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A NEW SEISMIC RETROFIT TECHNIQUE FOR INSTALLATION OF A STEEL BRACED FRAME INSIDE EXISTING RC FRAME

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ABSTRACT

In this paper, a new hybrid connection for installation of a steel braced frame inside an existing RC (reinforced concrete) frame is presented. Three one-bay one-story RC bare frames were planed to be retrofitted in such a way that each specimen represented one of the fundamental mechanisms including shear-frame behavior, overall-flexural behavior and shear-sliding behavior. The retrofitted frames were tested under constant axial forces and cyclic horizontal loading. The obtained experimental results exhibited that the proposed hybrid connection can successfully deliver a relatively high shear force between the installed steel braced and the existing RC frame. In addition, the hybrid connection increases the shear resistance of the boundary RC columns by means of jacketing steel plates and PC bars (high-strength bolts). Associated calculation approach for estimating the direct-shear capacity of the proposed hybrid connection technique is discussed in this paper.

Introduction

In the past strong earthquakes such as the Izmit-Turkey 1999, the Kobe-Japan 1995 and the Northridge-USA 1994, a large number of RC (reinforced concrete) structures severely damaged or completely collapsed due to the soft-story mechanism. Application of steel braced frames inside the existing RC frames is a method in increasing the lateral strength and stiffness of the soft-story RC frames. The main concern in utilizing this method is the approach to appropriately connect the steel braced frame to the existing RC frames (FEMA-547 2006). Application of post-installed anchor bolts is the most common detail of connection which is also adopted in the guideline by Japan Building Disaster Prevention Association (JBDPA 2001). However, drilling holes into the RC frame to install the anchors is a noisy, dusty and destructive procedure which provides some difficulty at the building site.

Previous investigations by Yamakawa (Yamakawa 2006) and Rahman (Rahman 2007) have demonstrated that by utilizing thick hybrid wall technique not only the lateral strength and stiffness of the soft-story frames increase but also the ductility of non-ductile RC frames significantly improves. Based on the concept of thick hybrid wall technique, a new hybrid connection between the RC frame

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and the steel braced frame is proposed in this study. The proposed hybrid connection not only can transfer a relatively high shear force between the existing RC frame and the steel braced frame, but also successfully prevents the possible shear failure of non-ductile RC columns by means of jacketing steel plates and PC bars (high-strength bolts). The capacity of the hybrid connection in transferring high shear force leads to design the stocky steel braces with inelastic buckling behavior, to capture a high degree of energy absorption during a strong earthquake. Three specimens which were designed with different retrofit schemes are described in this study. The specimens are retrofitted in such a way that each test specimen exhibits one of the fundamental mechanisms including shear-frame behavior, overall-flexural behavior and shear-sliding behavior. In addition to experimental verifications, the associated calculation approach is presented for calculating the direct-shear resistance of the proposed hybrid connection at the top connection.

Test Plan

The general procedure for installation of a steel braced frame inside a RC frame is illustrated in Fig. 1. In this procedure, the channel-shaped steel plates jacket the boundary RC columns and steel columns with the help of PC bars (high-strength bolts). Also, two steel plates sandwich the top RC beam and the steel beam. The sandwiching steel plates were stitched together by means of PC bars crossing through the provided holes on the RC beam and the steel beam. After installation of steel plates and PC bars, high-strength grout was cast in the space among the steel plates, RC members and steel members, to provide sufficient rigidity in the connection zones. Finally, the PC bars are fastened with hand force.

In this study, three test specimens were planed to be retrofitted by the proposed method (see Fig. 2). The scale factor of the test specimens was 1/4~1/3, to model a low-rise school building designed according to pre-1971 Building Standard Law of Japan. The reinforcement details and the frame dimensions of all of the RC frames are identical. The reinforcement's details of the RC frames are shown in Fig. 3. In all of the test specimens at first the RC frame was cast and cured, and then, after at least 28 days, the retrofitting procedure was implemented. The horizontal cyclic loading and vertical constant loads of $N=0.2\sigma_B bD$ (per column) were simultaneously acted on the frame specimens during the experimental tests.

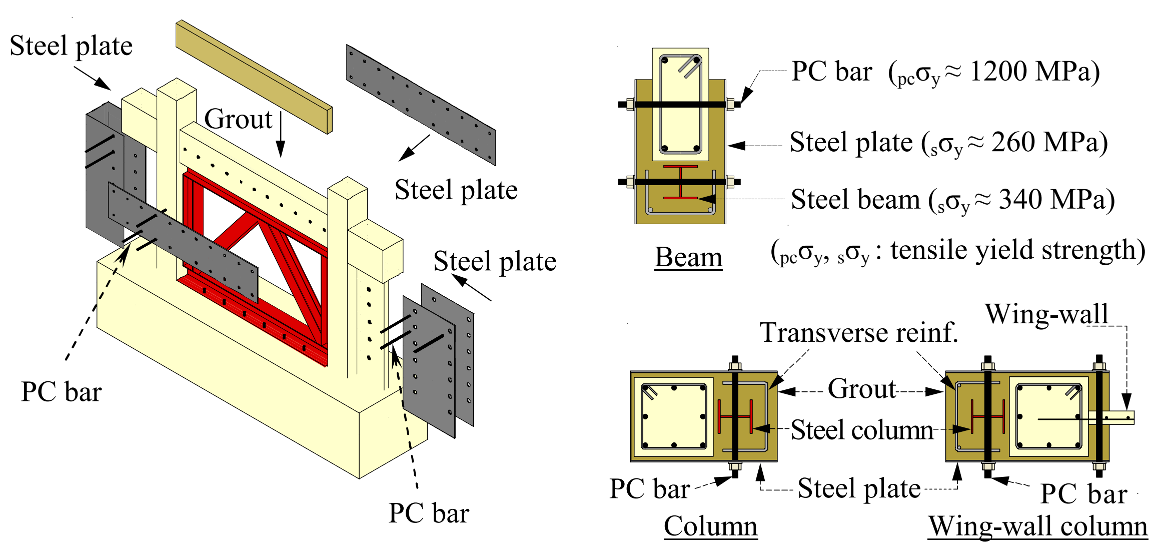
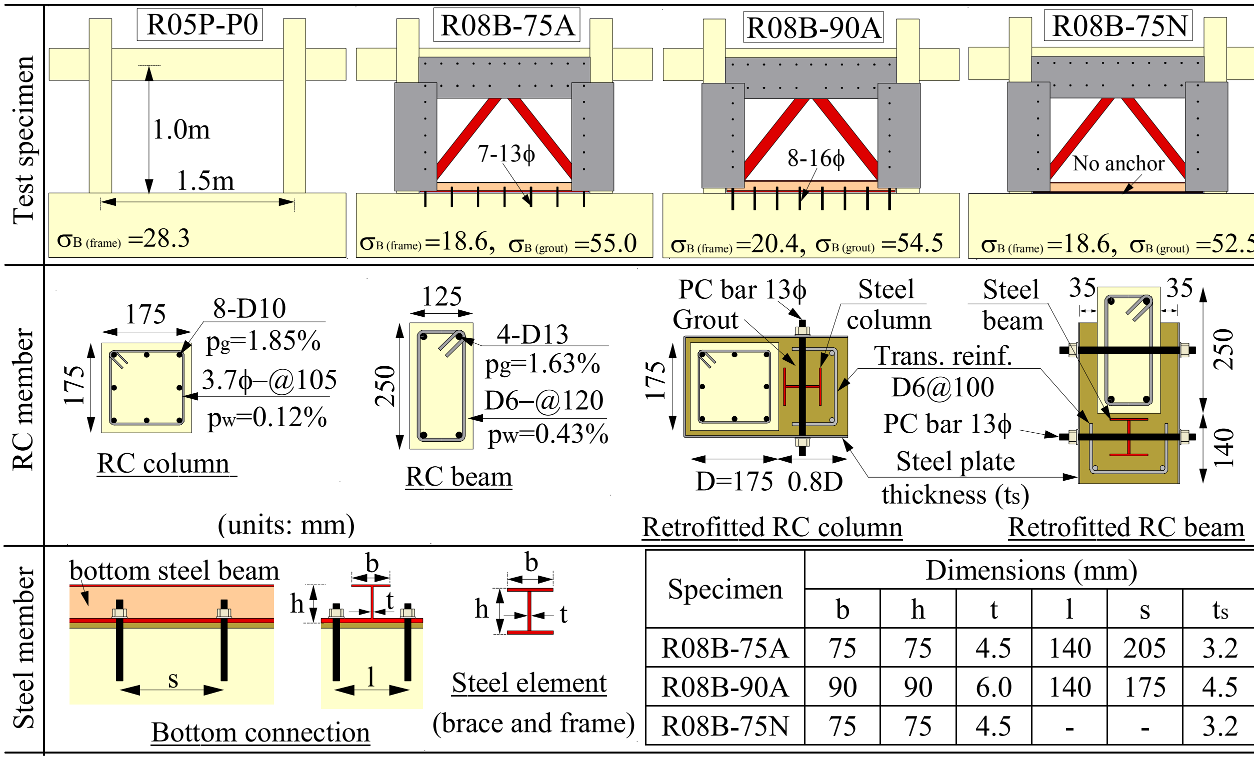


Figure 1. Proposed retrofit technique



Specimen	Dimensions (mm)					
	b	h	t	l	s	ts
R08B-75A	75	75	4.5	140	205	3.2
R08B-90A	90	90	6.0	140	175	4.5
R08B-75N	75	75	4.5	-	-	3.2

Figure 2. Details of the test specimens

As shown in Fig. 2, the specimen R05P-P0 is a non-retrofitted RC frame. This specimen is used as the benchmark specimen.

The specimen R08B-75A was retrofitted by steel braced frame with the help of the proposed hybrid connection. In retrofitting the test specimen, a fabricated steel braced frame (BH 75x75x4.5x4.5mm) was installed inside the RC frame. Holes were made on the steel columns and the steel beam to cross the PC bars. The channel-shaped steel plates (t=3.2mm) jacketed the boundary RC columns. U-shaped hoops were arranged on the exposed faces of the jacketing steel plates to prevent the spalling of grout at the compression zone. After installation of jacketing steel plates, PC bars (13ϕ) crossed through the provided holes on the steel plates and steel columns. Also, at the top, the plane steel plates sandwiched the RC beam and the steel beam with the help of PC bars crossed through the drilled holes on the RC beam and the punched holes on the steel beam. Then, high-strength grout (σ_B=55.0MPa) were cast in the provided space among the RC members, the steel members, and the steel plates, to provide sufficient rigidity in the connection parts. The PC bars were fastened with hand force. In this specimen, the bottom steel beam of the steel braced frame was connected to the stub by means of anchors (7-13ϕ). For this specimen, it is expected that the shear-frame mechanism (buckling of steel brace in compression, and flexural behaviors of the RC columns and the steel columns) would be the dominate mechanism.

The specimen R08B-90A was retrofitted in the almost same way as operated for the specimen R08B-75A. The differences are the dimensions of steel braces, and the size and the arrangement of the bottom anchors. In the specimen R08B-90A, the steel braces (BH-90x90x6.0x6.0) are stocker and stronger than those of the specimen R08B-75A (BH-75x75x4.5x4.5). Also, the bottom anchorage (8-16ϕ) in the specimen R08B-90A is stronger compared to the specimen R08B-75A. The main objective of this retrofit design is to observe the overall-flexural behavior.

The retrofit procedure of the specimen R08B-75N is the same as the specimen R08B-75A. But

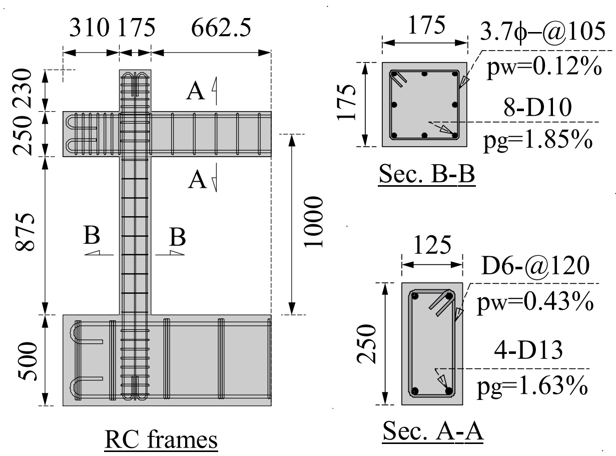


Figure 3. Reinforcement details

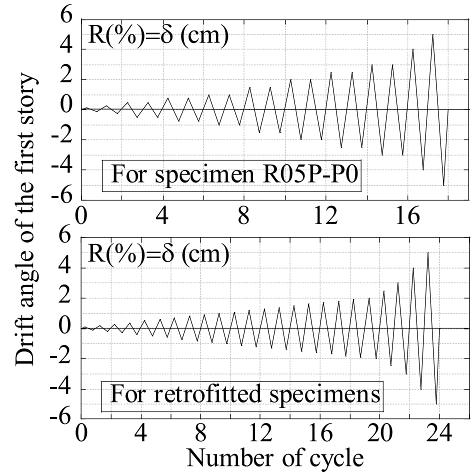
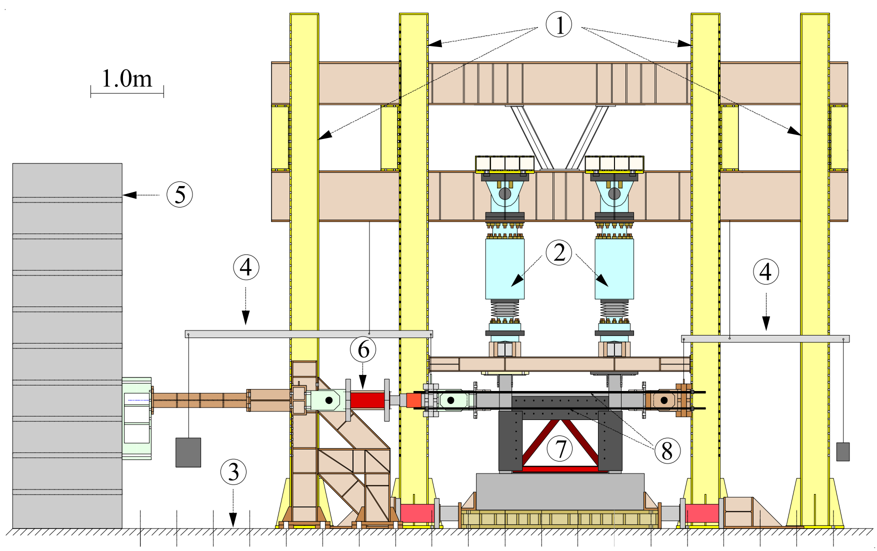


Figure 4. Loading programs



Devices:

1. Vertically loading reaction steel frame
2. Servohydraulic actuator
3. Strong floor
4. Counter balance
5. Horizontally loading reaction RC wall
6. Hydraulic oil jack
7. Test specimens
8. Long PC bar

Figure 5. Test setup

in this specimen, the bottom steel beam was freely placed on the stub. The objective of this retrofit design is to observe the shear-sliding behavior at the bottom of the retrofitted frame.

The displacement-controlled horizontal loading programs and the test setup are given in Figs. 4 and 5, respectively.

Experimental Results

The crack patterns of the test specimens at the final stages of the loading tests, their dominant mechanisms, and their *V*-*R* relationships are shown in Fig. 6.

In the non-retrofitted specimen R05P-P0, flexural cracks appeared at the ends of the RC columns and at the ends of the top RC beam at drift angle of about R=0.5% and R=1.0%, respectively. At drift angle of R=0.67%, the longitudinal reinforcements of the RC columns started yielding. Shear cracks at the columns generated at about R=1.5% and widened progressively with increasing drift angle. At drift angle of R=2.5% in the push direction (+) of the first cycle, the width of the shear crack in the right-hand column was about 5mm. In the pull direction (-) of the same cycle at drift angle of R=2.0%, the right-hand column collapsed suddenly due to the shear failure.

In the specimen R08B-75A, flexural cracks generated at the bottom of RC columns at the drift angle of R=0.2%. The longitudinal reinforcements at the bottom of RC column yielded at drift angle of R=0.46%. Buckling-shaped deformations initiated in the steel braces at drift angle of R=0.8%, and at drift angle of R=1.3% considerable plastic rotations occurred at about mid-length of the steel braces. Moreover, local buckling was observed at the ends of the steel braces where the steel braces embedded in the additional high-strength grout. Occurrence of local buckling at the ends of the steel braces in the post-buckling stage resulted from inelastic bending of the brace due to P-Δ moment produced by compression axial force. As shown in Fig. 5, shear-frame behavior, mechanism (a), was the dominant mechanism. In the retrofitted specimen, the lateral strength increased to about 5.6 times of the non-retrofitted specimen R05P-P0. The lateral resisting force of the specimen maintained greater than $0.8V_{\max}$ up to finishing loading test at the drift angle of R=3.0%. The experimental result of this test specimen demonstrated that firstly proposed hybrid connection can successfully transfer high shear force between the top RC beam and the steel beam. Secondly, in the post-bucking stage, the hybrid connection could sustain the provided vertical unbalanced force at the joint of the braces. Thirdly, by retrofitting the RC columns by jacketing steel plates, the shear failure (as happened in the non-retrofitted specimen R05P-P0) was perfectly prevented.

The specimen R08B-90A was retrofitted by the steel braced frame with the stocky braces. Since the steel braced frame was strong, the overall-flexural mode, mechanism (b), appeared in its global response. At the drift angle of R=0.9%, the lateral resisting force reached to its ultimate value which is 7.4 times of the lateral strength of the non-retrofitted specimen R05P-P0. The longitudinal reinforcements of the RC columns started breaking at the drift angle of R=1.8% due to

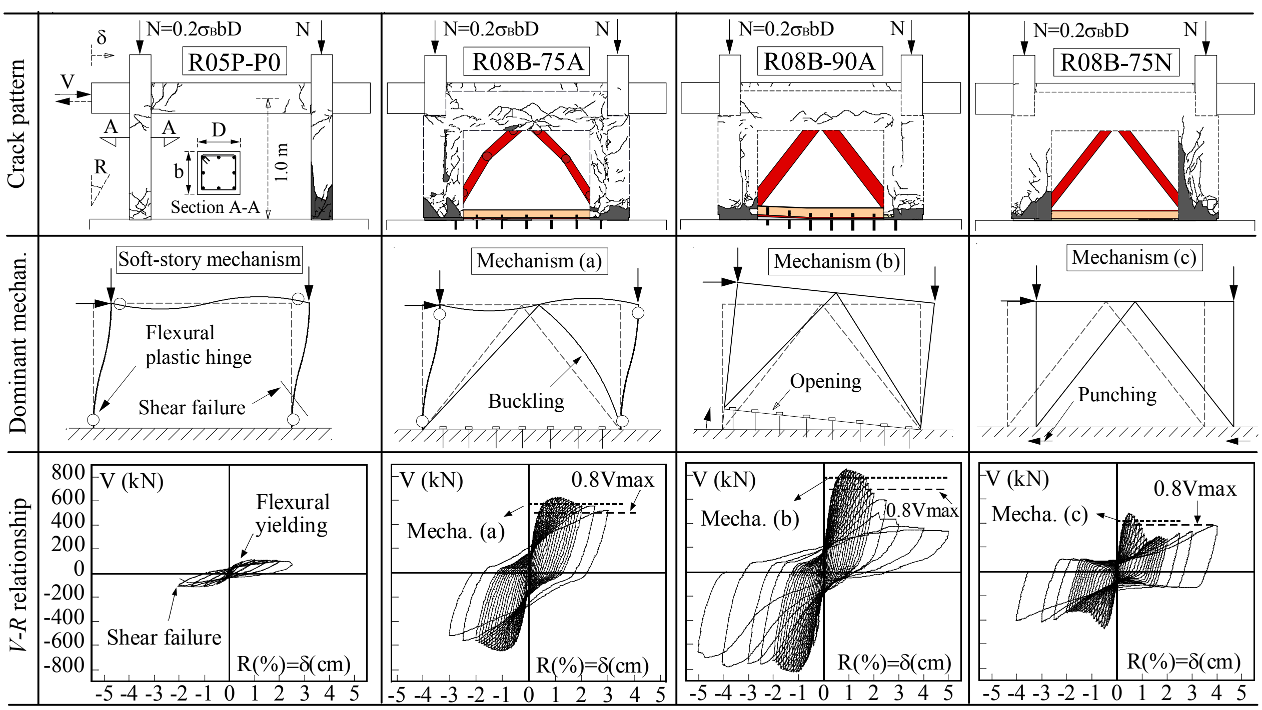


Figure 6. Experimentally obtained results of the test specimens

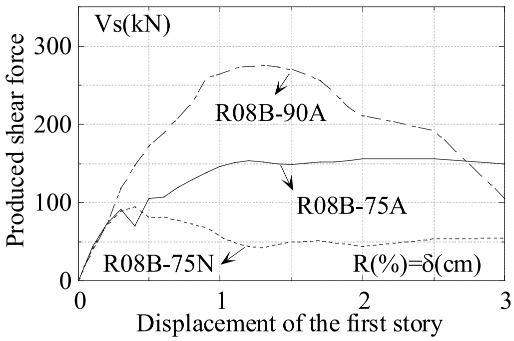


Figure 7. Shear force of one steel plate at the top connection

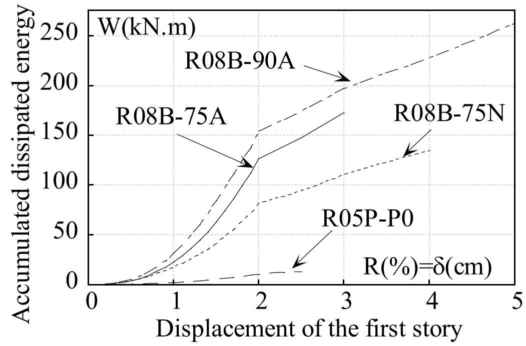


Figure 8. Accumulated dissipated energy of the test specimens

their alternately stretching and buckling at the localized zones at the base of RC columns. The breakage of longitudinal reinforcements had continued up to the drift angle of R=4.0% at which all of the reinforcements of the left-hand column broke. The longitudinal reinforcements of the RC beam had not yielded during loading test. This specimen also demonstrated the capacity of the proposed hybrid connection to deliver relatively high direct-shear force at the top connection.

In the specimen R08B-75N, the bottom steel beam was freely placed on the stub to observe the possible sliding at that surface. After yielding the longitudinal reinforcements and widening the flexural cracks at the bottom of RC columns, at drift angle of R=0.5%, the sliding initiated at the base. The shear resistance force at the base of the specimen derived from shear-punching resistance of RC columns and shear-friction between the steel beam and the stub. From drift angle of R=0.5% to drift angle of R=1.3%, the lateral strength gradually decreased due to the deterioration of shear-friction resistance. By increasing the loading test from moderate to large drift angle (1.3%<R<4.0%), the lateral resisting force again gradually increased due to contacting the jacketing steel plate with the stub.

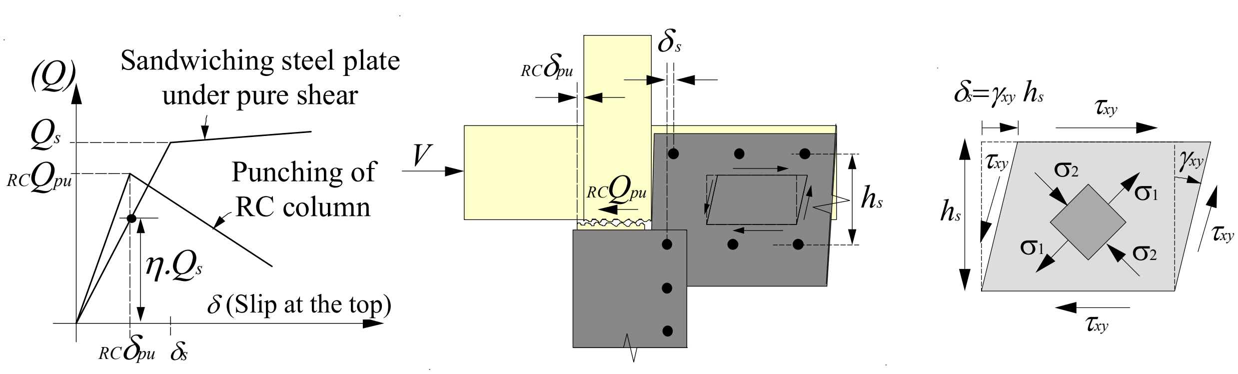
The direct-shear resistance at the top of the retrofitted frame is carried by the boundary RC columns and the hybrid connection. To find out the contribution of sandwiching steel plate in carrying the direct-shear force, three-components strain gauges were attached on the centers of the sandwiching steel plates. The shear forces in the steel plates were calculated regarding a uniform shear strain along the lengths of the sandwiching steel plates. In Fig. 7, the produced shear forces in the steel plates are shown. For example in the specimen R08B-90A in that the lateral resisting force is the highest one, the contribution of steel plate is the greatest. In the specimen R08B-90A, the maximum shear force induced in only one steel plate is about 276kN which is 33% of the ultimate lateral resisting force of the specimen. The accumulated dissipated energies of the test specimens are shown in Fig. 8. It is evident that after retrofitting, the energy dissipations of the test specimens significantly increased.

Direct-Shear Resistance of the Hybrid Connection at the Top

In this study, the most important objective is the mechanism of transferring shear force between the floor RC beam and the steel beam. The horizontal shear force is delivered from the floor RC beam to the steel braced frame through the shear-friction of the boundary RC columns and direct-shear resistance of the hybrid connection. Shear-friction resistances of the RC columns can be calculated through the common guidelines such as Japan Building Disaster Prevention

Association (JBDPA 2001), American Concrete Institute (ACI-318 1995), etc. In this section the attention mainly focuses on the direct-shear resistance of the hybrid connection. In superimposition of the shear-friction resistance of RC columns and the horizontally direct-shear resistance of hybrid connection, it should be taken into consideration that the produced shear force in the steel plates reaches to its ultimate strength or not. Since the produced shear force in the steel plate depends on the relative horizontal deformation, the resistance contribution of sandwiching steel plate should be carefully verified. On the other hand, the resistance contribution of the steel plates depends on the relative horizontal deformation at which the punching failure is likely to happen in the RC columns. Moreover, the geometry of the steel plate affects the contribution factor. This mechanism strongly influences the shear resistance contribution of sandwiching steel plates in case of real buildings in which the dimensions of sandwiching steel plates are relatively large. Experimental investigations by Hofbeck (Hofbeck 1961) on push-off test specimens showed that the shear-punching resistance of the test specimens reached to their ultimate strengths at relative slip deformation in the range of 0.2mm< ${}_{RC}\delta_{Pu}$ <0.5mm. In this study, the average slip displacement of ${}_{RC}\delta_{pu}$ =0.4mm is considered as the deformation index at which the ultimate direct-shear resistance of RC columns and the hybrid connection should be calculated.

As shown in Fig. 9, it should be verified that how much shear force are carried by the sandwiching steel plates Q_s and the boundary RC columns ${}_{RC}Q_{pu}$. This calculation approach is used for taking into consideration the displacement compatibility between the hybrid connection and the boundary RC columns. For considering this importance, the contribution reduction factor η for the sandwiching steel plate should be calculated. In calculating the contribution reduction factor η of the sandwiching steel plates, it is assumed that a pure shear deformation filed produces along the length of the steel plate. An element of the sandwiching steel plate is shown in Fig. 9. The maximum produced shear stress in the element is presented in Eq. 1. The shear yielding stress τ_y of the element is given in Eq. 2, according to Von Mises criteria. The maximum principle stress is governed by the buckling stress of the steel plate or the yielding strength. The buckling strength of the steel plate can be estimated by the formulation calculated by Timoshenko (Timoshenko 1961) based on the theory of elasticity (see Eq. 3). In Eq. 3, k_s is the buckling coefficient which depends on the geometry and boundary condition of the steel plate. As shown in Fig. 10, the PC bars provide the simple support condition for the steel plate. The buckling coefficient k_s for a steel plate which is simply supported on its four edges is presented in Eq. 5 (Galambos 1988). The provided horizontal deformation in the steel plate under pure shear at its yield strength or buckling strength is given in



Contribution of steel plate in direct shear resistance | Shear resistance at the top | Pure shear state

Figure 9. Contribution of sandwiching steel plate in direct-shear resistance

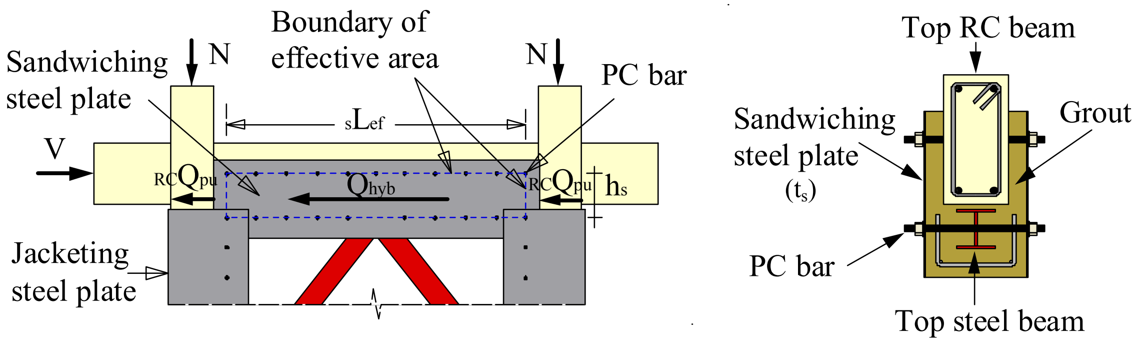


Figure 10. The geometry details of the hybrid connection

Eq. 6. By obtaining the shear deformation in the steel plate and assuming that the punching failure will happen in the boundary RC columns at the relative slip of ${}_{RC}\delta_{Pu}$ =0.4mm, the contribution reduction factor of the sandwiching steel plates can be calculated through Eq. 7.

$$\tau_{xy,\max} = \min\left\{ {}_s\tau_y,\ {}_s\tau_{cr} \right\} \tag{1}$$

$$ {}_s\tau_y = {}_s\sigma_y / \sqrt{3} \tag{2}$$

$$ {}_s\tau_{cr} = k_s \frac{\pi^2 E_s}{12(1-v^2)(h_s/t_s)^2} \tag{3}$$

$$\gamma_{xy,\max} = \frac{\tau_{xy,\max}}{G} \tag{4}$$

$$k_s = 5.34 + \frac{4.00}{({}_sL_{ef} / h_s)^2} \tag{5}$$

$$\delta_{s,\max} = \left|\gamma_{xy,\max}\right| h_s \tag{6}$$

$$\eta = \frac{{}_{RC}\delta_{pu}(\cong 0.4\text{mm})}{\delta_{s,max}} \le 1.0 \tag{7}$$

where $\tau_{xy,\max}$: maximum produced shear stress; ${}_s\tau_y$: shear yielding stress; ${}_s\tau_{cr}$: critical buckling stress; ${}_s\sigma_y$: yield stress; k_s: buckling coefficient; E_s: Young's modulus of elasticity; ν: Poisson's ratio; h_s: height of steel plate; t_s: thickness of the steel plate; G: shear modulus; ${}_sL_{ef}$: length of steel plate; $\delta_{s,\max}$: maximum horizontal shear deformation in the steel plate; $\gamma_{xy,\max}$: shear strain in the steel plate; η: contribution reduction factor; ${}_{RC}\delta_{Pu}$: relative displacement at the top of the RC columns at which the punching failure is likely to happen.

As presented in Eq. 8, the horizontal direct-shear capacity of the hybrid connection depends on the capacities of the assembled elements, namely the sandwiching steel plates, the stitching PC bars, and the infilling grout. The PC bars transfer the shear forces from the top RC beam and also from the steel beam to the sandwiching steel plates through dowels actions. Under dowel resistance, direct-shear yielding of PC bars is likely to happen. The direct-shear capacity of the PC bars can be estimated through Eq. 9. Moreover, the grout which is embedding the PC bars in the hybrid connection zone should have sufficient rigidity and strength to keep the PC bars without any relative movement. On the other hand, when the PC bars resist against the produced shear force, the split of surrounding grout under bearing stresses is likely to happen. So, the bearing capacity of the additional grout in the hybrid connection zone should be checked through Eq. 10 which is adopted by JBDPA (JBDPA 2001) for bearing capacity of concrete. Another important mechanism is the fracture of the steel plate holes. As presented in Eq. 11, the bearing capacities of the steel plate holes are calculated according to the provisions by AISC-LRFD (AISC 1994), which considers both the tear fracture of the steel plate's materials and the deformation around the bolts holes.

Regarding the geometry of the sandwiching steel plate (its effective length ${}_sL_{ef}$, and thickness t_s, see Fig. 10), the steel plate may yield or buckle under the produced shear force. The yield strength ${}_sQ_y$ and the buckling strength of the steel plate ${}_sQ_{bu}$ are suggested to be calculated through Eq. 12 and 13, respectively, considering a pure shear strain field along its effective length ${}_sL_{ef}$.

$$Q_{hyb} = 2\times min\left\{{}_{pc}Q_y,\ {}_{gr}Q_b,\ {}_sQ_b,\ \eta Q_s\right\} \tag{8}$$

$$_{pc}Q_y = n_{pc}\,.\,a_{pc}\,.\,{}_{pc}\sigma_y/\sqrt{3} \tag{9}$$

$$_{gr}Q_b = n_{pc}\,.\left(0.4\sqrt{E_c.\sigma_B}\right).a_{pc} \le n_{pc}\left(245(N/mm^2)\right).a_{pc} \tag{10}$$

$$_sQ_b = n_{pc}\,.(2.4\ {}_s\sigma_u\,.d\,.t_s) \tag{11}$$

$$Q_s = {}_sQ_y = t_s\,.\,{}_sL_{ef}\,.\,{}_s\sigma_y/\sqrt{3} \qquad if\ (h_s/t_s)<\beta \tag{12}$$

$$Q_s = {}_sQ_{bu} = k_s\frac{\pi^2E_s}{12(1-\nu^2)(h_s/t_s)^2}\,.\,{}_sL_{ef}\,.t_s \qquad if\ (h_s/t_s)\ge\beta \tag{13}$$

$$\beta = \sqrt{\frac{k_s\pi^2E_s}{12(1-\nu^2)_s\sigma_y}} \tag{14}$$

where Q_{hyb}: capacity of hybrid connection; ${}_{pc}Q_y$: direct-shear yielding of PC bars; ${}_{gr}Q_b$: bearing capacity of grout embedding the PC bars; ${}_sQ_b$: bearing force between the PC bars and steel plates; Q_s: shear resistance of steel plate; η: reduction contribution factor (see Eq. 7); n_{pc}: number of PC bars in a horizontal row; a_{pc}: section area of a PC bar, ${}_{pc}\sigma_y$: yield strength of PC bars; E_c: Young's modulus of grout; σ_B: compressive strength of grout; ${}_s\sigma_u$: ultimate strength of the steel plate in pure tension; d: diameter of PC bars; t_s: thickness of steel plate; ${}_sL_{ef}$: effective length of steel plate; β: slenderness parameter; ${}_s\sigma_y$: yield strength of steel plate.

Conclusions

In this paper a new hybrid connection method for installation of a steel braced frame inside a RC frame is presented. One non-retrofitted one-bay one-story RC frame and three RC frames retrofitted by the proposed method were tested under constant axial forces and cyclic horizontal loading. Based on the experimental results, the following conclusions can be briefly explained;

1) In the non-retrofitted specimen R05P-P0, after flexural yielding, shear failure happened in the RC column. But, after retrofitting the RC columns by the jacketing steel plates, the shear failure were perfectly prevented in the boundary RC columns.
2) The proposed hybrid connection successfully delivers the horizontal direct-shear force from the steel braced frame to the RC frame. Moreover, the hybrid connection perfectly sustains the unbalanced downward force at the joint of the braces in the post-buckling stage, as observed in the behavior of the specimen R08B-75A.
3) In the specimen R08B-90A, it was observed that the steel braced frame effectively resists against the provided overturning moment.
4) The shear punching failure at the base of the specimen R08B-75N demonstrated the necessity of utilizing appropriate anchorage system at the base of the retrofitted frames.
5) The associated calculation approach, for estimating the horizontal direct-shear resistance of the hybrid connection at the top, is proposed.

Acknowledgments

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