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Method of reinforcement for joints between steel roofs and RC columns in existing buildings

Yuko Shimada[a,*], Satoshi Yamada[b], Shoichi Kishiki[b], Takashi Hasegawa[c], Toru Takeuchi[d]

[a] Faculty of Engineering, Chiba University, 1-33, Yayoi-town, Inage-Ward 263-8522, Chiba, Japan
[b] FIRST, IIR, Tokyo Tech, 4259 Nagatsuda, Midori-Ward, Yokohama 226-8503, Japan
[c] Building Research Institute, 1 Tachihara, Tsukuba 305-0802, Japan
[d] Department of Archi. and Building Eng., Tokyo Tech, 2-12-1 O-okayama, Meguro-Ward, Tokyo 152-8550, Japan

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ABSTRACT

The aim of this study was to validate the reinforcement method used for joints connecting a steel roof and a reinforced concrete (RC) frame in existing buildings based on a series of cyclic loading tests. Three reinforcement methods were investigated in the study: one using a steel jacket and high-strength steel rods (Type-1), another using a steel jacket and post-installed adhesive anchors (Type-2), and a third using a steel plate and post-installed adhesive anchors (Type-3). The results obtained show that the Type-1 method is most effective in increasing force, because shear force can be transferred from the RC frame to the steel roof; thus, a steel jacket can support the entire top of the RC column and integrate the roof joints into the RC frame. The Type-2 and Type-3 methods, however, are not effective in the reinforcement of RC columns because shear force cannot be transferred, and concrete failure occurred easily.

1. Introduction

Steel roofs on reinforced concrete (RC) frames are widely used in open-space structures, such as gymnasiums, stadiums, stations, and airports. A typical type of damage that is induced in such structures is concrete edge failure at the joint between the RC frame and the steel roof. Concrete edge failure in school gymnasiums was reported in the 2011 Tohoku Earthquake [1], the 2013 Lushan Earthquake [2], and the 2016 Kumamoto Earthquake [3], as shown in Fig. 1. It was known that these concrete edge failures can easily occur in joints and column bases with insufficient edge distance [4–6]. Concrete edge failure was affected by edge distance and the number of anchor rods and reinforcing bars [5]. If concrete edge failure occurs, the lateral force is not transferred from the steel roof to the RC frame. As a result, the steel roof swings considerably [7], suspended members and nonstructural walls fall down. This is dangerous as it could lead to a breakout concrete block that may fall from overhead.

It has also been shown that concrete edge failure can be prevented with a sufficient edge distance based on conducting tests of the roof joints [8]. Only in new buildings can a sufficient edge distance be set; however, there are roof joints with an insufficient edge distance in many existing buildings. This is because RC frame and steel roof were designed separately; therefore, the concrete edge failure of roof joints resulting from the large lateral force caused by the relative displacement between the RC frame and the steel roof, as shown in Fig. 2, was not considered during the design. These roof joints were often placed at a position offset in the top of RC column for the inner direction of the building, to avoid interfering with the reinforcing bars of the concrete beam. As a result, the edge distance of the roof joint is insufficient. Another issue in existing buildings is that concrete shear failure occurs easily in roof joints with thick concrete cover at the top of the RC column. Separate design between RC frame and steel roof also made the issue; for example, concrete cover added to adjust of steel roof elevation. Roof joints with an insufficient edge distance or a thick concrete cover need to be reinforced to increase resistance force and prevent concrete failure. Specific reinforcement measures for roof joints have not been generally established. Some retrofit techniques of concrete columns used in existing buildings or bridges are useful to roof joints. To consider rapid retrofit of roof joints in public buildings, only techniques executable from interior of a building are considered as the reinforcement of roof joints.

Steel jacketing retrofit (i.e., steel plates surround the entire of concrete column and the gap between steel plate and concrete column is filled with grout) is effective to enhance the ductility of the concrete column [9]. The techniques and design criteria of steel jacketing retrofit have been shown only for the circular jacket and the elliptical jacket for

* Corresponding author.
E-mail address: yshimada@faculty.chiba-u.jp (Y. Shimada).

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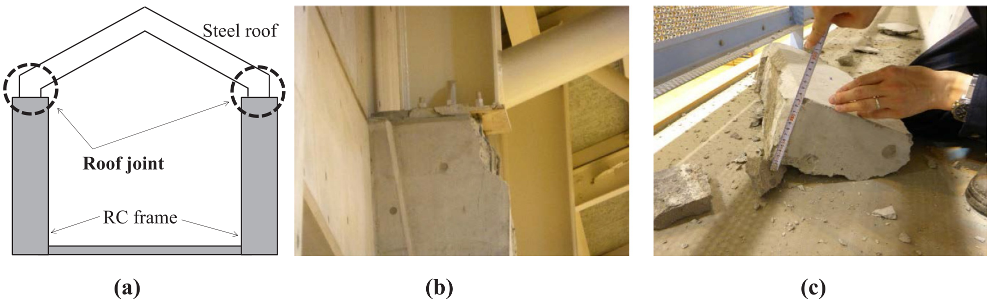


Fig. 1. (a) Location of the roof joint, (b) edge failure of roof joint, and (c) concrete block falling from the roof joint.

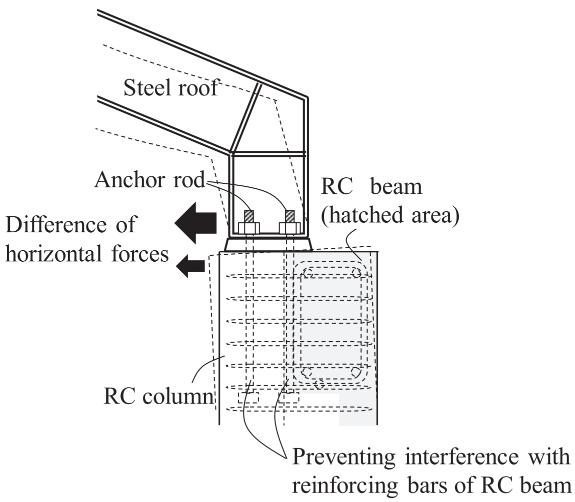


Fig. 2. Deformation and position of roof joint.

rectangular column, but not for the rectangular jacket [9]. A new type of steel jacketing retrofit of concrete column without filled grout was presented recently in [10]. This no-grout steel jacket has advantageous energy dissipation capacity and workability; however, the technique was evaluated only for circular columns.

A series of tests on steel jacketing retrofit using steel plate, steel angle, and bolts for a rectangular column was presented in [11,12]. This research also tested U-shaped partial steel jacket with through bolts and C-shaped partial steel jacket with adhesive anchor bolts [11]. The retrofit with U-shaped jacket showed a similar response to the whole steel jacket; the retrofit with C-shaped jacket did not prevent shear failure and did not improve displacement ductility. The results presented above suggest that techniques of partial steel jacketing retrofit are useful for roof joints; however, some improvements to prevent early failure and to grow energy dissipation capacity with simple jacket and fixing method are needed.

The purpose of this study was to validate the reinforcement method for the roof joint to prevent concrete edge failure. Based on a previous studies and damage survey, three reinforcement methods were selected. Each reinforcement method has some advantages and disadvantages in terms of strength and workability. The effect of each method was evaluated from elastic area to failure through a cyclic loading test.

2. Reinforcement methods

Three methods of reinforcement against a roof joint as shown in Fig. 3(a) examined in this work are as follows.

(1) Type-1 uses a steel jacket surrounding the top of the RC column, as shown in Fig. 3(b). In this type, it is possible to transfer force from the steel roof to the RC frame, as shown by the black arrows in Fig. 3(b), because the steel jacket surrounds the entire top of the RC frame and high-strength steel rods anchor to the RC frame. The high-strength steel rods need to be positioned appropriately to be able to transfer force to the RC frame. The construction works for this reinforcement method require attention to the force of the steel jacket and position of the high-strength steel rods.
(2) Type-2 uses a steel jacket support fixed by post-installed adhesive anchors on two sides of the RC column, as shown in Fig. 3(c). In this type, it is possible to transfer lateral force from the steel roof to the RC frame, as shown by black arrows in Fig. 3(c), by the shear resistance of the post-installed adhesive anchors after the concrete failure of the roof joint occurred. The construction works for this reinforcement method require attention to set the post-installed adhesive anchors. To use the steel jacket effectively, these anchors should be installed with an embedded length to reach farther inside than the reinforcing bars of the RC column. To avoid concrete edge failure, the position of the post-installed adhesive anchors should be set between two anchor rows.
(3) Type-3 uses a steel plate set at the front of the top of the RC column with post-installed adhesive anchors, as shown in Fig. 3(d). In this type, lateral force can transfer from the steel roof to the RC frame by the pull-out resistance of the post-installed adhesive anchors. If the post-installed adhesive anchors are installed with an embedded length to reach farther inside than the reinforcing bars of the RC column, they are effective at preventing concrete failure. The construction work for this reinforcement method only requires embedding the anchors in front of the RC column; however, the post-installed adhesive anchors should be installed to reach inside the RC beam. The position of post-installed adhesive anchors should be set between vertical anchor rods and reinforcing bars.

3. Cyclic loading test

3.1. Test specimen

To evaluate the effectiveness of the reinforcement methods described above, a series of cyclic loading tests in the roof joint were conducted. The specimens represented the joint between the steel roof and the top of the RC column with four anchor rods. In all specimens, the details of the top of the RC column and anchors were common. The section of concrete of the specimens had eight longitudinal reinforcing bars with 25-mm nominal diameter. The reinforcing bars were welded to a steel plate with a 25-mm thickness in the bottom of the specimen. The transverse stirrups with 10-mm nominal diameter had a pitch from 20 to 70 mm, as shown in Fig. 4. The base mortar in the 270 × 270-mm section was filled between the RC column and the base plate of the steel roof with a 50-mm thickness. The mortar was cast one week after the RC column. The depth from the top surface of the RC column to the top of the reinforcing bar was 100 mm, which is twice the minimum requirement of the thickness of cover concrete in Japan.

Four M20 rolled threaded anchor rods with 500-mm lengths (embedded length of 420 mm) were used. A nut was attached at the bottom of each anchor. The edge distance was 125 mm because the anchor

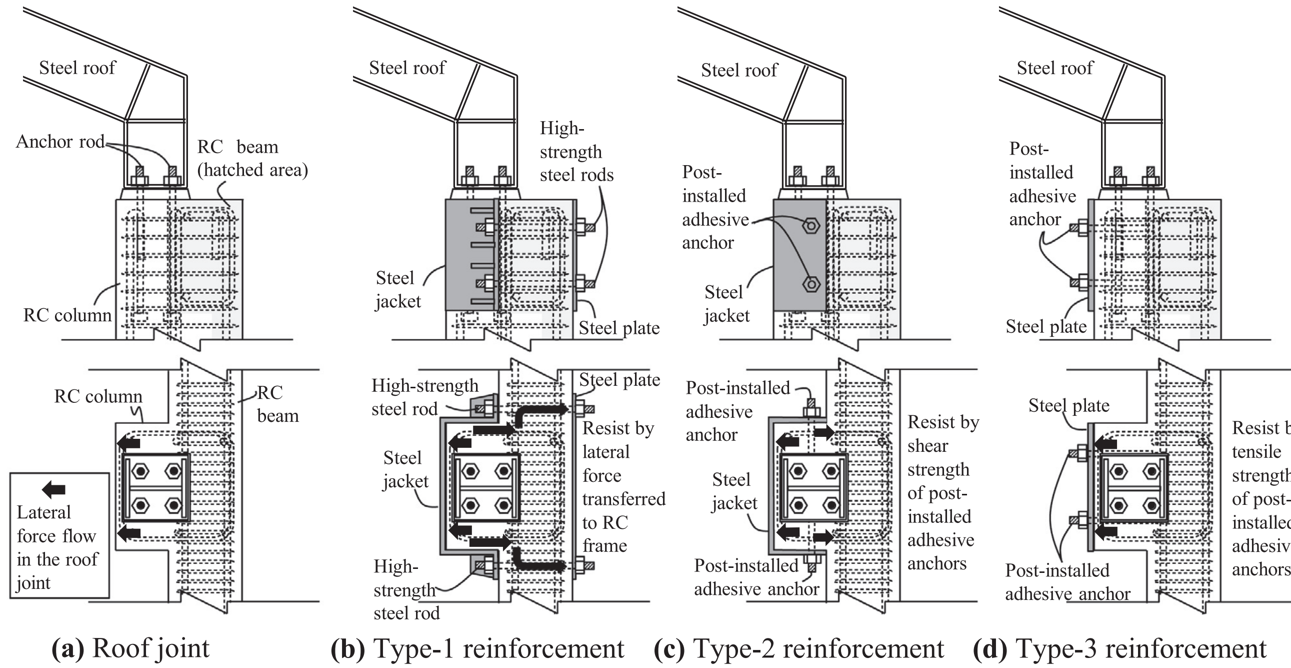


Fig. 3. (a) Roof joint without any reinforcement, (b) roof joint with reinforcement using steel jacket and high-strength steel rods (Type-1), (c) roof joint with reinforcement using steel jacket and post-installed adhesive anchors parallel to RC beam (Type-2), and (d) roof joint with reinforcement using a steel plate and post-installed adhesive anchors on the interior side of RC column (Type-3).

group at the roof joint in a general gymnasium in Japan is placed at a position offset from the center of the RC column to the inside of the building. This edge distance represents the detail of the roof joint in the gymnasium that experienced edge failure during an earthquake. In this study, the two anchor rods closer to the front of the specimen are called the "forward anchors," and the other two anchor rods are called the "rear anchors," as shown in Fig. 4(a).

Table 1 shows the material property list of the concrete and base mortar. The compressive force of the concrete was designed to be 24 N/mm^2 to represent that of general RC frames of gymnasiums in Japan.

The forces of the anchor rods, stirrup, high-strength steel rod, and steel plate are shown in Table 2.

There were five specimens in this test, as shown in Fig. 5. E-01 shows a specimen without reinforcement, that represents the common part, as shown in Fig. 3. E-02 is a specimen with Type-1 reinforcement attached to a common part, and E-03 is a specimen with Type-2 reinforcement attached to a common part. Both E-04 and E-05 are specimens with Type-3 reinforcement attached by post-installed adhesive anchors to common parts. E-04 specimen has a large embedded length, while E-05 specimen has a shorter one. This is the main difference between these two specimens.

Table 1
Material properties of concrete and mortar.

Concrete				Mortar		
Age (Day)	Comp. strength (N/mm^2)	Fracture strength (N/mm^2)	Young's modulus (N/mm^2)	Test date (Day)	Comp. strength (N/mm^2)	Fracture strength (N/mm^2)
34	27.0	2.75	25,000	27	16.7	1.93
57	29.4	3.02	25,000	50	21.1	2.16

*Each value is the average of three test pieces.

The details of reinforcement in each specimen are as follows. The E-01 specimen was designed so that edge failure of concrete occurred. The maximum of shear force of the specimens was expected to be determined by the edge failure of concrete in front of the forward anchors, as shown by the broken line in Fig. 4.

The E-02 specimen (Type-1) was designed so that lateral force from the steel roof can transfer to RC frame by a steel jacket surrounding the

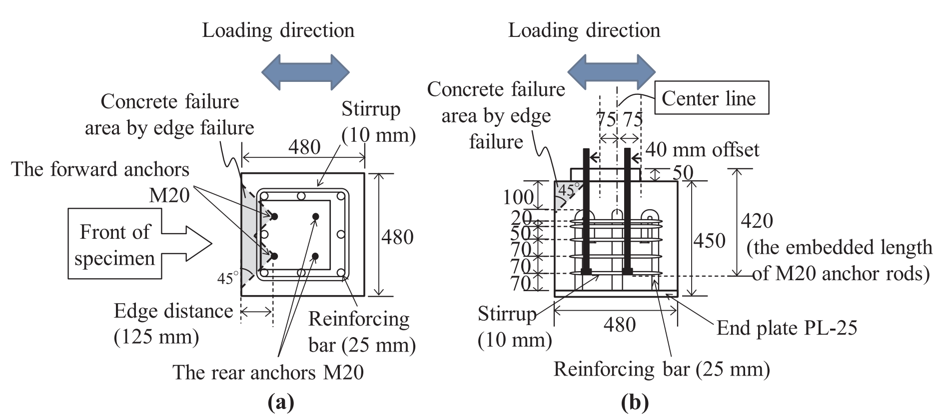


Fig. 4. (a) Top view of common part in specimen and (b) side view of common part in specimen.

Table 2
Material properties of anchor rods, high-strength steel rod, and steel plate.

Application		Yield strength [N/mm^2]	Tensile strength [N/mm^2]
Anchor rod	M20 (Specified min. tensile strength 400 N/mm^2)	358	449
Reinforcing bar	25 mm* (Specified min. yield strength 345 N/mm^2)	390	560
Stirrup	10 mm* (Specified min. yield strength 295 N/mm^2)	339	472
High-strength steel bar	13 mm* (Specified min. yield strength 785 N/mm^2)	1039	1115
Post-installed adhesive anchor	22 mm* (Specified min. yield strength 345 N/mm^2)	393 (E-03)	548 (E-03)
		396 (E-04&05)	552 (E-04&05)
Steel restraint & steel plate	PL-9 (Specified min. yield strength 325 N/mm^2)	412	524

* Nominal dinameter.

entire top of the RC frame. In the specimen, a steel jacket with a height of 400 mm from the top surface of the RC column was attached in the inner front, and both sides of the RC column. Its height was designed to cover the area affected by concrete edge failure, as shown by the gray area in Fig. 4. Four high-strength steel rods (13 mm in nominal diameter) were set to get reaction force from the jig. The diameter was determined because two steel rods could retain elastic under the estimated concrete edge failure force, which was described in section 3.3. To transfer lateral force from the entire steel jacket to RC frame, two high-strength steel rods were installed on the each of upper and lower rows. The upper two high-strength steel rods were 65-mm position from the top surface of RC column, and the lower two high-strength steel rods were 335-mm position from the top surface. Low prestress for each high-strength steel rod (about 1kN) was set to support the roof joint against the lateral force by only the steel jacket. This was set to confirm that Type-1 reinforcement method was useful even in the case of reinforcing the existing building with thin RC walls which cannot apply pre-tension in the high-strength steel rods.

The E-03 specimen (Type-2) was designed so that lateral force from the steel roof can transfer to RC column by shear resistance of the post-installed adhesive anchors through a steel jacket. In the specimen, a steel jacket was attached in the three inner sides of the RC column, similar to the E-02 specimen. Four post-installed adhesive anchors (22 mm in nominal diameter) were set to anchor the steel jacket to both sides of the RC column. Two post-installed adhesive anchors were 65-mm position from the top surface of RC column, and the other two were 335-mm position from the top surface. The effective embedded length of these post-installed adhesive anchors was determined by the edges of them reaching the M20 anchor rod position to resist the concrete edge failure of the RC column by their lateral force.

The E-04 and E-05 specimens (Type-3) were designed so that lateral force can transfer from the steel roof to the RC frame by the pull-out resistance of the post-installed adhesive anchors. In the both specimens, a steel plate with a 400-mm height was attached only inner front of the RC column by four post-installed adhesive anchors (22 mm in nominal diameter). For the E-04 and the E-05 specimens, two post-installed adhesive anchors were 65-mm position from the top surface of RC column, and the other two were position from the top surface. In the E-04 specimen, the effective embedded length of these post-installed adhesive anchors was determined by their edges being farther in than the rear anchor position to resist concrete edge failure of the RC column resulting from pull-out. In the E-05 specimen, however, these post-installed adhesive anchors were about half of the edge distance to examine the difference of maximum force based on the embedded length.

3.2. Test procedure

The test setup is shown in Fig. 6 and the three-dimension condition of test setup is also shown Fig. 7. The test apparatus is consisting two orthogonal rigid steel frames as shown in A–A′ section of Fig. 6. The bottom plate of the specimen was connected by high-strength bolts to the loading block, which could move in a horizontal direction. A horizontal load was applied by hydraulic jacks that were connected to the loading block supported on the sliders. The top the specimen was connected to the jig, which represented the base plate of the steel roof. The base plate was 250 × 250 mm square with 40-mm thickness. The

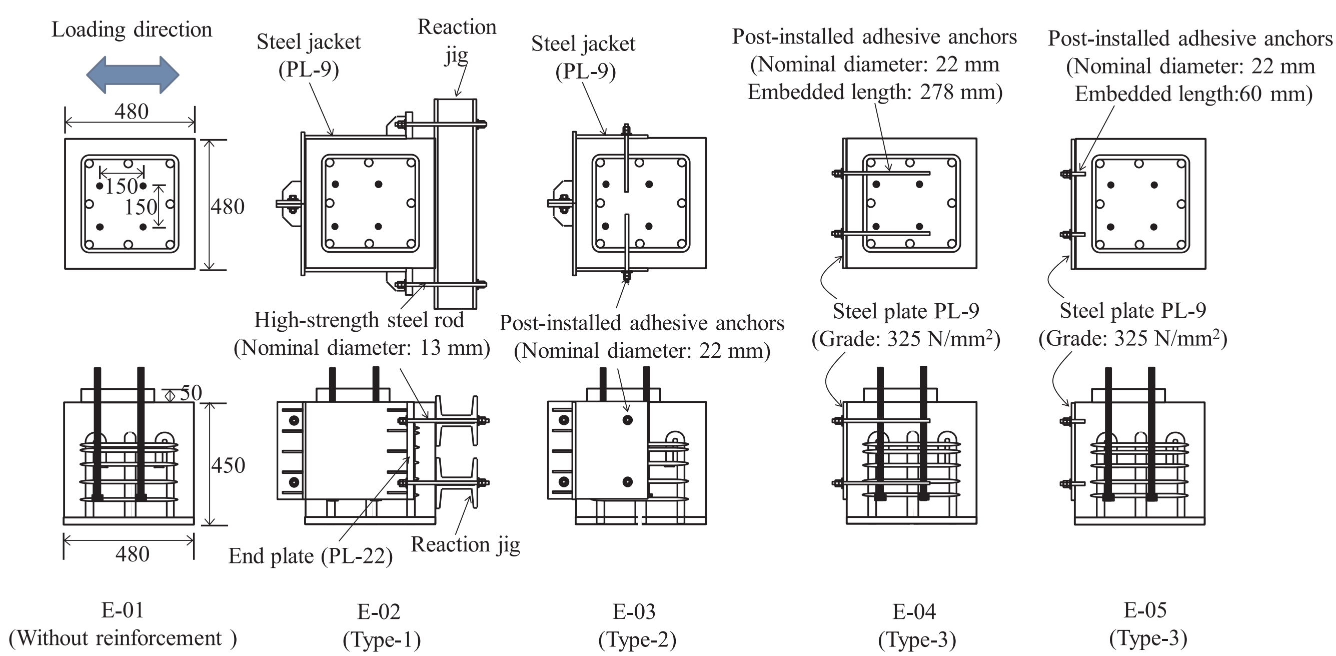


Fig. 5. Specimen detail.

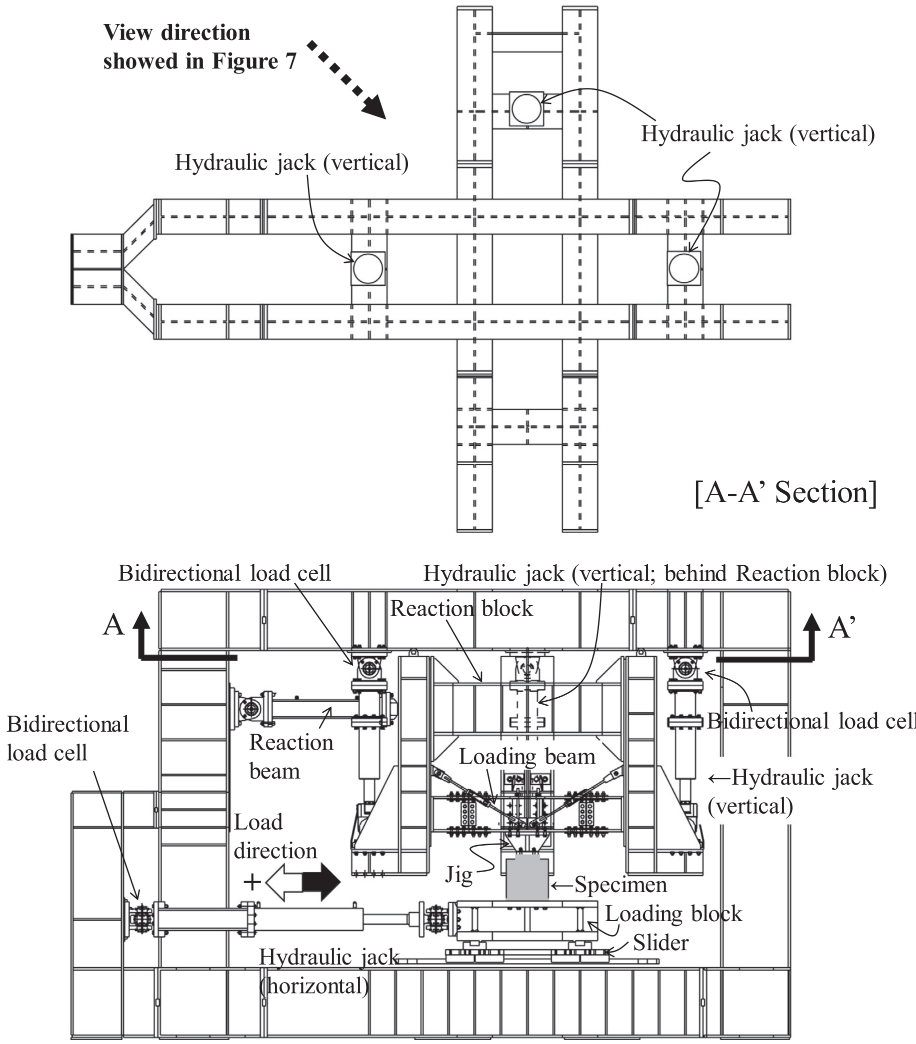


Fig. 6. Test setup (A-A' section and side view).

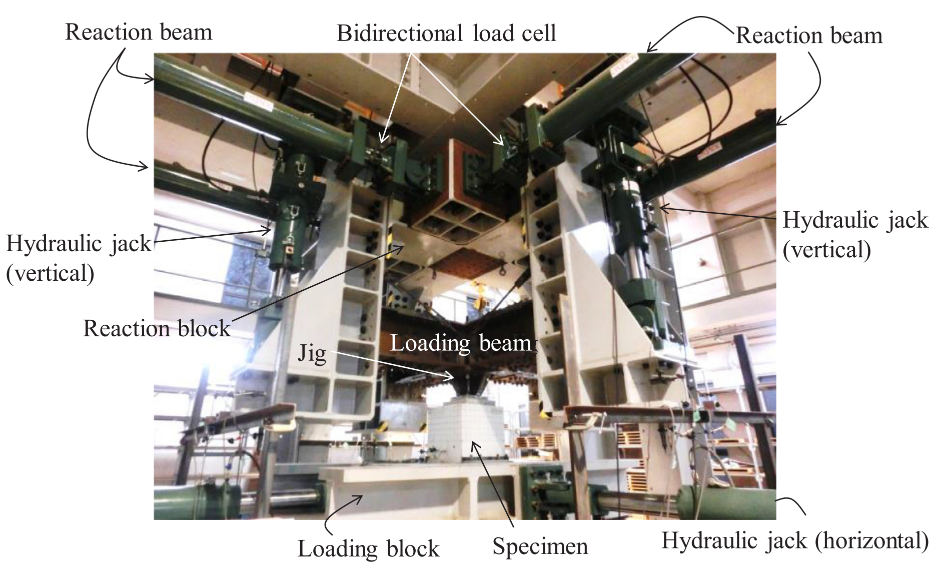


Fig. 7. 3D condition of test setup.

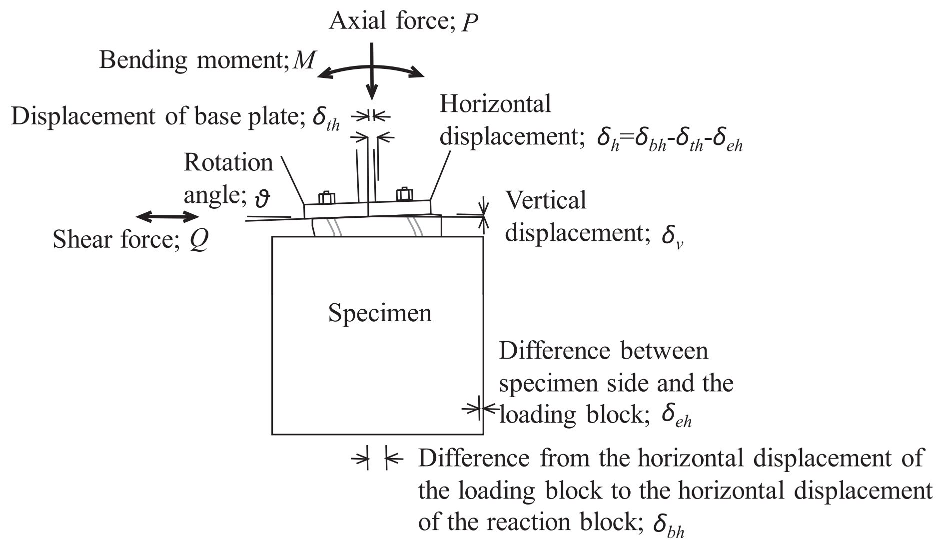


Fig. 8. Measurement item (main).

jig on the specimen was connected to the reaction block through a loading beam. A vertical load was applied by three hydraulic jacks that were connected through the reaction block as shown in A-A' section of Fig. 6. These hydraulic jacks for vertical load control applied constant axial force to the specimen.

The shear force (Q) and the axial force (P) in the specimen were obtained using the sum of the surrounding components of the horizontal and vertical forces. These were measured by the bidirectional load cells at the end of the hydraulic jacks and reaction beams. The bending moment (M) at the top of the specimen at the center of the base plate was obtained by the shear force of the horizontal hydraulic jack and reaction beams. The displacement of the reaction block was considered when evaluating moment arms. Loading was controlled by the applied horizontal load until significant concrete failure of the specimen was observed. After that, loading was controlled by the horizontal displacement of the specimen.

As shown in Fig. 8, the horizontal deformation of the specimen (δ_h) was calculated as the difference between the horizontal displacement of the loading block and that of the reaction block (δ_{bh}), and the difference between the horizontal displacement of the specimen side and that of the loading block (δ_{eh}). The vertical deformation of the specimen (δ_v) was calculated as the vertical displacement of the reaction block, because both the reaction frame and the loading block were regarded as rigid. The rotation angle of the specimen (θ) was measured using the vertical displacements at both sides of the reaction block and their distance.

To evaluate the reinforcement methods, the following measurement item was added. Inside the concrete, there were strain gauges glued on each of the stirrups to measure the lateral force transferred. Each anchor had strain gauges glued at two section to measure stress. As shown in Fig. 9, in the E-02 specimen, two strain gauges were attached on the top and bottom sides of each high-strength steel rod to measure the tension force.

At the beginning of loading, a 50-kN compression load was applied to the specimen as a constant axial force representing the weight of the steel roof. The applied axial force was kept within ± 10 kN of the target value by controlling the stroke of the vertical hydraulic jacks. Keeping the axial force constant, a horizontal force was applied incrementally. Loading amplitude was incremented 30 kN, which is approximately 20% of the nominal yield shear force of the anchor rods. The loading in each amplitude was repeated twice. After concrete failure occurred, the loading protocol changed to controlling using horizontal deformation. To intend to occur concrete edge failure, the amplitude of horizontal deformation for the positive load direction increased incrementally.

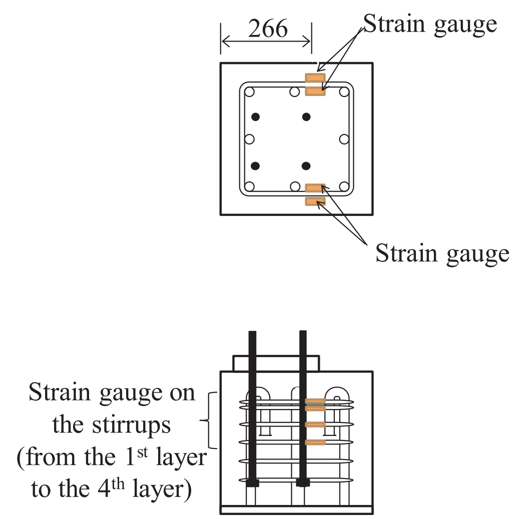


Fig. 9. Strain gauge position (to evaluate reinforcement effect).

Table 3
General data of test results.

Specimen	Max. shear force (Q_{max}) (kN)	Concrete edge failure strength (Q_c) (kN)	Estimated concrete compressive strength at test age of concrete (N/mm^2)	Age (Day)
E-01	122.5	137.5	27.7	41
E-02	269.4	138.0	27.9	43
E-03	190.7	139.7	28.7	50
E-04	146.8	140.7	29.1	54
E-05	137.7	140.9	29.2	55

However, for the opposite direction, constant deformation was applied to the specimen to avoid any unintentional failure disturbed the effect of reinforcement method by increasing deformation. In the specimens with reinforcement, the force of specimen increased after the concrete failure occurred because of the out-of-plane deformation of the steel jacket.

3.3. Test results

Table 3 shows the force for each specimen. The maximum shear

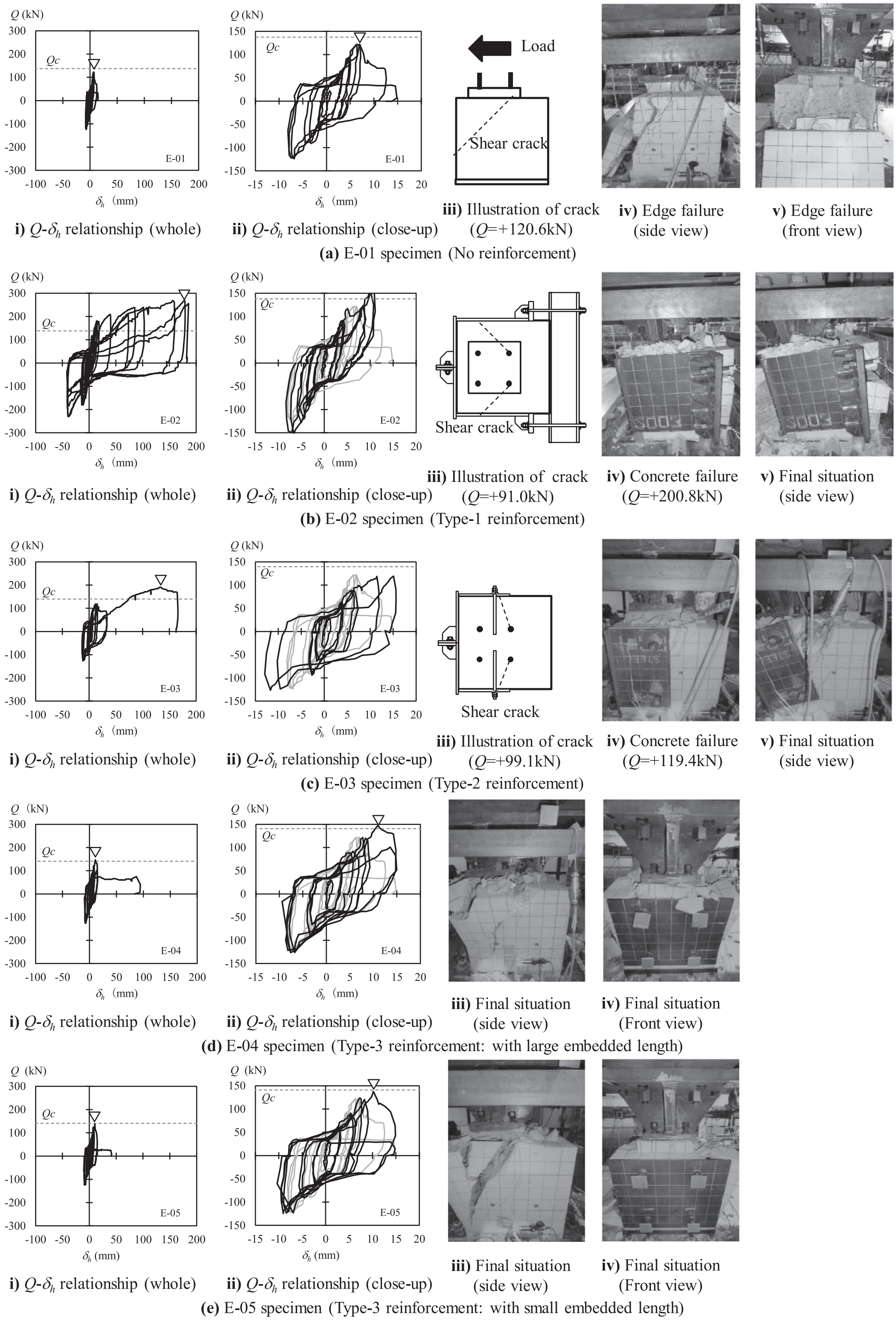


Fig. 10. Relationships between force and horizontal deformation and failure in all specimens.

force (Q_{max}) was obtained by the test results. Q_c is the estimated concrete edge failure force by design formula, as shown Eq. (1) in the design specification of composite structures in Japan [13], which used the presumed force of concrete obtained from linear interpolation of material test results according to the test age.

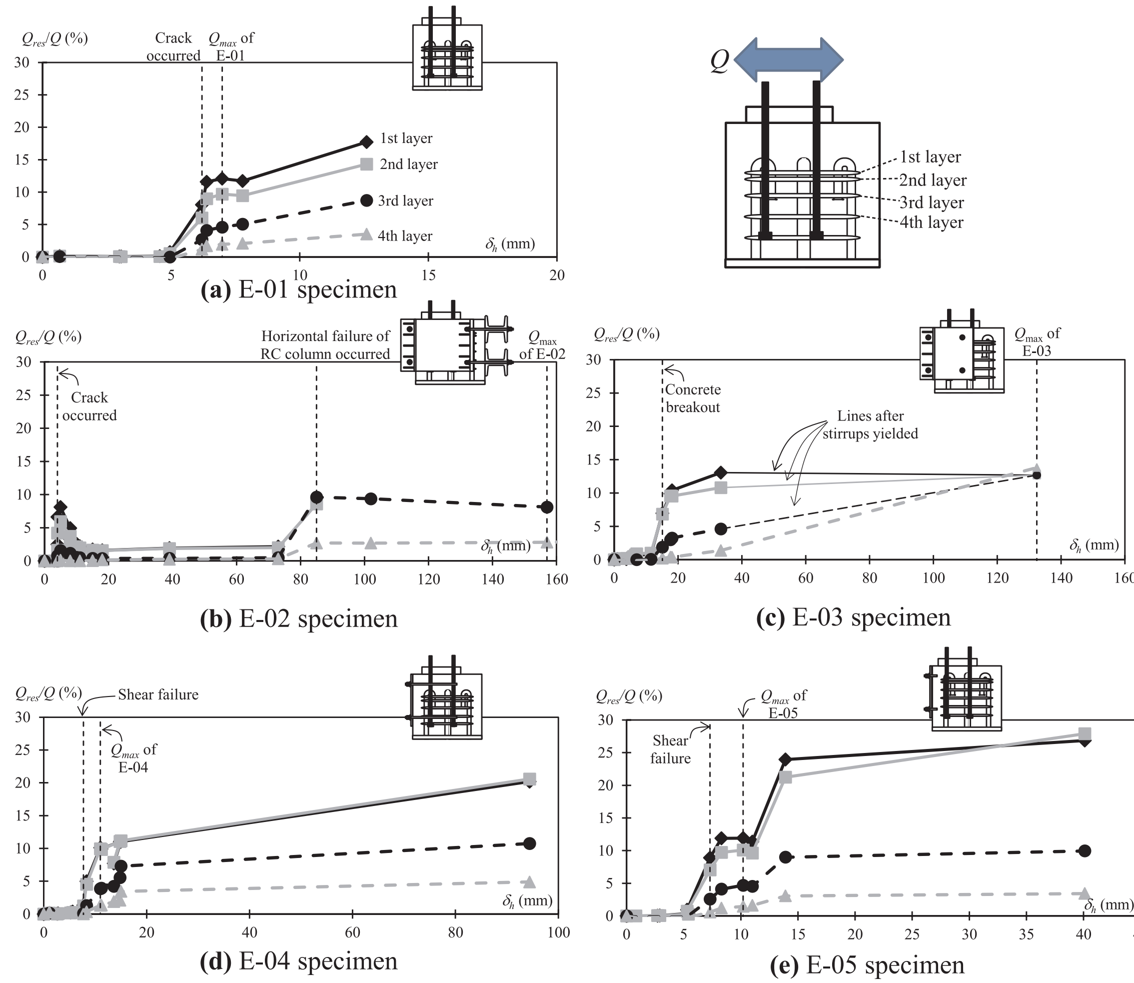
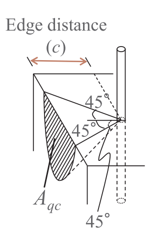


Fig. 11. Relationships between the resistance force and the horizontal displacement: (a) E-01 specimen, (b) E-02 specimen, (c) E-03 specimen, (d) E-04 specimen, and (e) E-05 specimen.



$$Q_c = {}_c\sigma_t A_{qc} \tag{1}$$

${}_c\sigma_t$: Concrete tensile strength

A_{qc} : Section area scooped out by concrete edge failure ($=0.5\pi c^2$) as shown in left figure.

c : Concrete edge distance

Here, Q_c is twice the concrete breakout force being determined on the forward anchor rods, because the shear forces of all the anchor rods are considered to transfer equally until concrete edge failure occurs.

Fig. 10 shows the relationships between shear force and horizontal deformation in each specimen. Each white triangle mark (∇) in Fig. 10 shows the maximum shear force of roof joint under positive loading (Q_{max}). The broken horizontal line shows the estimated concrete edge failure strength (Q_c). The gray hysteresis in the close-up relationship in the E-02, E-03, E-04, and E-05 specimens shows the hysteresis of the E-01 specimen.

For the specimen without reinforcement, E-01, shear cracks, as shown in Fig. 10(a) (iii), were observed on the RC column when the specimen's force reached +120.6 kN, and the concrete edge failure from near the rear anchors occurred when the specimen reached the maximum force in positive loading (+122.5 kN). Here, "+" in front of the force value indicates positive load direction, which pushes out the front of the specimen. As the specimen's force decreased after the maximum force, some concrete blocks in the front of the specimen experienced a downward pushing out by the forward anchors, as shown in Fig. 10(a)(iv).

In the E-02 specimen (Type-1 reinforcement), shear cracks from the rear anchors on the top surface of the RC column were observed when the specimen's force reached the first peak (+117.2 kN), as shown in Fig. 10(b)(iii). After the first peak, the specimen's force decreased; however, the specimen's force increased again as the loading continued. When the specimen's force reached +200.8 kN, the quite large concrete failure of the specimen occurred between the upper reaction jigs' position and lower reaction jigs' position, as shown in Fig. 10(b) (vi). Ultimately, the concrete failure fractured the lower high-strength steel rods, and the specimen's force was suddenly lost; however, falling concrete was not observed. The out-of-plane deformation of the steel jacket was observed as a result of the subduction of the base plate into the concrete under full collapse of the base mortar.

In the E-03 specimen (Type-2 reinforcement), shear cracks were observed from the rear anchor position to the post-installed adhesive anchor on the side of the concrete, as shown in Fig. 10(c) (iii) when the specimen's force slightly decreased after it reached +99.1 kN. These concrete cracks propagated as loading continued. When the specimen's force reached the second peak at +119.4 kN, the concrete breakout occurred from the rear anchors. Despite the concrete breakout, the specimen's force increased again. One of the reasons is that the steel jacket rotated around the lower edge, as shown in Fig. 10(c)(iv), with out-of-plane deformation according to the pushout of the fractured concrete. Finally, the specimen's force reached +190.7 kN during monotonic loading after cyclic loading.

In the E-04 and E-05 specimens (Type-3 reinforcement), shear cracks on the concrete side of the rear anchors were generated, similar to the E-01 specimen, when the specimen's force reached the maximum. After that, large deformation was applied for positive load direction to confirm the concrete failure mode, and the E-04 specimen with a large effective embedded length kept a force of 60–70 kN. Finally, concrete fell down from the side of the specimen, as shown in Fig. 10(d) (iii) and (iv). The E-05 specimen with a short effective embedded length maintained a force of 20–30 kN and falling concrete also was observed from the side of the specimen at the end of the test, as shown in Fig. 10(e)(iii) and (iv). The steel plate of the E-04 specimen was bent at a position 10 mm from the bottom; however, the steel plate of the E-05 specimen did not bend. This means the steel plate in the E-05 specimen was not effective in resisting shear force.

In a close-up examination of the relationship between shear force and horizontal displacement, all reinforced specimens showed similar hysteresis to that of the E-01 specimen. Therefore, the reinforcement methods affect the hysteresis only after concrete breakout occurs.

4. Comparison of the effects of reinforcement methods

In the following section, the resistance to lateral force in each reinforcement method is discussed. Fig. 11 shows the relationships between the ratio of resistance force transferred in the stirrups to the specimen's force (Q_{res}/Q) and the horizontal deformation of the specimen (δ_h). The resistance force (Q_{res}) is calculated by the average strain on the stirrups in each layer. Each plot in Fig. 11 is extracted from the hysteresis using the method illustrated in Fig. 12.

In the E-01 specimen, the resistance force in each stirrup increased significantly when shear cracking of concrete occurred. The resistance force of the stirrups in the first layer was the largest, and the deeper the stirrup's positions from the top of concrete surface, the smaller their resistance force became. After the specimen reached the maximum force (Q_{max}), the resistance force of the stirrups did not change. The sum of the resistance force of the stirrups when the specimen reached Q_{max} was 28% of the specimen's force.

The resistance force of the stirrups in the E-02 specimen increased significantly; however, the value dropped to near zero after cracks in the concrete were generated. This was because concrete cracking makes the lateral force transfer not through vertical anchor rods, but through the steel jacket and the high-strength steel rods; therefore, the lateral force transferred in the RC column is reduced. When the large concrete failure occurred between the upper rods' position and the lower rods' position, the resistance force of the stirrups increased again. The sum of the resistance force of the stirrups was 21% of the specimen's force.

In the E-03 specimen, the resistance force of the stirrups increased after the concrete shear failure occurred. The sum of the resistance force of the stirrups was 16% of the specimen's force. Because of the concrete failure, the stirrups received large tensile force and yield. In Fig. 11(c), the thin lines show the resistance force of the stirrups after yielding, i.e., the product of yield stress and the effective section area of the stirrup. Until specimen force reaches the maximum force (Q_{max}), these stirrups of four layers all yield. The steel jacket could not unite the top of the RC column because of the shear cracking of the concrete around the post-installed adhesive anchors. Hence, the steel jacket of the E-03 specimen attached by post-installed adhesive anchors did not transfer shear force.

The resistance force of the stirrups in the E-04 and E-05 specimens also increased when shear failure of the RC column occurred. When each specimen reached the maximum force (Q_{max}), the sum of the resistance force of the stirrups was similar to that in the E-01 specimen. The results show that post-installed adhesive anchors cannot effectively resist lateral force regardless of the anchor length.

5. Conclusion

To validate the effect of reinforcement in the roof joint between an RC frame and a steel roof, the seismic performance of three reinforcement methods were examined by cyclic loading tests. These reinforcement methods enable rapid retrofit from the interior of buildings by using simple, fabricated steel jackets and bolts and they do not require grout filling. According to obtained results, the reinforcement that used the steel jacket attached on the inner sides of the RC column and the high-strength steel rods significantly increased in maximum force and displacement of the roof joint. This is because the combination of the steel jacket and high-strength steel rods. Two important points for reinforcement are as follows.

(1) The steel jacket should surround the entire top of the RC column, or RC part just under roof joint.
(2) Lateral force of the steel jacket should be transferred from the RC main frame by through bolts, which is formed by the RC column and the RC beam and excludes the roof joint zone.

It is important to transfer lateral force to the RC main frame, not only to the RC column. If this rule is kept, the reinforcement method with steel jacket and high-strength steel rods can be used even when the RC beam and RC column have the same width. The through bolts to transfer lateral force to the RC main frame do not necessarily need to be prestressed. This is helpful to use Type-1 reinforcement method in the existing building with thin RC walls. To effectiveness of the reinforcement, the through bolts should have balanced position, and the diameter and material property enough to resist to concrete failure. If lateral force cannot be transferred to the main frame, the reinforcement is not effective, and concrete blocks could fall down from the top of the RC column, as shown in earthquake damage reports.

The minimum reinforcement recommendation for the roof joint design is the future research because the quantitative considerations based on additional loading test or analysis are needed.

CRediT authorship contribution statement

Yuko Shimada: Validation, Investigation, Writing - original draft. **Satoshi Yamada:** Supervision, Project administration. **Shoichi Kishiki:** Data curation, Writing - review & editing. **Takashi Hasegawa:** Funding acquisition, Conceptualization. **Toru Takeuchi:** Methodology,

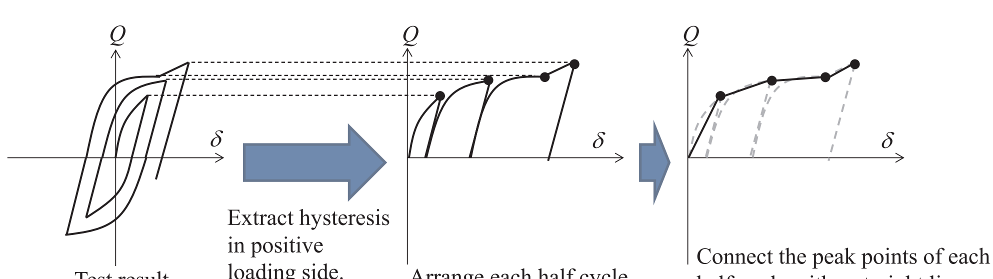


Fig. 12. Method of extracting the line connecting peak points of half cycles from hysteresis.

Formal analysis.

Declaration of Competing Interest

The authors declared that there is no conflict of interest.

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