Seismic Performance of Frame Retrofitted Utilizing Lateral Active Confinement Technique Together with Steel Braces

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SUMMARY:
A new seismic retrofitting method utilizing lateral active confinement technique together with K-shaped steel brace is proposed. The retrofit technique is that the tops and bottoms regions of existing RC columns constructed under the old standards are jacketed by steel plates actively utilizing pre-tensioned high strength steel bars, and then K-shaped steel brace and RC columns are simply jointed together through those pre-tensioned steel bars also. Cyclic lateral loading tests were carried out to confirm the difference of seismic performance between retrofitted specimen and non-retrofitted specimen. The retrofit specimen showed excellent seismic performance through experimental tests, even if the non-retrofitted specimen failed in a shear manner.

Keywords: seismic retrofit, active confinement, steel brace

1. INTRODUCTION

A seismic retrofitting design method of the dry type connection that the both end regions of an existing RC column constructed under the old standards were pressed by steel plates actively utilizing pre-tensioned high strength steel bars through steel corner blocks and then the column was simply jointed together with steel brace, was reported (Li and Esaki, 2010). Experimental results indicated that the lateral capacity of the retrofitted specimen was about 2.4 times of that of non-retrofitted one. And sufficient ductility of the retrofitted one could be expected even under large deformation. However, in order to put this kind of retrofitting method to practical use, it is necessary to verify whether the seismic performance of RC frame retrofitted utilizing this technique under cyclic loading test is effective or not.

In this study, the seismic retrofitting effect of a RC frame strengthened utilizing K-shaped steel braces was examined by comparison with the experimental result of a non-retrofitted one exhibited shear failure (Li, 2011).

2. TEST PROGRAM

The list of RC frame specimens is shown in Table 2.1. The mechanical properties of reinforcement utilizing in this experiment are listed in Table 2.2. The details of retrofitted specimen are illustrated in Fig. 2.1. The reinforcing details of column are also shown in this figure. The RC frame was assembled by two columns and two large stiffness H-shaped steel beams. As first, the columns were cast separately, and then the top stubs of them and the steel beam were joined together utilizing high strength steel bars. Subsequently, the bottom stubs were connected together by the same method as the top stub. In these two specimens, for the column, cross section is 200 mm × 200 mm, clear height is 800 mm, shear span to depth ratio is 2.0, and span length is 1,800 mm. In each column, 4-D13 has been arranged as longitudinal reinforcement (longitudinal reinforcement ratio, p_g = 1.27%) and reinforcing steel 4ϕ has been placed as hoop at 155 mm interval (hoop steel ratio, p_w = 0.08%). In this study, in order to confirm lateral force transfer mechanism between RC columns and steel braces, the

Table 2.1. Specimens

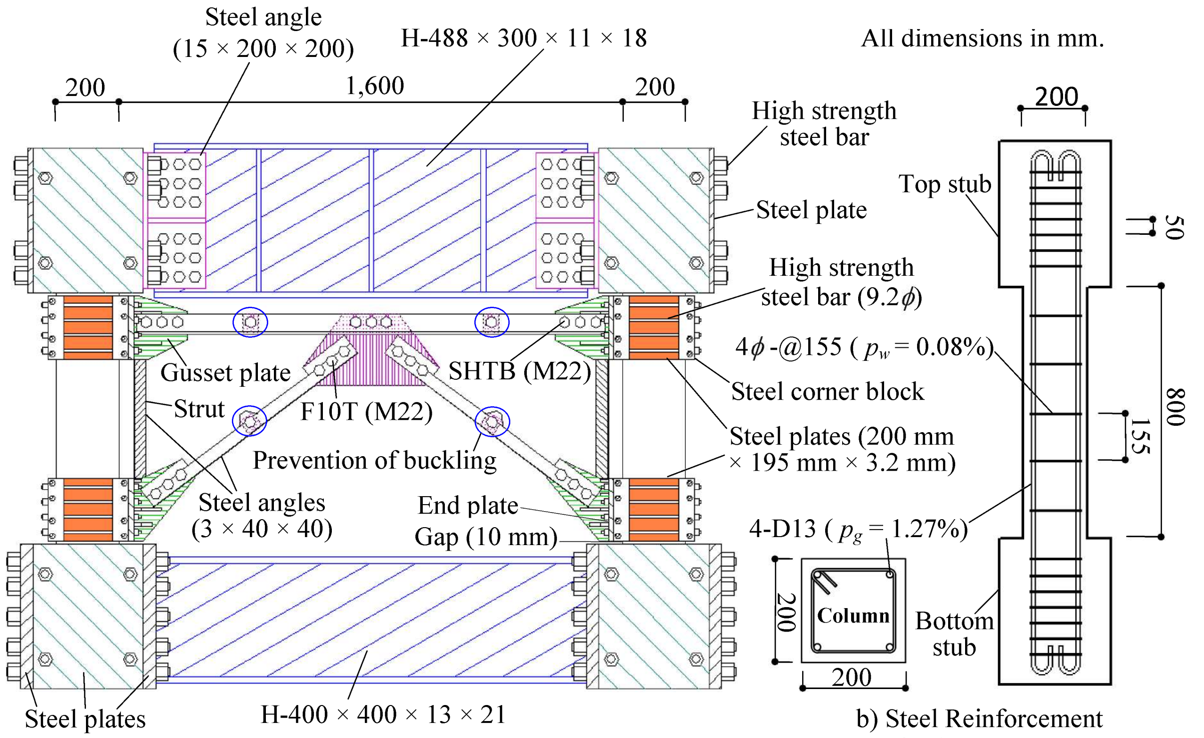
	10-F	11-FKB
Retrofit		By high strength steel bars, steel plates and K-shaped steel brace.
σ_B	27.8 MPa	30.2 MPa
Common details	Hoop: 4ϕ-@155 (p_w = 0.08%), Column rebar : 4-D13 (p_g = 1.27%), Axial force ratio $N/(b \cdot D \cdot \sigma_B)$ of per column = 0.	

Note: σ_B = concrete compressive strength, p_w = hoop steel ratio, p_g = longitudinal reinforcement ratio, N = axial force of column, b = width of column, D = depth of column.

Table 2.2. Properties of Reinforcement

Reinforcement			a (cm^2)	f_y (MPa)	ε_y (%)	E_S (GPa)
Rebar	D13	10-F	1.27	365	0.24	151
		11-FKB		334	0.19	179
Hoop	4ϕ		0.13	199	0.10	197
High strength steel bar	9.2ϕ		0.66	1080		201
Steel plate	3.2 mm			311	0.16	192
Steel angle	3 mm × 40 mm × 40 mm		2.34	317	0.18	173
	8 mm × 65 mm × 65 mm		9.76	309	0.16	197

Note: a = cross-sectional area, f_y = yield strength of steel, ε_y = yield strain of steel, E_S = modulus of elasticity.



a) Specimen of 11-FKB

b) Steel Reinforcement of column

Figure 2.1. Details of retrofitted specimen 11-FKB

initial axial force ratio of each column was set as zero.

10-F is a non-retrofitted test specimen. Test specimen 11-FKB is reinforced by steel plates, high strength steel bars and K-shaped steel braces. In this retrofit technique, steel plates of 200 mm (height) × 195 mm (width) × 3.2 mm (thickness) were pasted on to the top and bottom regions of columns from four sides by adhesive. And then, steel corner blocks, in which the holes were made beforehand, were placed in four corners of the column and outside of the steel plates. After that, high strength steel

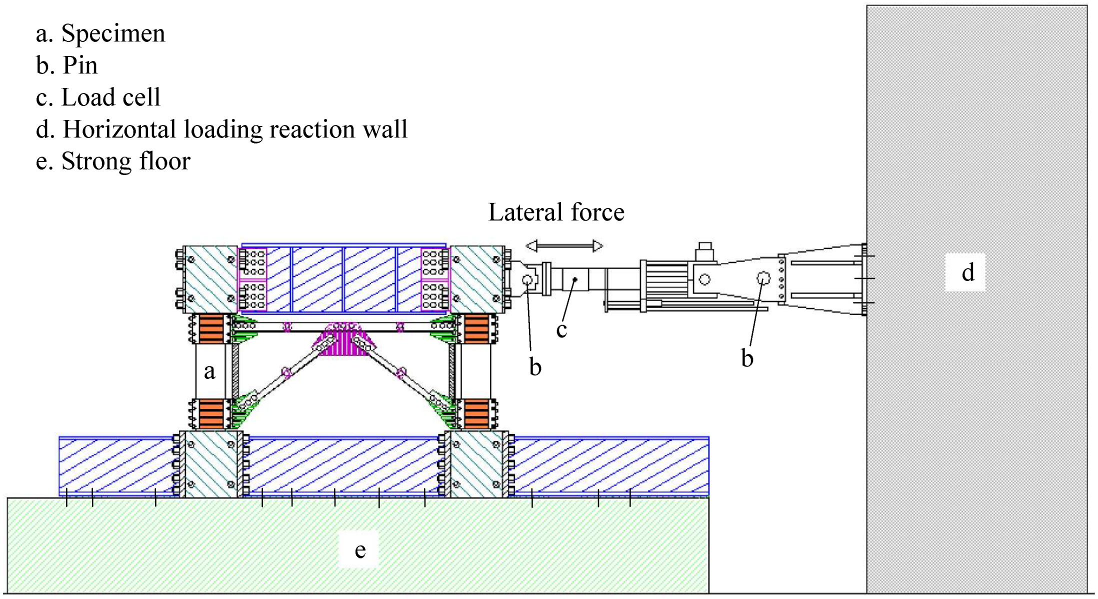


Figure 2.2. Details of test setup

bars of 9.2ϕ were penetrated through the holes. Finally the column was pressed by the steel plates utilizing high strength steel bar prestressing. The steel bars surrounded the column like external hoops. The prestressing was introduced into the steel bar by torque wrench through corner blocks and nuts. However, a gap of 10mm was set between steel plate and stub (see Fig. 2.1). And, the end steel plate with thickness of 16 mm that was welded to the gusset plate was also jointed to steel corner blocks utilizing pre-tensioned high strength steel bars (see Fig. 2.1). In addition, the end steel plates in top and bottom regions were jointed together by strut (two steel angles of 3 mm × 40 mm × 40 mm) utilizing 4 high strength bolts (M12). After that, the gusset plates were clamped by steel angles (3 mm × 40 mm × 40 mm) from both side utilizing 3 high strength bolts of M22 (F10T). In order to improve the lateral capacity of 11-FKB, it is important to prevent the slip of brace joints. To prevent the slip, it is a good way to increase the coefficient of friction for the brace joint surface. In this research, all of the brace joint surfaces had been made to be covered with red rust. And, the centre of the steel angle had been fixed on the steel plate in which a hole was made beforehand by a high strength bolt of M22 (F10T) in order to prevent the buckling of the brace (see Fig. 2.1). Moreover, brace joint was strengthened by welding steel plate near the bolt hole of steel angle for overcoming the partial loss of section. The distance between central of the bolts of diagonal member is regarded as the effective buckling length, and the slenderness ratio of the steel brace is 53 while both ends are considered as pin joints. The top chord was composed of two steel angles (8 mm × 65 mm × 65 mm), and it was connected to the gusset plates by super high strength bolts of M22 (SHTB). The buckling prevention in the same way as diagonal member had been made at the top chord member also (see Fig. 2.1). The diameter of every high strength steel bar retrofitted for column was 9.2ϕ. And the pretressing strain of the steel bars was about 4300μ. The prestressing force was about 57 kN per steel bar. This technique can be applied quickly and easily without utilizing heavy machinery on-site.

The test was carried out utilizing the loading apparatus illustrated in Fig. 2.2. Lateral loading cycles included three successive cycles at each story drift angle of R = 0.5%, 1.0%, 1.5%, 2.0%, 2.5% and 3.0%. The R is defined as δ/h. Where, δ is the story drift, and h is the clear height of column.

3. EXPERIMENTAL AND ANALYTICAL RESULTS

While retrofitted test specimen 11-FKB was set up, initial shear crack with width of 0.3 mm as shown in Fig. 3.1 formed in the middle of column, which was near loading jack and is defined as column B undermentioned. The observed crack patterns of both specimens after cyclic loading test are illustrated

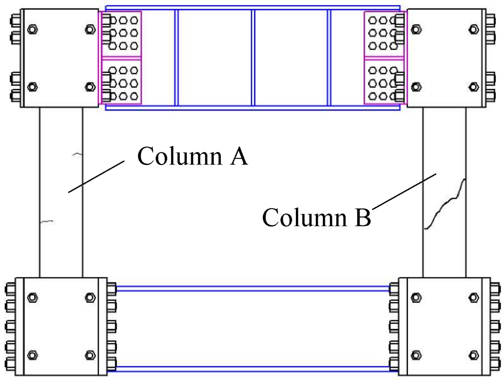


Figure 3.1. Observed cracking patterns of 11-FKB before cyclic loading test

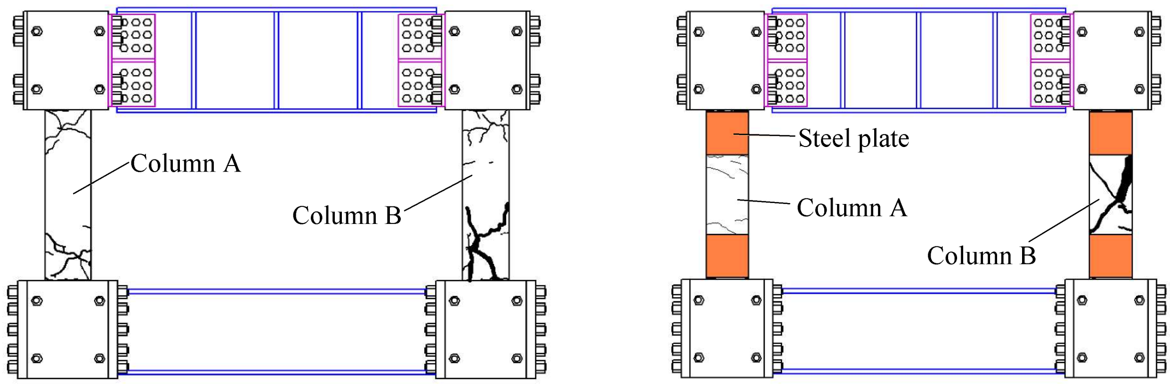


Figure 3.2. Observed cracking patterns of 10-F and 11-FKB after cyclic loading test

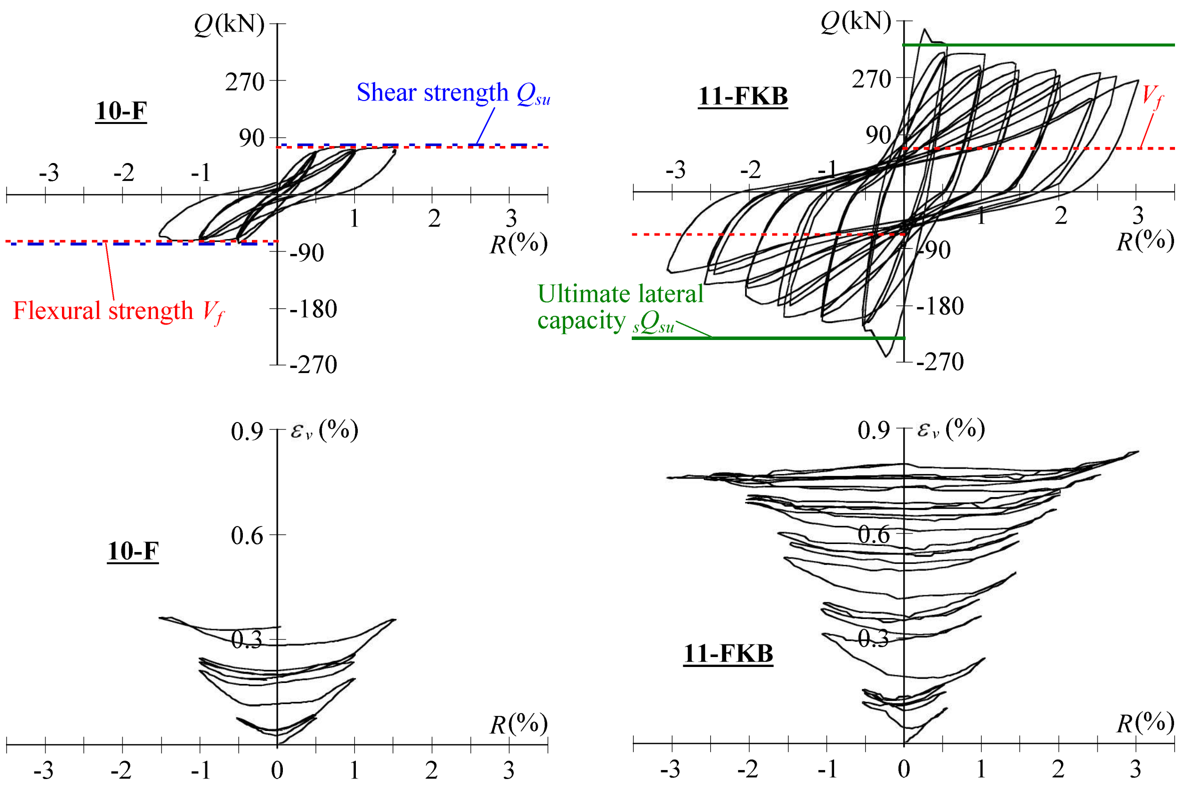


Figure 3.3. Measured *Q*-*R* and ε_v-*R* relationships of 10-F and 11-FKB

in Fig. 3.2. The relationship between experimental lateral force Q and story drift angle R of each test specimen is shown in Fig. 3.3. And the relationships between average strain ε_v along the column axis and R of these two specimens are also shown in this figure. In the Q-R curve of 10-F, the dotted line is calculated flexural strength of frame by the simplified equation V_f (Architectural Institute of Japan, 1999), and the dot-dash-line is ultimate shear strength of frame Q_{su} (The Building Disaster Prevention Association of Japan, 2001).

The V_f is defined by the following expression:

$$V_f = \frac{4M_u}{h} \tag{3.1}$$

Where M_u is the ultimate strength for bending moment of column, and h is the clear height of column.

In the non-retrofitted specimen 10-F, flexural cracks at the ends of columns and flexural-shear cracks near the ends of columns were observed when story drift angle R approximated 0.5%. The rebars around critical sections of columns yielded while R approached 1.0%, and the experimental lateral capacity ${}_{exp}Q_{max}$ of 77kN was over the V_f. Subsequently, the widths of flexural-shear cracks in the two columns increased suddenly while R was on the way to 1.5% (see Fig. 3.2) and this specimen failed in a shear manner because of poor transverse reinforcement. However, the flexural-shear cracks of the column B were wider than those of the column of opposite side, which is defined as column A undermentioned. Because of no initial axial force acting on the column, the decreasing of average vertical strain ε_v and lateral force resistance capacity of this specimen were not rapid while shear failure happened.

On the other hand, in the Q-R curve of 11-FKB, the horizontal solid line is ultimate lateral capacity ${}_sQ_{su}$ of frame (The Building Disaster Prevention Association of Japan, 2001) retrofitted by steel braces and the dotted line is V_f in case of non-retrofit. The ${}_sQ_{su}$ is the sum of the V_f and ultimate lateral strength of braces. Moreover, ${}_sQ_{su}$ can be expressed by Eqn. 3.2 below:

$${}_sQ_{su} = V_f + N_t \cdot cos\theta + N_c \cdot cos\theta \tag{3.2}$$

Where θ is slope angle of the steel brace, N_t and N_c shall be calculated utilizing Eqn. 3.3 and Eqn. 3.4 as follows respectively:

$$N_t = F \cdot A' \tag{3.3}$$

$$N_c = f_{cr} \cdot A' \tag{3.4}$$

Where A' is effective cross-sectional area of the steel brace (Architectural Institute of Japan, 2008), F is standard strength (yield strength) of brace whose unit is N/mm^2 and f_{cr} is limit compressive stress of brace (The Building Disaster Prevention Association of Japan, 2001). Moreover, f_{cr} is given as follows:

$$f_{cr} = \left[1 - 0.4\left(\frac{\lambda}{\Lambda}\right)^2\right] \cdot F \quad (\lambda \le \Lambda) \tag{3.5}$$

$$f_{cr} = \frac{0.6F}{\left(\frac{\lambda}{\Lambda}\right)^2} \quad (\lambda > \Lambda) \tag{3.6}$$

Where λ is effective slenderness ratio, and Λ is critical slenderness ratio (The Building Disaster

Prevention Association of Japan, 2001), which should be calculated utilizing the following formula:

$$\Lambda = \pi \cdot \sqrt{\frac{E}{0.6F}} \tag{3.7}$$

Where E is modulus of elasticity for the steel brace.

In test specimen 11-FKB, it is feared that punching failure shall happen at the ends of both columns. Therefore, it is necessary to check the punching shear strength ${}_pQ_c$ of this specimen. The ${}_pQ_c$ is defined by the following expression (The Building Disaster Prevention Association of Japan, 2001):

$${}_pQ_c = K_{min} \cdot \tau_o \cdot b_e \cdot D \tag{3.8}$$

Where b_e is the effective width considering the transverse member of the column subjected to punching shear, D is the depth of the column subjected to punching shear and K_{min} can be estimated as:

$$K_{min} = \frac{0.34}{0.52 + a/D} \tag{3.9}$$

Where a is the distance from the acting point, where lateral force is transferred from steel brace to column assuming that the force acts concentratively, to boundary surface between column and beam. Because of high confinement of the end regions for the column, a was considered as the gap between the steel plate and the stub in this study. On the other hand, τ_o is given as follows:

$$\tau_o = 0.98 + 0.1F_{c1} + 0.85\sigma \quad (0 \le \sigma \le 0.33F_{c1} - 2.75) \tag{3.10}$$

$$\tau_o = 0.22F_{c1} + 0.49\sigma \quad (0.33F_{c1} - 2.75 < \sigma \le 0.66F_{c1}) \tag{3.11}$$

$$\tau_o = 0.22F_{c1} + 0.49 \times 0.66F_{c1} \quad (\sigma > 0.66F_{c1}) \tag{3.12}$$

Where F_{c1} is design strength of concrete for existing structure. And, σ shall be expressed by Eqn. 3.13 below:

$$\sigma = p_g \cdot f_y + \sigma_o \tag{3.13}$$

Where p_g is the ratio of the total main reinforcement cross-sectional area for the column to $b_e \cdot D$. In addition, σ_o can be calculated by the following expression:

$$\sigma_o = \frac{N}{b_e \cdot D} \tag{3.14}$$

Where N is the axial force of column at mechanism whose value is positive in case of compression. In this study, ${}_pQ_c$ was equal to 363kN. And, it was over the ${}_sQ_{su}$, 272kN.

In the test specimen 11-FKB reinforced by steel plate, high strength steel bar and K-shaped steel braces with friction-type high-tension bolted joints, lateral stiffness and lateral force increased rapidly at the initial period. And the experimental lateral capacity in the positive loading direction ${}_{exp}Q^{+}_{max}$, 258kN, was measured at story drift angle R = +0.27%. At the same time, the compressive steel brace buckled and lateral force decreased rapidly. The deflection of the top chord increased rapidly at that time also. And the longitudinal reinforcement of column base (Column B) in the tensile side yielded. Moreover, the experimental lateral capacity ${}_{exp}Q_{max}$ reached the ${}_sQ_{su}$, and the ${}_{exp}Q_{max}$ was about 3.8 times of the V_f. After that, at R=0.5%, the flexural cracks at all the ends of columns and a new narrow shear crack at the region without retrofitting by steel plate of column B were observed. Subsequently, in the negative loading direction, buckled brace was tensioned and experimental lateral force reached the lateral capacity of the negative side, -262kN, at R=-0.24%. Simultaneously, the longitudinal reinforcement at both ends of column A yielded also. And then, for the loading of the positive and negative directions, the experimental lateral force of this specimen was declining gradually with the increasing of R. And, the width of initial shear crack which happened in the column B before cyclic loading test was suddenly increased at R=-1.5% to -2.0%. At R=3.0%, a crack in the steel angle at the initial buckled position was observed, and the experiment was discontinued. With regard to this specimen, there was hardly any damage except for plastic hinge zones in the column A, however, shear failure happened and big damage was observed in the column B. It seemed that this kind of failure mechanism was due to the initial damage of the column B when 11-FKB was set up. On the other hand, the slip not only between steel corner blocks and steel plates but also between gusset plates and steel angles did not occur during cyclic loading test. Moreover, the increment of tensile strain ε_v of this specimen with the increase of R was more remarkable than that of 10-F.

The presentation of measured strain of hoops and steel angles for specimen 11-FKB is illustrated in Fig. 3.4, and these measured results are also shown in this figure. In the retrofitted specimen, the most critical zones are the centre regions of two columns. The variation of the strain of hoop located in the centre of column A (No. 1) was small with the increasing of R. It was proven that column A was secure after retrofitting utilizing this kind of technique. However, the hoop in the same position for column B (No. 2) yielded. This result was due to the initial damage of this column as 11-FKB was set up. It is desirable that the centre region of the column is also pressed by steel plates actively utilizing pre-tensioned high strength steel bars through steel corner blocks, in preparation for the coming huge earthquake, such as 2011 Tōhoku earthquake and tsunami. On the other hand, the strain of every measurement position almost reached the yielding level in the tensile side for the diagonal members.

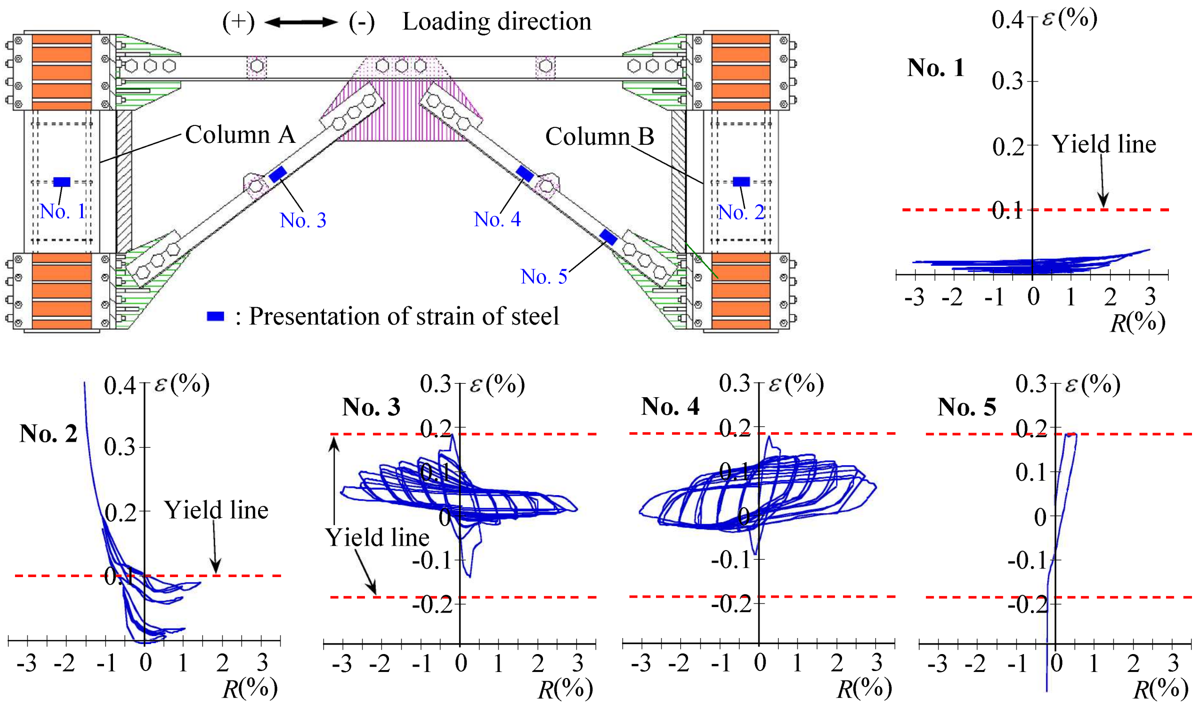


Figure 3.4. Measured strain of hoops and steel angle members

4. DETERMINATION OF PRETENSION FORCES OF STEEL BAR AND BOLT

In order to improve the lateral capacity of 11-FKB, it is important to prevent the slip not only between steel corner blocks and steel plates but also between gusset plates and steel angles of brace joints. To prevent the slip, there are two essentials which should be grasped. One is to increase the coefficient of friction for connection. In this study, all of the joint surfaces had been made to be covered with red rust that was mentioned above. And the coefficient of friction is bigger than 0.45 (Architectural Institute of Japan, 2006). For safety, it was considered as 0.45 in this research. The other is to determine the initial pretension forces of high strength steel bars and high strength bolts. Since the sliding of steel brace connection has to be prevented, the frictional resistance due to the tensile forces of high strength bolts shall be larger than the tensile force of the brace for design of 11-FKB. And, the relationship between them can be expressed by Eqn. 4.1 below:

$$A \cdot \sigma_{at} \leq m \cdot \mu_1 \cdot N_{tt} \tag{4.1}$$

Where A is total cross-sectional area of the steel brace, σ_{at} is the average tensile stress of the cross section for brace, m is the number of the shear surface, μ_1 is the coefficient of friction for brace connection and N_{tt} is total tensile force of high strength bolts. However, considering the tensile force of high strength bolt tends to loosen during cyclic loading, Eqn. 4.2 is recommended

$$A \cdot \sigma_u \leq n_b \cdot m \cdot \mu_1 \cdot N_i \tag{4.2}$$

Where σ_u is the fracture stress of steel brace, n_b is the number of the high strength bolts and N_i is initial pretention force of high strength bolt. In this study, N_i was determined according to torque control method. And, the N_i can be calculated by the following expression:

$$N_i = \frac{T_r}{k \cdot d_1} \tag{4.3}$$

Where T_r is tightening torque for high strength bolt, k is the torque coefficient and d_1 is the nominal diameter of bolt.

The frictional resistance between the end steel plate and the steel corner block is shown in Fig. 4.1.

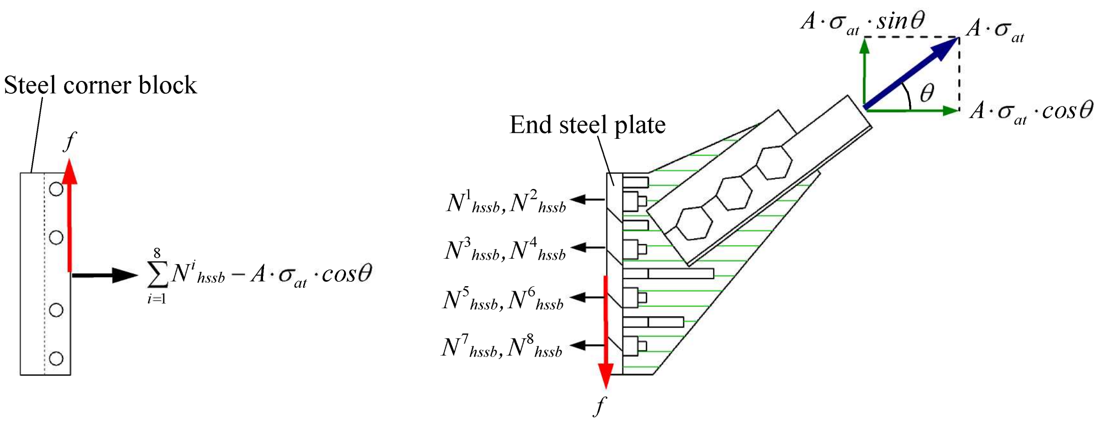


Figure 4.1. Frictional resistance between end steel plate and steel corner block

For safety, the effect of strut is disregarded in this resistance mechanism. And, Eqn. 4.4 shall be satisfied.

$$A \cdot \sigma_{at} \cdot sin\,\theta \leq f \tag{4.4}$$

Where f is the friction resistance between steel corner block and end steel plate which can be calculated utilizing the following formula:

$$f = \mu_2 \cdot \left(\sum_{i=1}^{8} N^i{}_{hssb} - A \cdot \sigma_{at} \cdot cos\,\theta \right) \tag{4.5}$$

Where μ_2 is the coefficient of friction between steel corner block and end steel plate and $N^i{}_{hssb}$ is the tensile force for the high strength steel bar of number i. But, considering the tensile force of high strength steel bar tends to loosen during cyclic loading, Eqn. 4.6 is recommended

$$A \cdot \sigma_u \cdot sin\,\theta \le \mu_2 \cdot \left(n_{hssb} \cdot N_{hssbi} - A \cdot \sigma_u \cdot cos\,\theta \right) \tag{4.6}$$

Where n_{hssb} is the number of the high strength steel bars, N_{hssbi} is the initial pretension force of steel bar. In this study, $A \cdot \sigma_u \cdot sin\,\theta$ was equal to 126kN. And, it was under the $\mu_2 \cdot \left(n_{hssb} \cdot N_{hssbi} - A \cdot \sigma_u \cdot cos\,\theta \right)$, 132kN.

5. DISCUSSION ON TOP CHORD

While a lateral force is applied to 11-FKB, one brace is tensioned, and the other one is compressed. Within a small R, buckling of the compressive brace does not accrue, and the axial force of tension brace F_t tends to be identical with the axial force of compressive brace F_c. After buckling happens, the difference in vertical component between F_t and F_c tends to increase, and the deflection of the top chord becomes larger with the increase of R. While this difference is larger than the plastic collapse load P_u, the plastic hinge will accrue at top chord. In this study, two kinds of failure mechanisms as shown in Fig. 5.1 are discussed. In model a, two ends are considered as fixed joints. On the other hand, in model b, two ends are considered as pin joints. The plastic collapse load of model a, P_{ua}, is larger than that of model b, P_{ub}. The P_{ub} can be calculated according to principle of virtual displacement, and it shall be expressed as follows:

$$P_{ub} = \frac{4M_{ptc}}{L} \tag{5.1}$$

Where L is the effective length of top chord (c/c spacing of joints) and M_{ptc} is the full plastic moment of top chord which can be calculated utilizing the following formula:

$$M_p = f_{ytc} \cdot Z_{ptc} \tag{5.2}$$

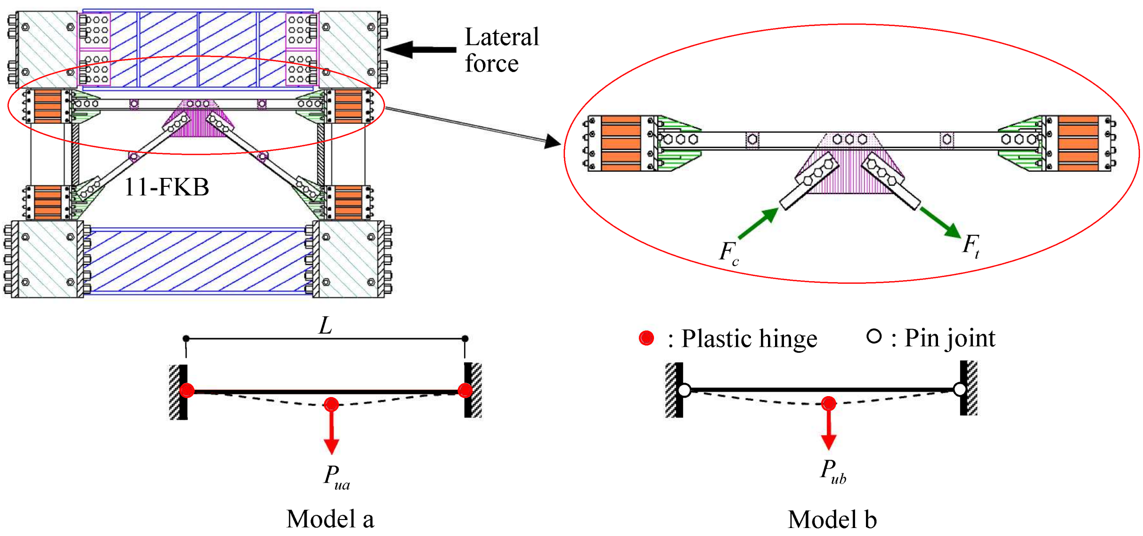


Figure 5.1. Failure mechanism of top chord

Where f_{ytc} is the yield strength of top chord and Z_{ptc} is the section modulus of it.

In order to expect the seismic performance of this kind of retrofit technique, it is necessary to keep the top chord in a state of elastic. And the P_{ub} is suggested to be satisfied with Eqn. 5.3 given as below:

$$P_{ub} \geq \left(F - f_{cr}\right) \cdot A \cdot sin\,\theta \tag{5.3}$$

Where F is standard strength of brace and f_{cr} is limit compressive stress of it. In this study, P_{ub} was equal to 15.2kN. And, it was over the $\left(F - f_{cr}\right) \cdot A \cdot sin\,\theta$, 11.3kN.

6. CONCLUSIONS

An existing RC frame constructed under the old standards, the ends of columns were jacketed by steel plates actively utilizing pre-tensioned high strength steel bars through steel corner blocks and then the columns were simply jointed together with K-shaped steel brace, was tested under the reversed cyclic lateral force. The results of this experiment lead to the following conclusions:

1) For the retrofitted specimen, lateral capacity and lateral stiffness were improved remarkably. This improvement of such seismic performance was mostly attributed to K-shaped steel brace.

2) It is recommended that the center region of the column is also pressed by steel plates actively utilizing pre-tensioned high strength steel bars through steel corner blocks, in preparation for the coming huge earthquake.

3) The slip not only between steel corner blocks and steel plates but also between gusset plates and steel angles can be avoided if the pretension forces of high strength steel bars and high tension bolts are reasonable.

4) In order to expect the seismic performance of this kind of retrofit technique, it is recommended that the top chord is in a state of elastic.

5) The maximum lateral strength, which can be expected in this retrofit technique, shall be the sun of punching shear strength of column of both sides.

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