AN EXPERIMENTAL STUDY ON STRENGTHENING ASR-DAMAGED REINFORCED CONCRETE MEMBERS WITH CARBON FIBER SHEET

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ABSTRACT

It has been confirmed that there is no significant reduction in flexural strength due to the confining effects of chemical prestress although ASR (alkali-silica reaction) causes cracks to occur in concrete structures. However, repairs are being carried out to inhibit the progress of cracks and steel corrosion caused by moisture. As cracks propagate, they reduce the shear strength of concrete, thus give rise to concern regarding their influence on the shear resistance of RC members. Steel jacketing method is a possible method to repair such ASR damaged RC members. There are some researches and applications of this method. However, installment of steel plates is not so easy work because of the considerably heavy weight of the plate.

In these days , CF (sheet made from continuous carbon fiber) has come to use in the civil engineering field to repair and strengthen RC members. The advantage of strengthening method with CFRP (CF impregnated and bonded with epoxy resin) is that it is easy to construct. CF is very light and flexible so it is easy to apply even in narrow space. On the other hand, comparing the construction cost with the steel plate bonding method, it will be expected that strengthening with CFRP will reduce construction cost under adequate strengthening design.

In the present study, two series of experiments were conducted and focused on the use of CF as the strengthening material for concrete structures. In the Series-1 experiment, the confining effects obtained by strengthening concrete with CF was confirmed, while in the Series-2 experiment, the strengthening effects of CF was examined using beam specimens.

MATERIALS

The mix proportions of concrete used in the experiment are shown in table 1. Cement used was ordinary Portland cement, while river sand (FM-2.73) from the Yasu River was used as fine aggregates, and crushed stone (FM-6.36, MS-20mm) produced from Takatsuki as coarse aggregates. 2900cc/m^3 of AE water reducing agent (25% solution) and 1450cc/m^3 of AE promoter (1% solution) were used in the mixture and expansion agent (E*) was added to expansive admixture concrete (expansive concrete) to simulate ASR-caused expansion. Table 2 shows the properties of the concrete, and table 3 shows the properties of the reinforcing bars. CF used for strengthening consisted of two types, the high-strength type and the high-elasticity type (of unit weights of 100, 200 and 300g/m^2). Their properties are shown in Table 4.

Table 1 Mix proportions of concrete

Type of concrete	W/C	s/a	Quantity of material(kg/m^3)				
	(%)	(%)	Water	Cement	E*	Sand	Crushed stone
Normal concrete	60	46	174	290	-	824	1008
Expansive concrete	60	46	174	217	73	821	1006

Table 2 Concrete specimens properties

Type of concrete	Compressive strength (kgf/cm^2)		Elastic modulus (kgf/cm^2)	
	28days	During the test	28days	During the test
Normal concrete	379	390	3.01×10^5	3.12×10^5
Expansive concrete	26	19	-	-

Table 3 Reinforcement properties

Reinforcement	Diameter	Yield strength	σ (tensile strength)	E (elastic modulus)
		(kgf/cm^2)		
Stirrup	D6	3500	5500	2.1×10^6
Compression bar	D10	4000	5900	2.1×10^6
Tension bar	D16	3900	5800	2.1×10^6

Table 4 The characteristic of carbon fiber

No	Type	Unit weight	Specific	Thickness	σ_{CF}	E_{CF}	σ_{CFRP}
		(g/m^2)	gravity	(mm)	(kgf/cm^2)		
①	High strength (CT)	100	1.80	0.056	49000	2.32×10^6	39000 (80%)
②		200		0.111			
③		300		0.167			
④	High elasticity (CM)	100	1.84	0.054	42000	4.45×10^6	29000 (70%)
⑤		200		0.108			
⑥		300		0.161			

SPECIMENS AND TEST METHOD

Series-1 Experiments

The experiments were carried out on 100 (diameter) x 200mm (height) cylindrical specimens. 24hours after casting, the casing were removed and the specimens were cured under water for 7 days. The specimens were then being air-cured, and after 3 weeks they were strengthened with CFRP. The strengthening was done by bonding the specimens in the circumferential direction with CF using epoxy resin. A 50mm development length was provided at the sheets ends. Three specimens were prepared for each type of the mix. After 28days the specimens were tested in uniaxial compression.

Series-2 Experiments

150x240x2000mm RC beams were used, with their corners chamfered (r=10mm) to prevent the strength from reducing due to folding of CF at the corners. Fig. 1 illustrates the shape of the beams. 3 types of beams were fabricated by varying the amount of stirrups in the 440mm shear span on both sides of the beam. The form works were removed after seven days, and the beams

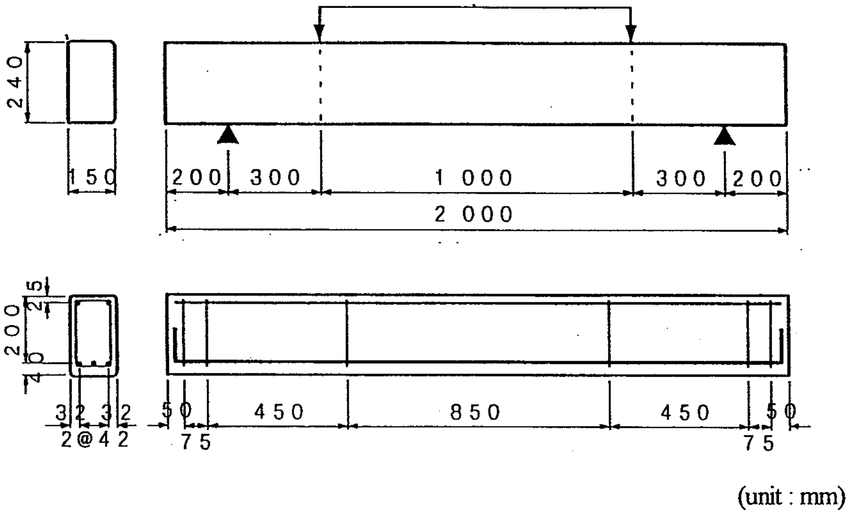

Fig.1 The shape of beam specimen

Table 5 Beam Specimen Types

No	Specimen No (*1)	Concrete Type	Shear reinforcement: Stirrup	CF (Carbon fiber sheet): Type of reinforcement	CF (Carbon fiber sheet): Winding pattern	pw•σw (*2) (kgf/cm²)
1	N15	Normal concrete	D6@15cm	—	—	9.85
2	N15-100Z			CT-100×1layer	Zebra	19.00
3	N15-100				Over all surface	29.40
4	N15-200			CT-200×1layer		48.95
5	N-200		—			39.09
6	N15-E100Z		D6@15cm	CM-100×1layer	Zebra	17.49
7	N15-E100				Over all surface	26.17
8	N5		D6@ 5cm	—	—	29.56
9	N5-300×2-1			CT-300×2layers	Over all surface	146.84
10	N5-300×2-3					
11	E15	Expansive concrete	D6@15cm	—	—	9.85
12	E15-100			CT-100×1layer	Over all surface	29.40
13	E15-200			CT-200×1layer		48.95
14	E-200		—			39.09

*1) D16×3 bars were used as tension reinforcement for all beams.
CT-300 (CT type CF of unit weight of 300g/m²)×1layer was applied for specimen No.9.
CT-300×3layers was applied for specimen No.10.

*2) pw・σw: amount of shear reinforcement (from Eq.3) were sprayed with water and air-cured. After 28 days, CF were bonded around the beams with epoxy resin. In order to confirm the strengthening effects, one or two layers of CF were bonded around the beams. CF(3) as shown in Table 4 was used for beams that were to be strengthened for flexure as well. Table 5 lists all the beam specimens.

Symmetrical 2-point loading with a distance of 1000mm between them were applied on the beam with an effective span of 1600mm, see fig.1. Beam deflections and strains in the reinforcing bars and CFRP were measured. The ratio of effective depth to shear span (a/d) was 1.5.

RESULTS

Series-1 Experiments

Table 6 shows the results of compression tests. : The result shows the values for the compressive strength (σ_C), the maximum longitudinal strain of concrete (Max. ε_V) and maximum transverse strain of the CFRP (Max. ε_H). Significant cracks were observed in the expansive concrete, resulting in a vast decrease in its compressive strength.

The load vs. longitudinal strain curve for concrete and load vs. transverse strain curve for the CFRP are shown in Figs. 2 and 3, respectively. Although the increase in the ultimate strain and strength of the concrete is in directly proportion to the applied amount of CF, the confining effects of the CFRP for the elastic zone of concrete seems small. The initial Young's modulus and yield load were found to be about the same as those of non-strengthened concrete, regardless of the CF type or the amount of CF used.

It can be seen that the ultimate longitudinal strain increases as the amount of CF increases depending on CF type. Meanwhile, the ultimate transverse strain of CFRP is influenced by CF type only, differing little by the amount of CF used for strengthening. It implies that this rapture strain is governed by the rapture strain of CFRP. However, compared with the ultimate transverse strain estimated from the Young's modulus and tensile strength of CF, the ultimate transverse strains obtained from the present experiments show a slight decrease. Possible reasons of this reduction are due to that the strains occurring in the antiplane of CFRP causes the curved surface formed by the fibers, and the longitudinal strains occurring in the orthogonal fiber direction inplane of CFRP in concrete.

Figure 4 shows Fcf/Fo (Compressive strength of reinforced concrete/Compressive strength of non-reinforced concrete) vs. $\rho cf \cdot \sigma ucf / Fo$ (Volumetric ratio of CF × Ultimate unit stress of

Table 6 Result of compression test

Specimen	Concrete	Reinforcement	σ_C (*1)	Increase of strength(*1)	Max. ε_V(*2) (longitudinal)	Max. ε_H(*2) (transverse)
V	Normal concrete	-	379	-	2000	-
CT-1		CT100	414	35	12600	13100
CT-2		CT200	575	196	14700	13100
CT-3		CT300	677	298	19700	12400
CM-1		CM100	422	43	5200	4700
CM-2		CM200	512	133	9800	5700
CM-3		CM300	592	213	11700	5600
E-1	Expansive concrete	-	26	-	-	-
E-2		CT300	376	350	-	-

*1) unit of strength : kgf/cm^2 , *2) unit of strain : $\times 10^{-6}$

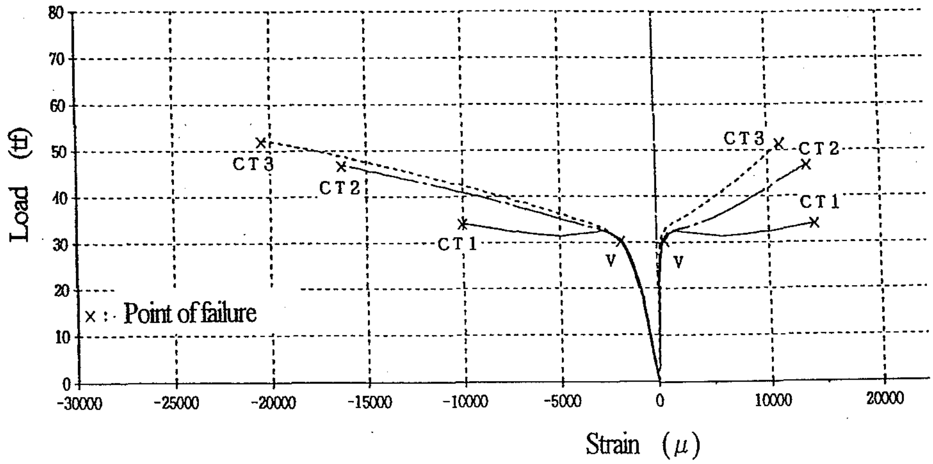

Fig.2 Load vs. strain (CT type CFRP)

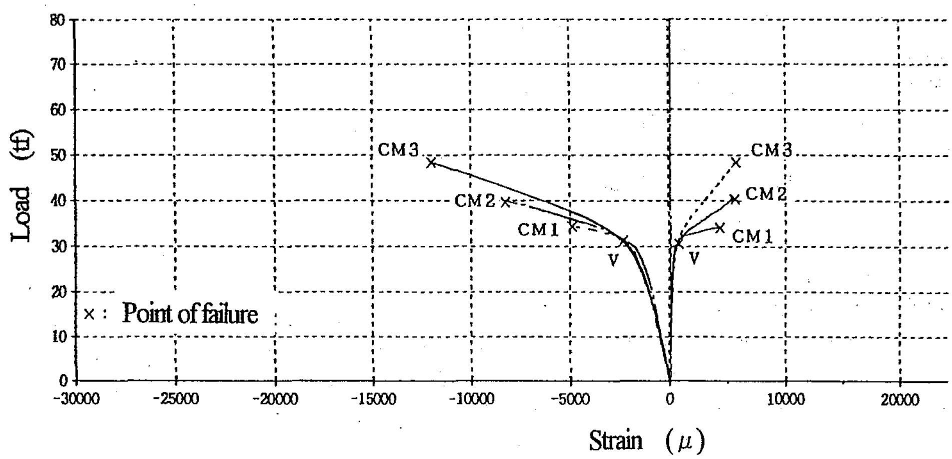

Fig.3 Load vs. strain (CM type of CFRP)

CF/Strength of non-reinforced concrete).

When the amount of CF for strengthening is zero, Fcf/Fo should be 1, but Fcf/Fo obtained from the regression formula was less than 1. Thus it is considered that the confining effects of CFRP cannot be readily effective within the initial elastic range. It can also be seen from Fig. 4 that the confining effects can be evaluated from $\rho cf \cdot \sigma cf$, and that the effects of high-strength fibers (CT)are greater than those of high-elasticity fibers (CM).

The evaluation expression for compressive strength of concrete confined by lateral hoops as proposed in literature 1) is given as Eq. (1). Comparing with the gradient obtained from this expression, the gradient obtained from the present analytical expression reduced by 57% for CT type, and 41% for CM type. In addition to this, the tensile strength of CFRP is 80% of CF for

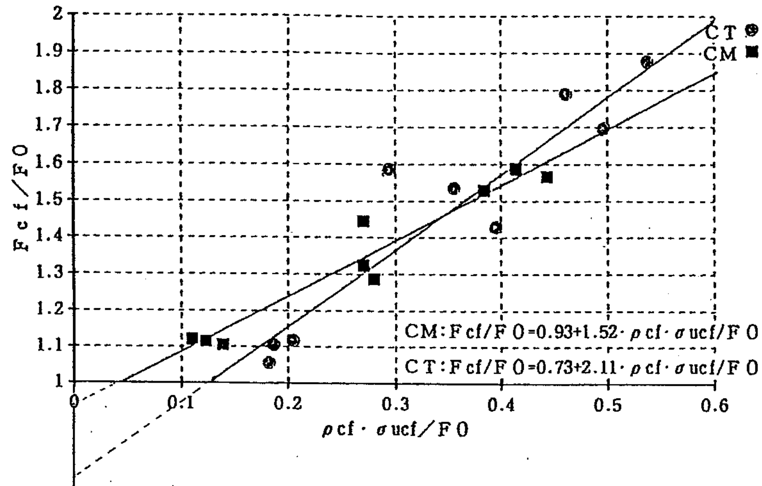

Fig..4 Confining effect of CFRP

CT type and 70% for the CM type (see table 4). Taking this reduction into account, the gradient ratios of Eq. (1) obtained from the analytical expression will be about 70% and 60%, respectively. This implied that, in calculating the amount of CF required for strengthening, it is desirable to conduct calculations separately for CT type and CM type, but, the average value, 0.65 may be practically used as the coefficient of reduction.

$fcc/fco = 1+3.70 \cdot \kappa \cdot \rho s \cdot fyh/fco$ Eq. (1)

where, fcc: Maximum compressive strength of confined concrete,

fco: Compressive strength of unconfined concrete,

s: Volumetric ratio of hoop reinforcement,

fyh: Yield strength of hoop,

κ: Coefficient in the case of circular confine (=1).

Series-2 Experiments

Table 7 shows the experimental results of beam loading test. Eq. (2) was used for calculating shear strength (Pvu) and Eq. (3) for calculating the amount of shear strengthening ($pw \cdot \sigma w$). In equation 3, the coefficient of reduction for the tensile strength of carbon fiber was set to 0.65, based on the findings of the Series-1 experiments.

$Pvu = \alpha \cdot Vc + Vw$ Eq. (2)

where, α: Coefficient due to arch action (=1.5),

Vc: Shear strength of concrete ($=0.94 \cdot \beta d \cdot \beta p \cdot (f'c)1/3 \cdot (0.75+1.4\ (a/d)) \cdot bw \cdot d$),

Vw: Shear strength of strengthening material ($= (pw \cdot \sigma w) \cdot bw \cdot z$).

$pw \cdot \sigma w = ps \cdot \sigma s + pcf \cdot \lambda \cdot \sigma cf$ Eq. (3)

$ps = As / (bw \cdot s)$, $pcf = bcf/bw$

where, σs: Yield strength of stirrup,

σcf: Tensile strength of carbon fiber sheet,

As: Sectional area of stirrup,

bcf: Thickness of carbon fiber sheet,

s: Spacing of stirrups,

λ: Coefficient of reduction for CFRP (= 0.65),

bw: Sectional width of beam

Table 7 Result of beam loading test

No	Theoretical value P'u (tf)		Test Result (tf)				pw・σw	Pu / P'u
	Pmu	Pvu	Pcr	Py	Pu	Mode	(kgf/cm²)	
1		24.00	12.00	—	26.50	SD	9.85	1.10
2		28.77	13.00	—	30.40	SD＊	19.00	1.06
3		34.20	13.00	31.25	33.80	BSD＊	29.40	0.99
4		44.40	14.00	31.00	34.80	BD	48.95	1.06
5	32.92	39.26	13.50	32.80	36.20	BD	39.09	1.10
6		27.99	12.00	—	29.80	SD＊	17.49	1.06
7		32.52	13.50	31.00	33.80	SD	26.17	1.03
8		34.28	13.00	31.50	33.00	BD	29.56	1.00
9	37.56	95.47	15.00	32.50	41.80	BD＊	146.84	1.11
10	45.44		15.50	38.30	53.10	BD＊	146.84	1.17
11		24.00	17.00	—	28.90	SD	9.85	1.20
12		34.20	17.50	31.00	32.40	BSD＊	29.40	0.95
13	32.92	44.40	16.50	30.50	31.70	BD	48.95	0.96
14		39.26	17.50	28.80	31.40	BD	39.09	0.95

Pmu: bending failure load, Pvu: shear failure load, Pcr: load when diagonal crack appeared, Py: bending yield load, Pu: failure load, SD: shear failure, BD: bending failure, BSD: after bending yield, shear failure, ＊: failure by breaking of CFRP.

As the amount of CF for shear strengthening increases, the failure mode is transformed from the shear- type to the flexural-type. Although, in some cases, breaking of CFRP tends to give rise to brittle failure, the measured ultimate strengths were confirmed to be essentially on the safe-side with respect to the theoretical values.

For all beam specimens, the load at diagonal cracking was found to be of the same level, implying that the strengthening effects of CFRP become significant after the initial diagonal crack has occurred.

Fig. 5 shows the relationship between the stirrup strain and the amount of CF used for shear strengthening for different load levels. The fact that the stirrup strain decreases with increasing amount of CF indicates that CF plays a role in carrying a part of shear force, reducing stress on the stirrups. This ensures that CF are acting effectively in resisting shear. Fig. 6 shows the load-deflection diagram for specimens strengthened against flexure. It can be seen that both the flexural rigidity and the ultimate strength improve as the amount of CF for flexural strengthening increases, thus providing evidence that CFRP has strengthening effects against flexure as well.

In terms of beam strengthening, no significant difference was observed between CT type and CM type. This may be attributed to the fact that the strengthening effects develops only after the initial diagonal crack occurred, and therefore, the confining effects remain practically the same for the low-strain range.

As for the expansion agent-added concrete beams, in which, the expansion is confined by the stirrups and acted as chemical prestress, the strength of the non-strengthened specimens (No.11) was found to be higher than that of normal concrete beams (No.1). Since the strengthening effects gained with the use of CFRP in expansive concrete provide performance levels similar to

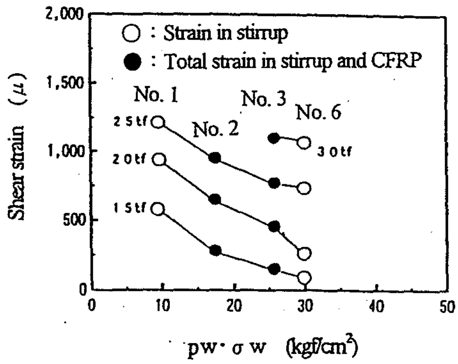

Fig.5 Shear strain vs. p w • σ w

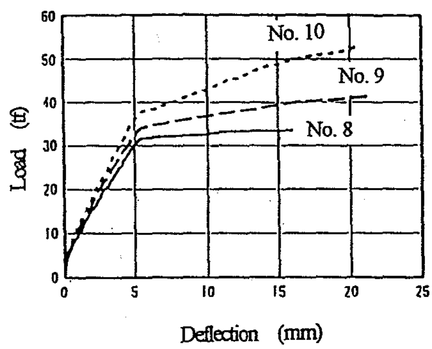

Fig.6 Load vs. deflection

those of normal concrete beams, CFRP can be expected to provide excellent shear strengthening effects for ASR-damaged members.

CONCLUSION

The main result obtained from the present study are as follows:

(1) By bonding CF around concrete members, the compressive strength of concrete can be increased. The increase in strength is dependent on the tensile strength of CF and the amount of CF used for strengthening.

(2) Bonding around of CF causes the ultimate longitudinal strain of concrete to increase dramatically in proportion to the amount of CF. Application of this method is expected to be useful in strengthening ASR-damaged concrete members.

(3) Since bonding around of CF also causes a remarkable increase in the ultimate transverse strain, it is possible to confine ASR-caused concrete expansion as well. The ultimate transverse strain, however, depends on the type of CF used and not on the amount of CF.

(4) When CF are used as shear reinforcements, the shear strength increases in proportion to the amount of CF used for strengthening.

(5) When CF are bonded around concrete members damaged by expansion, CFRP effectively resist shear forces in the same way as when they are used for normal concrete members.

(6) When CF are bonded only to the side surfaces of beam to serve as flexural reinforcements, the flexural strength of the beam increases in proportion to the amount of CF.

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