner layers. The 18 inner bars and 36 center bars were terminated at 480 mm and 840 mm from the base, respectively, with the 36 outer bars being extended to the top without termination. Tie bars with 3 mm diameter were provided at every 75 mm interval in the outer, center and inner layers, however they were provided at every 37.5 mm only above 1,050 mm and below 225 mm in the outer layer. Longitudinal reinforcement ratio and the volumetric tie reinforcement ratio at the upper termination are 0.9% and 0.11%, respectively, in the five models in A series.

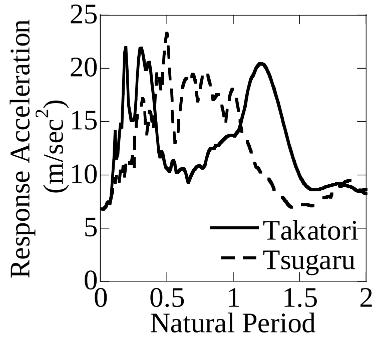
On the other hand, three models in B series are 1/4.5 scaled models of a 1.8 m diameter column model which is scheduled to be excited using E-Defense (C1-2 model) as a part of the large scale shake table experimental program of National Research Institute for Earth Science and Disaster Prevention [Nakashima et al 2008]. Forty, forty and twenty 6 mm diameter deformed longitudinal bars were set in the outer, center and inner layers, respectively. Tie bars with 3mm diameter were provided at every 140 mm interval in the outer, center and inner layers, however they were provided at every 70 mm only above 1,050 mm and below 210 mm in the outer layer. Longitudinal reinforcement ratio and the volumetric tie reinforcement ratio at the upper termination are 1.0 % and 0.06 %, respectively, in the three models in B series. A flexure and shear strength ratios at the upper termination zone based on Japanese design code [JRA 2002] are 0.84 and 0.78 in A and B series, respectively.



(a) Near-field ground acceleration (Takatori record)



(b) Middle-field ground acceleration (Tsugaru record)



(c) Response acceleration

Figure 2 Ground Motions for Hybrid Loading Experiments

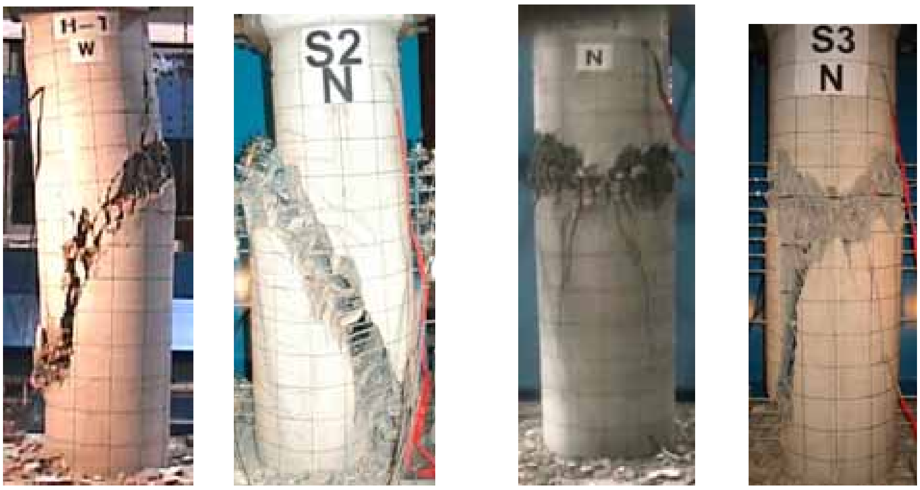
Five loading protocols were used for the A series; 1) pushover loading, 2) unilateral cyclic loading, 3) bilateral cyclic loading, 4) hybrid loading under a near-field ground acceleration and 5) hybrid loading under a middle-field ground acceleration. On the other hand, three loading protocols were use for B series; 1) pushover loading, 2) unilateral cyclic loading and 3) hybrid loading under a near-field ground acceleration. Since axial stress of a prototype pier at the plastic hinge due to the deck weight is 1.75 MPa for A series and 1.2 MPa for B series, the five models in A series and three models in B series were loaded under a constant vertical load of 220 kN and 145 kN, respectively, so that the same intensity of vertical stress is generated in the models.

In the pushover loading, A-1 model in A series and B-1 model in B series were loaded to failure under displacement control. In the cyclic loadings, A-2 and B-2 models were subjected to unilateral cyclic loading while A-3 model was subjected to bilateral cyclic loading. Loading displacement was stepwisely increased from 0.5 % drift (=8.4 mm) to failure with an increment of 0.5 % drift. The models were loaded three times at each loading displacement. A circular orbit was used in the bilateral cyclic loading.

In the hybrid loading, A-4 and B-3 models were subjected to a ground acceleration recorded near JR Takatori Station during 1995 Kobe, Japan earthquake, and A-5 model was subjected to a ground acceleration recorded near Tsugaru Bridge during 1983 Nihon-kai Chubu, Japan earthquake. The records near JR Takatori Station and Tsugaru Bridge are denoted hereinafter as Takatori record and Tsugaru record, respectively. Takatori record has a short duration with several long pulse accelerations while Tsugaru record has a long duration with many cycles of acceleration (refer to Fig. 2). They were selected as a typical near-field ground motion and a middle-field ground motion in this study. However because the intensity of Tsugaru record was small, its intensity was 2.5 times increased after scaling the acceleration to 15 % (=1/7) and 22.2 % (=1/4.5) the original intensity in the hybrid loading for A series and B series, respectively.

3. FAILURE MODE UNDER PUSHOVER AND CYCLIC LOADINGS

Fig. 3 shows damage of A1 and B1 under the pushover loading and A-2 and B-2 under cyclic loading. Under pushover loading, flexural cracks and diagonal shear cracks were first developed at the upper termination of main reinforcements in both A-1 and B-1. Subsequently a major



(a) A-1 (b) B-1 (a) A-2 (b) B-2

(1) Pushover loading (2) Cyclic loading

Figure 3 Damage after Pushover and Cyclic Loadings

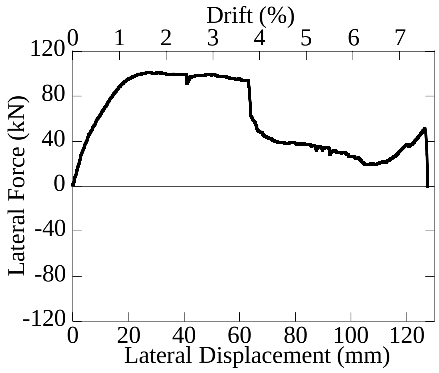
diagonal shear crack extended from the upper termination zone to the bottom of the column and A-1 and B-1 finally failed in shear. Because the concrete and main reinforcements in the compression zone did not suffer damage, the compression was directly transferred to the base by the compression strut action. This developed a major shear crack extended from the upper termination to the base.

On the other hand, under the cyclic loading, flexural cracks and diagonal shear cracks were first developed in A-2 at the upper termination zone at 1.5 % drift loading. Then 12 longitudinal reinforcements buckled at the right side surface (refer to Fig. 4(2a)) near the upper termination at 2.0 % drift loading. Once 12 bars buckled, the damage concentrated at only the upper termination zone without major extension of the diagonal shear cracks down to the base. Failure of covering and core concrete as well as local buckling of main reinforcements at the upper termination zone developed due to cyclic compression and tension progressed before major diagonal cracks extended from the upper termination to the base.

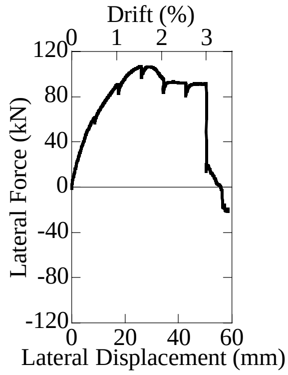
It is important to note that two columns with the same properties failed in different modes; A-1 failed in shear due to a diagonal shear crack extended from the upper termination to the base under the pushover loading while A-2 failed in shear on both surfaces after local buckling of main reinforcements under the cyclic loading.

In B-2, flexural cracks and diagonal shear cracks were first developed at the upper termination of main reinforcements at 1.5 % drift loading, and 13 longitudinal bars on the right surface buckled at 2.0 % drift near the upper termination. The failure up to this point is very close to the failure developed in A-2. However several major diagonal shear cracks were developed under loading in the other direction and one of them sharply extended to the base of the column with the diagonal shear cracks in the other direction being not seriously extended. Consequently the failure mode of B-2 is not the same with the failure mode of B-1 which simply failed in shear in one direction.

Fig. 4 shows the lateral force vs. lateral displacement hystereses of A-1 and B-1 under pushover loading and A-2 and B-2 under cyclic loadings. The peak lateral force capacity is nearly 100 kN in the four models at 1.5 % drift. The lateral force capacity starts to sharply deteriorate due to shear failure at 3.8 % drift in A-1 and 3.0 % drift in B-1 under the pushover loading. On the other hand,

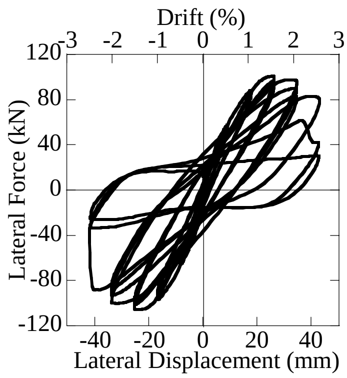


(a) A-1

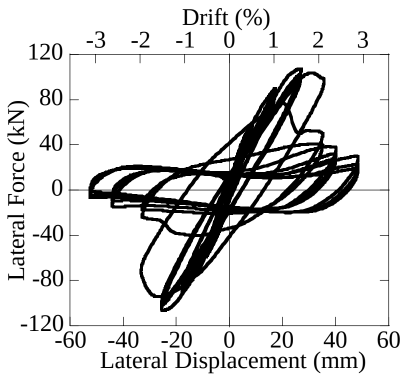


(b) B-1

(1) Pushover loading



(a) A-1



(b) B-1

(2) Cyclic loading

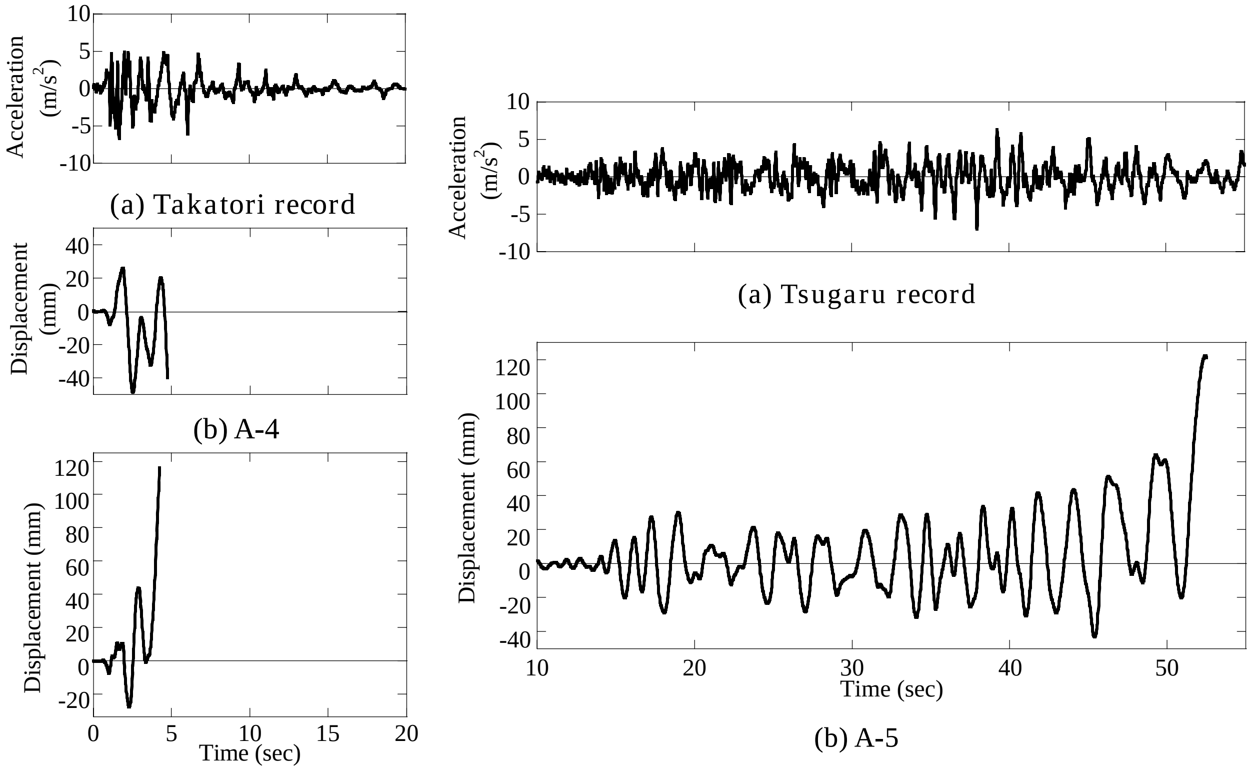
Figure 4 Lateral Force vs. Lateral Displacement under Pushover and Cyclic Loadings

the lateral force capacity starts to deteriorate at 2.5 % drift in A-2 and 2.0 % drift in B-2 under the cyclic loading. The displacement capacity at where the lateral force capacity starts to deteriorate is smaller under the cyclic loading than the pushover loading.

4. FAILURE MODE UNDER HYBRID LOADING

Figs. 5 and 6 show damage and response displacement of A-4 and B-3 which were subjected to Takatori record and A-5 which was subjected to Tsugaru record under the hybrid loading. A-4 and B-3 failed in pure shear while A-5 failed in shear but like flexure. A sharp increase of response displacement in one direction due to large long-period pulse ground motion resulted in shear failure from the upper termination zone to the base of the column. On the other hand, repetition of cyclic response displacement with similar amplitudes in the positive and negative directions resulted in shear failure around the upper termination zone which looks like flexural failure. The dependence of failure mode on the type of ground motions may be explained from the difference of failure modes between the pushover and cyclic loadings.

Fig. 7 shows the lateral force vs. lateral displacement hystereses under the hybrid loadings. The peak lateral force capacity is nearly 100 kN at about 1.5 % drift in both A-4 and B-3. The lateral force capacity and the drift when the peak lateral force is developed are very close to those under the pushover and cyclic loadings. On the other hand, the lateral force capacity starts to sharply



(a) Takatori record

(b) A-4

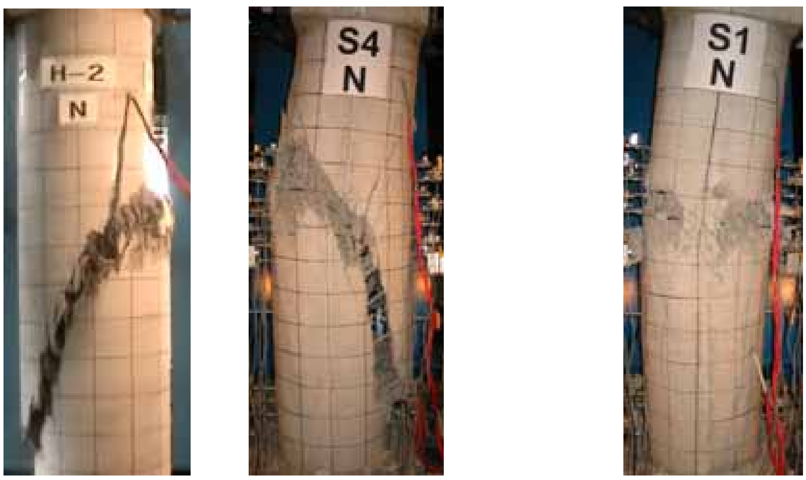
(c) B-3

(a) Tsugaru record

(b) A-5

(1) Near-field ground acceleration (2) Middle-filed ground acceleration

Figure 5 Response Displacement at 1,680 mm from the base

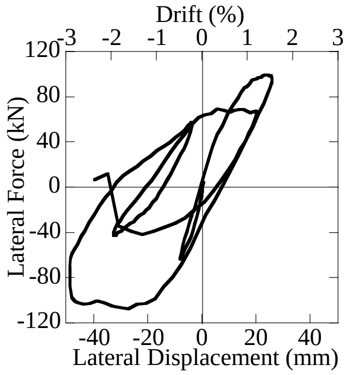


(a) A-4 (b) B-3 (a) A-5

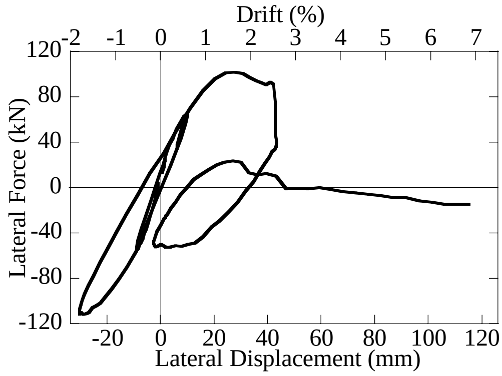
(1) Near-field ground acceleration (2) Middle-field ground acceleration

Figure 6 Damage after Hybrid Loadings

deteriorate at 2.9 % drift in A-4 and 2.5 % drift in B3 under the Takatori record. Obviously slightly less drift in B-3 than A-4 at which shear failure started to occur is due to the smaller shear and flexural strength ratio. It is noted that deterioration of the lateral force capacity over 2.5% drift in A-5 occurred more gradually than that in A-4 and B-3.

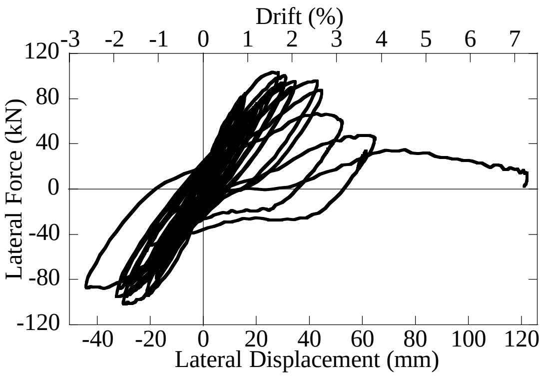


(a) A-4



(b) B-3

(1) Near field ground acceleration



(a) A-5

(2) Middle field ground acceleration

Figure 7 Lateral Force vs. Lateral Displacement under Hybrid Loadings

6. CONCLUDIONS

To clarify the failure mechanism of RC bridge columns with termination of main reinforcements at two heights, a series of loading experiments was conducted on eight models with two sets of shear and flexural strength ratios. Based on the results presented herein, the following conclusions may be deduced:

1. A-1 model with a shear and flexure strength ratio at the upper termination of 0.84 failed in shear which was directed from the upper termination to the base under the pushover loading, while A-2 model with the same shear and flexure strength ratio failed in shear which occurred around the upper termination zone without extension of diagonal cracks under the cyclic loading. On the other hand, B-1 and B-2 models with a shear and flexure strength ratio at the upper termination of 0.78 failed in shear which was directed from the upper termination to the base under both the pushover loading and cyclic loading.

2. The difference of failure modes between A-1 and A-2 was resulted from the damage at the upper termination zone. Under the pushover loading, because the concrete and main reinforcements in the compression zone did not suffer damage, the compression was directly transferred to the base by the compression strut resulting in a major shear crack extended from the upper termination to the base. This resulted in shear failure in A-1. On the other hand, under cyclic loading, failure of covering and core concrete as well as local buckling of main reinforcements at the upper termination zone which were developed due to cyclic

compression and tension progressed before major diagonal cracks extended from the upper termination to the base. This resulted in failure around the termination zone in A-2.

3. The failure mode dependence on the loading protocol in B-1 and B-2 models with a shear and flexure strength ratio of 0.78 is less than that in A-1 and A-2, but the difference of failure modes still exists between B-1 and B-2. A major diagonal crack extended from the upper termination to the base which resulted in a complete shear failure in B-1, however longitudinal reinforcements buckled before a major diagonal crack extended to the base in B-2.

4. A-4 model with a shear and flexure strength ratio at the upper termination of 0.84 which was subjected to a near-field ground acceleration failed in shear directed from the upper termination to the base, while A-5 model with the same shear and flexure strength ratio which was subjected to a middle-field ground acceleration failed in shear developed only around the upper termination zone without extension of diagonal cracks. On the other hand, B-3 model with a shear and flexure strength ratio at the upper termination of 0.78 which was subjected to a near-field ground motion failed in shear directed from the upper termination to the base. A large response displacement in one direction resulted in a similar action developed in the pushover loading in A-4 and B-3, while a number of repetitive response in both directions resulted in a similar action developed in the cyclic loading in A-5.

REFERENCES

Ikehata, S., Adachi, K., Yamaguchi, T. and Ikeda, N. (2001). Seismic performance of RC bridge piers based on quasi-static dynamic loading tests. Proc. Japan Concrete Institute, 23 (3), 1255-1260.

Japan Road Association (2002). Part V Seismic design-Design specifications of highway bridges, Maruzen, Tokyo, Japan.

Kawashima, K., Unjoh, S. and Iida, H. (1993). Seismic inspection and seismic strengthening methods of RC bridge piers at mid-height where main reinforcements are terminated with inadequate anchoring length, Report of PWRI, 189, Public Works Research Institute, Ministry of Construction.

Yamamoto, T., Ishibashi, T., Otsubo, M. and Kobayashi, S. (1984). Experimental studies on seismic resistance of a pier with reinforcement terminated in a tension zone. Proc. Concrete Engineering, JSCE, 348/V-1, 61-70.