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TEST AND BEHAVIOR OF STEEL BEAM AND REINFORCED CONCRETE COLUMN CONNECTIONS

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SUMMARY

The objective of this research is to investigate the seismic behavior of steel beam to RC column connections with or without the floor slab, and to act as a proof test for the design of connections of a three-story three-bay reinforced concrete column and steel beam (RCS) in-plane frame at the NCEER, Taiwan tested in the year of 2002 by the international research corporation between Taiwan and USA. Totally six cruciform RCS joint sub-assemblages were constructed and tested. Parameters such as composite effect of slab and beam, stirrups in the panel zone, effect of cross beam, loading protocol, and analytical model for the shear transfer in panel zone were all investigated in this study.

INTRODUCTION

RCS moment frame systems consist of reinforced concrete columns and steel beams. Using RC rather than structural steel as columns can result in substantial material cost savings, increased structural damping and lateral stiffness of the building. To date, RCS connections can be characterized as two main categories: beam through type and column through type. Based on literatures, beams continuously passing through column panel zone (beam through type) behaved in a ductile manner under seismic loading; however, orthogonal moment connection in the panel zone may be labor intensive. Column through type using diaphragms or cover plates to connect steel beams and column may facilitate field construction, however, extra effort in connection details to ensure a better seismic capacity in terms of strength and ductility is needed.

Since 1989, researches on RCS composite system have been started by Deierlein [1], and Sheikh [2] in Texas University, where 15 beam-through-type connections without slab were tested. Two distinguished failure modes were pointed out such as panel zone yielding and bearing failure of column concrete due to the cyclic loading as shown in Figure 1. In 1993, Konno [3] tested a series of RCS connections without slab. Research parameters included hoop details in panel zone, column axial load, and bearing strength of concrete. Test results showed that seismic capacity of RCS systems is not less than reinforced concrete or steel structures. Since 1997, corporations for research on RCS system have been conducted in US and Japan such as Baba [4], Kim [5], Nishiyama [6], Parra-Montesinos [7], and Bugeja [8].

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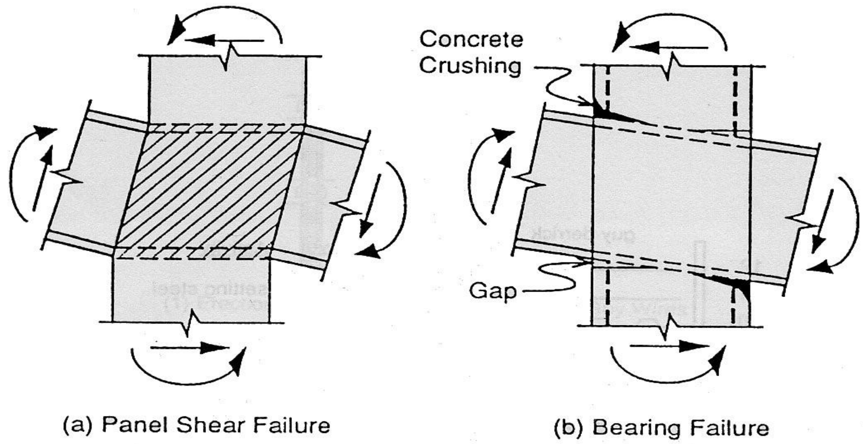


Figure 1 Failure mechanism of inner panel based on Kanno [3]

To study the composite effect of slab and steel beam, Yu [9] have tested several composite steel beams to steel column or steel reinforced concrete column (SRC) connections. Test results showed that composite effect may vary with types of connection, distribution of shear stud, floor thickness and amount of RC steel in slab. In general, shallow beam depth used in low-to-mid-rise building tends to have larger composite effect. Test results also revealed that slab provided lateral support for the beam flange to prevent torsional buckling. In addition, floor concrete also increased the flexural stiffness for composite beams. Beside this research, Liu [10] also conducted 6 tests of composite beam to steel or SRC column connections. Investigated parameters included connection details, shear stud and floor reinforcement. Test results showed that composite effect sustained until drift angle reached 0.04 radians.

In 2002, research corporations between Taiwan and US proposed to test a full size three-story-three-bay in-plane RCS frame. Before this test, seismic behavior of beam-column connections needs to be clarified. Based on literatures, beam-through type connections may have better seismic performance than that of the column-through type. Therefore, six beam-through type composite beam-column sub-structures were designed and tested. Investigated parameters include composite effect of slab, hoop details in panel zone, effect of transverse beam and loading protocol.

EXPERIMENTS

In the full-size plane frame, the span of columns was 7 meters center to center with 4 meters of story height. Based on loading combinations, beam sections from roof to the first floor of frame were designed to be H396X199X7X11, H500X200X10X16 and H596X199X10X15, respectively. In the sub-structure tests, all specimens representing beam-column connections in the first floor of in-plane frame, therefore, have the same dimension with steel beam H596X199X10X15 in size and 65X65cm columns reinforced with 12-#11 longitudinal bars. Based on the research of Kanno [11], panel zone of beam-through-type connections can be divided into two elements: inner and outer element. Failure modes in the inner element can be panel shear yielding or bearing failure of column concrete; while failure modes in the outer element may be bond failure of longitudinal reinforcement or panel shear yielding. To prevent these premature failures for all specimens, two retrofit techniques were applied as shown in Figure 2. To prevent bearing failure of column concrete, band plates (BP) were embedded around column right above or beneath the steel beam. To enhance the shear transfer in panel zone, face-bearing plates (FBP) were fillet welded to the beams at column face.

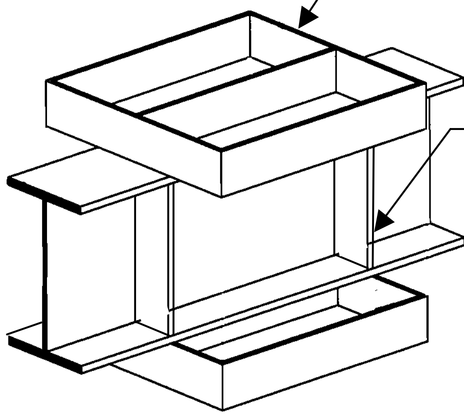


Figure 2 Two retrofit techniques in the panel zone of beam-through type connections

In the numbering of six specimens, first character, I, represents interior column connections. The second character, C and N, means connections with or without cross beam in the orthogonal direction, respectively. The third character represents shape of hoops reinforced in the panel zone. As shown in Figure 3, U, L and Square shape of hoops were used in the specimens with or without cross beam intersected in panel zone, respectively. The fourth character distinguishes different loading protocol with C to be cyclic loading and P represents near-fault pulse-type loading. If fifth character S added, it means composite beam with slab. Figure 4 shows the beam and column details for all specimens. Figure 5 shows the distribution of shear studs on composite beam, and temporary brace for the pour of floor concrete. As shown in the figure, slab was reinforced with #3 bars spanned 30 cm in the bottom layer and wire mesh with 100x100 mm in spacing at the upper layer. Material strengths of steel are summarized in Table 1. Table 2 shows the compressive strength for concrete. As show in Figure 4, column and a shorter beam were precast together at factory for easy transportation and then spliced with an extended beam at laboratory. Specimen ICLC was the first to be tested, however, slip of bolts at the beam splice occurred during test. Therefore, the cover plates at the beam splice were all fillet welded for the remaining tests.

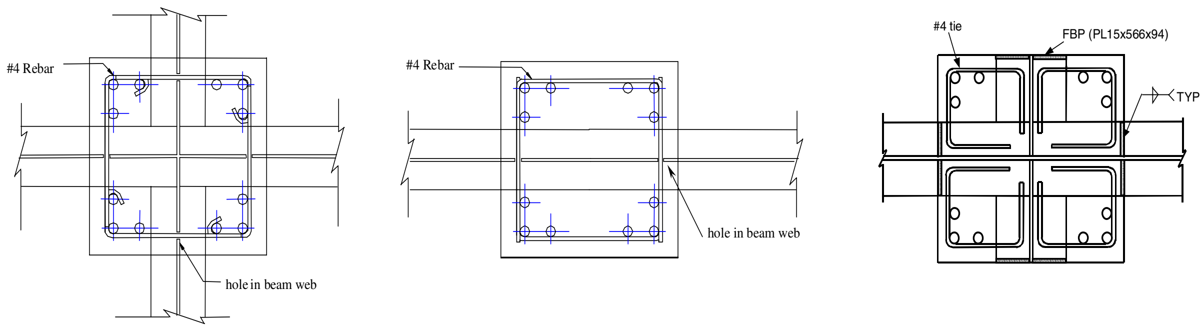
Table 1 Material strength for steel

Item	Rebar(#3)	Rebar(#4)	Rebar(#11)	Plate(10mm)	Plate(15mm)
Fy (MPa)	442.3	430.7	443.3	478.5	444.2
Fu (MPa)	650.3	680.6	674.6	598.7	568.0

Figure 6 shows the test apparatus. Before test, the hydraulic jack at top of column applied a 1000 kN constant axial load. Then hydraulic actuators at each beam end applied the cyclic load with displacement control in the form of triangular waves as shown in Figure 7. During test, horizontal actuator at top of column held the column in position, but allowed it to rotate accordingly. For specimen ICLPS, loading protocol simulates the waveform of near fault excitations as shown in Figure 8, based on the report of Krawinkler [12].

Table 2　Concrete strength

Specimen	Column(f_c')MPa	Slab (f_c')MPa
ICLCS	48.9	22.5
INUCS	54.5	24.3
ICLPS	49.9	21.0
ICLC	52.4	-
ICSC	42.7	-
INUC	54.3	-



(a)Stirrup in panel zone (4 L Shape)　(b)Stirrup in panel zone (2 U Shape)　(c)Stirrup in panel zone (4 Square tie)

Figure 3 Shape of stirrups in the panel zone

Test results show that all specimens performed in a ductile manner with plastic hinge formed at the beam end near the column face, where local buckling took place successively at beam flange and web. For specimens with slab, composite effect disappeared after 3% of drift because of insufficient shear transfer provided by shear studs. Due to the lateral support of floor slab, lateral torsional buckling at top flange was suppressed, however, bottom flange buckled and even fractured during final stage of test. Visual observation revealed that all specimens except ICLPS performed similarly with each test concluded at the drift of 6% where fracture of beam bottom flange and only minor damage such as cracks observed at column and panel zone. In the test of two specimens INUCS and ICLCS, fracture of bottom beam flange and separation of beam and slab was visualized during test, but these phenomena were not observed till the end of test for specimen ICLPS. For specimens without slab, local buckling at top and bottom beam flange was observed at the same time.

Figure 9 shows the hysteretic curve of load and displacement at east beam end for all specimens. Based on this figure, the strength and stiffness of each specimen was summarized in Table 3. Under positive bending, it was found that initial stiffness and ultimate strength of composite beam averagely increased 67% and 27% respectively, compared to steel beam without slab. Under negative bending, the average ultimate strength of specimens with slab is 1.02 times of specimens without slab. Loaded by near-fault loading protocol, the post-peak deterioration of specimen ICLPS is less than that of other specimens loaded by incremental loading protocol.

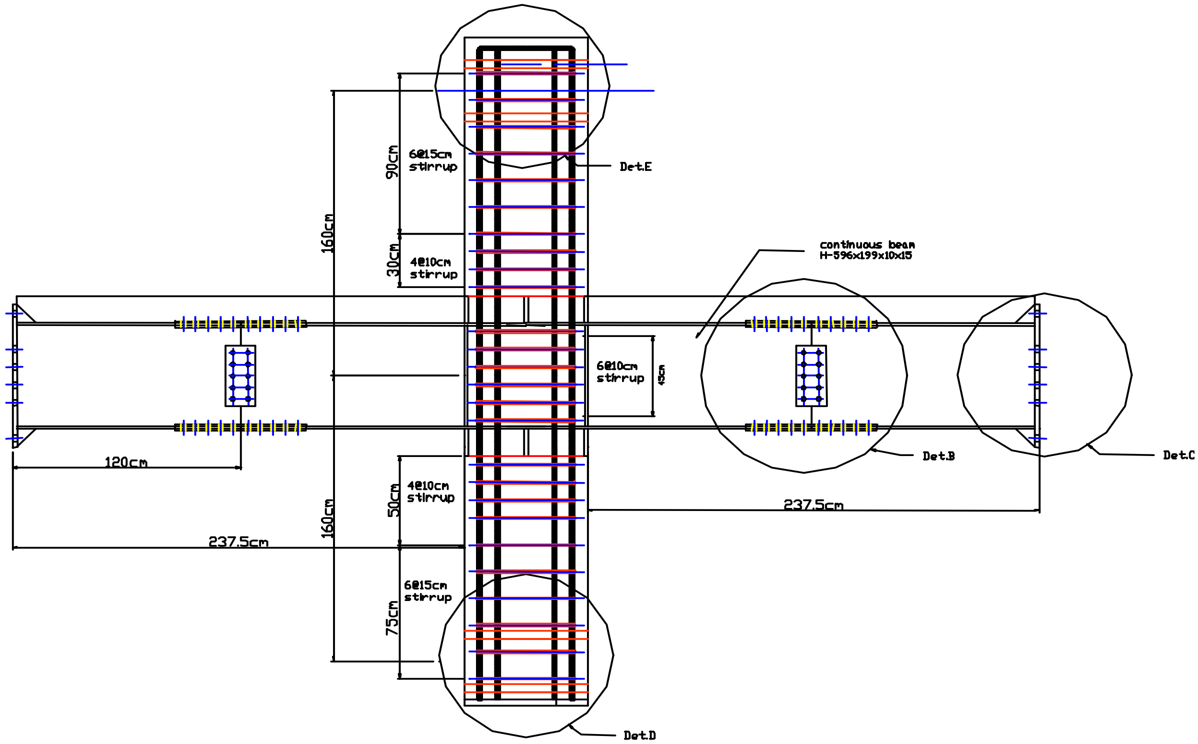


Figure 4 Details of beam-column joint

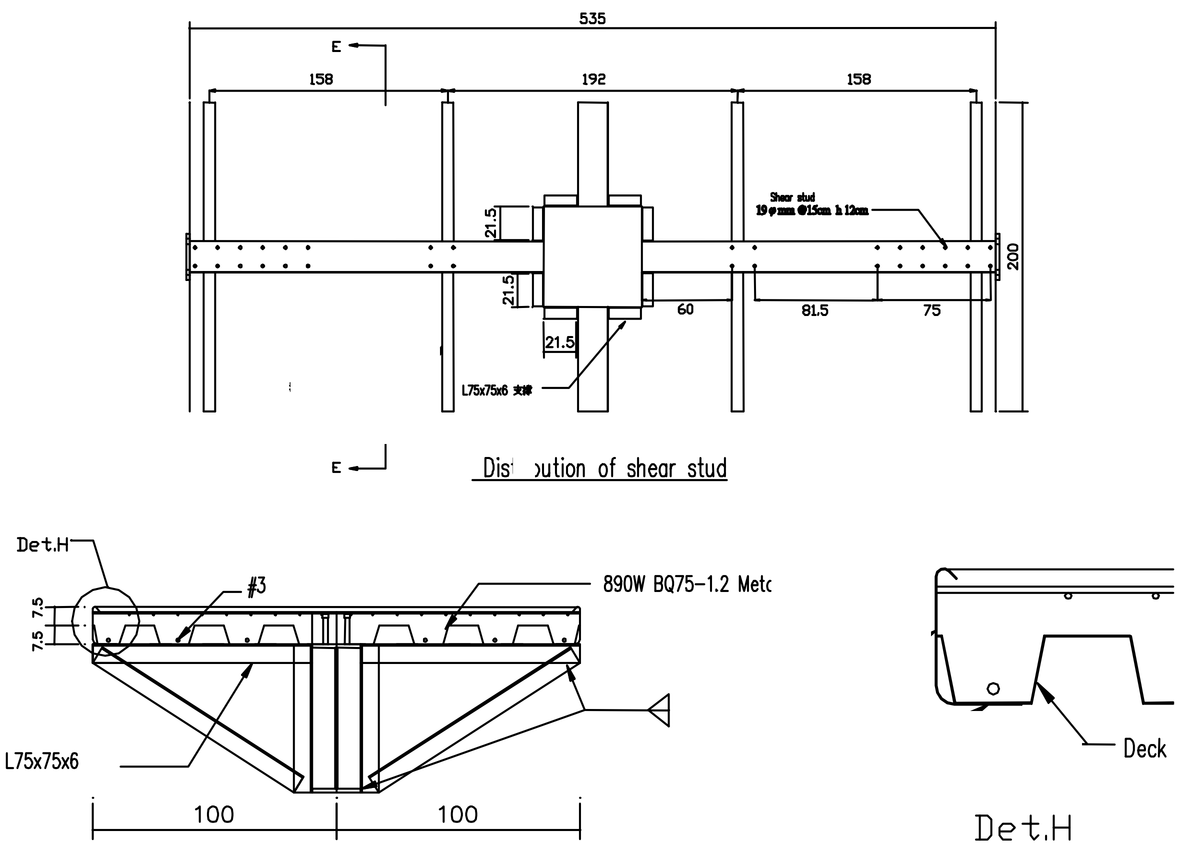


Figure 5 Details of composite beams

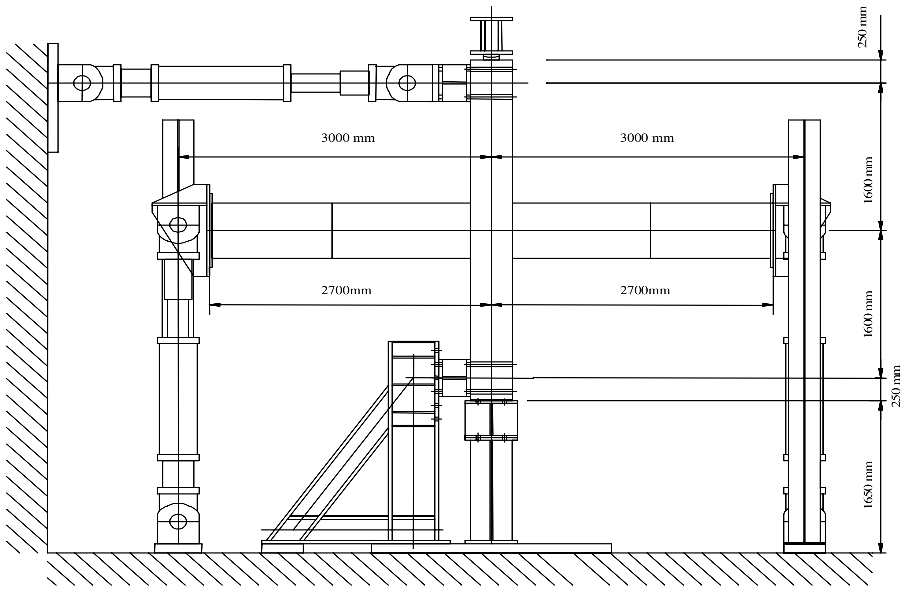


Figure 6 Test apparatus

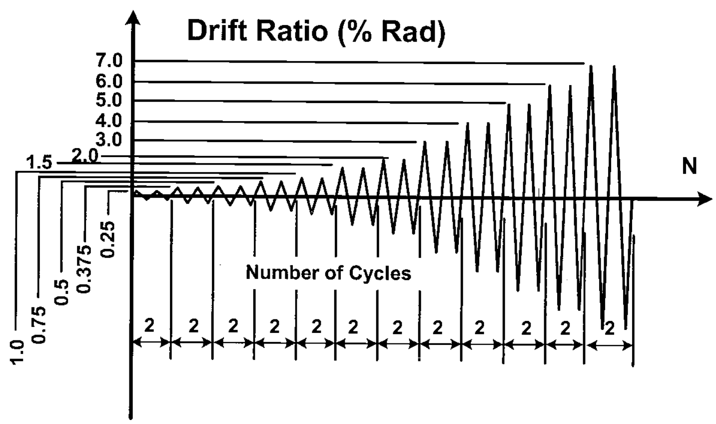


Figure 7 Loading protocol for specimens except ICLPS

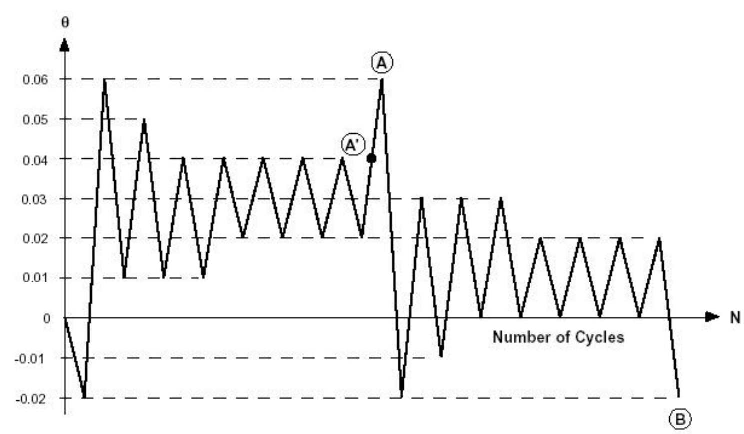


Figure 8 Loading protocol for specimen ICLPS

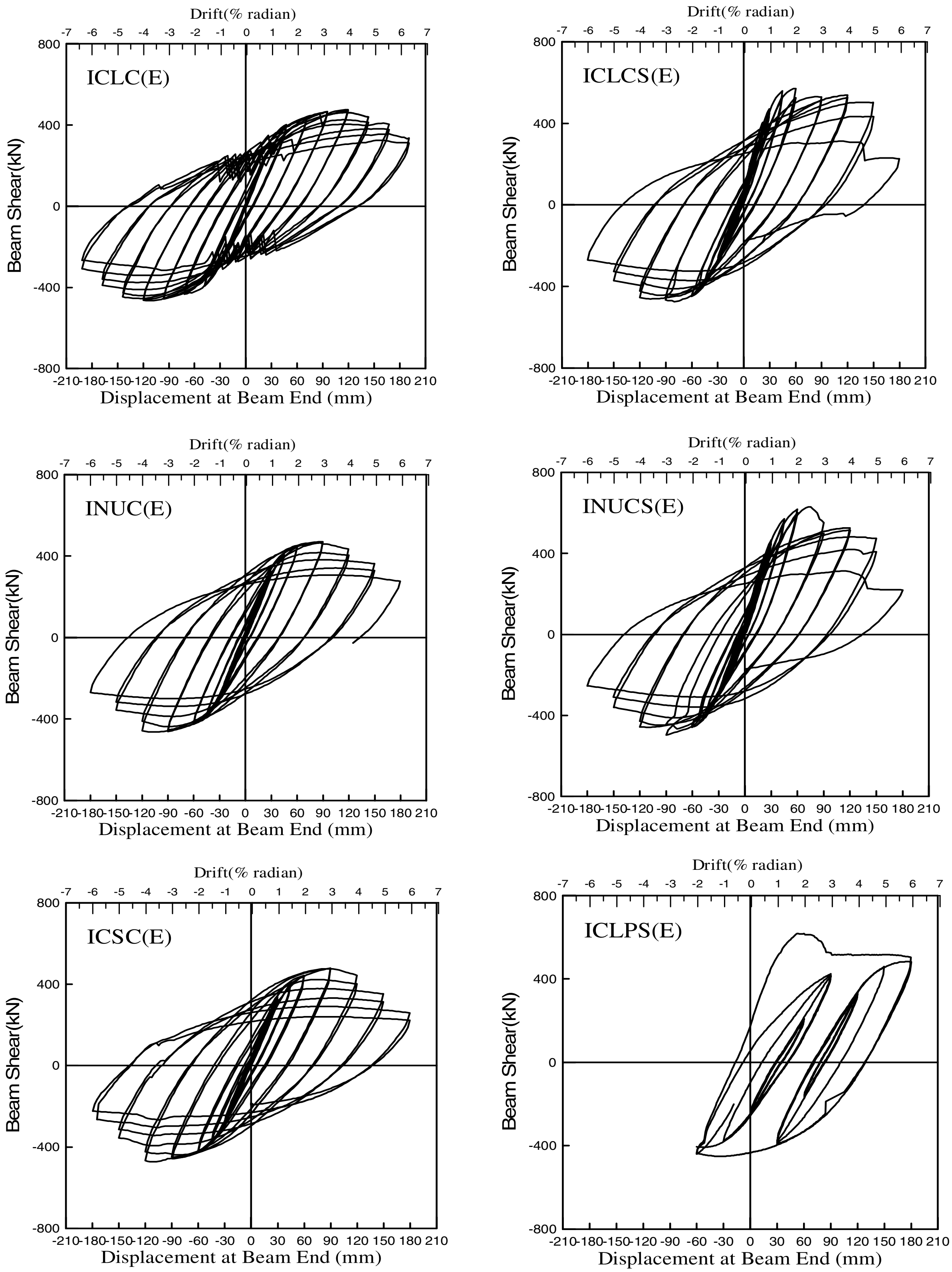


Figure 9 Hysteretic curves and analytical predictions.

FORCE DEFORMATION SIMULATIONS

Drain-2DX program was applied to simulate force-deformation behavior of RCS beam-column connections. Based on the measurement installed around the panel zone, deformations of connection consist of flexural deflection due to column and beam, in addition to the panel zone deformation. As shown in Fig. 1, panel zone deformations consist of distortions due to bearing of column concrete and panel shear. Based on research of Para-Montesinos [7], panel shear can be resisted by the superposition of three components such as concrete strut in inner and outer elements, and steel beam web. Compatibility in this disturbed region shows that

$$\varepsilon_c = \frac{\varepsilon_x + \varepsilon_y}{2} + \frac{\varepsilon_x - \varepsilon_y}{2}\cos(2\theta_p) + \frac{\gamma_{xy}}{2}\sin(2\theta_p) \tag{1}$$

$$\varepsilon_t = \frac{\varepsilon_x + \varepsilon_y}{2} + \frac{\varepsilon_x - \varepsilon_y}{2}\cos\left[2\left(\theta_p + 90^\circ\right)\right] + \frac{\gamma_{xy}}{2}\sin\left[2\left(\theta_p + 90^\circ\right)\right] \tag{2}$$

where $\gamma_{xy} = \tan\left(2\theta_p\right)\left(\varepsilon_x - \varepsilon_y\right)$, ε_c and ε_t = principal compressive and tensile strain of concrete strut, ε_x and ε_y = horizontal and vertical strain of concrete strut, and θ_p is the main concrete strut angle for the inner and outer elements calculated based on geometry shown as follows.

$$\theta_{inner} = a\tan\left(\frac{d_{beam}}{h_c}\right) \tag{3}$$

$$\theta_{outer} = a\tan\left(\frac{1.25 d_{beam}}{h_c}\right) \tag{4}$$

where d_{beam} = depth of steel beams and h_c = width of columns. Empirical values of $k_{tc} = -\varepsilon_t / \varepsilon_c$ varied with different details such as tee or cruciform joints, inner or outer elements are provided in the paper of Para-Montesinos [7]. By using Equations (1) to (4) and k_{tc} value, shear strain γ_{xy} and ε_c can be obtained. For inner element, stress-strain relationship of strut concrete can be expressed as

$$f_c(\varepsilon_c) = f_c'\left[2(\varepsilon_c/\varepsilon_0) - (\varepsilon_c/\varepsilon_0)^2\right] \qquad \varepsilon_c \le \varepsilon_0 \tag{5}$$

$$f_c(\varepsilon_c) = f_c'\left[1 - Z(\varepsilon_c - \varepsilon_0)\right] \qquad \varepsilon_c > \varepsilon_0 \tag{6}$$

where f_c' =concrete compressive strength (MPa), ε_c =concrete strain corresponding to f_c, ε_0 =concrete strain corresponding to f_c' and Z =parameter to define the post-peak slope of descending strength, for inner element Z=50 and

$$\varepsilon_0 = 0.001648 + 0.0000165 f_c' \tag{7}$$

Therefore, effective strut concrete stress is expressed as

$$\left(f_c\right)_{eff} = f_c(\varepsilon_c) k_c \beta \tag{8}$$

where k_c considers the confine effect that is applied to the concrete. In general, k_c is equal to 2.0. If a cross-beam intersected in the panel zone, k_c is equal to 2.3. Where β considers the soften effect of concrete strength due to the orthogonal tension in panel zone. Based on the research of Vecchio [13],

$$\beta = \frac{1}{0.85 - 0.27\dfrac{\varepsilon_t}{\varepsilon_c}} = \frac{1}{0.85 + 0.27 k_{tc}} \tag{9}$$

Then, the shear strength resisted by inner element is

$$V_{ih} = 0.3 (f_c)_{eff} h_c (b_f - t_w) \tag{10}$$

where h_c =column width, b_f =width of beam flange and t_w =width of beam web.

The calculation of shear strength resisted by outer element is the same as inner element except the adjustments of parameters Z , β and k_c . When stirrups are applied in the panel zone k_c is equal to 1.1. In addition to stirrups, application of steel band plates increases k_c to 1.5. If steel band plates are applied alone without stirrups, k_c is adjusted to be 1.3. Under the above circumstantial condition, Z is equal to 150. If steel plates are applied to cover the panel zone, k_c is equal to 2.0 for inner and outer element, and Z is equal to 50. Therefore, shear strength resisted by outer element is calculated as

$$V_{oh} = 0.3 (f_c)_{eff} h_c b_0 \tag{11}$$

where b_0 =effective width of outer element defined in the research of Para-Montesinos [7]. Shear strength resisted by beam web can be calculated as

$$V_{wh} = \int_0^{h_c} \tau_{web}(x) dx t_w \tag{12}$$

where $\tau_{web}(x) = \gamma_{web}(x) G_s \le \dfrac{f_y}{\sqrt{3}} = \tau_y$. Then, the shear strength of panel zone is the superposition of three components as

$$V_{pzs} = V_{ih} + V_{oh} + V_{wh} \tag{13}$$

Figure 10 shows the predictions of shear force-strain curve by this model together with experiments for the specific specimens. The prediction was calculated based on 42 MPa of concrete strength and 420 MPa yield strength for steel. It was found that the predictions might have higher stiffness than the tests. However, above analytical procedure based on the research of Para-Montesinos [7] did not account for the distortions of band plates that were applied to prevent the bearing failure of column concrete. Therefore, the stiffness of band plates can be estimated by

$$k_{bp} = 3EI_{bp} \big/ L_{bp}{}^3 \tag{14}$$

where I_{bp} is the flexural inertia of three plates and L_{bp} is the half-length of band plates as shown in the Figure 2. Figure 11 shows that the stiffness estimated by analytical results may be appropriate when compared with the experiments. Therefore, stiffness of panel zone k_{pz} due to concrete bearing and panel shear may be combined and expressed as

$$1/k_{pz} = 1/k_{bp} + 1/k_{pzs} \tag{15}$$

where k_{pzs} is the shear stiffness as shown in the Figure 10. Figure 12 shows the comparison of total panel stiffness between predictions and the tests. As shown in Figure12, proposed methods for calculating panel zone stiffness that account for distortions due to concrete bearing and panel shear is appropriate.

In the simulation of force deformation behavior of beam-column connections by using Derain-2DX, the above analytical predictions for shear force-strain relationship in panel zone can be applied instead of rigid joints. However, this shear force-strain relationship needs to be transformed into moment-rotation relationship of springs due to the loads applied at beam tip in tests as shown in Figure 13. The beam end total moments ΔM and panel shear V_{pz} can be related as

$$V_{pz} = \frac{\Delta M}{d_{pz}}\left(1 - \frac{d_{pz} \times L}{H(L - h_c)}\right) \tag{16}$$

where d_{pz} is depth of panel zone, L is the beam length between two inflection points (actuators), H is column height between two inflection points and h_c is column width. Beam end rotations due to panel shear θ_{pz} and shear strain in panel zone γ can be related as

$$\theta_{pz} = \gamma\left[1 - \frac{d_{pz} L}{H(L - h_c)}\right] \tag{17}$$

Based on Equations (16) and (17), panel shear force and strain relationship can be transformed into beam end moment and rotation relationship that were simulated by four springs in Drain-2DX program as shown in Figure 13. Figure 13 also shows a compression only link element to compensate the composite effect due to floor slab. Based on LRFD regulations, the stiffness of link element was calculated by the difference of bare steel beam without slab and composite steel beam with slab, and transformed into the direction of link element. For the column stiffness, effective moment inertia (70%) was used to account for flexural cracks. Figure 14 shows the predictions of force-deformation by Drain-2DX and compares with test results for the specimens ICLCS and ICLC. It is found that the force-deformation simulations agree very well with tests for both specimens. Table 3 summarizes the analytical shear strength in panel zone based on the research of Kanno [11], Para-Montesinos [7] and AIJ [14]. It was found that Kanno [11] suggestions is tends to be conservative for the design of connections, when compared with the tests.

Table 3 Strength and stiffness of specimens

Specimens	Experimental Results							Analytical Panel Shear (kN)		
	Moment in East Beam (kN-m)		Moment in West Beam (kN-m)		Stiffness K (kN/m)		Panel Shear (kN)	K & D	P & W	AIJ
	Positive	Negative	Positive	Negative	Positive	Negative				
ICLCS	1539	1283	1609	1236	17592	10643	3884	3440	5557	5671
INUCS	1701	1342	1652	1337	17571	12257	4134	3572	5760	5790
ICLPS	1669	1218	1661	1307	14999	10666	4012	3458	5746	5694
ICLC	1286	1256	1274	1229	10883	10463	3457	3527	5935	5749
INUC	1293	1276	1275	1229	12321	11515	3477	3570	5760	5546
ICSC	1272	1253	1244	1204	12418	10776	3408	3229	5083	5788

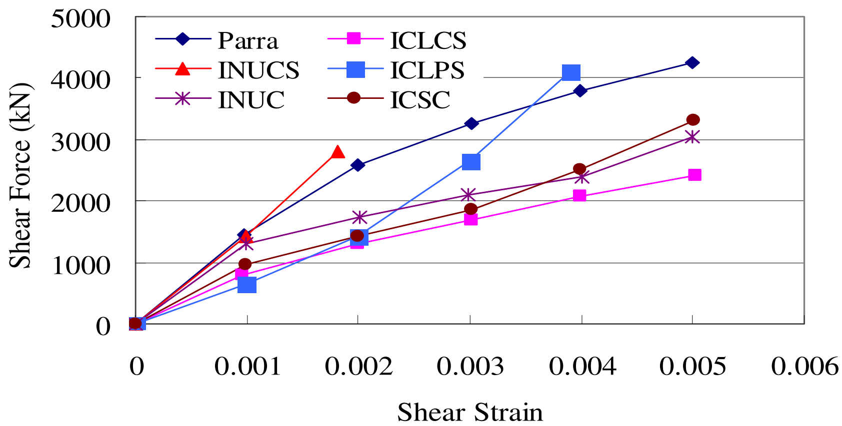


Figure 10 Shear force and strain relationship due to panel shear

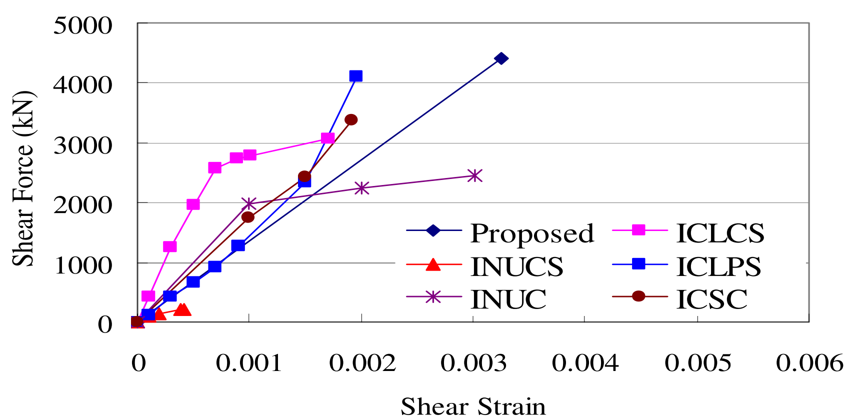


Figure 11 Shear force and strain relationship due to bearing

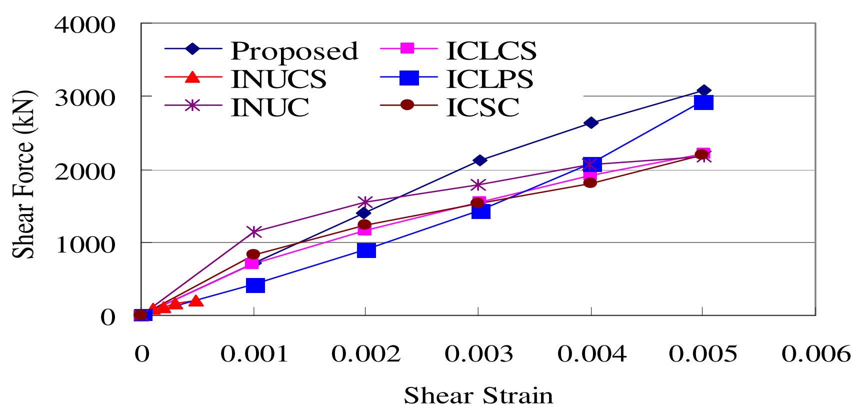


Figure 12 Proposed shear force and strain relationship due to bearing and panel shear

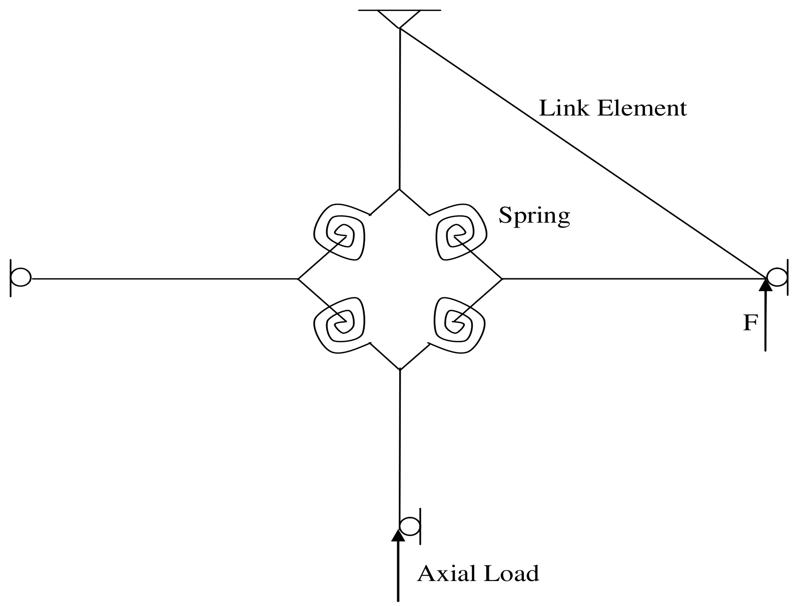


Figure 13 Schematic graph showing model of beam-column substructures

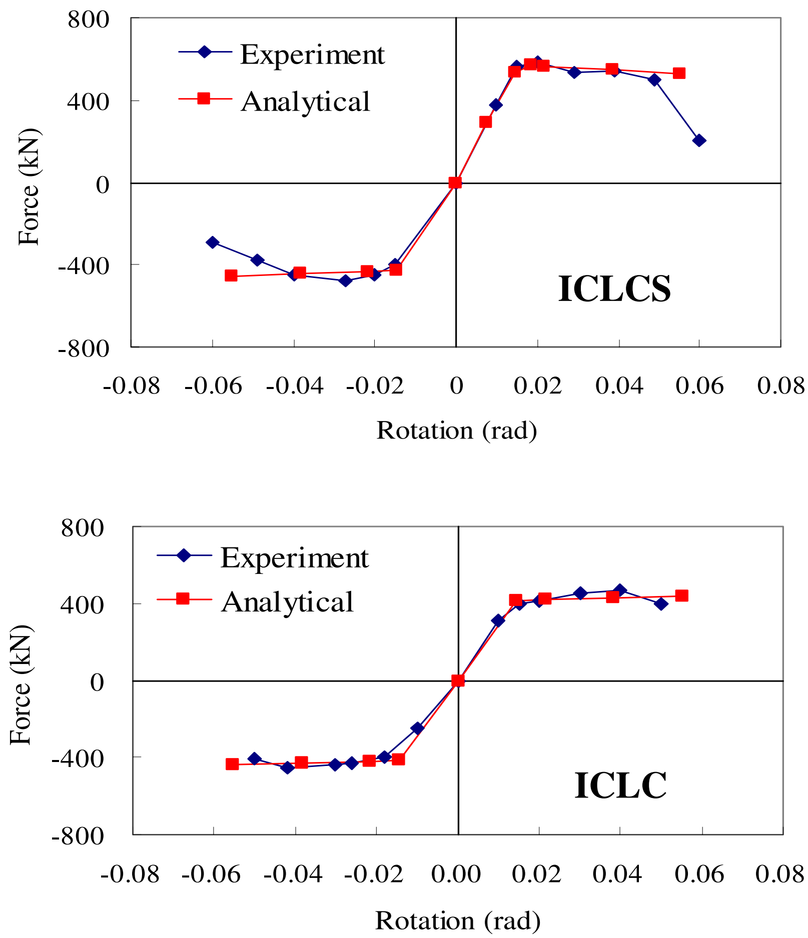


Figure 14 Predictions of force-deformation for specimens ICLCS and ICLC

CONCLUSIONS

Test results show that all specimens performed in a ductile manner with plastic hinge formed in the beam end. Under positive bending, it was found that initial stiffness and ultimate strength of composite beam averagely increased 67% and 27% respectively, compared to steel beam without slab. Under negative bending, similar ultimate strength of specimens with or without slab was obtained. This composite action disappeared after 3% drift of loading and then lateral strength slowly deteriorated until fracture of bottom flange. Substructure loaded by near-fault protocol performed well showing good strength and ductility slightly better than that of other tests where fracture of bottom flange and separation of beam and slab was visualized during test. Moreover, test performance revealed that cross beams and shape of stirrups in the panel zone have only marginal effect on the shear transfer in panel zone due to the strong column and weak beam design for all specimens.

Based on the comparison of force-deformation simulation and test results, it was found that distortions due to column concrete bearing in addition to distortions due to panel shear can appropriately predicted the total shear stiffness in the panel zone of RCS connections. Adding a compression only link element to simulate composite effect of slab, Drain-2DX program can simulate the envelop of force-deformation of composite RCS beam column substructures. To evaluate the shear strength in the panel zone, three models such as Kanno [11], Para-Montesinos [7] and AIJ [14] were applied. Test results showed that Kanno [11] suggestions are over conservative for the design of connections.

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