Bond Deformability of Reinforced Concrete Members

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SUMMARY:
The usual earthquake resistant design philosophy of ductile frame buildings allows the beams to form plastic hinges adjacent to beam-column connections. In order to carry out this design philosophy, the ultimate bond or shear strength of the beam should be greater than the flexural yielding force and should not degrade before reaching its required ductility. The behavior of RC members dominated by bond or shear action reveals a dramatic reduction of energy dissipation in the hysteretic response due to the severe pinching effects. After flexural yielding, plastic hinges develop near both ends of these beams and reversed cyclic loading produces a progressive deterioration of bond that may lead to failure at cyclic bond stress levels lower than the ultimate stress under monotonic loading. In addition, flexural bond stress increases with the increasing of plastic hinge length induced during positive and negative loadings. The designer has to consider the effect of bending moment and shear when designing for bond since the bond seldom acts on its own but rather in combination with flexure and shear. In this study, a method was proposed to predict the deformability of reinforced concrete members with short-span-to-depth-ratios failing in bond after flexural yielding. Repeated or cyclic loading produces a progressive deterioration of bond that may lead to failure at cyclic bond stress levels lower than the ultimate stress under monotonic loading. Accumulation of bond damage is supposed to be caused by the propagation of micro-cracks and progressive crushing of concrete in front of the lugs. The proposed method takes into account bond deterioration due to the degradation of the concrete in the post yield range. In order to verify the bond deformability of the proposed method, the predicted results were compared with the experimental results of RC members, reported in the technical literature. Comparisons between the observed and calculated bond deformability of the tested RC members, showed reasonable agreement.

Keywords: bond deformability, bond strength, slip, reinforced concrete members, cyclic loading

1. INTRODUCTION

Repeated or cyclic loading produces a progressive deterioration of bond that may lead to failure at cyclic bond stress levels lower than the ultimate stress under monotonic loading. Accumulation of bond damage is supposed to be caused by the propagation of micro-cracks and progressive crushing of concrete in front of the lugs. Degradation of bond primarily depends on the peak slip in either direction reached previously. Other significant parameters are rib pattern, concrete strength, confining effects, number of load cycles, and peak value of slip between which the bar is cyclically loaded.

Figure 1 shows the photos of the damaged buildings failing in shear or bond after earthquake. The behaviors of the RC members failing in shear or bond after flexural yielding show a reduction of ductility and energy dissipation in the hysteretic response after flexural yielding. The strength of the beam failing in shear suddenly dropped, while that of the beam failing in bond gradually decreased. After flexural yielding, the shear cracks developed diagonally to the member axis, while the bond cracks developed along the longitudinal reinforcing bars. In case of structures that deform primarily in the flexural mode, the response is governed by well-rounded hysteretic load-deformation curves because the response of such elements is governed mainly by the properties of the reinforcing steel bars. By comparison, RC members that deform primarily in the bond or shear mode show significant pinching around zero load, and severe strength deterioration in their hysteretic loops.

The AIJ (Architectural Institute of Japan) design guidelines (1990) have a provision to prevent splitting bond failure of continuous bars. The guidelines divide the design for RC members into two groups; design for a member without yield hinges or with a yield hinge at one end and design for a member with the yield hinges at both ends. The guidelines provide a design method of the combination of two bond failure mechanisms: flexural and truss mechanisms. Some researchers discussed the splitting bond failures of the RC beams subjected to severe seismic loads. For example, Ichinose (1995) studied a mechanism of splitting bond failure in RC columns and provided a way to prevent it. Masuo (1993) derived formulas for evaluating the ultimate shear strength of RC members subjected to anti-symmetrical bending moment taking into account the splitting bond strength. However, these researches did not provide a method to predict the ductile capacity of RC members failing in bond.

The research reported in this paper provides a method to predict the ductility of RC beams with short-span-to-depth-ratios failing in bond after flexural yielding. The proposed method takes into account bond deterioration due to the degradation of the concrete in the post yield range.

Figure 1. RC buildings failing in shear or bond

2. BOND FAILURE MECHANISM

The designer has to consider the effect of bending moment and shear when designing for bond since the bond seldom acts on its own but rather in combination with flexure and shear. Particularly the behaviors of RC beams dominated by shear action, such as coupling beams, are strongly related with bond force. Figure 2(a) shows the crack pattern as well as the plastic hinge regions of a RC beam subjected to reversed cyclic shear and moment, while Fig. 2(b) shows the variation in the steel stress along the span of the beam subjected to an anti-symmetric moment distribution. The bond stress due to flexure, τ_f, is measured as the rate of change in force in reinforcing bars:

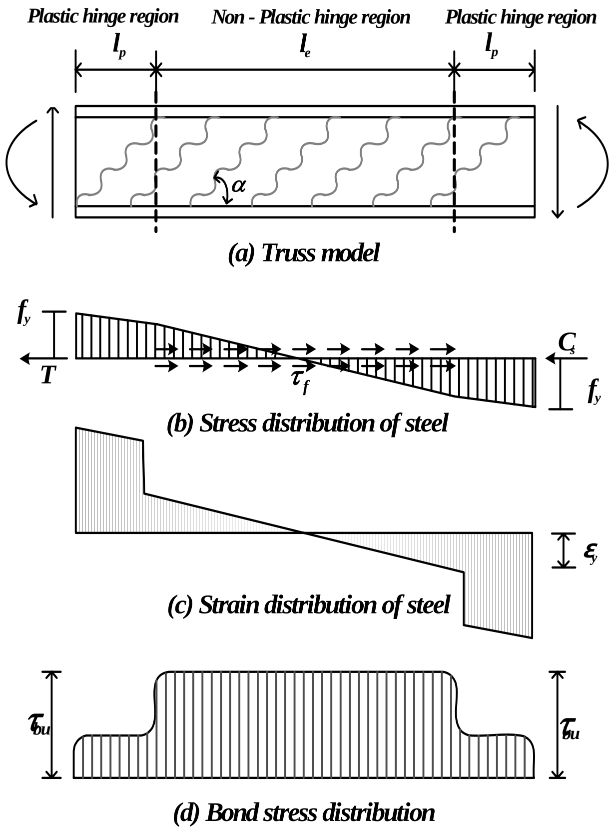


Figure 2. A truss model and strain distribution of steel bars

$$\tau_f = \frac{T + C_s}{\sum o \cdot l_e} = \frac{A_b \cdot \Delta f_s}{\sum o \cdot l_e} \tag{2.1}$$

where T and C_s are tensile and compressive forces of the longitudinal bars, respectively; $\sum o$ is the nominal surface area of the steel; l_e is the effective anchorage length of steel bars; and A_b is the sectional area of the steel bar. After plastic hinges develop near both ends of the beam, the stress, f_s, in Eq.(2.1) can be replaced by the yield stress of the steel bars, f_y.

$$\tau_b = \frac{2A_b f_y}{\sum o \cdot l_e} \tag{2.2}$$

When plastic hinges develop, the flexural bond stress, τ_f, along the longitudinal reinforcing bar increases because the effective anchorage length of steel bars, l_e, is shorten.

In order to assure a truss mechanism, bond stress, τ_t, required for this mechanism must be smaller than bond strength. The bond stress required for truss mechanism is obtained from the equilibrium of forces illustrated in Fig. 3.

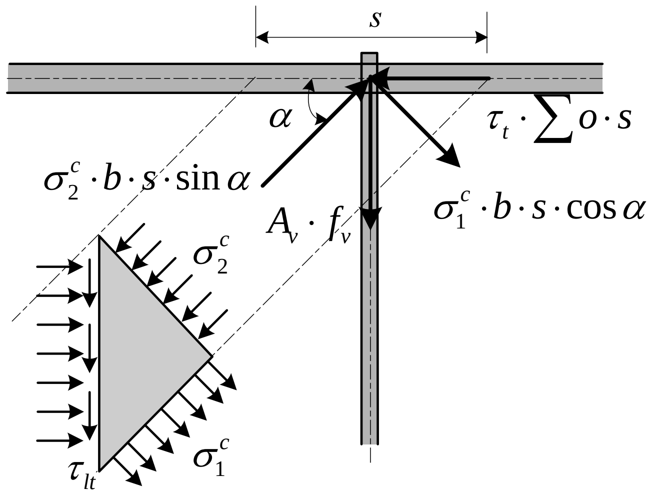


Figure 3. Equilibrium conditions of a truss model

$$\tau_t = \frac{V}{\sum o \cdot jd} = \frac{\left(\sigma_2^c \cos^2\alpha - \sigma_1^c \sin^2\alpha\right)b}{\sum o} \tag{2.3}$$

where σ_2^c and σ_1^c are the principal compressive and tensile stresses of concrete, respectively; b is the width of section; jd is the lever arm; and α is the angle formed between the direction of the principal compression stress of concrete and the longitudinal steel direction.

3. BOND DETERIORATION

Repeated or cyclic loading produces a progressive deterioration of bond that may lead to failure at cyclic bond stress levels lower than the ultimate stress under monotonic loading. Accumulation of bond damage is supposed to be caused by the propagation of micro-cracks and progressive crushing of concrete in front of the lugs as shown in Fig. 4(a) (Eligehausen et al, 1983). Cycles with reversed loading produce degradation of bond strength and bond stiffness that is more severe than the same number of load cycles with unidirectional repeated loading. Degradation primarily depends on the peak slip in either direction reached previously. Other significant parameters are rib pattern, concrete strength, confining effects, number of load cycles, and peak value of slip between which the bar is cyclically loaded.

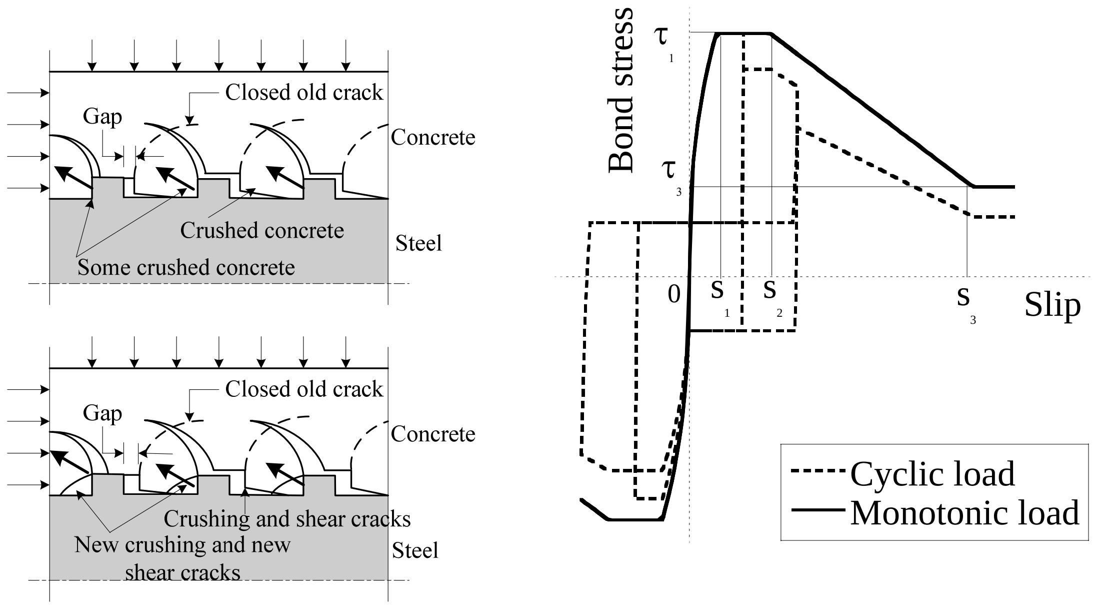
The analytical models of the local bond stress-slip relationship for cyclic loading were proposed by Morita and Kaku (1973), Tassios (1979), and Eligehausen et al. (1983). The first analytical model for reversed cyclic bond behavior was proposed by Morita and Kaku (1973). After cycling between arbitrary slip values, it is assumed that the monotonic envelop is reached at slip values larger than the peak value in the previous cycle and followed thereafter. Tassios' model(1979) is an improvement so far as the descending branch of the local bond stress-slip relationship is given and the influencing of local cycles on bond deterioration for slip values smaller than or equal to the peak slip value of the previous cycle is taken into account. Eligehausen et al.'s model (1983) takes into account the significant parameters that appear to control the behavior observed in tests. Reduced cyclic envelops are obtained from the monotonic envelop by decreasing the characteristic bond stress τ_1 and τ_3 as shown in Fig. 4(b) through reduction factors which are formulated as a function of the so called damage parameter, d. It is assumed that the damage parameter is a function of the total dissipated energy only.

4. DEFORMABILITY EVALUATION METHOD

RC members subjected to reversed cyclic load that have relatively short shear span-to-depth ratio, such as short columns or coupling beams, are inclinable to fail in shear or bond. The shear contribution of concrete of these members decreases as the number of loading cycles or the rotation of member increases, while the bond between concrete and steel bar deteriorates because of the crushing of concrete in front of the lugs. In this paper, the deflection (or rotation of member) of the RC members failing in shear or bond was calculated. The smaller value of the two calculated maximum deflections (or rotations of member) was considered as the ductile capacity of these members.

$$\Delta = Min\left[\Delta_s, \quad \Delta_b\right] \text{ or } R = Min\left[R_s, \quad R_b\right] \tag{4.1}$$

where Δ_s and Δ_b are the maximum deflection of the RC members failing in shear and bond, respectively, and R_s and R_b are the rotation of the RC members failing in shear and bond, respectively.



(a) Bond mechanism, reversed cyclic loading (b) Analytical model for bond vs. slip relationship

Figure 4. Bond mechanism and analytical model for bond vs. slip relationship (Eligehausen et al., 1983)

4.1. Calculation Method of Ductile Capacity of the Member Failing in Shear

Lee and Watanabe (2003) proposed a model to predict the shear deterioration of RC beams subjected to reversed cyclic loadings. Unlike the MCFT (1986) and the RA-STM (1988), which predict the shear strength and corresponding deformation of RC panels using the monotonic stress vs. strain curves of steel reinforcing bars, the proposed method takes into account the effect of reversed cyclic loading in the formulation by increasing the value of the axial strain, ε_l, with increasing cycles after yield. In this model, the axial strain, ε_l, in the plastic hinge region was calculated using the assumed rotation of member, R_m. Once a axial strain was assumed, the analytical shear force, V, in the beam could then be computed using the procedures of the MCFT or the RA-STM. In this approach, the maximum value of the calculated shear force V is taken as the potential shear strength of the beam V_p, which corresponds to a given fixed value of ε_l. If the value of V_p is greater than the shear value that corresponds to the formation of the plastic hinge, a new value of the longitudinal axial strain is assumed and the steps are repeated until the potential shear value V_p equals the shear force at onset of flexural yielding, V_f. When V_p reaches V_f, the corresponding deflection (or rotation) assumed was taken as the maximum deformation of the beam.

The monotonic shear force vs. deflection curve predicted according to the calculation procedure is shown in Fig. 5. The predicted relationship was compared to the observed cyclic shear force vs. deflection relationship of a beam. In Fig. 5, the potential shear strength, V_p, is obtained by interpolating the shear strengths calculated using the procedure of the RA-STM, in which the fixed values of ε_l, ***A*** through ***E***, are given. For example, when ε_l is set to 0.0025, the calculated shear strength, V_p, using the RA-STM is 280kN. Similarly, V_p, equals 125kN when $\varepsilon_l = 0.0299$. The shear strength predicted by Hsu's RA-STM was found to be larger than observed, because the magnitude of the longitudinal strain, ε_l, calculated from the monotonic stress-strain curve of the steel reinforcing bars was significantly less than observed. Using this approach, when V_p reaches the shear force at flexural yielding, defined in this paper as V_f, the corresponding deflection (or, rotation) is taken to be that corresponding to the maximum ductile capacity of the beam as shown in Fig. 5.

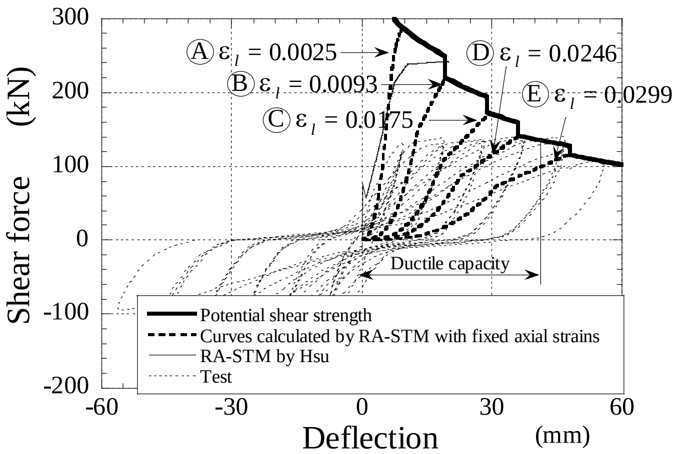


Figure 5. Potential shear strength and shear force calculated by RA-STM.

4.2. Calculation Method of Ductile Capacity of the Member Failing in Bond

The AIJ guidelines adopt a combination of two bond actions; bond action for flexural mechanism and for truss mechanism. In this study, the combination of two bond actions in the AIJ guidelines is also used. For the design of a member without yield hinges or with a yield hinge at one end, like a cantilever member, one of bond stresses due to flexure, τ_f, or due to truss mechanism, τ_t, should be less than bond strength in order to prevent bond splitting filure. However, for the design of a member without with the yield hinges at both ends, both bond stresses, τ_f and τ_t, should not more than bond strength. In the former design, the difference of steel stress, Δf_s, in Eq.(2.1) was calculated summing of the yield stress, f_y, and the stress, f_s, calculated by flexural analysis using plane remaining plane assumption. In the latter design, Δf_s, in Eq.(2.1) was replaced by $2f_y$.

The bond strength , τ_{bu}, proposed by Fujii and Morita (1982) was used.

$$\tau_{bu} = \tau_{co} + \tau_{st} \tag{4.2}$$

τ_{co} is a contribution of concrete and is given as:

$$\tau_{co} = 0.313(0.4b_i + 0.5)\sqrt{f_c'} \qquad \text{(MPa)} \tag{4.3}$$

τ_{st} is a contribution of shear reinforcement and is given as:

In case of the corner splitting

$$\tau_{st} = 0.313\frac{50A_s\sqrt{f_c'}}{s \cdot d_b} \quad \text{(MPa)} \tag{4.4}$$

In case of the side splitting

$$\tau_{st} = 0.313\frac{\left(\frac{40}{N_t} + \frac{10N_u}{N_t} + \frac{30N_s}{N_t}\right)A_s\sqrt{f_c'}}{s \cdot d_b} \quad \text{(MPa)} \tag{4.5}$$

where b_i is the coefficient related to the mode of bond failure; f_c' is the compressive strength of concrete; A_s is the sectional area of the shear reinforcement covering the corner steel; N_s is the number of the flexural steel bars directly hooked by supplemental ties; N_u is the number of the flexural steel bars not hooked; and N_t is the total number of the flexural steel bars directly hooked.

The analytical model for bond stress vs. slip relationship of concrete subjected by Eligehausen et al. (1983) was adopted in this paper.

4.3. Calculation Procedures

The calculation procedure for the proposed method is illustrated in Fig. 6. The first step in the flow chart involves the assumption of the rotation of member, R_m, which is defined as $R_m = \Delta / l$. The second step involves the calculation of the axial strain, ε_l, in the plastic hinge region of a given RC beam subjected to reversed cyclic loading using Eq. (4.6), which will be explained in details in the next sections. Before the third step, the failures are divided into two modes: bond or shear failure. In case of bond failure, the slip, S, in the plastic hinge region is calculated using the axial strain and the plastic hinge length (Lee and Oh (2005)). Once bond strength, τ_{bu}, and flexural bond stress, τ_f, are calculated in Step 5 and 6 for bond failure, respectively, the bond resistance and bond force can then be calculated as:

$$V_{bu} = \tau_{bu} \cdot \sum o \cdot jd \tag{4.6a}$$

$$V_f = \tau_f \cdot \sum o \cdot jd \tag{4.6b}$$

If the value of V_{bu} is greater than the shear value, $V_{flexure}$, that corresponds to the formation of the plastic hinge, a new value of the longitudinal axial strain is assumed and the steps are repeated until the potential shear value V_{bu} (or V_f) equals the shear force at onset of flexural yielding, $V_{flexure}$.

When V_{bu} (or V_f) reaches $V_{flexure}$, the corresponding deflection (or rotation) assumed in step 1 is taken as the maximum deformation, R_b, of the beam failing in bond.

In case of shear failure, once a transverse strain value is assumed, the analytical shear force, V, in the beam can then be computed in step 4 using the procedures of the MCFT or the RA-STM. In this approach, the maximum value of the calculated shear force V is taken as the potential shear strength of the beam V_p, which corresponds to a given fixed value of ε_l. If the value of V_p given by step 6 is greater than $V_{flexure}$, a new value of the longitudinal axial strain is assumed and the steps are repeated until the potential shear value V_p equals the shear force at onset of flexural yielding, V_f. When V_p reaches $V_{flexure}$, the corresponding deflection (or rotation) assumed in step 1 is taken as the maximum deformation, R_s, of the beam failing in shear.

The smaller values between R_b and R_s was considered as the maximum deformation of the beam with relatively short shear span-to depth ratio.

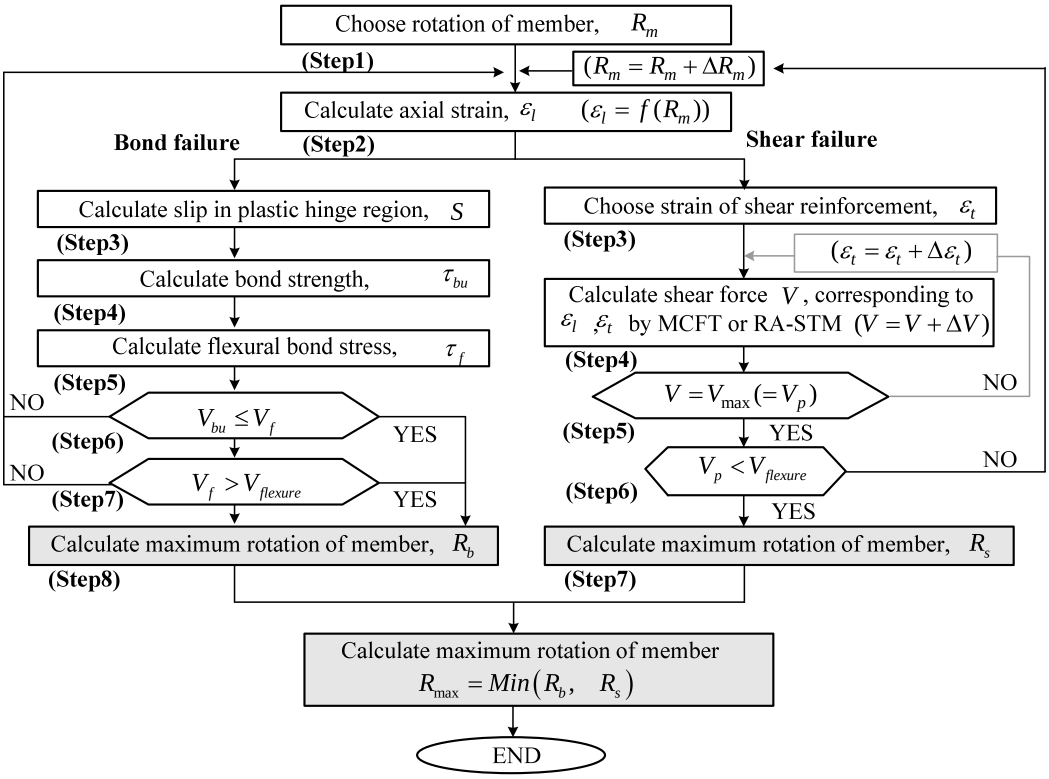


Figure 6. Calculation procedures.

5. VERIFICATION OF THE PROPOSED DUCTILE CAPACITY ANALYSIS

The calculated ductile capacity of the proposed method was verified against the observed results of two RC beams subjected to reversed cyclic loading. Both beams were designed to fail in bond or in shear after flexural yielding. Each tested specimen consisted of two regions: a test region and a loading region consisting of two loading stubs. The tested beams BB1 and BB3 were 150 mm wide and 300 mm deep. Table 1 shows the material properties of the beams in details (Choi and Lee (2012)). The specimens were loaded at 600mm far from the lower stub and one plastic hinge was developed in lower end of the beams.

Table 5.1. Specification of specimens and material properties.

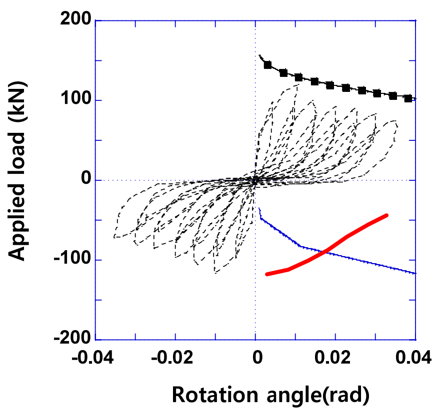
Beam	f_c' (MPa)	Tensile Longitudinal bar		Stirrup			
		f_{ly} (MPa)	ρ_l (%)	D	f_{ty} (MPa)	s (mm)	ρ_t (%)
BB1	36.2	435.0	1.97	Φ 5.5	384.0	82	0.390
BB3	36.2	435.0	1.97	Φ 5.5	384.0	143	0.224

D: nominal diameter of shear reinforcement; f_{ty} : yield stress of shear reinforcement, s : spacing of shear reinforcement; ρ_t : shear reinforcement ratio; f_{ly} : yield stress of tensile longitudinal reinforcement; ρ_l : tensile longitudinal reinforcement ratio; f_c' : compressive strength of concrete.

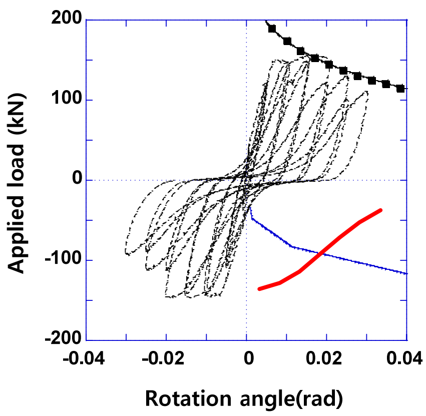
Five linear displacement transducers were attached to face of the hinge region of the test beam to measure curvature, longitudinal and transverse axial deformation as well as shear deformation. In addition, the strains of the transverse and longitudinal steel bars in the test region were measured by strain gauges attached to the surface of the steel bars.

In the test, the RC beams were loaded monotonically up to the point that corresponds to the first yield rotation, R_{em}, followed after that by a series of rotation controlled cycles in the inelastic range comprising three full cycles to each of the following specified rotations of $\pm 2R_{em}$, $\pm 3R_{em}$. The test was terminated when the resisting force in the post-peak load-deformation curve dropped to about 80% of the peak-recorded strength.

The specimens, BB1 and BB3 were failed in bond after flexural yielding. Figure 7 show the predicted and experimental load versus rotation responses for BB1 and BB2. In these figures, the predicted shear strengths were obtained according to the calculation procedure as shown in Fig. 6. The potential shear strength and bond strength of truss mechanism gradually decrease as the rotation of member increases.



(a) BB1



(c) BB2

Figure 7. Comparison between the observed and calculated rotations.

In Figs. 7(a) and (b), the shear strength calculated by shear mechanism was higher than that by bond mechanism and the ductility of beam BB1 and BB3 was determined by the ductility by bond mechanism. By comparison, Fig. 7 shows a good agreement between the observed and predicted ductility of BB1 and BB2. In addition, the proposed method correctly predicted the failure modes of two beams.

6. CONCLUSIONS

Reversed cyclic loading accelerates splitting bond failure in RC members because of the deterioration of bond and the shortening of the effective anchorage length of steel bars. This paper presented an analytical method to predict the ductile capacity of RC members failing in shear or bond. The proposed method took into account the decrease in the effective compressive strength of the concrete induced by reversed cyclic loading for shear mechanism and the deterioration of bond for bond mechanism. The calculated ductile capacity of the proposed method was verified against the observed results of 3 RC beams subjected to reversed cyclic loading. By comparison, the proposed analysis predicted the ductility of three beams with reasonable agreement.

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