論文 BOND BEHAVIOR IN RC TENSION MEMBERS BASED ON THE CHANGE OF CONCRETE FRACTURE CHARACTERISTICS WITH TEMPERATURE

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ABSTRACT: Macroscopic bond behavior identified as tension-stiffening effect in RC members is investigated under various environmental conditions in terms of temperature and moisture through a series of uni-axially loaded tension tests. The experimental result showed somewhat diverse local response on concrete but rather slight variation on macroscopic tension stiffening in stabilized cracking regions when temperature changes. Characteristic response under elevated temperature (75℃) could be grasped as being attributed to varying fracture characteristics and consequent high deformability of concrete.
KEYWORDS: bond behavior, temperature, fracture characteristics, deformability, micro-defects

1. INTRODUCTION

For computational assessment of structural serviceability under mechanical loads, so-called "tension-stiffness" effect occupies the central position. As a matter of fact, a number of models have been proposed in the past decades [4,5,7]. At present, modeling of tension-stiffness is highly reliable under short-term loadings even if yield of reinforcement is actualized. Owing to greatly improved computational tools, the macroscopic tension stiffness can be derived from microscopic states of bond nonlinearity. The systematic verification was crucial for checking its performance.

However, this verification does not confirm applicability to RC structures exposed to special environmental conditions such as high temperature, where macro as well as local change of mechanics is anticipated in nature. Engineering needs to assess the long-term structural performance are raised. Here, we have to take into account cracking under service loads. For generic space-averaged tensile modeling, as the first step to discuss this issue, a series of uni-axial tension tests under high temperature (75℃) was performed. Special attention will be directed to local bond kinematics associated with fracture property and consequent deformability of continuum concrete as well as overall mechanical features of tension members.

2. TESTING PROGRAM

2.1 TEST SERIES AND CONCRETE PROPERTY

The test series was determined considering the actual engineering service situation of RC structures under uncommon temperature conditions, such as the concrete containment vessel and the support of the reactor vessel in the nuclear power station[2]. In this situations, temperature is not only a key factor to control the global response of RC structures, but moisture condition in concrete and its movement play also an important role.

Tab.1 presents the test condition and concrete property of each specimen. The specimens with high temperature history were heated up to 75℃. Sealed condition for "HT-sealed" was especially provided by spreading grease on concrete and wrapping with saran. The rest of high temperature specimens were

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exposed to natural drying during heating process and loading. Two specimens were dried under normal room temperature, 20℃ with relative humidity of 70%. Drying process started at 28 days age and was sustained just before loading test during different period. Free shrinkage without steel bar, measured from identical geometry with same concrete, was recorded as 350 μ for 18 days and 680 μ for 214 days from "NT-dry1" and "NT-dry2", respectively. Four pre-cracks due to severe shrinkage were observed on "NT-dry2" specimen, but no pre-crack was observed on "NT-dry1".

Material property of concrete corresponding to each environmental condition of specimens was tested using standard cylinders. Compression and splitting test at high temperature was performed within acceptable range of temperature error based on the consideration of thermal coefficient of concrete, 10 μ/℃. The yield strength and Young's modulus of steel at 20℃ was 3900kgf/cm^2 and 1967000 kgf/cm^2, respectively. Those properties were decreased at 75℃ by 3.8% and 2.4%, respectively. Thermal coefficient of steel obtained was 12 μ/℃.

Tab.1 *Test condition and Concrete Property of each specimen*

Specimen	Temp. history(℃)	Moisture condition	Weight Loss(%)	fc' (MPa)	R*	ft (MPa)	R*	Ec (GPa)	R*
NT-wet	20	Wet	0.00	39.8	1.00	3.13	1.00	28.1	1.00
NT-dry1	20	Dry	1.31	43.6	1.10	3.05	0.97	26.9	0.96
NT-dry2	20	Dry	2.19	43.3	1.09	3.08	0.98	24.2	0.86
HT-dry	20-75	Dry	2.11	40.2	1.01	2.73	0.87	23.2	0.83
HT-sealed	20-75	Sealed	0.23	37.8	0.95	2.86	0.92	28.1	1.00
H-NT-dry	20-75-20	Dry	2.10	42.9	1.08	3.58	1.14	28.0	1.00

(R* : the ratio to the value of "NT-wet")

2.2 TEST SETUP AND METHOD

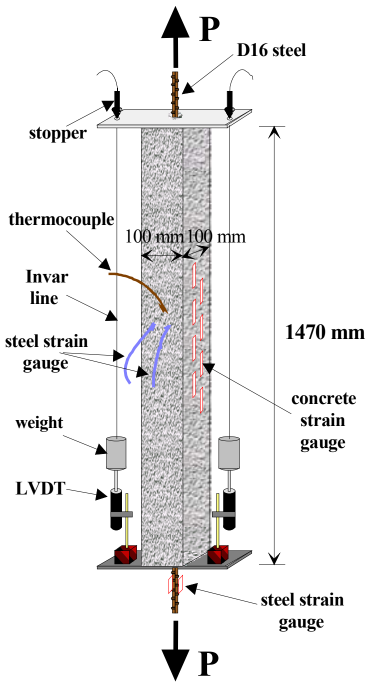


Fig.1 *Test setup*

Fig.1 shows the geometry and instrumentation for one-axial tension test specimen. All of the specimens had a length of 1470mm with unbonded zone of 50mm at both ends and a rectangular cross section of 100mm by 100mm. A single deformed reinforcing bar of D16, was provided at the center of the cross section. The reinforcement ratio ρ was 1.99% and concrete cover to bar diameter ratio c/d_b was 2.63. The diameter of reinforcing bar was selected to prevent the effect due to splitting cracks because splitting cracks are not so significant when c/d_b is larger than 2.5[1].

End plates were fixed to 50mm steel rods inserted into concrete of bond-free zone and total elongation of the specimen was obtained from the length between two end plates using displacement transducers. Steel and inner temperature was measured using multi-functional steel-strain gauge inside concrete with little effect on bond characteristics and cracking pattern. Concrete strain gauges were attached on two surfaces to obtain the local response of concrete in pre- and post-cracking region. Eight strain gauges, with 60mm gauge length, were glued at the middle part on each surface. Continuous local response of concrete was measured from strain gauges attached as zigzag pattern at the location of 1cm apart from the longitudinal centerline. For the specimens tested under high temperature, special treatment of coating was carried out to protect gauges from severe moisture movement.

The specimens heated up to 75℃, was enclosed with a thin adiabatic blanket, which played a role in mitigating external heat made from heater and transmitting it evenly to the specimen. Then, heating pad and another adiabatic blanket with much higher performance than the internal one, were installed around the specimen in due order. The outmost adiabatic blanket obstructed heat loss to the minimum and blocked up a bad influence on LVDTs.

Temperature was controlled manually with checking the gauged difference between external and internal temperature within the limits of 5℃ for preventing the unexpected effect by major cracking due to temperature gradient. Total heating time was about 6 hours for all the specimens. After attaining homogeneous target temperature in the specimen, thermally stable state was maintained for 30 minutes. No external restraint was applied to the specimens during heating and cooling process.

3. TEST RESULT

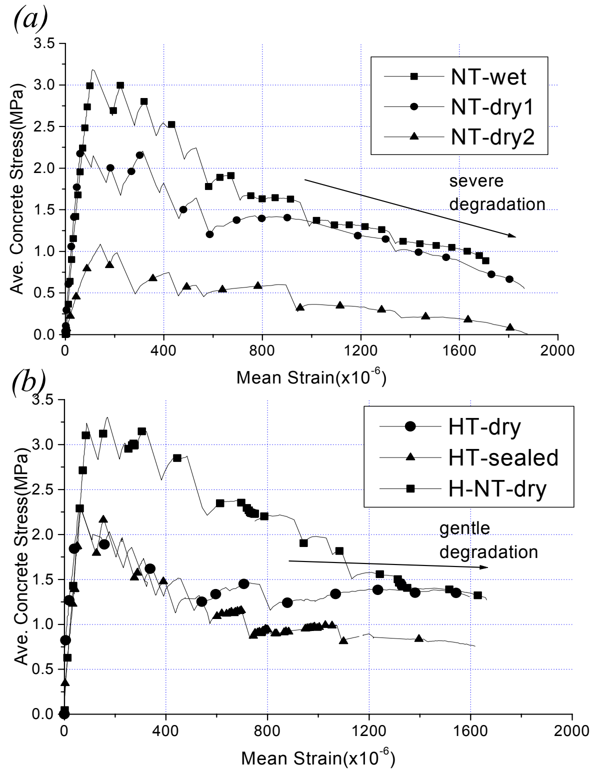


Fig.2 *Comparison of Average Stress-Strain Relationship of Concrete*
(a) Normal Temperature Specimens;
(b) Specimens with high temperature history

3.1 Average Stress-(mean)Strain Relationship of Concrete

The average stress-(mean)strain behavior of concrete(σ_c-ε_m) in the elastic steel strain range for different environmental conditions are compared in Fig. 2. The spatially averaged concrete stress was obtained by subtracting the load carried by steel bar from total load and dividing the result by the concrete cross-sectional area. The load carried by reinforcement was calculated from average strain based on the total elongation. Although all the specimens experienced one cyclic unloading and reloading process almost at every 15kN, only envelope curve is compared in Fig.2.

As shown in Fig.2, tension stiffening behavior was mainly controlled by the cracking strength of concrete. Although there was no noticeable difference in global trend for post cracking behavior, more stable response with little degradation was obtained after several major cracks had developed. In this test, major crack development was almost terminated at 800 μ of mean strain. While the averaged concrete stress decreases gradually in the stabilized cracking region in case of normal temperature specimens, further internal damage is hardly brought out after major cracking stabilized for the specimens with high temperature.

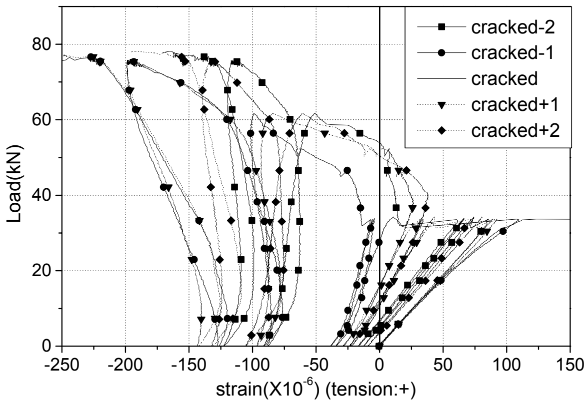
From test result, moisture loss process seems to affect the internal fracture characteristics of concrete and consequent global response of RC structure, diversely depending on temperature condition. Although severe shrinkage at high temperature was anticipated, the cracking strength of “H-NT-dry” that experienced closed temperature loop, accorded with that of concrete itself without the restraint from steel bar. Furthermore, the strength of high temperature specimens was almost identical regardless of moisture content. However, cracking strength of normal temperature specimens was severely governed by the constraint of steel against shrinkage. It means that sharply activated moisture movement in the entire concrete under high temperature builds up large amount of distributed micro-defects and large portion of restraint energy was vanished through the formation of them.

Crack numbers and its spacing were also observed. Generally, seven major cracks were occurred.

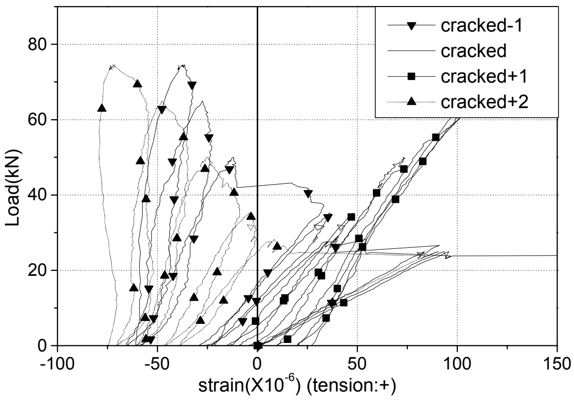
However, “HT-sealed” had 8 major cracks and 10 cracks were observed in “NT-dry2” including 4 pre-cracks due to shrinkage. Cracking pattern was a little irregular for the specimens with high temperature history, that is, the difference between maximum and minimum spacing was large. From this fact, it was considered that the bond transfer for high temperature specimens is relatively weak and the occurrence of some cracks was like the type of secondary cracking, which starts from internal cracks. However, it is very noticeable fact that tension-stiffening curve is globally dependent on only the cracking strength even when the local concrete property changes severely.

3.2 Local Concrete Behavior at the Surface

(a)



(b)



(c)

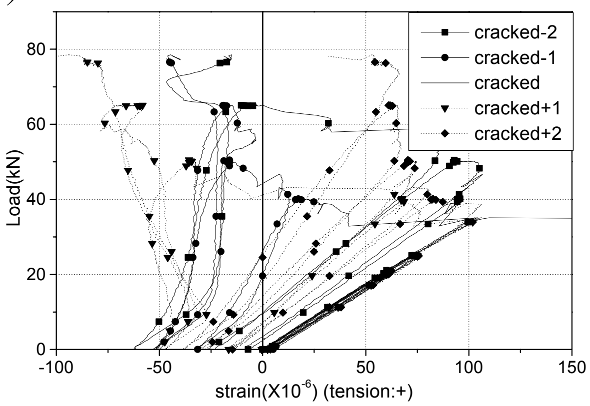


Fig.3 *Comparison of Local Concrete Response at Surface*
(a) NT-wet;(b) HT-dry;(c) H-NT-dry

Fig. 3 shows the variation of local strain at concrete surface with applied load. In Fig.3, “cracked” means the cracked section where strain jumps up and gauge fails. The strain values of each two gauges are presented in both directions longitudinally from cracked section and these gauges are indicated as “+” or “-”. For reasonable comparison, the part with almost same spacing was chosen from the total gauged length of 480mm. Before cracking, concrete shows almost linear elastic behavior uniformly in tension and the strain of non-cracked part suddenly drops when crack happens. After crack happens, the strain value gradually migrates to compressive direction with increasing the external load. This tendency is common for all the specimens. However, the degree of this tendency and the amount of residual strain value when the specimens are unloaded is very different by temperature condition. In case of normal temperature specimens, the unloaded residual compressive strain was nearly two times lager than that of specimens with high temperature history when unloaded from the same load level. With regard to the inclination of unloading and reloading curves, less movement to compressive direction is characteristic feature for the specimens with high temperature history. It is very remarkable that the local strain response of “H-NT-dry” that experienced high temperature was very similar to that of “HT-dry” tested under high temperature. “H-NT-dry” specimen, however, showed very similar behavior to “NT-wet” specimen in global tension stiffening response.

4. DISCUSSION

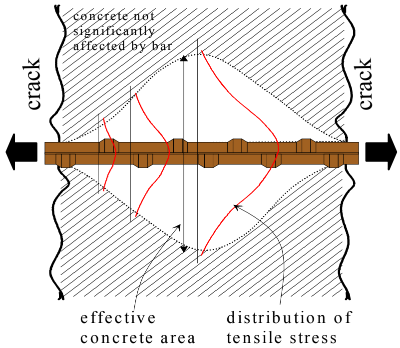


***Fig.4** Conceptual Illustration of Dispersion of Tensile Stress*

The obtained experimental results can be grasped as being attributed to varying fracture characteristics and consequent high deformability of concrete due to temperature variation. Transfer of bond force from steel to concrete between cracks can be described as illustrated in Fig.4 [3]. Tensile stress of concrete becomes large at the location closer to the center of cracked sections. Here, non-uniform stress field with gradients develops along the transverse direction. However, the effective concrete area and stress distribution inside it may be altered depending on boundary conditions, crack spacing and concrete properties, especially, fracture characteristics which is directly related to the load transfer capacity of the force from reinforcing steel.

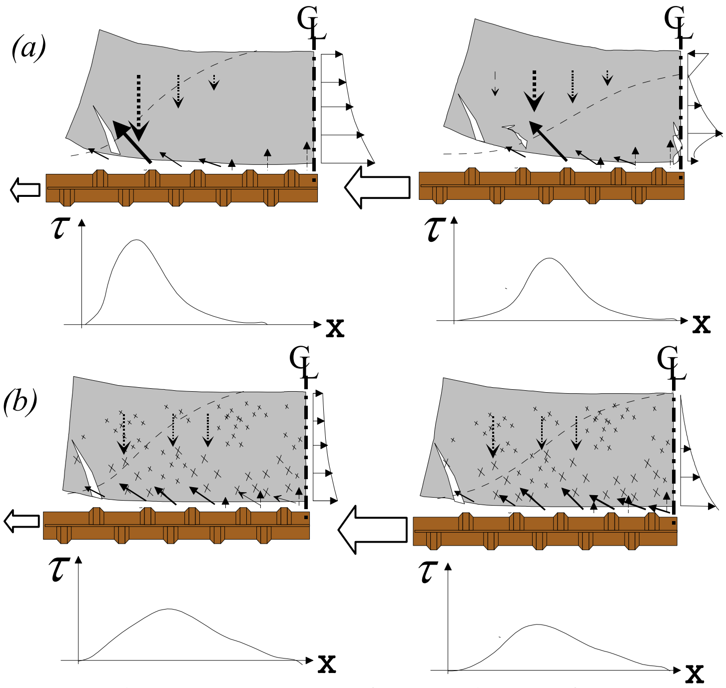


***Fig.5** Schematic Diagram for Load Transfer and Equilibrium State in Concrete*
(a) Normal Temperature Specimens;
(b) Specimens with High Temperature History

The internal equilibrium state around the interface of steel and concrete is conceptually shown with local damaging in Fig.5. Concrete in this free body diagram sustains static equilibrium by the sum of forces as shear from steel, internal tension at center section, radial component of out-of-plane resistance and steel reaction normal to longitudinal direction. Depending on the fracture property associated with micro- damage, different evolution of local nonlinearity and deformation is presented with different magnitudes of external loads after major cracking. In case of non-damaged concrete such as normal temperature specimens, the resistance of concrete to external load is very concentric near crack surface. Although such a localized resistance makes high bond stiffness and high tensile stress around middle part in cracked concrete block, it raises highly localized damage. It finally induces relatively large size of internal fracturing because of high level of stress state at some concentric points. From this reason, progressing damage is shown in tension-stiffening curve even after major cracking has been stabilized. Local failure sometimes produces one or two additional major or secondary cracking at high strain levels, which was shown around 1400 μ in this test. From the test result for dried specimens, drying under normal temperature does not seem to build distributed internal micro damage of concrete around steel bar. Moisture loss from concrete surface forms weak points rather in the exterior than inside concrete, and those can easily be a trigger of localized cracking with the effect of steel restraint. The above mechanism can be regarded as the reason of gradual degradation of tension stiffness for all normal temperature specimens without high temperature history and of large crack numbers of dried specimens.

On the other hand, initial micro-damage, which is anticipated that the specimens with high temperature history considerably includes, disperses the load carrying mechanism near steel bar with changing load transfer path. It produces lower bond stiffness but the resistance to the force from steel bar is effectively distributed with less concentration. This mechanism makes concrete entirely more deformable with relatively small size of internal fracturing. One noticeable point here is the transverse stress gradient as shown in Fig.5. Compared to the normal temperature specimens, gradient of tensile stress is much slighter even though the peak is lower. Because micro-cracks increases effective fracture toughness[6], stress concentration at the tips of relatively large size of cracks is easily released and redistributed. This makes the effective area be enlarged even though the level of tensile stress is very low. This distributed resistance mechanism can make concrete more deformable with no further severe damage even with load increase. This mechanism can be considered as the main reason for the relatively stabilized tension-resistance mechanism at high strain levels for damaged specimens by temperature.

Local strain response at concrete surface supports the above mechanical image. As shown in Fig.5(a), concentrated resistance develops steep strain gradient along transverse direction and high compressive strain at concrete surface is induced. This action is accompanied with large-size of internal damage that is highly irreversible and large residual strain consequently becomes remained. On the other hand, distributed resistance due to micro-cracks makes effective area for tension to be large with gentle transverse gradient. It makes the development of compressive strain at concrete surface less severe and large irrecoverable strains also less accumulated. Local strain response of “H-NT-dry” similar to “HT-dry” rather than “NT-dry”, can be explained by this distributed resistance mechanism induced by high temperature history.

5. CONCLUSIONS

The effect of local change of concrete fracture property on macroscopic bond behavior due to extreme environmental loads was investigated through a series of RC tension prism specimens. Generally speaking, tension-stiffening curve depended mainly on concrete strength, but the behavior in stabilized cracking region slightly changed with less further degradation due to the elevated concrete deformability caused by micro-defects. Effectiveness of deformable concrete on the serviceability and durability performance seems to be necessary to be investigated as a further study.

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