SEISMIC PERFORMANCE OF RECTANGULAR COLUMNS AND INTERLOCKING SPIRAL COLUMNS

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ABSTRACT :

Seismic performance of RC rectangular columns was clarified in comparison with interlocking spiral columns based on a shake table experiment as a part of NEES and E-Defense collaboration on bridge project. It is found from the experiment that the interlocking spiral columns and rectangular columns with cross ties performed satisfactorily under the design and ultimate level excitations.

KEYWORDS: bridge, seismic design, shake table experiment, E-Defense, NEES

1. INTRODUCTION

Past researches show that interlocking spiral columns are effective for enhancing the ductility capacity of bridge columns [for example, Tanaka and Park 1993, Fujikura and Kawashima 2000, Yagisita, Tanaka and Park 2000, Ohtaki et al 2000]. Uniform lateral confinement by circular spirals and the cross sections which chamfer corners enhance the ductility capacity. On the other hand, rectangular columns with cross ties are widely implemented in Japan after the 1995 Kobe, Japan earthquake. Based on the Japanese Road Association (JRA), cross ties having the same bar diameter with ties are set at less than 1m distance at every 150 mm spacing. Failure at corners under bilateral excitation is a weak point of rectangular columns, but cross ties enhance the ductility capacity by isolating damage at corners.

Rectangular columns have generally higher flexural strength and stiffness than interlocking spiral columns if the section size is the same and the same amount of reinforcements are used. Assembling spirals becomes difficult as column size increases larger than 3-4 m. A static cyclic loading experiment on a 2.5 m wide square section column with cross ties exhibited stable seismic performance under repeated loading [Hoshikuma et al 2001].

Based on the above background, the seismic performance of two RC rectangular column models was clarified in comparison with two interlocking spiral column models based on a shake table experiment. The experiment was conducted at the Richmond Field Station, University of California, Berkeley as a part of NEES and E-Defense collaboration on bridge project. Two rectangular column models with cross ties and two interlocking spiral column models which had the same section sizes were constructed in the Tokyo Institute of Technology, and they were shipped to the Richmond Field Station. The basic design procedures and details used to proportion the four models followed the basic requirements of JRA and California Department of Transportation (Caltrans) codes [JRA 2002, Caltrans 2004a and b]. Using a near-field ground motion recorded at JR Takatori Station during 1995 Kobe, Japan earthquake, the four models were excited using essentially the same loading sequence. Progress of damage and the seismic performance of the rectangular columns and interlocking spiral columns were clarified in terms of variation of fundamental natural periods, moment capacities, column displacements and residual displacements during the repeated excitation sequence.

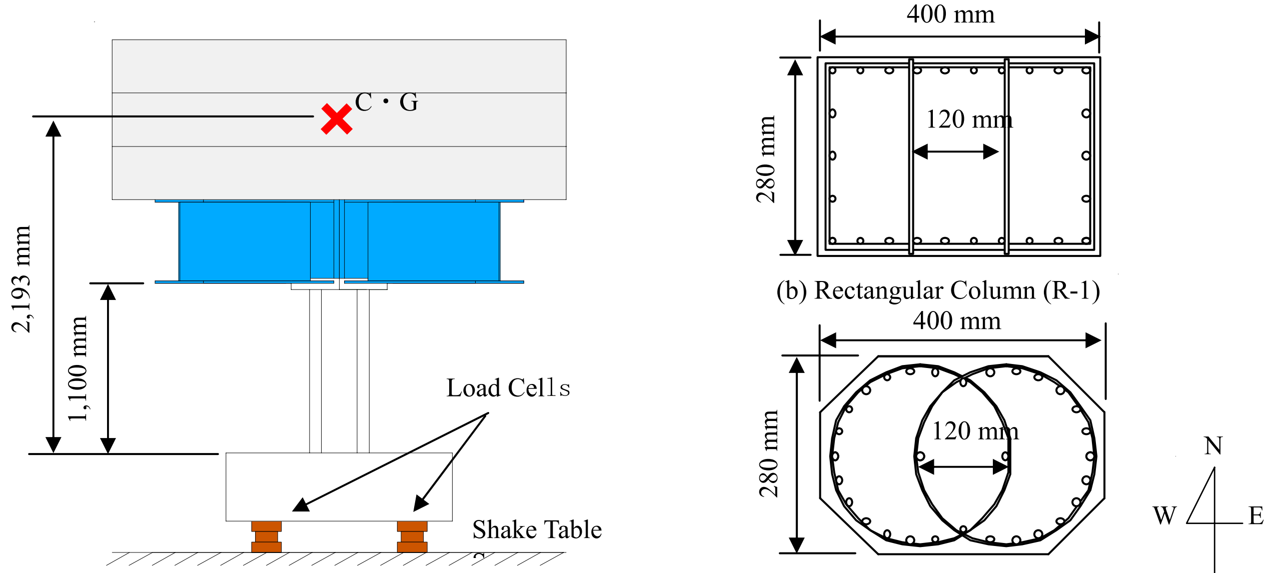


Figure 1 Sections of R-1 and I-1

Photo 1 Setup of Column Model on the Shake Table

2. EXPERIMENTAL MODELS

One of the two sets of the rectangular column and the interlocking spiral column had a cross-section of 400 mm $\times$ 280 mm, as shown in Figure 1, which are denoted hereinafter as R-1 and I-1, respectively. The other set of the rectangular column and interlocking spiral column had a cross-section of 440 mm $\times$ 280 mm, which are denoted as R-2 and I-2, respectively. The sections of interlocking spiral columns correspond to the section of rectangular columns with the four corners being chamfered. The models supported three concrete mass blocks with the total weight of 226.6 kN as shown in Photo 1, thus the axial stress of the columns at the plastic hinge was 2.02 MPa and 1.84 MPa in R-1 and R-2 and 2.28 MPa and 2.05 MPa in I-1 and I-2, respectively. The four models are 1,100 mm high, therefore the distance from the column base to the gravity center of the three mass blocks is 2,193 mm.

They were designed based on JRA and Caltrans codes assuming the scaling factors of the geometry, time and stress of 1/6, $1/\sqrt{6}$ and 1/6, respectively. Consequently, the scaling factor for the acceleration is equal to 1.0. In the modeling of the specimens, it was originally intended to design R-1 and R-2 based on the JRA code and I-1 and I-2 based on the Caltrans codes. However there were important limitations in the modeling of the specimens resulted from the preferred size of columns and different intensity of the design ground motion between US and Japan. Shake table experiments for models with different sizes and intensity of the design ground motion do not produce meaningful results for comparison of the seismic performance of the rectangular columns and the interlocking spiral columns, therefore it was decided that the models should have the same section size and the response acceleration corresponding to the flexural yield is nearly 2.0 m/s^2. The models were then designed by combining the requirements of JRA and Caltrans codes as follows.

Target strength of concrete was 30 MPa. The strength of concrete based on cylinder tests at the day of the experiment was 32.2 MPa and 32.0 MPa for R-1 and I-1, respectively. Deformed bars with nominal strength of 345 MPa (SD345) having a diameter of 10 mm and 6 mm were used for longitudinal bars and tie bars/spirals, respectively. The yield stress, yield strain and the tension strength were 400 MPa, 2,200 μ and 570 MPa for 10 mm diameter bars and 360 MPa, 2,000 μ and 550 MPa for 6 mm diameter bars, respectively.

Two cross ties having the same diameter with ties were provided in R-1 and R-2 in the longitudinal direction based on JRA. Cross ties were not required in those models in the transverse direction because of shorter width. The center-to-center distance of spiral bars in section for interlocking spiral column was set as 1.0 and 1.2 times the radius of spirals in section (=120 mm and 160 mm) in I-1 and I-2, respectively based on Caltrans. Tie spacing of the columns at the plastic hinge needs to be less than 150 mm and 200 mm in JRA and Caltrans codes, respectively. Therefore it was set here 150/6=37 mm in the models following the Caltrans. Based on JRA, the volumetric reinforcement ratio ρ_s by the following equation is recommended not to exceed 1.8%. This is because there must be a certain limitation for the enhancement of the ductility capacity by only increasing the amount of ties.

$$\rho_s \equiv \frac{4A_h}{sd} \tag{1}$$

where, A_h is the nominal section area of ties and cross ties, s is tie and cross tie spacing, and d is the effective distance between ties and/or cross bars. Consequently, the volumetric tie (spiral) reinforcement ratio is 2.85 % and 2.57 % in R-1 and R-2, respectively, which are larger than the recommended ρ_s values. Obviously Japanese columns are generally larger in section, therefore ρ_s is not generally in such a high value.

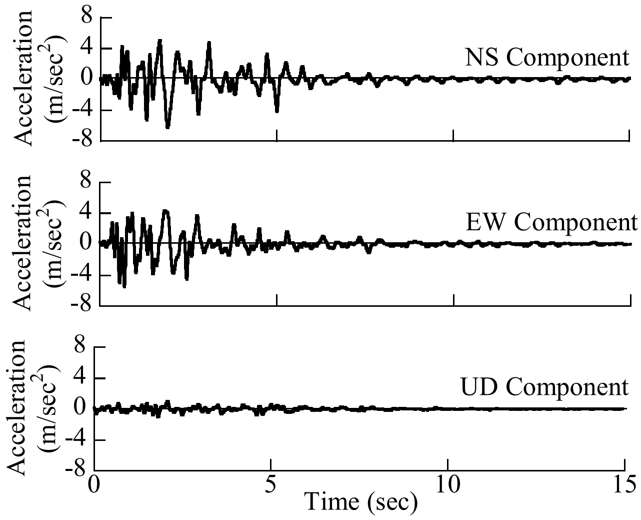
It is not known how d should be evaluated in Eq. (1) for the interlocking spiral columns. Therefore if a volumetric tie reinforcement ratio $\tilde{\rho}_s$ is simply defined as a ratio of the volume of concrete and the volume of confining reinforcements, $\tilde{\rho}_s$ becomes 1.15 % and 1.05 % In R-1 and R-2 and 1.30 % and 1.17 % in I-1 and I-2, respectively. This means that the amount of spirals in I-1 and I-2 are 0.15 % (=1.30-1.15 %) and 0.12 % (=1.17-1.05 %) larger in ρ_s than the amount of ties and cross ties in R-1 and R-2, respectively.

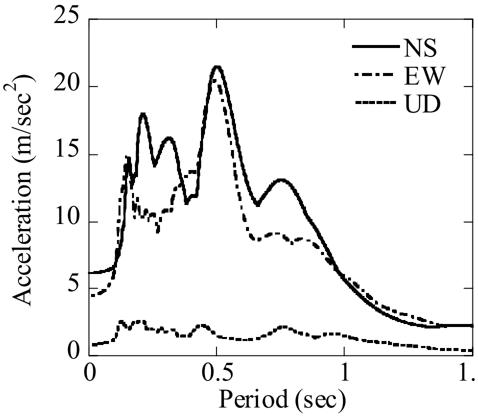
Number of longitudinal bars which were determined based on the above requirement that the response acceleration corresponding to the yield should be nearly 2.0 m/s^2 was 26 in R-1 and R-2 and 30 in I-1 and I-2. Effective contribution of longitudinal bars around the rectangular shape to the flexure strength resulted in 4 bars less in R-1 and R-2 than I-1 and I-2. Four longitudinal bars were set in the interlocked zone in I-1 and I-2 based on Caltrans. Consequently, the main reinforcement ratio becomes 1.66 % and 1.51 % in R-1 and R-2, and 2.16 % and 1.94 % in I-1 and I-2, respectively. Because the recommended main reinforcement ratio is 0.8-2.0 % in JRA and 1.0-4.0 % in Caltrans, the main reinforcement ratios of the four models satisfy those requirements.

Response of the models is presented with an emphasis on R-1 and I-1 in this paper.

3. SEQUENCE OF EXCITATIONS

The models were subjected to 3D excitation using the shake table at Richmond Field Station. NS, EW and UD components of the ground accelerations recorded at JR Takatori Station during 1995 Kobe, Japan earthquake (refer to Figure 2), were imposed in the longitudinal, transverse and vertical directions, respectively. Time axis of the original accelerations was scaled by $1/\sqrt{6}$ based on the above scaling rule. The models were first excited so that they remain in the elastic level (elastic level excitation). Then the models were subjected to the

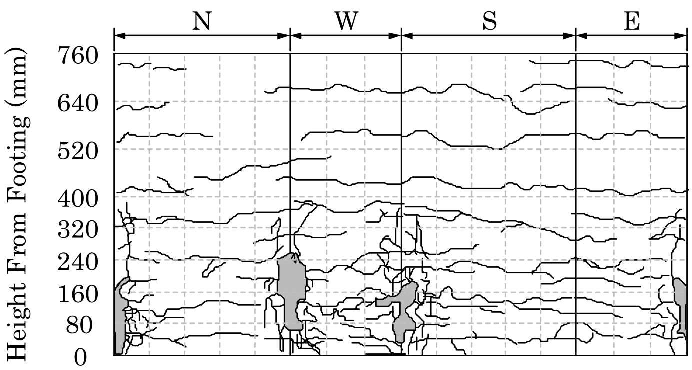




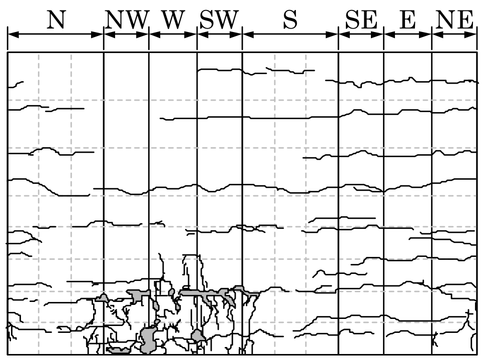
(b)Response Acceleration (ξ =0.05)

(a) Accelerations

Figure 2 Original JR Takatori Station Accelerations



(a) R-1



(b) I-1

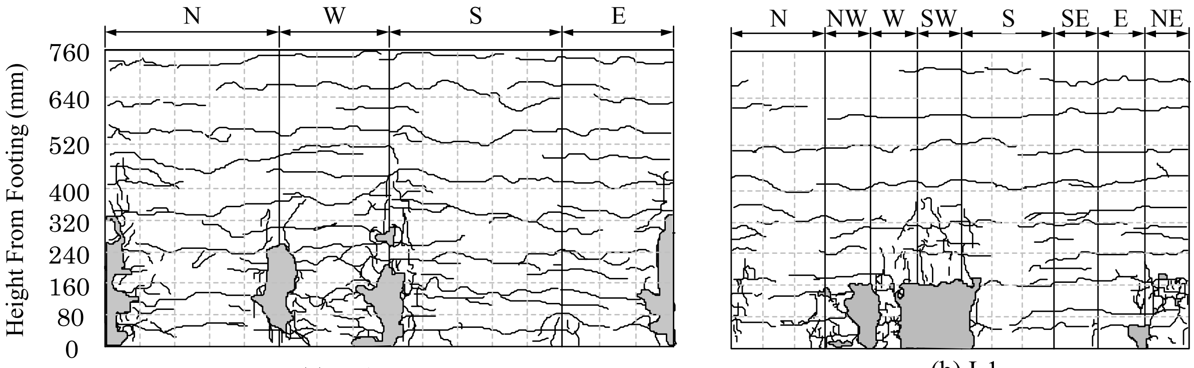
Figure 3 Damage of R-1 and I-1 after the Design Level (1) Excitation was Completed

scaled JR Takatori ground acceleration with an intensity which is scaled so that computed column displacement reaches the initial yield displacement, the design displacement and the ultimate displacement based on JRA. They are denoted hereafter as “yield level,” “design level (1)” and “ultimate level (1)” excitations, respectively. The intensity of table accelerations at the yield, design and ultimate level excitations were 14 %, 90 % and 120 % the scaled JR Takatori Station accelerations, respectively.

Because damage of the models after those excitations was still limited, the design level and ultimate level excitations were repeated which are noted as “design level (2)” and “ultimate level (2)” excitations. Then the models were further subjected to 140 % and 160 % the scaled JR Takatori Station accelerations which correspond to 1.17 and 1.33 times the ultimate level excitations. Finally, the 1.33 times the ultimate level excitation was repeated until the models became unstable.

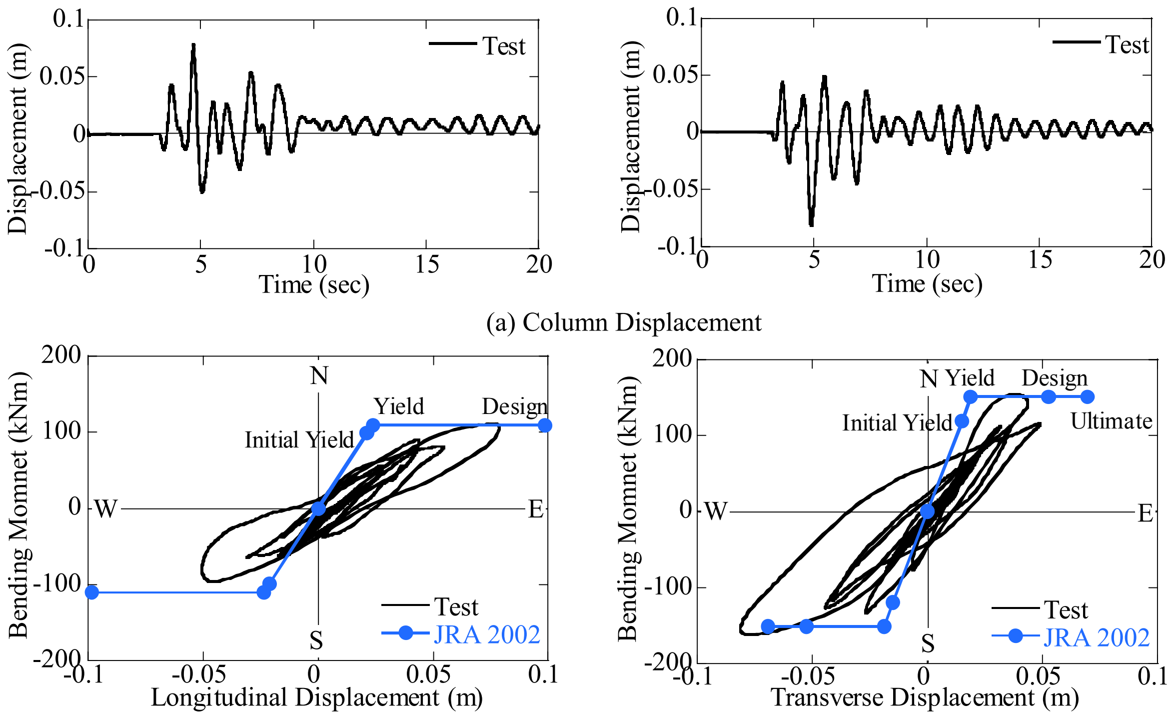
4. SEISMIC RESPONSE OF R-1 AND I-1 MODELS

R-1 and I-1 suffered neither local buckling nor rupture of longitudinal bars under the design level (1) excitation and the ultimate level (1) excitation although covering concrete spalled at the corners as shown in Figures 3 and 4. Figures 5 and 6 show the responses of R-1 and I-1, respectively, under the design level (1) excitation. The peak mass block displacements are 73.0 mm (3.33 % drift) and 90.0 mm (4.10 % drift) in the longitudinal and transverse directions in R-1, while they are 78.8 mm (3.59 % drift) and 81.2 mm (3.70 % drift) in the



(a) R-1 (b) I-1

Figure 4 Damage of R-1 and I-1 after the Ultimate Level (1) Excitation was Completed



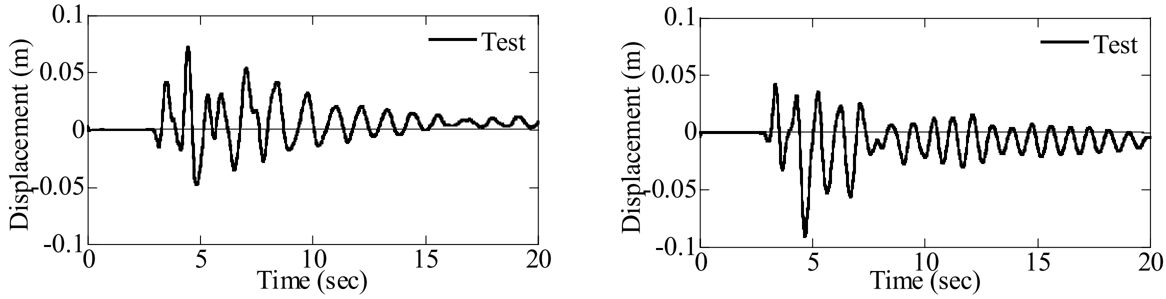
(a) Column Displacement

(b) Moment vs. Column Displacement Hystereses

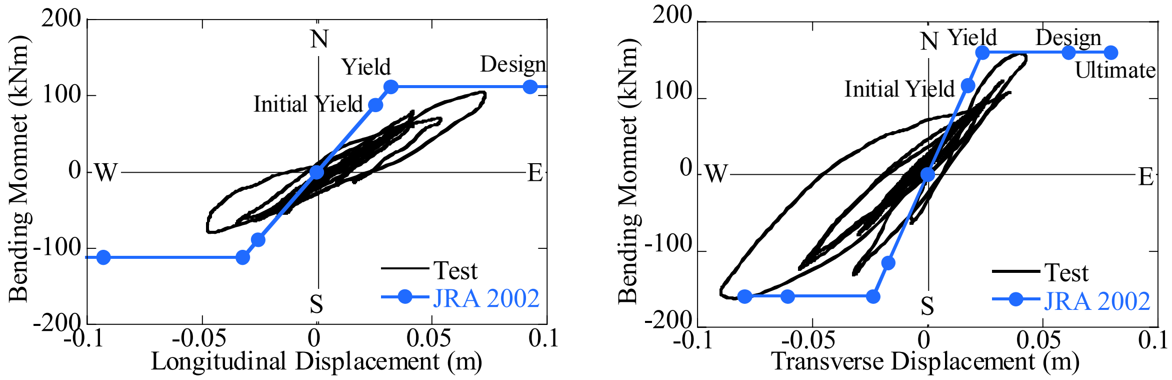
Figure 5 Seismic Responses of R-1 during Design Level (1) Excitation
(Left: NS Direction, Right: EW Direction)

longitudinal and transverse directions in I-1. The peak bending moments at the bottom of the model columns are 111.4 kNm and 161.6 kNm in the longitudinal and transverse directions in R-1 while they are 104.8 kNm and 162.8 kNm in the longitudinal and transverse directions in I-1. Two columns performed quite satisfactorily for the design level excitation. The seismic performance R-1 and I-1 under the ultimate level excitation was in the similar level. It is interesting to note that strains of spirals, ties and cross bars did not yield under those excitations.

Under the stronger than expected excitations, R-1 was subjected to a 1.17 times the ultimate level excitation and five 1.33 times the ultimate excitation, while I-1 was subjected to a 1.17 times the ultimate level excitation and three 1.33 times the ultimate excitation as shown in Figure 7. Because residual drift became significant in I-1 in the longitudinal direction, excitation by 1.33 times the ultimate level excitation was terminated after 3rd excitation was completed. Photos 2 and 3 compare the damage of R-1 with I-1 after the design level (2) excitation and the 3rd 1.33 times the ultimate level excitation. During the 1.17 times the ultimate level

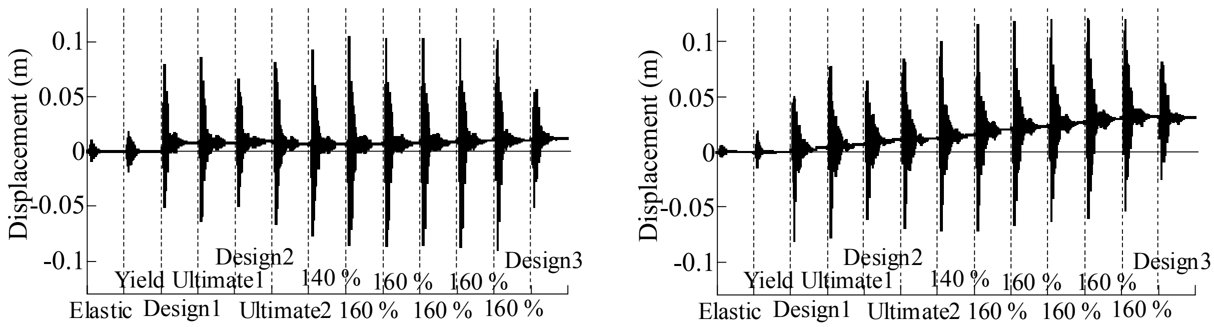


(a) Column Displacement

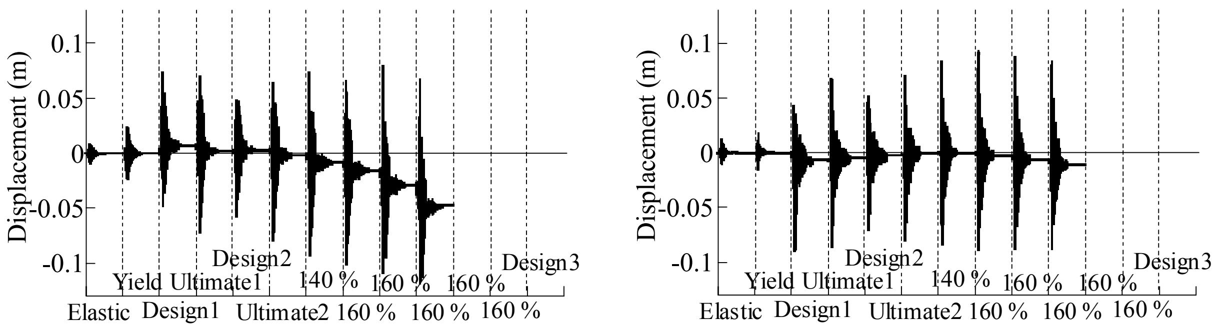


(b) Moment vs. Column Displacement Hystereses

Figure 6 Seismic Responses of I-1 during Design Level (1) Excitation
(Left: NS Direction, Right: EW Direction)



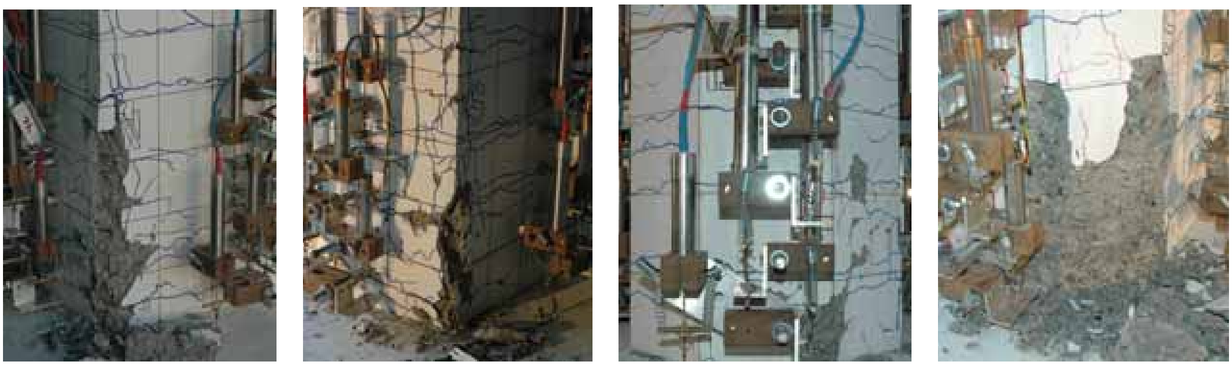
(a) R-1



(b) I-1

Figure 7 Response Displacement of R-1 and I-1 (Left: NS Direction, Right: EW Direction)

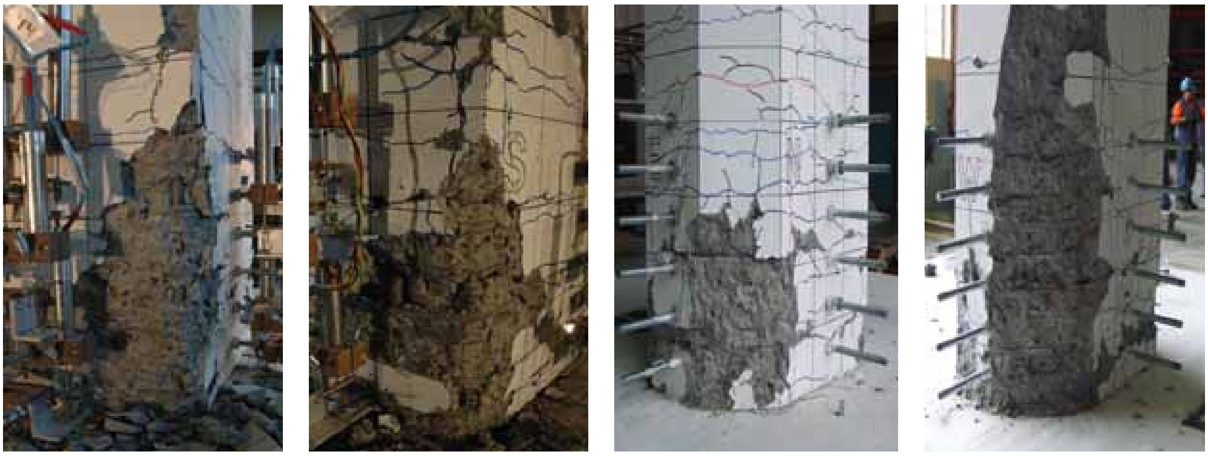
excitation, progress of damage was minor in I-1, while R-1 suffered a certain compression failure at the core concrete at SW corner in the plastic hinge. During the 3rd 1.33 times the ultimate level excitation, a



(1) NE Corner (2) SW Corner (1) E Face (2) SW Face

(a) R-1 (b) I-1

Figure 2 Damage of R-1 and I-1 Models after the Design Level (2) Excitation was Completed



(1) NE Corner (2) SW Corner (1) NE Face (2) SW Face

(a) R-1 (b) I-1

Photo 3 Damage of R-1 and I-1 Models after the 3rd 1.33 Times the Ultimate Level Excitation was Completed

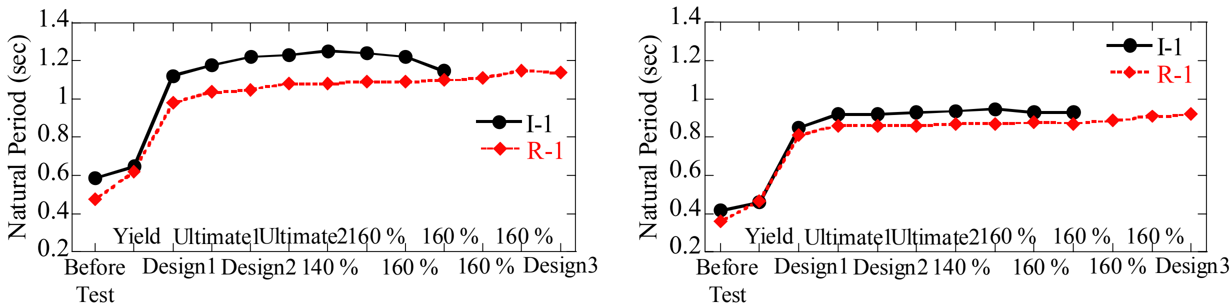


Figure 8 Variation of Fundamental Natural Period after each Excitation
(Left: NS Direction, Right: EW Direction)

longitudinal bar ruptured and two longitudinal bars locally buckled in R-1 at the SW corner due to the combined bilateral excitation, while a part of core concrete as well as the covering concrete spalled but neither local buckling nor rupture of any longitudinal bars occurred in I-1. The lateral confinement at corners is a weak point of R-1, while the damage at corers can be mitigated in I-1 because corners are chamfered.

However the damage at a corner in R-1 has to be carefully evaluated. The most important point is that the local buckling and rupture occurred under repeated larger than expected excitations. In fact, visible local buckling of a longitudinal bar first occurred during 1st 1.33 times the ultimate level excitation in R-1. As shown in Figure 8 which compares variation of the fundamental natural periods of R-1 and I-1, the response of R-1 was stable during the entire excitations. Any significant deterioration of the stiffness did not occur in R-1. It is noted that the column stiffness was slightly higher in R-1 than I-1 during the entire excitations. The lateral confinement still remained at the most section in R-1 even after a corner suffered damage. The section of R-1 after damage at a corner become close to the section of I-1 because the section of interlocking spiral column is chamfered at corners.

5. CONCLUSIONS

A series of shake table experiments was conducted to clarify the seismic performance of rectangular columns with cross ties and interlocking spiral columns. Based on the results presented herein, the following conclusions may be deduced;

1) Damage of R-1 and I-1 models under the design level (1) excitation and the ultimate level (1) excitation was only spalling of covering concrete and neither buckling nor fracture of longitudinal reinforcements occurred. Both interlocking spiral columns and rectangular columns performed satisfactorily under the design and ultimate level excitations.
2) Under repeated stronger than expected excitations, damage of R-1 was more significant at a corner than I-1 after 3rd 160 % the scaled Takatori accelerations. However R-1 exhibited stable response without any deterioration of column restoring force during the entire range of excitations. Furthermore the fundamental natural period after each excitation was higher in R-1 than I-1. This is attributed to the fact that the rectangular section of R-1 after damage of a corner becomes close to the section of I-1.

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