EFFECT OF CONCRETE STRENGTH ON LONG-TERM CRACK WIDTH OF CONCRETE MEMBERS

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ABSTRACT: The effect of concrete strength on long term cracking of concrete member was investigated experimentally. Two concrete compressive strengths of 45 and 100 N/mm^2 were adopted. Comparison between calculation and experiment results showed that, in high strength concrete members, JSCE Code equation overestimated maximum crack width specially as reinforcement stress became small. In additon, Considering bond stiffening and fracture energy in tension at cracked concrete section would improve the accuracy of calculating of crack width.

KEYWORDS: Crack Width, Crack Spacing, Long-Term, High strength concrete, Bond.

1. INTRODUCTION

Cracking of reinforced concrete members has received enormous research attention, as durability and serviceability of concrete structures is often limited by crack formation. Excessive crack widths can impair corrosion resistance of exposed structures. Therefore, control of cracking may be important for serviceability. However, it is difficult to predict the number and flexure crack width because of the complex processes that are involved. Therefore, there is no universal approach for estimating these quantities. Various codes are based on regression analysis of test data and others are based on simplified models of mechanics crack formation. The predictions that result from these models are accompanied by an inherently high uncertainty and limitations. Nowadays, tendency for both smaller and economic reinforced concrete sections increases. In addition, the use of high strength concrete (HSC) has been widely spread, where, HSC has better durability, lower creep and lower permeability. Furthermore, HSC permits accelerated construction, and provides a better solution for reducing sizes and weights of concrete structural elements. Therefore, the evaluation of the specifications, that control the crack width and spacing and the capabilities of these specifications to serve, becomes necessary. This evaluation should include controlling of cracking under short-term loading as well as under long-time loading, where, under sustained loads, crack width increases with time.

In this study, the effect of concrete strength on long term cracking of concrete member was investigated experimentally. Concrete members were reinforced as well as prestressed reinforced. In addition to concrete members, specimens for creep test and shrinkage test with the same cross section as concrete members were also made. Crack spacing, crack width and strain distribution of tension reinforcement in the longitudinal direction were measured and discussed including the applicability of JSCE Code equation to estimate maximum crack spacing and maximum crack width. The effect of bond on long-term crack width was discussed.

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2. OUTLINE OF EXPERIMENT

Sixteen beams specimens were designed to investigate the crack width and spacing. All the beams are geometrically identical with varying reinforcement ratios. **Fig.1** shows the cross section details. All the beams are investigated under two concentrated sustained point loads. The sustained load was applied by using a system combined of screw shaped PC bars, nuts, rigid plates, springs and load cells. Setup of the experiment is shown in **Fig 2**.The sustained load was maintained by monitoring the measured magnitude by the load cell, and was adjusted within ±3% of the designed value by retensioning the PC bars by applying torque to the nuts.

High-early-strength portland cement was used. The coarse aggregate was a crushed gravel with maximum size of 20mm, specific gravity of 2.63 and fineness modulus of 6.68. The fine aggregate was a river sand with specific gravity of 2.61 and fineness modulus of 2.69.

Two types of concrete mixes NL and HL were used of compression strength 45 and 100 N/mm^2 , respectively, at the age of 28 days under curing temperature of 20^oC. The concrete mix proportions are tabulated in **Table 1**. Four kinds of laterally lugged deformed bars with bar sizes of 10,13,16 and 19mm with elastic moduls of 201,202,205 and 204 GPa were used in the tests as main compressive and tensile reinforcements. For each of the two concrete mix, eight beams were applied to different prestressing levels to obtain different levels of tension stresses in the tension reinforcement (**Table 2**).

Table 1 Mix.Proportions

Mix	w/c (%)	s/a (%)	SF/B (%)	Proportions (kg/m^3)					Additives (g/m^3)		Fc' N/mm^2
				W	C	SF	S	G	AE Agent	SuperPlastic. Agent	
NL	55	46	-	165	300	-	843	1000	1200	6000	45
HL	25	45	10	165	540	60	766	640	-	12200	100

SF: Silika fume.
B : Binder

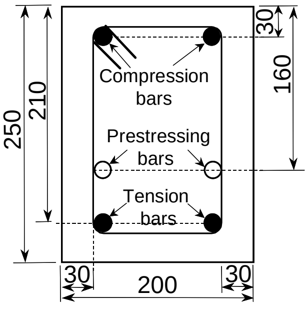

Fig. 1 Cross section of specimens

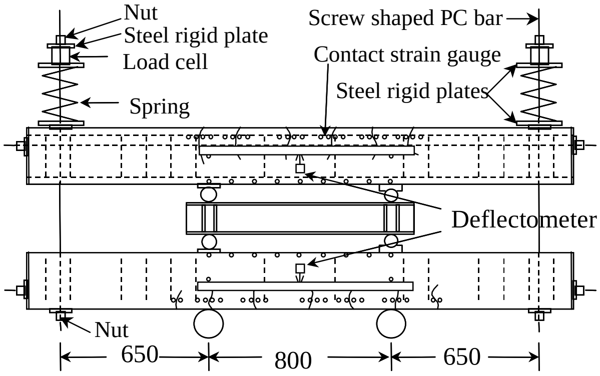

Fig. 2 Experimental setup

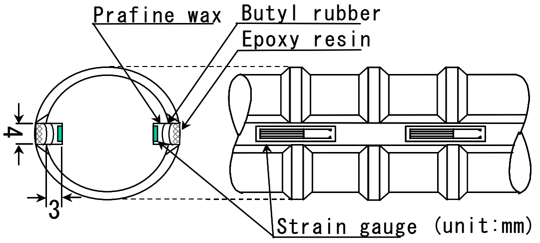

Fig. 3 Method of measurement of reinforcement strain

The concrete tension strain at the same level of tension reinforcement was measured by contact strain gauge. In order to measure tension steel strain distribution, the tension reinforcements were grooved and provided with strain gauge as illustrated in **Fig. 3**. Steel strains were measured along the pure bending zone only. During the experiment period under sustain load, deflection, steel strain and concrete surface strain were recorded. In addition, Both crack width and spacing were measured and finally crack patterns were observed. The experiment period extended to 1000 loading days.

Table 2 Reinforcements and loading configurations

Specimens	Reinforcement	Tension Steel As		Compression Steel		PC bar		Moment	Calculated short-term tension steel stresses (N/mm^2)		Effective concrete area Ae (mm^2)
		(mm^2)	(%)	(mm^2)	(%)	(mm^2)	(%)	(kN.m)	NL	HL	
NL10 and HL10	D19-0-0	573	1.36	-	-	-	-	20.2	182	181	15427
NL12 and HL12	D19-0-D16	573	1.36	397.2	0.95	-	-		182	181	15427
NL20 and HL20	D16-7.1-0	397.2	0.95	-	-	80	0.19	9.2	161	156	15603
NL22 and HL22	Dm16-7.1-D16	315.1	0.75	397.2	0.95	80	0.19		193	182	15685
NL30 and HL30	D13-9.0-0	253.4	0.6	-	-	128	0.3	20.2	119	140	15814
NL32 and HL32	Dm13-9.0-D16	186.1	0.44	397.2	0.95	128	0.3		140	160	15814
NL40 and HL40	D10-10.7-0	142.7	0.34	-	-	180	0.43	9.2	70	64	15857
NL42 and HL42	Dm10-10.7-D16	142.7	0.34	397.2	0.95	180	0.43		54	54	15857

3. RESULTS AND DISCUSSIONS

The measured maximum crack width and spacing were compared with those calculated according to the JSCE code equation (1) for both short-term and long-term [1].

$$w = \underbrace{k_1(4c + 0.7(c_s - \phi))}_{\text{L}}\left(\frac{\sigma_{se}}{E_s} + \varepsilon_{csd}\right) \qquad (1)$$

Where: w and L are the maximum crack width and spacing, k_1 is a constant depending on geometric details of the surface of reinforcement, c is concrete cover, c_s is center-to-center distance of steel bars, ϕ is diameter of steel bar, σ_{se} is tension stress increment of reinforecement from the state of zero stress in concrete as the same position of the tension reinforcement to the state under load considered, E_s is modulus of elasticity of steel bar and ε_{csd} is strain to take into account the influence of creep and drying on crack widths considering the environmental condition for durability. For long term calculation, JSCE recommend its value to be 150×10^{-6} for steel corrosion.

From equation (1), it is clearly shown that the influence of the variation of concrete strength is not considered.

Figs. 4 shows the relation between the measured and calculated maximum crack spacings and widths. Where $L_{max,meas}$ and $L_{max,JSCE}$ are maximum measured and calculated crack spacing, respectivelly, and $W_{max,meas}$ and $W_{max,JSCE}$ are maximum measured and calculated crack width, respectivelly.

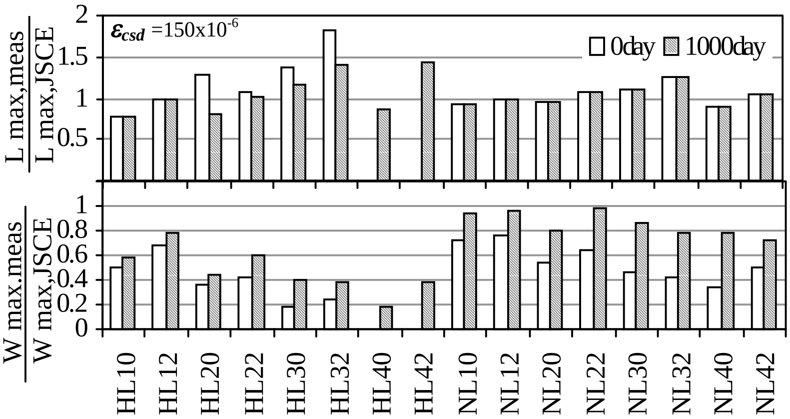

Fig. 4 Ratio between measured and calculated maximum crack spacings and widths

As shown in **Fig.4**, at 1000 days after loading, normal strength specimens (NL) gave a good agreement for both maximum crack spacing and width comparing with that calculated by JSCE code equation, as the ratios for the measured to the calculated were 0.7-1.0 with average of 0.85 for crack width and 0.9-1.3 with average of 1.0

for crack spacing. However, in case of HL specimens there was a large deviation between the calculated and those measured. In order to get a clear explanation about the cause of this deviation, the relation between the calculated stresses in tension steel at cracked section and the ratio between the measured and the calculated crack spacings and widths in long-term was examined, the results are shown in **Fig.5**. It is noted that crack spacing for HL specimens scattered around the value of 1.00, and were in range of 0.8-1.4 with average value of 1.05. The standard deviation in case of HL specimens was 0.26, which is larger than that of NL specimens, which was 0.12. It seems that, in long term, the variation of concrete compression strength does not affect the accuracy of the JSCE code in prediction of crack spacing.

On the other hand, for HL specimens, the calculated maximum crack widths were always larger than those measured. The difference between them increases, as steel stress decreases. The ratio of the calculated to the measured crack width varied in range of 0.2-0.8, Therefore, the JSCE code equation for maximum crack width calculations needs modifications to cover the case of high strength concrete members, although it is accepted in case of normal strength concrete.

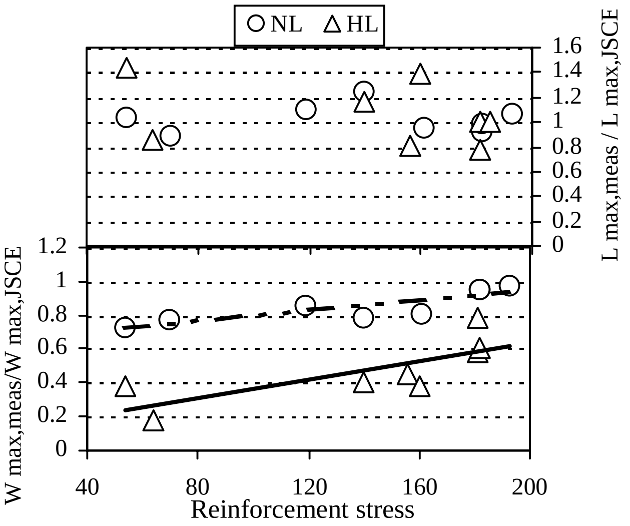

Fig. 5 Relation between steel stresses and ratio between measured and calculated maximum crack spacings and widths in long-term

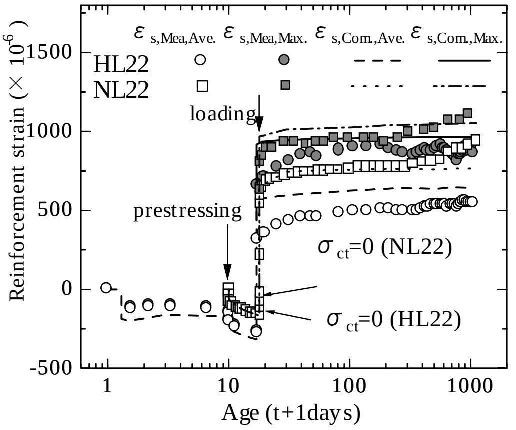

Fig.6 Comparison of HL22 and NL22 for time-dependent strain[2]

The measured steel strains in the bending zone of two specimens NL22 and HL22 of normal and high strength concrete,respectively, were chosen to clear the difference in cracking behavior between the NL and HL specimens. The two specimens are identical in geometric properties and reinforcement. In addition, the calculated tension steel stresses at cracked section were nearly the same and with values of 193 and 182 N/mm^2 for NL22 and HL22 (Table 2), respectively.

As shown in **Fig.6**, Sato et.al compared between the HL22 and NL22 for time-dependent strain for both the measured and calculated strains[2]. In the figure, $\varepsilon_{s,M,Ave}$ and $\varepsilon_{s,M,Max}$ are the average and maximum tension reinforcement strains obtained by the measurements, and $\varepsilon_{s,C,Ave}$ and $\varepsilon_{s,C,Max}$ are those obtained by calculation. Contribution of concrete in tension at cracked section was not considered in calculations. In case of high strength concrete, unlike normal strength concrete, the calculated strains were always larger than that measured. This difference indicates the contribution of concrete in tension at the cracked section that acts to make steel stresses less than the calculated values, probably, due to the cohesive stress in the fracture process zone depending on crack opening displacement. Strain of reinforcement steel increased with time due to the combination effects between creep and shrinkage as well as new development of micro cracking softening the macroscopical stiffness of tension concrete [2]. The contribution of concrete in tension of high strength concrete is larger than that of normal strength concrete.

The contribution of bond in cracking behavior could be obtained from the difference between the measured reinforcement strains at the cracked section and the average strains. It is shown that, the contribution of bond for HL22 is larger than that of NL22, where bond stiffness for high strength concrete is larger than that of normal strength concrete, as demonstrated in **Fig.7** [3]. In the figure, MSC and HSC are the same, in this study, as (NL) normal and (HL) high strength concrete, respectively, and. the specimens are the same as those of this study. Bond-slip relationship was obtained from the strain distribution measured in reinforcing bars outside the vicinity of cracks. For the same bond stresses, slip in case of high strength concrete is less than that for normal strength concrete. Consequently, normal strength concrete exhibits wider crack width than that of high strength concrete under the same bond stress conditions.

Fig.8 shows the variation of bond contribution, represented as the difference between the measured reinforcement strain at the cracked section and the average strain for both short and long term considering the variation of the effective reinforcement ratio (As/Ae). According to the cross section dimension and reinforcement arrangements, the effective area (Ae) were taken equal to the area of concrete surrounding the tension steel and having the same centroid as that of steel (equation 2).

$$Ae = 2\,b\,(h - d) - As \qquad (2)$$

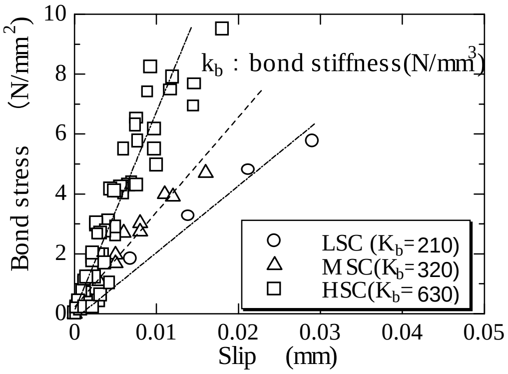

Fig. 7 Relation between bond stress and slip [3]

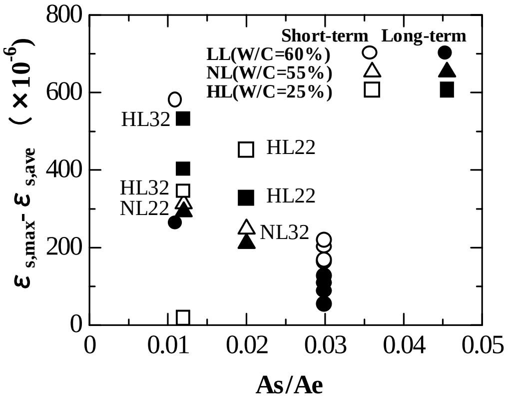

Fig. 8 Relation between effective steel ratio and the differnece between the maximum steel strain at cracked section and the mean steel strain.

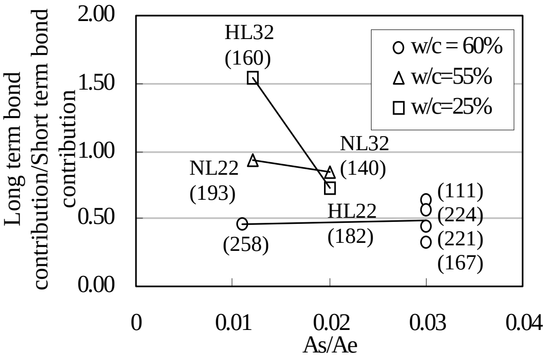

Fig.9 Effect of As/Ae on the change of bond contribution after long term

Where, *Ae* is effective area of concrete, *b* and *h* are the width and the height of the rectangular concrete cross section, *d* is the distance from extreme compression fiber to centroid of tension reinforcement and *As* is the cross section area of the tension reinforcement.

Fig.8 includes, also, experimental results for beams specimens with the same cross section details, shown in **Fig.1**, without the prestressing bars. The specimens were made of concrete mix (LL) of w/c=60% [4].

As generally shown from **Fig.8**, in case of either short or long term, bond contribution in resisting growing of crack width decreased with increasing of the effective steel ratio. Moreover, excluding the short term result of w/c=0.60 at As/Ae=0.01, the larger concrete strength showed larger bond contribution, whatever the values of As/Ae was.

In order to assess the change in bond contribution in long term corresponding to the change of As/Ae, **Fig.9** shows the relation between As/Ae and the ratio between bond contribution in long-term divided by that of short-term. In the figure, the calculated short term steel stress is writen beside each point. From the figure, it is shown that, at As/Ae of 0.01, the long term change in bond contribution remarkably affected by the concrete strength. In case of w/c=0.25 (HL), for HL32, bond contribution increased after long time to 150% of that of short term, rather than the cases of other lower concrete strengths. However, when As/Ae varied from 0.01 to 0.02 companied with increase of tension steel stresses, the ratio of bond contribution for HL specimens decreased from 150% to 73% of its initial values in short term. However, in case on NL specimens, NL22 and NL32, despite of the decrease of tension steel stresses form 193 to 140 N/mm^2 and increasing of As/Ae from 0.01 to 0.02, there was a little decrease in the bond contribution from 95% to 85% of that in short term. On the other hand, specimens of w/c=60%, (LL) seemed to be not affected by either the variation of As/Ae ratio or the the variation of tension stress of steel, where it exhibited constant degradation percentage in long term in range of 35% to 65% with average of 50%. Therefore, it is obvious that there are mutual effect between As/Ae, steel stresese and concrete strength on bond contribution, specially, after long time, which needs many more speciemens to get the proper model.

4. CONCLUSIONS

Whithin the limits of the results obtained in this research, the following points could be concluded:

(1) JSCE code equation is suitable for conventional concrete.
(2) JSCE code equation overestimated maximum crack width as reinforcement stress became small,especially in case of high strength concrete members.
(3) Considering bond stiffening and fracture energy in tension at cracked concrete section would improve the accuracy of calculating of crack width and spacing.
(4) The mutual effect between the effective reinforcement ratio and steel stress on bond contribution, which may not affect the accuracy of the calculated crack width in case of normal strength concrete should be considered in case of high strength concrete, specially after long time.

5. REFERENCES

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