Earthquake-resistant fibre-reinforced concrete coupling beams without diagonal bars

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Abstract

Results from large-scale tests on fibre-reinforced concrete coupling beams subjected to large displacement reversals are reported. The main goal of using fibre reinforcement was to eliminate the need for diagonal bars and reduce the amount of confinement reinforcement required for adequate seismic performance. Experimental results indicate that the use of 30 mm long, 0.38 mm diameter hooked steel fibres with a 2300 MPa minimum tensile strength and in a volume fraction of 1.5% allows elimination of diagonal bars in coupling beams with span-to-depth ratios greater than or equal to 2.2. Further, no special confinement reinforcement is required except at the ends of the coupling beams. The fibre-reinforced concrete coupling beam design was implemented in a high-rise building in the city of Seattle, WA, USA. A brief description of the coupling beam design used for this building, and construction process followed in the field, is provided.

Keywords

Link beams, steel fibres, shear, coupled walls, displacement reversals.

1 Introduction

Coupled walls consisting of structural walls connected by link or coupling beams are a common structural system used in medium- and high-rise structures for earthquake resistance. Typical span-to-depth ratios for coupling beams range between 1.5 and 3.5. Given their low aspect ratio and the large rotation demands these beams can be subjected to during a major earthquake, the design of coupling beams has long represented a challenge for engineers.

Current design practice for coupling beams is primarily based on research conducted in the 1970s (see Paulay and Binney, 1974). Diagonal bars are provided such that they are capable of resisting the entire shear demand. Closely spaced transverse reinforcement is also required to provide confinement to the concrete and lateral bar support (Figure 1), either along the diagonal reinforcement cages, or to the entire member core. Although this design has been shown to perform well under large displacement reversals (Paulay and Binney, 1974; Naish et al., 2009), the construction of diagonally reinforced coupling beams is difficult and time consuming and may control the construction schedule in some cases.

The difficulties mentioned above make it clear that there is need for a simpler coupling beam design that can be implemented economically in the field. Given the tensile and compression ductility of tensile strain-hardening fibre-reinforced concrete, typically referred to as high-performance fibre-reinforced concrete (HPFRC), the use of such a material was thought to show promise as a means to provide shear resistance and confinement, allowing reduced reliance on bar-type reinforcement to ensure adequate seismic behaviour.

Figure 1: Typical design of earthquake-resistant coupling beams.

2 Research program

2.1 Tests of HPFRC coupling beams

The research on the use of HPFRC as a means to simplify reinforcement detailing in earthquake-resistant coupling beams spanned several investigations conducted for over a decade (Canbolat et al., 2004; Parra-Montesinos et al., 2010; Lequesne, 2011; Setkit, 2012; Lequesne et al., 2013). Thus, only tests and results most related to the coupling beam design ultimately adopted in a high-rise structure recently completed in the city of Seattle, WA, USA, will be presented.

Tests were conducted on isolated coupling beams, as shown in Figure 2. Each test specimen consisted of a coupling beam connected to two large reinforced concrete (RC) blocks simulating the edges of two structural walls. For testing convenience, each coupling beam subassembly was rotated 90 degrees. The tests were conducted at the University of Michigan Structural Engineering Laboratory. The bottom block in Figure 2 was anchored to a strong floor. The top block was connected to a hydraulic actuator, which was connected at the other end to a reaction wall. Lateral displacements were applied to the top block. The vertical steel links shown in Figure 2 restrained rotation of the top block, ensuring double curvature bending of the coupling beam. Also, these links provide some level of axial restraint, which in a real structure is provided by the structural walls and slab. Lateral displacement cycles of increasing drift magnitude were applied to the coupling beams up to failure. Drift, as reported herein, corresponds to the chord rotation of the coupling beam.

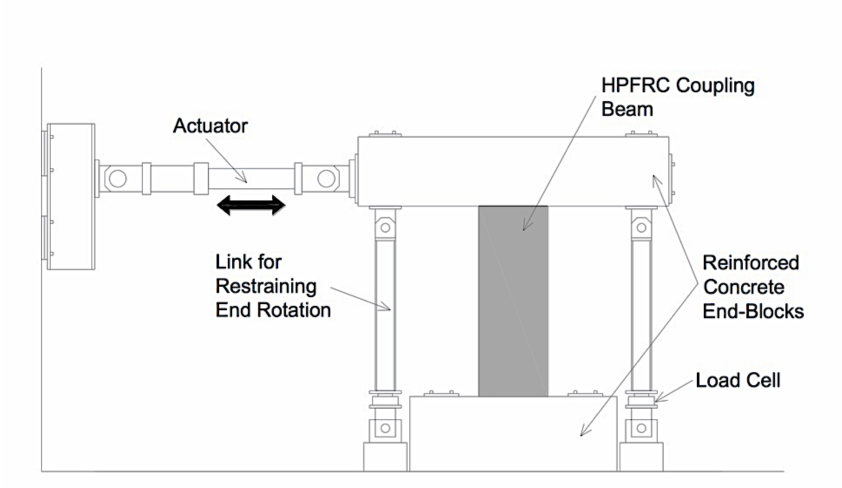


Figure 2: Coupling beam test setup.

Reinforcement details for the three coupling beam tests discussed herein are provided in Figure 3. Coupling beam span-to-overall depth ratio was either 2.2, 2.75, or 3.3. No diagonal reinforcement was used in any of the three coupling beams. As shown in Figure 3, closely spaced transverse reinforcement was provided at the ends of the coupling beams, over a distance half the beam depth from each end. This reinforcement complied with transverse reinforcement required for confinement at the ends of columns in special moment resisting frames, according to Chapter 21 of the 2011 ACI Building Code (ACI Committee 318, 2011). In the remaining span of the coupling beam, single closed hoops were provided. Assuming that shear strength in the HPFRC coupling beams is provided by the HPFRC material and hoops, the calculated shear stress demand on the HPFRC material in the middle portion of the beam, assuming yielding of the hoops, was $0.56\sqrt{f_c'}$, $0.22\sqrt{f_c'}$, and $0.30\sqrt{f_c'}$ (MPa) for beams with span-to-overall depth ratios of 2.2, 2.75, and 3.3, respectively.

All three coupling beams were precast. This was intended to make the proposed design more flexible, as precast construction may prove more advantageous in certain conditions. As the connection between the coupling beam and the walls is more critical in precast construction, the same design can be applied to cast-in-place coupling beams. To prevent interference with the wall boundary reinforcement, the precast portion of the HPFRC coupling beams extended only to the wall cover (see shaded grey area in Figure 3). Longitudinal bars were extended beyond the precast portion of the beam a full development length for anchorage. U-shaped bar reinforcement crossing the beam-wall interface was used to force most of the beam inelastic deformations to occur away from the cold joint, preventing concentration of damage at the cold joint and thus, a premature sliding shear failure.

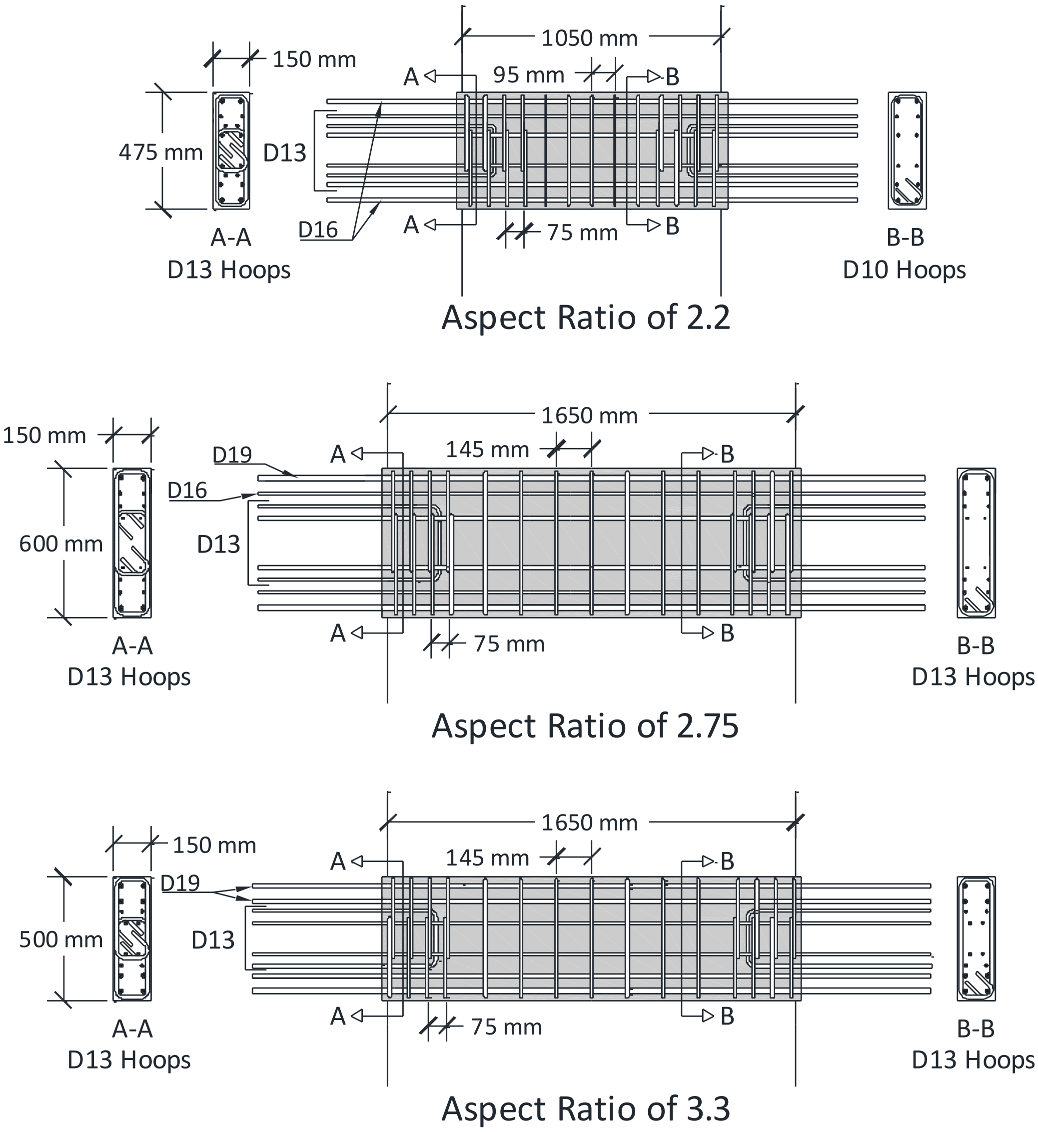


Figure 3: Reinforcement details for test coupling beams.

All specimens discussed herein were constructed with a single HPFRC material containing high-strength hooked steel fibres in a 1.5% volume fraction. The fibres used were 30 mm long and 0.38 mm in diameter, made out of a wire with minimum tensile strength of 2300 MPa. The HPFRC mixture proportions by weight were 1:0.875:2.2:1.2:0.8:0.005:0.038 for cement:fly ash:sand:course aggregate:water:high-range water reducing agent:viscosity modifying agent. The course aggregate was crushed limestone with a maximum size of 13 mm. This HPFRC mixture exhibited high workability, so only minimum vibration was required during casting. Detailed information about this mixture can be found elsewhere (Liao et al., 2006). As shown in Table 1, the compressive strength of this HPFRC mix, obtained from tests of 100 x 200 mm cylinders, was 63 MPa for the coupling beam with an aspect ratio of 2.2, and 68 MPa for the other two test coupling beams.

Table 1: Summary of test results

l_n/h	b, mm	h, mm	f_c'*, MPa	V_{max}, kN	$V_{max} / (bh\sqrt{f_c'})$, MPa	Drift Capacity**
2.2	150	475	63	570	1.00	5.8 %
2.75	150	600	68	540	0.73	5.7 %
3.3	150	500	68	500	0.81	6.8 %

* Concrete cylinder compressive strength at test day.
** Largest drift achieved in both directions with less than 20% strength loss
b: thickness; h: overall depth; V_{max}: maximum shear

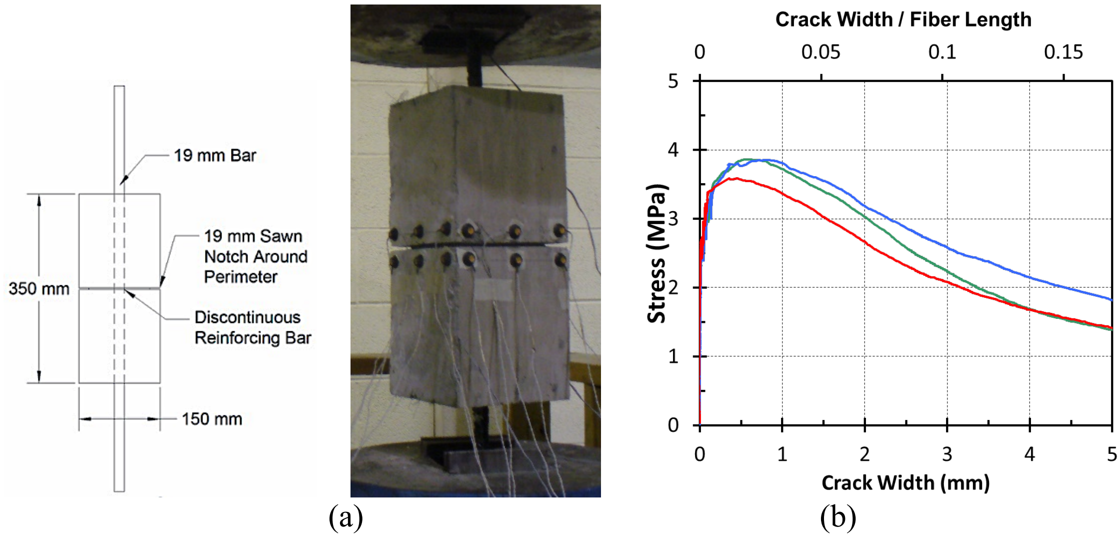


Figure 4: Proposed tensile test: (a) specimen details; (b) sample result.

2.2 Tensile and flexural behaviour of HPFRC

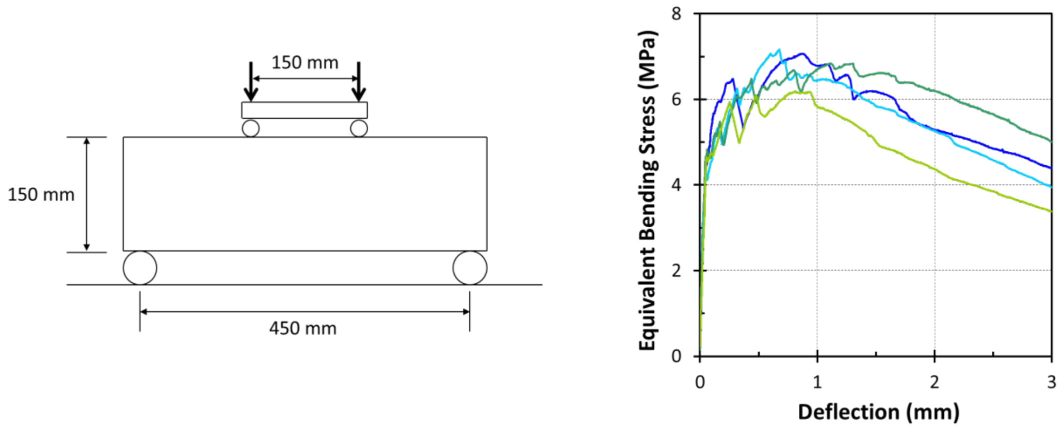
There is no standardized tensile test protocol for fibre-reinforced concrete. This is due primarily to difficulties associated with connecting a material specimen sufficiently large to be representative of field conditions to conventional testing equipment. There is also no agreement among researchers about the proper boundary conditions for such a specimen.

A tensile test developed by co-writers Parra-Montesinos, Lequesne, and Conforti was used to evaluate the tensile behaviour of an HPFRC with the same matrix mixture proportions, fibre type, and fibre content as that used in the test coupling beams. This test can be conducted using standard reinforcing bar test equipment and largely diminishes fibre alignment due to edge effects. The test specimens consisted of a 150 by 150 by 350 mm FRC prism with an embedded 19 mm reinforcing bar used for loading purposes (Figure 4a). The reinforcing bar was pre-cut in order to make it discontinuous at the centre of the specimen in order to reduce restraint against shrinkage. A 19 mm deep notch was sawn prior to testing around the perimeter of the specimen, at a location coinciding with the cut in the reinforcing bar. Tests in which the reinforcing bar was continuous were also conducted in order to better study the interaction between reinforcing steel and cracked FRC. The uncertainty in the estimation of the effect of stresses induced by restrained shrinkage on specimen behaviour, however, made the use of data from these continuous bar tests questionable.

For testing (Figure 4a), the reinforcing bars protruding from the ends of the FRC prism were held by standard bar grips. Specimens were loaded at a rate of 0.002 mm/s until cracking, followed by a rate of 0.006 mm/s until the average crack width exceeded 5 mm. Non-contact infrared-based instrumentation was used to determine crack widths. Figure 4b shows representative tensile stress versus average crack width responses obtained from specimens cast with the same concrete mixture used in the coupling beam tests. The specimens cracked at a stress that was typically between 2.3 and 3 MPa. The peak tensile strength, which was, on average, 30% higher than the first cracking strength, occurred at a crack width of approximately 0.5 mm. The fact that the peak stress was significantly higher than the first cracking stress indicates that this fibre-reinforced concrete was an HPFRC.

HPFRC samples with dimensions of 150 by 150 by 500 mm were also subjected to third-point flexural tests in accordance with ASTM C1609 (Figure 5a). Typical equivalent bending stress versus midspan deflection responses are shown in Figure 5b. Equivalent bending stress refers to the tensile stress calculated at the bottom of the section assuming linear elastic behaviour and uncracked section properties. Specimens consistently exhibited higher strength after first cracking, with peak strengths exceeding the first cracking strength by approximately 40%. The peak strength occurred at a midspan deflection of 0.9 mm on average.

The predicted flexural responses that would be obtained by using the tensile stress vs. crack width data shown in Figure 4b would have trends similar to the behaviour shown in Figure 5b. However, the calculated equivalent flexural stresses would generally be greater, as the tensile stress versus crack width data were obtained from a notched specimen, while the ASTM C1609 beams were unnotched.



(a) (b)

Figure 5: ASTM C1609 Flexural test: (a) schematic of test setup; (b) sample result.

3 Results and discussion

Figure 6 shows the average shear stress versus drift response for the three coupling beam specimens. Average shear stress was calculated as the applied shear divided by the beam cross sectional area. As shown in Figure 6, all three beams exhibited stable hysteresis up to drifts exceeding 5% (0.05 rad). No significant shear-related damage was observed in the middle region of the beam throughout the test, even though the peak shear stress ranged between 0.73 and $1.0\sqrt{f_c'}$ (MPa). It is worth mentioning that the maximum shear stress allowed in design in

Chapter 21 of the ACI Building Code (ACI Committee 318, 2011) is $0.83\sqrt{f_c{}'}$ (MPa). Drift at failure, which was defined as the largest drift achieved in both directions prior to a loss of strength of 20% or greater was close to 6% for the beams with an aspect ratio of 2.2 and 2.75, and slightly less than 7% for the beam with a 3.3 aspect ratio. A summary of the beam test results is shown in Table 1.

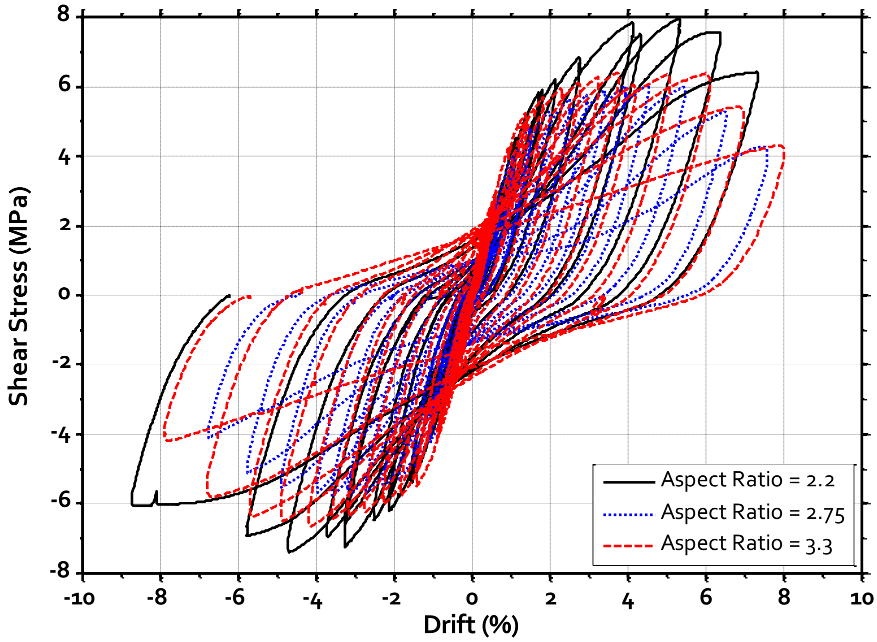


Figure 6: Shear stress versus drift behaviour for test specimens.

Figure 7: Damage in coupling beam with aspect ratio of 3.3 at 3% (left) and 5% (right) drift.

Plastic deformations and damage in the coupling beams concentrated at the beam ends, but no concentration of damage occurred at the cold-joint between the precast HPFRC beam and the cast-in-place wall concrete, indicating that the intermediate reinforcement served the intended purpose of strengthening this interface and forcing most of the inelastic deformations within the beam span. Failure of the coupling beams was caused by the development of wide flexural cracks, along which sliding deformations developed, leading to a significant loss of stiffness and strength. At coupling beam drifts of approximately 4%, however, damage in the coupling beams was relatively modest, which indicates that the HPFRC coupling beams are highly tolerant to large inelastic deformations. Figure 7 shows damage in the coupling beam with an aspect ratio of 3.3 at 3 and 5% drift.

4 Use of HPFRC coupling beams in a high-rise structure

The coupling beam design shown in Figure 3 was implemented in a high-rise building in the city of Seattle, WA, USA. The building, The Martin (Figure 8), is a 73 m tall, 24-story core-wall structure designed by Cary Kopczynski & Co from Bellevue, WA, USA. Approval of the use of HPFRC coupling beams was achieved through a peer-review process. Results from the tests described above were used as the basis for analysis and design. Because approval from building officials was obtained after construction had begun, the HPFRC coupling beam design was implemented from the 12th floor up.

Each floor level had five coupling beams and each HPFRC coupling beam had a 76 x 46 cm cross section, with a span length of 1.3 m for a span-to-height ratio of 2.8. Only longitudinal and transverse reinforcement was used in the beams (i.e., no diagonal bars), as in the test specimens discussed herein, which substantially facilitated construction. Design shear stresses in the coupling beams ranged from $0.33\sqrt{f_c'}$ to $0.71\sqrt{f_c'}$ (MPa) and transverse steel in the middle portion of the beam was designed such that the shear stress demand in the HPFRC material would not exceed $0.29\sqrt{f_c'}$, MPa. This was consistent with the coupling beams tested. As in the test specimens, special column-type confinement reinforcement was provided over half the beam depth from each beam end. Longitudinal reinforcement ranged from 5-No. 22M bars top and bottom for the coupling beams with lower shear stress demands to 3-No. 35M + 3-No. 32M top and bottom. Specified concrete strength ranged between 41 and 69 MPa.

Ready-mix fibre-reinforced concrete was used, with the fibres added at the concrete plant. The fibres were of the same type (high-strength hooked steel fibres) and used in the same dosage (1.5% volume fraction) as in the test specimens. Prior to addition of fibres, the mixture used exhibited self-consolidating properties. To verify that adequate fibre distribution could be achieved, small samples were cast and cut after hardening of concrete for visual inspection. A photo of fresh fibre-reinforced concrete used in the coupling beams is shown in Figure 9a. Rather than using a precast operation as in the experimental program, the coupling beams were cast-in-place, using a crane and bucket operation (Figure 9b). Given the fact that the HPFRC material did not penetrate into the core of the wall boundary region, intermediate reinforcement was used as in the precast coupling beams to force most of the inelastic deformations to occur away from the cold joint and thus prevent a premature sliding shear failure. Overall, the construction of cast-in-place HPFRC coupling beams proved to be a very practical and successful operation.

Figure 8: The Martin, a core-wall structure with HPFRC coupling beams in Seattle, WA, USA.

(a) (b)

Figure 9: (a) Fibre distribution in fresh fibre-reinforced concrete used in The Martin building (fibre length: 30 mm); (b) casting of fibre-reinforced concrete coupling beam in The Martin building.

5 Conclusions

Results from tests of large-scale coupling beams under simulated earthquake-induced displacements proved that a complete elimination of diagonal bars and a substantial reduction in confinement reinforcement is possible in coupling beams constructed with a high-performance fibre-reinforced concrete (HPFRC) and with span-to-overall depth ratios greater than or equal to 2.2. The behaviour of the test specimens was governed by flexural yielding at the beam ends with little shear-related damage. Ultimate failure of the coupling beams was caused by sliding along wide flexural cracks at the beam ends, but at coupling beam drifts greater than 5%.

The elimination of diagonal bars and reduction in transverse reinforcement proved to be advantageous from a construction viewpoint, as evidenced by the experience in the construction of a high-rise building in the city of Seattle, WA, USA, where cast-in-place HPFRC coupling beams were used.

For evaluation of the tensile behaviour of HPFRC mixtures, tensile tests on 150 x 150 x 350 mm notched prisms with an embedded bar discontinuous at the notched section proved to be simple and gave consistent results up to crack widths as large as 5 mm. This tensile test can be conducted using standard reinforcing bar test equipment and largely diminishes fibre alignment due to edge effects by allowing the testing of large prisms.

6 References

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