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Highly Damage-Tolerant Beam-Column Joints Through Use of High-Performance Fiber-Reinforced Cement Composites

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The feasibility of using high-performance fiber-reinforced cement composites (HPFRCCs) as a means to eliminate the need for confinement (transverse) reinforcement and the associated construction problems in beam-column connections subjected to earthquake-induced loading is evaluated. The fiber cementitious material used in this study contained ultra-high molecular weight polyethylene fibers in a 1.5% volume fraction, which represented the minimum value for which a tensile strain-hardening behavior was obtained from direct tension tests. Two large-scale subassemblies, consisting of beams framing into a column from two opposite sides, were tested under displacement reversals to evaluate the adequacy of the proposed connection design for use in zones of high seismicity. The two HPFRCC connections were subjected to peak shear stresses of 7.3 and 9.3 MPa, which corresponded to approximately 1.2 and 1.4$\sqrt{f'_c}$ *(MPa), respectively. Although the maximum beam shear stress corresponded to 0.2*$\sqrt{f'_c}$ *(MPa), no special transverse reinforcement detailing was provided in the beam plastic hinge regions. Experimental results indicate that HPFRCC beam-column connections perform satisfactorily under large shear reversals with excellent damage tolerance. The test specimens sustained drifts as large as 5.0% with beam rotation capacities in the order of 0.04 rad. Only minor joint damage was observed at the end of the tests, indicating that the ACI joint shear stress limit of 5/4*$\sqrt{f'_c}$ *(MPa) can be safely used in HPFRCC connections with no confinement reinforcement. Also, excellent bond between beam longitudinal bars and surrounding HPFRCC material was observed throughout the tests even though the connection design did not satisfy minimum anchorage length requirements specified in the ACI Building Code.*

Keywords: beam-column; joint; shear strength.

INTRODUCTION

Beam-column connections in reinforced concrete (RC) frame structures under earthquake-induced lateral displacements are generally subjected to large shear stresses that may lead to significant joint damage and loss of stiffness in the structure. Since the 1960s, several researchers (for example, Hanson and Connor 1967; Hanson 1971; Megget and Park 1971; Uzumeri and Seckin 1974; Meinheit and Jirsa 1981; Durrani and Wight 1982; Ehsani and Wight 1982) have devoted significant effort studying the behavior of joints under shear reversals, as well as on the development of design recommendations for ensuring adequate connection behavior in frame structures expected to undergo large inelastic deformations. Current design recommendations for RC beam-column joints in earthquake-resistant construction given by Joint ACI-ASCE Committee 352 (2002) focus on three main aspects: 1) confinement requirements; 2) evaluation of shear strength; and 3) anchorage of beam and column bars passing through the connection. Additionally, a strong column-weak beam behavior must be ensured, and frame members or regions expected to experience large reversed inelastic deformations must be properly detailed to ensure sufficient displacement capacity during earthquakes.

The ACI design recommendations for RC beam-column connections follow a strength-based approach, where the connection shear strength is checked against the expected force demands imposed by adjoining members. Using these recommendations, the joint is assumed to behave satisfactorily during earthquakes if its shear strength exceeds the shear demand, a strong column-weak beam mechanism is ensured, and sufficient transverse reinforcement and anchorage length for reinforcing bars passing through the connection are provided. The minimum amount and maximum spacing of joint transverse reinforcement are based on the requirements for critical regions of RC columns, which when combined with the longitudinal reinforcement from beams and columns, often lead to severe reinforcement congestion and construction difficulties. Further, the need to satisfy the anchorage length requirements for beam and column longitudinal bars may require either the use of large column and/or beam sections or a large number of small diameter bars, which might in turn increase reinforcement congestion in the connection. It is worth mentioning that satisfying the minimum ACI Code provisions does not prevent the formation of wide diagonal cracks in connections during large displacement reversals (Joint ACI-ASCE Committee 352 2002) and thus, these provisions are primarily intended to provide protection against loss of lives and structural collapse.

As seismic design of structures moves towards performance-based design, there is need for new structural members and systems that possess enhanced deformation capacity and damage tolerance, while requiring simple reinforcement details. The development of a highly damage-tolerant beam-column connection would allow structural engineers to design joints for moderate shear distortions (that is, 0.01 rad) while exhibiting little damage, reducing rotation demands in beam plastic hinges, and eliminating the need for post-earthquake joint repairs. One option for achieving this goal is to use fiber-reinforced cement-based materials with superior deformation capacity in beam-column connections. In recent years, strain-hardening or high-performance fiber-reinforced cement composites (HPFRCCs) with relatively low fiber-volume fractions ($V_f \leq 2.0\%$) have been developed (Li 1993; Naaman 1999). These composites generally exhibit tensile strain capacities between 1.0 and 5.0% depending on the type and amount of fibers used, matrix composition and

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matrix-fiber interface, and a compression behavior that resembles that of well-confined concrete. Thus, HPFRCC materials are ideal for use in regions of structures susceptible to experience large inelastic deformations and/or high shear stress reversals during a seismic event, such as plastic hinge regions of flexural members and beam-column connections. In addition, past research (Parra-Montesinos and Wight 2000) has shown superior bond between reinforcing bars and HPFRCCs under stress reversals compared with that in bars embedded in regular concrete, which would also make these materials attractive for reducing slip of reinforcing bars in RC beam-column connections.

In this research, the feasibility of achieving large displacement capacity and damage tolerance in frame structures designed with simple reinforcement detailing in beams and connections by using HPFRCC materials was evaluated. The reductions in transverse reinforcement requirements and associated labor, and more importantly, the achievement of highly damage-tolerant structures that would most likely require few or no post-earthquake repairs, would make the use of HPFRCCs in selected regions of frame structures attractive from both structural and economical viewpoints.

PREVIOUS WORK ON BEAM-COLUMN CONNECTIONS CONSTRUCTED WITH FIBER-REINFORCED CEMENT COMPOSITES

During the past 25 years, several research studies have been conducted to evaluate the effectiveness of using fiber-reinforced concrete or cement composites (FRCCs) to reduce reinforcement congestion while enhancing seismic performance in beam-column connections (for example, Henager 1977; Craig et al. 1984; Gefken and Ramey 1989; Jiuru et al. 1992; Filiatrault, Pineau, and Houde 1995; Bayasi and Gebman 2002). To date, the fiber cement composite materials used in research studies on beam-column connections have typically consisted of regular concrete with steel fibers in volume fractions ranging between 1.0 and 2.0%. Even though a superior tensile response is attained compared with regular concrete, these FRCCs exhibit a tensile softening response after first cracking as opposed to the strain-hardening behavior with multiple cracking observed in HPFRCCs. Results from previous studies have demonstrated that FRCCs with steel fibers in 1.2 to 2.0% volume fractions can be used as partial replacement of confinement reinforcement in beam-column joints. In addition, improved anchorage conditions in longitudinal beam and column bars passing through joints with steel-fiber concrete have been observed (Jiuru et al. 1992). The fact that these materials exhibit a tensile softening response after first cracking, however, limits their ability to sustain large shear stresses while preventing early damage localization, making regular FRCC materials not suitable for total elimination of transverse reinforcement in highly stressed RC joints.

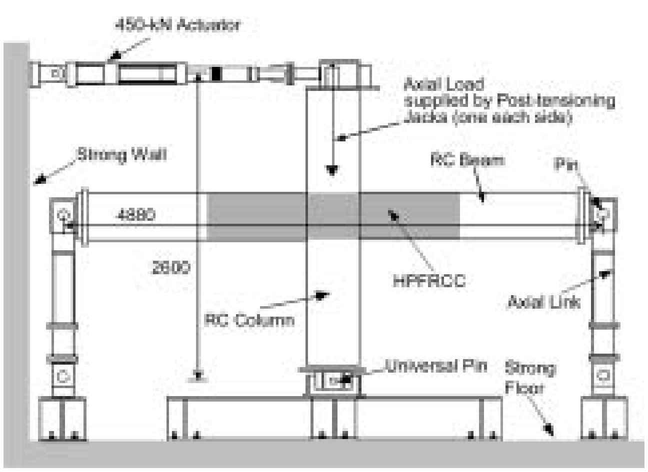


Fig. 1—Test setup.

RESEARCH SIGNIFICANCE

Results from this research provide evidence that supports the use of HPFRCC materials as a total replacement of confinement reinforcement in beam-column connections of RC earthquake-resistant frame structures. In addition, the use of HPFRCCs as a means to eliminate the need for special transverse reinforcement details in beam plastic hinges is evaluated.

EVALUATION OF PROPOSED HIGH-PERFORMANCE FIBER-REINFORCED CEMENT COMPOSITE CONNECTION DETAILING

In the proposed frame configuration, HPFRCC material would be used in the beam-column connection and adjacent beam region (over a length of twice the beam depth), the rest of the structure being constructed with regular concrete. For HPFRCC materials to be competitive for use in connections of earthquake-resistant RC frames, the following incentives were sought in the proposed HPFRCC connections in addition to providing enhanced damage tolerance: 1) total elimination of confinement (transverse) reinforcement in beam-column connection while maintaining comparable shear strength; 2) increased stirrup spacing in beam plastic hinge zones; and 3) reduction in reinforcement slip, minimum anchorage length for beam and column longitudinal bars passing through the joint, or both. Thus, in the proposed frame system, special transverse reinforcement detailing would only be provided in the column regions just above and below the beams, as required by the ACI Code. It is worth mentioning that no HPFRCC material is used in the column regions adjacent to the connection so as not to interfere with the typical concrete placing sequence of columns, which is generally performed separately from that of floor systems. To evaluate the adequacy of the proposed connection for use in earthquake-resistant construction, two approximately 3/4-scale beam-column subassemblies were designed and constructed satisfying the aforementioned criteria, and later tested under large displacement reversals.

Design of test specimens

The two beam-column subassemblies represented a connection where beams frame into the column from two opposite sides. Figure 1 shows a sketch of the test setup used in this investigation and overall specimen dimensions. HPFRCC material was used in the beam-column connection and adjacent beam regions over a length equal to twice the beam depth. The beam-column subassemblies were pinned at beam midspan and column midlength, assuming inflection points at these locations during a seismic event. The column cross section was 350 x 350 mm, and the beam was 150 mm wide and 350 mm deep. Because moderate column axial loads have been reported to enhance cracking shear strength in beam-column connections (Meinheit and Jirsa 1981), only a small axial load, corresponding to approximately 4.0% of the column axial load capacity, was applied to the column through hydraulic jacks.

Earthquake-induced displacements were simulated by imposing lateral displacements at the top of the column through a hydraulic actuator. The planned lateral displacement history included 20 reversed displacement cycles ranging from 0.5 to 5.0% (0.005 to 0.05 rad) drift (lateral displacement ÷ column height), with two cycles performed at each new drift level (Fig. 2). After this original displacement history was completed, the specimens were cycled twice to 6.0% drift. It should be mentioned that a drift of 6.0% is unrealistically high for RC frame structures, and thus these two additional cycles were applied only for the purpose of evaluating beam rotation capacity in the test specimens.

For a frame designed following a strong column-weak beam approach, the shear stress demand in the connection region can be estimated as

$$v_j = \frac{\sum \frac{M_{ub}}{jd} - V_c}{b_j h_c} \qquad (1)$$

where M_{ub} is the ultimate moment strength of the beams framing into the column in the loading direction; jd is the distance between the internal compression and tension force resultants in the beams; V_c is the column shear; b_j is the effective joint width; and h_c is the column depth. According to the Joint ACI-ASCE Committee 352 recommendations (2002), an average shear stress $v_j = 5/4\sqrt{f'_c}$ (MPa) is allowed in connections with only beams framing into the column from two opposite sides, given that a strong column-weak beam mechanism is ensured and adequate reinforcement detailing is provided. Thus, the flexural design of the beams in the test specimens was performed such that a comparable joint shear stress level would be imposed on the HPFRCC connection to better evaluate the feasibility of using HPFRCC materials as replacement for joint transverse reinforcement. Because the use of only top and bottom longitudinal reinforcement in the beams implies that the joint would not be crossed by any horizontal reinforcement, however, intermediate layers of beam longitudinal reinforcement were used in the connection region to enhance joint postcracking behavior. In addition, these intermediate layers of reinforcement would help in spreading beam inelastic deformations away from the column faces, as demonstrated by Abdel-Fattah and Wight (1987). The final beam flexural design for the test specimens is shown in Fig. 3. It is worth mentioning that the ratio between the column depth h_c and the diameter of the beam longitudinal reinforcement d_b was equal to 18.7, which is slightly below the minimum ratio of 20 specified in the 2002 ACI Building Code and Joint ACI-ASCE Committee 352 recommendations for Grade 420M steel.

For design purposes, the joint shear stress demand in the test specimens was estimated using actual material properties for the steel reinforcement and assuming no contribution from the HPFRCC material to ultimate beam moment strength. The assumption of no moment contribution from the HPFRCC material is reasonable for beams with longitudinal reinforcement ratios larger than 1.0% and subjected to large plastic hinge rotations (≥0.03 rad), as was the case of the two beam-column test subassemblies, because significant fiber pullout would have occurred at these rotation levels. The RC columns in the test specimens were designed such that the subassembly behavior would be governed by a strong column-weak beam mechanism. The nominal column moment strength-beam strength ratios for Specimens 1 and 2 were 2.2 and 1.6, respectively (Table 1).

The design of beam transverse reinforcement outside the plastic hinge regions was performed according to Chapter 11 of the 2002 ACI Code. The same transverse reinforcement design was then used in the beam plastic hinge regions, which violated the maximum spacing requirements specified in

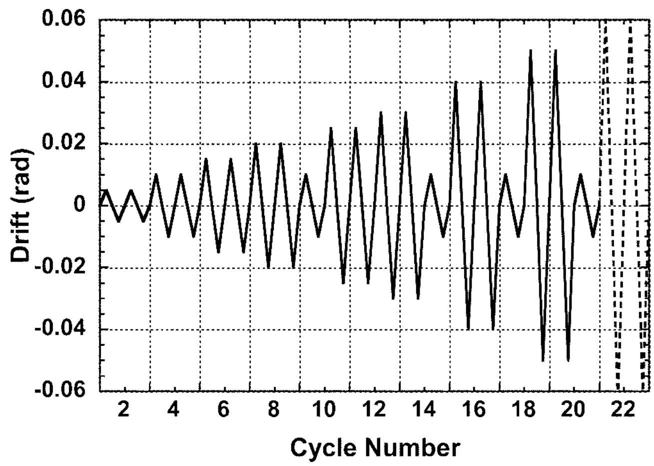


Fig. 2—Lateral displacement history.

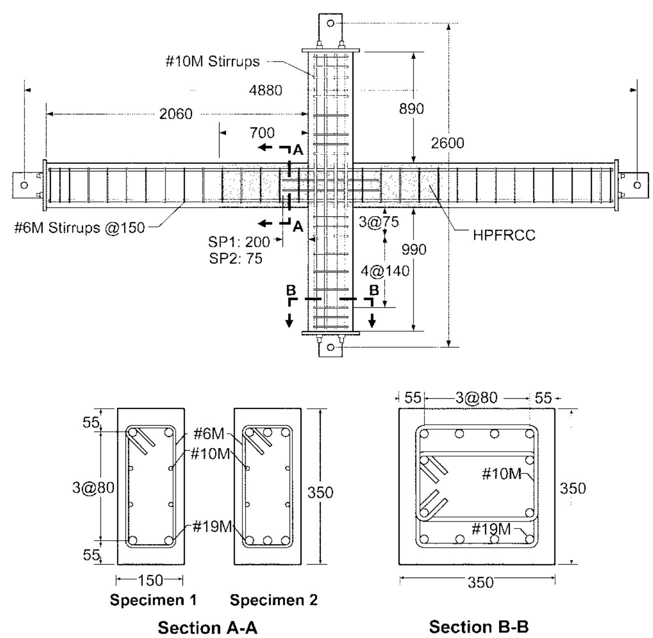


Fig. 3—Specimen details.

Chapter 21 of the ACI Code for earthquake-resistant construction. The HPFRCC material in the beam plastic hinges, however, was expected to provide sufficient confinement and shear strength such that no special reinforcement detailing would be needed to sustain large inelastic rotation reversals. The design of transverse reinforcement in the RC column was performed according to Chapter 21 of the 2002 ACI Code and was kept the same for both test specimens (Fig. 3).

MATERIAL PROPERTIES AND SPECIMEN CONSTRUCTION

Each beam-column subassembly was constructed with two types of cement-based composites: 1) HPFRCC (beam-column connection and adjacent beam plastic hinge regions); and 2) ready mixed concrete (column and elastic beam regions). The HPFRCC material contained 38 mm-long and 0.038 mm-diameter straight ultra-high molecular weight polyethylene fibers in a 1.5% volume fraction. This fiber volume fraction represented the minimum amount for which a tensile strain-hardening behavior was obtained from direct tension tests. The strength and modulus of elasticity for the fiber material were 2570 MPa and 117 GPa, respectively. Other components in the HPFRCC mixture included: cement (Type III), fly ash, flint sand ASTM 30-70, and water in the following proportions by weight: 1:0.15:1:0.5. A high-range water-reducing agent was also added to ensure good workability of the mixture. The ready mixed concrete had a specified concrete strength of 35 MPa, a 150 mm slump, and a 10 mm maximum size of limestone aggregate.

The test specimens were cast in a horizontal position. The beam-column joint and adjacent beam regions were placed first with HPFRCC material. After the fiber-cement composite hardened, ready mixed concrete was placed in the rest of the specimen. Thus, two cold joints, perpendicular to the beam axis, were present at a distance twice the beam depth from the column faces, while two other cold joints were located at the joint-column interfaces. Both the HPFRCC material and regular concrete were vibrated during casting with an electrical vibrator.

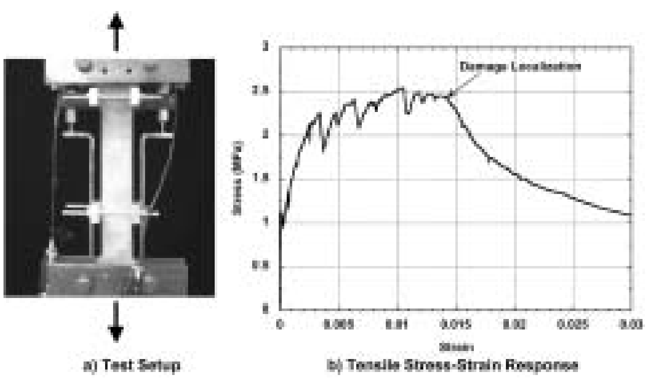


Fig. 4—Tensile tests on HPFRCC dog-bone-shaped specimens.

To determine the properties of the HPFRCC material used, compression tests on 75 x 150 mm cylinders, as well as direct tension tests on 25 x 50 mm dog-bone-shaped specimens (Fig. 4(a)), were performed. Table 1 lists the average compressive strength, average postcracking strength (peak postcracking strength), and tensile strain capacity (strain at peak stress) of the HPFRCC material. Figure 4(b) shows the tensile stress-strain response obtained from one of the dog-bone-shaped specimens constructed with the HPFRCC material used in Specimen 2. As can be seen, this particular material sample exhibited a tensile strain-hardening behavior up to approximately 1.5% strain (peak strength of 2.5 MPa), which translated into the formation of a multiple cracking pattern. For larger strains, damage localization (single crack opening) dominated the material response, leading to a tensile softening response up to total fiber pullout. The regular concrete used in the columns and beam regions away from the connection was obtained from a local ready mix concrete supplier. The average compressive strength for this concrete was 43.9 and 41.3 MPa for Specimens 1 and 2, respectively.

Grade 420M steel was used in the column reinforcement and longitudinal beam bars. The yield and ultimate strengths for the steel used in the No. 19M bars of Specimen 1 were 540 and 660 MPa, respectively, while yield and ultimate strengths of 435 and 695 MPa were obtained for the No. 19M bars in Specimen 2. Note that the yield strength of the No. 19M bars used in Specimen 1 was greater than the assumed bar stress of 520 MPa ($1.25f_y$) specified in the ACI Building Code and the Joint ACI-ASCE Committee 352 recommendations (2002) for calculating joint shear stress demand. The 30% overstrength observed in these bars, combined with a column depth equal to 18.7 beam bar diameters, led to bond stress demands substantially larger than those expected in RC beam-column connections, as will be discussed in detail in a following section. The properties of the No. 10M bars were only available for Specimen 1. These bars exhibited a yield strength of 500 MPa and a tensile strength of 770 MPa. The steel used for the No. 6M stirrups did not satisfy ASTM A 615M standards. This steel exhibited a nearly elastic-perfectly plastic stress-strain behavior with measured yield and ultimate strengths of 560 and 610 MPa, respectively.

BEHAVIOR OF HIGH-PERFORMANCE FIBER-REINFORCED CEMENT COMPOSITE BEAM-COLUMN SUBASSEMBLIES

Overall response

From the load-versus-drift hysteresis response obtained for the two specimens (Fig. 5), it can be seen that both connection subassemblies exhibited a stable behavior up to

Table 1—Summary of experimental results

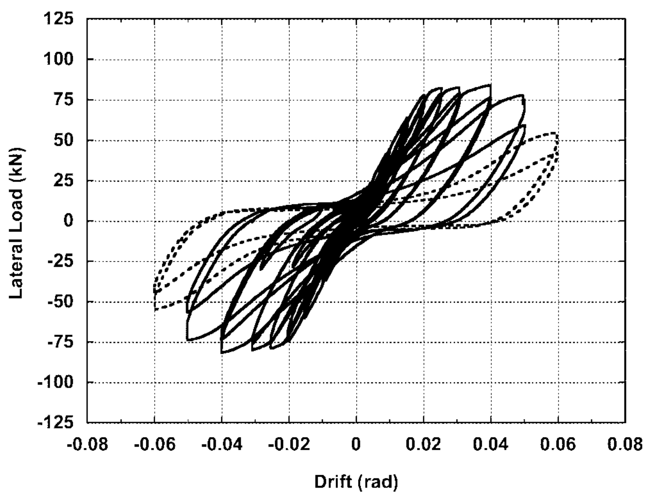
Specimen	High-performance fiber-reinforced cement composites (HPFRCCs) σ_{pc}, MPa	ε_{pc}	f'_c, MPa	$\frac{\sum M_{nc}}{\sum M_{ub}}$	$(vj)_{max}$ ($\sqrt{f'_c}$, MPa)	$(\gamma)_{max}$, rad	Drift capacity, rad*	Maximum beam rotation, rad*
1	2.7	0.010	39.3	2.2	1.2	0.002	0.05	0.045
2	2.2	0.013	42.7	1.6	1.4	0.008	0.06	0.045

*Drift capacity and peak beam rotation correspond to maximum values before strength decay greater than 20% of peak strength occurred.

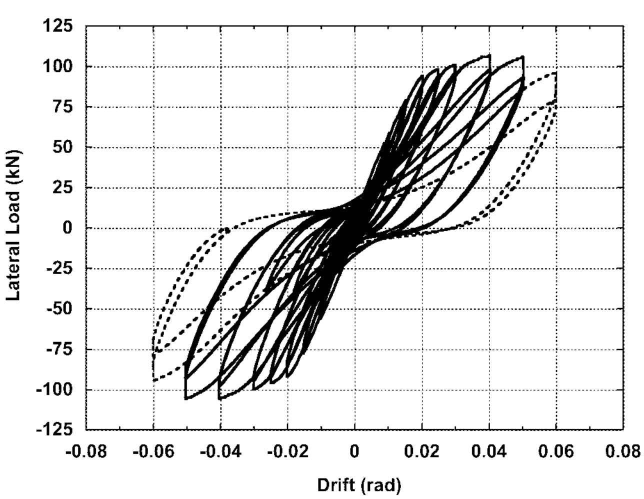
Note: σ_{pc} = postcracking (peak) strength; ε_{pc} = tensile strain capacity (strain at peak stress); M_{nc} and M_{ub} = column nominal moment capacity and beam ultimate moment strength, respectively; v_j and γ = joint shear stress and joint shear distortion, respectively.

displacement levels of 5.0% drift for Specimen 1 and 6.0% drift for Specimen 2, indicating that the proposed connection design is suitable for use in regions of high seismicity. The behavior of the two HPFRCC beam-column subassemblies was governed primarily by inelastic rotations in the beam regions adjacent to the column for drifts of 2.0% or larger, and the remaining elements of the connection subassembly behaved in a cracked elastic range or exhibited limited yielding. Cracking in the two specimens began at the beam ends during the cycles performed to 0.5% drift, with several flexural cracks spaced at approximately 1/4 of the beam depth. Joint diagonal cracking was first observed at 1.0% drift, and at 2.0% drift, only minor damage, characterized by a large number of hairline cracks and limited yielding in the beam longitudinal bars, was observed in both the joint and beam end regions (Fig. 6). During the cycles to 3.0% drift, additional diagonal cracks formed in the joints of the two specimens, with a maximum crack width of 0.25 mm for Specimen 1 and 1.0 mm for Specimen 2. In addition, first signs of damage localization in the HPFRCC material were noticed in the beam plastic hinge regions where a few flexural cracks opened with a maximum width of 3.0 mm for Specimen 1 and 1.25 mm for Specimen 2. For the cycles performed at 4.0 and 5.0% drift, damage concentrated primarily at the beam ends and the connection region exhibited only minor damage (Fig. 7). During the second cycle to 5.0% drift, crushing of the HPFRCC material in the beam plastic hinge regions of Specimen 1 was observed, leading to a 25% drop in the lateral strength of the subassembly. For Specimen 2, a similar observation was made during the second cycle to 6.0% drift, with an approximately 20% decay in lateral strength. At the end of the tests, both connection regions exhibited only minor damage that would not require repair. A larger number of diagonal cracks was observed in Specimen 2, however, primarily due to the larger shear stress demand imposed on the joint of this subassembly. Table 1 lists key results from the subassembly tests.

The displacement capacity of Specimen 1 was governed by the rotation capacity of the beam, which was the only source of inelastic deformation in this subassembly. On the other hand, Specimen 2 exhibited a stable response even during the first cycle to 6.0% drift because lower beam rotation demands were imposed on this specimen due to an increase in joint shear distortions, as discussed in the following section.

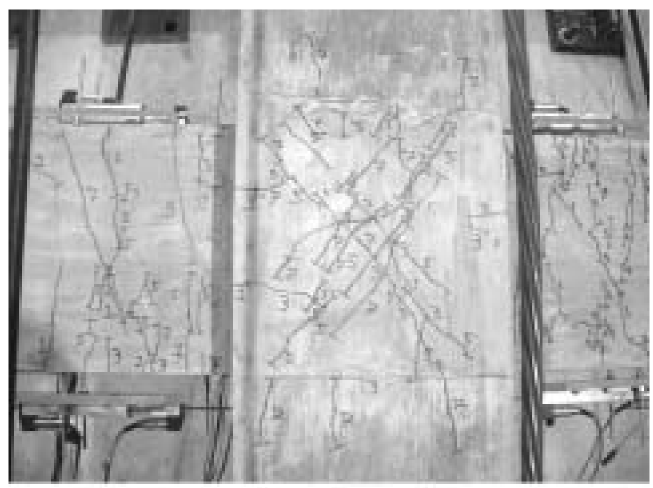


a) Specimen 1

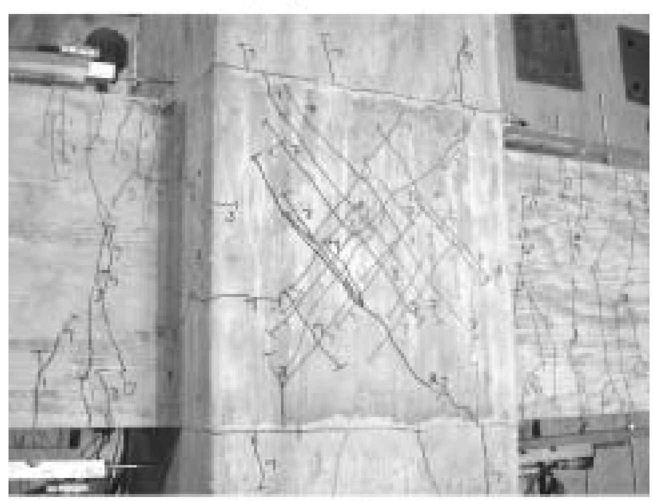


b) Specimen 2

Fig. 5—Lateral load versus displacement response.



a) Specimen 1



b) Specimen 2

Fig. 6—Joint damage at 2.0% drift.

Connection behavior

Specimen 1 was designed such that the peak joint shear stress would be approximately the same as the ACI Code maximum permitted limit of $5/4\sqrt{f'_c}$ (MPa), while the beams in Specimen 2 were designed such that the joint shear stress demand would exceed the ACI prescribed joint stress limit. The joint shear stresses were calculated using Eq. (1) with an effective joint width b_j calculated based on the Joint ACI-ASCE Committee 352 recommendations (2002). In addition, because the internal tensile force in the beams was the result of tensile stresses in the reinforcement and the HPFRCC material, a constant moment lever arm, $jd = 0.9d$, was used for simplicity.

Figure 8 shows the joint shear stress versus shear distortion response for the two test specimens. As can be seen, the behavior of the connection in Specimen 1 was nearly linear with a peak joint shear stress demand of approximately 7.3 MPa, which corresponded to $1.2\sqrt{f'_c}$ (MPa). At this shear stress level, the peak joint shear distortion was 0.002 rad, which translated into negligible joint damage with a maximum crack width of 0.6 mm (Fig. 7(a)). In Specimen 2, a peak shear stress demand of 9.3 MPa ($1.4\sqrt{f'_c}$) was imposed on the beam-column connection. In this specimen, the joint behaved in the cracked-elastic range up to a shear stress of approximately $1.2\sqrt{f'_c}$ (MPa), which is consistent with the behavior observed in Specimen 1. For larger shear stresses, limited joint inelastic deformations occurred with a peak shear distortion of approximately 0.008 rad when the peak shear stress of $1.4\sqrt{f'_c}$ was attained. Joint damage in Specimen 2 at the end of the test could be considered minor, as shown in Fig. 7(b), with a maximum crack width of 3.0 mm at 6.0% drift and a negligible residual crack width upon unloading. Thus, the observed joint behavior in Specimen 2 suggests that current joint shear stress limits specified in the ACI Code and Joint ACI-ASCE Committee 352 recommendations can be safely applied to HPFRCC beam-column connections with no transverse reinforcement.

To compare the behavior of HPFRCC beam-column connections with that expected in an RC joint designed with current standards, the joint shear stress-versus-joint distortion envelope curve for the two test specimens, and that obtained for Specimen X2 tested by Durrani and Wight (1982), are shown in Fig. 9. The shear stress values shown in Fig. 9 have been normalized by the maximum joint shear stress of $5/4\sqrt{f'_c}$ allowed by Joint ACI-ASCE Committee 352 and the ACI Building Code for the studied connection configuration. Durrani and Wight's Specimen X2 was selected for comparison

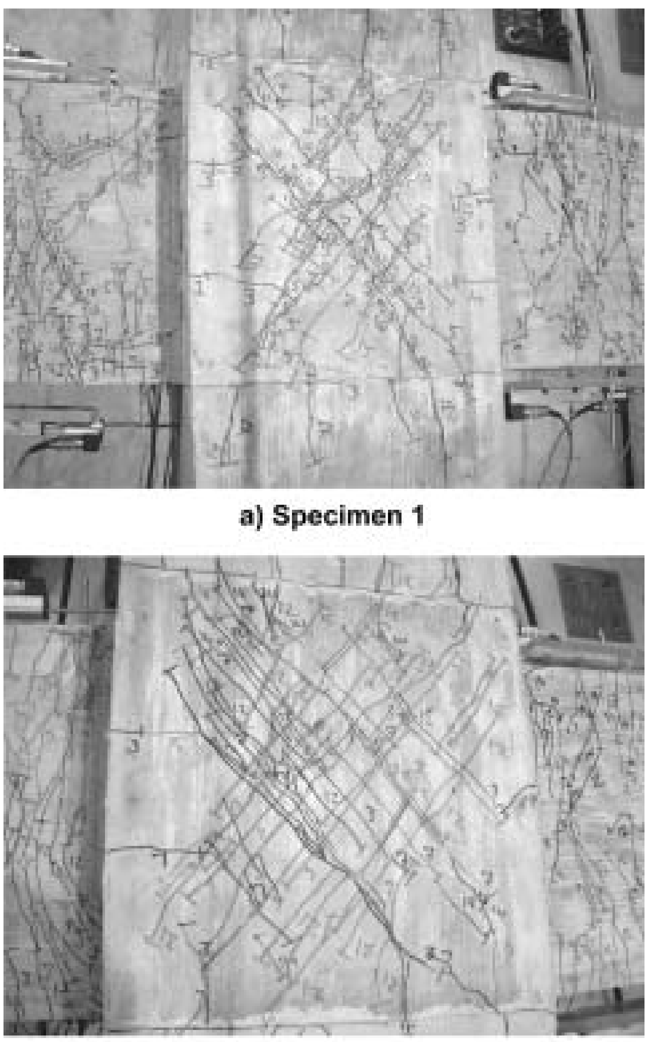


Fig. 7—Joint damage at 5.0% drift.

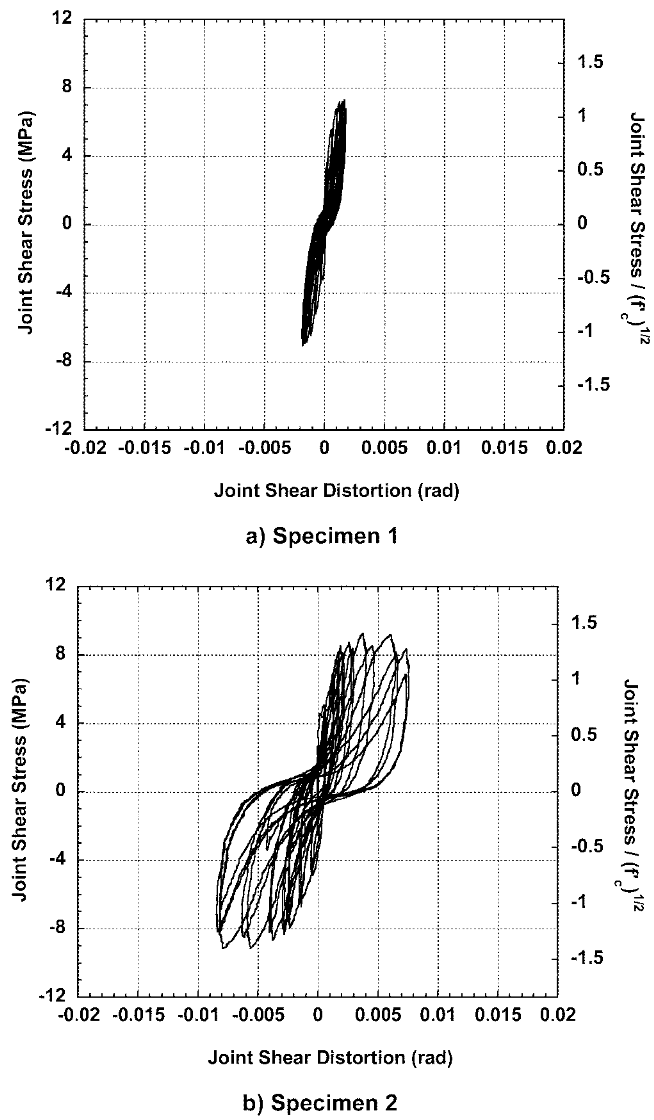


Fig. 8—Joint shear stress versus distortion response.

purposes because of the following reasons: 1) Specimen X2 represented a beam-column subassembly with beams framing into the column from two opposite sides; 2) joint details satisfied current ACI requirements (except for a joint hoop spacing of 100 mm in Specimen X2 compared with a maximum allowable spacing of 90 mm); 3) beam and column dimensions were similar to those in the two HPFRCC connection specimens; 4) peak joint shear stress demand in Specimen X2 was approximately equal to $5/4\sqrt{f'_c}$, based on ACI Committee 352 recommendations; 5) concrete compressive strength was similar to that of the HPFRCC material; 6) column axial load was similar to that in test specimens; and 7) joint shear stress-versus-distortion response was available to the authors.

From Fig. 9, it can be seen that the two HPFRCC connections were significantly stiffer than the RC joint tested by Durrani and Wight. Based on test results, the cracked-elastic joint stiffness for use in analysis and design of RC frames constructed with HPFRCC joints can be estimated as

$$(k_j)_{cracked} = \frac{V_j}{\gamma} = 0.7G_{HPFRCC}b_jh_c \qquad (2)$$

where V_j is the horizontal joint shear force; γ is the joint shear distortion; b_j is the effective joint width (per Joint ACI-ASCE Committee 352); h_c is the column depth; and G_{HPFRCC} is the shear modulus of elasticity of the HPFRCC material.

From the envelope curves shown in Fig. 9, it is seen that the HPFRCC joint of Specimen 2 exhibited a higher shear strength compared with Durrani and Wight's Specimen X2, which should be representative of the strength expected in planar RC interior beam-column connections. It should also be noticed that the connection of Specimen X2 exhibited a stress plateau at approximately the code-stress limit of $5/4\sqrt{f'_c}$ (MPa), indicating that large inelastic distortions (and damage) might occur in RC connections subjected to shear demands near the assumed joint shear strength. On the other hand, the connection in Specimen 2 exhibited a nearly linear elastic response with negligible joint damage up to joint shear stresses slightly larger than the stress limit of $5/4\sqrt{f'_c}$ (MPa). The fact that the connection region in Specimen 2 exhibited only minor damage at a shear distortion of 0.008 rad suggests that HPFRCC connections could be designed to experience limited inelastic deformations, reducing the rotation demands in the beam plastic hinges. It is worth mentioning that the joint shear stress decay in Specimens 1 and 2 shown in Fig. 9 was due to HPFRCC crushing in the beam plastic hinge regions, which led to a decay in beam moment strength, and thus to a reduction in joint shear stress demand.

Behavior of high-performance fiber-reinforced cement composite beam plastic hinges

As shown in Fig. 3, HPFRCC material was used in the beams over a length equal to twice their depth from the column face to eliminate the need for special transverse reinforcement in the beam plastic hinge regions. Beam transverse reinforcement consisted of No. 6M closed hoops at a spacing approximately equal to half the effective beam depth $d/2$ (150 mm), which is typically used in beam regions away from potential plastic hinges. The provided transverse reinforcement at $d/2$ spacing, although sufficient to resist the applied beam shear, was not adequate to provide confinement to the concrete core and bar support, based on the provisions in Chapter 21 of the ACI Code that required a hoop spacing of approximately 75 mm ($d/4$).

Because of the small deformations experienced by the beam-column connections and RC column, beam inelastic rotations accounted for most of the applied drift with the demand in Specimen 1 being slightly larger than that in Specimen 2 due to the lower joint distortions in Specimen 1, as discussed in the previous section. Figure 10 shows the moment-versus-plastic hinge rotation response for one of the beams in Specimen 2. Plastic hinge rotations were measured over a length of 250 mm from the column face (0.7 × beam depth). The corresponding beam average shear stress is also shown in Fig. 10. As can be seen, a stable hysteresis response was obtained up to rotations of approximately 0.04 rad, the rotation at which crushing of the HPFRCC material occurred, leading to a drop in beam moment strength and stiffness. In both test specimens, the ultimate beam moment capacity was accurately predicted by neglecting the tensile strength of the HPFRCC material. From potentiometer readings, the compressive strain capacity of the HPFRCC material was estimated as 0.01, and thus the rotation capacity of HPFRCC beam plastic hinges could be simply estimated as the section curvature corresponding to the peak compressive strain

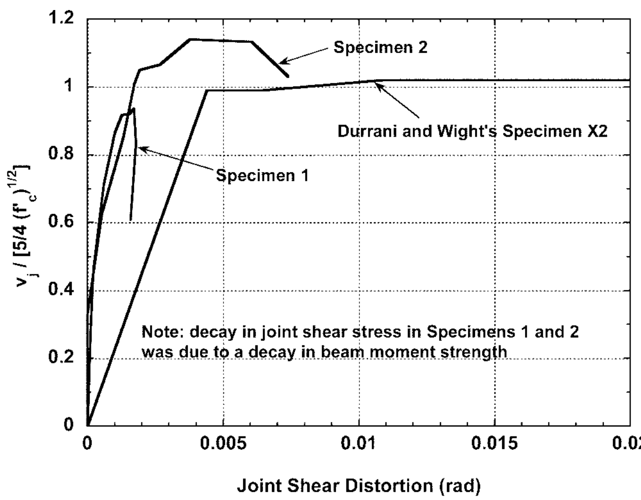


Fig. 9—Joint shear stress versus distortion envelopes.

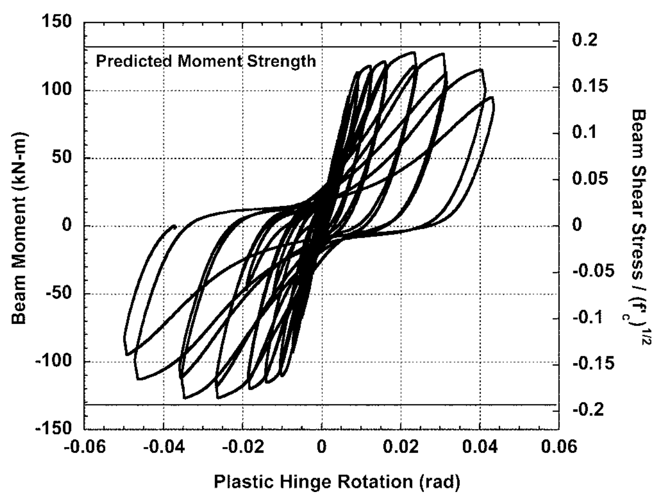


Fig. 10—Beam moment versus plastic hinge rotation response (Specimen 2).

(0.01) times the plastic hinge length (0.75 × beam depth, based on test results). It is worth mentioning that no signs of beam bar buckling were observed throughout the test.

The beam behavior observed in the two test specimens indicates that HPFRCC materials can be safely used in beam plastic hinge regions as a means to relax transverse reinforcement requirements to ensure adequate inelastic rotation capacity in RC flexural members. Even though the shear demand imposed on the beams of Specimens 1 and 2 was relatively low ($<0.2\sqrt{f'_c}$, MPa), test results from an ongoing investigation (Chompreda and Parra-Montesinos 2005) have indicated that HPFRCC flexural members containing ultra-high molecular weight polyethylene fibers in a 1.5% volume fraction and with no transverse reinforcement can sustain average shear stresses of up to $0.4\sqrt{f'_c}$(MPa) at rotations as large as 0.04 rad.

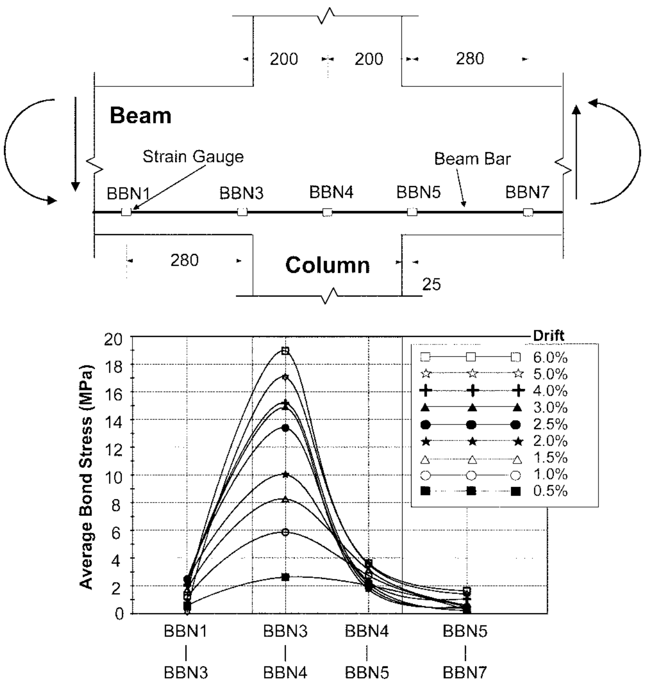


Fig. 11—Bond stress distribution over column depth.

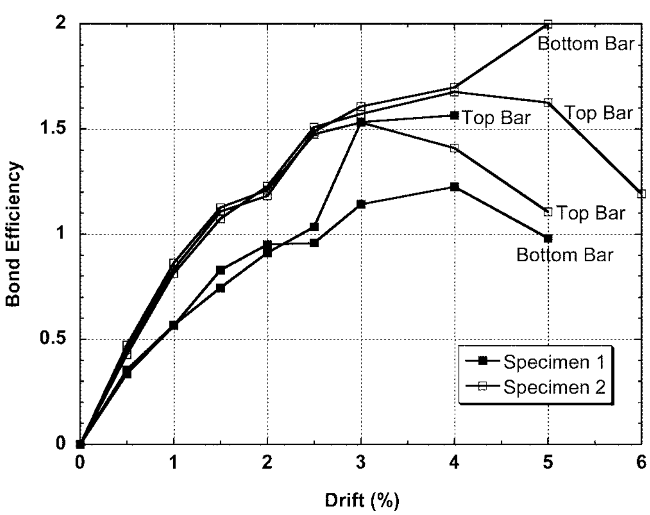


Fig. 12—Bond efficiency.

Reinforcement anchorage requirements in high-performance fiber-reinforced cement composite connections

Because of the change in moments in beam-column connections of RC frames subjected to earthquake-induced lateral loading, beam and column longitudinal bars are expected to be under tension on one side of the connection and compression or nearly zero tensile stresses on the opposite side, given that perfect bond is achieved between the steel bars and surrounding concrete. Peak average bond stresses in longitudinal bars passing through connections, however, are generally on the order of 5.5 MPa (Leon and Jirsa 1986) and thus, large beam and column sizes would often be required to achieve such an ideal behavior. Based on results from tests of beam-column subassemblies, Leon (1989) concluded that anchorage lengths of at least 28 bar diameters are required to achieve a nearly ideal bond behavior in beam and column longitudinal bars passing through RC connections. Joint ACI-ASCE Committee 352 recommendations currently specify that beam-column connections should be proportioned such that an anchorage length of at least 20 bar diameters is provided for Grade 420M longitudinal reinforcement passing through the joint. Thus, the use of 20 rather than 28 bar diameters is only expected to control, and not eliminate, reinforcing bar slip in the connection region.

To evaluate the potential of HPFRCC materials to improve bond conditions in beam and column longitudinal bars passing through RC connections, the beam and column depths in Specimens 1 and 2 were set equal to 18.7 column and beam bar diameters, respectively. Because large inelastic rotations occurred at the beam regions adjacent to the column in the two test specimens, particular emphasis was placed on the bond stresses developed in the longitudinal beam bars. Figure 11 shows the distribution of bond stresses over the column depth for a beam longitudinal bar in Specimen 2. Stresses in the reinforcing bars were determined using the modified Giuffre, Menegotto, and Pinto model proposed by Sakai and Kawashima (2003). Average bond stresses were calculated over the assumed beam plastic hinge length, as well as over the front and back half-column depths, as shown in Fig. 11. Average bond stresses as large as 19 MPa were calculated in the compression side of the connection, while average bond stresses ranging between 2 and 4 MPa were computed on the tension side. From the results obtained for various beam longitudinal bars in Specimens 1 and 2, a peak average bond stress over the entire column depth of 10 MPa was obtained. For design purposes, however, a minimum anchorage length of 16 bar diameters is recommended for use in HPFRCC beam-column connections to keep reinforcing bar slip to minimum levels.

The bond strength developed in the longitudinal beam bars at various drift levels was also evaluated through the use of a bond efficiency parameter (Leon 1989). In this research, bond efficiency was defined as the ratio between the average bond stress developed in a given reinforcing bar and the stress that would be required to produce yielding in the bar at one side of the connection and zero stress at the other side. Thus, bond efficiency values greater than 1.0 would require a stress variation in the reinforcing bar larger than the yield strength f_y over the connection region. Figure 12 shows a plot of bond efficiency versus drift obtained for various beam bars in Specimens 1 and 2. As can be seen, peak bond efficiency values equal to or greater than 1.0 were obtained for drifts ranging between 3.0 and 5.0%, indicating excellent bond behavior, even at large bar inelastic strains. It should be mentioned that bond efficiency values of approximately 0.7 were reported by Leon (1989) in RC connections with anchorage lengths of 20 bar diameters.

SUMMARY AND CONCLUSIONS

Results from a research program aimed at developing highly damage tolerant beam-column connections that require no confinement (transverse) reinforcement through the use of strain-hardening FRCCs or HPFRCCs were reported. The proposed connection design was evaluated through the testing of two large-scale beam-column subassemblies constructed with an HPFRCC material in the connection and adjacent beam regions under large displacement reversals. The following conclusions can be drawn from the results of this research:

1. Beam-column connections constructed with an HPFRCC material containing a 1.5% volume fraction of ultra-high molecular weight polyethylene fibers exhibited excellent strength, deformation capacity, and damage tolerance. The fact that the connections in Specimens 1 and 2 sustained peak shear stress demands of 7.3 and 9.3 MPa (1.2 and $1.4\sqrt{f_c'}$ [MPa]), respectively, indicates that current ACI shear stress limits for joints with beams framing into the column from two opposite sides are adequate for use in HPFRCC connections with no confinement (transverse) reinforcement. Specimens 1 and 2 exhibited a stable behavior up to 5.0 and 6.0% drift, respectively, with most of the inelastic activity concentrated at the beam ends. In addition, only minor joint damage was observed at shear distortions of up to 0.008 rad, suggesting that moderate distortions may be allowed in HPFRCC connections of earthquake-resistant structures, which could lead to a reduction in beam rotation demands;

2. No signs of bond deterioration in beam longitudinal bars passing through the connection were observed in the test specimens, even though the column depth represented 18.7 beam bar diameters and the beam bars were subjected to large inelastic strains. A peak average bond stress of approximately 10 MPa was developed in beam longitudinal bars with no noticeable reinforcement slip. For design purposes, a minimum anchorage length of 16 bar diameters is recommended for use in HPFRCC connections of frames subjected to large inelastic deformations; and

3. The use of HPFRCC materials in beam plastic hinge regions allowed an increase in transverse reinforcement spacing to half the effective beam depth. In both test specimens, a crushing strain of 0.01 was estimated for the HPFRCC material, which translated into a plastic hinge rotation capacity of approximately 0.04 rad.

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