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Experimental studies on shear performances of RC interior column-beam joints with high-strength materials

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ABSTRACT: Reinforced concrete interior column–beam joints using high–strength materials were tested to study the shear performances of the joint. The maximum joint shear strengths did not increase so much as compared with the increase of the concrete compressive strength, and they located nearly on the curve of 1.88 times the square root of concrete compressive strength in MPa units for various concrete strength. Loading history gave a large effect on the hysteresis characteristics after the maximum strength, and the strength decay which was observed under reversed cyclic loading was not observed under monotonic loading. The confinement effects by the joint core lateral reinforcement became remarkable at the large deformation level after the story drift angle of 1/50rad., and strains in the compressive concrete strut in the joint panel were restrained, and the shear compression failure in the joint was delayed.

1 INTRODUCTION

Reinforced Concrete (RC) buildings of more than twenty stories have been constructed using high–strength concrete in seismic zones in Japan. It is desired that the high–strength concrete technology should be developed to meet the future demand in construction. A five-year National Research Project (New RC) has been promoted by the Ministry of Construction of Japan since 1988 to establish the design and construction technology using super high–strength materials in RC buildings. In the research project, the development of the design method of the column–beam joints is very important for the aseismic design of RC buildings. The provisions for the design of the column–beam joints with normal–strength materials were first included into the AIJ Design Guideline for Earthquake Resistant RC Building Based on the Ultimate Strength Concepts in 1988 (Architectural Institute of Japan, 1988). But it is necessary to verify if the provisions are applicable to the column–beam joints with the high–strength materials and to establish the universal design method of the joints independent of the material strengths. In this study, the shear performances of the interior column-beam joints with super high–strength materials were investigated from the test results as a part of the National Research Project (New RC).

2 TEST PROGRAM

2.1 *Design of Tests*

In the AIJ Design Guideline, joint shear stress is restrained below $0.3\sigma_B$ (σ_B:concrete strength). However, this limit value of the joint shear stress exceeds the shear strength of the recent test results of RC column-beam joints with high–strength materials (Shiohara et al. 1989, Sugano et al. 1989). Therefore, the application of the limit value of $0.3\sigma_B$ to the joint shear stress of the joint with the high–strength materials is difficult. In this test, the shear performances of the interior column–beam joints with super high–strength materials were investigated in order to contribute to the development of a rational joint design method which is applicable to the joints with from normal to high–strength materials.

2.2 *Specimens*

Six one–third scaled interior column–beam joints (called OKJ–series) with high–strength concrete and reinforcement were tested. The properties of specimens

Table 1 Properties of specimens

Specimen	OKJ-1	OKJ-2	OKJ-3	OKJ-4	OKJ-5	OKJ-6
(a)Beam Top Bars / Bottom Bars	9-D13 / 7-D13		10-D13 / 10-D13	9-D13 / 7-D13	10-D13 / 10-D13	8-D13 / 7-D13
Stirrups	□2-D6 @50 P_w=0.63%					
(b)Column Total Bars	20-D13		22-D13	20-D13	24-D13	20-D13
Hoops	□2-D6 @40 P_w=0.53%					
(c)Joint Detailes	□Hoops		□Hoops&Sub Ties		□Hoops	
Hoops	4-D6×3sets @50 P_w=0.54%					
Concrete Strength F_c	78MPa		118MPa	78MPa		59MPa
Joint Shear Stress*1 τ_{py}	15.1MPa $=1.7\sqrt{F_c}$ $=0.19F_c$		18.9MPa $=1.7\sqrt{F_c}$ $=0.16F_c$	15.1MPa $=1.7\sqrt{F_c}$ $=0.19F_c$	18.9MPa $=2.1\sqrt{F_c}$ $=0.24F_c$	14.2MPa $=1.8\sqrt{F_c}$ $=0.24F_c$
Loading History	Cyclic	Monotonic	Cyclic			

* : Joint shear stress at beam yielding τ_{py}
$\tau_{py} = 2 \cdot Q_{py} / (D_c \cdot (b_c + b_b))$
Q_{py}:Joint shear force at beam yielding
D_c:Column depth, b_c:Column width, b_b:Beam width

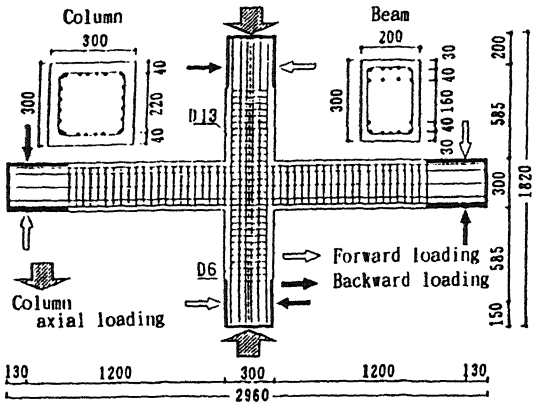


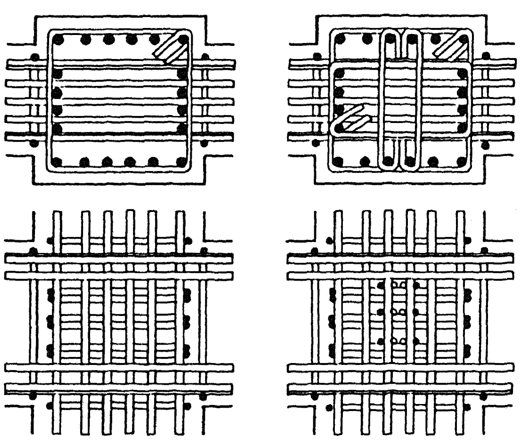
Fig. 1 Details of specimen OKJ–1

Table 2 Material properties

*:0.2%off set

Concrete Fc	Compressive Strength fc (MPa)	Strain at fc (μ)	Secant Stiffness at 25% Stress Lebel of fc (MPa)	Split Strength ft (MPa)
59	53.5	2230	36800	4.09
78	70.0	2960	35100	4.26
118	107.0	2860	43300	6.01

Bar Size	Yield Stress (MPa)	Yield Strain (μ)	Maximum Stress (MPa)	Young Modulus (MPa)
D6 (SD79)	*955	*7340	1140	182000
D13(SD69)	*718	*5870	767	186000



Specimen OKJ-1 Specimen OKJ-4

Fig. 2 Details in joint lateral reinforcement

are shown in Table 1. The arrangement of reinforcement of the specimens and member sections are shown in Fig. 1. The distance from the column center to the beam-end support was 1350mm, and the distance from the bottom support to the top horizontal loading point was 1470mm. The dimensions of beams and columns are 200 x 300mm, 300 x 300mm, respectively. The beams were reinforced by bars of diameter 13mm with a yielding strength of 718MPa, and the compressive strength of concrete was from 53.5 to 107Mpa, as shown in Table 2. The concrete was cast in forms in

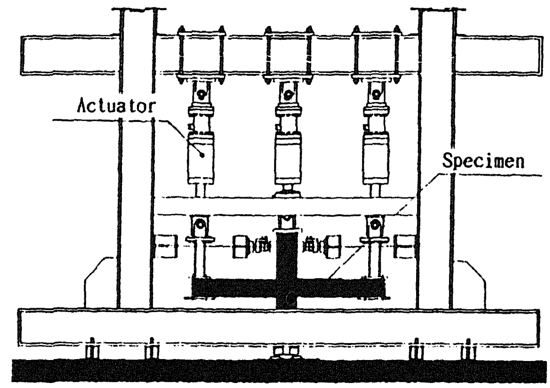


Fig. 3 Loading apparatus

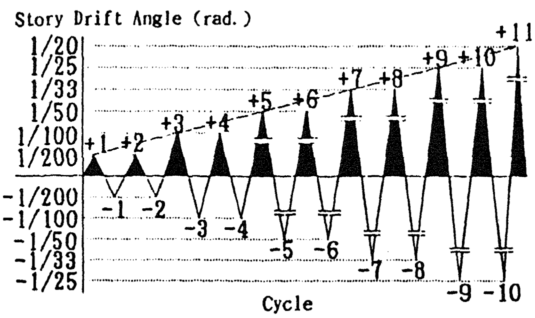


Fig. 4 Loading histories

sideways position. The effective joint area to resist shear was defined by the column depth and the average of the beam and column widths.

The joint lateral reinforcement ratio was 0.54%. The longitudinal reinforcement of columns in all specimens were designed to remain elastic.

Main parameters in the test were concrete compressive strength, σ_B = 53.5MPa, 70.0MPa, 107MPa, joint shear stress level; from 0.16 σ_B to 0.24 σ_B, loading histories; cyclic and monotonic, and lateral reinforcing details in the joint; with and without core lateral reinforcement (sub–ties) to increase the confinement of the joint, as shown in Fig.2.

2.3 *Loading and measuring methods*

Loading apparatus is shown in Fig. 3. Except for specimen OKJ–2 subjected to monotonic loading, two reversed cycles of loads were applied to the ends of two beams of the specimen under the each story drift angle of 1/200, 1/100, 1/50, 1/33, 1/25 rad., respectively, as shown in Fig. 4, with constant axial load (axial stress σ_0 = 0.12 σ_B). Tests were continued up to a final story drift angle of 1/20 rad.

The measurements were carried out for the story drift, deflections of columns and beams, joint shear distortions, strains in beam and column longitudinal bars and joint lateral reinforcement, and strains in the diagonal compressive concrete strut in the joint core.

Table 3 Test results

Specimen	Joint Shear Stress (MPa)						Failure Mode
	Joint Shear Crack			Maximum Load			
	τ_p *1	$_c\tau_p$ *2	*1/*2	τ_p *1	$_m\tau_p$ *3	*1/*3	
OKJ-1	7.53	7.26	1.04	14.24	15.73	0.91	B•J
OKJ-2	5.11	7.26	0.70	14.16	15.73	0.90	J
OKJ-3	4.80	8.65	0.55	17.80	19.44	0.92	J
OKJ-4	6.06	7.26	0.83	15.04	15.73	0.96	B•J
OKJ-5	5.84	7.26	0.80	14.80	15.73	0.94	J
OKJ-6	4.09	6.05	0.68	13.11	13.75	0.95	J

*1 : $\tau_p = 2 \cdot Q_p / (D_c \cdot (b_c + b_b))$
*2 : $_c\tau_p = F_t\sqrt{(1+\sigma_0/F_t)}, F_t = 0.5\sqrt{f_c}$
*3 : $_m\tau_p = 1.88\sqrt{f_c}$
Q_p: Joint shear force , D_c: Column depth
b_c: Column width , b_b: Beam width
σ_0: Column axial stress , f_c: Concrete strength

3. TEST RESULTS

3.1 *Failure progress*

Crack patterns at a story drift angle of 1/50 rad. of specimen OKJ–1 are shown in Fig. 5. In all specimens, the opening of joint shear cracks and splitting cracks along the column longitudinal bars from the end of joint shear cracks were remarkable.

Test results are shown in Table 3. In specimens OKJ–1, 4, beam flexural yielding occurred nearly at the story drift angle, Rs of 1/50, 1/33 rad., respectively. Afterward, they reached at the maximum strength at the same loading cycle. As the joint shear distortion, γ_p increased over 20 x 10^{-3} rad. at Rs = 1/33 rad., and the ratio of the joint shear distortion to the story drift reached over 30 %, it was considered that these two specimens failed in shear compression of the joint after the beam flexural yielding (called BJ). The other four specimens reached at the maximum strength at Rs = 1/33 rad. As the beam flexural yielding was not observed before the joint shear distortion increased, it was considered that these specimens failed in shear compression of the joint before the beam flexural yielding (called J).

3.2 *Joint shear strength*

Joint maximum shear stress–concrete compressive strength relations are shown in Fig. 6, as compared with the previous test results of interior column–beam joints using normal–strength and high–strength materials. The maximum joint shear stresses were 0.24 σ_B for specimen OKJ–6 with σ_B = 53.5MPa, about 0.2 σ_B for specimens OKJ–1, 2, 4, 5 with σ_B = 70.0MPa, and 0.17σ_B for specimen OKJ–3 with σ_B =107MPa. Therefore, it is shown that the joint shear strength is overestimated by the value of 0.3σ_B for the joint using concrete of which strength is higher than 40 Mpa. From Fig. 6, the maximum joint shear strength for various concrete strength located nearly on the curve of 1.88 $\sqrt{\sigma_B}$.

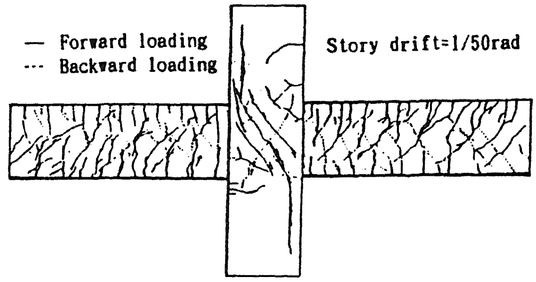


Fig. 5 Crack patterns of specimen OKJ–1

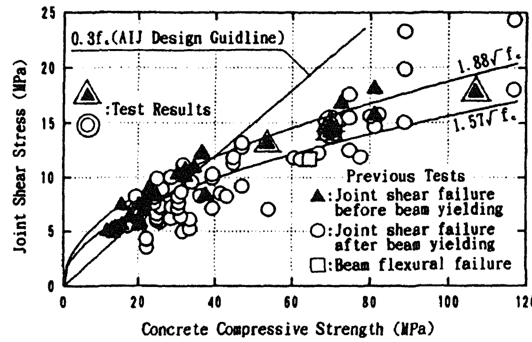


Fig. 6 Joint maximum shear stress – concrete compressive strength relations

3.3 *Hysteresis characteristics*

3.3.1 *Story drift*

Story shear–story drift relations are shown in Fig. 7. All specimens developed a fairly good spindle–shape hysteresis, and a remarkable pinching shape, which was caused by the bond deterioration of beam longitudinal bars through the joint, was not exhibited in the hysteresis loop.

As for specimen OKJ–1 of BJ type, beam flexural yielding occurred at the story drift angle, Rs = 1/50 rad., and the strength decayed under the reversed cyclic loading at Rs = 1/33 rad. As for specimen OKJ–2 subjected to monotonic loading, the strength decay was not observed until Rs = 1/20 rad., and the story drift and joint shear distortion at the maximum strength increased as compared with specimen OKJ–1 under reversed cyclic loading.

The strength of specimen OKJ–4 with joint core lateral reinforcement as a part of joint lateral reinforcement, as shown in Fig. 2, was not higher than that of specimen OKJ–1 with ordinary lateral reinforcement in the joint. However, the story drift angle of specimen OKJ–4 at the maximum strength increased to Rs = 1/33 rad., because of the confinement of joint concrete by the core lateral reinforcement.

The increase of the strength for specimen OKJ–5, which was designed as J type (joint shear failure type), was small with about 4% as compared with specimen OKJ–1 of BJ type.

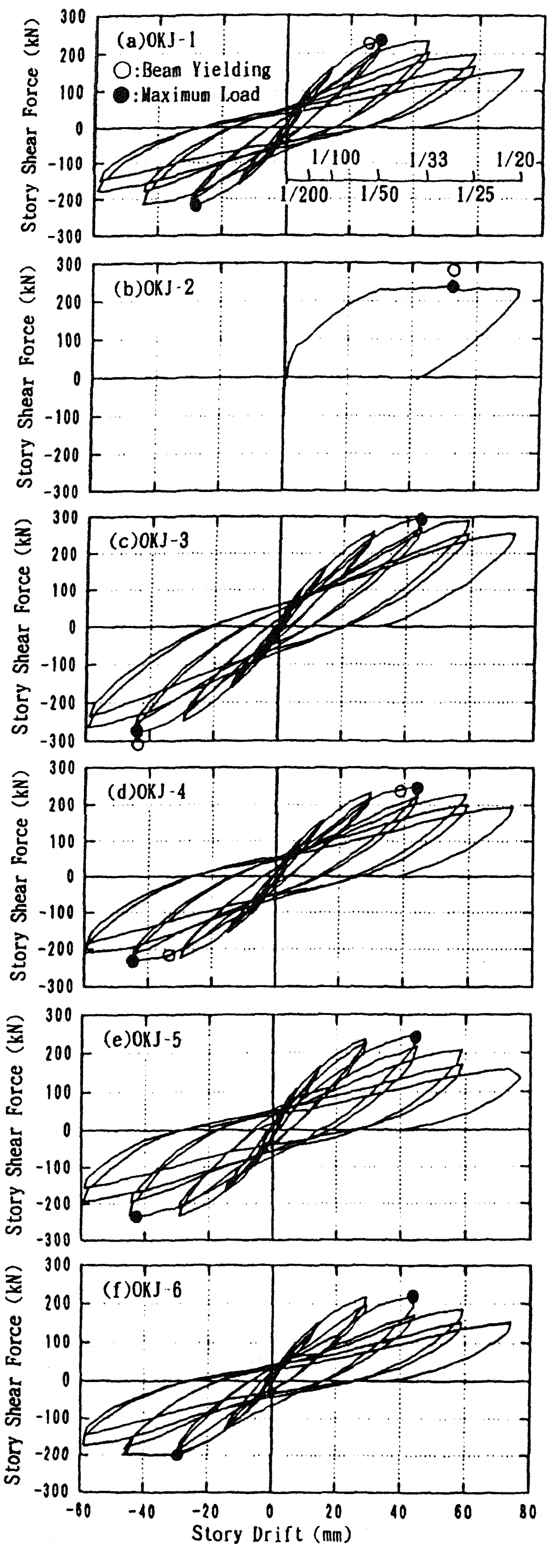


Fig. 7 Story shear force – story drift relations

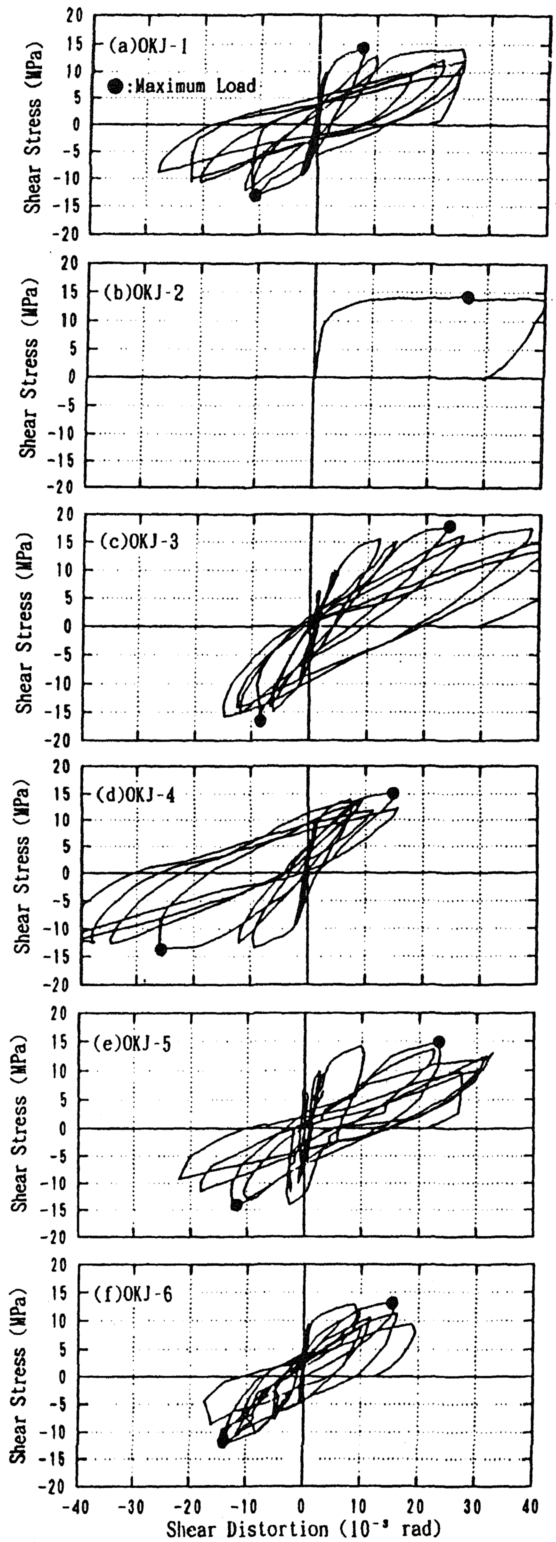


Fig. 8 Joint shear stress – shear distortion relations

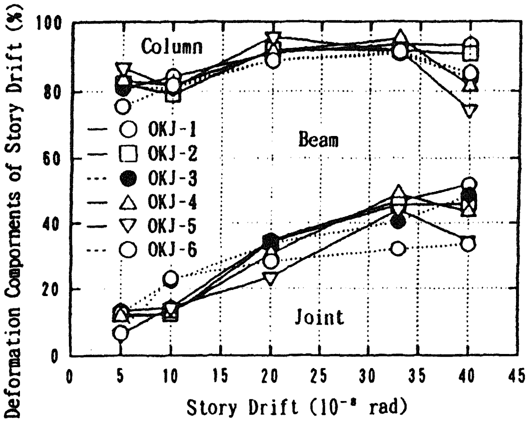


Fig. 9 Deformation components of story drift

3.2.2 *Joint shear distortion*

Joint shear stress–joint shear distortion relations are shown in Fig. 8. The joint shear distortion angle at the maximum strength increased over 20 x 10^{-3} rad., and it became a larger value than that of the joint with normal-strength materials, because the joint input shear level got higher from the use of high-strength materials. The joint shear distortion contributed to from 35% to 50% of the story drift at a story drift angle of 1/25 rad. (4%) for all specimens, as shown in Fig. 9.

3.3 *Strains in reinforcement*

3.3.1 *Strains in beam longitudinal bars*

Strain distributions of beam longitudinal bars (outside of top bars) through a joint at the peak of the first loading cycle at each story drift angle are shown in Fig. 10. The bond characteristics of these test specimens are kept within the least upper bound proposed in the AIJ Design Guideline. The bond characteristics of beam longitudinal bars through a joint were considered to be kept good in all specimens, because the phenomenon that the strain in the compressive reinforcement in the joint changed from compression to tension was not observed until the forward loading at the story drift angle, Rs = 1/50 rad. As for specimens OKJ-4, 6, strains in tensile reinforcement at the critical section reached over 0.005 at Rs = 1/50 rad., and the above-mentioned phenomenon appeared under the subsequent backward loading.

3.3.2 *Strains in joint lateral reinforcement*

Joint shear stress-strains in the joint lateral reinforcement relations are shown in Fig. 11 for specimens OKJ-1, 4. Joint lateral reinforcement parallel to a loading direction yielded in specimens OKJ-1, 3, 4 at Rs = 1/20, 1/33, 1/33 rad., respectively. The main difference between specimen OKJ-1 with the ordinary joint lateral reinforcement and specimen OKJ-4 with both ordinary and core lateral reinforcement was that the strains in the core lateral reinforcement in OKJ-4 increased firstly at Rs = 1/33 rad.

Lateral reinforcement orthogonal to a loading direction, which confined the joint core concrete yielded only in the core lateral reinforcement of specimen OKJ-3 at Rs = 1/25 rad., but strains in the core lateral reinforcement after Rs = 1/50 rad. as shown in Fig. 11. It is considered that the confinement effects by the joint core lateral reinforcement became remarkable at the large deformation level after Rs = 1/50 rad.

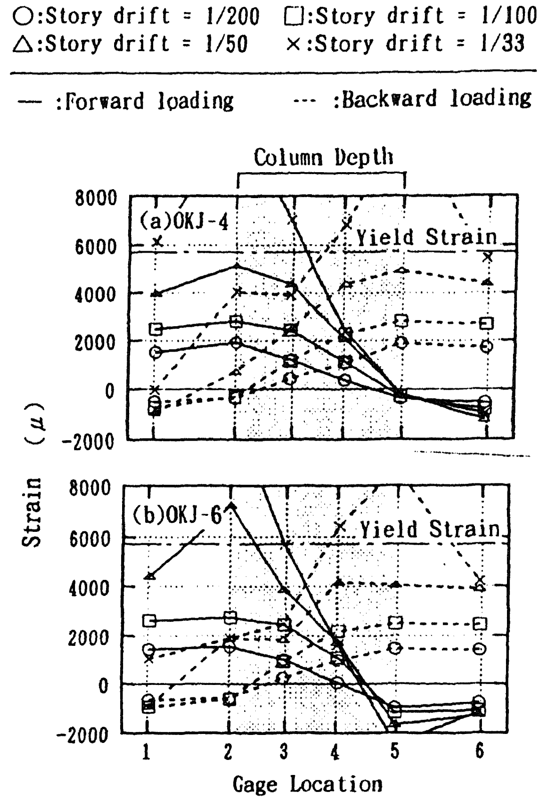


Fig. 10 Strain distributions of beam longitudinal bars through a joint

3.4 *Strains in joint compressive concrete strut*

Joint shear stress–strain along the compressive concrete strut of a joint relations are shown in Fig. 12. The strut strain in specimen OKJ-4 with core reinforcement in the joint is restrained to about 0.0025 from the confinement effect by core reinforcement even at Rs = 1/50 rad. On the other hand, the strut strain in specimen OKJ-1 without core reinforcement increased over 0.004 and came near the compression failure in the joint concrete at Rs = 1/50 rad. This difference at Rs = 1/50 rad. corresponded to the story draft angle where the confinement effects by the joint core lateral reinforcement became dominant.

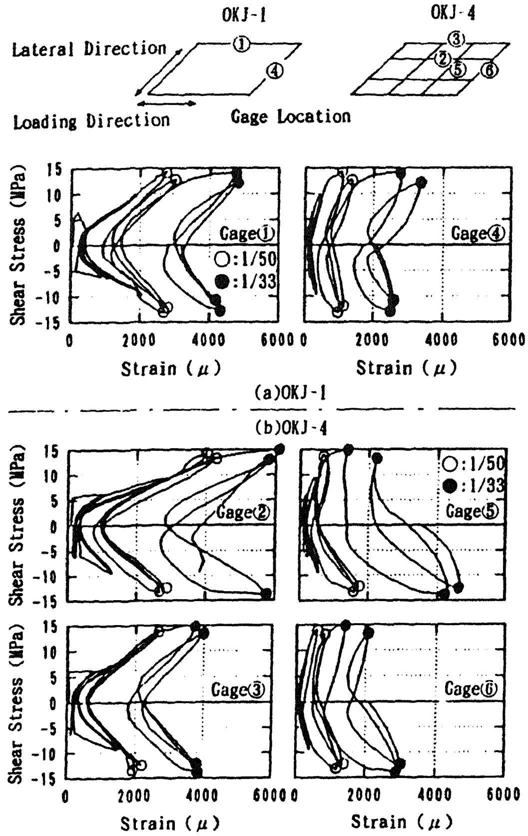


Fig. 11 Joint shear stress – strain in joint lateral reinforcement relations

4. CONCLUSIONS

Main findings in this study are as follows.

(1) The maximum joint shear strengths for from normal-strength to high-strength of concrete located nearly on the curve of 1.88 $\sqrt{\sigma_B}$, and they did not increase so much as compared with the increase of the concrete compressive strength, σ_B.

(2) It was possible to restrain the bond deterioration of beam longitudinal bars until the story drift angle of 1/100 rad., if the bond characteristics are kept within the least upper bound for avoiding bond deterioration proposed for the joint using normal-strength materials in the AIJ Design Guideline.

(3) Loading history gave a large effect on the hysteresis characteristics after the maximum strength, and the strength decay observed under reversed cyclic loading was not observed under monotonic loading.

(4) The confinement effects by the joint core lateral reinforcement became remarkable at the large deformation level after the drift angle of 1/50 rad., and strains in the compressive concrete strut in the joint panel were restrained.

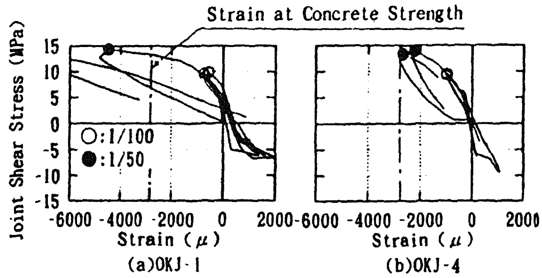


Fig. 12 Joint shear stress – strain along compressive concrete strut relations

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