- Technical Paper -

SEISMIC PERFORMANCE EVALUATION OF RC EXTERIOR BEAM-COLUMN JOINT WITH INSUFFICIENT ANCHORAGE WITH FE ANALYSIS

Murshalin AHMED*1, Rokhyun YOON *2 and Yasushi SANADA *3

ABSTRACT

In this paper, nonlinear 2-D finite element analysis was conducted on an exterior reinforced concrete (RC) beam-column joint specimen with insufficient beam longitudinal reinforcement anchorage, and the seismic capacity was predicted. The proposed model was verified by comparing the experimental and analytical results, showing that the load-deformation relationship and the failure mechanism agreed well. The present paper describes the effectiveness of modeling of the straight anchorage considering the bond-slip relationship between the reinforcement and concrete and its limitations.

Keywords: bond stress, exterior joint, FEM analysis, insufficient anchorage, pullout failure, reinforced concrete, seismic performance evaluation.

1. INTRODUCTION

In developing countries like Bangladesh, new infrastructures have been constructed at an ever-growing rate for the last four decades [1]. Proper designing is the first step in making the infrastructures resilient against seismic events. Bangladesh, which geographically lies in an active seismic zone, only introduced its national building code (BNBC) [2], in 1993. Before that ACI code was followed for the construction of new buildings. However, the government only started the implementation of BNBC, strictly in 2006. As a result, at present many buildings exist in Bangladesh are still struggling to meet the design standards specified in the national building code [3].

One example of such sub-standard constructions is the presence of insufficient and/or straight anchorage in the exterior beam-column joints. In a previous study, Wardi et al. [4] experimentally observed that such improper seismic detailing in the exterior joint exhibited unanticipated pre-mature pullout failure. RC buildings are designed considering many failure mode conditions. However, such kind of uncommon failure mode is not considered during the general design of a building.

The hypothesis behind the pullout failure mechanism of the exterior beam-column joint is, insufficient anchorage length of beam longitudinal reinforcement prevents the reinforcement from reaching its maximum tensile load-resisting capacity and leads to slippage of reinforcement due to bond failure between reinforcement and concrete. It is difficult to properly understand this failure mechanism and evaluate its seismic performance accurately due to the limited research in this field.

In this study, the exterior beam-column joint comprising of insufficient beam longitudinal reinforcement is modeled using a non-linear two-dimensional finite element method utilizing the concept of bonding between reinforcement and concrete and its deterioration with increasing load as well as deformation of the joint. The results are compared with the previous experimental test [4]. Successful modeling of beam-column joints with deficient anchorage is the first step toward understanding the complications that can arise from such improper design detailing.

2. SUMMARY OF EXPERIMENTAL TEST

The subject experimental test was performed at Osaka University by Wardi et al. [4]. The details and the results of the experimental test are explained in reference [4]. The parameters and results necessary for this study are summarized in this section.

2.1 Specimen Details

An experimental test was performed on a 70% scaled specimen representing the exterior beam-column joint of an existing RC building in Bangladesh. The specimen details are shown in Fig. 1, and the specifications of the specimen are shown in Table 1. The total height of the specimen was 2100 mm, and the length of the beam was 1125 mm. Insufficient anchorage was considered for the beam longitudinal reinforcement. Deformed rebar was used as the main and shear reinforcement during the specimen construction. The embedment length of the beam longitudinal reinforcement, anchored straightly, was considered 235mm, full depth of column. This embedment length represents 81% of the required embedment length by Bangladeshi National Building Code (BNBC) [2].

*1 Graduate Student, Graduate School of Engineering, Osaka University, M.E., JCI Member
*2 Assistant Prof., Graduate School of Engineering, Osaka University, Dr.E., JCI Member
*3 Professor, Graduate School of Engineering, Osaka University, Dr.E., JCI Member

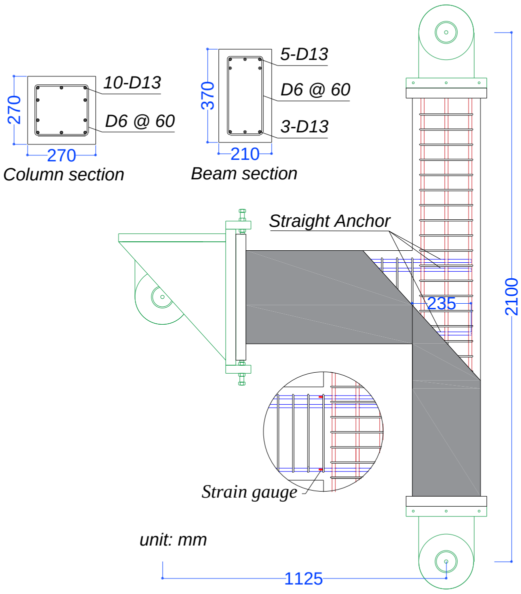


Fig. 1 Specimen details (unit: mm)

Table 1 Specification of the specimen

Column	b x D	270 mm x 270 mm
	Main rein.	10-D13
	Shear rein.	D6@60
Beam	b x D	210 mm x 370 mm
	Main rein.	8-D13
	Shear rein.	D6@60
Concrete com. strength/Steel yield stress	Concrete	11.0 N/mm^2
	D13	342 N/mm^2
	D6	408 N/mm^2
Young's Modulus	Concrete	8.7 kN/mm^2
	D13	174 kN/mm^2
	D6	171 kN/mm^2

2.2 Test Setup and Loading Program

The experimental test setup is shown in Fig. 2. The specimen was attached to the testing frame using pin hinges at column ends and a roller support at the beam end. To measure the shear force on the beam, a load cell was incorporated in the roller support. Two vertical jacks were used to apply a constant axial force of 72 kN (10% axial stress ratio) during the experiment. A horizontal jack was used to apply the reversed cyclic lateral loading on the specimen, as shown in Fig. 2. The lateral loading program is shown in Fig. 3.

2.3 Test Results

Figure 4 shows the experimental test results. The joint moment was obtained by multiplying the shear force measured by the load cell at the beam end and the distance between the column center and roller center at the beam end. Diagonal cracks appeared at the joint region in ± 0.5% rad drift cycle. The bottom beam reinforcement at the column end yielded during the drift cycle of – 0.75% rad. Specimen reached the maximum strengths at ± 1.5% rad drift cycle followed by an expansion on diagonal cracks to the upper column along

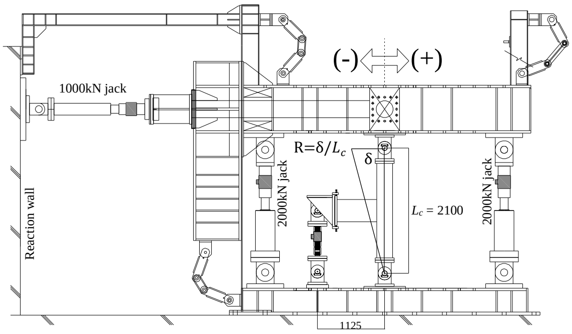


Fig. 2 Test set-up (unit: mm) [4]

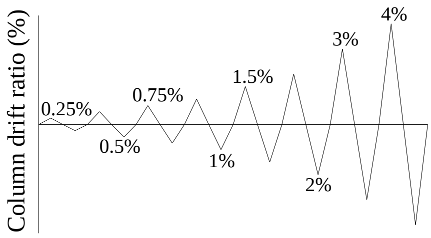
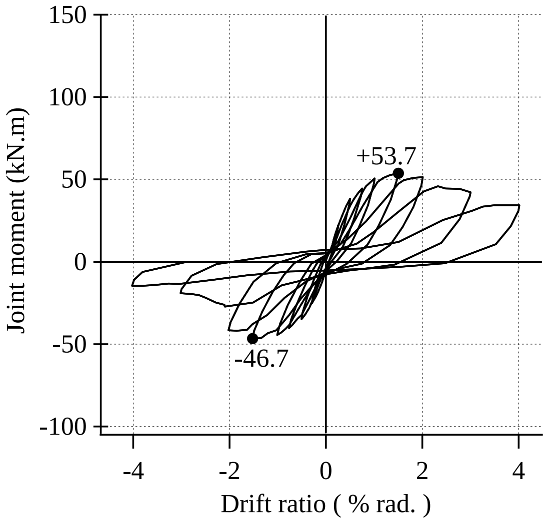
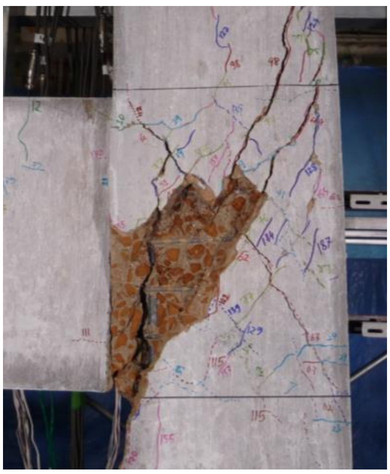


Fig. 3 Lateral loading history [4]



(a) Joint moment vs drift ratio relationships



(b) Final damage state

Fig. 4 Experimental test results [4]

the external longitudinal bars, indicating joint shear failure in the positive direction. While in the negative loading cycle, a wide vertical crack was observed on the beam-column joint face indicating anchorage failure of beam longitudinal reinforcement due to inadequate development length. In the subsequent negative loading cycle, a significant reduction in strength showed that anchorage failure was much more brittle in nature than the joint shear failure observed in the positive loading direction. Fig. 4b shows the final damage state of the specimen.

3. FEM ANALYSIS

3.1 FE Model Detail

The specimen to be analyzed was the specimen of the previous study [4], as shown in Fig. 1. An image of analytical modeling is shown in Fig. 5. Concrete and pin supports were modeled by four-node iso-parametric quadrilateral elements in 2-D discretization. Six degrees of freedom in each node were utilized for modeling quadrilateral elements. The pin supports at the ends of the beams and column were assumed to be rigid. Longitudinal reinforcements in the beam and the column were modeled by two-node discrete truss elements. The bond behavior was modeled by introducing a linkage element on the interface between the truss element and the concrete element. All other reinforcing bars were modeled by embedded elements, assuming the perfect bond. The blue-colored part shows the rigid steel section representing pin supports, as shown in Fig. 5. Concrete is shown in the grey-colored section. Longitudinal reinforcement in beam and column is shown by the red-colored section in Fig. 5.

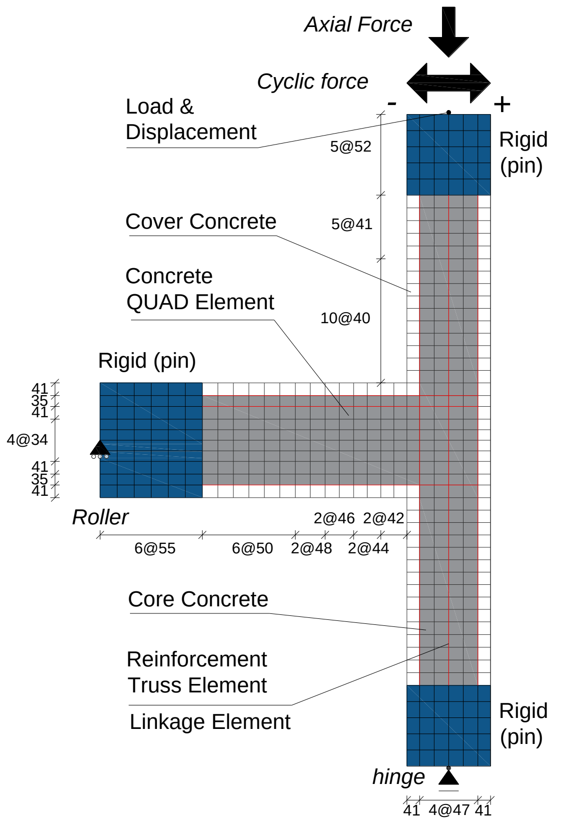
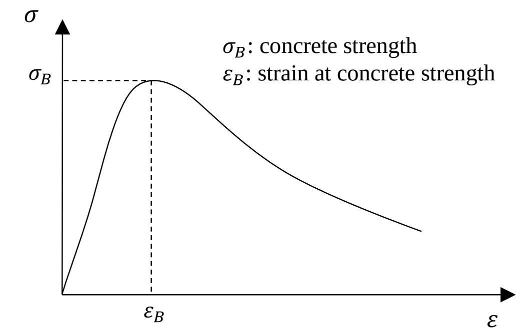


Fig. 5 FE model details (unit: mm)

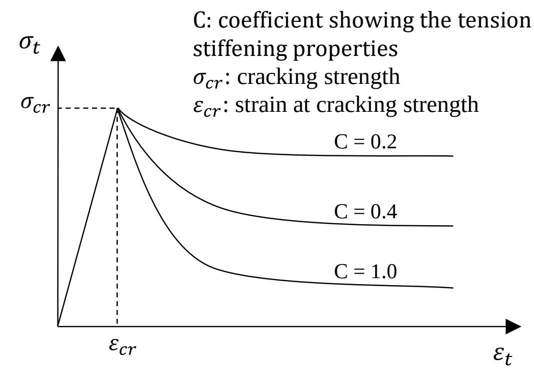
3.2 Material Modeling

(1) Concrete modeling

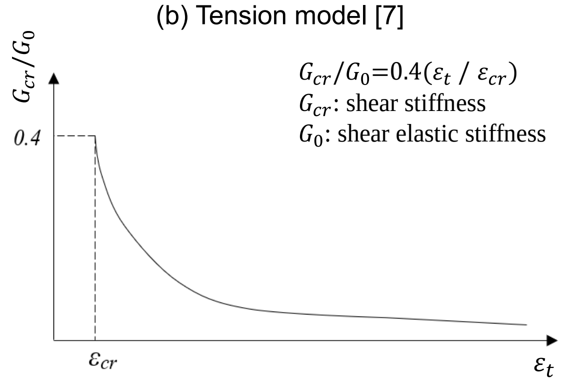
Figure 6 shows the material constitutive law of concrete. The compressive fracture energy was determined according to the Kupfer-Gerstle proposal [5]. The modified Ahmed model [6] was used to define the compressive stress-strain curve of the concrete, shown in Fig. 6a. In the tension zone, the stress-strain relationship is assumed to be linear up to cracking. Tension stiffening characteristics of concrete were considered and modeled according to Izumos' model [7], shown in Fig. 6b. C = 0.4 was used for core concrete and C = 1.0 was used for cover concrete. Shear transfer characteristics of concrete after cracking were modeled according to Al-Mahaidi's model [8], as shown in Fig. 6c. The hysteresis model for concrete under cyclic stress was modeled by Naganuma's model [9].



(a) Compression model [6]



(b) Tension model [7]



(c) Shear transfer model [8]

Fig. 6 Constitutive laws for concrete model

(2) Reinforcement modeling

The reinforcing bar was treated as an elastoplastic material. The average stress-strain relationship of the reinforcing bars maintains an elastic relationship for unloading/reloading until yielding. After the yielding of the reinforcing bars, the relationship loses its elasticity and becomes nonlinear. The reinforcing bar's constitutive law was modeled according to the modified Menegotto-Pinto model revised by Ciampi [10], shown in Fig. 7. Zieglar's kinematic hardening law [11] was used for defining strain hardening.

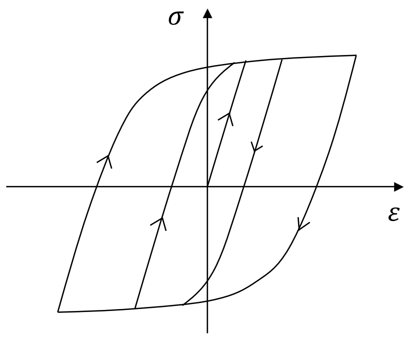


Fig. 7 Hysteresis model for reinforcement [11]

3.3 Bond-Slip Relationship

Bond-slip relationship between reinforcement and concrete was expressed as shown in Fig. 8. Naganumas' model [12] was used for defining the bond stress-slip characteristics between reinforcement and concrete for this FE model. Where, τ_u is the peak bond stress and S_u is the slip corresponding to peak bond stress. The bond-slip corresponding to the peak bond stress was assumed as 1.0 mm, referring to the study by Elmorsi [13]. Pullout condition was considered after the specimen reached peak bond strength and the bond strength degraded to 40% of the peak bond stress with a larger amount of slip. A curve model was used for the hysteresis model of the bond-slip relationship under cyclic stress.

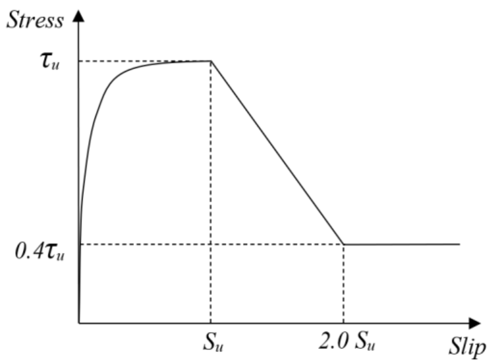


Fig. 8 Bond-slip model

The maximum bond stress value is required in the Bond-Slip model. It was obtained from the two independent method. Firstly, from the design guideline formula of the Architectural Institute of Japan and secondly, from experimental test results.

(1) Calculated value (AIJ guideline [14])

Equation 1 was used to calculate the maximum bond stress value according to the Japanese Design Guidelines [14].

$$\tau_u = 0.7(1 + \sigma_0/\sigma_B){\sigma_B}^{2/3} \qquad (1)$$

where, τ_u is maximum bond stress; σ_0 is axial stress of column; σ_B is the compressive strength of concrete.

(2) Experimental value

The maximum bond stress value was estimated indirectly from the previous experimental test [4] using the strain value of the reinforcement at the time of pullout failure. The averaged maximum bond stress can be calculated by the following equations (Eq. 2 to 3) using the tensile force of reinforcement. The reinforcement strain of the critical section was measured during the experiment using strain gauges, as shown in Fig.1. It was assumed that reinforcements behaved elastically until reaching the yielding limit. Finally, Equation 4 was used to find the averaged maximum bond stress τ_u acting on the reinforcement at the time of pullout failure.

$$F_t = \sigma \cdot A_s = E_s \cdot \varepsilon \cdot \pi \cdot (d^2/4) \qquad (2)$$
$$F_b = \tau_u \cdot \pi \cdot d \cdot l \qquad (3)$$
$$\tau_u = (E_s \cdot \varepsilon \cdot d)/(4 \cdot l) \qquad (4)$$

where, E_s is the modulus of elasticity; σ is the stress on the reinforcement at the beam critical section; ε is the strain on the reinforcement at the beam critical section, this was found from the strain gauge data during the experimental test; F_t is the tensile force acting on the reinforcement at the beam critical section; F_b is the gross bond force acting on the embedded reinforcement; d is the diameter of the reinforcement; and l is the embedment length of the reinforcement.

The value of peak bond strength obtained by Eq. 1 and Eq. 4 are 3.77 N/mm^2 and 5.35 N/mm^2 respectively. The bond stress from Eq. 1 is a design guideline recommendation, hence, conservative estimation compared to actually observed value by the experiment through Eq. 4.

3.4 Loading Program

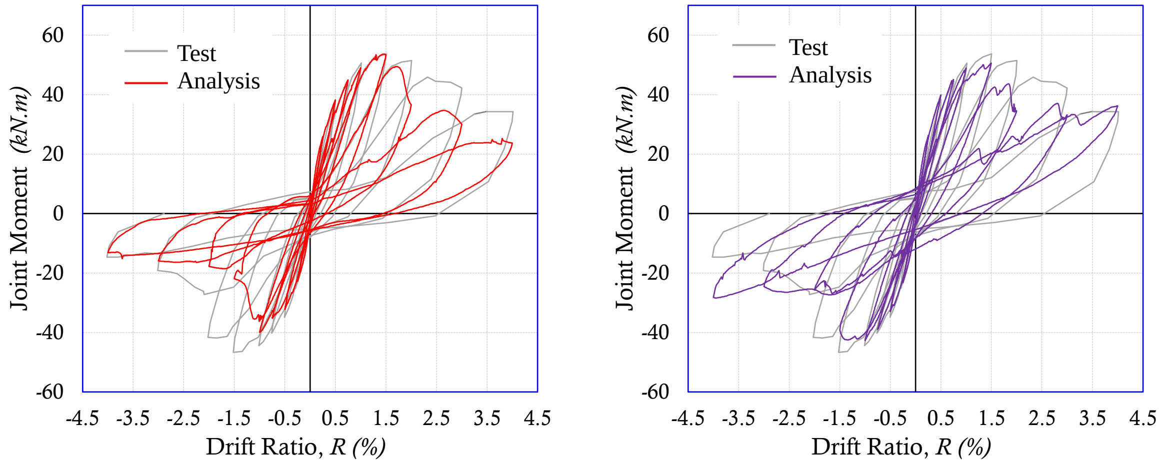
A constant axial loading of 72 kN was applied on the top of the FE model specimen. The applied vertical load was the same as the previous experimental test. The cyclic lateral load was applied through displacement control. Displacement was measured directly at the top of the FE model specimen at the column centerline, as can be seen in Fig. 5. The applied lateral loading program was the same as the previous experimental test as shown in Fig. 3.

4. FE ANALYSIS RESULT AND DISCUSSION

Figures 9, 10 and 11 show FE analysis results and compare it with the previous experimental test results.

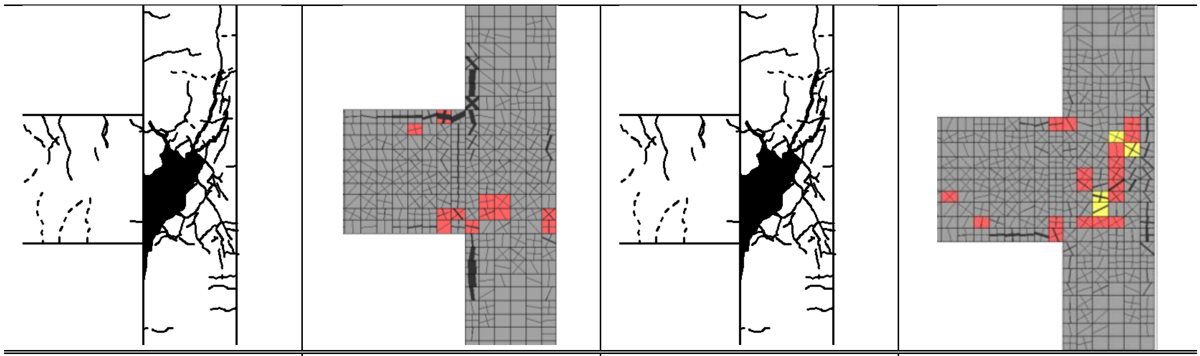
(1) FE analysis using calculated (design) bond value

The analytical specimen reached a maximum strength of 53.6 kN.m in the positive loading direction (99.8% of the experimental test) and -40 kN.m in the negative loading direction (85.7% of the experimental test) during the cyclic analysis, as shown in Fig. 9a. A large slip was observed in both positive and negative



(a) Calculated value of τ_u (Eq. 1) (b) Experimental value of τ_u

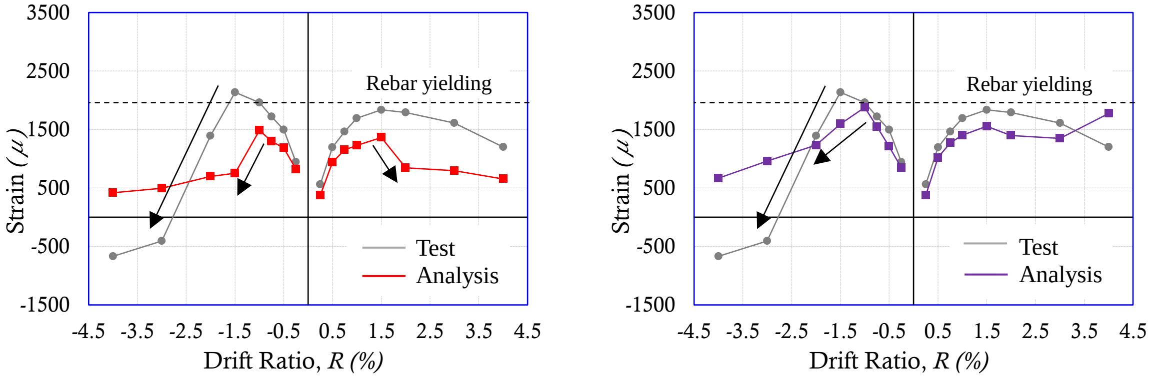
Fig. 9 Joint moment vs. drift ratio relationships and comparisons with the experimental results



(a) Calculated value of τ_u (Eq. 1) (b) Experimental value of τ_u

Element soften by compression, Element previously softened by compression

Fig.10 Comparisons between the experimental and analytical cracking patterns at final damage state (+/- 4.0%, peak drift)



(a) Calculated value of τ_u (Eq. 1) (b) Experimental value of τ_u

Fig. 11 Reinforcement strain at the critical section

loading directions. In the positive loading direction beam reinforcements slipped at +2.0% drift cycle. On the other hand, in the negative loading direction beam reinforcements slipped at -1.5%, as shown in Fig. 10a and Fig. 11a. The FE analysis was successful in predicting the pullout failure mode in the negative loading direction. However, the FE analysis also showed pullout in the positive loading direction whereas pullout failure was not observed in the positive direction during the experiment. No reinforcements yielded in the FE analysis and strain observed in the corresponding cycles was lower than that of the experiment.

(2) FE analysis using experimental bond value

The analytical specimen reached a maximum strength of +50.6 kN.m in the positive loading direction (94% of the experimental test) and -42.7 kN.m in the negative loading direction (97.6% of the experimental test) during the analysis, as shown in Fig. 9b. The maximum strengths were obtained at ± 1.5% rad drift cycle similar to experimental test. The reinforcement strain, shown in Fig. 11b, was also similar to the experimental test. In the positive loading direction reinforcement strain maintained a relatively similar value after reaching the maximum strength. In contrast, in the negative loading direction reinforcement strain dropped suddenly indicating pullout failure, as shown in Fig. 11b. In addition, the cracking diagram generated by the FE analysis was also roughly similar to the experimental test in the beam-column joint region, as shown in Fig. 10b. The experimental results could be accurately simulated by using the bond stress obtained from the experimental test.

The FE analysis utilizing bond value calculated by AIJ Guidelines conservatively estimated the beam-column joint behavior. The result is understandable because the AIJ Guidelines formula provides a conservative estimation of bond stress to ensure safety margin for design. In real conditions, the bond stress value is likely to be higher than estimated by AIJ Guidelines value. Whereas, FE analysis utilizing bond value from the experimental test simulated the seismic performance of the exterior beam-column joint with satisfactory accuracy.

5. CONCLUSIONS

An exterior beam-column joint with insufficient beam longitudinal reinforcement anchorage was modeled using Finite Element Method to replicate the characteristics of a previous experimental test. The major findings are summarized below.

(1) FE analysis showed pullout failure of beam longitudinal reinforcement with insufficient anchorage can be simulated by considering bond behavior between reinforcement and concrete.
(2) Bond stress calculated using the AIJ Design Guidelines, underestimates the exterior beam-column joint characteristics in terms of failure mode and maximum lateral strength estimation.
(3) Bond stress estimated from the sub-assembly experimental test was able to simulate the exterior beam-column joint characteristics well.
(4) Bond stress value has a significant contribution to the seismic performance of exterior beam-column joints with insufficient anchorage.

The FE model described in this study can be used to perform the seismic performance evaluation of exterior beam-column joints with insufficient beam longitudinal reinforcement anchorage.

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