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SEISMIC RESISTANCE OF THROUGH COLUMN TYPE CONNECTIONS FOR COMPOSITE *RCS* SYSTEMS

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ABSTRACT

Seismic resistance of beam to column connections for composite structural systems composed of reinforced concrete columns and steel beams is investigated as a part of studies on "Composite and Hybrid Structures" of the US–Japan cooperative earthquake research program which have been started in 1993 fiscal year. The term "composite RCS systems" is used for such structural systems in this paper. An aimed type of the RCS connections in this study is "the through column type" in which a column is confined by steel plates and flanges of beams are replaced with transverse and/or horizontal stiffeners at the connection. A total of three RCS connections was tested, in which a column was confined by cover plates and flanges of beams were replaced with horizontal stiffeners at the connection. The test results are reported and effects of the thickness of cover plates and the existence of extended face bearing plates on seismic performance of the connections are described in this paper. Stress transferring mechanisms in the connections are also discussed on the basis of the test results.

KEYWORDS

Composite RCS systems; beam–column connection; through column type; structural test; seismic resistance; shear resistant mechanism

INTRODUCTION

In recent years, composite RCS systems for which a building is composed of reinforced concrete columns and steel beams are widely noticed in the rationalization of the structural design and construction of buildings in both Japan and the US. In particular, the numerous large construction companies in Japan are well equipped and funded for research in RCS structural components and systems. As a results, numerously clever connection details have been developed to make the composite RCS systems practical (for example, AIJ, 1994). These systems are commonly designed according to structural standards published by the Architectural Institute of Japan (AIJ, 1987) and the Building Center of Japan (BCJ, 1994). However, many of the new connection details and systems developed are not covered by the existing standards. The establishment of original design procedures for RCS connections and systems is an urgent necessity in Japan.

A study on the composite RCS systems has been included into studies on "Composite and Hybrid Structures" of the US-Japan cooperative earthquake research program which have been started in 1993 fiscal year (Yamanouchi et al., 1994). In the Japanese RCS sub committee of the research project, researches on beam-column connections and systems were planed to develop the original design procedures for composite RCS systems. For the research on RCS beam-column connections, prototype details were selected to grasp primary stress transferring mechanisms and make design formulas of connections, in which the rationality and applicability for many of the existing connection details would be provided. The selected prototypes were ***the through beam type*** in which steel beams penetrated a reinforced concrete column at the connection and ***the through column type*** in which a column was confined by steel plates and flanges of beams were replaced with transverse and/or horizontal stiffeners at the connection.

This study forms a link in the chain of the research on RCS beam-column connections in the Japanese RCS sub committee. Its object is to clarify the effect of reinforcing details on the shear strength and shear resistant mechanisms for the through column type connections and to provide basic data for constituting a general shear design procedure for such connections. A total of three RCS beam-column connections was tested, in which a column was confined by cover plates and flanges of beams were replaced with horizontal stiffeners at the connection. The test results are reported and the stress transferring mechanisms in the connections are discussed in this paper.

EXPERIMENTAL PROGRAM AND SPECIMENS

A total of three specimens of about one-third scale was prepared, which simulated interior column to beam connections for a RCS frame, as listed in Table 1. All specimens were designed as the joint shear failure type. The dimensions and details of the specimens are shown in Fig. 1. The mechanical properties of material used are listed in Table 2. All specimens had reinforced concrete columns with a 300 mm square section and a 510 mm shear span length and steel beams with a section of 250×100×9×16 mm and a 1,350 mm shear span length, which was SM490 grade built-up. The longitudinal reinforcement in each column consisted of twelve 16 mm diameter SD685 grade deformed bars arranged symmetrically around the perimeter to give a total reinforcement ratio, p_g=2.56%. All columns also had a square welded hoop and two square welded sub-ties per set, of which the reinforcement ratio, p_w was 0.7%. Although the depth of beams was 250 mm, the depth of a connection may be defined as 330 mm due to the existence of band plates and/or extended face bearing plates (FBP) in both the top and bottom of beam flanges, of which the depth was 40 mm.

Table 1 Test Program

Specimen			BRI1	BRI2	BRI3
Connection	Web		PL6 (SS400)		
	Cover Plate		PL3.2 (SS400)	PL6 (SS400)	PL3.2 (SS400)
	Band plate		Height of 40 mm		
	Horizontal Stiffener		PL16 (SM490)		
	FBP	Thickness	PL9 (SM490)		PL3.2 (SS400)
		Width	100 mm		
	Transverse Beam		Web Only		
Beam	Cross Section		BH-250×100×9×16 (SM490)		
Column	Cross Section		300×300 mm		
	Longitudinal Bars		12-D16 (p_g=2.65%; SD685)		
	Hoop		4-D6 @60 (p_w=0.7%; SD785)		
Applied Axial Load			459 kN (=0.2·b·D·σ_B)		

Table 2 Properties of Material Used

Steel		Measured (N/mm^2)	
Item	Size (mm)	Yield	Maximum
Plate	3.2	332	409
	6.0	379	475
	9.0	391	549
	16.0	346	531
Deformed bar	D16	732	940
	D6	993	-----
Concrete		25.5 (Comp.)	

The variables investigated were the thickness of cover plates and the existence of FBP. The thicknesses of cover plates are 3.2 mm for Specimens BRI1 and BRI3 and 6.0 mm for Specimen BRI2, respectively. Specimens BRI1 and BRI2 also had extended FBP of 9.0 mm thickness. Therefore, effects of the thickness of cover plates on seismic resistance of the connections can be examined by comparing the behavior of Specimens BRI1 with BRI2. Effects of the existence of FBP can be also found by making a comparison between Specimens BRI1 and BRI3.

The specimen was supported by pins in both the top and bottom of columns and the left and right ends of beams to simulate a interior column to beam connection in a frame, in which the beams and columns had inflection points in the mid span and height, respectively, as shown in Fig. 2. The specimen was loaded by a manual jack which was horizontally held at the top of the column and applies cyclic forces to the column while a constant compression load was applied by an actuator, the magnitude of which was 459 kN ($=0.2 \cdot b \cdot D \cdot \sigma_B$; where b, D and σ_B are the width, depth and concrete compressive strength of columns, respectively). The left and right reaction stringers absorbed the shear forces in the beams caused by the load applied at the top of the column. The lateral loading was reversed in two cycles each at the story drift angle, R of 0.005, 0.01, 0.015, 0.02, 0.025, 0.03 and 0.04 rad., which corresponded to δ/L, where δ and L were the relatively vertical displacement and the distance between both ends of the beams, respectively.

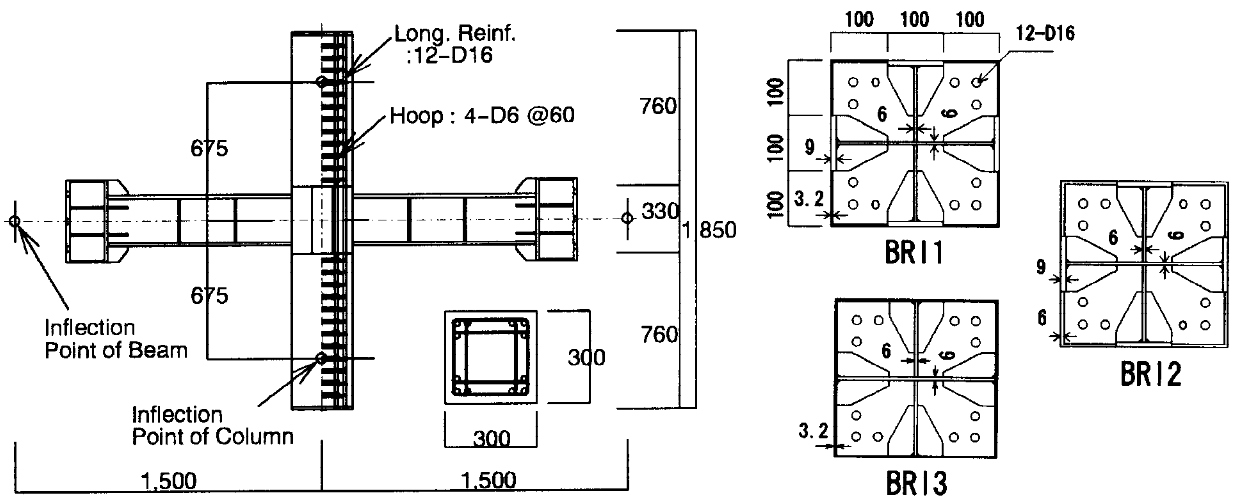

Fig. 1 Details of Test Specimens

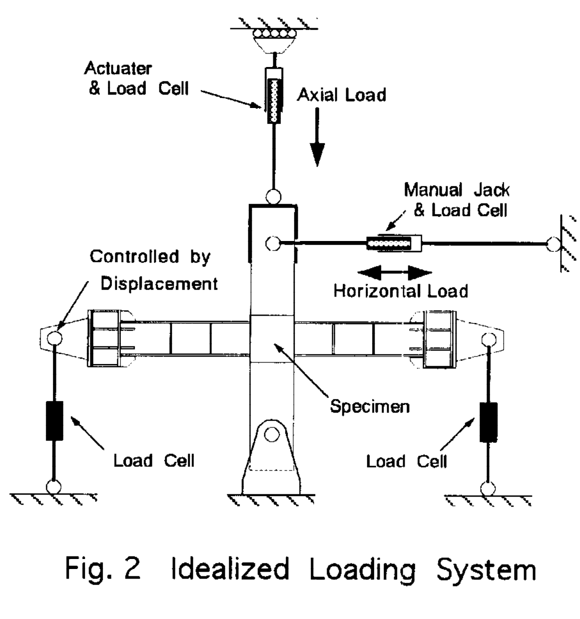

Fig. 2 Idealized Loading System

TEST RESULTS

Load versus Drift Angle Relations

Measured maximum strengths of each specimen are listed in Table 3 with calculated ultimate strengths of columns and beams expressed by converting into story shear forces. The ultimate strengths of columns was calculated by using the BCJ formula for flexure (BCJ, 1994) and the author's formula proposed for shear (Kuramoto and Minami, 1990). Figure 3 shows the load–drift angle relations for all specimens. Yielding in beam flanges was not observed even in the final stage of loading and joint yielding occurred in all specimens. A little slip–shaped but stable hysteresis loops in load–drift angle relations were observed in the specimens regardless of the thickness of cover plates or the existence of FBP. All specimens had the maximum capacity at the relative story drift angle , R of 0.02 rad. Deterioration in load carrying capacity with an increase of deflection amplitude after attaining to the maximum capacity was not observed up to R=0.04 rad. in the specimens. The average value of the maximum capacity in positive and negative loadings of Specimen BRI2 with cover plates of 6 mm thickness was about 10 kN larger than that of Specimen BRI1 with cover plates of 3.2 mm thickness. The average value of the maximum capacity of Specimen BRI1 with FBP was about 15 kN larger than that of Specimen BRI3 without FBP. It was also observed that not only the maximum capacity but also the capacity of energy consumption in the connection was increased by thickening cover plates and/or fitting FBP.

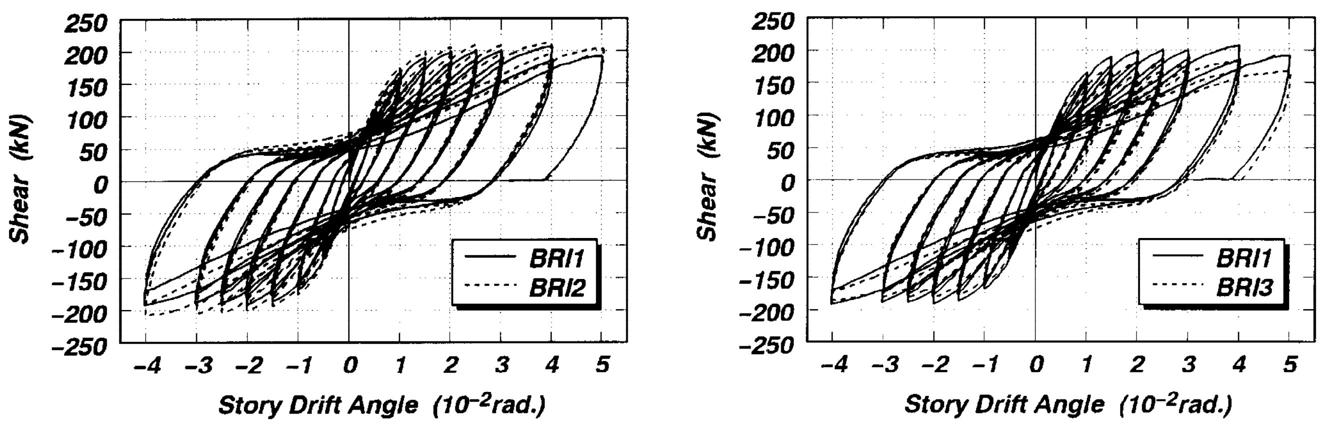

Fig. 3 Load–Drift Angle Relations

Table 3 Measured and Calculated Strengths

Specimen	Measured Maximum Load (kN)		Calculated Ultimate Strength (kN)		
			Column		Beam
	Pos.	Neg.	Flexure	Shear	
BRI1	199	−184			
BRI2	204	−200	405	394	235
BRI3	176	−178			

Behavior of Joint Elements

(a) Web Panel Shear stress distributions in a web panel resisting to the loading direction for each specimen are shown in Fig. 4. The distributions at the peak in negative loading of the cycles of 0.005, 0.01 and 0.015 rad. are plotted in the figure, in which the shear stresses have been obtained by converting measured strains of tri–axial gauges stuck in the panel with applying the Mises' yield criterion. In all specimens, the web panel yielded in shear at all points measured up to the loading cycle of 0.015 rad.

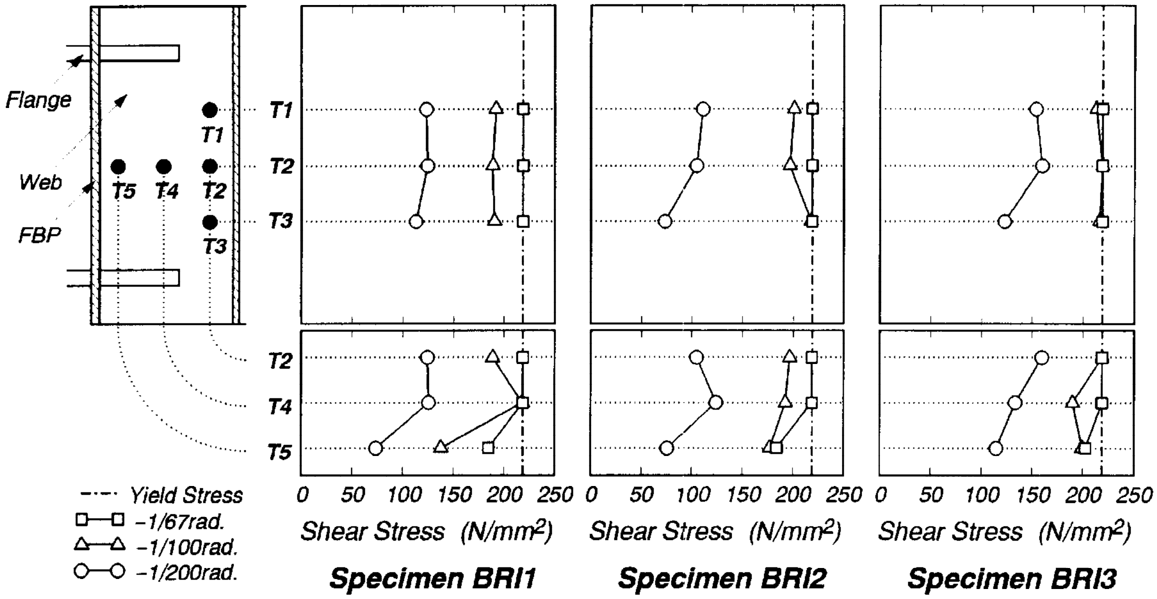

Fig. 4 Shear Stress Distribution in Web Panel

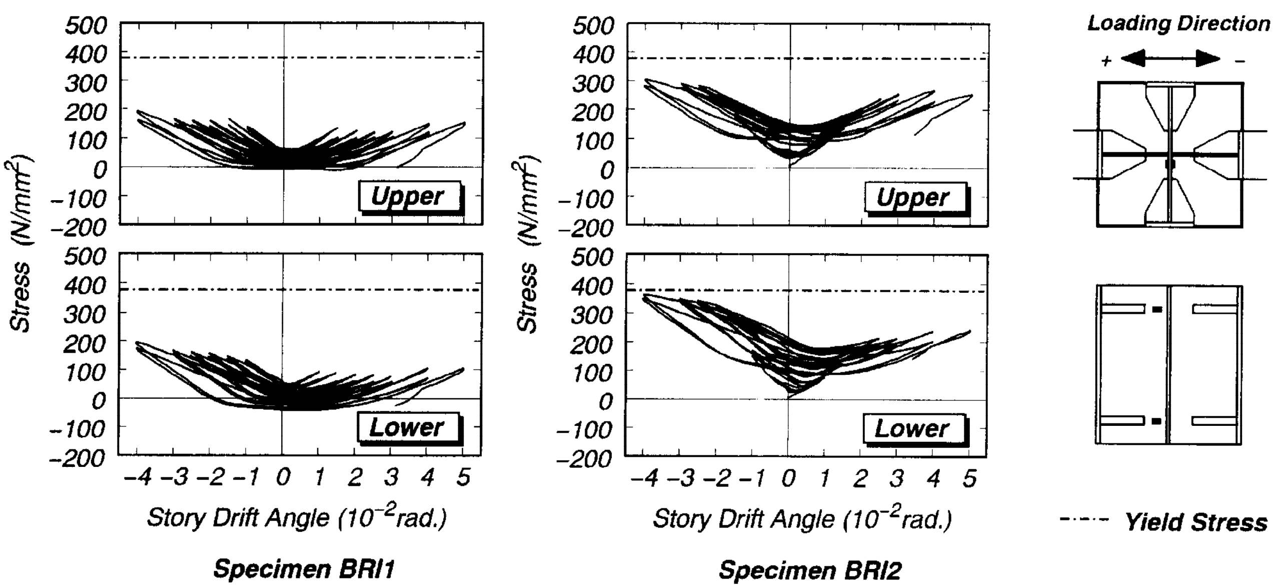

Fig. 5 Transition of Normal Stress in Transverse Web Panel with Increasing Story Drift Angle

Figure 5 shows the transition of normal stresses with an increase of the story drift angle in a transverse web panel for Specimens BRI1 and BRI2. The stresses have been obtained by converting measured strains of uni–axial gauges stuck horizontally in the panel. For both specimens, the transverse web panel did not yield and contributed almost the constant stress at each peak in loading cycles after the story drift angle, R of 0.015 rad., of which the values were about 200 and 300 N/mm^2 for Specimens BRI1 and BRI2, respectively. From these transition of stresses in the panel, it is obvious that the transverse web panel in connections of the cover plate type play an important role in resisting external forces.

(b) Face Bearing Plate (FBP) Relations between shear stresses in FBP fitted to transverse beams and story drift angles for Specimens BRI1 and BRI2 are shown in Fig. 6. The shear stresses have been obtained by converting measured strains of tri–axial gauges stuck in the FBP with applying the Mises' yield criterion.

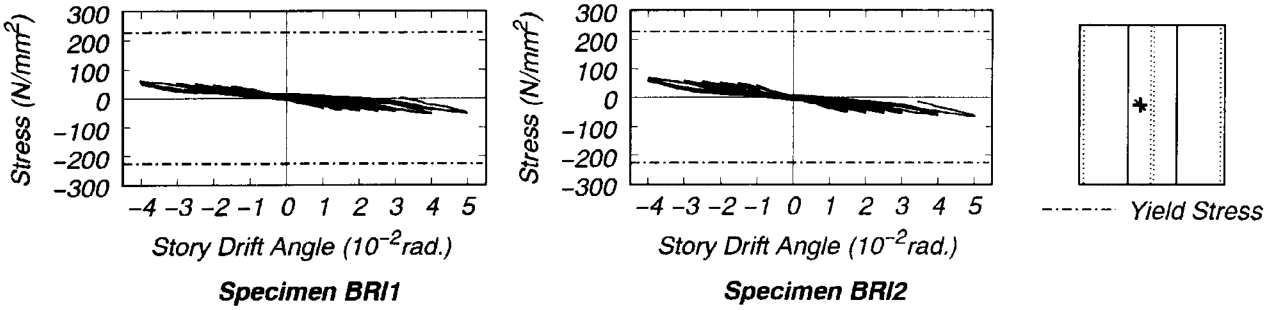

Fig. 6 Relations between Shear Stress in FBP and Story Drift Angle

For each specimen, the stresses in FBP at the peak of each loading cycle after R=0.015 rad. were almost constant in both positive and negative loadings. A little increase of the stresses with an increase of the thickness of cover plates was observed.

(c) Cover Plate Relations between normal stresses in cover plates and story drift angles for Specimen BRI2 are shown in Fig. 7. The stresses plotted are at the levels of beam flanges in cover plates resisting to the loading direction, as shown in the figure. Specimen BRI2 showed tensile stresses of about 70 N/mm^2 at the peaks in both positive and negative loading of each cycle after R=0.015 rad. In Specimens BRI1 and BRI3, the tensile stresses were about 100 and 120 N/mm^2, respectively. From these results, it was imagined that the cover plates resisted to the tension of beam flanges and the compression of a diagonal concrete strut formed in the connection.

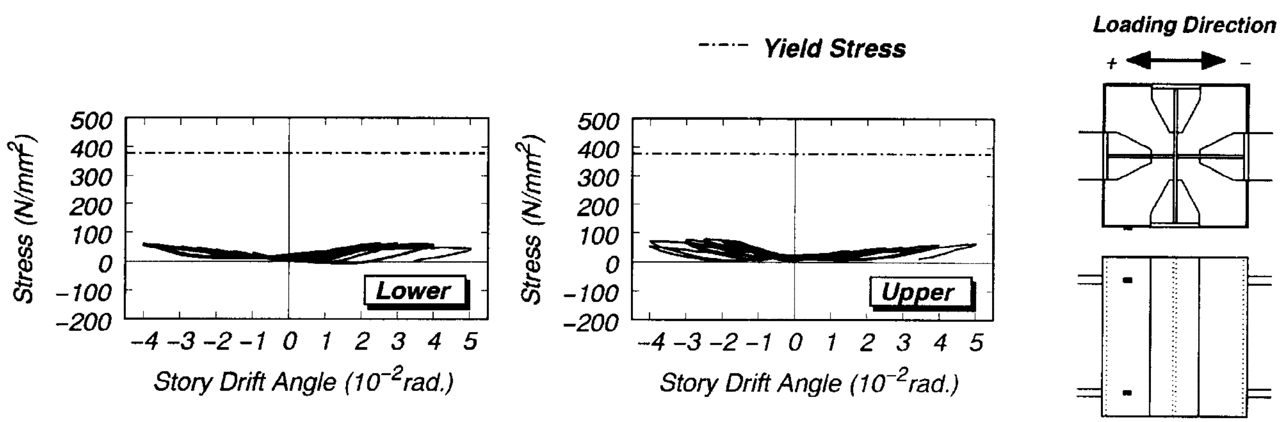

Fig. 7 Transition of Normal Stress in Cover Plates with Increasing Story Drift Angle for Specimen BRI2

CONSIDERATION ON STRESS TRANSFERRING MECHANISMS IN CONNECTIONS

Macroscopic models for stress transferring mechanisms in connections are designated based on the experimental results reported in the previous chapters. The models presented here are applicable to RCS beam to column connections failing in joint shear such as the connections tested in this study.

Connections of the cover plate type are assumed to consist of three kinds of stress transferring elements, a web plate, concrete sub struts and a concrete strut, as shown in Fig. 8. It can be assumed that the strength of the web plate element at the maximum capacity of the connection is given by shear yielding in the web panel as $A_w \cdot {}_s\sigma_{wy} / \sqrt{3}$ because yielding in the overall depth of the web panel have been observed for the cover plate type as shown in Fig. 4, where A_w and ${}_s\sigma_{wy}$ are the sectional area and yield stress of a web plate, respectively.

The concrete sub struts are assumed to be formed between upper and lower horizontal stiffeners fitted to a transverse web plate each other. The shear contributed by the sub struts at the maximum capacity can be estimated by using stresses in FBP as $A_{fb} \cdot \tau_{fb}$, where A_{fb} and τ_{fb} are the sectional area and shear stress of FBP, respectively. The measured stresses at the maximum capacity of the specimens were used as the shear stresses of FBP in examinations on the shear transferring rate of each resistant mechanisms mentioned later. Then the concrete strut is assumed to be formed on a diagonal line in the joint core through compression transferred from both beams and columns, bond forces in reinforcing bars and tensions in cover plates as shown in Fig. 8(c). In this case, the shear contributed by the strut can be obtained by subtracting the shears contributed by the web panel and concrete sub strut from the maximum shear of the connections.

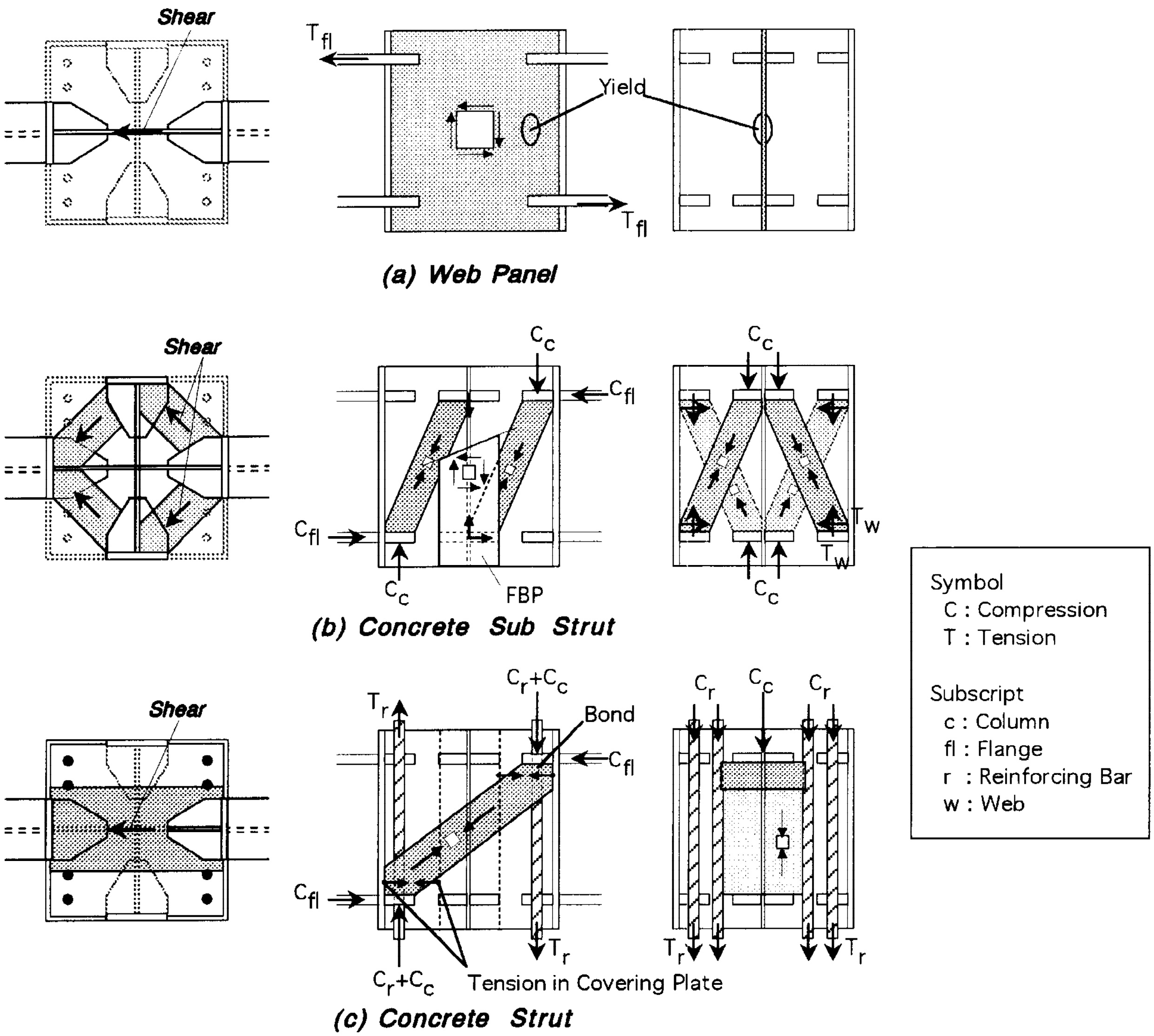

Fig. 8 Stress Transferring Mechanisms in Connections of the Cover Plate Type

Figure 9 shows the shear transferring rate of each resistant mechanisms for each specimen. The shear in connections is drawn by converting into the story shear in this figure. For all specimens, it is designated that thickening cover plates and fitting face bearing plates are effective to an increase of the shear strength of the connection due to an increase of shear contributed by not only the concrete strut mechanism but also the concrete sub struts mechanism.

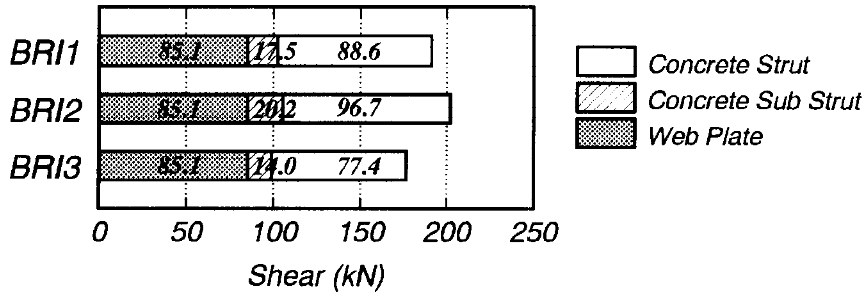

Fig. 9 Shear Contributed by Each Resistant Mechanism

CONCLUSIONS

The shear strength and seismic performance of composite RCS beam to column connections of the through column type were discussed in this paper. From experimental and analytical results presented above, the following conclusions can be drawn:

(1) Thickening cover plates and fitting face bearing plates are effective for an increase of the shear strength and improvement of the seismic performance of the connections.

(2) The web plate mechanism, the concrete sub struts mechanism and the concrete strut mechanism can be assumed as shear transferring mechanisms for connections with cover plates.

(3) The shear strength of the connections can be increased by thickening cover plates and/or fitting face bearing plates due to an increase of shear contributed by not only the concrete strut mechanism but also the concrete sub struts mechanism.

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