Hybrid structural steel beam – reinforced concrete column connection

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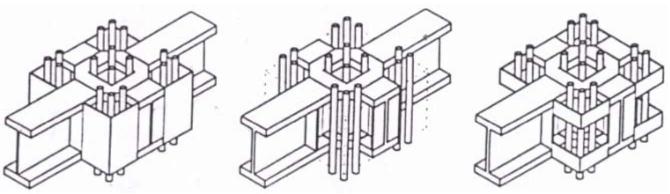
ABSTRACT: Nine specimens were tested to evaluate the behavior of a hybrid steel beam – reinforced concrete column connection under axial and antisymmetrical cyclic loads. The connection is made of welded plates forming an encasement that confines twelve reinforcing bars at the corners and four bars at the center core. Experimental and theoretical results on loads, displacements, stresses and strains are presented to describe the general structural performance of the connection.

1 INTRODUCTION

Newly developed hybrid frames made of reinforced concrete (RC) columns connected to steel beams have been used in Japan. These frames can provide practical and economical merits by combining longer steel beams with high compression resistant RC columns. To further develop such hybrid structures, it is necessary to determine the strength and ductility of the connection. This study focuses on beam-column connections, as shown in Figure 1,



JP1,2; JS1,2,5,6,7 JS3 JS4

Figure 1. Beam-column connections.

that consist of welded steel plates forming an encasement confining the concrete and the reinforcing bars. The connections are designed to effectively resist varying column axial loads especially in tension. Openings in the encasement are made to provide the necessary arrangements for reinforcing bars. A square tube at the center of the encasement serves as the inner core confinement and the outer plates act as outer confining reinforcements. The specimens are modeled after some beam-column connections of a nine-story building that can be subjected to antisymmetrical bending moments and varying axial loads.

1.1 *Objective*

The main objective of this research is to investigate the general structural performance of the hybrid connection and its design method.

2 EXPERIMENTAL INVESTIGATION

2.1 *Specimens*

The specimens are about 1/3 the actual size of the designed structural members at the second floor of a nine-story building. Table 1 shows the properties of the specimens. The column cross section is 320 mm

Table 1. Properties of specimens.

Specimen	JP1	JP2	JS1	JS2	
BxD(mm)	320x320				column
main bars	12-D13		12-D16		
core bars	4-D13		4-D16		
pg(%)	1.98		3.11		
stirrups	4-U5.1@50		4-U6.4@50		
pw(%)	0.50		0.75		
DxBxt1xt2	H-200x100x5.5x8		BH-200x100x6x25		beam
inner tube	□ 120x120x6		□ 125x125x6		connx
outer plates	3.2		2.3	4.5	
Specimen	JS3	JS4	JS5	JS6	JS7
BxD(mm)	320x320				
main bars	12-D16				
core bars	4-D16				
pg(%)	3.11				
stirrups	4-U6.4@50				
pw(%)	0.75				
DxBxt1xt2	BH-200x100x12x25				
inner tube	□ 125x125x9			□ 125x125x4.5	
outer plates	0.0	2.3	4.5	2.3	4.5

x 320 mm. Beam size is 200 mm x 100 mm. The column/beam depth proportion Dc/Db is 1.6 and the width proportion Bc/Bb is 3.2.

2.2 *Material Properties*

Table 2 shows the material test results. In flexural type specimens JP1 and JP2, D13(SD345) (diameter=13mm; yield strength=345Mpa) reinforcing bars and high strength ties U5.1(SBPD1275/1420) were used for RC columns. The concrete strength was designed to be 360kgf/cm^2. H200x100x5.5x8 (SS400) was selected for the beams, In shear type specimens JS1-JS7, D16(SD390) steel bars and high strength stirrups U4.6(SBPD1275/1420) were utilized for columns. The design strength was 210kgf/cm^2 for concrete. Steel beams were BH200x100x6x25 (SM490) and BH200x100x12x25 (SM490).

Table 2. Material properties.

Steel Plates				
plate properties	yield kgf/cm^2	ultimate kgf/cm^2	elongation %	part
SS400 t=2.3 mm	3610	4440	34	outer plate
SS400 t=4.5 mm	2900	3800	42	
SM490 t=6 mm	4030	5720	28	beam web
SM490 t=12 mm	3780	5550	28	
SM490 t=25 mm	3500	5400	48	flange
SM490 t=28 mm	3650	5440	31	diaphragm
SM490 t=9 mm	3640	5320	28	steel tube
SN490 t=4.5 mm	4060	5620	34	
SN490 t=6 mm	4170	5650	32	
Steel Bars				
designation	yield kgf/cm^2	ultimate kgf/cm^2	elongation %	part
D13	3680	5510	26	main/core
D16	4820	6880	19	bars
U6.4	13900	14900	9	stirrups
Concrete				
specimen	comp. strength kgf/cm^2	E kgf/cm^2 x10^{-5}	tensile strength kgf/cm^2	part
JP1	433	2.49		column
JP2	456	2.50		
JS1	217	2.14	22.0	
JS2	234	2.35	20.6	
JS3	238	2.23	23.7	
JS4	237	2.47	23.3	
JS5	190	2.12	21.1	
JS6	201	2.37	21.5	
JS7	211	2.27	21.1	

2.2 *Loading Method and Sequence*

As illustrated in Figure 2, an oil jack was used to apply the axial load to the column, and actuators at beam-ends provided the antisymmetrical bending moments. The loading cycle and sequence, as shown in Figure 3, started from a drift angle R=±1/800 and ended at R=±1/25. The drift angle was doubled after every two loading cycles. At R=±1/25, the cycle was done only once.

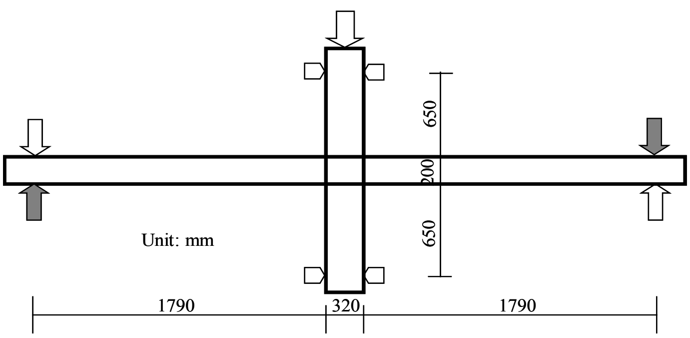


Figure 3. Loading method.

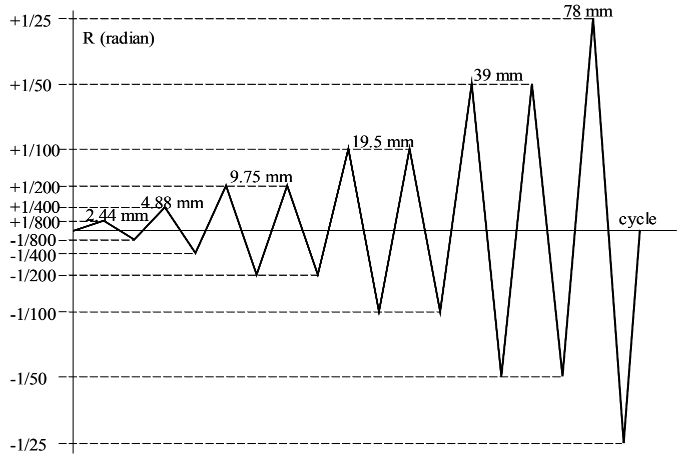


Figure 2. Loading cycle and sequence.

2.3 *Displacement and strain gauges*

Figure 4 shows the setup to measure relative displacements and curvatures in beams and column. Gauges to record shear deformations within the beam-column connection are shown in Figure 5.

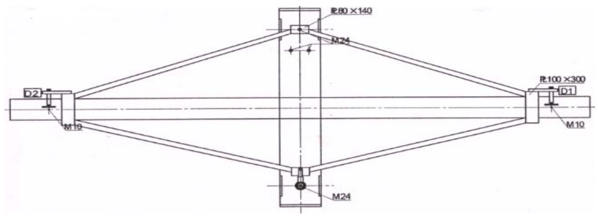


Figure 4. Setup of displacement gauges.

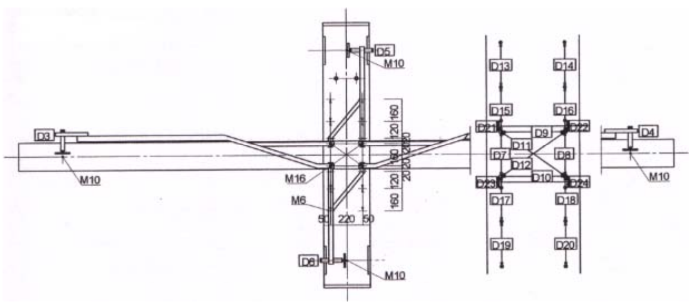


Figure 5. Shear deformation gauges.

Strain gauges placed at preferred locations to monitor strains on steel plates are shown in Figure 6.

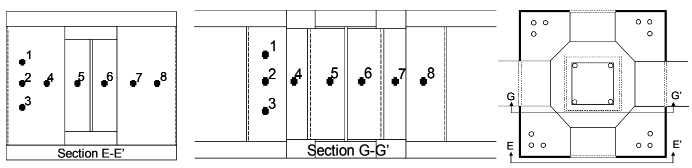


Figure 6. Strain gauges on steel plates.

3 TEST RESULTS

3.1 *Loads, displacements, cracking, and yielding*

Load-displacement diagrams are shown in Figure 7. In bending failure type specimens JP1 (cross shape) and JP2 (half cross shape), the maximum loads for

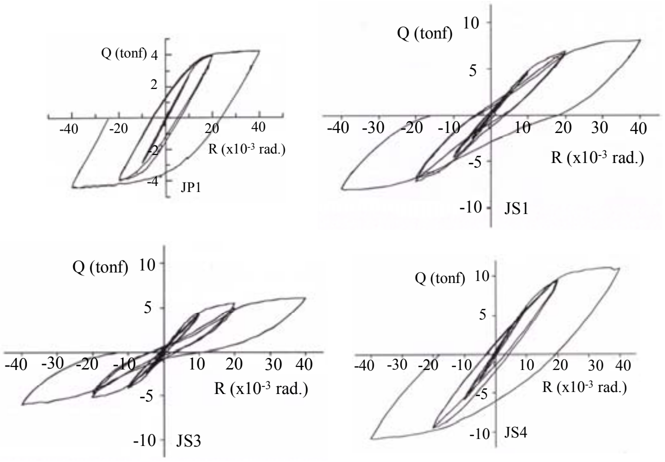


Figure 7. Load – displacement diagrams.

both specimens are 4.2 tonf during the positive cycle and 4.4 tonf during the negative cycle. More visible cracks occurred in the column of half-cross specimen JP2 than in JP1. Cracks in JP1 are very minimal and occurred only near the beam-column connection. There was no indication of yielding in the steel reinforcements of the column. However, the web of the beam was yielding. In a shear failure type specimen JS1 that has steel encasement around the beam-column connection, cracks due to bending occurred in the column during a drift R=1/100 cycle. During this cycle, the end of the steel beam and some parts inside the beam-column connection were yielding. Encasement plates yielded during the last cycle when R=1/25. Shear cracks on the column was seen during R=1/25 cycle. A maximum load of 8.1 tonf was reached during this cycle. The maximum load was still less than the ultimate flexural strength of the column. In specimen JS3, where outer plate encasement was not present, the flexural cracks in the column occurred when R=1/200. During this cycle, shear and flexural cracks were noticed at the beam-column connection. Yielding of steel plates inside the beam-column connection and at beam-ends occurred during R=1/100. A maximum load of 6.2 tonf was attained at R=1/25. The ultimate flexural strength of specimen JS3 was not reached. In specimen JS4, flexural cracks started to occur when R=1/200. The beam started to yield upon reaching R=1/100. Plates inside the beam-column connection began yielding during R=1/50. In this cycle, shear cracks appeared in the area of beam-column connection. A maximum load of 11.1 tonf was reached when R=+1/25. This maximum load was almost equal to the beam ultimate bending capacity.

3.2 *Strain distributions*

In flexural type specimens, the maximum strain in the square tubing was 900μfor cross shape specimen JP1 and 400μfor half cross shape specimen JP2. The outer plates of the beam-column connection indicated a maximum strain of 500μfor JP1 and 300μfor JP2. Figure 8 shows the strain values on the plates

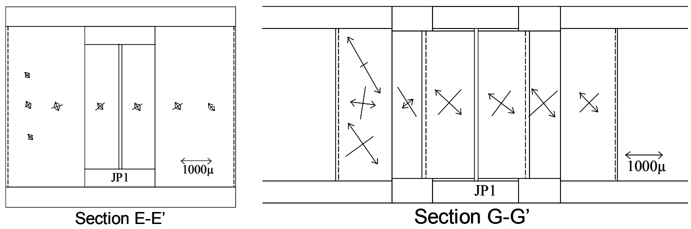


Figure 8. Typical strain indication in flexural type specimens.

inside the connection when R=1/25. Strain readings for shear type specimens JS1, JS3 and JS4 when R=1/25 are plotted in Figure 9. The maximum load

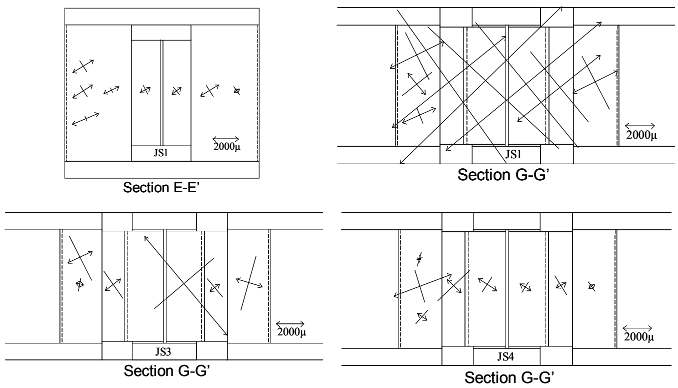


Figure 9. Typical strain indications in shear type specimens.

was thought to be governed by the beam-column connection. This was true for specimens JS1 and JS3 where the inner plates and square tubing indicated large strains. However, in JS4, the specimen that was thought to fail in bending at beam-ends, relatively small strains on the square tube were observed. Although strains on the outer plates of JS1 were smaller than those on the square tubing, yielding occurred on the outer plates.

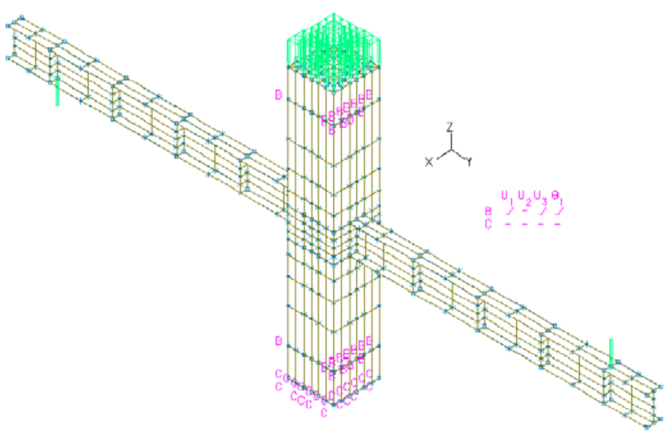


Figure 10. FEM Model with load and boundary conditions.

4 ANALYSIS

4.1 *Results of first analytical investigation*

A structural model was created using ADINA finite element analysis program to simulate the behavior of a flexural type specimen JP1. The model indicating the load and boundary conditions is shown in Fig. 10. Full fixity was assumed at the bottom end of the column. It was also assumed that there was no translation along y-axis at intermediate points near the top and bottom ends of the column where confining oil jacks were located. The model consisted of 2-node line elements for steel reinforcing bars, 4-node shell elements for steel plates, and 8-node solid elements for concrete for a total of 1340 nodes. Bilinear model with Von Mises yield condition was

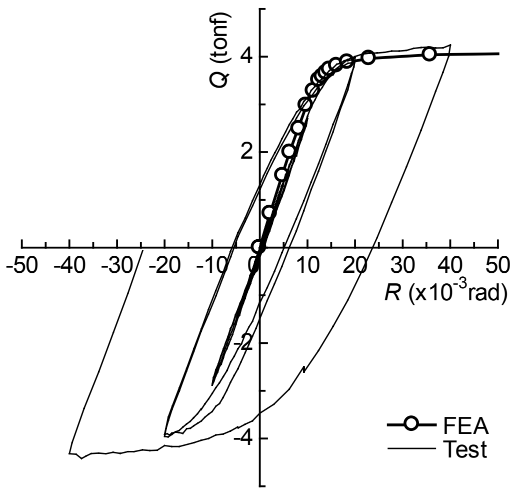


Figure 11. Analytical and experimental load-displacement relations for JP1.

used for steel elements. Figure 11 shows that the experimental and theoretical load-displacement relations for the beam of specimen JP1 are in good agreement. Both analytical and experimental investigations indicated that when R=1/25, the deflection of the beam is about 95% of the total deformation of the specimen as can be observed in Figure 12. During testing, cracks were observed on the surface of the column in the area of the beam-column connection but analytical results indicated that cracks

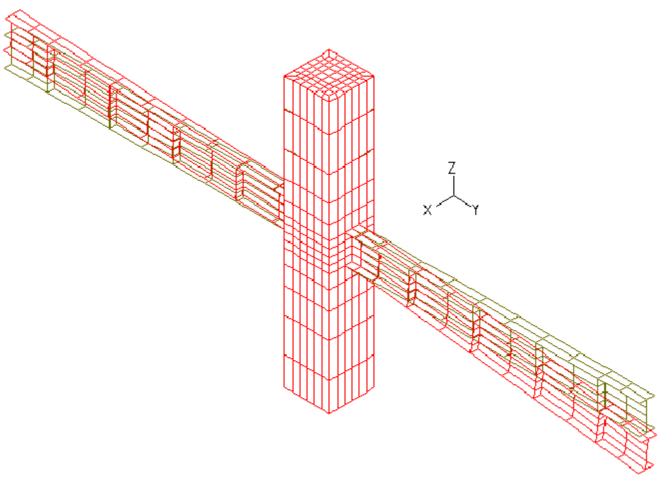


Figure 12. Analytical exaggerated deflection of JP1.

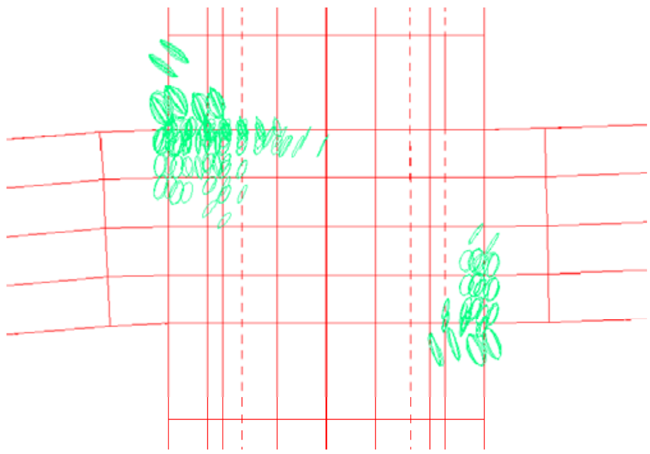


Figure 13. Theoretical crack formations in JP1.

formed beneath the surface as shown in Figure 13. Theoretical results for stresses on plates inside the connection were notably different from the experimental results. A response model for bond between concrete and steel plates was not provided. This may account for the difference between analytical and actual stress results in the inner part of the connection. Because of this, a second analytical investigation was done to reexamine the stress distributions on steel plates.

4.2 *Results of second analytical investigation*

Finite element models for beams and beam-column connections without concrete and reinforcing bars were created as shown in Fig. 14. Since it is the reinforced concrete column that supports and restrains the beam, the resistance of column was modeled as boundary pin supports along the outer and inner

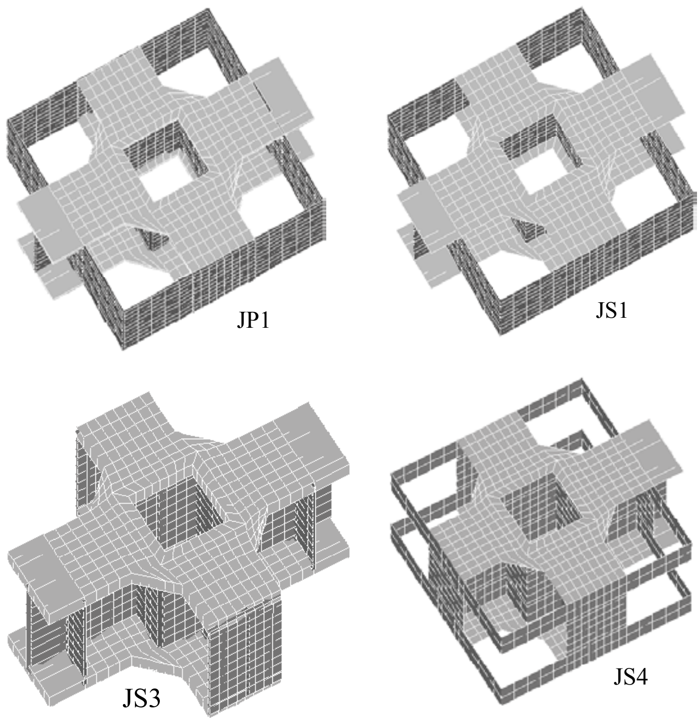


Figure 14. Typical models for beam-column connections.

square perimeters at the top and bottom of the connection. The main objective was to investigate the stress proportion and flow from the beam to the plates in the beam-column connection. Models that were identical in shapes and sizes to actual specimens were created using 4-node shell elements. Figure 15 and 16 show stress contours at maximum loads obtained from one-load-step calculations. It was observed that in all specimens, the stresses indicated by the strain gauges were closely approximated by analytical results. Slight differences in the stress results were found in the square tube and in the inner webs. The assumption for concrete as resisting boundary supports may account for the differences. However, with a good correlation in stress results, finite element models formed using exact sizes and shapes can be suggested to be the bases of actual structural design of this hybrid connection.

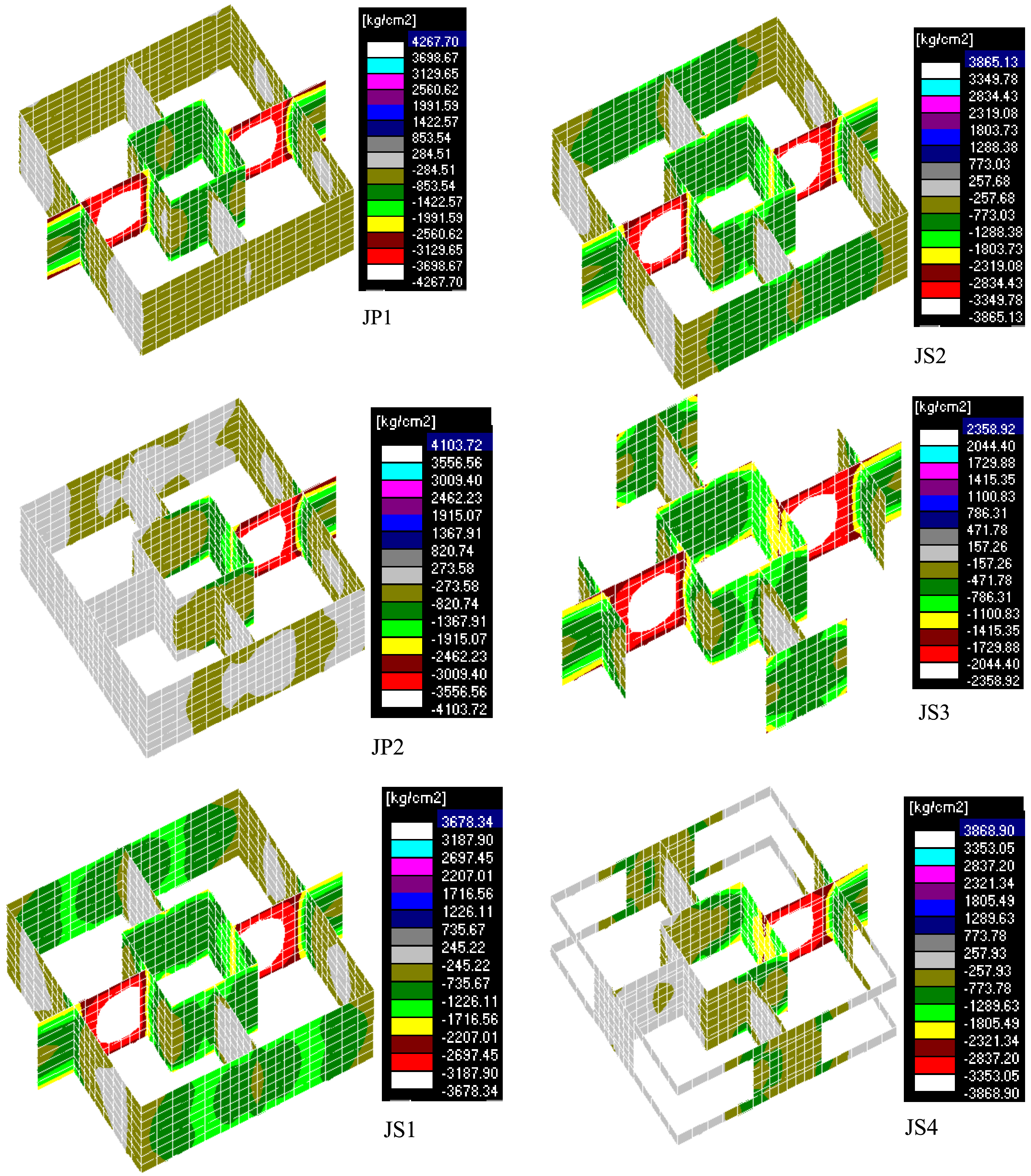


Figure 15. Stress distributions in the connections of specimens JP1, JP2, JS1, JS2, JS3, and JS4.

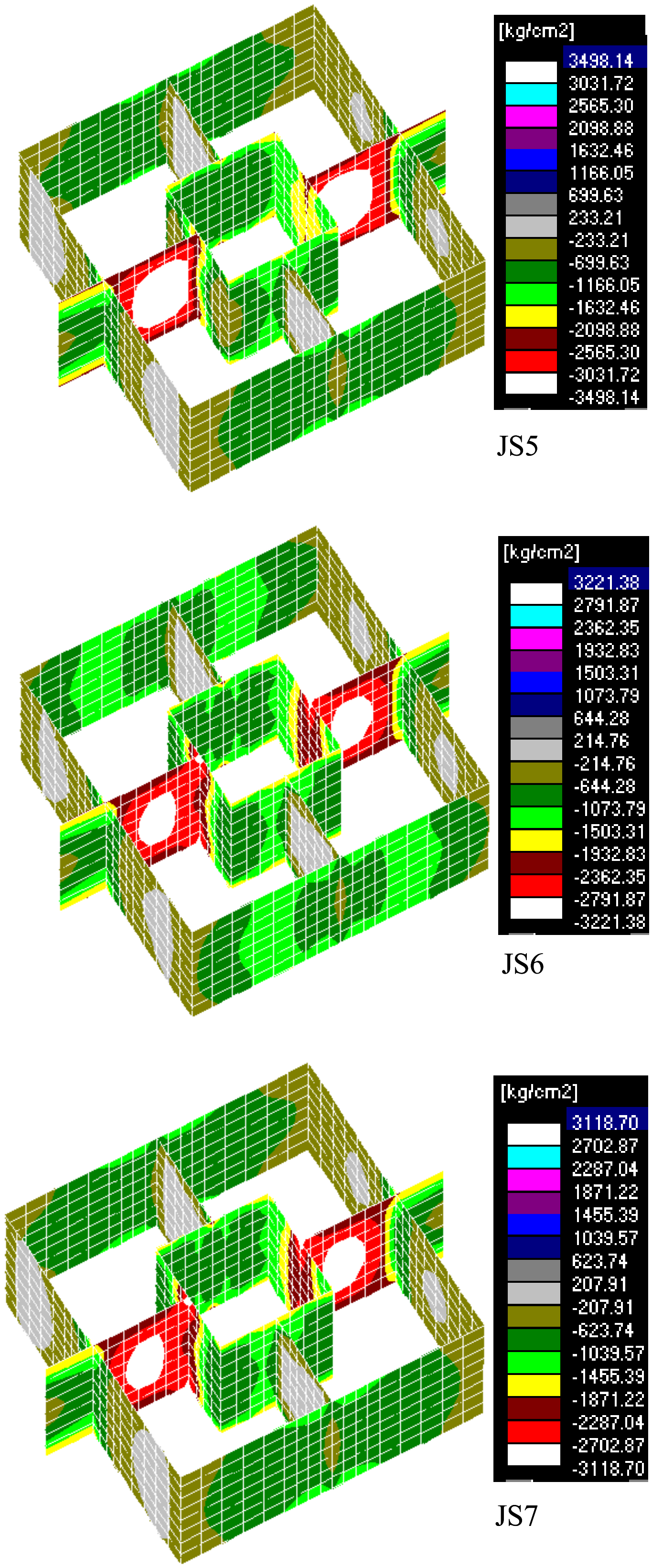


Figure 16. Stress distributions in the connections of specimens JS5, JS6, and JS7.

4.3 *Strength*

Empirical and experimental strengths are presented in Table 3. Existing equations were used to calculate the bending and shear strengths of beams and columns. The e function method of the Architectural Institute of Japan (AIJ) was used to calculate the bending moment capacity of the column. Its shear capacity was calculated using Urbon equation. The ultimate flexural strength of the beam and the shear strength were estimated using the equations provided by the AIJ. The shear strength of the beam-column connection was approximated using Sakaguchi equation. The actual shear strengths for specimens with outer confining plates were 1.04 to 1.31 times the calculated strengths using Sakaguchi equation for an average factor of about 1.16. For specimen JS3 without outer confining plates, the calculated strength was closer to the actual at a ratio of about 0.96. It is therefore possible to use Sakaguchi equation to determine the range of the shear strength of the hybrid connection.

Table 3. Experimental and analytical strengths.

	Specimen	JP1	JP2	JS1	JS2
column	axial(tonf)			67.0	
	flexural(tonf)	29.5	34.4	28.3	28.9
	shear(tonf)	22.2	23.0	29.3	30.1
	applied(tonf)	11.6	5.8	22.3	23.7
	ratio	1.91	3.97	0.79	0.82
beam	flexural(tonf)	3.5	3.5	10.1	
	shear(tonf)	30.6	30.6	20.9	
	max. load(tonf)	4.2	4.2	8.1	8.6
	ratio	1.2	1.2	0.8	0.85
conx	shear(tonf)	668	690	94.4	113.0
	applied(tonf)	65.0	32.1	124	132
	ratio	10.3	21.5	1.31	1.16

	Specimen	JS3	JS4	JS5	JS6	JS7
column	axial(tonf	67.0		57.0		
	flexural(tonf)	29.1	28.9	26.6	27.7	28
	shear(tonf)	30.2	30.1	29.1	29.8	28.9
	applied(tonf)	17.1	15.3	26.4	23.1	24.8
	ratio	0.59	0.53	0.99	0.83	0.89
beam	flexural(tonf)	11.4				
	shear(tonf)	39.3				
	max. load(tonf)	6.2	11.1	9.6	8.4	9.0
	ratio	0.54	0.97	0.84	0.74	0.79
conx	shear(tonf)	99.2	105	142	109	126
	applied(tonf)	94.9	84.9	147	129	138
	ratio	0.96	0.72	1.04	1.18	1.09

5 CONCLUSIONS

It can be concluded from the observed general performance that the presented hybrid beam-column connection provides an adequate strength and ductility. The hybrid connection method can be an alternative design in building frame type structures. Design can be made using finite element analytical model of the steel beam and steel plates in the beam-column connection with the reinforced concrete column action assumed as the boundary supports. The shear strength range of the connection can be calculated using Sakaguchi equation. Further investigation of the connection performance considering the bond between steel and concrete is recommended.