(51) Seismic Retrofit of Soft-story RC Frames by Steel Braced Frames with the Help of Hybrid Connection

Pasha JAVADI¹ • Tetsuo YAMAKAWA² • Koki MAEDA³ • Makoto KOBAYASHI⁴

¹AIJ member Graduate Student Graduate School of Engineering and Science University of the Ryukyus (Japan, Okinawa, 1-Senbaru, Nishihara, 903-0213) E-mail: pasha Javadi@yahoo.com

²AIJ member Professor Department of Civil Engineering and Architecture University of the Ryukyus (Japan, Okinawa, 1-Senbaru, Nishihara, 903-0213) E-mail: yamakawa@tec.u-ryukyu.ac.jp

³AIJ member Assistant Technician Department of Civil Engineering and Architecture University of the Ryukyus (Japan, Okinawa, 1-Senbaru, Nishihara, 903-0213) E-mail: k ryu univ@yahoo.co.jp

⁴AIJ member Graduate Student Graduate School of Engineering and Science University of the Ryukyus (Japan, Okinawa, 1-Senbaru, Nishihara, 903-0213) E-mail: makoto.13214@yahoo.co.jp

This paper presents a new hybrid connection for installation of a steel braced frame inside an existing RC frame. Three specimens were planed to be retrofitted with steel braced frames by using hybrid connection and also by utilizing the conventional method (studs and anchors). The hybrid connection is assembled from steel plates, PC bars and infilling grout. The specimens were tested under constant axial forces and cyclic horizontal loading. Experimental results exhibited that the hybrid connection can successfully transfer direct-shear force from the floor RC beam to the steel braced frame. The proposed hybrid connection obtains a desired structural performance and provides ease in operation. Calculation approach for estimating the direct-shear capacity of hybrid connection is presented based on mechanisms of its assembled elements.

Keywords: hybrid connection, steel braced frame, soft-story, retrofit, strengthening, experimental tests

1. INTRODUCTION

In the past strong earthquakes in Japan such as the 1995 Hyogoken-nanbu earthquake (commonly called the Kobe earthquake), a considerable number of RC buildings presented a risk of poor seismic performance during earthquakes¹⁾. The most important reason of collapses of the RC buildings was soft-story mechanism. The soft-story mechanism may happen in a building with a discontinuity in its lateral-resistance system. According to capacity-based concept which was proposed by Paulay et al.²⁾, the global response of a building will be restricted to the lateral strength of its softest story. Moreover, regarding the displacement-based concept, the relative lateral displacement in the soft story will be greater than those in other stories. This condition will be more critical in the case of soft-first-story buildings.

Nowadays, a large number of RC buildings in Japan and

other countries, which are located on high seismic zones, have the characteristics of soft-story mechanism. Considering this fact, it necessitates to improve seismic performance of soft-story buildings by increasing their lateral strength and stiffness. Among different building types, retrofit of school RC buildings, which were designed according to pre-1971 Building Standard Law of Japan, is the most important target not only for their regular usages but also for sheltering the survivors.

Application of a steel braced frame inside an existing RC frame is a method to improve seismic performance of vulnerable soft-story RC buildings. However, connection approach for installation of a steel braced frame inside a RC frame is a primary concern of this method. Application of post-installed anchors and studs is the most conventional approach for connecting the steel braced frame to the existing RC frame. However, drilling holes into the RC frame to install the anchors is a noisy and dusty procedure that provides

some difficulties at the construction site. In addition, in case of non-ductile RC frame, even after retrofitting by the conventional method, brittle shear failures are likely to happen in the boundary RC columns which carry gravity loads.

Previous experimental investigations by Yamakawa et al.^{3), 4), 5)} have demonstrated that by utilizing steel-jacketed thick hybrid wall, not only the lateral strength and stiffness of soft-story frame significantly improve, but also possible shear failures of RC columns are perfectly prevented. In following the previous investigations, in this study, a new connection system called "Hybrid Connection" will be introduced. In the hybrid connection technique, plain steel plates sandwich the steel beam and the RC beam with the help of PC bars (high-strength bolts). Also, channel-shaped steel plates jacket the boundary RC columns and steel columns through PC bars.

To verify the efficiency of the proposed technique in improving seismic performance of soft-story frames, experimental investigations were conducted on soft-story RC frames retrofitted by steel braced frame. The geometric dimensions of the test specimens were 1/4~1/3 of a real low-rise school building designed according to pre-1976 Standard Law of Japan. The experimental results confirmed the efficiency of the proposed retrofit technique in improving the lateral strength and stiffness of a soft-story RC frame with the help of hybrid connection. Moreover, the calculation of direct-shear resistance of hybrid connection is presented regarding mechanisms of its assembled elements.

2. PROPOSED APPROACH

General procedure in installation of a steel braced frame inside an existing RC frame is shown in Fig. 1. In the proposed retrofitting method, a steel braced frame is installed inside an existing RC frame. Transverse plain steel plates, at the top of the frame, sandwich the RC beam and the steel beam with the help of PC bars. Channel-shaped steel plates jacket the boundary RC columns and steel columns by means of PC bars. After installation of steel plates, spaces among the steel plates, the RC frame and the steel frame are filled by high-strength grout. After hardening of the additional highstrength grout, the PC bars are fastened with hand force. To prevent the possible sliding at the base of the retrofitted frames anchor bolts should be installed at the bottom. The main objective of this investigation is to experimentally verify the seismic performance of hybrid connection.



Fig. 1 General procedure of the proposed retrofit technique

3. TEST PLAN

Details of original RC frames are shown in Fig. 2. In all of the test specimens, details of the RC frames are identical. The shear span-to-depth ratio (M/(VD)) of the RC columns was 2.5, and that of the RC beams was 2.65, regarding their clear spans. The ratio of transverse reinforcements of the RC columns was the poor the value of $\rho_w=0.12\%$. Due to the poor amount of transverse reinforcements in RC columns, shear failures of RC columns are likely to happen in case of the non-retrofitted frame. Retrofit details of the test specimens are shown in Fig. 3. Four test specimens are planed to be verified experimentally. In all of the test



Fig. 2 Details of reinforcement (unit:mm)



Fig. 3 Details of test specimens

specimens, at first the original RC frame was cast and cured and then, after at least 28 days, the retrofitting implementation was carried out.

The **specimen R05P-P0** is a non-retrofitted frame which is used as a benchmark specimen to compare its experimentally obtained results with those of the retrofitted specimens.

The specimen R08B-75P was retrofitted by steel braced frame. In the retrofitting procedure, a fabricated steel frame with chevron-shape braces (inverted V-shape, BH-75x75x4.5x4.5) was installed inside the RC frame. Channelshaped steel plates (t=3.2mm) jacketed the boundary RC columns and steel columns, and then the PC bars crossed the provided holes on the steel plates. Transverse reinforcements (D6-@100mm) were arranged at the exposed face of the jacketing steel plates to prevent the spalling of grout at that zone. About 10 mm gaps were considered between the jacketing steel plate and RC column and steel column to ensure that additional grout can easily flow and fill the discontinuity between them. At the top connection, two transverse steel plates (t=3.2mm) sandwich the RC beam and steel beam with the help of PC bars (13ϕ) . Also, at the top hybrid connection, the additional high-strength grout filled the existing empty space between the steel plates and RC beam to provide the requirement rigidity. The steel columns were connected to the stub through base plates

which were connected to the stub with anchors $(4-16\phi)$. For this part of connection, holes were drilled on the stub, and then anchor bolts cross through the provided holes on the base plates and inserted to epoxy-injected holes in the stub.

The **specimen R08B-75D** is retrofitted by a steel braced frame in a same way as specimen R08B-75P, but its connection to the stub is different. In the specimen R08B-75D, stud and anchor dowels (conventional technique) were used at the base of specimen. The headed stud dowels (13 ϕ -@94mm) were welded to the bottom steel beam. Also, the headed anchor dowels (13 ϕ -@94mm) were installed to the stub. The saw-tooth-shape stud dowels and anchor dowels were arranged between each other. After installation of stud dowels and anchor dowels, additional grouts were cast between them. The connection of this specimen at the bottom follows the typical details suggested by Japan Building Disaster Prevention Association (JBDPA)⁶.

Retrofitting procedure of the **specimen R08B-75N** is almost in the same way as specimen R08B-75D. The difference is that, in specimen R08B-75N, the bottom steel beam of steel frame was placed on the stub without any anchorage system to observe the possible shear-sliding at that surface.

All specimens were tested under cyclic horizontal loading with a displacement-controlled program and constant



<u>Note</u>: 1. Vertically loading reaction steel frame; 2. Servohydraulic actuator; 3. Strong floor; 4. Counter balance; 5. Horizontally loading reaction wall; 6. Hydraulic oil jack; 7. Test specimen; 8. Long PC bar

Fig. 4 Test setup and loading programs

vertical loads ($N=0.2\sigma_B bD$, per column) with a loadcontrolled program. Schematic view of test setup and the

Steel elements		a (mm ²)	σ _y (MPa)	ε _y (%)	E _s (GPa)
Rebar	$D10^*$	71	412	0.21	195
	D10	71	355	0.18	201
	D13 [*]	127	341	0.17	200
	D13	127	342	0.17	201
Hoop or stirrup	3.7¢*	11	643	0.32	199
	3.7ф	11	617	0.33	188
	$D6^*$	32	393	0.22	176
	D6	32	449	0.29	153
Dowel	13¢ (SD345)	127	371	0.20	183
	16¢ (SD345)	199	366	0.21	176
PC bar	13φ	133	1220	0.66	200
Steel plate	t = 3.2 mm	-	263	0.12	211
	t = 4.5 mm	-	337	0.16	206
	t = 16.0 mm	-	299	0.15	199

Table 1 Properties of steel elements

Notes: a= cross-section area; σ_y = yield stress; ε_y = yield strain; E_s= Young's modulus; (*) for specimen R05P-P0 only.

	Concrete of RC frame			Additional grout		
Specimen	$\sigma_{\rm B}$	3	Ec	$\sigma_{\rm B}$	3	Ec
	(MPa)	(%)	(GPa)	(MPa)	(%)	(GPa)
R05P-P0	28.3	0.22	27.2	-	-	-
R08B-75P	21.2	0.16	27.6	52.2	0.36	23.0
R08B-75D	21.2	0.16	27.6	52.2	0.36	23.0
R08B-75N	18.6	0.20	23.2	52.5	0.39	21.6

Notes: σ_B = compressive strength obtained by cylinder test specimen, ϵ_c = strain at σ_B ; E_c = modulus of elasticity.

displacement controlled programs are illustrated in Fig. 4. The horizontal cyclic loading was planned to perform at drift angles in the range of R=0% to $R=\pm 2\%$ with incremental amplitude of $\pm 0.1\%$, and then, at drift angles of $R=\pm 2.5\%, \pm 3.0\%, \pm 4.0\%$ and $\pm 5.0\%$, respectively. The properties of steel, concrete and grout materials are listed in Tables 1, 2.

4. EXPERIMENTAL RESULTS

Crack patterns at final drift angles and *V-R* relationships of the test specimens are given in Fig. 5. The crack patterns show the cracks, at final step of the loading, which were drown after removing the steel plates. Also, the backbone curves of the specimens are shown in Fig. 6.

In the non-retrofitted **specimen R05P-P0**, flexural cracks appeared at the ends of columns and at ends of the top beam at drift angle of about R=0.5% and R=1.0%, respectively. The longitudinal rebars in the columns started yielding at about R=0.67%. 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 5 mm. In the pull direction (-) of the same cycle at drift angle of R=2.0%, the right-hand column collapsed suddenly in shear failure.

In specimen R08B-75P, flexural cracks generated at bottom of RC columns at drift angle of R=0.3%. The longitudinal reinforcements at the bottom of columns yielded at drift angle of R=0.8%. Buckling-shape



Fig. 5 Crack patterns at final drift angle and V-8 relationships of the test specimens

deformation initiated in the steel braces at drift angle of R=0.6%, and, at drift angle of R=1.2%, considerable plastic rotation occurred at about mid-length of the steel braces. Moreover, local bucklings were observed at the ends of steel braces where the steel braces embedded in additional highstrength grout. Occurrence of local buckling at the ends of steel braces in the post-buckling stage resulted from inelastic bending of the brace due to $P-\Delta$ moment produced by compressive axial force. On the other hand, there should be a high-degree of rotational constraint at the end of the steel brace to cause the local buckling. According to experimentally observed behavior and on the safe side, it is suggested that the end part of the steel brace where it is embedded in the additional high-strength grout is considered as an intermediate state between the fixed-fixed condition and the pined-pined condition. The lateral resisting force gradually enhanced up to the ultimate strength at drift angle of R=0.9%. After the lateral resisting force reached to the ultimate strength, strength deterioration appeared in the V- δ response. The strength deterioration can be related to post-buckling strength deterioration of steel brace in compression and to shear sliding displacement at base of the specimen. Also, Bauschinger effect (material nonlinearity) in steel braces has some influence on the strength deterioration under cyclic loading. However, the lateral resisting force maintained greater than $0.8V_{max}$ until drift angle of R=2.5%. At drift angle of R=4.0%, the anchors of left-hand column broke and the loading test terminated. Since the axial strength of the compression brace after its buckling considerably decreased while the tension brace perfectly

resisted in tension, a vertical unbalance force produced in the brace joint at the top. The experimental observed behavior exhibited that the applied hybrid connection at the top beam could successfully sustain the provided unbalanced force in the post-buckling stage. The crack pattern of the specimen shows that only some hair-line cracks formed at the beam connection (see Figs 5, 7). It is also worthy to remind that by application of proposed method, the possible brittle shear failure of RC columns perfectly prevented in this specimen.

In the **specimen R08B-75D**, the flexural cracks appeared at the base of boundary RC columns at drift angle of R=0.1%. The longitudinal reinforcements at the bottom of the RC columns yielded at drift angle of R=0.5%. During loading test, cracks gradually formed on grout of the bottom







Specimen R08B-75D



Specimen R08B-75P



Specimen R08B-75N Fig. 7 Crack Patterns of test specimens

connection. The lateral resisting force of the test specimen increased up to drift angle of R=0.7%. Then, the lateral resisting force suddenly decreased due to punching failure of the bottom connection and boundary RC columns. At drift angle of R=1.3%, the stud dowels broke at their ends where they were welded to the bottom steel beam due to fatigue phenomenon under cyclic loading. Finally, the loading test terminated at drift angle of R=2.0%. It is worthy to note that in this specimen, at the top, hybrid connection is utilized while, in the bottom, the conventional connection is applied. The value of shear force at top and the bottom are equal, but it is evident that the hybrid connection successfully transferred the direct-shear force while the conventional connection failed.

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 of longitudinal reinforcements and widening the flexural cracks at 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 the 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 resisting force gradually decreased due to deterioration of shear-friction resistance. By continuing the loading test for moderate to large drift angles (1.3<R<4.0), the lateral resisting force again gradually increased because of contacting the jacketing steel plate with the stub.

5. HYBRID CONNECTION RESISTANCE

In a RC frame retrofitted by steel braced frame, the horizontal shear force mainly transfers from the floor RC beam to the top steel beam. So, in this paper, the attention focuses on the method of calculating the direct-shear resistance of hybrid connection at the top of the steel braced frame. The shear resistance of boundary RC columns jacketed by channel-shaped steel plate is deferred to the future study. The calculation approaches for direct-shear capacities at the bottom of specimens (studs and dowels system) are referred to the provisions by JBDPA⁶. The previous experimental investigations by Yamakawa et al.⁷⁾ demonstrated that, at the top, boundary RC columns and hybrid connection resist against provided shear force. The direct-shear resistance of RC columns can be explained by shear-friction theory which is used in the most design and retrofit guidelines such as JBDPA⁶. Different elements are assembled in the hybrid connection. Horizontal shear capacity of the hybrid connection depends on the capacities of the assembled elements, namely, sandwiching steel plates, stitching PC bars and filling grout. The capacity of hybrid connection can be estimated through Eq. (1).

$$Q_s = 2 \text{ (both sides)} \times \min \left\{ {}_{PC}Q_y \text{ , }_{g}Q_b \text{ , }_{s}Q_b \text{ , }_{s}Q_y \text{ , }_{s}Q_{bu} \right\} (1)$$

Where ${}_{PC}Q_{y}$: yield capacity of PC bars; ${}_{g}Q_{b}$: bearing capacity of embedding grout; ${}_{s}Q_{b}$: bearing capacity of holes on the steel plate; ${}_{s}Q_{y}$: yield capacity of sandwiching steel plate; ${}_{s}Q_{bu}$: buckling capacity of sandwiching steel plate. The PC bars transfer the shear force from the top RC beam and also from the steel beam to the sandwiching steel plates. Under this mechanism, direct-shear yielding of PC bar is likely to happen. The total direct shear capacity of the PC



Fig. 8 Hybrid connection at the top of steel frame

bars can be estimated through Eq. (2).

$${}_{s}Q_{y} = \frac{1}{\sqrt{3}} n_{PC} a_{PC} p_{C} \sigma_{y}$$
⁽²⁾

Where n_{pc} : number of PC bars; a_{PC} : cross-section area of a PC bar; $p_C\sigma_y$: yield stress of PC bars. 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. So, the bearing capacity of additional grout in hybrid connection zone should be checked through Eqs (3a) and (3b) which are adopted by JBDPA⁶ for bearing capacity of concrete.

$${}_{g}\mathcal{Q}_{b} = 0.4\sqrt{E_{g}\sigma_{B}} n_{PC} a_{PC}$$
(3a)

$$0.4\sqrt{E_g\sigma_B} \le 245(N/mm^2) \tag{3b}$$

Where E_g : Young modulus of grout; σ_B : compressive strength of grout; n_{PC} : number of PC bars; a_{PC} : cross-section of a PC bar. The sandwiching steel plates should have sufficient strength to resist against the provided direct-shear force. Based on the geometry of sandwiching steel plates, three mechanisms namely, yielding of steel plate, buckling of steel plate or fracture of holes, are likely to happen. Yield strength of steel plate can be estimated through Eq. (4), considering a pure shear deformation field along the effective length of the steel plate.

$${}_{s}Q_{y} = \frac{1}{\sqrt{3}} t_{s} L_{ef s} \sigma_{y} \tag{4}$$

Where t_s : thickness of steel plate; L_{ef} : effective length of steel plate; ${}_{s}\sigma_{y}$: yield stress of steel plate in pure tension. In estimating total direct-shear resistance at the top of the retrofitted frame, it is assumed that the shear-friction resistances of the boundary RC columns are added to the shear capacity of the hybrid connection. This assumption will be valid for small relative deformation only, because for large relative deformation, the shear friction resistance of the RC columns may deteriorate through the possible punching

failure. According to this mechanism, and for small relative deformation, elastic buckling strength of steel plate is estimated only and their ultimate post-buckling strength under relatively large deformation is ignored. The elastic buckling capacity is calculated based on formulation by Timoshenko et al.⁸⁾ regarding the theory of elastic stability (see Eq. (5a)). The effective area of sandwiching steel plate is limited to the positions of PC bars and it is assumed that PC bars provides simple support condition at boundary effective zone of the steel plates.

$${}_{s}Q_{bu} = k_{s} \frac{\pi^{2} E_{s}}{12(1-\nu^{2})(h_{s} / t_{s})^{2}} L_{ef} t_{s}$$
 (5a)

$$k_s = 5.34 + \frac{4.00}{\left(L_{ef} / h_s\right)^2}$$
(5b)

Where k_s : buckling coefficient; E_s : Young modulus of steel plate; v: poison's ratio of steel plate; h_s : effective height of steel plate; t_s : thickness of steel plate; L_{ef} : effective length of steel plate. The associated buckling coefficient presented in Eq. (5b) was suggested by Galambos⁹ for a steel plate which is simply supported on its four edges and loaded in pure shear. The bearing capacity of holes is presented in Eq. (6) according to the provision by AISC¹⁰ without consideration of resistance factor.

$${}_{s}Q_{b} = 2.4 \, {}_{s}\sigma_{u} \, d_{pc} \, t_{s} \, n_{pc} \tag{6}$$

Where ${}_{s}\sigma_{u}$: ultimate stress of steel plate in pure tension; d_{pc} : diameter of PC bar; t_{s} : thickness of steel plate; n_{pc} : number of PC bars. As an example the calculation capacity of hybrid connection for specimen R08B-75P is presented in Appendix.

6. SUMMARY AND CONCLUSIONS

The experimental investigations were conducted on onebay one-story RC frames. The specimens were tested under constant axial forces and cyclic horizontal loading. Three specimens were retrofitted by steel braced frames and one specimen is a non-retrofitted one. In the retrofitted specimens, hybrid connection was utilized at the top of the steel braced frames and boundary columns, and, at the bottom, different connection types were used. The following conclusions can be briefly explained;

(1) In the non-retrofitted specimen R05P-P0, it was observed that, after flexural yielding, shear failure occurred in the RC columns. However, after utilizing hybrid connection, the shear failures of RC columns prevented, because the hybrid connection plays two important roles as a connector and a shear-enhancing device for the adjacent RC member.

- (2) In the retrofitted specimen R08B-75P, at the top, the steel braced frame was connected to the RC beam through the proposed hybrid connection, and at the bottom, the base plate and anchor bolts were used to install the steel braced frame. The experimental result demonstrated that the hybrid connection has sufficient direct-shear resistance to capture the ultimate capacities of steel braces which were strong and stocky.
- (3) In the retrofitted specimen R08B-75D, at the top, the steel braced frame was connected to the RC beam through the proposed hybrid connection, and at the bottom, the conventional method was utilized. The experimental results showed that the hybrid connection could sustain the provided direct-shear resistance while the conventional method failed in transferring the shear force. Also, the steel braces did not reach to their ultimate strengths.
- (4) In the retrofitted specimen R08B-75N, at the top, the hybrid connection was applied, but the bottom steel beam of the steel braced frame was placed freely on the stub. The experimental results exhibited the necessity of utilizing anchorage system at the base of the retrofitted specimen to prevent the brittle punching failure at base.
- (5) Associated design equations are suggested for hybrid connection regarding the possible mechanisms of the assembled elements.

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APPENDIX

As an example the shear capacity of hybrid connection at the top of the specimen R08B-75P is calculated as follows;

From Eq. (2);

$$PCQ_y = \left(\frac{1}{\sqrt{3}} \times 10 \times 133 \times 1220\right) \times 10^{-3} = 937 \ kN$$

From Eq. (3b) and Eq. (3a), respectively;

$$\left\{0.4\sqrt{(23.0 \times 10^3) \times 52.2} = 438.3 \, N \, / \, mm^2\right\} \not\leq 245 \, N \, / \, mm^2$$

$${}_g Q_b = (245 \times 10 \times 133) \times 10^{-3} = 326 \, kN$$

From Eq. (4);

$${}_{s}Q_{y} = \frac{1}{\sqrt{3}} \times 3.2 \times 1250 \times 263 = 607.4 \ kN$$

From Eq. (5a) and Eq. (5b), respectively;

$${}_{s}Q_{bu} = 5.4 \times \frac{\pi^{2} \times (211 \times 10^{3})}{12 \times (1 - 0.3^{2})(172.5/3.2)^{2}} \times 3.2 \times 1250 = 1417 kN$$

$$k_{s} = 5.34 + \frac{4.00}{(1250/172.5)^{2}} = 5.4$$

From Eq. (6);

$$_{s}Q_{b} = 2.4 \times 337 \times 13 \times 3.2 \times 10 = 336 \ kN$$

From Eq. (1);

$$Q_s = 2 \times \min \left\{ PCQ_y \ , \ _gQ_b \ , \ _sQ_b \ , \ _sQ_y \ , \ _sQ_{bu} \right\} = 2 \times 326 = 652 \, kN$$

It should be noted that in calculation of the total direct-shear resistance at the top, the capacity of hybrid connection should be added to the punching resistance of boundary RC columns.

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