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Seismic Assessment of Beam-to-Column Interaction Utilizing Headed Bars

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ABSTRACT :

Experimental research was performed to evaluate the applicability of headed bars with small heads in exterior beam-column joints. A total of twelve pullout tests and two, full-scale, cyclic beam-column joint tests were carried out to examine pullout and seismic anchorage behavior of headed bars. The variables selected for pullout tests were the loading condition, the head type (size and shape), and the technique to attach the head to the bar end. The pullout test results revealed that all types of head sizes/shapes and head-attaching techniques achieved the proper anchorage under monotonic and repeated loads. The seismic tests indicated that both beam-column joints failed in a ductile manner, exhibiting similar patterns of cracking and the same level of lateral strength. The joint using headed bars showed better seismic performance in terms of the damage extent, lateral drift capacity and energy dissipation, compared with that using hooked bars. These experimental results demonstrate that headed bars with small heads ($A_{brg}/A_b \approx 2.6$) could be used as an alternative to hooked bars for the design of exterior beam-column joints.

KEYWORDS: Headed bars; beam-column joints; pullout; seismic performance; cyclic loading

1. INTRODUCTION

In reinforced concrete structures, use of 90-degree standard hooks is common when insufficient embedment depth is available for straight bar anchorage. The development length in tension for standard hooks (l_{dh}) ranges only approximately 30 to 50% of that for straight bars (l_d). However, the bends and tails of the hooked bars tend to create reinforcing congestion, particularly in a region where all beam and column main bars pass through or terminate (e.g., exterior or knee joint). The congestion problem gets even worse with a quite large amount of joint transverse reinforcement that is demanded for earthquake-resistant structures. Reinforcement has to be simplified without sacrificing structural performance for better concrete placement. As such, headed reinforcement is quickly becoming a preferred means for anchorage and development. Simplified reinforcing detailing eventually will lead to reduced construction time with consequent huge savings in costs.

New provisions for the development length and details for headed bars have been added to ACI 318-08. The development length in tension for headed bars (l_{dt}) is defined as:

$$l_{dt} = (0.191\psi_e f_y d_b) / \sqrt{f'_c} \geq \text{the larger of } 8d_b \text{ and } 152 \text{ mm} \tag{1}$$

where f_y is the specified strength of headed bars in MPa unit, f'_c is the design concrete strength in MPa unit, d_b is the bar diameter, and $\psi_e = 1.2$ for epoxy-coated reinforcement and 1.0 for other cases. Eqn. (1) results in the development length of approximately 80% of that required for hooked bars by the ACI 318-08 code. A reduction factor of (A_s required)/(A_s provided) may be applicable to Eqn. (1). Although it has been observed that the head size and shape influenced anchorage capacity (Wallace et al., 1998; Thompson, 2002; Chun et al., 2007), Eqn. (1) is not the function of those terms. Rather, they are indirectly accounted for in ACI 318-08.

For the joint design, ACI 352R-02, "Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures," can be used, where the development length of l_{dt} is defined as:

$$l_{dt} = (0.179 f_y d_b) / \sqrt{f_c'} \text{ for Type 1 joint ; } (0.152 f_y d_b) / \sqrt{f_c'} \text{ for Type 2 joint} \quad (2)$$

In Eqn. (2), a stress multiplier is included to account for over-strength and strain hardening of reinforcement (i.e., $\alpha = 1.25$). If spacing of joint transverse reinforcement is less than or equal to $3d_b$, Eqn. (2) is multiplied by 0.8. In ACI 352R-02, Type 2 joint is defined as the joint subjected to moderate-to-high seismic risks, while Type 1 joint is defined as the joint subjected to low seismic risk. As noted in Eqn. (2), the development length (l_{dt}) given by Eqn. (2) is taken as 75% of l_{dh} of ACI 352R-02, yielding approximately 80% of that given by Eqn. (1). Eqn. (2) was developed based on various test results (ACI 352R-02), and was targeted towards the special case of beam-column joints. The shorter development length from Eqn. (2) versus Eqn. (1) appears to be reasonable, given the fact that substantially higher concrete breakout (or bearing) capacity exists for headed bars anchored within the diagonal strut of the well-confined joint (Chun et al., 2007). As well, joint and column transverse reinforcement plays a role in preventing brittle concrete breakout (Choi, 2006).

Use of the circular head with (A_{brg}/A_b) of approximately 3 to 4 is common, where A_{brg} is the net bearing area of the head and A_b is the bar area. The (A_{brg}/A_b) of 4 is the minimum head size limit specified by ACI 318-08. Prior experimental research (Chun et al., 2007) has shown that this head size is appropriate to ensure anchorage, substantially reducing steel congestion. In this study (that was planned when no clear provisions existed), the head size was investigated as one of main research variables to provide design guidelines for the use of headed bars. Both circular and square head shapes were examined to observe the impact of head shape on anchorage.

Along with two companion experimental studies (Choi et al., 2002; Choi, 2006) where several parameters affecting anchorage strength were investigated, an attempt was made to find a quantitative answer to the question of which combination of development length and head size should be used. This study aims to test the general applicability of headed bars with small heads in exterior beam-column joints.

2. EXPERIMENTAL PROGRAM

Table 1 Measured steel material properties

	d_b [mm]	E_s [GPa]	f_y [MPa]	ε_y [strain]	f_u [MPa]
D10	10	204	570	0.028	697
D19-Pullout	19	181	465	0.026	721
D19-JH	19	200	481	0.024	575
D19-JK	19	200	460	0.023	580
D25	25	204	407	0.020	602

2.1. Materials

Headed deformed bars with a bar diameter of 19 mm (D19) were used in this study. Three types of head geometries used for pullout tests are detailed in Table 1. For reversed cyclic tests of the beam-column joint, small circular heads (CS; see Table 2) were chosen based on the results of the pullout tests. All headed bars and heads were made of steel with a specified yield stress (f_y) of 400 MPa.

Measured material properties of steel and concrete are summarized in Table 1. Headed and hooked bars used for the seismic tests had similar actual yield strengths ($1.2f_y$ and $1.15f_y$, respectively, where f_y is the specified yield strength, 400 MPa). Different concrete mixes were used for pullout tests and seismic tests. For each test, two and three concrete cylinders were tested and averaged (35.2 and 29.1 MPa), respectively.

2.2. Pullout tests of single-headed bars

In the prior pullout tests on various embedment depths (Choi et al., 2002), an embedment depth (h_d) of $10d_b$

was shown to be sufficient to develop the yield strength of the single-headed bars, along with minimum head size of (A_{brg}/A_b = 3). In the new study, the same embedment depth (h_d) of $10d_b$ (190 mm) was used to examine other variables such as head types (see Table 2), head-attaching techniques (welding vs. threading) and loading conditions (monotonic vs. repeated). Table 2 summarizes the parameters tested.

Table 2 Summary of test parameters and results for pullout tests

I.D.	Loading type	Head attachment	Head type	Head type	f'_c [MPa]	h_d [mm]	L_{peak} [kN]	L_{peak}/f_y
Mnt-CST	Monotonic	Threading	CS	CS	35.1	190	165	1.13
Mnt-CLT	Monotonic	Threading	CL	CL	35.1	190	171	1.28
Mnt-SQT	Monotonic	Threading	SQ	SQ	35.1	190	163	1.18
Mnt-CSW	Monotonic	Welding	CS	CS	35.1	190	148	1.11
Mnt-CLW	Monotonic	Welding	CL	CL	35.1	190	196	1.47
Mnt-SQW	Monotonic	Welding	SQ	SQ	35.1	190	152	1.14
Mnt-NH1	Monotonic	N.A.	N.A.	N.A.	35.1	285	115	0.86
Mnt-NH2	Monotonic	N.A.	N.A.	N.A.	35.1	285	104	0.78
Rp-CST	Repeated	Threading	CS	CS	35.1	190	144	1.08
Rp-CLT	Repeated	Threading	CL	CL	35.1	190	181	1.35
Rp-CSW	Repeated	Welding	CS	CS	35.1	190	142	1.06
Rp-CLW	Repeated	Welding	CL	CL	35.1	190	179	1.34

Note: CS; Circular small; CL; Circular large; SQ; Square; NH = No head; f'_c = measured concrete strength; h_d = embedment depth measured from the concrete surface to the bearing face of the head; L_{peak} = peak pullout load monitored; f_y = measured yield stress of steel

For 4 of 12 specimens, three cycles of loading and unloading in tension were repeated at the stress levels of 0.25, 0.5, 0.75, 1.0, and $1.25f_y$, where f_y is the measured yield stress of steel (465 MPa). This was done to analyze the difference in anchorage behaviors under different loading conditions. Subsequent to the repeated loading steps, monotonic pullout loading was applied until failure (i.e., loss of pullout load capacity).

Each specimen consisted of a single-headed bar embedded in the concrete block with dimensions of 700 x 700 x 700 mm. The headed bar was subjected to tension using a hydraulic jack with a capacity of 500 kN. A load cell was located under the jack to record the tension force. The head slips at the back of the head and at the loaded end were measured using Linear Variable Displacement Transducers (LVDTs). For attachment of the LVDT to the head, a 5 mm diameter steel rod was welded to the back surface of the head. The rod was then inserted inside the PVC tube, which was embedded in the concrete. Two strain gauges were mounted on each headed bar at the location of 50 mm away from the concrete surface (in the air).

2.3. Reversed cyclic tests of beam-column joints

To evaluate the application of headed bars with small heads in exterior beam-column joints as compared to hooked bars, cyclic subassembly tests were conducted. Two, full-scale, joint subassemblies were constructed; one specimen with headed bars (JH) and the other with 90-degree hooked bars (JK). Figure 1 shows reinforcement and dimensions of the specimens. The story height was 3.6 m, and the center-to-center span length was 5.25 m. Reinforcement was detailed such that beam yielding would occur prior to joint failure (based on a joint shear capacity of ($1\sqrt{f_c'}$ x b_jh), where b_j is the effective joint width and h is the joint depth). The same bar size (D19) used in pullout tests conducted as part of this study was also used for seismic tests. The small head size (A_{brg}/A_b = 2.6), circular head shape, and threaded head-to-bar connection were chosen based on the observations of new and prior companion pullout tests (Choi et al., 2002; Choi, 2006).

2.3.1 Anchorage and development

The development length (l_{prov_1}) provided for JH specimen is 285 mm ($15d_b$ or $0.168f_yd_b/\sqrt{f'_c}$), measured from the beam-joint interface to the bearing face of the head. For JK specimen, l_{prov_1} also is 285 mm ($15d_b$ or

$0.176 f_y d_b/\sqrt{f'_c}$), measured from the interface to the outside edge of the hook. The dimension used for JH is similar to that ($15d_b$ or $0.168 f_y d_b/\sqrt{f'_c}$) needed to develop $1.2f_y$ of multiple-headed bars in the prior pullout tests (Choi, 2002), but smaller than that ($0.191 f_y d_b/\sqrt{f'_c}$) required for a headed bar embedded in normalweight concrete by ACI 318-08, §12.6.1 (new provision).

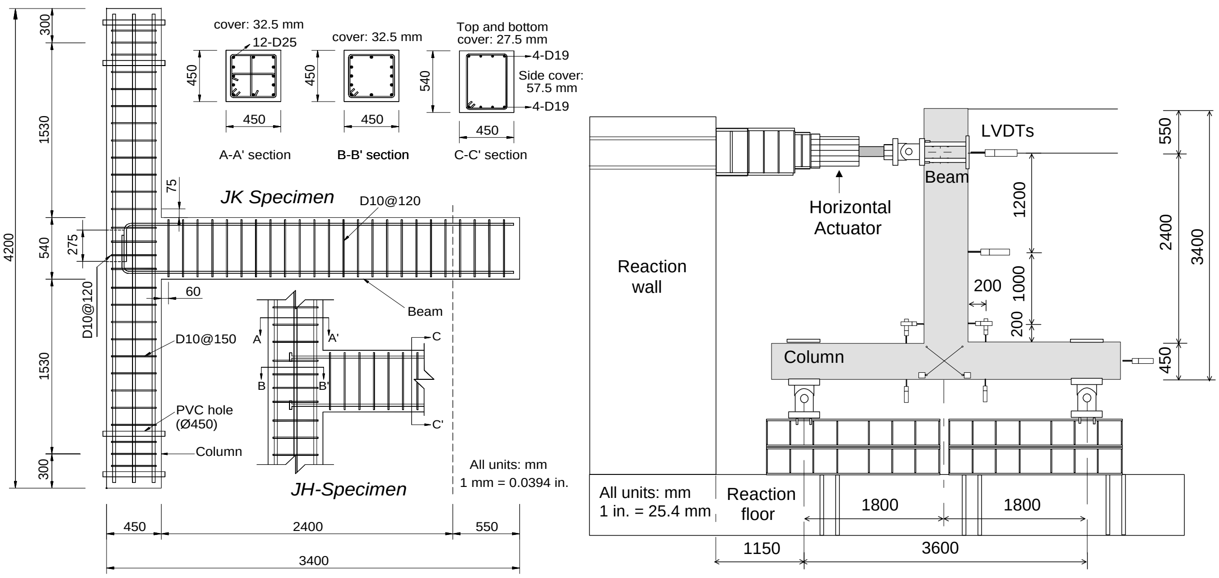

Figure 1 Joint test specimen and test setup

For Type 2 joints, because the critical section should be considered located at the edge of the inner part of the joint core, the effective development length (l_{prov_2}) is equal to 257.5 mm (285 mm – clear cover of 27.5 mm) for JH. It should be noted that the definitions of l_{dt} are different in ACI 318-08 and 352R-02. The former defines l_{dt} taken as the length measured from the critical section to the "bearing face" of the head, while the latter as the length to the "outside end" of the head (i.e., ACI 318 defined development length + head thickness). Therefore, according to the ACI 352R-02 definition, l_{prov_2} for JH is calculated as 276.5 mm (= 257.5 + 19 mm). This value is greater than the ACI 352 defined l_{dt_2} of 225 mm. On the contrary, for JK specimen, l_{prov_2} of 257.5 mm is smaller than l_{dh_2} of 300 mm, where l_{dh_2} is the ACI 352 defined development length for a hooked bar anchored in a Type 2 joint.

The location of the head and hook extension did not comply with ACI 352R-02 recommendations (Section 4.5.2.1) or ACI 318-08 commentary (R12.6). The outside edges of the head and the hook were located at 113.5 and 132.5 mm, respectively, from the back of the joint core (versus 51 mm per ACI 352R-02). Note that the 2008 version of ACI 318 began to explicitly state that the headed bar should extend to the far side of the joint core (Commentary R12.6 and Fig. R12.6(b)).

2.3.2 Joint shear capacity and confinement

Joint shear force demand (V_u = 500 kN) was only about 50% of nominal joint shear capacity (V_n = 990 kN). The joint core was moderately confined as per common practice in Korea, where low-to-moderate seismic regions exist (Fig. 1). Joint transverse reinforcement placed satisfied ACI 352R-02 (§4.2.1) recommendations for "Type 1" joint. One of the objectives of this research is to observe seismic behavior of beam-column joints with a moderate amount of confining steel under unexpected extreme earthquakes. On the other hand, the column core was well-confined, as per ACI 352 Type 2 detailing, to ensure no damage to the column.

2.3.3 Testing and instrumentation

By subjecting the beam end tip to reversed cyclic displacements, the test setup simulated earthquake-induced lateral drifts (Fig. 1). The horizontal actuator with a capacity of 1,000 kN was used to displace the beam end, with column ends pinned. Though application of axial force is known to improve bond behavior (Chun et al., 2007), no axial force exerted on the column as it is appropriate to test a worst bond condition in this study. Displacement control was applied at drifts of approximately ±0.4, 0.7, 1.0, 1.7, 2.7, and 3.5% with three cycles at each drift level. The selected drift levels are slightly inconsistent with ACI Innovation Task Group testing protocol (ACI T1.1R-01), but the number of cycles (three) conformed to ACI T1.1R-01. A total of 11 LVDTs were installed for each test to measure beam displacements at two different locations, column axial movement, beam and column end rotations, as well as joint distortion (Fig. 1). Strain gauges were affixed on reinforcing bars at selected locations to verify if bar yielding or strain hardening occurred, and the corresponding drift level for each specific bar strain.

3. PULLOUT TEST RESULTS

Figure 2 depicts the relations of pullout load versus bar slip measrued at the loaded end under monotonic loading, comparing behavior between three different head types (Table 2). The load-slip relations were relatively similar between the specimens using threaded head-to-bar connections (Fig. 2(a)). The CBF was not observed for all specimens. Pullout failure occurred for straight anchorage; however, no loss in anchorage strength was observed until reaching the onset of strain-hardening of all headed bars (from strain readings). The specimens maintained the anchorage strength (by head bearing) even after significant bond deterioration, leading to ductile failure. The bond deterioration was evidenced by a reduction in stiffness of the load-slip relations, and by the observation that the stiffness softening has occurred when splitting cracks began to form.

Results of bar slips (average: 0.09 and 0.32 mm) at 70 and 95% of the ultimate forces demonstrate excellent anchorage behavior for all three head types under monotonic loads, as reported by the CEB-FIB Model Code 90 (§9.1.1.3) that requires those within 0.1 and 0.5 mm, respectively. Table 2 indicates that Mnt-SQT (monotonic; square; threading) achieved slightly higher anchorage strength (~170 kN; $1.28A_sf_y$) than that (~165 kN; ~$1.23A_sf_y$) obtained for Mnt-CST (monotonic; circular small; threading) or Mnt-CLT (monotonic; circular large; threading), where f_y is the measured yield stress (465 MPa). This was anticipated because the net bearing areas (A_{brg}) of the heads were different.

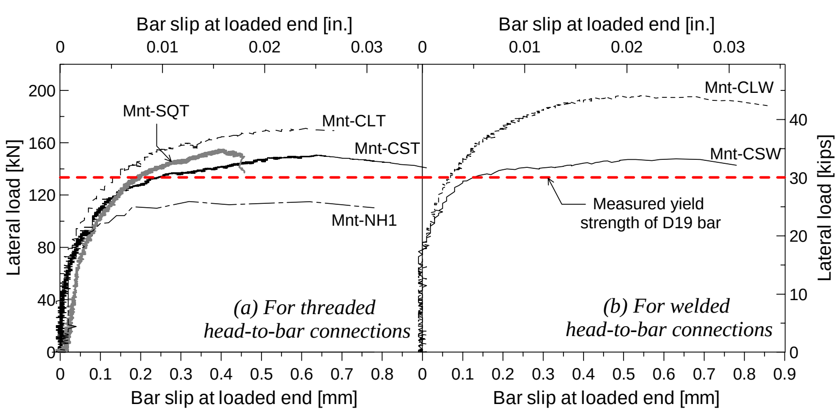

Figure 2 Pullout test data

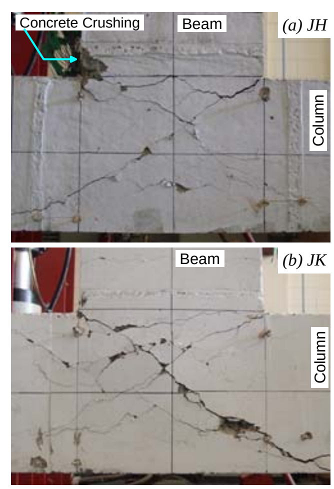

Figure 3 Joint damages

Figure 2(b) provides similar comparisons for the case where the welded head-to-bar connection was used. While excellent anchorage behavior was exhibited for all specimens, the anchorage strength was particularly increased to ~190 kN ($1.47A_sf_y$) for the large head (A_{brg}/A_b = 4.5). Due to the local strain-hardening at the welded connection location, it appears that the larger tensile force was transferred. The anchorage strengths for the welded and threaded connections were similar for small heads (Table 2).

Similar test results were observed under repeated loading. There were no discrepancies in anchorage behavior (stiffness and strength) between the threaded and welded connections. All headed bars were reached to the yield stress, although relatively low anchorage strengths (~140 kN; ~1.05$A_s f_y$) were recorded for the small head (A_{brg}/A_b = 2.6). The average head slips of 0.06 and 0.30 mm at 70 and 95% of the ultimate forces were within tolerable limits. The strengths began to degrade at head slips exceeding 0.5 mm. These data and the post-test observation indicate that "local" concrete crushing eventually has occurred just before the (small) head, but a concrete breakout failure (CBF) did not occur. The cone-type failure (i.e., CBF) is typically characterized by a sudden loss of the pullout load to near zero levels right after reaching the peak, which was not seen in the load-slip relations.

4. SEISMIC TEST RESULTS

4.1. Observed beam-column joint behavior

Observed crack patterns for both JH and JK specimens were similar up to about 2.7% drift; however, during 3.5% drift cycles, the extent of joint damage for JK was more apparent than that for JH (Fig. 3(a) vs. 3(b)). This might be influenced by different bond qualities, as other conditions were exactly the same.

During 0.4% drift cycles, flexural cracks formed on the beam adjacent to the joint. For drift ratios greater than 0.7%, these cracks became significant and were observed up to the midpoint of the beam. Diagonal joint cracks occurred beyond a drift level of 0.7%. The deterioration of JK was characterized by joint cover spalling and by a gradual accumulation of diagonal strut damage between 2.7 and 3.5% drift ratios (Fig. 3(a)). In contrast, JH showed a typical flexural failure of the beam with limited joint deterioration (Fig. 3(b)). Significant damage in the diagonal joint strut for JK might be due in part to a relatively moderate degree of joint confinement, which in fact did not adversely affect the overall behavior of JH. The only difference between JH and JK was the anchorage condition; therefore, it is believed that inadequate anchorage of hooked bars might have accelerated the strut failure of JH specimen at approximately 3% drift. This will be discussed in detail by examining test data in the following subsection.

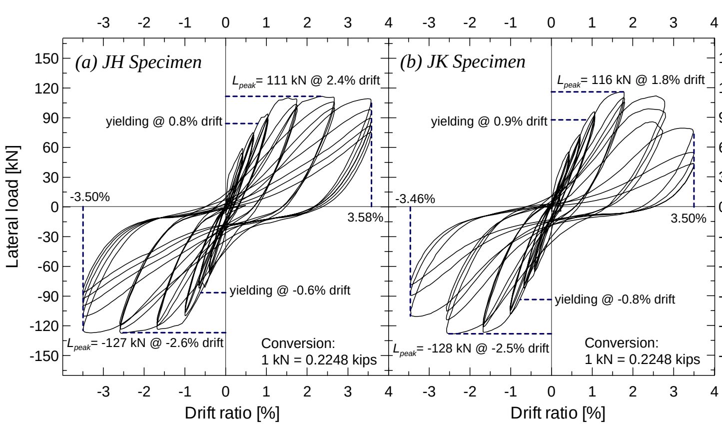

Figure 4 Lateral load-drift relations

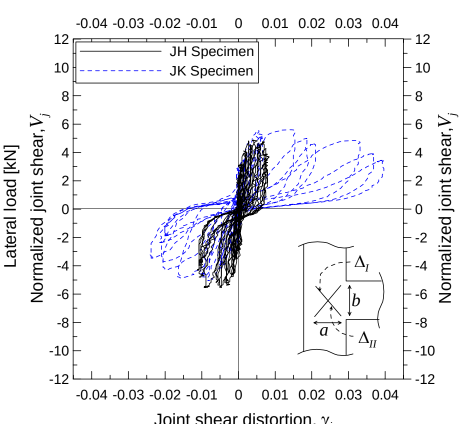

Figure 5 Joint shear-distortion relations

4.2. Data monitored from cyclic testing

The relationship of lateral load versus story drift is shown in Fig. 4, illustrating that both JH and JK specimens failed in a flexural mode, as also evidenced by substantial beam flexural cracking along with all tension steel yielding. Based on readings from strain gauges that were attached to beam longitudinal bars at the beam-joint interface, first yielding occurred at about 0.6 to 0.9% drifts. This indicates that the development lengths for both headed and hooked bars were sufficient to develop their full strengths (A_sf_y) in the linear range of the load-deformation curve. The lateral stiffness and peak lateral loads were similar for both specimens, exceeding the nominal beam moment strengths (M_n) by at most a factor of 1.25, where M_n is calculated using as-measured material properties (f'_c = 29.1 MPa; f_y = 481 and 460 MPa for JH and JK, respectively).

The peak lateral loads were reached at drifts of approximately 2 to 2.5%. The peak lateral load was maintained until about 3.5% for JH, whereas for JK, after reaching the peak the lateral load subsequently declined to 75% of the peak at +3.25% drift. ACI T1.1R-01 defines the failure criteria as a drop to 75% of the peak lateral load. Therefore, the performance of JK specimen did not meet ACI T1.1R-01 criteria that the failure should be precluded at drift ratios less than 3.5%.

Two potential reasons might exist for relatively poor seismic performance noted from the JK testing: 1) insufficient development length of the hooked bar and 2) moderate joint confinement. Even though the edge of the hooked bar was not located (133 mm) within 51 mm away from the outside of the joint core (as recommended by ACI 352R-02, §4.5.2.1), it appears that the bend of the hook was still located inside the main diagonal strut. That is based on the observation that the performance of JH with the same embedment depth, however, was satisfactory. As mentioned earlier, the hooked bars were provided with a shorter development length than the ACI 352 defined length. This adversely affected the anchorage behavior in the nonlinear range of deformation, particularly beyond 2.5% drift. On the contrary, the JH test results indicate that the same embedment length of $15d_b$ ($0.168f_yd_b/\sqrt{f'_c}$) was enough for headed bars with small heads (A_{brg}/A_b = 2.6).

The lack of joint confinement (Type 1 detailing) eventually resulted in joint shear failure at a drift of 2.5 to 3% in conjunction with poorer anchorage behavior for JK (versus JH). It is interesting to note that even with the same condition (joint shear demand and confinement), JH with headed bars experienced no loss of the lateral load capacity until the end of testing (up to 3.5% drift; Fig. 4). In part, this appears due to the fact that the column transverse reinforcement (Type 2 detailing) and confined concrete "above" the joint contributed to resist the bearing stress acting against the head. Given the less compressive stress applied to the diagonal strut "inside" the joint, inelastic joint shear deformation was smaller than that of JK specimen during the 2.7 and 3.5% drift cycles (Fig. 5). The less joint shear deterioration is also demonstrated in Fig. 3(b) (vs. Fig. 3(a)). This finding is important as it signals a possibility for reduction of "transverse reinforcement" in the exterior interstory joints with headed bars, while providing equivalent seismic performance. This also points out that the anchorage behavior of headed bars was not adversely affected by reduced joint confinement. The potential for the reduction of the joint reinforcement when utilizing headed bars warrants further investigation.

Due to the limitation of the actuator stroke, the drift ratio could not be increased further. Instead, two more cycles were repeated at 3.5% drifts for JH specimen. At the third and fifth cycles of 3.5% drift, peak lateral loads were reduced by 17 and 30% of the first cycle peak load, respectively (Fig. 4). The 17% reduction for the third cycle at 3.5% drift meets the ACI T1.1R-01 acceptance criteria, and the 30% reduction at the fifth cycle can be considered not critical. All other acceptance criteria of ACI T1.1R-01 were satisfied.

The greater energy dissipated for JH vs. JK during the drift cycles. The better energy dissipation capacity "after initial yielding" for JH supports the improved anchorage behavior of headed bars embedded in the exterior joint, for the given development length. The apparent degree of pinching shown in Fig. 4(a) vs. 4(b) is also consistent with this behavior. The energy dissipation capacity of reinforced concrete beams is one of key aspects for the design of ductile moment frames. Based on the results described in this and in the previous paragraphs, it is concluded that use of headed bars, with l_{prov_1} of $13d_b$ ($0.168f_yd_b/\sqrt{f'_c}$) and (A_{brg}/A_b) of 2.6, was effective in transferring the beam moment to the exterior column without loss up to 3.5% drift.

5. SUMMARY

1. Based on twelve pullout tests of single-headed bars, the head size of (A_{brg}/A_b) of at least 2.6 was effective to achieve adequate anchorage or bearing, provided that the development length was $10d_b$ or $0.124f_yd_b/\sqrt{f'_c}$.

2. The larger head (A_{brg}/A_b = 4.5) was, the larger was the anchorage strength.

3. The loading condition (monotonic vs. repeated), head shape (circular vs. square), and head-attaching technique (threading vs. welding) did not impact the anchorage behavior substantially during pullout.

4. The test results of the joint subassemblies support the applicability of headed bars with small heads (A_{brg}/A_b = 2.6) in exterior beam-column joints, and the new ACI 318-08 provision on headed bars.

5. The exterior joint containing headed bars with a development length of $15d_b$ or $0.168f_yd_b/\sqrt{f'_c}$ and with head size of (A_{brg}/A_b) of approximately 3 was capable of transferring probable moments and forces in the members without loss up to 3.5% drift, and also met ACI T1.1R-01 acceptance criteria. On the other hand, the joint with hooked bars did not.

6. The aforementioned results were achieved even with moderate (Type 1) joint confinement. This indicates that reduced joint confinement does not impact adversely on the headed bar anchorage in interstory joints, likely due to the bearing stress acting against the concrete above the joint. This implies a possibility that transverse reinforcement in the exterior interstory joint can be reduced when headed bars are utilized.

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