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Flexural Behavior of Reinforced Recycled Concrete Beams

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Abstract

In order to evaluate whether concrete with recycled aggregate can be applied for concrete structures, flexural loading tests of reinforced recycled concrete members were carried out. The recycled coarse aggregate and the recycled fine aggregate were produced mainly from various reinforced concrete members of a building structure as well as from 300 mm cubic concrete specimens. The properties of concrete with recycled aggregates, such as strength, Young's modulus, shrinkage, creep and shrinkage-induced stress depending on the combination of natural and recycled aggregates, curing condition, and water to binder ratio, were discussed based on a comparison with the properties of concrete with virgin aggregates. Furthermore, the flexural behavior of reinforced recycled concrete beams was also discussed through comparison with the behavior of companion virgin concrete beams in which the tension reinforcement ratio, curing condition, and water to binder ratio of concrete, are the main factors. The results indicate the mechanics-based possibility of utilizing recycled concrete for reinforced concrete structures under the proper design and within the proper limit of application.

1. Introduction

According to the White Paper on the Environment 2000 released by the Ministry of the Environment (White Paper 2000), concrete waste amounted to 35 Mt, out of which 1.3 Mt was scrapped. This indicates that 96% of concrete waste is recycled in Japan. However, almost all of the concrete waste that is recycled is used for pavement base or back filling for retaining walls, which does not necessarily require high performance compared with structural concrete. Such limited use of concrete waste is attributable to not only the unclear quality of the original concrete but also low and scattered quality due to high porosity and impurity. From the viewpoint of promoting resource saving, energy saving, and environmental preservation, it is essential to study how waste concrete can be used effectively as a structural material.

Some studies concerning concrete and reinforced concrete (RC) made of recycled aggregate have been performed (Torii *et al.* 1984, Nanba *et al.* 1995, Sato *et al.* 2000). It has been indicated that instantaneous deflection of reinforced concrete beam after fatigue loading increases while ultimate flexural moment does not decrease, and that bond stiffness as well as shear strength decreases in some cases (Mukai *et al.* 1978, Yagishita *et al.* 1991). It is, however, difficult to quantitatively estimate the effects of recycled aggregate on the structural behavior of RC members, because data related to the

material properties and mix proportions of the original concrete are hardly accessible in many cases, and the original aggregates in the target recycled aggregate concrete are different from the aggregates in the reference concrete.

In utilizing recycled aggregate as a structural aggregate, inferior concrete quality caused by the significant amount of porosity in clinging mortar or paste is a major concern. Based on this fact, high-quality recycled coarse aggregate is generally recognized to be aggregate from which as much mortar as possible has been removed, making it better for structural application. While this approach affords higher quality recycled aggregate, it is highly onerous in terms of energy and cost, and raises the need to develop a method for the utilization of the removed mortar. From the viewpoint of a feasible recycling system, this approach holds little promise. The aim of the present study is to investigate the applicability of recycled aggregate containing mortar or cement paste to structural materials, by comparing the flexural behavior of RC members produced from recycled aggregate concrete with that of RC members using conventional aggregates.

Firstly, the physical properties of recycled aggregate concrete (RAC), whose original aggregate was the same as that of the reference virgin concrete, were compared with those of the virgin concrete (VC). This series of experiments was named 'Series A'. The original concretes were made with water-to-cement ratios (W/C) of 0.45 and 0.63, and two types of coarse recycled aggregate and two types of fine recycled aggregate were prepared using a jaw crusher and impact crusher. Using these recycled aggregates obtained from the original concretes with different W/C, several type of recycled aggregate concrete with water-to-cement ratios of 0.6 and 0.25 were produced. The physical properties of these RACs as well as the flexural behaviors of reinforced

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concrete beams using these RACs were compared with those of the VC, and the effects of the types of RAC on the physical properties of concrete as well as the flexural behaviors of RC beams were discussed.

Secondly, aiming for practical use, RAC with recycled aggregate from the real market was investigated through comparison with the properties of virgin concrete. This series of experiments was named 'Series B'. The recycled aggregate was mainly from the beams, columns, and slabs of reinforced concrete (RC) buildings built in 1961 and 1967 in Hiroshima City. The W/C of the original concrete was estimated as ranging from 0.55 to 0.65. The recycled aggregate was produced by a jaw crusher and impact crusher and the level of crushing was almost the same as that of the recycled aggregate of Series A. In this series, different water-to-cementitious materials ratios (W/C or W/B), namely 0.6, 0.45 and 0.3, were set as parameters, and based on the fact that the compressive strength of RAC can be controlled with W/C, the physical properties of RAC were evaluated relative to compressive strength. Additionally, from the structural point of view, the physical properties of RAC and the flexural behavior of RC beams with RAC was evaluated through JSCE design code and the present analysis method, and the applicability of RAC for practical use was discussed.

2. Experiments

2.1 Materials and mixture proportions

As tabulated in **Table 1**, two kinds of original concrete with W/C of 0.45 and 0.63, whose symbols are OC45 and OC63, respectively, were prepared to make recycled fine

and coarse aggregates. The coarse and fine aggregates consisted of crushed hard sandstone and river sand.

Table 2 lists the above-mentioned natural aggregates and recycled aggregates along with several physical properties. Two kinds of coarse aggregate, RG-1 and RG-2, and two kinds of recycled fine aggregate, RS-1 and RS-2, were produced from OC45 and OC63, respectively. RG-3 and RS-3 are mixes featuring 1/3 parts of RG-1 and 2/3 parts of RG-2, and 1/3 parts of RS-1 and 2/3 parts of RS-2, respectively.

The G-1 and S-1 aggregates for reference virgin concrete V-01 in **Table 3** are the same as those in the original concrete, and the G-2 and S-2 aggregates for V-03 in the same table are natural aggregates produced at different sites from G-1 and S-1.

The other recycled aggregates (RG-4 and RS-4) in Series B were from a real site.

Table 2 also gives the ratios of mortar content to total coarse aggregate in recycled coarse aggregate and cement paste content to total fine aggregate in recycled fine aggregate. The ratios of mortar and cement paste contents in the 30% to 50% range were used in an attempt to utilize recycled concrete waste as effectively as possible.

Table 1 Mixture proportions of original concrete for producing recycled aggregate in Series A.

Symbol	Type of aggregate		W/C	s/a (%)	Unit weight (kg/m ³)					
	G	S			W	C	S	G	SP ^{*1}	AE ^{*2}
OC45	G-1	S-1	0.45	43	170	378	749	1004	3.4	—
OC63	G-1	G-1	0.63	47	167	267	867	988	—	0.67

*1: Superplasticizer, *2: Air-entraining agent

Table 2 Properties of aggregates.

Series	Symbol		Density at saturated surface dry state (g/cm ³)	Oven-dry density (g/cm ³)	Absorption (%)	F.M.	Mortar/paste content (%)
A		JIS A 5308 (Coarse agg.)	—	≥2.5	≤3.0	—	—
		JIS A 5308 (fine agg.)	—	≥2.5	≤3.5	—	—
	G-1	Crushed hard sandstone (Oume ^{*1})	2.66	2.64	0.69	6.73	—
	S-1	River sand	2.63	2.56	2.63	2.97	—
	RG-1	Recycled agg. (W/C=0.45) ^{*2}	2.41	2.29	6.13	6.62	50.8
	RG-2	Recycled agg. (W/C=0.63) ^{*1}	2.41	2.29	5.28	6.53	50.3
	RG-3	Recycled coarse aggregate composed of 1/3 parts of RG-1 and 2/3 parts of RG-2					
	RS-1	Recycled agg. (W/C=0.45) ^{*2}	2.32	2.11	9.94	3.17	38.2
	RS-2	Recycled agg. (W/C=0.63) ^{*3}	2.3	2.07	11.02	3.27	33
B	RS-3	Recycled fine aggregate composed of 1/3 parts of RS-1 and 2/3 parts of RS-2					
	G-2	Crushed hard sandstone (Iwase ^{*1})	2.64	2.62	0.82	6.68	—
	S-2	Land sand	2.6	2.56	1.44	2.64	—
	RG-4	Recycled agg. ^{*4}	2.46	2.32	6.18	6.37	43.4
	RS-4	Recycled agg. ^{*4}	2.23	1.98	12.51	2.66	24.9

*1: Area of production

*2: Recycled aggregate was produced by jaw crusher and impact crusher from the original concrete using aggregate G-1 and S-1 (Mixture proportion is OC45 in Table 2). The age of concrete at crushing is 655 days.

*3: Recycled aggregate was produced by jaw crusher and impact crusher from the original concrete using aggregate G-1 and S-1 (Mixture proportion is OC63 in Table 2). The age of concrete at crushing is 655 days.

*4: Recycled aggregate was produced mainly from the beams, columns and slabs of two RC buildings built at 1961 and 1967 in Hiroshima city. Water to cementitious material ratio of original concrete was estimated as 0.55-0.65. Recycled aggregate was produced by jaw crusher and impact crusher.

These ratios of content were obtained through a test method for insoluble residue in recycled aggregate using hydrochloric acid.

Table 3 lists the mix proportions and names of 21 kinds of virgin and recycled concretes. The capital 'V' stands for reference concrete with virgin coarse and fine aggregates, 'CR' stands for concrete with recycled coarse aggregate and virgin fine aggregate, 'FR' stands for concrete with virgin recycled coarse aggregate and recycled fine aggregate, and 'CFR' stands for concrete with recycled coarse and fine aggregates. In three cases, expansive additive (EX) was used for concrete in an attempt to improve flexural performance of RC beams with recycled aggregate. In several cases, pigment with 2% of cement content was blended into recycled aggregate concrete to observe the rupture of aggregate after cracking.

2.2 Physical properties of concrete

The compressive strength and Young's modulus of $\phi 100 \times 200$ mm cylinders of concrete, and the splitting tensile strength of $\phi 150 \times 200$ mm cylinders of concrete were determined according to the corresponding JIS (Japanese Industrial Standard) methods. The specimens were cured after demolding under the following conditions: 20°C water curing (standard curing), sealed curing at room temperature (sealed curing), sealed with saturated paper

at room temperature (wet curing), and exposure to room atmosphere after 1-week wet curing (drying condition).

The effects of recycled coarse and fine aggregates on shrinkage strain was tested using specimens with a length of 500 mm and the same cross section as the reinforced concrete (RC) specimens, as described in greater detail later (**Fig. 1**). Shrinkage strain was measured by embedded strain gauges of a size of $\phi 20 \times 104$ mm and low elastic modulus of 39 N/mm². An embedded strain gauge and thermocouple were located at the center of the specimen in parallel to the longitudinal direction. Measuring was performed just after drying in the case of the recycled concrete with W/C of 0.6, and just after placing in the case of the high-strength concrete with W/C of 0.25, namely HV-01 and HCFR-01. The measurement for the latter aimed to confirm that recycled aggregate can be effective in decreasing autogenous shrinkage as an internal curing material (Paillere *et al.* 1989). Autogenous deformation as well as drying shrinkage of concrete were determined by subtracting thermal strain from measured strain. The thermal expansion coefficient was assumed to be 10×10^{-6} for the thermal strain of concrete.

Shrinkage-induced stress was evaluated by measuring the strain of reinforcing bars (rebars) in RC specimens, as described in greater detail later. Shrinkage-induced stress in concrete at the extreme bottom fiber due to restraint of

Table 3 Mixture proportions of original and recycled concretes.

Case	Symbol	Type of aggregate		W/C	s/a (%)	Unit weight (kg/m ³)									
		G	S			W	C	S	G	SP*1	AEWR*2	AE*3	D*4	EX*5	P*6
A	V-01	G-1	S-1	0.60	46.9	170	283	860	900	—	0.57	1.42	—	—	—
	CR45-01	RG-1	S-1	0.60	47.2	167	278	872	900	—	0.42	0.83	—	—	5.57
	CR60-01	RG-2	S-1	0.60	47.2	167	278	872	900	—	0.70	0.70	—	—	5.72
	FR60-01	G-1	RS-2	0.60	47.0	185	308	732	948	—	0.62	1.54	—	—	—
	CFR-01	RG-3	RS-3	0.60	46.9	176	293	755	885	—	0.73	0.73	—	—	5.86
	HV-01	G-1	S-1	0.25	40.0	161	645	656	996	9.68	—	—	—	—	—
	HCFR-01	RG-3	RS-3	0.25	40.0	161	645	581	902	9.67	—	—	—	—	5.86
B	V30-03	G-2	S-2	0.30	41.7	178	593	647	932	5.93	—	—	1.78	—	—
	CR30-03	RG-4	S-2	0.30	41.7	178	593	647	853	5.34	—	—	1.19	—	11.9
	CFR30-03	RG-4	RS-4	0.30	40.8	177	588	543	870	10.6	—	—	1.77	—	11.8
	V45-03	G-2	S-2	0.45	41.7	171	381	727	1048	—	—	2.28	—	—	—
	CR45-03	RG-4	S-2	0.45	41.7	171	381	727	958	0.76	—	0.76	—	—	7.61
	CFR45-03	RG-4	RS-4	0.45	41.8	170	378	625	960	2.27	—	—	—	—	7.55
	V60-03	G-2	S-2	0.60	47.7	187	311	840	935	—	1.56	—	—	—	—
	CR60-03	RG-4	S-2	0.60	47.0	164	292	859	848	—	1.28	—	—	—	5.84
	CFR60-03	RG-4	RS-4	0.60	46.8	186	309	706	886	—	1.93	—	—	—	6.18
	VEX45-03	G-2	S-2	0.45	41.7	171	351	727	1048	—	—	3.08	—	30	—
	CREX45-03	RG-4	S-2	0.45	41.7	171	351	727	958	0.76	—	0.76	—	30	7.61
	CFREX45-03	RG-4	RS-4	0.45	41.8	170	348	625	960	2.27	—	—	—	30	7.55

*1: Superplasticizer, *2: Air-entraining and water-reducing agent, *3: Air-entraining agent, *4: Anti-forming agent

*5: Expansive additive (lime type), *6: Pigment

the rebar is determined by the equilibrium requirement for concrete and rebar as well as the assumption of linear strain distribution. The dimensions of the specimens are presented in Fig. 1.

Additionally, to explain the creep of recycled concretes compared with that of virgin concretes, 24 specimens were produced with a section of $150 \times 200 \text{ mm}^2$ and length of 300 mm, as well as 2 specimens for each of the 3 loading ages of 7, 28, 180 days under wet and drying conditions. The setup for the creep test is illustrated in Fig. 2. The creep strain was measured by an embedded strain gauge identical to those used for shrinkage measurements. The stress-to-strength ratio applied to the specimens was fixed to 0.1 for all loading cases.

Fracture energies of concretes were obtained in accordance with the method proposed by RILEM (RILEM 1985), in which the specimen size was $100 \times 100 \times 400 \text{ mm}$. Applied load was measured by a load cell with 250 kN capacity, deflection at the span center was measured by a displacement transducer with a minimum graduation of $1/1000 \text{ mm}$, and crack opening displacement was measured by clip-gauge with an accuracy of $1/1000 \text{ mm}$. The fracture energy G_f was calculated with the equation $G_f = (W_0 + mg\delta_0) / A_{lig}$ (N/m), where W_0 is the integration of load-displacement relationship (Nm), $m = m1 + m2$ (kg), $m1$ is the mass of the beam between supporting points with a distance of 300 mm, $m2$ is the mass of the loading equipment, g is the acceleration of gravity, δ_0 is the displacement at fracture, and A_{lig} is the fractured area. The ages at testing were 3 and 28 days, and 5 specimens were tested for each concrete.

2.3 Beam specimens and loading test method

Thirty-seven beams were prepared in order to investigate the effects of the following factors on the flexural behaviors, i.e., W/C of original concrete used for producing recycled aggregate, usage of recycled fine aggregate and/or recycled coarse aggregate, curing condition, W/C of recycled concrete, and tension reinforcement ratio. All of the RC flexural beam specimens are listed in Table 4. Ten specimens were produced from virgin concrete and twenty-seven specimens from the recycled concrete. The reinforced concrete specimens for shrinkage-induced stress testing are also listed in Table 5.

The size of the specimens for flexural behavior is $150 \times 200 \times 2800 \text{ mm}$. The details are shown in Fig. 1. The length of the zone subjected to pure bending moment was fixed to 800 mm and the full span was 2200 mm for all the flexural beams. The mechanical properties of reinforcement are listed in Table 6.

The setup of the loading test is shown in Fig. 1. All the specimens were loaded at two points symmetrically about the center section. Flexural beams were unloaded and reloaded twice prior to cracking, then loaded at every increment of 50 N/mm^2 in terms of tension reinforcement stress prior to yielding in tension reinforcing bars, and

Fig. 1 Details of specimens for tests of loading, shrinkage strain and shrinkage-induced stress, and setup for loading test.

Fig. 2 Hydraulic load system for creep test.

thereafter at every deflection increment of $2\delta_y$ up to failure. Here, δ_y is deflection at yielding.

In the instantaneous loading test for every RC member, the magnitude of the loads was measured by a load cell with 100 kN capacity and deflection at the span center was measured by a displacement transducer with a minimum graduation of $1/1000 \text{ mm}$. The crack widths of flexural beams were measured at the same level as tension reinforcement by π -shaped displacement meters with 100 mm gauge length and minimum graduation of $1/1000 \text{ mm}$.

Table 4 Reinforced concrete specimens for flexural loading test.

Series	Symbol	C ^{*1}	W/C	Tension reinforcement		Loading age (day)	f'_c ^{*4} (N/mm ²)	f_t ^{*5} (N/mm ²)
				Reinforcement	p_t (%)			
A	V-01-10WB	w ^{*2}	0.6	2@D10	0.59	65	30.6	2.9
	V-01-10DB	d ^{*3}	0.6	2@D10	0.59	121	32.5	3.0
	CR45-01-10WB	w	0.6	2@D10	0.59	57	30.4	2.6
	CR45-01-10DB	d	0.6	2@D10	0.59	108	28.4	2.4
	CR60-01-10WB	w	0.6	2@D10	0.59	142	34.5	2.8
	CR60-01-10DB	d	0.6	2@D10	0.59	134	31.8	3.3
	V-01-13WB	w	0.6	2@D13	1.06	65	30.6	2.9
	V-01-13DB	d	0.6	2@D13	1.06	121	32.5	3.0
	CR45-01-13WB	w	0.6	2@D13	1.06	57	30.4	2.6
	CR45-01-13DB	d	0.6	2@D13	1.06	108	28.4	2.4
	CR60-01-13WB	w	0.6	2@D13	1.06	142	34.5	2.8
	CR60-01-13DB	d	0.6	2@D13	1.06	134	31.8	3.3
	FR60-01-13WB	w	0.6	2@D13	1.06	79	24.5	2.4
	FR60-01-13DB	d	0.6	2@D13	1.06	78	23.8	2.0
	CFR-01-13WB	w	0.6	2@D13	1.06	87	23.5	2.3
	CFR-01-13DB	d	0.6	2@D13	1.06	86	23.5	2.0
	HV-01-13DB	d	0.25	2@D13	1.06	73	68.7	3.0
	HCFR-01-13DB	d	0.25	2@D13	1.06	93	68.1	2.3
	V-01-16WB	w	0.6	2@D16	1.65	65	30.6	2.9
	V-01-16DB	d	0.6	2@D16	1.65	121	32.5	3.0
	CR45-01-16WB	w	0.6	2@D16	1.65	57	30.4	2.6
	CR45-01-16DB	d	0.6	2@D16	1.65	108	28.4	2.4
	CR60-01-16WB	w	0.6	2@D16	1.65	142	34.5	2.8
	CR60-01-16DB	d	0.6	2@D16	1.65	134	31.8	3.3
B	V30-03-WB	w	0.3	2@D13	1.06	99	106.4	6.3
	CR30-03-WB	w	0.3	2@D13	1.06	76	69.0	3.9
	CFR30-03-WB	w	0.3	2@D13	1.06	99	53.8	3.7
	V45-03-WB	w	0.45	2@D13	1.06	60	57.0	3.0
	CR45-03-WB	w	0.45	2@D13	1.06	88	46.5	3.0
	CFR45-03-WB	w	0.45	2@D13	1.06	70	35.5	2.6
	V60-03-WB	w	0.6	2@D13	1.06	105	40.2	3.5
	CR60-03-WB	w	0.6	2@D13	1.06	41	32.9	2.7
	CFR60-03-WB	w	0.6	2@D13	1.06	106	29.4	2.3
	VEX45-03-WB	w	0.45	2@D13	1.06	61	55.3	3.6
	CREX45-03-WB	w	0.45	2@D13	1.06	93	46.6	3.4
	CFREX45-03-WB	w	0.45	2@D13	1.06	66	35.2	2.5

*1: Curing condition, *2: Wet curing, *3: Drying condition, *4: Compressive strength at loading age,

*5: Splitting tensile strength

The long-time behavior under sustained load of RC beams loaded at the age of 28 days was also investigated. RC beams V-01-13WB, V-01-13DB, CFR-01-WB, and CRF-01-DB, which are $150 \times 200 \times 2800$ mm³ in size,

with a pure moment zone of 800 mm, and reinforcement ratio of 1.06%, with different moisture condition, i.e. drying condition and wet curing, were investigated. The details of RC beams are given in Fig. 3.

Table 5 Reinforced concrete specimens for shrinkage-induced stress test.

Series	Symbol	C ^{*1}	W/C	Tension reinforcement		f'_c ^{*4} (N/mm ²)	f_t ^{*5} (N/mm ²)
				Reinforcement	p_t (%)		
A	CFR-01-DS	d ^{*2}	0.60	2@D19	2.39	23.5	2.0
	CFR-01-WS	w ^{*3}	0.60	2@D19	2.39	23.5	2.3
	HCFR-01-DS	d	0.25	2@D19	2.39	68.1	2.3
	HV-01-DS	d	0.25	2@D19	2.39	68.7	3.0
B	VEX45-03-WS	w	0.45	2@D19	2.39	55.3	3.6
	CFREX45-03-W	w	0.45	2@D19	2.39	35.2	2.5

*1: Curing condition, *2: Drying condition, *3: Wet curing,

*4: Compressive strength, *5: Splitting tensile strength

Table 6 Mechanical properties of reinforcing bars.

	Type	Young's modulus	Yielding stress
		kN/mm ²	N/mm ²
Series A	D10SD295A	181.9	332
	D13SD295A	193.2	353
	D16SD295A	170.0	342
	D19SD345	171.5	359
Series B	D13SD295A	187.3	331
	D19SD345	187.3	356

The external force was loaded by tendon with screwing nuts, and the magnitude of the loads was measured by a load cell with 100 kN capacity installed between the nut and spring. The loaded force is equivalent to the force when the calculated stress in reinforcement is 100 N/mm² at the cracked section. Deflection at the span center was measured by a displacement transducer with a minimum graduation of 1/1000 mm. The average curvature of RC beams was determined from the displacement at the center of the pure moment zone on the assumption that deformation of the pure moment zone is a

circular arc. Chips for contact gauges, whose intervals are 40 mm, were attached on the side of the beam at the same depth as the reinforcement in order to measure the crack width. The minimum graduation of the contact-type strain gauge was 1/400 mm.

Ages at loading of the VC and CFR beams were 21 and 25 days in terms of temperature adjusted concrete age (CEB-FIP, 1990), respectively.

2.4 Analysis of time dependent behavior of RC beam

For the discussion of RC beams under sustained load, the numerical analysis proposed by Sato (Sato *et al.* 1992, Sato *et al.* 1998a) was applied. This analysis method is based on basic bond simultaneous equations expressed by Eq. (1) and Eq. (2), and the nonlinear bond stress-slip relationship shown in Table 7 (Muguruma *et al.*, 1967). The equations are formulated from the equilibrium requirements for axial force and bending moment at an arbitrary section between two cracks shown in Fig. 4, and the bond stress-slip relationship is determined by giving the bond strength and slip at bond strength. The effective Young's modulus method was adopted for the evaluation of creep of concrete, and the drying shrinkage effect was incorporated into basic equations. Bond creep was considered by increasing slip at bond strength, and the drop in bond strength in the vicinity of crack was also considered (Fig. 5) by decreasing the bond strength linearly toward the crack, whose length was 1.5 d_s (d_s : bar diameter).

Fig. 5 Drop in bond strength in vicinity of crack.

Fig. 3 Setup for sustained loading test.

The strain and stress distributions at an arbitrary section shown in **Fig. 4** will be determined if the tensile steel strain ε_{st} , concrete strain ε_{ct} at the same depth as the tensile steel, and stress related neutral axis depth y are given. Therefore, strains ε_{st} and ε_{ct} are obtained from two equilibrium equations for axial force and bending moment as a function of y . Differentiation of slip s with respect to x is given by the difference between tensile reinforcement strain and concrete strain at the same depth as the steel bar as follows:

$$\frac{ds}{dx} = \varepsilon_{st}(y) - \varepsilon_{ct}(y) = \frac{\{1 + G_{cr}(y)/G_c(y)\} \{M + \Delta M_{sc}(y) + \Delta M_{st}(y)\}}{E_e I_e(y)} (d - y) (1) + \frac{G_{ss}(y)}{G_c(y)}$$

The variables are described in Notation.

Further differentiating Eq. (1) with respect to x , substituting differentiations of ε_{st} and ε_{ct} into the equation and replacing $d\varepsilon_{st}/dx = (U_s/A_s E_s) \tau_b(s, x)$, we obtain the following basic differential equation:

$$\frac{d^2 s}{dx^2} = \left[1 + \frac{\frac{\partial}{\partial y} \left\{ \frac{G_{cr}(y)}{G_c(y)} \frac{M + \Delta M_{sc}(y) + \Delta M_{st}(y)}{E_e I_e(y)} (d - y) + \frac{G_{ss}(y)}{G_c(y)} \varepsilon_{cs} \right\}}{\frac{\partial}{\partial y} \left\{ \frac{M + \Delta M_{sc}(y) + \Delta M_{st}(y)}{E_e I_e(y)} (d - y) \right\}} \right] \frac{U_s}{A_s E_s} \tau_b(s, x) \quad (2)$$

As bond stress is a function of s and x , Eqs. (1) and (2) with variables s and y can be solved simultaneously under the boundary conditions; $s = 0$ at center section between cracks and $ds/dx = \varepsilon_{st} - \varepsilon_{ct} = \varepsilon_{st} - \varepsilon_{cs}$ at the crack. The strain and stress distribution at the crack was calculated considering concrete tension in accordance with a fictitious crack model in which tension softening stress-crack opening displacement was assumed to be linear. Iterative computation was carried out to obtain coincidence of the assumed crack width at the crack with that obtained by solving Eqs. (1) and (2).

Hence by using y and s , the strains of concrete and steel, and then, the stress of those as well as bond stress will be obtained.

Table 7 Model of bond for virgin concrete.

Bond stress-slip curve	$\tau_b(s, x) = \tau_{b, \max}(x) \cdot \exp \left(\frac{\ln \{ (e-1)s/s_{\max}(t, t_0) + 1 \}}{(e-1)s/s_{\max}(t, t_0) + 1} \right)$
Bond strength	$\tau_{b, \max} = 14.0 \text{ N/mm}^2$
Slip at $\tau_{b, \max}(x)$	$s_{\max}(t_0, t_0) = 0.24 \text{ mm}$
Drop in bond strength	$\tau_{b, \max}(x) = \frac{s_r/2 - x}{1.5d_s} \tau_{b, \max}, \{ (s_r/2 - 1.5d_s) \leq x \leq s_r/2 \}$
Creep coefficient of bond	$\phi_b(t, t_0) = \begin{cases} 0.32t & (0 \leq t \leq 1(\text{day})) \\ 0.32 + \frac{1.32(t-1)}{1.64(t-1) + 134} & (1(\text{day}) \leq t) \end{cases}$

Fig. 4 Schematic diagram of distributions of strain, stress and crack opening displacement in analysis (Sato *et al.* 1998).

3. Properties of recycled concrete

3.1 Compressive Strength, Young's modulus and splitting tensile strength

Figure 6 shows the compressive strength at 28 days of concrete in Series A under different curing conditions. Regardless of the curing condition, little decrease in compressive strength was observed in concrete produced from recycled coarse aggregate, compared with V-01. On the other hand, there was a 15-20 % decrease, regardless of the curing condition, in concrete produced from FR60 except for FR60 cured in 20°C water. In the case of CFR, about 20-45% smaller compressive strength was observed. The compressive strength of HCFR-01 was 7-23% smaller than that of HV-01 cured in 20°C water at 28 days.

In Fig. 7, the compressive strength of concrete at 28 days cured in 20°C water of Series A and Series B, and the lines obtained by least mean square approximation for CR and CFR data in Series B are plotted as a function of the cement-to-water ratio (C/W). All the types of RAC can be plotted linearly as a function of C/W. These results

Fig. 6 Compressive strength of recycled concrete compared with that of virgin concrete (Series A).

Fig. 7 Relationship between compressive strength of concrete at 28 days cured in 20°C water and cement-to-water ratio.

indicate that it is possible to control compressive strength by W/C, even though there may be a maximum compressive strength when the recycled aggregate is used in practical range of water-to-cement ratio.

Above a C/W of 3, it is generally known that the compressive strength of concrete is very sensitive to the properties of aggregate, and large differences can be seen between the compressive strengths of W/C = 0.3 and W/C = 0.25. However, this figure clearly shows that the compressive strengths of the concretes in Series B are comparable to those in Series A. This result indicates that the recycled aggregate of Series B, which was from the real market, has almost the same quality as the recycled aggregate of Series A. Differences in compressive strength between VC and CFR in Series A were much smaller than in Series B.

The smaller compressive strength of RAC is due to the large amount of porosity in recycled aggregate, which is capable of significant water absorption. From this point of view, compressive strength can be explained by the ratio of total water content in concrete to mass of cement (Fumoto *et al.* 2002). This concept makes it possible to evaluate the effect of the quality of recycled aggregate through absorbed water. Figure 8 shows the relationship between compressive strength at 28 days and cement and total water content ratio. The total water content (TW) is calculated using the percentage of absorption of each aggregate and mass of aggregate in concrete. It can be concluded from Fig. 8 that the compressive strength of the concrete with recycled aggregate can be evaluated by the cement-to-total water content ratio (C/TW), and the quality of the recycled aggregate used in Series B is comparable as that in Series A from the viewpoint of the relationship between the cement-to-total water ratio and compressive strength.

Young's modulus of concrete with recycled aggregate is smaller than that of virgin concrete due to the larger amount of cement paste in concrete. For RC design, the Young's modulus is generally evaluated from the compressive strength. Therefore, the experimental results of Young's modulus of concretes made of aggregates in both Series A and Series B are plotted as a function of

Fig. 8 Relationship between compressive strength of concrete at 28 days cured in 20°C water and cement-to-total water content ratio.

compressive strength in Fig. 9. Here, an equation of Young's modulus shown in Fig. 9 is obtained by regression analysis for values of Young's modulus depending on compressive strength given in JSCE code (JSCE 2002a). According to Fig. 9, the data of Young's modulus of CFR are 0.79 times the standard values, and those of CR are 0.85 times the standard values. Both CR and CFR show increases in Young's modulus with increases in compressive strength.

In Fig. 10, splitting tensile strength versus compressive strength of concretes made of aggregates in both Series A and Series B is shown with the standard value proposed by the JSCE (JSCE 2002b). Nearly all the tensile strength data of V are above the JSCE curve, while those of CR and CFR almost coincide with the JSCE curve with a small amount of scattering except for one case. Considering the variety of experimental data, 0.7 times the value of the JSCE was the minimum value of splitting tensile strength of RAC.

3.2 Volume change and shrinkage-induced stress of concrete

Figure 11 illustrates development of shrinkage strain under the drying condition and wet condition of each concrete with a W/C of 0.6. Also shown in this figure is the strain obtained from the code equation of the JSCE (JSCE 2002c). Shrinkage development varied in rate among V, CR, FR, and CFR, and was particularly significant for CFR. CFR showed about 1300 μ , which was almost twice as large as that of V-01, after 400 days under drying condition. FR60-01 showed close development to CR45-01 and CR60-01 at the period until 130 days and CR45-01 shows about 800 μ , which was about 1.3 times as large as that of V-01, after 1 year drying.

Figure 12 indicates the time-dependent autogenous shrinkage strain up to 28 days and shrinkage strain under drying condition after 28 days of HCFR-01 compared with HV-01. Autogenous shrinkage strain develops more rapidly and to a greater extent in HCRF-01 compared to HV-01 within the age of 1 day. This more rapid shrinkage is assumed to have resulted from the aggregate supplied water, which promoted hydration at a higher rate. Afterwards, continuous water supply from recycled aggregate decelerated the shrinkage strain to 40% of that of virgin concrete at the age of 28 days under sealed condition. Thus, it was confirmed that recycled aggregate has eminent properties for controlling autogenous shrinkage to a considerable degree. On the contrary, in the drying environment following seal removal, the whole shrinkage strain in HCFR-01 developed to a large extent and reached almost the same shrinkage strain as HV-01 after 100 days' drying. This appears to have resulted from the fact that the larger amount of porosity caused by water supply from recycled aggregate, and the large amount of

Fig. 9 Young's modulus of concrete as function of compressive strength.

(*: Equation of Young's modulus is obtained by regression analysis for values of Young's modulus given in JSCE code (JSCE 2002a))

Fig. 10 Relationship between splitting tensile strength and compressive strength.

Fig. 11 Comparison between virgin and recycled concretes with W/C of 0.6 for drying shrinkage strain.

Fig. 12 Comparison between virgin and recycled high-strength concretes with W/C of 0.25 for shrinkage strain.

porosity in the old mortar in the recycled aggregate facilitate moisture transport.

Figure 13 shows the specific creep of CFR-01 and V-01 under drying condition and wet condition. According to the figure, specific creep under wet condition was 62μ in the case loaded at the age of 7 days, 44μ at 28 days, and 38μ at 180 days after 50-day-loading, which means that creep development decreases as the age increases. Specific creep under drying, however, did not always become smaller in proportion to the passage of time. In **Fig. 13**, drying specific creep after 230-day loading of CFR-01, whose loading age was 28 days, shows almost the same values as those of specimens loaded at the age of 7 days. The drying specific creep of CFR-01 at 180 days shows relatively small values at 200 days after loading compared with those of others.

Figure 14 illustrates shrinkage induced stress of concrete at the bottom fiber of RC beams made of HV-01-DS, HCFR-01-DS, CFR-01-DS, CFR-01-WS, VEX-45-03WB, CFREX45-03-WB, VEX45-03-WS, and CFREX45-03-WS, which were calculated using measured strains in tension reinforcements. Autogenous shrinkage stress in HCFR-01-DS was larger than that of HV-01-DS in response to rapid shrinkage development at very early ages before 1 day. However the shrinkage-induced stress of HCFR-01-DS was reduced to be nearly half that of HV-01-DS at the age of 28 days, while shrinkage strain of HCFR-01 developed rapidly and the rate of shrinkage development of HCFR-01 was about

twice as high as that of HV-01 after drying started, shrinkage-induced stress in HCFR-01-DS was produced with approximately the same rate as that in HV-01-DS and, consequently, the stress in HCFR-01-DS resulted in 70% of the stress in HV-01-DS at 30 days after drying. The reason for this phenomenon may be explained by the smaller Young's modulus and the larger creep of RAC.

When expansive additive is used, the effect of the additive depends on the reinforcement ratio. In a beam with a reinforcement ratio of 1.06%, stored compressive stress in recycled concrete was almost the same as that of virgin concrete at 50 days, while the stress in concrete differed under the reinforcement ratio of 2.39%. This is attributable to the smaller Young's modulus, larger creep, and nonlinear characteristic of creep of recycled concrete with expansive additive.

3.3 Fracture energy

Figure 15 represents the typical experimental results of the load-COD relationship of V-01 and CFR-01 tested at 28 days. Additionally, each of the two figures presents the calculated averaged fracture energies of V-01 and CFR-01 of 5 specimens at 3 and 28 days according to polylinear tension softening analysis (Kitsutaka 1998), as well as the calculated fracture energy from the compressive strength at 28 days according to the JSCE code (JSCE, 2002d) and data from a technical committee report of JCI (JCI 1993). The experimental results, which are well known to have marked variety, are considerably

Fig. 13 Comparison between virgin and recycled concretes with W/C of 0.6 for specific creep under wet and drying conditions.

Fig. 14 Comparison between virgin and recycled concretes with W/C of 0.6, 0.45 and 0.25 for stress in concrete at extreme bottom fiber in RC beam specimen due to shrinkage. (Left: Effect of autogenous shrinkage and drying shrinkage, Right: Effect of expansive additive).

Fig. 15 Comparison between virgin and recycled concretes with W/C of 0.6 for typical tension softening curve at 28 days, and fracture energy.

larger than those of the JSCE, but the ratio of fracture energy of CFR-01 to those of V-01 is about 0.7, while 0.75 was reported when the concrete is made with coarse recycled aggregate by Kunieda (Kunieda *et al.* 1999).

4. Flexural behavior of RC beams made of recycled aggregate concrete

4.1 Instantaneous behavior (Sato *et al.* 1998b, and 1999)

Table 8 summarizes the flexural properties under serviceability condition including the deflection of beams when the stress in the tension rebar is 200 N/mm², and the cracking moment (M_{cr}), average and maximum crack width (w_{av} , w_{max}) and spacing (l_{av} , l_{max}) of cracking, and cracking moment are calculated using the average splitting tensile strength of concrete during the loading test of RC beams. This table lists the predicted values of the deflection when the stress in the tension rebar is 200 N/mm² in the RC section evaluated with the following effective moment of inertia of transformed cross section I_e proposed by Branson (Branson, 1963):

$$I_e = \left(\frac{M_{crd}}{M_{dmax}} \right)^3 I_g + \left\{ 1 - \left(\frac{M_{crd}}{M_{dmax}} \right)^3 \right\} I_{cr} \quad (3)$$

where M_{crd} is the critical flexural moment when a flexural crack occurs in the cross section, M_d is the design flexural moment to be used in the computation of displacement and deformation, M_{dmax} is the maximum value of design flexural moment to be used in the computation of displacement and deformation, I_g is the moment of inertia of gross cross section about its centroid, and I_{cr} is the moment of inertia of cracked cross-section around its centroid.

Additionally, the predicted values of l_{max} and w_{max} obtained with the following JSCE code equation (JSCE 2002e) are also presented:

$$l_{max} = 1.1k_1k_2k_3 \{ 4c + 0.7(c_s - \phi_{RB}) \} \quad (4)$$

$$w_{max} = l_{max} (\sigma_{se} / E_s + \varepsilon'_{csd}) \quad (5)$$

where l_{max} is the crack spacing, k_1 (=1.3) is a coefficient for the effect of surface shape of RB, k_2 is a coefficient for an effect of concrete properties, calculated by: $k_2 = 15 / (f'_c + 20) + 0.7$ where f'_c is the compressive strength in positive value,

k_3 is a coefficient for the number of layers of tension RB (n), which is calculated by $k_3 = 5(n+2)/(7n+8)$, c is the cover (mm), c_s is the distance between centroids of RBs, ϕ_{RB} is the nominal diameter of RB, w_{max} is the maximum crack width, σ_{se} is the increment of stress in RB from the state in which concrete stress at the depth of RB is zero, E_s is Young's modulus of RB, and ε'_{csd} is a design value for the effect of shrinkage and creep in concrete on crack width increases and set to 150×10^{-6} in general cases.

Figure 17 illustrates how recycled aggregates typically affect deflection of RC beams with a tension reinforcement ratio of 1.06% cured under drying condition and under wet condition, respectively. This figure shows the relationship between bending moment and deflection until rebar yielding. Deflections of recycled concrete beams under both wet and drying conditions are obviously larger compared to those of virgin concrete beams and the former exceed the calculated values at crack. This is attributable to the lower Young's modulus as well as lower bond stiffness of recycled concrete. Further, markedly larger deflection of recycled concrete beam may be due to bond deterioration owing to shrinkage induced micro-cracks.

Figure 18 shows the experimental data on deflection when the stress in the tension rebar is 200 N/mm² in the RC section compared with the values calculated with Eq. (5). This figure indicates that the deflections when the stress of the rebar is 200 N/mm² of RC beams with RAC as well as VC can be evaluated with Branson's equation as well as Young's modulus of the concrete, and that the deflections of RC beams with RAC are larger than those of VC, while they are predictable by the same equation.

In Fig. 19, the ratio of $l_{\max}$ observed in RAC beams to $l_{\max}$ in VC beams at the rebar stress of 200 N/mm^2 shows a range of 0.92-1.37 in the case of CR and a range of 0.74-1.26 in the case of CFR, which indicates that the type of recycled aggregate has almost no influence on

crack spacing. Additionally, Fig. 20 shows that the ratio of $w_{\max}$ measured in RAC beams to $w_{\max}$ in VC beams ranges from 0.57 to 1.3 in the case of CR and from 1.1 to 1.7 in the case of CFR, while the same ratio for $W/C = 0.6$ in Series B was an exceptionally high 1.70. **Table 8**

Table 8 Characteristics of crack and deflection under serviceability condition.

Symbol	C	M_{cr} (kNm)		l_{av} (mm)	$l_{\max}$ (mm)		w_{av} (mm)	$w_{\max}$ (mm)		δ_{200} (mm)		
		Exp.	Cal.	Exp.	Exp.	Cal.J	Exp.	Exp.	Cal.J	Exp.	Cal.J	Cal.R
V-01-10WB	w	1.7	3.0	146	184	207	0.03	0.08	0.24	1.8	1.8	4.8
V-01-10DB	d	2.1	3.1	173	198	205	0.05	0.14	0.23	2.7	1.7	4.8
CR45-01-10WB	w	2.8	2.7	159	213	207	0.04	0.09	0.24	2.0	2.3	4.9
CR45-01-10DB	d	1.7	3.0	140	187	210	0.06	0.11	0.24	3.0	2.0	4.9
CR60-01-10WB	w	2.7	3.4	151	189	203	0.04	0.09	0.23	1.5	1.4	4.8
CR60-01-10DB	d	2.1	2.8	107	172	206	0.04	0.08	0.23	2.0	2.1	4.8
V-01-13WB	w	3.6	3.1	118	137	200	0.07	0.12	0.22	3.7	4.3	5.1
V-01-13DB	d	2.1	3.2	102	125	198	0.08	0.12	0.22	4.6	4.2	5.2
CR45-01-13WB	w	2.8	2.9	126	165	200	0.06	0.12	0.22	4.4	4.7	5.3
CR45-01-13DB	d	1.9	3.1	122	148	203	0.08	0.13	0.23	4.9	4.5	5.3
CR60-01-13WB	w	2.0	2.9	113	163	196	0.07	0.11	0.22	4.0	4.5	5.2
CR60-01-13DB	d	1.4	3.5	87	131	198	0.08	0.11	0.22	4.1	4.0	5.2
FR60-01-13WB	w	2.0	2.7	114	169	208	0.07	0.13	0.23	4.4	4.5	5.1
FR60-01-13DB	d	-	2.3	120	164	209	0.09	0.14	0.23	5.1	5.0	5.3
CFR-01-13WB	w	1.5	2.7	113	173	210	0.09	0.15	0.23	5.1	4.6	5.2
CFR-01-13DB	d	1.0	1.7	82	138	203	0.09	0.14	0.26	5.6	5.4	6.0
HV-01-13DB	d	3.3	1.3	114	139	174	0.10	0.13	0.31	3.7	4.9	6.2
HCFR-01-13DB	d	2.2	1.1	94	162	175	0.09	0.14	0.29	4.6	5.0	6.0
V-01-16WB	w	3.4	3.2	122	141	192	0.10	0.15	0.23	4.7	5.9	6.1
V-01-16DB	d	1.6	3.3	117	129	190	0.10	0.15	0.22	6.4	5.9	6.2
CR45-01-16WB	w	3.2	3.0	109	125	193	0.07	0.11	0.23	5.7	6.2	6.4
CR45-01-16DB	d	1.9	3.3	102	119	195	0.10	0.13	0.23	6.7	6.1	6.4
CR60-01-16WB	w	1.3	3.0	120	157	188	0.09	0.13	0.22	4.9	6.0	6.2
CR60-01-16DB	d	1.8	3.7	91	125	191	0.07	0.10	0.22	5.4	5.8	6.2
V30-03-WB	w	7.2	4.6	149	201	164	0.03	0.09	0.21	1.4	1.6	5.2
CR30-03-WB	w	4.5	4.3	141	217	174	0.05	0.09	0.21	2.0	2.4	5.1
CFR30-03-WB	w	3.7	3.6	114	164	181	0.07	0.12	0.21	3.4	3.6	5.2
V45-03-WB	w	4.3	4.8	145	205	179	0.05	0.12	0.20	2.6	2.3	4.8
CR45-03-WB	w	3.5	3.8	102	153	186	0.06	0.10	0.21	2.7	3.4	5.0
CFR45-03-WB	w	2.1	2.5	101	139	194	0.09	0.14	0.22	4.5	4.6	5.2
V60-03-WB	w	3.3	4.1	123	157	190	0.05	0.09	0.21	2.9	3.0	4.9
CR60-03-WB	w	2.7	2.9	99	139	194	0.10	0.13	0.22	4.1	4.3	5.4
CFR60-03-WB	w	2.4	2.9	130	154	201	0.10	0.15	0.23	5.1	4.6	5.3
VEX45-03-WB	w	4.5	5.0	127	183	180	0.04	0.07	0.16	1.9	2.8	4.1
CREX45-03-WB	w	4.5	4.4	107	181	186	0.04	0.08	0.16	2.5	3.7	4.3
CFREX45-03-WB	w	3.6	4.3	100	134	195	0.07	0.10	0.17	3.7	4.2	4.4

l , w , δ : Spacing, width of cracking and deflection at $\sigma_s=200 \text{ N/mm}^2$, σ_s : Tension stress of tension reinforcement, av., max.: Average and maximum values, Exp.: Experimental values, Cal.J ($l_{\max}$ and $w_{\max}$): Calculated values by the JSCE's equation, (JSCE 2002d), Cal.J (δ_{200}): Calculated values by Branson's equation (Branson, 1963), Cal.R: Calculation assuming flexural stiffness is equivalent to that of cracked section, M_{cr} : Flexural cracking moment

Fig. 17 Comparison between virgin and recycled concrete beams with W/C of 0.6 for typical relationship between bending moment and deflection under drying and wet conditions.

Fig. 18 Experimental deflection at $\sigma_s = 200 \text{ N/mm}^2$ in RC section versus calculated deflection by Branson's equation.

Fig. 19 Effect of recycled aggregate and curing condition on maximum crack spacing of $\sigma_s = 200 \text{ N/mm}^2$. (Values on bars indicate ratio to value of reference virgin concrete beam).

Fig. 20 Effect of recycled aggregate and curing condition on maximum crack width of $\sigma_s = 200 \text{ N/mm}^2$. (Values on bars indicate ratio to value of reference virgin concrete beam).

includes the $l_{\max}$ and $w_{\max}$ values calculated by the JSCE method (JSCE 2002e), in which shrinkage induced strain in tension reinforcing bars measured at loading (and released at cracking) is incorporated into calculations. Based on this table, it is concluded that experimental values of $w_{\max}$ are considerably smaller than calculated values independent of the difference of the aggregate, and therefore, specifying the quality of recycled aggregate used in this test, the JSCE's equation estimates experimental values in a conservative manner.

4.2 Time-dependent behavior

Table 9 lists the average curvature of the pure moment zone under instantaneous loading until σ_s of the reinforcing bar reaches 100 N/mm^2 in the RC section of V-01-13DB, V-01-13WB, CFR-01-13DB, and CFR-01-13WB, and the average crack spacing and maximum crack spacing. These beams were used for testing of long-time behavior under sustained load, as shown in Fig. 3. The beams under wet condition did not crack until σ_s of the reinforcing bar reached 100 N/mm^2 in the RC section. On the other hand, V-01-13DB and CFR-01-13DB beams under drying condition cracked at 1.9 kNm and 1.0 kNm, respectively.

The average curvature of CFR-01-13DB just after the application of instantaneous bending moment, whose magnitude is 3.5 kNm and corresponds to 100 N/mm^2 in terms of tension rebar stress, was about 2 times as much as that of V-01-13DB, and in the case of wet condition, the average curvature of CFR-01-13WB was about 25% larger than that of V-01-13WB.

The average flexural crack spacing of V-01 is about 1.6 times that of CFR-01. This is due to the larger tensile strength and bond stiffness of VC compared to RAC.

The time-dependent average curvatures of RC beams, which were measured for about 1 year, are plotted in Fig. 21. This figure shows that there was no increase in beams under wet condition. This phenomenon could be explained by the smaller creep and drying shrinkage (Figs. 11 and 13). After the 1-year loading, the increase in average curvature of CFR-01 after the application of a sustained load was about 1.8 times that of V-01. Figure 21 plots the results of the analysis, which is for predicting long-term deformations and crack width in reinforced concrete flexural members under sustained load, comparing them with the experimental results. The experimental data of CFR-01 and V-01, i.e. Young's modulus of concrete (18.1 and 24.4 kN/mm^2), and average crack spacing (91 and 147 mm, as listed in Table 9) were used for this analysis as input values, respectively. Additionally the drying shrinkage shown in Fig. 11 and the specific creep at the loading age of 28 days in Fig. 13 were also used in the computation. The fracture energies adopted in the computation for behavior at the cracked section were 100 N/m for V-01 and 65 N/m for CFR-01, respectively. These values were assumed by considering experimental data, values calculated by the JSCE code method (JSCE, 2002d), and data shown in a technical

Table 9 Comparison between virgin and recycled concretes beams with W/C of 0.6 for average curvature and crack spacing at $\sigma_s=100$ N/mm² in RC section under drying and wet conditions.

Symbol	Average curvature (μ /mm)	l_{av} (mm) ^{*1}	l_{max} (mm) ^{*2}
V-01-13DB	2.17	147	192
V-01-13WB	1.16	- ^{*3}	-
CFR-01-13DB	4.46	91.2	130
CFR-01-13WB	1.72	-	-

*1: Average crack spacing, *2: Maximum crack spacing,

*3: No crack observed

Fig. 21 Comparisons between CFR-01 and V-01 as well as experiment and computation for average curvature.

committee report of JCI (JCI 1993) in which the minimum ratio of fracture energy of CFR to V was 0.65 in the experiment.

The comparison between the experiment and analysis showed that the analysis underestimated the experiment after about 130 days for both beams, while the former offered a very high prediction accuracy before 130 days. One of the reasons for this discrepancy may be attributable to the adoption of the effective Young's modulus method for estimating concrete creep, because higher stress in concrete at an earlier elapsed time after loading cannot be estimated by this method.

Figure 22 shows that the measured crack width over time of CFR-01 is larger than that of V-01 though the crack spacing of CFR-01 is smaller than that of V-01 (**Table 9**). However, the increasing tendency of crack width of CFR-01 is similar to that of V-01 and their increased values at about 300 days after the application of a sustained load were both 0.06 mm. The measured crack widths of CFR-01 were remarkably larger than the computed ones as shown in the same figure. This is likely due primarily to the fact that the bond characteristics such as bond strength and bond slip used for CFR-01 were assumed to be the same as those of V-01 while the former should be inferior to the latter. Modeling of the bond behavior for RAC is required to improve the prediction accuracy.

Fig. 22 Comparisons between CFR-01 and V-01 for maximum and average crack widths as well as experiment and computation for average crack width.

4.3 Strength, plastic deflection and ultimate moment of RC beams

Figure 23 shows typical examples of crack patterns at the failure observed in drying-cured flexural beams CFR-01-13DB, HCFR-01-13DB with a reinforcement ratio of 1.06% made of both recycled fine and coarse aggregates, compared with those of drying-cured flexural beams made of virgin concrete. The solid lines indicate cracks observed before yielding and the dotted lines crack observed after yielding, respectively. No noticeable difference in crack pattern between reinforced recycled and virgin concrete beams can be observed. All the beams showed flexural failure except for the HV-01 and HCFR-01 beams, which showed flexural-shear failure as shown in **Fig. 23**.

Typical examples of the effects of the recycled aggregate on the plastic deflection of beams with a reinforcement ratio of 1.06% are shown for the drying-cured case in **Fig. 24**. As expected, recycled aggregate did not affect ultimate flexural deflection following tension reinforcement yielding.

To estimate the performance of plastic deformation of reinforced recycled concrete beams, ductility factors obtained from all the beams are summarized in **Table 10**, in which ductility factors were defined as the rate of deflection just before sudden drop in applied load to deflection at yielding of steel bars. The yielding moment, deflection at yielding, ultimate moment at maximum, and deflection just before sudden drop in moment obtained from all beams, are also tabulated in this table. **Figure 25** shows a comparison of the ductility factors of RAC beams with those of VC in Series A. This figure indicates that the ductility factor never decreased even when RAC of normal strength was used, and that the ductility factor can be increased when RAC strengthened by a lower W/C is used.

The ultimate moments (M_u) calculated by the JSCE's code method (JSCE 2002e), in which the measured yielding stress of reinforcing bars is applied, are compared with those obtained by measurement in **Fig. 26**. The stress-strain of concrete in the JSCE code is com-

Fig. 23 Crack patterns of flexural beams.

Fig. 24 Bending moment - deflection relationship after yielding of beam with drying curing.

posed of parabola up to 2000×10^{-6} and then perfect plasticity up to 3500×10^{-6} at extreme compression fiber.

According to this figure, experimentally obtained ultimate moments are equal or greater than the ones obtained through calculation. Therefore, it is concluded that flexural capacity does not decrease through the use of recycled aggregate if the anchorage length is sufficient to pull the reinforcing bar up to failure and the steel bars yield before failure of concrete in compression.

5. Conclusions

The physical properties of recycled aggregate concrete (RAC) and the flexural behavior of RC beams consisting of said RAC were experimentally investigated to evaluate the applicability of recycled aggregate concrete to reinforced concrete members. Two kinds of recycled aggregates were used: one was obtained from the same natural aggregate as that in the reference virgin concrete (VC), in order to figure out the effects of recycled aggregate on the physical properties and flexural behavior of plain and reinforced recycled aggregate concretes. Considering the nature of aggregates available on the real market, the other was recycled aggregate from actual buildings, in which case the properties of the original aggregate is unclear.

Fig. 25 Comparison of ductility factors between reference concrete beam and recycled aggregate concrete beam.

Fig. 26 Comparison between experiment and calculation for ultimate moment M_u .

The following conclusions were drawn within the limit of the present study.

(1) Compressive strength

i) In Series A, the compressive strength of concretes with coarse recycled aggregate (CRC) was greater than 0.9 times that of the reference concrete when $W/C = 0.6$, which means that the effect of coarse recycled aggregate on the compressive strength of recycled aggregate concrete is small. On the other hand, the compressive strength of concretes with coarse and fine recycled aggregates (CFRC) was about 0.55-0.80 times that of reference concrete. This result indicates that the effect of fine recycled aggregate on the compressive strength is large. In the case of high-strength concrete ($W/C = 0.25$), the ratio of compressive strength of CFRC to that of VC was about 0.77-0.93.

ii) According to the experiment of Series A and B, the compressive strengths of CRC as well as CFRC had a linear relationship with the cement-to-water ratio irrespective of the type of the original aggregate and original concrete. This tendency is the same as that of VC and

Table 10 Strength and plastic deflection of flexural beams.

Symbol	M_y (kNm)		δ_y (mm)			M_u (kNm)		δ_u (mm)	δ_u/δ_y
	Exp.	Cal.J	Exp.	Cal.J	Cal.R	Exp.	Cal.J	Exp.	
V-01-10WB	7.4	6.9	6.2	6.0	8.1	8.0	7.3	94.2	15.2
V-01-10DB	7.7	6.9	7.4	5.9	8.1	9.1	7.3	164.4	22.2
CR45-01-10WB	7.4	6.9	6.4	6.7	8.3	8.5	7.3	94	14.7
CR45-01-10DB	7.1	6.9	7.8	6.3	8.3	8.9	7.3	106.9	13.7
CR60-01-10WB	7.7	6.9	6.7	5.5	8.1	9.3	7.3	98.9	14.8
CR60-01-10DB	7.8	6.9	7.6	6.4	8.1	9.5	7.3	116.4	15.3
V-01-13WB	13.5	12.7	8.9	8.9	9.2	13.7	13.2	49.8	5.6
V-01-13DB	13.2	12.7	9.3	9.0	9.3	14.0	13.3	80.1	8.6
CR45-01-13WB	12.9	12.6	8.5	9.3	9.6	13.9	13.2	52.8	6.2
CR45-01-13DB	13.2	12.6	10.1	9.3	9.6	14.1	13.2	75.8	7.5
CR60-01-13WB	12.5	12.7	8.6	9.1	9.3	14.1	13.4	72.2	8.4
CR60-01-13DB	13.4	12.7	10.1	8.9	9.3	15.1	13.3	73.3	7.3
FR60-01-13WB	12.5	12.7	9.7	9.0	9.2	13.1	13.0	44.5	4.6
FR60-01-13DB	11.9	12.6	9.6	9.4	9.6	13.7	12.9	65.2	6.8
CFR-01-13WB	12.4	11.9	10.4	8.6	8.8	14.1	12.3	70.1	6.7
CFR-01-13DB	12.1	11.9	11.1	9.2	9.7	13.9	12.6	70.1	6.3
HV-01-13DB	13.6	12.7	9.7	8.3	9.5	16.3	13.2	102.2	10.5
HCFR-01-13DB	12.9	12.6	10.0	8.5	9.4	16.4	13.2	131.2	13.1
V-01-16WB	18.9	18.9	10.6	10.6	10.7	19.4	19.3	34.8	3.3
V-01-16DB	19.3	18.9	11.2	10.7	10.8	19.5	19.4	45.2	4.0
CR45-01-16WB	18.9	18.8	10.3	11.1	11.2	19.2	19.3	40.4	3.9
CR45-01-16DB	18.9	18.8	12.2	11.1	11.2	19.5	19.1	44.7	3.7
CR60-01-16WB	18.8	18.9	11.1	10.7	10.8	19.9	19.6	38.0	3.4
CR60-01-16DB	19.7	18.9	11.2	10.7	10.8	21.9	19.4	63.7	5.7
V30-03-WB	13.2	12.3	6.9	5.7	8.3	15.6	13.2	84.8	12.3
CR30-03-WB	12.5	12.1	7.5	6.8	8.4	15.3	13.0	76.4	10.2
CFR30-03-WB	13.0	12.0	8.2	7.9	8.7	15.4	12.9	86.8	10.6
V45-03-WB	12.9	12.2	7.4	6.5	8.0	15.0	12.9	81.8	11.1
CR45-03-WB	13.2	12.0	7.7	7.7	8.4	14.8	12.8	72.0	9.4
CFR45-03-WB	12.6	11.9	9.5	8.5	8.7	13.7	12.6	67.1	7.1
V60-03-WB	11.9	12.1	7.1	7.3	8.2	15.8	12.7	101	14.2
CR60-03-WB	12.8	11.9	9.2	9.4	9.5	15.3	12.5	86.8	9.4
CFR60-03-WB	12.2	11.8	9.2	8.7	9.0	14.1	12.5	59.6	6.5
VEX45-03-WB	13.6	12.2	6.8	7.0	7.3	15.3	12.9	100.5	14.8
CREX45-03-WB	13.0	12.1	7.4	7.8	7.7	15.1	12.8	95.7	12.9
CFREX45-03-WB	11.8	11.9	8.6	8.4	8.0	13.5	12.6	61.4	7.1

δ_y : Deflection at tension reinforcing bars yielding, δ_u : Deflection just before sudden drop in moment, Exp.: Experimental values; Cal.J: Calculated values by JSCE code, Cal.R: Calculated values assuming flexural stiffness is equivalent to that of cracked section, M_y : Yielding moment, M_u : Ultimate moment. δ_u/δ_y : Ductility factor

accounts for the fact that the compressive strength of RAC is controllable with the cement-to-water ratio. From the present study, $f'_c = 4.23 + 18.1(C/W)$ for CRC, and $f'_c = 3.42 + 13.9(C/W)$ for CFRC were obtained.

(2) Young's modulus

iii) It was found from Series A and B that the curves of Young's modulus as a function of compressive strength of RAC were similar to that of VC. This proves that the Young's modulus of RAC can be evaluated with com-

pressive strength. The standard data set of Young's modulus and compressive strength proposed by JSCE was applicable with reduction coefficients of 0.85 and 0.79 in the case of CRC and CFRC, respectively.

(3) Splitting tensile strength

iv) It was found from Series A and B that a curve of splitting tensile strength as a function of compressive strength of RAC can be evaluated by JSCE code, while CFRC with $W/C = 0.25$ shows exceptionally small values.

(4) Volume change

v) From the drying shrinkage test in Series A, the ratios of drying shrinkage strain after 1-year-drying of CRC and CFRC to that of VC were 1.3 and 2.5, respectively. From these experimental results, shrinkage-induced cracking and resultant jeopardized durability can be problems for CFRC. These are a matter for further study. Drying shrinkage strain of CRC can be evaluated by JSCE code when total water content including absorbed water in recycled aggregate is taken into account. Prediction of drying shrinkage strain of CFRC is also an issue for the future.

vi) Specific creep of CFRC (in Series A) in wet and drying conditions was about 1.5 and 2.5 times that of VC after 1-year-loading. The specific creep of CFRC in wet condition at the loading age of 7 days was about 150×10^{-6} after 1-year-loading, which will not become problem.

(5) Flexural behavior

vii) Deflections of RC beams with RAC (in Series A and B) were larger than those with VC under the conditions of same moment and same water-to-cement ratio. Branson's equation satisfactorily evaluated the deflections of RC beams using RAC irrespective of the type of original aggregate and original concrete when stress in the tension rebar is 200 N/mm^2 in the RC section, while it tended to underestimate the deflection as moment increased. This phenomenon can be explained by the smaller bond stiffness of RAC.

viii) Crack spacing of CRC and CFRC (in Series A and B) was 0.92-1.37 times and 0.74-1.26 times that of VC, and there was no significant difference in crack spacing between RC beams with VC and those with RAC. Crack spacing in drying condition was smaller than that in wet condition.

ix) The crack width of RC beams with CRC and CFRC (in Series A and B) was 0.57-1.3 times and 1.1-1.7 times that of VC, respectively, and the crack width in RC beams with RAC was greater than that of RC beams with VC. These values, however, were not larger than the predicted values by JSCE code. On the condition that the tension rebar is 200 N/mm^2 in RC section, the crack width in an RC beam with RAC cannot be a problem for durability.

x) Under the sustained bending moment, which was equivalent to 100 N/mm^2 in tension rebar stress in the RC section, RC beams with CFRC under wet condition did not show cracking nor an increase in deflection for 1 year. But in drying condition, RC beams with CFRC developed many cracks as well as an increase in deflection. The increased deflection of RC beams with CFRC was 2 times that of RC beams with VC. The crack width of RC beams with CFRC under sustained loading was not different from that of RC beams with VC, and the increase in crack width was also the same as that of RC beams with VC, 0.06 mm after 1-year-loading. According to these results, the application of RAC to RC with severe deflection requirement needs particular countermeasures, such as no use of fine recycled aggregate, restriction of

occupancy rate of recycle aggregate, and use of compression rebars.

xi) The ductility factors of RC beams with RAC (in Series A and B) were almost the same as those of RC beams with VC.

xii) The ultimate moment of RC beams with RAC (in Series A and B) was almost the same as that of RC beams with VC on the condition that their concrete has the same water-to-cement ratio, and yielding of rebar precedes failure of concrete in compression. Further, the ultimate moment can be predicted by using JSCE code irrespective of the type of original aggregate and original concrete.

(6) Applicability to a structural concrete

Recycled concretes used in the present study were lower in compressive strength and Young's modulus, and larger in creep and drying shrinkage compared to the corresponding virgin concretes. However, all the recycled concretes had a compressive strength higher than the value of 21 N/mm^2 except for the case of FR60-01 in Fig. 6, while having a water-to-cement ratio of 0.6. Moreover, the compressive strength depends linearly on the cement-to-water ratio, and Young's modulus and the splitting tensile strength can be expressed by a function of the compressive strength. Therefore, it is possible to design the mixture proportions of recycled concretes in the same way as conventional concrete whose compressive strength exceeds the minimum characteristic strength of 18 N/mm^2 specified for structural concrete by JSCE, JIS and so on.

In the case of reinforced flexural beams, instantaneous and long-term deformations of the reinforced recycled concrete beams were larger than those of the reference beams, while instantaneous and long-term crack widths of the former were not significantly larger than those of the latter. These characteristics of the recycled concrete beams could be predicted by the previous methods with partial modifications. Furthermore, the effect of recycled aggregate on the deterioration of ultimate bending moment and ductility was hardly observed because the beams failed in flexure or in flexural-shear after yielding of tension reinforcing bars, and thus the ultimate bending moment can be predicted by a conventional method.

Based on the above facts, it is concluded that recycled aggregates having properties equal to or superior to those of the recycled aggregates used in the present study may be applied to structural concrete from a mechanical point of view, while deflection must be controlled by considering the physical properties of recycled aggregate concrete if necessary.

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Notation

M : Applied sustained bending moment
 $\Delta M_{sc}(y)$: $\{y_{ct} - (d' - y)\} A_s E_s \varepsilon_{cs}$
 $\Delta M_{st}(y)$: $\{y_{ct} - (d - y)\} A_s E_s \varepsilon_{cs}$
 d, d' : Distance from extreme compressive fiber to tensile and compressive reinforcing bar
 ε_{cs} : Drying shrinkage strain
 $I_e(y)$: $I_{cr}(y) - G_{cr}(y)y_{ct}$, $y_{ct} = I_c(y)/G_c(y)$
 $I_{cr}(y), G_{cr}(y)$: $I_c(y) + n_e I_s(y) + n_e I_s(y)$,
 $G_c(y) + n_e G_s(y) + n_e G_s(y)$
 $I_c(y), I_s(y)$: Second moments of area of concrete and steel in compression zone about N. A.
 $I_c(y), I_s(y)$: Second moments of area of concrete and steel in tension zone about N. A.
 $G_c(y), G_s(y)$: First moments of area of concrete and steel in compression zone about N. A.
 $G_c(y), G_s(y)$: First moments of area of concrete and steel in tension zone about N. A.
 A_s, A_s : Areas of reinforcements in compression and tension zone
 $E_s, E_c(t_0)$: Elastic modulus of reinforcement and concrete at the age t_0
 E_e, n_e : $E_c(t_0)/\{1 + \phi(t, t_0)\}$, E_s/E_e
 $\phi(t, t_0)$: Creep coefficient of concrete age t for loading at age t_0
 $G_{ss}(y)$: $n_e(A_s + A_s)(d - y)$
 $\tau_b(s, x), U_s$: Bond stress, perimeter of tension reinforcement
 f_{ct} : Tensile strength of concrete
 $w(z)$: $(h - z - y - y_{ce})w_s/(d - y - y_{ce})$, h : height of RC member
 w_s : Crack opening displacement at the depth of tension reinforcement
 G_f, U_{ult} : Fracture energy, $2G_f/f_{ct}$
 s_r : L length of element

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