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Study on Time-Dependent Behavior of RC Beams with Flexural Cracks Generated at Early Age

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Abstract

In this research, to clarify the time-dependent deformational behavior of RC beams with flexural cracks generated at an early age, continuous flexural loading of RC beams was conducted, and the effects of shrinkage property, age of cracking, and environmental conditions were experimentally investigated. A calculation method of total crack width utilizing the results of numerical simulation was proposed. Using this method, tensile deformation of RC beams at an early age was analyzed.

The results of this study demonstrated that compressive deformation of concrete due to drying creep causes a significant increase in the tensile stress of reinforcement. Especially under high compressive stress/strength ratio, large creep deformation, which might be due to micro-cracks generated at ITZ between the cement paste and aggregate, should be properly considered in numerical simulation. Furthermore, deterioration of the bond between compressive reinforcement and the surrounding concrete might be another reason for the large creep deformation.

Moreover, it was assumed that concrete between flexural cracks generated at a very early age could deform with less bond deterioration following the deformation of tensile reinforcement. This might be due to the generation of micro-cracks at ITZ between the cement paste and aggregate, which might prevent internal cracks around reinforcement leading to deterioration of the bond.

1. Introduction

Recently, in Japan, instances of severe damage to concrete structures such as a large numbers of cracks due to excessive shrinkage of concrete has been reported (JSCE 2008).

However, the time-dependent behaviors of RC structures with cracks generated at an early age have not been clarified sufficiently. Therefore, this research investigates the time-dependent behavior of RC beams with flexural cracks generated at an early age through various experiments examining the influences of the shrinkage property of concrete, age of cracking, and environmental conditions on the time-dependent behavior of RC beams.

Further, through FEM numerical simulation, the effects of creep deformation of concrete in compression on the time-dependent behavior of RC beams are analyzed. A new methodology to calculate total flexural crack width utilizing FEM simulation results is proposed, and the behavior of early age concrete in tension is analyzed through comparison of calculated results and experimental results in terms of total crack width.

2. FEM simulation system used in this research

2.1 Basic scheme (Maekawa *et al.* 2008)

In this research, an FEM simulation system integrating microscopic thermo-hydro physics and macroscopic nonlinear mechanics is utilized for investigating the time-dependent behavior of RC beams (**Fig. 1**). The responses are successively updated along the time history in the software considering the link between the microscopic characteristics of concrete composites and structural mechanics and damage induced by loads and weather actions. With this simulation tool, the long-term deformation mechanism of PC bridges has been clarified (Maekawa *et al.* 2011). In the present research, to discuss time-dependent behavior of RC beams, total crack width, w_{all} , will be calculated by integrating the gap between mean strain of reinforcement and mean strain at concrete surface along the crack spacings. The meaning of each variable will be explained in 2.4.

To consider the bond effect between concrete and reinforcement after cracking, the relationship between the average stress and the average strain of concrete is defined with stiffening parameter “ c ” (0.4 for deformed reinforcement). In areas where the bond is not effective, cracks are localized. Therefore, to express the effects of crack localization, a “zoning method” is applied and the fracture energy of concrete and the element size are considered in tension softening of the concrete in the plain concrete zone (An 1994). In this research, the RC zone (bond effective area) was decided to be the area double the cover thickness from the bottom of RC beams.

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The “zoning method” has been proposed for analyzing shear failure of RC beams. In this research, flexural deformation is the dominant behavior, and therefore there must be little effect of zoning on the results of structural analyses. Actually, the authors confirmed that the effect of the size of the RC zone was negligible.

To consider the shrinkage of cement paste, in the case of high-middle humidity, condensed water in the pores forms a meniscus at the gas-liquid boundary, and negative pressure caused by the surface tension becomes a driving force for shrinkage. In the case of low humidity, tensile force is considered due to the increment of surface energy in gel particles, and plastic deformation is modelled due to movement of the interlayer water. Additionally, the ink bottle effect due to geometric structure in the pores is also considered.

To consider the shrinkage of aggregate, the stiffness of aggregate is defined as a function of the mean density of fine aggregates and coarse aggregates. The shrinkage of aggregate is defined as a function of the maximum shrinkage strain when the aggregate is absolutely dried, and the degree of saturation of the aggregate.

2.2 Possible factors to be considered in investigating time-dependent behavior of RC beam since early age

To analyze shrinkage deformation of concrete after cracking accurately, evaporation of water has to be considered depending on crack width. In this simulation,

evaporation of water from micro cracks can be considered by the following equation (Yoneda *et al.* 2013).

$$J = -B \cdot (D_p \nabla P_l + D_T \nabla T) \quad (1)$$

where, J is a crack-dependent diffusivity, D_p is intrinsic moisture conductivity with respect to the pore pressure gradient, D_T is intrinsic moisture conductivity with respect to the temperature gradient, P_l is pressure. B is coefficient considering the acceleration effect of crack, and it is given from the following equation,

$$B = D \cdot (\varepsilon - 300/10^6) \times 10^3 + 1.0 \quad (2)$$

where ε is crack-width equivalent tensile strain, D is intrinsic parameter set as constant.

In the experiment of this research, flexural cracks of about 0.2 mm width were generated at an early age. However, these cracks were naturally generated, and it was presumed that increment of water evaporation from these cracks was small. Therefore, the authors judged that the model by Yoneda *et al.* for micro cracks would be sufficient for the purposes of our simulation.

In addition, it has been reported that creep deformation in concrete can be generated not only due to the water flow in pores, but also due to the effect of micro defects between cement paste and aggregates (Honda *et al.* 2013; Watanabe *et al.* 2011). The model for concrete under sustained loading was proposed by El-Kashif (Maekawa

Fig. 1 Conceptual scheme of application of numerical simulation in this research.

et al. 2008), but has not been verified for early age concrete. There is a possibility that, at a very early age, creep strain can develop due to the generation of micro cracks at interfacial transition zone (ITZ).

Moreover, when the concrete in the compression zone is under high stress/strength ratio at an early age, bond deterioration between concrete and reinforcement can occur. In that case, substantial compressive stress of concrete becomes larger, leading to larger creep deformation.

2.3 Cracking criterion strength

In this simulation, excessive unnecessary cracks can be generated immediately after casting of concrete. To avoid this unnecessary cracking, an imaginary tensile strength, “cracking criterion strength,” is set in this research. When the tensile strength calculated from compressive strength obtained in the numerical simulation is smaller than the cracking criterion strength, especially at

a very early age, the judgment of cracking is conducted by the cracking criterion strength (Fig. 2). On the other hand, when the tensile strength calculated from compressive strength obtained in the numerical simulation exceeds the cracking criterion strength, the judgment of cracking is conducted by the calculated tensile strength. In this study, the cracking criterion strength was calculated from the experimental compressive strength at the age when continuous bending loading was applied to RC beams.

2.4 Calculation method of total crack width

The simulation tool used in this research is essentially based on the smeared crack model. Therefore, it is difficult to predict the generation and the propagation of individual macro cracks. In this research, as shown in Fig. 3, Total crack width is a calculated value based on the results of numerical simulation. The total crack width is calculated by integrating the difference between the mean strain of reinforcement and the surface strain of concrete in the constant moment area. On the other hand, summation of flexural crack width is an experimentally obtained value which is the summation of whole crack widths in the equivalent moment zone of the RC beam. Comparing the calculated total crack width with the experimentally observed crack width, the time-dependent deformation of RC beams is analyzed. Here, in Fig. 3, tensile strain is defined as positive, and compressive strain is defined as negative.

In this calculation method, to obtain the strain of reinforcement, the tension stiffening effect is considered. The following two factors are the driving force for the strain at the surface of concrete between flexural cracks.

- The difference of elastic strain between concrete and reinforcement due to shrinkage and creep of concrete
- Strain caused by external force

However, the second factor is known to be negligible (Kakuta 1970). Thus, in our calculation method, this factor is not considered.

Fig. 2 Conceptual scheme of cracking judgment.

Fig. 3 Calculation method of total crack width using numerical simulation.

In order to calculate strain at the concrete surface, Ishibashi *et al.* (1994) proposed the assumption of an imaginary concrete member called “a small divided member (mean crack spacing $\times$ member width $\times$ 1/5 of girder height)”. According to Ishibashi’s proposal, this small divided member was assumed to shrink freely with its six surfaces exposed to the environment at 20°C and R.H. of 70%. The reason for the high accuracy of this simple method for predicting the maximum crack width of real structures might be because the increment of dry shrinkage due to accelerated evaporation from cracks might be canceled by the restraint from the reinforcement (Seki *et al.* 2010). In this research, based on Ishibashi’s method, the strain at the surface of concrete is numerically simulated and used for predicting the crack width. The environmental conditions for the simulation were set as 20°C and R.H. of 60% in line with the experimental conditions. The length of a small divided member is decided as the observed mean crack spacing. The boundary condition of this member is set as the sealed condition until the age of demolding of the RC beam, and after flexural cracks are generated, the six surfaces are exposed to drying conditions.

It was revealed that when micro-cracks were generated in very high numbers in the simulation, especially in the case of the large shrinkage property of concrete, the shrinkage behavior of concrete was not accurately simulated. Therefore, only when the free shrinkage of small divided members is calculated, cracking criterion strength as described in section 2.3 is set as 99.9 N/mm² to prevent the generation of micro cracks. Using this method, it was found that the shrinkage behavior of plain

concrete was well predicted (Komatsu *et al.* 2013).

2.5 Size of element

In this simulation, the average stress-strain relationship is applied. Therefore, it is recommended that the element size of concrete should be larger than the maximum particle size of coarse aggregates (20 mm). However, in past research (Komatsu *et al.* 2013), it was pointed out that the shrinkage behavior of concrete with high shrinkage potential could not be predicted well when the size of the outermost layer was set as 20 mm. Micro cracks propagated from the surface into 5 mm depth in a 150 $\times$ 150 $\times$ 400 mm cube after demolding, which caused discrepancy between the simulated results and experimental results. This discrepancy was solved by setting the size of the outermost layer as 5 mm. In this research, to prevent the propagation of unnecessary micro cracks for accurate simulation, the size of the outermost layer was set as 5 mm. In the longitudinal direction of RC beams, the maximum size of elements was set as 30 mm.

Furthermore, in this research, different cracking loads were experimentally obtained according to the type of coarse aggregate. For investigating the effect of timing of cracking and crack propagation on the increment of tensile stress of reinforcement, strip elements (10 mm size) were arranged as the trigger of crack propagation (Fig. 4). In the equivalent moment zone, those cracking trigger elements were arranged so that the number of the trigger elements became equal to the number of observed flexural cracks. W/C of concrete in the trigger element was set as 60%, which was higher than that of concrete in surrounding elements.

Fig. 4 Meshing for RC beam.

Table 1 Mix proportions.

Type	Coarse aggregate	W/C (%)	s/a (%)	Unit content (%)				AE mixture (g/m ³)	AE water reducing agent (g/m ³)
				W	C	S	G		
G _s	Sandstone	50	46	155	310	810	983	217	310
G _l	Limestone		50	150	299	892	922	0	898

Table 2 Parameters of RC beams.

Name of RC beam	G _s (3)	G _s (28)	G _s (3)S	G _l (3)	G _l (28)
Coarse aggregate	Sandstone			Limestone	
Demolding [day]	2.8	27.8	2.0	2.8	27.8
Loading (cracking)[day]	3.6	28.2	3.2	3.6	28.2
Environment [°C, RH]	20°C, 60%		Sealed	20°C, 60%	

3. Outline of experiment and properties of concrete

3.1 Details of RC beam and method of loading

In this experiment, the density of fine aggregate used is 2.53 g/cm³. The densities of coarse aggregates, limestone coarse aggregate and sandstone coarse aggregate are 2.69 g/cm³ and 2.58 g/cm³, respectively. The maximum size of coarse aggregate is 20 mm. **Table 1** lists the mix proportions of concrete. The fresh properties of those concretes were almost constant (slump 15±1.5 cm, air content 4±1.5%). Curing condition was constant temperature and humidity environment (20°C, RH=60%).

The experimental parameters of RC beams are shown in **Table 2** and **Fig. 5**. The dimensions of RC beams were made almost the same as in the experiment by Ishibashi *et al.* (1994). In order not to disturb the bond between concrete and reinforcement, a thin channel was made along the longitudinal rib of reinforcement, and strain gauges were attached in that channel at 30 mm intervals. The channel was then filled with epoxy resin for waterproofing.

The mean tensile stress of reinforcement was calculated as follows. Before the reinforcement yielded, the mean stress was calculated using the mean strain from observed strains. On the other hand, after the reinforcement yielded, the stress was set as the yield stress at the yield locations. All measured strains were below the strain at the start of the strain hardening range. The mean

stress of reinforcement was calculated by the integration of stress distribution expressed as a parabola using the nearest three points (Shima 1986).

In this experiment, two-point loading was applied by controlling the strains of PC bars for loading. After applying sustained loading, no additional flexural cracks were generated during the investigation. The mean tensile stress level of reinforcement was controlled as a value around 180 N/mm², which was almost the same stress level of an actual RC box girder (Higashi-Sendai viaduct: Tohoku Shinkansen structure), which suffered severe flexural and shrinkage cracks (Matsuoka *et al.* 1981).

In the constant moment area, strain gauges were attached at the compressive edge and at 50 mm from the compressive edge. The deflection was measured at the center of the span.

After the loading, to investigate development of internal cracks around tensile reinforcement, an epoxy resin (low viscosity) with red ink was injected into the flexural cracks. After the epoxy resin hardened in the flexural cracks, the tension in PC bars was released. Concrete cores including flexural cracks were taken from the RC beams. Damage of the concrete around tensile reinforcement was observed by cutting concrete cores.

3.2 Shrinkage property and strength property of concrete

In the numerical simulation used in this research, the maximum shrinkage strain of aggregate in absolute dry

Fig. 5 Dimension of RC beams.

condition is required to simulate the shrinkage of concrete. In this research, by inverse analysis of the results of shrinkage tests of concrete by the JISA1129 method, the maximum shrinkage strain of aggregate was estimated ($G_s = 2000 \times 10^{-6}$, $G_l = 500 \times 10^{-6}$). Shrinkage properties of concretes with sandstone and limestone were about 1300×10^{-6} and 600×10^{-6} , respectively. The simulation results showed good agreement with experimental data (Fig. 6).

The stress and strain relationships in the compressive strength test are shown in Fig. 7. In the compressive strength test, friction between the loading plates and the specimen was removed through the use of a Teflon sheet. The crack surfaces of the specimens loaded at 28 days are shown in the same figure. Stiffness of aggregate was calculated following the rule of thumb from experimental works (Zhu 2004). A 1/8 model with circular column element was used in the simulation shown in Fig. 7.

The results show that the specimens with sandstone showed more nonlinear behavior in early stress level compared with those with limestone. Furthermore, for the specimens with limestone, the maximum load was underestimated in the simulation.

Regarding the crack pattern, in the specimens with

Fig. 6 Results of shrinkage behavior of concretes with different aggregates.

Fig. 7 Results of static loading for compression.

sandstone, almost all cracks were along ITZ. On the other hand, in the specimens with limestone, many cracks penetrating coarse aggregate were observed. In past research, it has been reported that bond strength between coarse aggregate and mortar differs depending on the aggregate type (Kawakami 1991).

4. Test results of continuous loading of RC beams

4.1 Deformational behavior of RC beams in static loading

The mean tensile stress of reinforcement during static loading is shown in Fig. 8. The numerical simulation results agreed well with the experimental results, which means that the mechanical properties of concrete were appropriately simulated based on the input data. In the results of this simulation, the stiffening factor was $c = 0.4$. The strains at compressive edge and the strains at the middle of the compressive zone obtained in the numerical simulation also agreed well with the experimental data. It was confirmed that, at the age of the static loading, the material properties of concrete and bonding behavior expressed by tension stiffening were appropriately calculated in this simulation.

Deformational behaviors of RC beams in static loading are summarized in Table 3. In the RC beam loaded at 3.6 days, the results of simulation using different stiffening factor ($c = 0.2$ to 0.9) showed almost same results. This may be due to the low tensile strength (1.1 N/mm^2 : half of the tensile strength at 28.2 days) in the simulation. In the RC beam loaded at 3.6 days, the increment of tensile stress of the reinforcements was so large that the tension stiffening effect was not much affected.

As shown in Table 3, in the RC beam using sandstone, cracking load was smaller compared with the RC beam with limestone. In the RC beam using sandstone, flexural crack spacing was smaller than that of the RC beam using

Table 3 Deformation of RC beams immediately after static loading.

Aggregate used in the beams	Sandstone		Limestone	
Beam name	G _s (3)	G _s (28)	G _l (3)	G _l (28)
Loading age [days]	3.6	28.2	3.6	28.2
Cracking load [kN]	11	14	14	20
Number of cracks	7	6	6	6
Mean crack spacing [mm]	115.3	121	157	155.8
Mean crack width [mm]	0.17	0.14	0.18	0.14
Deflection [mm]	3.41	2.54	2.78	1.78
Increment of stress of reinforcement when cracks are generated [N/mm ²]	205.2	190.2	185.5	180.2

limestone at the same loading age. As discussed regarding Fig. 7, in the specimen using sandstone, bond strength at ITZ was weaker than that in the specimen using limestone from the observation of crack propagation. Based on past research (Kakuta 1970), maximum flexural crack spacing was defined as the function of maximum bond strength and tensile strength of concrete.

$$l_{\max} = \frac{2k_1 A_e \sigma_{ct}}{u \bar{\tau}_{\max}} \quad (3)$$

where $l_{\max}$: maximum crack spacing, k_1 : coefficient of tensile stress distribution of concrete in the perfect bonding cross section, A_e : effective cross section of concrete, σ_{ct} : tensile strength of concrete, $\bar{\tau}_{\max}$: bond strength, u : perimeter of reinforcement.

It was confirmed in this research (Table 3) that the difference in bond strength at ITZ influenced cracking (tensile) strength. On the other hand, the authors guess that bond strength at ITZ had less influence on bond strength $\bar{\tau}_{\max}$. The reason for this negligible influence is that bond strength, $\bar{\tau}_{\max}$, is mainly governed by bearing resistance at lateral ribs. Bearing resistance in which the mechanical behavior of concrete in compression is

dominant must be less affected by bond strength at ITZ. Therefore, the difference of $l_{\max}$ of the RC beams using different coarse aggregates was mainly due to the difference of σ_{ct} .

4.2 Time-dependent behavior of RC beams

4.2.1 Influence of age of cracking on time-dependent behavior of RC beam

The influence of the age of cracking on the increment of the mean tensile stress of reinforcement is shown in Fig. 9. Deflections of the RC beams are also shown in the same figure. In the RC beam loaded at 3.6 days, significant increment of the mean tensile stress of reinforcement was observed. In past research (Ishibashi *et al.* 1994), it was reported that the increment of tensile stress of reinforcement after cracking was small (increment of tensile stress of reinforcement after generation of flexural cracks was about 18.8 N/mm² when continuous loading was started at 14 days.). Figure 9 shows that the increment of tensile stress of reinforcement after cracking is not negligible when continuous loading is started at a very early age.

Fig. 8 Results of static loading.

4.2.2 Influence of shrinkage property of concrete on time-dependent behavior of RC beam

The influence of the shrinkage property of concrete on the mean tensile stress of reinforcement is shown in Fig. 10 (continuous loading was started at 3.6 days). In the RC beam whose concrete had larger shrinkage property (sandstone coarse aggregate), larger increment of the mean tensile stress of reinforcement was observed.

4.2.3 Influence of environmental condition on time-dependent behavior of RC beam

The influences of environmental conditions on the mean tensile stress of reinforcement and the strain of concrete at the compressive edge are shown in Fig. 11. In the sealed RC beam, smaller increment of the reinforcement

stress was observed. The strain of concrete at the compressive edge was also smaller in the sealed RC beam. In the sealed RC beam, decreasing of the mean tensile stress of reinforcement was observed, which may be due to the generation of a shear crack. The generation of a shear crack was also confirmed in the results of numerical simulation.

The distributions of the tensile strain of reinforcement for two beams exposed to different environmental conditions are shown in Fig. 12. In the RC beam exposed to the environmental conditions of 20°C and R.H. = 60% (Gs(3)), reinforcement yielded in some points. On the other hand, in the sealed RC beam (Gs(3)S), reinforcement remained in the elastic range. This difference may be due to the influence of environmental conditions

Fig. 9 Effect of age of cracking on mean stress development of reinforcement.

Fig. 10 Effect of shrinkage property on mean stress development of reinforcement.

Fig. 11 Mean tensile stress of reinforcement and strain of concrete at compressive edge in RC beams with different environmental conditions in sustained loading.

Fig. 12 Strain distribution of reinforcement in constant moment zone.

during continuous loading on the creep deformation of concrete in compression. This will be further discussed in section 5.2.

5. Analysis of time-dependent behavior of RC beams with numerical simulation

5.1 Mechanism of increment of tensile stress of reinforcement

The time-dependent behavior of the RC beam with limestone aggregate loaded at 3.6 days is shown in **Fig. 8** (compressive stress/strength ratio = 41%). In the simulation, stiffening factor “*c*” was changed in the range between 0.2 and 0.9. A smaller “*c*” value indicates better bond between reinforcement and concrete. The influence of tension stiffening on the time-dependent behavior was small, which was because the tensile strength of concrete was small (1.1 N/mm² at 3.6 days and 2.1 N/mm² at 28.2 days). Kakuta pointed out the causes for the increment of reinforcement stress after cracking as follows (Kakuta 1970).

- Compressive creep deformation of concrete causing the neutral axis to get closer to the compressive edge so that the equilibrium is satisfied between compression force of concrete and tension force in reinforcement

- Deterioration of bond between tensile concrete and reinforcement

The second cause above will not be the main mechanism in this research, because the tension stiffening effect is originally small for young age RC beams. Thus, the authors believe that the first cause above will be the main mechanism for the significant increment of reinforcement.

5.2 Mechanism of creep deformation at high compressive stress/strength ratio

The results of the numerical simulation of the increment of the mean tensile stress of reinforcement in the RC beam loaded at 3.6 days differed significantly from the experimental results. This suggests that there must be some important factors not considered in the simulation system which importantly affect the time-dependent behavior of young age concrete. The authors’ hypotheses

are as follows.

- Due to the deterioration of bond between concrete and reinforcement in the compressive zone, the substantial compressive stress of concrete was increased.

- Micro cracks were generated at ITZ between the cement paste and aggregate in the concrete, which caused large compressive creep deformation.

These two hypotheses were simply expressed in numerical simulation as follows. First, the reinforcement in the compressive zone was removed. Second, the W/C of concrete in the compressive zone was increased from 50% to 60%. As one of the evidences for the second assumption, Honda reported that under the condition of a high stress/strength ratio, the bond at ITZ remarkably deteriorated (Honda 2013). As shown in **Fig. 13**, the simulation result of strain of concrete at the compressive edge became closer to the experimental data by increasing W/C from 50% to 60%.

The authors adopted this simple method to obtain more accurate time-dependent development of tensile stress. The main objective of using this simple method was to analyze time-dependent behavior of concrete in the tensile zone discussed below. These hypotheses should be verified in future.

Figure 14 shows the increment of the mean tensile stress of reinforcement in the RC beam with limestone aggregate loaded at 3.6 days. The red numbers show the stress/strength ratio (σ_c / f_c) at the loading age. Considering both phenomena above, the simulation results (stress/strength ratio = 69%) agree well with the observed results.

Figure 15 shows the results of the RC beam with sandstone aggregate. The red numbers show the stress/strength ratio (σ_c / f_c) at the loading age. Even though both phenomena were considered, the simulation results (compressive stress/strength ratio = 69%) were still underestimating the observed result. In the RC beam with sandstone aggregate, the flexural cracking load was lower than that of the RC beam with limestone aggregate. This may be due to the difference of bond strength between coarse aggregate and mortar as mentioned in past research (Kawakami 1991) and as shown in the crack propagation on the fracture surface in the compressive

Fig. 13 Mean tensile stress of reinforcement and strain of concrete at compressive edge in RC beam using limestone after continuous loading.

strength specimen (Fig. 7). In the case of the RC beam with sandstone aggregate, the lower cracking load might

Fig. 14 Stress development of reinforcement of RC beam with lime-stone aggregate.

Fig. 15 Stress development of reinforcement of RC beam with sandstone aggregate.

have influenced the larger creep deformation of concrete at the compressive zone, which caused the further increase of tensile stress of reinforcement.

To investigate the effects of weaker bond at ITZ, the same numbers of cracking trigger elements of 10-mm width as those of observed flexural cracks were provided, as shown in Fig. 5. The W/C of concrete in the trigger element was 10% larger than that in other elements. Consequently, as shown in Fig. 15, significant increment of the mean tensile stress of reinforcement (compressive stress/strength ratio = 98% at half a day after the starting of sustained loading) was observed. However, in this simulation, the rate of increase of tensile stress was still smaller than the reality. The reason for this might be that the propagation of flexural cracks was not appropriately simulated. Even in the RC beam whose surfaces were sealed (Gs(3)S), the rate of increase of tensile stress was smaller than the reality.

It was revealed that, in the RC beam loaded at 3.6 days, the tensile stress of reinforcement increased due to large creep deformation of concrete at high stress/strength ratio in the compressive zone.

5.3 Investigation of propagation of flexural crack width

To investigate the time-dependent behavior of concrete in the tensile zone of RC beams during continuous loading, the time-dependent development of crack width was investigated. To calculate flexural crack width using the results of FEM simulation, a concept called total crack width in the constant moment zone is proposed.

Fig. 16 Development of total crack width using different types of coarse aggregate and different flexural cracking ages.

For calculating flexural crack width, stress of tensile reinforcement in FEM simulation results was used where good agreement of tensile stress of reinforcement was confirmed in the RC beams loaded at 3 days (stress/strength ratios of $G_s(3)$ and $G_t(3)$ were 69% and 98%, respectively). For calculating flexural crack width in RC beams loaded at 28 days, stress of tensile reinforcement in FEM simulation results was used where good agreement of tensile stress of reinforcement was confirmed between simulation and experimental results.

The total crack widths calculated using the method shown in Fig. 3 are indicated in Fig. 16. For calculation results, two lines are exhibited. The upper continuous line shows the total crack width considering free shrinkage of the concrete between flexural cracks, and the lower dotted line shows the total crack width without considering shrinkage of concrete. Observed values of mean crack width are also shown in the same figure. The crack width was measured at the same locations at the tensile edge.

In the RC beam loaded at 3.6 days, the simulation results without considering the shrinkage of concrete between flexural cracks more closely approximated reality. On the other hand, in the RC beam with limestone aggregate loaded at 28.2 days, the simulation results considering free shrinkage of the concrete between flexural cracks were close to the experimental results.

In past research, it was revealed that micro defects at ITZ influence creep strain in young age mortar specimens using the AE method when the stress/strength ratio is kept constant (Watanabe 2011). The authors think that in the RC beam loaded at 3.6 days, tensile deformation of concrete following the deformation of reinforcement due to continuous loading become larger because of the occurrence of micro defects at ITZ. The ductile tensile deformation of concrete with micro defects was derived due to the lesser degradation of the bond between reinforcement and surrounding concrete (Fig. 17). Therefore, good agreement was observed between the reality and the simulation results not considering shrinkage of concrete between flexural cracks due to the restraint by reinforcement.

On the other hand, in the RC beam loaded at 28.2 days, the simulation results without considering the shrinkage of the concrete between flexural cracks more closely

approximated reality. In past research, it was pointed out that the bond between reinforcement and surrounding concrete deteriorated and was accompanied by internal cracks (Goto 1971) (Fig. 17). It was also pointed out that calculation assuming free shrinkage of concrete agreed well with the observed crack width at the surface (Seki *et al.* In the RC beams loaded at different age, cross sections with flexural cracks were observed as shown in Fig. 18. The purple line shows the location of flexural cracks and the pink area shows the un-bonded area between concrete and reinforcement where epoxy resin was induced (Hayashi 2009). It was clarified that in the RC beam loaded at 3.6 days, deterioration of the bond around reinforcement was less compared with the case in the RC beam loaded at 28.2 day.

In the RC beam using sandstone aggregate loaded at 28.2 days, the total crack width in the simulation result considering the shrinkage of concrete between flexural cracks was larger compared to the reality. Even when concrete was sufficiently matured, the bond strength between coarse aggregate and paste could affect the generation of the micro defects in the tensile concrete around reinforcement. The authors guessed that in the case of $G_s(28)$, internal cracks around reinforcement might have been restricted due to micro defects, which might have caused smaller shrinkage of concrete.

6. Conclusions

The conclusions obtained from this research are as follows.

1. The significant increment of the tensile stress of reinforcement after cracking was experimentally observed when RC beams were loaded at 3.6 days. The shrinkage property of concrete and the environmental conditions greatly affected the increment of reinforcement stress.
2. The significant increment of the tensile stress of reinforcement might be due to the drying creep deformation of concrete in compression.
3. An evaluation method of the time-dependent behavior of concrete in the tensile zone was proposed by comparing the calculated total crack width utilizing FEM simulation results and the observed total crack width. In the RC beam loaded at 3.6 days, the development of

Fig. 17 Schematic illustration of concrete deformation mechanism around tensile reinforcement.

Fig. 18 Bond between concrete and reinforcement in RC beams (G_s (3) and G_s (28)) loaded at different ages.

internal cracks around reinforcement might have been restrained due to the large deformability of concrete caused by micro defects at ITZ. The tensile deformability of concrete following the deformation of reinforcement due to continuous loading must have been caused from less degradation of the bond between the reinforcement and surrounding concrete. The shrinkage of concrete between flexural cracks might have been restrained by this better bond.

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