TESTING OF FIBRE REINFORCED PRECAST CONCRETE BEAM-COLUMN JOINTS

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ABSTRACT

An innovative method of constructing equivalent monolithic moment resisting frames from precast elements is proposed. This method places in-situ concrete connections in the joint core regions, and uses ductile fibre reinforced cementitious composites (DFRCC) to ensure the robustness of these connections whilst significantly reducing reinforcement congestion and connection complexity. Although DFRCC is a relatively expensive product it is felt that the small volume of the connection and the ease of construction will make this method a practical alternative for real structures. Two beam-column joint sub-assemblies were constructed using this method and subjected to simulated seismic loading. The test units behaved well until very high drift and ductility levels were reached. Failure of the test units was either by buckling of the beam plastic hinges or shear failure of the joint core, depending on the anchorage detail of the beam longitudinal reinforcement in the joint core.

Precast Concrete Moment Resisting Frames in New Zealand

Throughout the world precast concrete is recognised as an attractive construction solution with several advantages over in-situ concrete construction. These include faster construction, better quality, reduced formwork requirements and a reduced requirement for skilled labour on site (fib Task Group 7.3 2003). In New Zealand precast concrete has been commonly used for structural purposes including moment resisting frames since the 1980's (Bull 1999) and remains popular today (fib Task Group 7.3 2003).

The majority of structures in New Zealand must be designed for seismic resistance. To achieve this New Zealand design standards (NZS 1170.5:2004; NZS 3101:2006) require structures to be designed according to the capacity design principles developed by Park and Paulay (1975). When applied to moment resisting frames this design philosophy generally requires plastic deformation to be limited to hinges forming in the beams, and for columns and joints to be designed so that their strength is guaranteed to be greater than the actions that could be imposed on them when a plastic mechanism forms. This design method provides excellent seismic resistance; however the beam-column joints of such structures tend to be very congested with reinforcement for two reasons:

- A large number of longitudinal bars from the beams must pass through the joint since the diameter of such bars is restricted in order to prevent bond failure from occurring in the joint core.
- Large quantities of transverse reinforcement must be used to prevent joint shear failure from occurring.

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Congestion typical of New Zealand beam-column joints is shown in Fig. 1. Researchers have previously attempted to alleviate reinforcement congestion at beam-column joints by using fibre reinforcement to reduce the need for conventional transverse reinforcement, although until recently these attempts have been largely unsuccessful (Parra-Montesinos et al. 2005).

Figure 1.   Reinforcement congestion in a beam-column joint core.

According to the *fib* (2003) precast concrete structures including moment resisting frames can be designed either to emulate the behaviour of cast in-situ structures ("equivalent monolithic" construction) or to have deformations concentrated at the connections between precast elements ("jointed" construction, as exemplified by the PRESSS project (Priestley et al. 1999)). Precast concrete moment resisting frames in New Zealand have almost exclusively been constructed so as to emulate in-situ construction (Park 1995). The performance of structures designed to emulate in-situ construction relies on the connections between precast elements having strength and stiffness at least equal to the adjacent precast elements.

Three methods are commonly used to construct equivalent monolithic frames in New Zealand. These were identified by Park (1995) and are shown in Fig. 2. While the performance of the methods shown in Fig. 2 is not in doubt (Beattie 1989; Restrepo et al. 1995), each method is affected by one or more of the following problems:

- Large column depths required to anchor discontinuous beam reinforcement
- Requirement to assemble complex, congested joint core reinforcement on site
- Tight tolerances required to ensure reinforcement can be inserted into ducts
- Use of heavy, awkward shaped precast elements

Whilst these problems have not prevented the use of systems 1-3, it was felt that construction could potentially be simplified by developing an improved construction methodology.

Simplified Moment Resisting Frames using DFRCC

Researchers at the University of Auckland have developed an improved method for constructing precast moment resisting frames. This method is felt to overcome the fundamental shortcomings of systems 1-3 by taking advantage of the properties of ductile fibre reinforced cementitious composites (DFRCC).

Ductile Fibre Reinforced Cementitious Composites

There has recently been much interest worldwide in cement based materials that exhibit strain hardening properties when subjected to axial tension. These materials are described variously in the literature as "High Performance Fibre Reinforced Cementitious Composites" (HPFRCC); "Engineered Cementitious Composites" (ECC) and "Ductile Fibre Reinforced Cementitious Composites" (DFRCC). The use of these names appears to be geographically based. In this paper the name DFRCC will be used, as it is the opinion of the authors that this name is the most descriptive of the properties of the material.
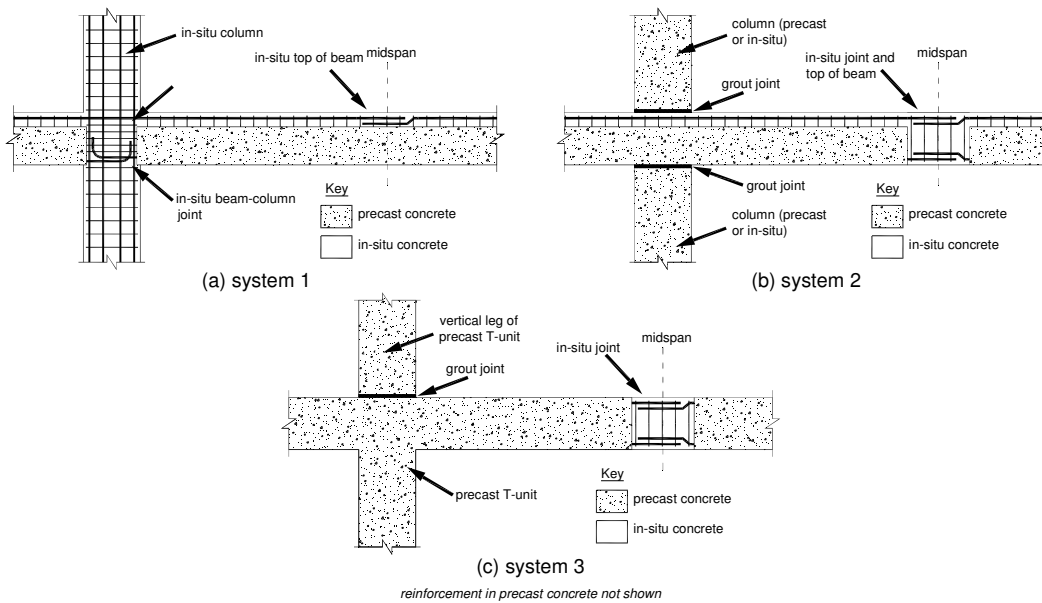
Figure 2.   Methods used to construct precast concrete frames in New Zealand (after Park 1995).

Fig. 3 shows the tensile stress-strain response of four cementitious composites tested by Li et al. (1994). This figure shows the poor performance of plain concrete (PC), and also the tension softening response of ordinary fibre reinforced concrete (FRC, with 1% by volume of hooked steel fibres). When ordinary fibre reinforced concrete is used in real structures this tension softening response leads to early damage localisation, and often poor structural performance. Fig. 3 also shows the tensile stress-strain response of two types of DFRCC, labelled SPECC and DRECC. Whilst clearly having quite different properties these materials both display a strain hardening response, which is considered the key characteristic of DFRCC. A second key characteristic of DFRCC is that the volume fraction of fibres used is typically around 2%, which allows easy mixing and placing of the material. Li (2003) provides more detail on the design and characteristics of DFRCC.
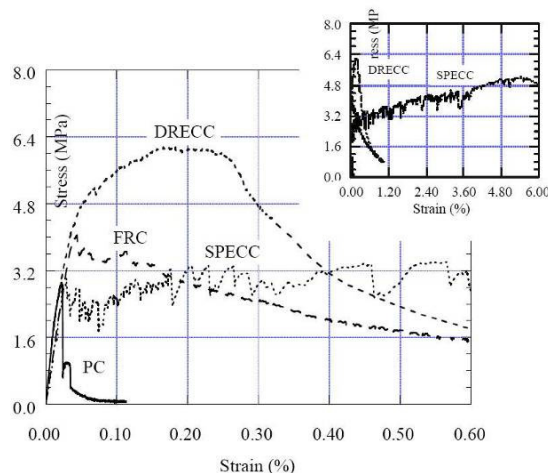

Figure 3.   Tensile stress-strain response of cementitious composites reported by Li et al. (1994).

When used in structures or structural components the toughness and ductility of DFRCC gives numerous advantages over more traditional materials, including (Li 2003):

- reduction or elimination of conventional shear reinforcement;
- ability to sustain large deformations without damage/deformation concentrating at a single crack;
- reduced reliance on the bond between reinforcement and concrete due to the ability of DFRCC to transmit tensile stresses even after cracking has occurred;
- elimination of spalling even when very large cyclic deformations are imposed on a structure.

These advantages translate particularly well to structures expected to be subjected to severe cyclic loading.

Improved Method for Constructing Precast Moment Resisting Frames

The development of a new method for connecting precast elements to form moment resisting frames had two key aims. These were to avoid the use of large, awkward shaped precast elements, and to avoid the need to assemble complex reinforcing details on site. To meet the first requirement it was decided to place the connection between elements in the joint core, which allowed all precast elements to be of a rectangular shape. Using traditional rules to design such a connection would require the assembly of complex joint reinforcement on site, as in system 1 from Fig. 2. The new connection method avoids this by using DFRCC to provide the necessary shear strength in the in-situ connection/joint core, without the need for conventional shear reinforcement. The design concept is shown in Fig. 4.

Compared to ordinary concrete DFRCC is expensive, and it is recognized that construction is a highly cost sensitive industry. However, it is envisaged that the relatively small quantity of DFRCC used by the connection method shown in Fig. 4 (approximately 10% of the total volume of the frame) in combination with enhanced speed of construction will make this method economical when considered holistically.
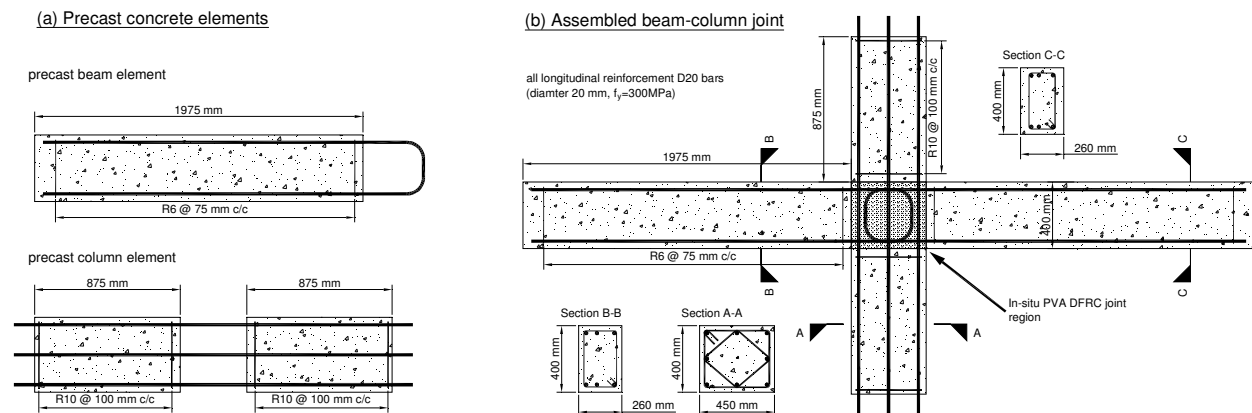

Figure 4.   Precast beam-column joint design used for PVA-1 and PVA-2.

Two beam-column joints (PVA-1 and PVA-2) were constructed based on the design shown in Fig. 4. Apart from the exceptions discussed below, these units were designed in accordance with the New Zealand concrete design standard (NZS 3101:2006). The precast beam and column elements of units PVA-1 and PVA-2 were cast in a single pour. The concrete used was an ordinary readymix concrete supplied by Firth Industries with a specified 28 compressive strength of 35 MPa, a specified slump of 120 mm and maximum aggregate size of 19 mm. All reinforcement used was supplied by Pacific Steel and had a nominal yield strength of 300 MPa. The joint core region, which also functioned as the connection between precast elements, was poured in-situ using PVA fibre reinforced concrete.

PVA-1 and PVA-2 were nominally identical, except for the reinforcement detailing used in the joint region. The beam longitudinal reinforcement of PVA-1 was anchored in the joint core with a "U" bend as shown in Fig. 4 and Fig. 5(a). The length of this "U" development length was based on the hooked development

length specified by NZS 3101 for exterior beam-column joints. This was felt to be conservative due to the high bond strength of fibre reinforced concrete. For PVA-2 the anchorage detail was changed to a 300 mm (15 bar diameters) straight development length for the reinforcement of each beam, as shown in Fig. 5(b). This detail did not meet the requirements of NZS 3101, which does not allow straight development lengths adjacent to a plastic hinge region.
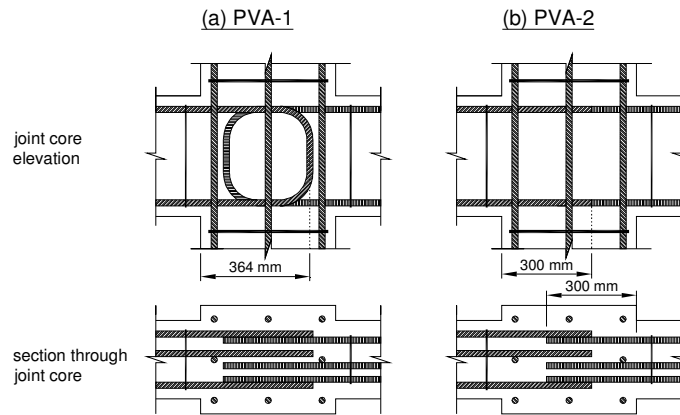
Figure 5.   Joint core reinforcing details of PVA-1 and PVA-2.

Neither PVA-1 nor PVA-2 contained any conventional transverse reinforcement in the joint core. As mentioned it was envisaged that the DFRCC used in the joint core would provide sufficient shear strength to transfer forces between the beams and columns and to allow formation of plastic hinges adjacent to the joint. The shear stress imposed on the joint was 3.4 MPa, which was approximately 34% of the maximum value permitted by NZS 3101. If the joints had been detailed conventionally according to NZS 3101 it would have been necessary to provide shear reinforcement with a strength of 343 kN in the joint region (for instance 4 sets of 3 legged 10 mm stirrups with a yield strength of 500 MPa). This quantity of shear reinforcement would have been required for both PVA-1 and PVA-2.

PVA Fibre Reinforced Concrete

The DFRCC used in this project contained 2% polyvinyl alcohol (PVA) fibres by volume. PVA is considered to be a particularly suitable material from which to manufacture fibres for DFRCC (Wang and Li 2005). The fibres used were Kuralon RECS 15/8, developed and manufactured by Kuraray specifically for use in DFRCC. The fibres have a length of 8 mm, a diameter of 0.04 mm and tensile strength of 1600 MPa. Full fibre properties are available from Kuraray's website (2006). The mix design used to manufacture the DFRCC is outlined in Table 1 and was closely based on one provided by Kuraray. Between the construction of PVA-1 and PVA-2 the type of sand used in the DFRCC was changed from a typical sand used in regular concrete to a much finer sand with over 80% of particles having a diameter less than 300 μm. This change was made because the original DFRCC mix was not displaying the desired multiple cracking behaviour.

The material properties of this DFRCC have not yet been properly characterised as no direct tensile tests have been conducted. Cylinder tests have shown a 28-day compressive strength of approximately 70 MPa, while results from the testing of small beams are shown in Fig. 6. The beams were loaded in 4-point bending, had a cross section of 50 mm square and a span of 270 mm. For comparative purposes Fig. 6 shows results of beams made from DFRCC using the ordinary sand, fine sand and also beams with no fibres.

Since the tensile stress-strain response of the material is not yet known it is inappropriate to report flexural strength as a stress value, and Fig. 6 instead shows applied force plotted against displacement. It can be

seen that adding PVA fibres to the beams doubled their flexural strength and dramatically increased their deformation at failure. With the exception of one beam, Fig. 6 shows only a minor improvement in the behaviour of specimens using fine sand compared to those using ordinary sand. However, the failure of the two sets of beams was quite different. Beams containing ordinary coarse sand cracked at a single location, after which all deformation was concentrated at this point. In contrast, beams containing fine sand were seen to develop multiple well distributed cracks before damage localisation and failure occurred at a single crack. While multiple cracking in flexural beam specimens is not necessarily indicative of tensile strain hardening properties (Li and Wang 2003) it is believed that this multiple cracking is the cause of the improved "post elastic" response visible in Fig. 6 for the beam specimens from PVA-2.

Table 1.   DFRCC mix design.

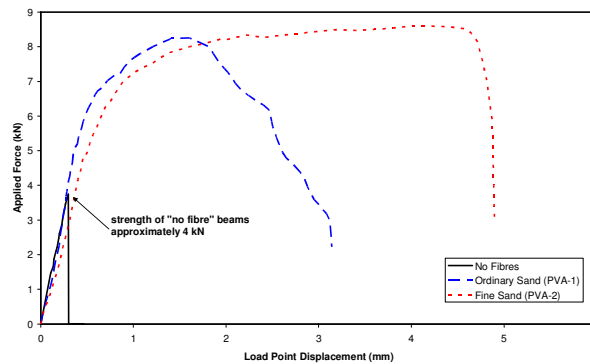| Material	Quantity (kg/m^3)
Sand	600
Cement	600
Fly ash	600
Water	230
Super plasticiser	18
Fibres	26

Figure 6.   Force-displacement response of 50x50x270 mm concrete beams.

Test Method

Units PVA-1 and PVA-2 were installed in a test frame designed to apply cyclic loading to simulate the effect of an earthquake. The frame used is shown in Fig. 7. The beams and columns are pin supported at their ends to allow rotation. The beams are restrained in the vertical plane by supports with integral load cells. The top of the column is restrained in the horizontal plane but allowed to move vertically, while the column base is supported on rollers and displaced cyclically by the actuator shown. This induces moments in the beams and simulates the forces that the sub-assembly would be subjected to if it was part of a moment resisting frame during an earthquake. The test setup does not allow axial loads to be applied to the column.
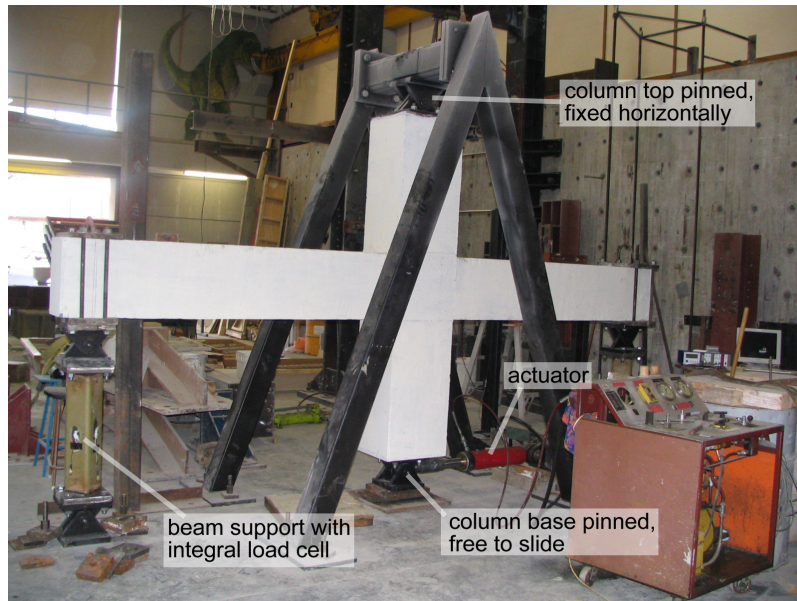
Figure 7.   Test setup.

The two sub-assemblies tested were extensively instrumented with a variety of displacement transducers. These transducers allowed overall displacements of the sub-assembly to be monitored and also allowed flexural and shear deformations of the beams, columns and joint core to be measured. Transducers were also installed to monitor slip of the beam longitudinal reinforcement within the joint core. A cyclic load history was used as illustrated in Fig. 8. Load cycles were defined as increments of interstorey drift, starting at 0.5% and increasing by intervals of 0.5% and 1.0% depending on the previous displacement imposed. The test unit was deemed to have failed during a cycle if it was unable to support 80% of the maximum load supported during any previous load cycle in the same direction of loading.
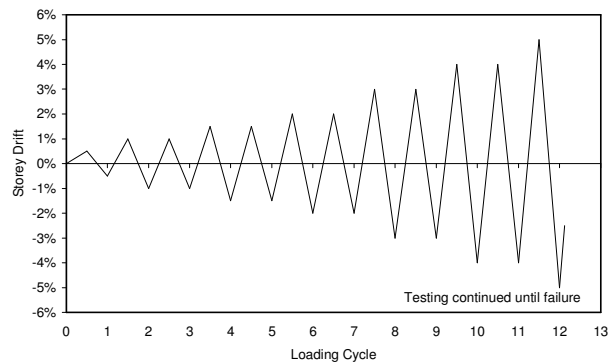

Figure 8.   Load history applied to test units.

Experimental Results

The performance of PVA-1 and PVA-2 when subjected to a cyclic load history was satisfactory. For both units the interstorey drift at first yield was approximately 0.64%, at which point stable, bi-directional plastic hinges formed in the beams on both sides of and adjacent to the joint core. Fig.9 shows the force-displacement history of the two units, from which it can be seen that the hysteresis loops of both units were stable until the imposed interstorey drift exceeded 4%, which corresponded to a displacement ductility of $\mu = 6.25$. These levels of interstorey drift and displacement ductility exceed the respective

limits of 2.5% drift and μ = 6 that can be used in design according to New Zealand standards (NZS 1170.5:2004; NZS 3101:2006).
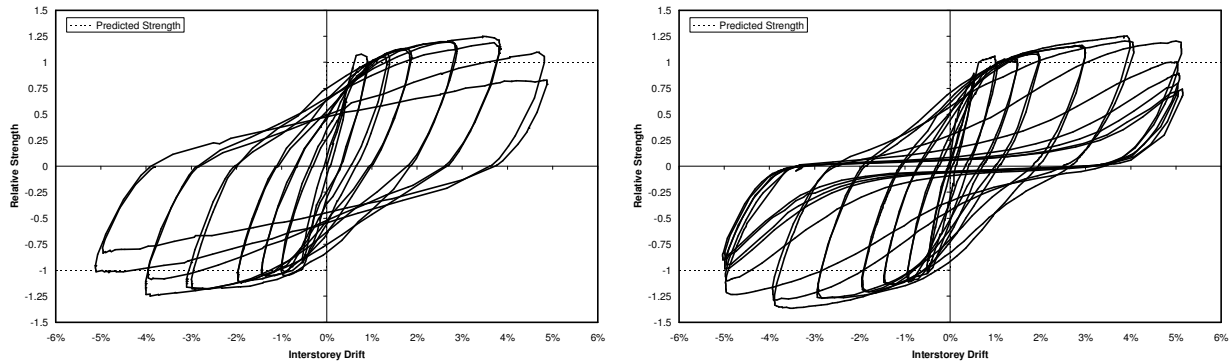

Figure 9.   Force-displacement history of unit PVA-1 (left) and PVA-2 (right).

According to the definition of failure given previously, units PVA-1 and PVA-2 both failed during the second cycle to 5% interstorey drift. The cause of these failures was different for the two units. PVA-1 failed as a result of longitudinal reinforcement buckling in the beam plastic hinges. In contrast PVA-2 failed when opening of wide cracks in the joint core resulted in a loss of stiffness. This joint shear failure is visible in Fig. 9 as the hysteresis loops of PVA-2 became increasingly pinched during the multiple cycles conducted to 5% interstorey drift.

Deformation of Joint Core

The most significant departure from code-specified design procedures found in units PVA-1 and PVA-2 was the total absence of conventional transverse reinforcement in the joint region. It is therefore of interest to examine the deformation of the joint cores of units PVA-1 and PVA-2. This deformation is shown in Fig. 10.
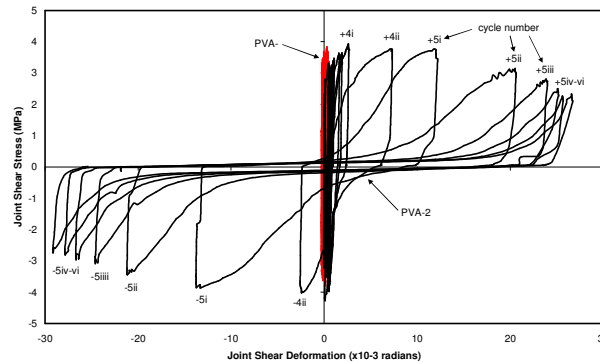

Figure 10.   Stress-deformation history of unit PVA-1 and PVA-2.

From Fig. 10 it is evident that at all points during testing the joint core deformation of PVA-1 was negligible, at no point exceeding 0.0006 radians. In contrast the joint core of PVA-2 deformed significantly, but only during cycles to 4% interstorey drift or more. This verifies the previous assertion that joint shear failure occurred in PVA-2. The difference between the condition of the joint cores of PVA-1 and PVA-2 during the cycles to large interstorey drifts is clearly evident in Fig. 11. While the joint of PVA-1 is largely uncracked, that of PVA-2 is crossed by a matrix of fine cracks which developed during earlier load cycles before damage localised at the wide diagonal crack visible in Fig. 11 during cycles to 4% and 5% interstorey drift.

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Figure 11.   Joint core condition of PVA-1 (left) and PVA-2 (right) at 5% interstorey drift.

Given that the properties of the DFRCC used in the joint cores of PVA-1 and PVA-2 were similar (see Fig. 6), and that the imposed joint shear stress for both units was similar it must be concluded that the poorer joint shear performance of PVA-2 was a result of the altered reinforcement anchorage detail used (see Fig. 5). The specific reason for this behaviour change has not yet been identified, but is believed to be related to stress concentration at the points where the beam longitudinal reinforcement terminated in unit PVA-2.

Anchorage of Beam Reinforcement in Joint Region

For a beam-column joint to perform in a desirable manner it is important that the beam longitudinal reinforcement remain well anchored in the joint core when subjected to cyclic loading. PVA-1 and PVA-2 used different methods to anchor the beam longitudinal reinforcement in the joint core.   PVA-1 had hooked anchorages designed to comply with New Zealand design requirements, while PVA-2 had non-compliant straight bar anchorages (see Fig. 5). During testing the effectiveness of these anchorages was monitored by measuring reinforcement movement within the joint core. This data is shown in Fig. 12.
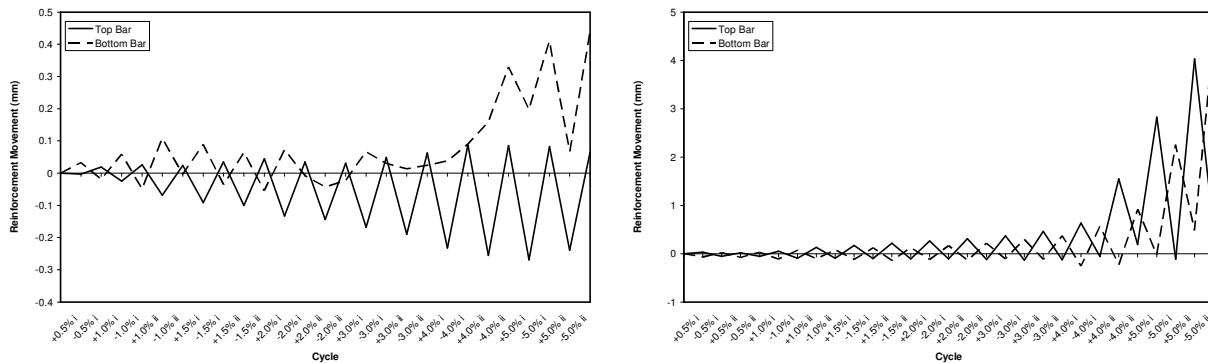

Figure 12.   Beam longitudinal reinforcement slip in PVA-1 (left) and PVA-2 (right).

Fig. 12 shows that throughout testing the beam reinforcement of PVA-1 remained well anchored in the joint core. This was expected because the anchorages were conservatively designed to meet New Zealand standards, and fibre reinforced concrete is known to anchor reinforcement more effectively than plain concrete (Parra-Montesinos et al. 2005). Fig. 12 also shows that the beam reinforcement of PVA-2 remained well anchored during testing. The apparent large reinforcement movement during cycles to 4.0% and 5.0% drift were not actually reinforcement slip; rather false readings were generated due to the

deformation of the joint core during these cycles. The performance of the reinforcement anchorage in PVA-2 exceeded expectations, as straight bar anchorages do not typically perform well when located adjacent to plastic hinges, and are not allowed in such locations by New Zealand standards.

Conclusions

An innovative method of constructing precast concrete moment resisting frames has been developed, which relies on the properties of DFRCC rather than conventional transverse reinforcement to resist shear forces in the beam-column joint core. When subjected to cyclic loading two beam-column joints constructed using this method performed in a satisfactory manner until imposed drift and ductility levels exceeded those allowed by New Zealand design standards, despite the absence of joint shear reinforcement normally required by these standards. The ultimate failure of the beam-column joints was affected by the detail used to anchor the beam longitudinal reinforcement in the joint core, although both details used were shown to adequately anchor the beam reinforcement. Currently there is insufficient test data available to recommend design rules for frames designed using the method outlined in this paper. Developing such recommendations is the focus of ongoing research at the University of Auckland.

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