Retrofitting of RC Frames by Steel Braced Frames Utilizing a Hybrid Connection Technique

Pasha Javadi, Tetsuo Yamakawa

*Journal of Advanced Concrete Technology*, volume 11 (2013), pp. 89-107

---

Related Papers Click to Download full PDF!

**Seismic Retrofitting Methods Newly developed for Railway Concrete Structures**
Tadayoshi Ishibashi, Takeshi Tsuyoshi, Kaoru Kobayashi

*Journal of Advanced Concrete Technology*, volume 2 (2004), pp. 65-76

---

**Investigation of a Hybrid Technique for Seismic Retrofitting of Bare Frames**
Md. N. Rahman, Tetsuo Yamakawa


---

**Seismic Retrofit of Reinforced Concrete Building Structures with Prestressed Braces**
Susumu Kono, Takeshi Katayama

*Journal of Advanced Concrete Technology*, volume 7 (2009), pp. 337-345

---

**Seismic Damage of and Seismic Rehabilitation Technique for Railway Reinforced Concrete Structures**
Tadayoshi Ishibashi, Daisuke Tsukishima

*Journal of Advanced Concrete Technology*, volume 7 (2009), pp. 287-296

---

Click to Submit your Papers

Japan Concrete Institute  [http://www.j-act.org](http://www.j-act.org)
Retrofitting of RC Frames by Steel Braced Frames Utilizing a Hybrid Connection Technique

Pasha Javadi¹ and Tetsuo Yamakawa²

Abstract

This study presents a new connection technique to retrofit existing RC frames by steel braced frames. The proposed connection which is called the “hybrid connection” plays two important structural roles. The first role is to provide connection between an existing RC frame and a steel braced frame. The second role is to increase shear strength and axial compression capacity of boundary RC columns of retrofitted frames. The hybrid connection consists of steel plates, high-strength bolts, and high-strength grout. Experimental investigations were conducted on four one-bay one-story RC frames and one one-bay two-story RC frame which were retrofitted by the proposed method. Moreover, one one-bay one-story RC frame was tested as a non-retrofitted benchmark frame. Experimental results of the specimens indicated that the proposed hybrid connection transferred relatively high direct shear forces between the RC frames and the installed steel braced frames to obtain lateral capacity of the steel braces. In addition, the proposed hybrid connection prevented possible shear failure of the boundary RC columns with the help of the utilized steel plates. Taking into consideration experimental results and observations, simplified design approaches are presented to estimate lateral strength of fundamental failure mechanisms of retrofitted RC frames.

1. Introduction

In past earthquakes such as the 1994 Northridge-USA, the 1995 Kobe-Japan, and the 1999 Izmit-Turkey, many reinforced concrete (RC) buildings significantly damaged or completely collapsed because of formation of the soft-story mechanism. In the soft-story mechanism, considerable lateral deformation occurs in a story that has a lower lateral strength and stiffness compared to its adjacent stories. In Japan, the Building Standard Law (BSL 1981) was revised in 1981 to upgrade required lateral resistances of buildings with irregular stiffness distribution in plan and/or along height of buildings. However, in Japan and other countries located on high seismic zones, many existing reinforced concrete buildings are still vulnerable to the soft-story mechanism.

Application of steel braced frames is one of the common retrofit techniques for existing soft-story RC frames. Experimental investigations by Sugano and Fujimura (1980) and Higashi et al. (1980) indicated effectiveness of installed steel braces in increasing lateral strength of retrofitted RC frames. Kawamata and Ohnuma (1980) suggested a practical retrofit scheme for strengthening existing reinforced concrete frames by eccentric steel braces. Ohishi et al. (1988) and Sekiguchi et al. (1988) presented practical applications for retrofitting RC frames by steel braced frames with direct connection which has been used as the typical retrofit scheme in the retrofit guideline by the Japan Building Disaster Prevention Association (JBDPA 2001). In the method used by JBDPA (2001), a steel braced frame is indirectly connected to a RC frame by means of studs and anchors which are interlocked at the connection zone with the help of high-strength grout. Studs are welded to the boundary zone of the steel frame, and anchors are inserted into epoxy-injected holes provided on the boundary zone of the RC frame. Then, high-strength grout is cast in the boundary zone between the steel frame and the RC frame to interlock the studs and anchors. Another approach was presented by Jones and Jirsa (1986) and adopted in the guideline by the Federal Emergency Management Agency (FEMA-547 2006). In this method, a steel braced frame is directly connected to a RC frame by means of anchors inserted into epoxy-injected holes on the boundary zone of the RC frame. However, in both methods by JBDPA (2001) and FEMA-547 (2006), the primary concern is connection between a RC frame and a steel braced frame. In fact, a limited number of anchors cannot provide required shear force between a steel frame and a RC frame to obtain lateral capacity of steel braces. Furthermore, in the conventional methods (JBDPA 2001, FEMA-547 2006), an installed steel braced frame only increases lateral strength and stiffness of a retrofitted frame, while RC columns still suffer from possible shear failure in the case of non-ductile RC columns. So, in the current study, it is aimed at proposing a new connection method to provide required direct shear forces between a RC frame and a steel frame to prevent possible shear failure of RC columns.
A practical technique for seismic retrofitting of soft-story RC frames by adding wing-walls or panel-walls was proposed by Yamakawa et al. (2006a, 2006b), and Rahman and Yamakawa (2007). In the proposed method which is called the “thick hybrid wall technique”, channel-shaped steel plates jacket boundary RC columns and are extended to bays of RC frames by additional plain steel plates. The steel plates are connected together by means of high-strength bolts. In fact, the steel plates and high-strength bolts make steel formworks inside bays of RC frames. Then, additional concrete is cast in the provided steel formworks. One of the important technical points of the thick hybrid wall technique is to provide appropriate connections between additional wing-walls and existing RC columns. Considering structural advantages of the connection in the thick hybrid wall technique, a new connection method which is called the “hybrid connection technique” was proposed for retrofitting existing RC frames by steel braced frames (Yamakawa 2009; Yamakawa 2010). The proposed hybrid connection consists of steel plates, high-strength bolts, and high-strength grout. In the current study, complementary experimental investigations are conducted to verify all possible failure mechanisms of RC frames retrofitted by steel braced frames with the help of the proposed hybrid connection. The experimental studies were conducted on five one-bay one-story RC frames and one one-bay two-story RC frame. The scale factor of the test specimens is 1/4-1/3, to model a low-rise school building designed according to the pre-1971 Building Standard Law of Japan (BSL 1971). The experimental results indicated that the hybrid connection transferred relatively high direct shear forces between the RC frames and the installed steel braced frames.

![Fig. 1 An overview of the conventional connection method [unit: N, mm].](image-url)

- **Specimen R08B-F60**
  - $N = 0.2hD\sigma_y$ (Conventional connection)
  - Steel frame
  - Sizes of the steel frame and braces: BH-75x60x4.5x4.5
  - $\sigma_y = 22.6$ MPa, for the RC frame
  - $\sigma_y = 333$ MPa, for the steel braced frame
  - Failure mechanism: Firstly, flexural plastic hinges formed in the RC columns, and the steel braces buckled in compression and yielded in tension. Then, sudden degradation occurred in the $V-R$ relationship due to shear punching failure happened at the top of the left-hand RC column and at the top connection, and shear failure happened at the top of the right-hand RC column.

- **Specimen R08B-F75**
  - $N = 0.2hD\sigma_y$ (Conventional connection)
  - Steel frame
  - Sizes of the steel frame and braces: BH-75x75x4.5x4.5
  - $\sigma_y = 19.7$ MPa, for the RC frame
  - $\sigma_y = 354$ MPa, for the steel braced frame
  - Failure mechanism: Sharp degradation occurred in the $V-R$ relationship due to shear punching failure happen at the top of the left-hand RC column and at the top connection, and shear failure happened at the top of the right-hand RC column.

- **Details of connections**
  - Anchor (D13)
  - Spiral hoop (D13)
  - Stud (D13)
  - Steel beam
  - Longitudinal and shear reinforcements in the RC columns are 8-D10 ($P_s=1.85\%$) and 3.76@@105 ($P_s=0.12\%$), respectively.
  - Longitudinal and shear reinforcements in the RC beams are 4-D13 ($P_s=1.63\%$) and 6@@120 ($P_s=0.43\%$), respectively.

- **Notes**: $\lambda = kl/r$, where $k$: effective length factor (for this case, $k=1$); $l$: length of steel braces (see the specimens); $r$: gyration of the steel brace section about its weak axis.
Furthermore, the hybrid connection prevented shear failure of RC columns. In addition to experimental investigations, corresponding design approaches are suggested to calculate lateral strengths of fundamental failure mechanisms of retrofitted frames.

2. An overview of the conventional method

The conventional connection method for retrofitting an existing RC frame by a steel braced frame is suggested in the guidelines by the Japan Building Disaster Prevention Association (JBDPA 2001). In order to give an overview of the conventional method, two RC frames retrofitted by steel braced frames utilizing the conventional connection are presented in Fig. 1 (Maeda and Yamakawa 2012). As shown in Fig. 1, the steel braced frames are indirectly connected to the RC frames by means of studs and anchors which are interlocked at the connection zones with the help of high-strength grout. In the retrofitting operation, studs are welded to the boundary of the steel frame, and anchors are inserted into the epoxy-injected holes provided on the boundary of the RC frame. Then, high-strength grout is cast in the boundary zones between the steel frame and the RC frame to interlock the installed studs and anchors. The specimens R08B-F60 and R08B-F75 shown in Fig. 1 were tested under constant axial forces and cyclic horizontal loading (Maeda and Yamakawa 2012). As shown in Fig. 1, the difference between the two specimens is the sizes of the steel braced frames.

In the specimen R08B-F60, the slenderness ratio of the steel braces is \( \lambda = 63 \) (see Fig. 1). In this specimen at \( R = 0.7\% \), steel braces buckled in compression and yielded in tension. Moreover, flexural plastic hinges formed at the top and at the bottom of the RC columns. By proceeding the loading test, at the drift angle of \( R = 2.5\% \), the lateral resistance \( V \) suddenly dropped due to shear punching failure at the top of the left-hand RC column and shear failure at the top of the right-hand RC column. Simultaneously, direct shear failure happened at the top connection.

In the specimen R08B-F75, the slenderness ratio of the steel braces is \( \lambda = 49 \) (see Fig. 1). In the \( V-R \) relationship of the specimen, as the lateral resistance \( V \) reached to its maximum value, rapid degradation happened due to shear punching failure at the top of the left-hand RC column and shear failure at the top of the right-hand RC column. Simultaneously, direct shear failure happened at the top connection.

From the experimental results of the two specimens, it is obvious that serious damages, such as punching failure, shear failure and etc., are strongly likely to happen in a RC frame retrofitted by steel braced frame with the help of the conventional connection. In fact because of low direct shear resistance of the conventional connection, it is difficult to reflect lateral capacity and behavior of steel braces in the global response of the retrofitted frames. Therefore, it is required to propose a connection with relatively high direct shear strength to capture lateral capacity of steel braces.

3. Experimental program of the proposed method

3.1 Typical retrofitting procedure of the test specimens

Typical procedure of the proposed retrofit method is illustrated in Fig. 2. The retrofitting operation is identical in all test specimens. Details of the retrofit schemes of the test specimens are explained in Section 3.3. The typical retrofitting procedure of the test specimens are expressed in the following items;

- **Placing the steel braced frame inside the RC frame:**
The fabricated steel braced frame with the inverted-V shaped braces is placed inside the existing RC frame.

![Fig. 2 Typical retrofitting procedure of the proposed technique.](image)
The steel braced frame is primarily fixed inside the RC frame by means of two base plates. Each base plate is connected to the stub through anchor bolts. The anchor bolts are inserted into epoxy-injected holes drilled in the stub.

- **Providing the steel formworks of the hybrid connections at the boundary columns:** Two different retrofit schemes are considered for boundary RC columns. One scheme is for columns with wing walls and another scheme is for columns without wing-walls. In the case of wing-wall columns, two plain steel plates are used (No. (2) in Fig. 2). In the case of columns without wing-walls, channel shaped steel plates are utilized (No. (3) in Fig. 2). After installation of the steel plates, the high-strength bolts are passed through the steel plates into the holes punched on the steel columns (No. (5) in Fig. 2).

- **Providing the steel formwork of the hybrid connection at the top beam:** The top beam of the RC frame is partially connected to the steel beam through the proposed hybrid connection. Two lateral plain steel plates sandwich the RC beam and the steel beam (No. (4) in Fig. 2). Two horizontal series of high-strength bolts stitch the two lateral sandwiching steel plates. One series of the high-strength bolts is passed through the top RC beam, and another series is passed through the top steel beam.

- **Casting high-strength grout:** After installation of the steel plates and the high-strength bolts, high-strength grout is cast in the provided spaces in the formworks (No. (6) in Fig. 2). The steel plates are used as formworks for casting the grout. To ensure that the grout can easily flow and uniformly fill the provided spaces, about 10 mm gap is considered between the outer faces of the RC columns and the steel plates.

### 3.2 Fundamental failure mechanisms of the retrofitted specimens

Idealized deformations of the fundamental mechanisms are illustrated in Fig. 3. It should be noted that it is assumed the RC subs are fixed to a rigid floor, however, in real buildings uplift of foundations is likely to happen. Specific structural characteristics of the fundamental failure mechanisms are shown in Fig. 3 and are explained as follows;

- **Mechanism (a):** In this mechanism, the steel braces yield in tension and buckle (or yield) in compression. The RC frame and the steel frame behave in a flexural manner.

- **Mechanism (b):** In this mechanism, direct shear failure happens simultaneously at the top hybrid connection zone and at the top of the RC columns.

- **Mechanism (c):** In this mechanism, direct shear failure happens simultaneously at the bottom of the RC columns and in the anchor bolts of the base plates.

- **Mechanism (d):** In this mechanism, overall-overturning...
behavior occurs in the retrofitted frame due to vertical slip of the longitudinal reinforcements at the bottom of the boundary RC column and due to vertical slip of the anchor bolts.

### 3.3 Reinforcement details and retrofit schemes of the test specimens

Many reinforced concrete buildings collapsed during the 1995 Kobe-Japan earthquake due to brittle shear failure in columns. The same failure mode was observed in school buildings after the 1968 Tokachi-oki earthquake in Japan. The Building Standard Law of Japan was revised in 1971 (BSL 1971) to require close spacing of lateral (tie) reinforcements in columns (Otani 2004). Details of the original RC frames of the test specimens are given in Fig. 4. The scale factor of the RC frames is about 1/4 to 1/3 of a low-rise school building designed according to the pre-1971 Building Standard Law of Japan (BSL 1971). As shown in Fig. 4, geometrical dimensions and reinforcement patterns of all one-bay one-story frames are identical. The clear shear span-to-depth ratio \( M/(VD) \) of the RC columns is 2.50, and that of the RC beams is 2.65. Since the RC frames are designed according to the pre-1971 Building Standard Law of Japan (BSL, 1971), the RC columns contain insufficient amount of transverse reinforcements of \( pw = 0.12\% \), and consequently shear failure is likely to happen in the RC columns. In the one-bay two-story test specimen, the dimensions and reinforcements of the first story is the same as those in the one-bay one-story specimens. Retrofit details of the test specimens are illustrated in Fig. 5. Specific retrofit schemes of the test specimens (in Fig. 5) are explained as follows:

The specimen R05P-P0 is a non-retrofitted one-bay one-story RC frame. This specimen is the benchmark specimen that represents behavior of soft-story RC frames.

The specimen R09B-F is retrofitted by the steel frame without steel bracing members. This specimen is planned to verify structural performance of the RC frame and the steel frame in the absence of steel bracing members.

The specimen R09B-P is retrofitted by the steel braced frame with the help of the hybrid connection technique. The steel braced frame is connected to the RC frame at the boundary RC columns and at the top beam. This specimen is planned in such a way that the mechanism \( a \) (in Fig. 3) would appear in its experimental response.

The specimen R09B-CT is retrofitted by the steel braced frame with the help of the hybrid connection technique. Sandwiching steel plates at the top hybrid connection have the thickness of \( t = 1.2 \text{ mm} \) that are thinner than those in other specimens. The specimen is planned in such a way that the mechanism \( b \) (in Fig. 3) would appear in its experimental response. In some buildings, the presences of wing-walls beside RC columns make it impossible to use channel-shaped steel plates. Considering this fact, in the specimen R09B-CT, the RC column and the steel column are sandwiched by two lateral plain steel plates instead of a channel shaped steel plate.

The specimen R10B-CP is retrofitted by the steel braced frame with the help of the hybrid connection technique. The anchor bolts of the base plates have a small diameter of \( D = 10 \text{ mm} \). Because of relatively low direct shear resistances of anchor bolts at the base, direct shear failure is likely to happen at that surface. This specimen is planned in such a way that the mechanism \( c \) (in Fig. 3) would appear in its experimental response.

The specimen R10B-CO is retrofitted by the steel braced frame with the help of the hybrid connection. This specimen is planned in such a way that the mechanism \( d \) (in Fig. 3) would appear in its experimental response. To provide a relatively high overturning moment at the base, a two-story frame is decided to be tested instead of a one-story frame. The anchor bolts at the base plates have the small diameter of \( D = 10 \text{ mm} \), and consequently the total tension resistance of anchor bolts against the overturning moment is relatively low in comparison with other retrofitted specimens.
Channel-shaped steel plate ($t_s$)

Specimen R05P-P0

1.0 m

1.5 m

Specimen R09B-F

Mechanism (a)

Specimen R09B-CT

Mechanism (b)

Specimen R09B-P

Specimen R09B-F

Specimen R09B-CT

Mechanism (b)

Specimen R10B-CP

Mechanism (c)

Specimen R10B-CO

Mechanism (d)

1.5 m

1.0 m

Grout

U-shaped reinf. (D6-@100)

Column section

125

140

175

175

High-strength bolt (13φ)

Steel plate ($t_d$)

U-shaped reinf. (D6-@100)

Beam section

140

250

175

175

Grout

High-strength bolt (13φ)

Notes: $T$: thickness of steel plates at the boundary columns, $t_s$: thickness of steel plates at the top connection, $l_{ex}$ & $h_{ex}$: effective length and height of steel plates at the top connections respectively, $l_a$: anchoring length of anchors, $D_a$: diameter of anchors, $h \times b \times t_f \times t_w$: section sizes of steel members.

Fig. 5 Test plans of the specimens.
3.4 Material properties
Properties of concrete and grout materials on the day of the loading tests are given in Table 1. Properties of steel materials are listed in Table 2.

3.5 Test setups and loading programs
Test setups and horizontal displacement-controlled programs of the test specimens are given in Fig. 6. The vertical loads and horizontal displacements were simultaneously applied on the specimens. Two servohydraulic actuators applied the constant vertical load of \( N = 0.2bD\sigma_B \) (where, \( bD \): section dimensions of the RC column and \( \sigma_B \): cylinder strength of concrete) per column. A double-acting hydraulic oil jack pushed and pulled the test specimens under the displacement-controlled programs. In the case of the one-bay two-story specimen, at each horizontal loading stage, 5/11 and 6/11 of the base shear force were applied on the first and second story respectively, in order to provide the lateral distribution of seismic forces according to the Building Standard Law of Japan (BSL 1981). For the non-retrofitted specimen R05P-P0, the cyclic loading test was conducted at drift angles in the range of \( R = \pm 0.5\% \), \( \pm 1.0\% \), \( \pm 1.5\% \), \( \pm 2.0\% \), \( \pm 2.5\% \), and \( \pm 3.0\% \) for two successive cycles, and \( \pm 0.125\% \), \( \pm 0.25\% \), \( \pm 4.0\% \), and \( \pm 5.0\% \) for one cycle. In the RC frames retrofitted by the steel braced frames, to precisely observe structural behavior of the retrofitted frames, the horizontal cyclic displacement-controlled program was planned to be performed at drift angles with smaller increment compared to that of the non-retrofitted frame. The displacement-controlled program of the retrofitted frames is in the range of \( R = \pm 0.1\% \), \( \pm 0.2\% \), ..., \( \pm 1.9\% \), \( \pm 2.0\% \), \( \pm 3.0\% \), \( \pm 4.0\% \) and \( \pm 5.0\% \) for one cycle. It should be emphasized that displacement-controlled programs of non-retrofitted frame and the retrofitted frames are different, however, this difference can not considerably influence the comparison between structural behavior of the non-retrofitted frame and the retrofitted ones, especially from the viewpoint of maximum lateral resistance.

<table>
<thead>
<tr>
<th>Table 1 Properties of concrete and grout materials.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
</tr>
<tr>
<td>Specimen</td>
</tr>
<tr>
<td>R05P-P0</td>
</tr>
<tr>
<td>R09B-F</td>
</tr>
<tr>
<td>R09B-P</td>
</tr>
<tr>
<td>R09B-CT</td>
</tr>
<tr>
<td>R10B-CP</td>
</tr>
<tr>
<td>R10B-CO</td>
</tr>
</tbody>
</table>

Notes; \( \sigma_B \): cylinder strength of concrete, \( \varepsilon_c \): concrete strain at \( \sigma_B \), \( E_c \): Young’s modulus of concrete, \( \sigma_{bg} \): cylinder strength of grout, \( \varepsilon_g \): grout strain at \( \sigma_{bg} \), and \( E_g \): Young’s modulus of grout.

<table>
<thead>
<tr>
<th>Table 2 Properties of steel materials.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel element</td>
</tr>
<tr>
<td>Longitudinal rebars and anchors</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Hoops and stirrup</td>
</tr>
<tr>
<td>High-strength bolts</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Steel plates</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

Notes; \(^*\) and \(^*\) indicate the material belong to R05P and R09B series of the specimens respectively, non-starred materials belong to R10B series, \( a \): cross section area, \( \sigma_y \): yield strength, \( \sigma_u \): ultimate strength, \( \varepsilon_y \): yield strain, \( E \): Young’s modulus of elasticity, and \( t \): thickness of steel plate.
Details of the test setups:

Fig. 6 Test setups and displacement-controlled programs.
4. Experimental results and observations

The photos of the test specimens after finishing loading tests and removing the steel plates are given in Fig. 7. The crack patterns and the $V$-$R$ relationships of the test specimens are shown in Fig. 8. The experimental observations of the test specimens are explained one-by-one as follows:

In the specimen R05P-P0, flexural cracks appeared at the end of the columns, and at the end of the beam at the drift angle of about $R = 0.5\%$ and $R = 1.0\%$, respectively. Shear cracks at the columns generated at about $R = 1.5\%$ and widened progressively with increasing drift angles. In the cycle with the target amplitude of $R = 2.0\%$, the right-hand column collapsed suddenly in shear failure at the drift angle of $R = -1.7\%$.

In the specimen R09B-F, flexural cracks generated at the bottom of the RC columns and at the both ends of the RC beam at the drift angles of $R = 0.2\%$ and $R = 0.3\%$, respectively. At the drift angle of $R = 1.4\%$, the shear cracks at the columns generated at about $R = 1.5\%$ and widened progressively with increasing drift angles. In the cycle with the target amplitude of $R = -2.0\%$, the right-hand column collapsed suddenly in shear failure at the drift angle of $R = -1.7\%$.

In the specimen R09P-P0, flexural cracks appeared at the end of the columns, and at the end of the beam at the drift angle of about $R = 0.5\%$ and $R = 1.0\%$, respectively. Shear cracks at the columns generated at about $R = 1.5\%$ and widened progressively with increasing drift angles. In the cycle with the target amplitude of $R = -2.0\%$, the right-hand column collapsed suddenly in shear failure at the drift angle of $R = -1.7\%$.

In the specimen R09B-P, the RC frame is retrofitted by the steel braced frame with help of the proposed hybrid connection. During the cyclic loading test, at the drift angle of $R = 0.2\%$ and $R = 0.3\%$, flexural cracks generated at the base of the RC columns and at the ends of the RC beam, respectively. Buckling-shape deformation initiated in the steel braces at the drift angle of $R = 0.9\%$. Flexural cracks appeared at the top of the RC columns at $R = 1.2\%$. The maximum lateral resistance of $V_{\text{max}} = 535kN$ that is about 5 times of that of the non-retrofitted specimen R05P-P0 obtained at the drift angle of $R = 1.3\%$. By increasing the lateral deformation, significant buckling happened in the compression brace, and consequently the compression resistance of the brace gradually decreased. Regarding the discrepancy between the compression resistance and the tension resistance of the braces in the post inelastic-buckling stage, an unbalanced downward force produced at the braces joint. Finally, at the large drift angle of $R = 5.0\%$, shear failure happened at the RC beam because of the aforementioned unbalanced downward force. The dominant mechanism of this specimen is the mechanism (a) illustrated in Fig. 3.

In the specimen R09B-CT, the thickness of the steel plates at the top hybrid connection is $t_s = 1.2mm$ that is thinner than those in other specimens. Because of applying relatively thin steel plates at the top hybrid connection, direct shear failure was expected at that zone. In the cyclic loading test, horizontal cracks appeared at the top of the RC columns at the drift angle of $R = 0.4\%$. The thin steel plates at the top connection started buckling at $R = 0.6\%$. The lateral resistance reached to the maximum value of $V_{\text{max}} = 473kN$ at $R = 0.8\%$. In the $V$-$R$ relationship, a sharp strength degradation can be observed because of direct shear failure at the top hybrid connection and shear punching at the top of the boundary RC columns. The dominant mechanism of this specimen is the mechanism (b) illustrated in Fig. 3. The buckling shape of the steel plate after finishing the loading test is shown in Fig. 9.

In the specimen R10B-CP, the anchors (4-D10 at each base plate) are relatively weaker than those in the specimens R09B-P and R09B-CT (4-13 high-strength bars). Because of utilizing weak anchorage system at the base of the steel braced frame, direct shear failure is likely to happen at that zone. In the cyclic loading test, at the drift angle of $R = 0.1\%$, horizontal cracks appeared at the base of the RC columns. The anchors of the base plates yielded at the drift angle of $R = 0.2\%$ due to the provided shear force. At drift angle of $R = 0.3\%$, the longitudinal reinforcements started yielding because of shear punching at the bottom of the RC columns. The lateral resistance reached to the maximum value of $V_{\text{max}} = 551kN$ at $R = 0.9\%$. Afterwards, the lateral resistance sharply decreased, because the anchors at the base plates started breaking under shear sliding. The dominant mechanism of this specimen is the mechanism (c) illustrated in Fig. 3.

In the specimen R10B-CO, the anchors (4-D10 at each base plate) are relatively weaker than those in the specimens R09B-P and R09B-CT (4-13 high-strength bars). The retrofit details of this specimen are the same as that in the specimen R10B-CP. However, since the specimen R10B-CO is a two-story frame, the height of the applied lateral force is higher, and consequently the provided overturning moment is greater. In the cyclic loading test, at the drift angle of $R = 0.2\%$, the anchors of the base plates yielded in tension due to the provided overturning moment. At the drift angle of $R = 0.3\%$, flexural cracks appeared at the base of the RC columns and longitudinal reinforcements started yielding. Fine shear cracks generated at the shear wall of the second story at $R = 0.8\%$. The lateral resistance reached to the maximum value of $V_{\text{max}} = 476kN$ at $R = 0.9\%$. Under overall-overturning behavior of the specimen, the anchors and the longitudinal reinforcements at the bottom of RC columns started breaking at the drift angle of $R = 1.3\%$ and $R = 1.9\%$, respectively. The dominant mechanism of this specimen is the mechanism (d) illustrated in Fig. 3.

From the backbone curves shown in Fig. 10 (a), it is obvious that both the lateral strength and stiffness of the retrofitted frames increased compared to those of the non-retrofitted frame R05P-P0. The lateral strength of the retrofitted specimen R09B-P in which the steel braces buckled increased to about 5 times of the lateral strength of the non-retrofitted specimen R05P-P0. Since, in the specimen R09B-P, plastic buckling happened in the stocky steel braces (the slenderness ratio of $\lambda = 35$),
Fig. 7 Photos of the specimens after finishing loading test and removing steel plates.

Specimen R05P-P0
Flexural plastic hinge
Shear failure
Flexural behavior followed by shear failure

Specimen R09B-F
Flexural plastic hinge
Flexural behavior of RC frame and steel frames

Specimen R09B-P
Plastic buckling
Flexural behavior of RC frame and steel frame, tension yielding and buckling of steel braces
Mechanism (a) **

Specimen R09B-CT
Direct shear failure
Direct shear failure at the top hybrid connection and at the RC columns
Mechanism (b) **

Specimen R10B-CO
Overturning
Overall-overturning
Mechanism (d) **

Specimen R10B-CP
Direct shear failure
Direct shear failure at the base
Mechanism (c) **

* The pointed plastic hinge locations are under push loading from the left-hand side to the right-hand side.
** The associated mechanism is illustrated in Fig. 3.
Notes: Crack patterns shown cracks after finishing loading tests and removing steel plates. * The mechanisms shown on the crack patterns are under the push loading only. ** Associated mechanisms are illustrated in Fig. 3. $R = \delta/h$ (where $\delta$ = lateral displacement of the first story, and $h$ = height of the first story), $N = 0.2hD\sigma_b$ (where $hD$ = sectional area of RC columns and $\sigma_b$ = concrete cylinder strength).

Fig. 8 Crack patterns and hysteresis responses of the test specimens.
moderate strength degradation was observed in the post-peak stage of the lateral resistance, especially up to the drift angle of $R = 2.0\%$.

The accumulated absorbed energies of the specimens are given in Fig. 10 (b). The accumulated absorbed energy is derived from the summation of the amount of the energy dissipated in each cycle of the loading. The amount of the energy dissipated in the specimen R09B-P in which buckling happened in the steel braces is the highest. It should be noted the displacement-controlled program of the non-retrofitted specimen R05P-P0 is different with that of the retrofitted specimens (in Fig. 6). Therefore, the difference in the loading programs causes a slight discrepancy in the comparison of the accumulated absorbed energies.

5. Ductility index

Application of the proposed hybrid connection provides superior structural performance for retrofitted RC columns especially in view point of ductility. Ductility of the specimens is quantified by an indicator called the “ductility index”. The specimens are categorized as two groups according to their failure types. One group represents the specimens with flexural failure in columns, and its specified ductility index ($F_1$) is given in Eq. (1) (JBDPA 2001). Another group represents the specimens with brittle failures such as punching and/or shear failure in boundary RC columns, and its corresponding ductility index ($F_2$) is given in Eq. (2) (Yamamoto 1985, and Hattori 2004).

$$F_1 = \left\{ \begin{array}{ll}
1.0 + 0.27 \frac{R_{\text{su}} - R_{R_0}}{R_y - R_{R_0}} & \text{in case } R_{\text{su}} < R_y \\
0.75(1 + 0.05R_{\text{su}} / R_y) & \text{in case } R_{\text{su}} \geq R_y 
\end{array} \right.$$

$$F_2 = 0.6 + 100R_{\text{su}} \leq 3.2$$

where $R_{\text{su}}$: the ultimate inter-story drift angle (%) at which lateral resistance decreases to 80% of maximum lateral resistance, $R_y$: the ultimate inter-story drift angle (%) at which lateral resistance reaches to the maximum lateral resistance, $R_y$: the yield deformation in terms of inter-story drift angle (%), which principally shall be taken as $R_y=1/150$ (JBDPA 2001).

The ductility indices of the specimens are given in Table 3. The comparison between propositionally-detailed specimens R09B-P and conventionally-detailed specimen R08B-F75 with the same steel braces sizes is more considerable. This comparison expresses that even though the sizes of installed steel braced frames $(75 \times 75 \times 4.5 \times 4.5)$ in both specimens are identical, the ductility index ($F_1=3.1$) of the specimen R09B-P with the hybrid connection is higher than the ductility index ($F_1=1.4$) of the specimen R08B-F75 with the conventional connection. The comparison quantitatively describes the role of the hybrid connection in improving the ductility of retrofitted RC columns. It should be noted that among different specimens retrofitted by steel braced frames utilizing the hybrid connection, the specimen R09B-P in which the steel braces buckled in compression and yielded in tension (the mechanism (a) in Fig. 3) has the most desired seismic performance with the high ductility index of $F_1=3.1$. The comparison between the conventionally-detailed specimens R08B-F75 and the propositionally-detailed specimen R09B-P clarifies two superior structural performances of the proposed connection to obtain high lateral capacity for retrofitted frame, and to increase shear strength of retrofitted RC columns with the help of steel plates. To obviously show the variation of the ductility index in the test specimens, the ductility indices versus the ultimate deformations are plotted in Fig. 11.
6. Calculation approaches

Four fundamental mechanisms (in Fig. 3) are taken into consideration for a RC frame retrofitted by a steel braced frame with the help of the proposed hybrid connection. The specimens were planned and tested to verify structural performance of each failure mechanism. In this section, the methods of calculating the lateral strength of the mechanisms are explained, and simplified design procedures are suggested.

6.1 Calculation method of the mechanism (a)

The idealized deformation of the mechanism (a) is shown in Fig. 12. In the mechanism (a), the lateral strength $V_a$ is mainly governed by tension and compression strengths of steel braces. The RC frame also participates in the global lateral strength. In the specimen R09B-P in which the mechanism (a) is the dominant mechanism, the strain gauges were attached on the steel frame at potential flexural plastic hinge regions. The gauges’ data did not show the formation of flexural plastic hinges in the steel frame at the ultimate lateral strength ($R_{mu} = 1.3\%$). Therefore, the participation of the steel frame in the global lateral strength of the retrofitted frame is conservatively ignored. The global lateral resistance $V_{RC}$ of the mechanism (a) is given in Eq. 3:

$$ V_a = (F_y + F_{bu}) \cos \theta + V_{RC} $$

where $V_{RC}$: global lateral strength of the mechanism (a), $F_y$: yield strength of the steel brace, $F_{bu}$: buckling strength of the steel brace, $\theta$: inclination of the brace toward the horizontal axis, and $V_{RC}$: flexural lateral strength of the RC frame.
6.2 Calculation method of the mechanism (b)

In the mechanism (b) shown in Fig. 13, direct shear failure happens at the top of the steel braced frame and at the top of the boundary RC columns. As given in Eq. 4, the direct shear strength $V_b$ consists of the direct shear strength of the hybrid connection $Q_{hyb}$ and the shear punching strength of the two boundary RC columns $2Q_{pu}$. 

$$V_b = Q_{hyb} + 2Q_{pu} \quad (4)$$

6.2.1 Direct shear strength of the hybrid connection $Q_{hyb}$:

Horizontal shear force between the top RC beam and the top steel beam is transferred by the hybrid connection. The hybrid connection consists of different contributing elements, namely steel plates, high-strength bolts, and infilling grout. So, the direct shear strength of the hybrid connection depends on the minimum value of the strengths of the contributing elements that is formulated in Eq. 5.

$$Q_{hyb} = 2 \times \min \{Q_{ps}, Q_{bs}, Q_{yb}, Q_{bg} \} \quad (5)$$

where $Q_{ps}$: shear strength of steel plates, $Q_{bs}$: bearing strength of steel plates’ holes, $Q_{yb}$: shear yield strength of bolts, and $Q_{bg}$: bearing strength of grout. In the shear transfer mechanism, the sandwiching steel plates deliver the provided horizontal shear force between the RC beam and the steel beam. The shear strength of steel plates $Q_{ps}$ is given in Eq. 6. Regarding the geometry of the sandwiching steel plates, the steel plate may yield or buckle under the produced shear force. The shear yield strength of steel plates $Q_{sy}$ is given in Eq. 7, and the elastic buckling strength of steel plates $Q_{crs}$ is written in Eq. 8. It was assumed that the steel plates are under a pure shear state. The shear buckling strength of steel plates is calculated based on the formulation by Timoshenko (1994). As shown in Fig. 14, the effective area of the sandwiching steel plates is limited to the positions of the bolts. It is assumed that bolts provide a simple support condition at the boundary of the effective area of the steel plates. Influences of the geometry and the boundary condition of the steel plates are taken into consideration in the buckling coefficient $k_s$. The buckling coefficient $k_s$ presented in Eq. 9 was suggested by Galambos (1988) for a rectangular steel plate which is simply supported on its four edges.

$$Q_{ps} = \min \{Q_{es}, Q_{crs} \} \quad (6)$$

$$Q_{es} = \frac{1}{\sqrt{3}} t_s l_e \sigma_{sy} \quad (7)$$

$$Q_{crs} = k_s \frac{\pi^2 E_s t_s l_e}{12(1-\nu^2)(h_{es})^2} \quad (8)$$

$$k_s = 5.34 + \frac{4.00}{(h_{es})^2} \quad (9)$$

where $Q_{ps}$: shear strength of the steel plate, $Q_{sy}$: shear yield strength of the steel plate, $Q_{crs}$: shear elastic buckling strength of the steel plate, $t_s$: thickness of the steel plate.
plait, \( h_{ps} \): effective length of the steel plate (in Fig. 14), \( \sigma_p \): tensile yield strength of the steel plate, \( k_b \): buckling coefficient, \( E_s \): Young’s modulus of the steel plate, \( \nu \): Poisson’s ratio, and \( h_{ps} \): effective height of the steel plate (in Fig. 14).

Bolts of the hybrid connection transfer direct shear force from the top RC beam and from the steel beam to the sandwiching steel plates. Direct shear yielding of the bolts is likely to happen under transferring shear force. The direct shear yielding of the bolts can be estimated through Eq. 10 (JBDPA 2001). Another important mechanism in transferring the direct shear force is the fracture of the steel plate around the holes. As given in Eq. 11, the bearing capacity of the steel plates’ holes is calculated according to the provisions by AISC-LRFD (1994) which considers both the tear fracture of steel plate’s material and the deformation of the steel plate around the bolts. Moreover, the grout embedding bolts in the hybrid connection zone should have sufficient strength and rigidity to strongly lock the bolts. On the other hand, the split of the surrounding grout under bearing stress of the bolts should be prevented. Consequently, the bearing strength of grout \( Q_{bg} \) should be checked through Eq. 12 (JBDPA 2001).

\[
Q_{jb} = 0.7 n_b \sigma_y \sigma_{jb} \\
Q_{ac} = 2.4 n_b t_z d_z \sigma_{ac} \\
Q_{as} = \left\{ 0.3 \sqrt{E_s \sigma_y \sigma_{as} \right\} n_b \sigma_y \leq 245 n_b \sigma_y \text{ (in N, mm)}
\]

where \( Q_{jb} \): shear yield strength of bolts, \( n_b \): number of bolts, \( \sigma_y \): cross-section area of a bolt, \( \sigma_{jb} \): tension yield strength of bolts, \( Q_{ac} \): bearing strength of steel plates’, holes, \( \sigma_{ac} \): ultimate tension strength of steel plates, \( t_z \): thickness of the steel plate, \( d_z \): diameter of a bolt, \( E_s \): Young’s modulus of grout, and \( \sigma_{as} \): cylinder strength of grout.

### 6.2.2 Shear punching resistance of a RC column \( Q_{pu} \)

The shear punching (direct shear) resistance of RC columns is calculated based on the method presented in the provisions by the Japan Building Disaster Prevention Association (JBDPA 2001). In calculation of the direct shear resistance \( Q_{pu} \), the influence of reinforcements cross the failure plane and external stresses normally act on the failure plane are considered. The calculation value of the direct shear strength of RC columns is given in Eq. 13. The factor of \( k_{min} \) in Eqs 13 & 14 represents the minimum estimation of the direct shear strength. In Eq. 14, \( a \) is the shear punching span between the direction of the applied shear force and the potential shear punching plane. According to the geometry of the proposed retrofit scheme, the shear punching span at the top of the retrofitted RC columns is considered as \( a = 10 \text{ mm} \).

\[
Q_{pu} = k_{min} \tau_0 b D
\]

\[
k_{min} = 0.34 / (0.52 + a / D)
\]

\[
\tau_0 = 0.98 + 0.1 \sigma_y + 0.85 \sigma_y \leq \sigma_y < 0.33 \sigma_y - 2.75
= 0.22 \sigma_y + 0.49 \sigma_y < 0.33 \sigma_y - 2.75 \leq \sigma_y < 0.66 \sigma_y
= 0.66 \sigma_y \sigma_y > 0.66 \sigma_y
\]

\[
\sigma = p_1 \sigma_y + N / (bD)
\]

where \( k_{min} \): the minimum value of the punching reduction factor, \( \tau_0 \): shear stress at the punching plane, \( b \): the effective width of the column resisting against the direct shear force considering the connected members in the orthogonal direction, \( D \): the depth of the column, \( a \): the shear punching span, \( \sigma \): normal stress act on the shear punching plane, \( p_1 \): cylinder strength of concrete, \( p_1 \): ratio of longitudinal reinforcements cross the shear punching plane, \( \sigma_y \): yield stress of longitudinal reinforcements, and \( N \): axial force.

### 6.3 Calculation method of the mechanism (c)

As shown in Fig. 15, in the mechanism (c), the direct shear failure happens at the base of the retrofitted frame. The direct shear strength at the base of a retrofitted frame is given in Eq. 17. Strengths of the boundary RC columns \( Q_{pu} \) and the anchors \( n_a Q_a \) (where \( n_a \) is the number of the anchors and \( Q_a \) is the direct shear resistance of an anchor) provide the required direct shear strength at the base.

\[
V_c = Q_{pu(1)} + Q_{pu(2)} + n_a Q_a
\]

Calculation of the direct shear strength \( Q_{pu} \) of RC columns is the same as that explained in Section 6.2.2. Direct shear strength of an anchor is given in Eq. 18 according to provisions by JBDPA (2001). The direct shear strength of an anchor \( Q_a \) depends on its yield shear strength \( Q_{as(1)} \) (in Eq. 19) and the bearing strength of surrounding concrete \( Q_{as(2)} \) (in Eq. 20).

\[
Q_a = \min \left\{ Q_{as(1)}, Q_{as(2)} \right\} Q_a / a_s \leq 294 \text{MPa}
\]

\[
Q_{as(1)} = 0.7 \sigma_{as} a_s
\]

\[
Q_{as(2)} = 0.4 \sqrt{E_s \sigma_y} a_s
\]
where $\sigma_y$: tension yield strength of anchors, $a_{sa}$: section area of an anchor, $E_c$: Young’s modulus of concrete, $\sigma_b$: cylinder strength of concrete.

6.4 Calculation method of the mechanism (d)

As shown in Fig. 16, in the mechanism (d), overall-overturning behavior appears in the retrofitted frame. When the retrofitted frame is pushed from the left-hand side to the right-hand side, the left-hand RC column and the anchors yield in tension, and the compression zone falls in the right-hand RC column and at the base of the compression brace. By taking moment about point O (shown in Fig. 16), the lateral strength $V_d$ of the mechanism (d) can be obtained that is written in Eq. 21. Based on the provisions by JBDPA (2001), the tension strength of an anchor $T_a$ depends on tension yield strength of the anchor $\sigma_y$, cone failure strength of the surrounding concrete $T_{a1}$, and bond failure strength between the anchor and the surrounding concrete $T_{a2}$ which are given in Eqs 23 -25, respectively.

\[
V_d = (a_g \sigma_y L + NL + n_{la} T_{a1} - F_c \sin \theta L_a) / h \tag{21}
\]

\[
T_a = \min \{T_{a1}, T_{a2}, T_a\} \tag{22}
\]

\[
T_{a1} = \sigma_y a_u \tag{23}
\]

\[
T_{a2} = 0.23 \sqrt{\sigma_b} A_c \text{ (in N, mm)} \tag{24}
\]

\[
T_{a3} = \pi a_0 D_a (l_a - D_a) \text{ (in N, mm)} \tag{25}
\]

\[
r_a = 10 \sqrt{\sigma_b} / 21 \tag{26}
\]

where $V_d$: lateral strength of the mechanism (d), $a_g$: section area of all longitudinal reinforcements of the tension-side columns, $\sigma_y$: tension yield strength of longitudinal reinforcements, $n_{la}$: number of anchors at the tension-side base plate, $\sigma_w$: tension yield strength of an anchor, $a_u$: section area of an anchor, $\sigma_b$: concrete strength, $A_c$: projected area of concrete cone failure surface of a single anchor (refer to JBDPA (2001)), $r_a$: bond strength of an anchor against pull-out force (MPa), $D_a$: diameter of an anchor, $l_a$: embedment length of an anchor, and other geometrical parameters are shown in Fig. 16.

7. Comparisons between the experimental and calculated results

The experimental results and the calculated lateral strengths of the specimens are given in Fig. 17. The calculated lateral strengths are based on the approaches presented in Section 6.

In the non-retrofitted specimen R05P-P0, flexural behavior of the RC frame is the dominant mechanism. However, shear failure finally happened in the RC columns. As shown in Fig. 17, from the calculated lateral strengths, the dominant behavior of the frame can be recognized.

In the specimen R09B-F, flexural behavior of the RC frame and the steel frame is the dominant mechanism. The calculated lateral strength of the mechanism (a) (flexural behavior of the RC frame and steel frame) is near the experimentally-obtained lateral strength. According to experimental results, it can be concluded that the jacketing steel plates effectively increased the shear resistance of the RC columns. It should be noted that in the non-retrofitted specimen R05P-P0 shear failure happened in the RC columns, but in this specimen the shear failure was perfectly prevented because of utilizing jacketing steel plates in the boundary columns.

In the specimen R09B-P, flexural behavior of the RC frame, tension yielding and buckling of the steel braces appeared in the experimental observations. As it is expected, the experimentally-obtained lateral strength of the specimen is near the calculated lateral strength of the mechanism (a).

In the specimen R09B-CT, since the thickness of the steel plates at the top hybrid connection is relatively thin ($t = 1.2\text{mm}$), direct shear failure happened at the top of the steel braced frame. The dominant mechanism of this specimen is the mechanism (b). As shown in Fig. 17, the calculated lateral strength of the mechanism (b) is close to the experimentally-obtained lateral strength.

In the specimen R10B-CP, because the anchor bolts of the base plates are relatively weak in direct shear resistance, direct shear failure happened at the base of the steel braced frame and at the bottom of the RC columns. As it is expected, the calculated lateral strength of the mechanism (c) is near the experimentally-obtained lateral strength.

In the specimen R10B-CO, since the anchor bolts of the base plates are relatively weak in tension resistance, the overall-overturning behavior appeared in the specimen. The dominant mechanism of this specimen is the mechanism (d). Because the overturning behavior is mainly generated by vertical slip of the longitudinal reinforcements at the base of the RC column in the tension side, the longitudinal reinforcements experienced high concentrated strain at that zone. Considering this
fact, the effect of strain-hardening of longitudinal reinforcements was considered in calculating the lateral strength of the mechanism (d). As shown in Fig. 17, the calculated lateral strength of the mechanism (d) and the experimentally-obtained lateral strength agree well.

8. Design procedure

Based on the calculation approaches of the fundamental failure mechanisms discussed in Section 6, the design flowchart to obtain the mechanism (a) as the dominant mechanism is presented in Fig. 18. According to experimental results of four fundamental failure mechanisms, the mechanism (a) is the most suitable one. In the retrofitted frame R09B-P in which the mechanism (a) was the dominant mechanism, not only the lateral strength and stiffness of the retrofitted frame increased, but also the ductility and energy absorption improved.

9. Summary and conclusions

In this paper, structural behaviors of the RC frames retrofitted by the steel braced frames are verified experimentally. The steel braced frames were installed inside the RC frames by a new connection method called the “hybrid connection technique”. Experimental investigations were performed to obtain four fundamental failure mechanisms of retrofitted RC frames. In addition to the experimental investigations, associated calculation procedures to estimate lateral strength of the fundamental failure mechanisms are suggested based on the provisions commonly used. The calculated lateral strengths by the proposed calculation procedures agree with the experimentally-obtained results. According to experimental observations and results, following conclusions are briefly expressed;

1) The hybrid connection can effectively transfer a relatively high direct shear force between an existing RC frame and an installed steel braced frame.
2) The hybrid connection not only plays a role as a connection between a RC frame and a steel frame, but also increases shear resistance and axial compression capacity of boundary RC columns.
3) The behavior of the specimen R09B-P in which the steel braces yielded in tension and buckled in compression (the mechanism (a)) shows superior struct-
tural performance by increasing the lateral strength and stiffness, as well as improving the ductility. So, the retrofit scheme of the specimen R09B-P with the dominant mechanism (a) can be suggested as the desired retrofit scheme for practical applications.

Acknowledgements

The authors would like to express their deepest gratitude to Prof. Chiaki Matsui, Professor Emeritus at Kyushu University for his valuable advice. The authors are indebted to Dr. Kozo Nakada, Associate Professor at University of the Ryukyus, for his valuable cooperation. The investigation reported herein was carried out possible by the grants supported in 2009 and 2010 from the Ministry of Land, Infrastructure, Transport, and Tourism of Japan. Also, this research was partially supported by the Grant-in-aid for the Scientific Research (A) in 2008 from Japan Society for the Promotion of Science (JSPS).

References

ACI Committee 318, (2008), “Building code requirement for structural concrete and commentary.” American Concrete Institute, Michigan DC.

of Steel Construction Inc., Chicago, USA.

Building Center of Japan, Japan. (in Japanese)

Building Center of Japan, Japan. (in Japanese)


