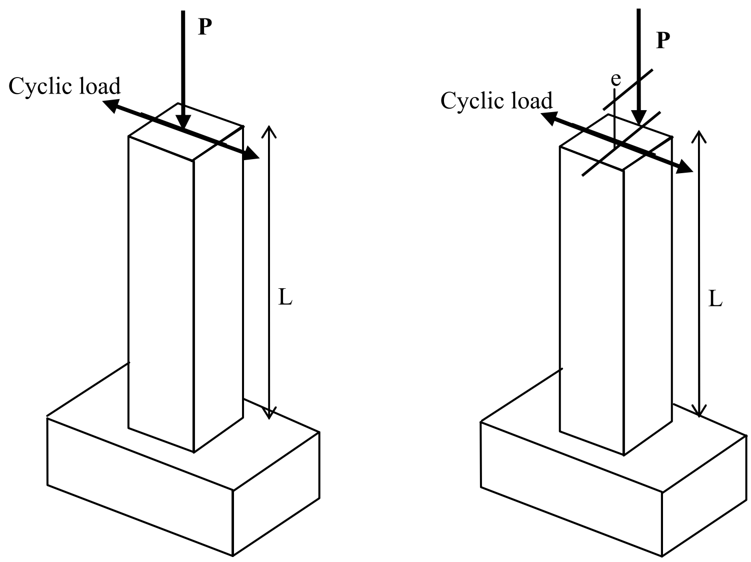
MATERIALS AND METHOD

Design of Test Specimens

Test specimens were constructed prototype reinforced concrete columns by scaling down full size bridge columns having 600 mm × 600 mm and 800 mm × 800 mm full size cross sections. Six specimens were conducted prototype reinforced concrete columns having the same dimension of 400 mm × 400 mm and total height of 2050 mm while the effective height of 1550 mm. The longitudinal reinforcement consisted of 13 mm diameter deform bars with yield strength of 395 MPa (SD345). Tie reinforcement is of 6 mm diameter round bars with yield strength of 245 MPa (RB245) using 135 degree hooks with a development length of 75 mm. The concrete strength is in the range from 29.61 to 32.36 MPa. The amount of lateral reinforcement was varied to investigate its confining effect on strength and ductility of the specimens. The footing was constructed with dimension 700 × 1150 mm and height of 400 mm. To monitor the behavior during loading test, all specimens were instrumented strain gages, load cells and displacement transducers. The column model details are shown in Fig. 20 and the details of the column characteristic are summarized in Table 3.



a) Specimens A1, B1, C1 and D1 b) Specimens A2 and B2

Figure 20 Column model

Table 3 Detailing of column test specimens

ID in the reference	A1	A2	B1	B2	C1	D1
Section size, $b \times h$ (mm)	400 x 400 (Square)					
Effective height, L(mm)	1550					
Effective depth, d (mm)	360					
Longitudinal reinforcement ratio, ρ_l (%)	1.27					
Ratio of tie reinforcement $\{(A_{sh}/(sh_c), \rho_s\ (\%)\}$	0.37		0.37		0.09	0.19
Volume ratio of tie bar $\{(V_{core}/V_{sh}), \rho_v(\%)\}$	0.75		0.93		0.19	0.50
Cylinder strength of concrete $(f_c^{'})$ (MPa)	32.36		29.61		32.36	29.61
Longitudinal reinforcement (f_y =390 MPa)	16D13 SD345					
Tie reinforcement (f_y = 245 MPa)	1-R6@50		2-R6@100		1-R6@200	1.5-R6@150
Axial force (kN)	384 kN					
Axial force index $\{P/(f_c^{'}A_g)\}$	0.074		0.081		0.074	0.081
Eccentricity (e/h)	0	0.15	0	0.15	0	0

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Tie reinforcement ratios, ρ_s:(A_{sh}/sh_c), of specimens A1, A2, B1 and B2 were the same and were designed corresponding to 25% of the minimum total cross-section area of rectangular hoops and cross ties based on AASHTO with seismic performance for zone 3 (0.19g < a < 0.29g) and zone 4 (a > 0.29g), being the larger of Equation 2 and 3. To investigate the effect of cross tie reinforcement, cross-tie stirrups were provided in specimen B1 and B2. The tie reinforcement ratio of specimen D1 was designed corresponding to the minimum hoops based on AASHTO with non-seismic performance and specimen C1 was designed based on AASHTO with non-seismic performance and having larger spacing of tie bars than that of specimen D1 and without the cross ties.

To investigate the effect of eccentric loading, specimens A2 and B2 were loaded by eccentric loading both axial load and lateral cyclic load. Besides, specimens A1 and B1 were loaded by concentric loading both axial load and lateral cyclic load. All properties and details of test specimens are shown in Table 3, Fig. 21 and Fig. 22.

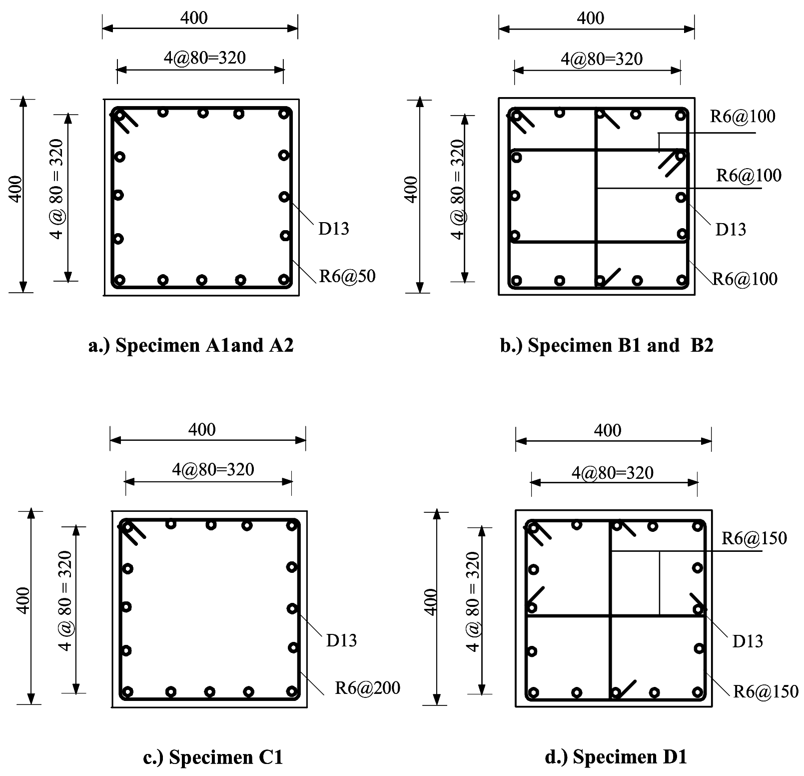


Figure 21 Cross section of test specimens

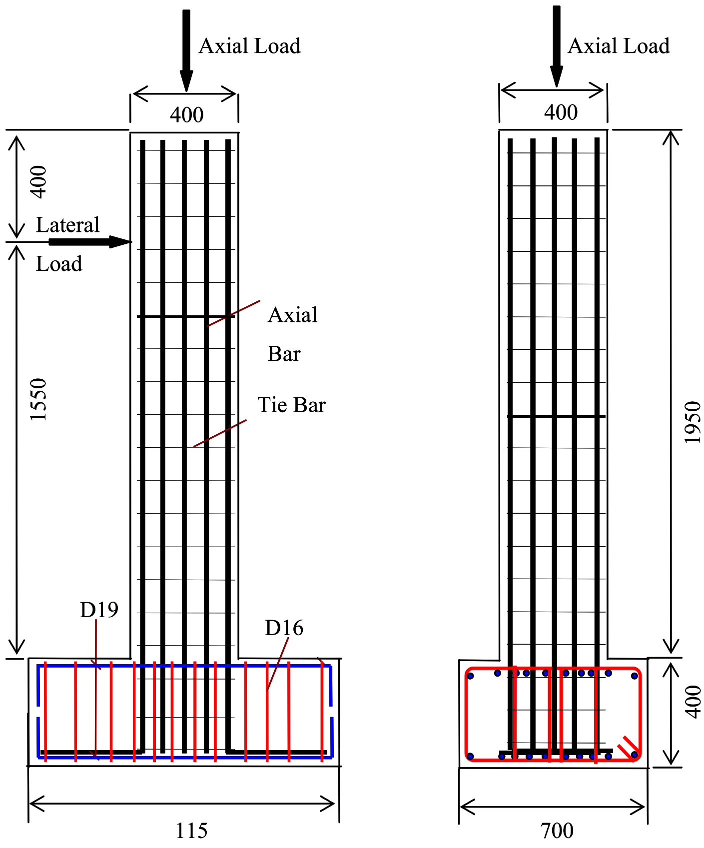


Figure 22 Elevation of test specimen

Specimens A1 and A2

Specimens A1 and A2 were conducted prototype reinforced concrete columns by scaling down 2/3 of full size bridge columns having 600 mm × 600 mm cross sections to be a rectangular section of 400 mm × 400 mm. The tie reinforcements were designed corresponding to 25% of the minimum total cross-section area of rectangular hoops and cross ties based on AASHTO with seismic performance for zone 3 (0.19g < a < 0.29g) and zone 4 (a > 0.29g), being the larger of Eq. 1 and 2. Tie bars reinforcement with a 6 mm diameter (R6) were provided at 50 mm spacing in the entire column height. These specimens were subjected to a constant axial load of 384 kN and a lateral cyclic load. To investigate the effect of both lateral and axial eccentric loading, the specimen A1 was loaded by concentric loading but specimen A2 was loaded by eccentric loading respectively. This eccentricity (*e*) is equal to 0.15*d* in which *d* is the column width. This eccentricity was measured from the center of the column to the point in which both axial load and cyclic load were applied. The cyclic load was applied in the perpendicular direction to the eccentricity.

Tie reinforcement ratio is as flowing:

$$\frac{A_{sh}}{sh_c} = \frac{63.4}{50 \times 340} \qquad = \qquad 0.0037$$

Shear capacity was calculated following:

$$V_n = V_c + V_s + V_p$$

$$V_c = 0.8 \times A_e \times 0.1\sqrt{f_c^{'}}$$

$$V_c = 0.8 \times 400 \times 400 \times 0.1\sqrt{32.36}\,/1000 \qquad = \qquad 72.81 \qquad \text{kN}$$

$$V_s = \frac{A_v f_y d}{s}$$

$$V_s = \frac{63.4 \times 245 \times 360}{50}/1000 \qquad = \qquad 111.84 \qquad \text{kN}$$

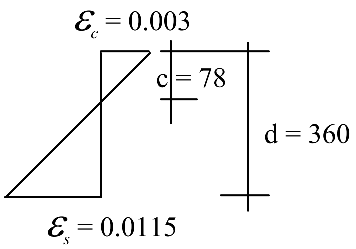
$$V_p = 0.2P$$

$$V_p = 0.2 \times 384 \qquad = \qquad 76.8 \qquad \text{kN}$$

$$V_n = V_c + V_s + V_p$$

$$V_n = 72.81 + 111.84 + 76.8 \qquad = \qquad 261.45 \qquad \text{kN}$$

Nominal moment strength (M_n) subjected to the axial load 384 kN was calculated as following:



P_n = 384 kN

M_n = 179.67 kN

Figure 23 Strain diagram of specimens A1, A2 and C1

Shear demand (V_d) is calculated by dividing M_n with column height of 1.55 m.

$$V_d = \frac{179.67}{1.55} \times 1.2 = 139.1 \qquad \text{kN}$$

<u>**Specimens B1 and B2**</u>

Specimens B1 and B2 were conducted prototype reinforced concrete columns by scaling down 2/3 of full size bridge columns having 600 mm × 600 mm cross sections to be a rectangular section of 400 mm × 400 mm. The tie reinforcements were designed corresponding to 25% of the minimum total cross-section area of rectangular hoops and cross ties based on AASHTO with seismic performance for zone 3 (0.19g < a < 0.29g) and zone 4 (a > 0.29g), being the larger of Equation (2) and (3). To investigate the effect of cross-tie reinforcement, tie bars reinforcement with a 6 mm diameter (R6) and cross-tie stirrups were provided at 100 mm spacing in the entire column height. These specimens were subjected to a constant axial load of 384 kN and a lateral cyclic load. Also to investigate the effect of both axial eccentric loading and lateral cyclic load, the specimen B1 was loaded by concentric loading and specimen B2 was loaded by eccentric loading respectively. This eccentricity (*e*) is equal to 0.15*d* in which *d* is the column width. This eccentricity was measured from the center of the column to the point in which both axial load and cyclic load were applied. The cyclic load was applied in perpendicular direction to the eccentricity.

Tie reinforcement ratio is as following:

$$\frac{A_{sh}}{sh_c} = \frac{63.4}{50 \times 340} \qquad = \qquad 0.0037$$

Shear capacity is calculated as following:

$$V_n = V_c + V_s + V_p$$

$$V_c = 0.8 \times A_e \times 0.1\sqrt{f_c^{'}}$$

$$V_c = 0.8 \times 400 \times 400 \times 0.1\sqrt{29.61}/1000 \qquad = \qquad 69.65 \qquad \text{kN}$$

$$V_s = \frac{A_v f_y d}{s}$$

$$V_s = \frac{128.6 \times 245 \times 360}{100}/1000 \qquad = \qquad 111.84 \qquad \text{kN}$$

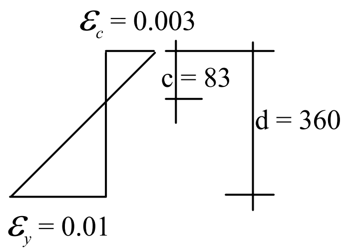
$V_p = 0.2P$

$V_p = 0.2 \times 384$ $\quad = \quad 76.8 \quad$ kN

$V_n = \; V_c + V_s + V_p$

$V_n = \; 69.65 + 111.84 + 76.8$ $\quad = \quad 258.29 \quad$ kN

Nominal moment strength (M_n) subjected to the axial load 384 kN was calculated the same as specimen A1 and A2



$P_n = 384$ kN

$M_n = 174.017$ kN

Figure 24 Strain diagram of specimens B1, B2 and D1

Shear demand (V_d) is calculated by dividing M_n with column height of 1.55 m.

$$V_d = \frac{174}{1.55} \times 1.2 = 134.71 \quad \text{kN}$$

Specimen C1

Specimen C1 was conducted prototype reinforced concrete columns by scaling down 2/3 of full size bridge columns having 600 mm × 600 mm cross sections to be a rectangular section of 400 mm × 400 mm. The tie reinforcement was designed corresponding to the minimum hoops based on AASHTO with non-seismic performance having larger spacing of tie bars than that of specimen D1 and without the cross ties. Tie bars reinforcement with a 6 mm diameter (R6) were provided at 200 mm spacing in the entire column height. This Specimen was subjected to concentric loading by a constant axial load of 384 kN and a lateral cyclic load.

$$\frac{A_{sh}}{sh_c} = \frac{63.4}{200 \times 340} \quad = \quad 0.0009$$

Shear capacity was calculated as following:

$$V_n = V_c + V_s + V_p$$

$$V_c = 0.8 \times A_e \times 0.1\sqrt{f_c^{'}}$$

$$V_c = 0.8 \times 400 \times 400 \times 0.1\sqrt{32.36}/1000 \quad = \quad 72.81 \quad \text{kN}$$

$$V_s = \frac{A_v f_y d}{s}$$

$$V_s = \frac{63.4 \times 245 \times 360}{200}/1000 \quad = \quad 27.96 \quad \text{kN}$$

$$V_p = 0.2P$$

$$V_p = 0.2 \times 384 \quad = \quad 76.8 \quad \text{kN}$$

$$V_n = V_c + V_s + V_p$$

$$V_n = 72.81 + 27.96 + 76.8 \quad = \quad 177.57 \quad \text{kN}$$

Nominal moment strength (M_n) subjected to the axial load 384 kN was calculated the same as specimen A1 and A2.

P_n = 384 kN

M_n = 179.67 kN

Shear demand (V_d) is calculated by dividing M_n with column height of 1.55 m.

$$V_d = \frac{179.67}{1.55} \times 1.2 = 139.1 \quad \text{kN}$$

Specimen D1

Specimen D1 was conducted prototype reinforced concrete columns by scaling down 1/2 of full size bridge columns having 800 mm × 800 mm cross sections to be a rectangular section of 400 mm × 400 mm. The tie reinforcement was designed corresponding to the minimum requirement based on ASSHTO code with non-seismic performance. Tie bars reinforcement and cross ties with a 6 mm diameter (RB6) were provided at 150 mm spacing in the entire column height for investigating the effect of cross-tie reinforcement. This specimen was subjected to concentric loading by a constant axial load of 384 kN and a lateral cyclic load.

$$\frac{A_{sh}}{sh_c} = \frac{95.1}{150 \times 340} \quad = \quad 0.0019$$

Shear capacity was calculated as following:

$$V_n = V_c + V_s + V_p$$

$$V_c = 0.8 \times A_e \times 0.1\sqrt{f_c^{'}}$$

$$V_c = 0.8 \times 400 \times 400 \times 0.1\sqrt{29.61}/1000 \quad = \quad 69.65 \quad \text{kN}$$

$$V_s = \frac{A_v f_y d}{s}$$

$$V_s = \frac{95.1 \times 245 \times 360}{150}/1000 \quad = \quad 55.92 \quad \text{kN}$$

$$V_p = 0.2P$$

$$V_p = 0.2 \times 384 \quad = \quad 76.8 \quad \text{kN}$$

$$V_n = V_c + V_s + V_p$$

$$V_n = 69.65 + 55.92 + 76.8 \quad = \quad 202.37 \quad \text{kN}$$

Nominal moment strength (M_n) subjected to the axial load 384 kN was calculated the same as specimen B1 and B2.

$$P_n = 384 \quad \text{kN}$$

$$M_n = 174.017 \quad \text{kN}$$

The shear demand (V_d) is calculated by dividing M_n with column height of 1.55 m.

$$V_d = \frac{174}{1.55} \times 1.2 = 134.71 \quad \text{kN}$$

Table 4 Shear capacity and shear demand

Specimens	Shear Capacity (kN)				M_n (kN-m)	Shear Demand V_d (kN)
	V_c	V_s	V_p	V_n		
A1	72.81	111.84	76.80	261.45	179.67	139.10
A2	72.81	111.84	76.80	261.45	179.67	139.10
B1	69.65	111.84	76.80	258.29	177.26	134.71
B2	69.65	111.84	76.80	258.29	177.26	134.71
C1	72.81	27.96	76.80	177.57	179.67	139.10
D1	69.65	55.92	76.80	202.37	177.26	134.71

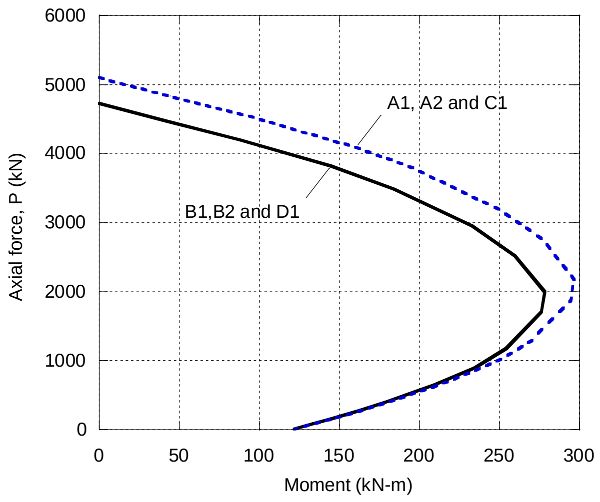
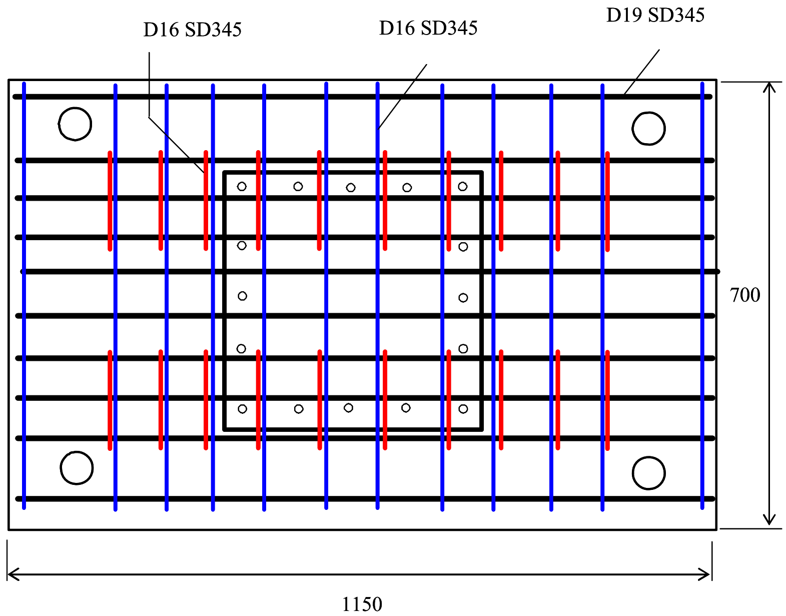


Figure 25 Interaction diagram of test specimens

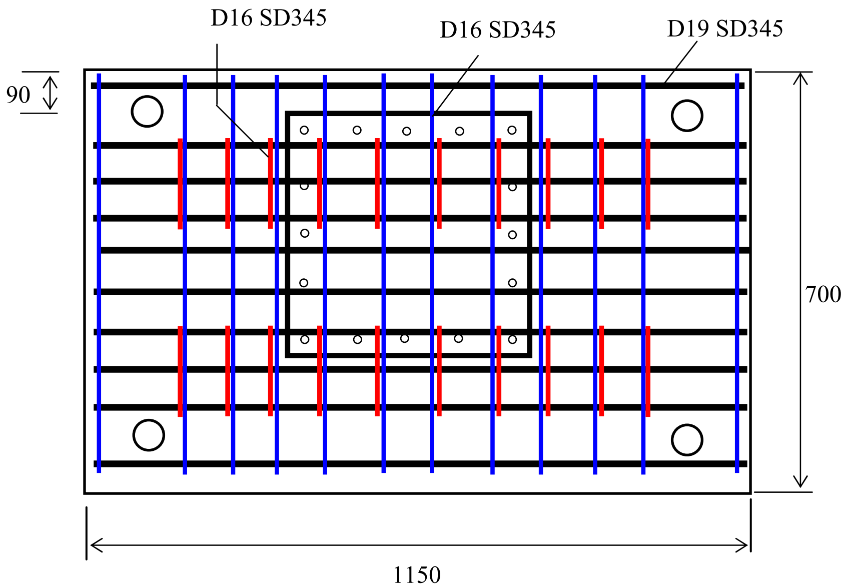
Footing Design

Each column specimen was constructed close to the reinforced concrete footing which was designed by not allowing shear failure and flexural yielding to occur under the estimated extreme loading condition. The footing dimension was 1050 mm long by 700 mm wide by 400 mm deep. The specified concrete compressive strength was 30 MPa. The reinforcement steel was 16 mm and 19 mm diameter deform bars with a nominal strength of 395 MPa. The footings were rocked to prevent negative and positive moment by means of tie-downs with four of 40 mm diameter pre-stressing bars with 25 tons stressing force each. A typical footing detail is showed in Fig. 25

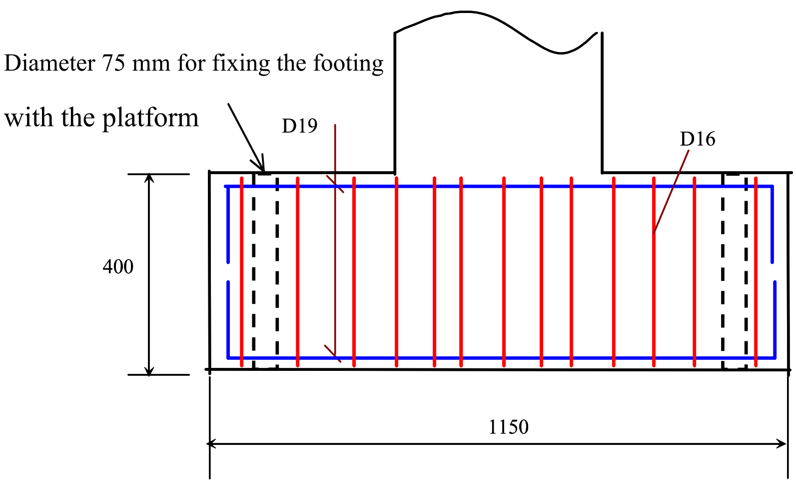


a.) Footing plan of specimens A1, B1, C1 and D1

Figure 26 Footing details



b.) Footing plan of specimens A2 and B2



c.) Footing section

Figure 26 (Continued)

Loading Condition

The actuator, MTS Mode 244 force rating 500 kN was used to apply the axial load. The actuator, MTS Mode 243 force rating 650 and 500 kN for compression and tension respectively was used to apply the lateral load as shown in Fig. 27.

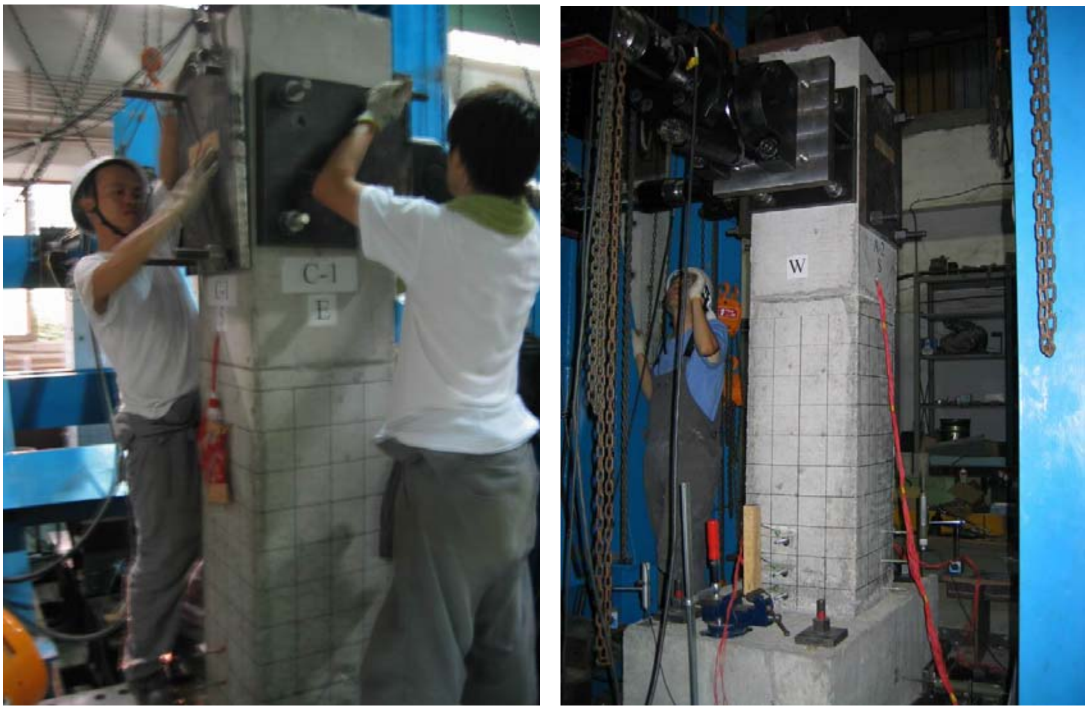


Figure 27 Actuators setup

Each specimen was subjected to a constant axial load of 384 kN corresponding to axial force index of 0.8. For the seismic loading, the unilateral cyclic load was applied in east and west direction at the top of column whose effective height was 1550 mm measured from the top of footing. The cyclic load based on displacement control corresponding to a stepwise decrease and increase loading with the preliminary loading of 0.25% drift ratio. Then loading started from 0.5% drift ratio with an increment of 0.5% drift ratio in each cycle until failure. The number of loadings was one and the time step was 30 second in each cycle. The load history is shown in Fig. 28

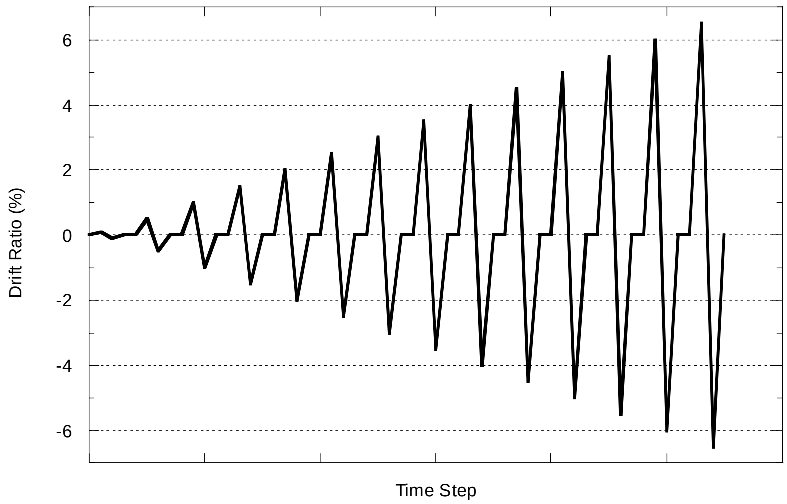


Figure 28 Applied lateral load history

The occurred failure was defined when the lateral load applied was reduced at least 25 % of maximum measured capacity. For the specimens A1, A2, B1 and B2, the maximum of cyclic load displacement control reached 6.5% drift ratio while the specimen C1 and D1 reached 5% and 5.5% drift ratio respectively.

Instrumentation

To collect data from each test, the specimens were instrumented with attached strain gages at longitudinal bars and confinement steel located in the plastic hinge zone. An array of strain gages are shown in Fig. 29 and Fig. 23.

For specimen A1 and A2, the attached strain gages were placed at fore longitudinal bars in the first and last layers at 50 mm and 150 mm height from the bottom of the column respectively. For confinement steel, the strain gages were placed at the first, third and fifth tie sets from the bottom of the column.

For specimen B1 and B2, the strain gages were placed at six longitudinal bars in the first and last layers at 50 mm and 150 mm height from the bottom of the column respectively. For confinement steel, the strain gages were placed at the first, third and fifth tie sets from the bottom of the column.

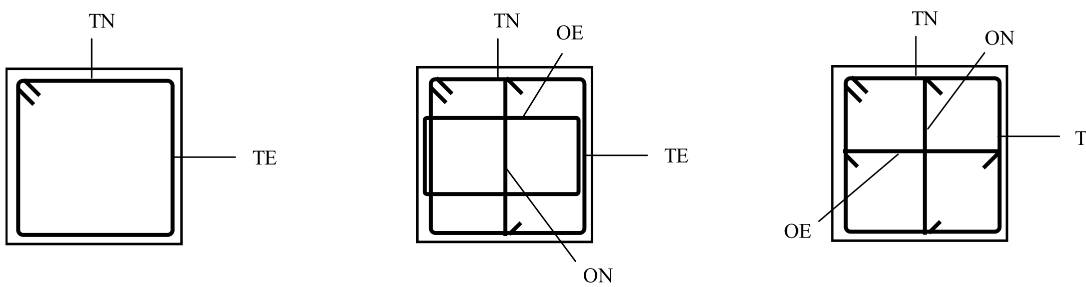
For specimen C1 the strain gages were placed at four longitudinal bars in the first and last layers at 50 mm and 150 mm height from the bottom of the column respectively. For confinement steel, the strain gages were placed at the first, second and third tie sets from the bottom of the column.

For specimen D1 the strain gages were placed at four longitudinal bars in the first and last layers at 37 mm and 140 mm height from the bottom of the column respectively. For confinement steel, the strain gages were placed at the first, second and third tie sets from the bottom of the column.

a.) Specimen A1, B1, C1 and D1

b.) Specimen A2 and B2

Figure 29 Strain gages of axial reinforcements



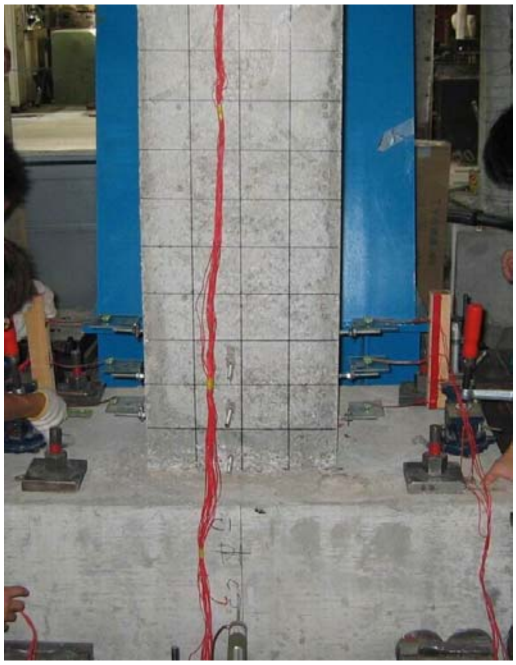
a.) Specimen A1, A2 and C1

a.) Specimen B1 and B2

a.) Specimen D1

Figure 30 Strain gages for confinement steels

a.) Strain gauging of reinforcement



b.) Curvature measurement

Figure 31 Instrumentation

All attached strain gages are connected with the data logger which obtains the strain of reinforcements during experiment. While the horizontal actuator is applying cyclic load at the top of column, the displacement is also measured controlled by the computer set.

Specimen Casting and Handling

This experiment was conducted at the Kawashima Laboratory, Department of Civil Engineering, Tokyo Institute of Technology, Tokyo, Japan from June 2003 until October 2003. The specimens were constructed at a workshop close to laboratory. The first specimen sets built were specimen A1, A2 and D1 and the second specimens set were B1, B2 and C1 respectively. All reinforcement steel was cut and bent following bar cutting list. The column steel tied with the transverse steel around main bars was installed then the steel of footing was placed.

The strain gages and electrical wire were installed on the reinforcements along the plastic hinge length. The steel cage of specimen was lifted and placed on the plywood surface. The formwork of column and footing made by wood was erected and coated by applying oil to plywood surfaces which were in contact with concrete. Four sleeves with galvanized threaded pipes diameter 75 mm were placed vertically inside the footing to fix the footing with the platform during the test with steel bars stressing force 25 tons each. At the top of specimens, four sleeves with galvanized threaded pipes diameter 50 mm were placed horizontally for fixing with actuators.

Casting concrete was ready-mixed concrete supplied by a local plant. In each batch, the concrete samples were kept for slump measure and for compressive strength test by six cube molds standard 150 mm dimension.

After moisture curing for 7 days, the specimens were moved to the laboratory. Then the test was conducted after casting concrete between 24 days to 36 days.

Test Observation

To investigate the experiment results, each specimen was investigated for the following:

1. Failure patterns such as crack patterns, spalling of the cover concrete, rupture of tie bars, local buckling and rupture of longitudinal bars.
2. Corresponding lateral force and lateral displacement hysteretic responses for every increment of displacement interval from 0.05 until failure
3. Strains in vertical reinforcement steel and tie reinforcements
4. Axial load level