SHEAR TESTING AND COMPARATIVE DESIGN STANDARDS OF STEEL FIBRE REINFORCED HOLLOWCORE SLABS

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SUMMARY

Prestressed concrete Hollowcore Slabs (PCHS) exhibited significant relative displacements leading to early shear failures following the 2016 Mw7.8 Kaikōura earthquake. Due to the multi-directional nature of the ground accelerations, which exceeded 1.0 g horizontal in several locations, the lightweight floor units were commonly observed to have also suffered web shear cracking across multiple commercial structures. As a result, Stahlton[1] commissioned the University of Canterbury (UC) to carry out extensive research into the application of steel fibre reinforcement to increase shear capacity and improve residual post-peak strength. In total, 26 units were tested with: 12 x 200 mm, 14 x 300 mm height specimens. Within each size group, four units were conventional high strength concrete; four were reinforced with short steel fibres and four with long steel fibres. Substantial improvements in toughness and post-peak shear resistance were detected with the inclusion of steel fibres. Hence, it was concluded that the incorporation of steel fibres in PHCS is able to enhance the post-peak performance, i.e. residual strength, but does not provide an important improvement in the maximum shear resistance.

INTRODUCTION

Inspections following the 2016 Kaikōura earthquake exposed important damages in precast prestressed hollowcore floors. Contrasting with 2010-11 Canterbury Earthquakes, in which observed damage was mostly due to diaphragm action (Corney et al., 2014), Kaikōura earthquake triggered failures in PHCS units (Henry et al., 2017). A typically observed damage pattern was transverse cracks. These crack patterns are usually initiated by flexural-shear, or web-shear failure mechanism, which are predominantly brittle owing to shear forces in PHCS are purely resisted by high-strength plain concrete and prestressed strands. Therefore, residual shear strength of PHCS results essential to carry gravity loads and reduce the local or global collapse risk in the seismic event or aftershocks.

Previous research on steel fibre reinforced concrete (SFRC) hollowcore slabs reported improvements in ductility, peak shear strength and residual shear strength (Paine, 1998,

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Tasligedik, 2015, Dudnik et al., 2017, Simasathien and Chao, 2015). Experimental tests by Stahlton (Tasligedik, 2015) informed increases by about 20% in the maximum shear resistance of PHCS. However, the results were not conclusive due to the limited number of tests (8 Specimens) and test setup used. To assess these findings in more detail, an experimental programme was commissioned to the University of Canterbury to further research on the shear behaviour of SFRC-PHCS.

The following paper presents the summary results of 26 full-scale shear tests on PHCS. The experimental programme comprises 200mm and 300mm height PHCS. Normal concrete (NC) and SFRC with two steel fibre dosages, 13 kg/m3 and 26 kg/m3 (0.17% and 0.33% by volume) were investigated. Moreover, for each series of specimens two monotonic and two cyclic loading tests were carried out. Comparisons between experimental results and design standard (NZS 3101.1&2:2006) are presented.

DESIGN SHEAR STRENGTH OF SFRC HOLLOWCORE SLABS (NZS 3101.1&2:2006)

New Zealand concrete structures standard (NZS 3101.1&2:2006) refers to an approach advocated by RILEM TC162-TDF (2003) for SFRC members design. According to the standard, this methodology is applicable to determine the shear resistance of SFRC prestressed members (Appendix C5.A4.2). Nonetheless, the formulation of this approach presented in the NZS 3101, is not explicitly aimed to determine PHCS shear resistance. Therefore, NZS3101 section 19.3.11 is preferred to determine the shear contribution of the concrete and steel fibres contribution is determined according to (NZS 3101.1&2:2006).

Based on these assumptions, the shear resistance of SFRC hollowcore slabs is given by the equation:

$$V_n = V_c + V_{fd} \tag{1}$$

With:

V_c	the shear contribution of the member without shear reinforcement (NZS 3101.1&2:2006 - Sec. 19.3.11)
V_{fd}	the shear contribution of the steel fibre shear reinforcement (NZS 3101.1&2:2006 - Sec. C5.A4.2.1)
V_n	the design shear resistance.

TEST SETUP AND INSTRUMENTATION

Figure 1 shows the test setup used in the experimental program for the PHCS shear tests. Each specimen was subjected to a concentrated load applied at a distance equal to the shear span "a" from the centre of the pinned support (Table 1). An actuator with a capacity of 1000 kN applied the load over a spreader beam with four stiffeners welded every 180mm. A hinge assembly was used in the connection actuator- spreader beam to accommodate the specimen rotation during the test. To distribute more evenly the load applied by the spreader beam through the slab surface, a commercial plaster mixture was used. The pinned and roller supports were detailed to assure the correct behaviour. Commercial bearing pads were located on the top of the bearing steel plates emulating the support conditions used in actual construction. The test setup layout for each specimen series is shown in Figure 2 and Table 1.

The specimens were instrumented with thirteen linear potentiometers and four inclinometers to measure displacements and rotations, respectively. The location of potentiometers and inclinometers is shown in Figure 3. In addition, a rotatory potentiometer was used to control the displacement applied by the actuator. The deflection was monitored through nine potentiometers (channel 2-10). The selected stations were at mid-span (Station 1), point load position (Station 2), and the middle of the shear span (Station 3). Inclinometers were located at both ends and in both lateral faces (R1-4). In order to measure tendons slip, four potentiometers located at the front face of the specimen (channel 11-14), two measuring tendon displacements (channel 11-12); and two measuring slab horizontal displacements (channel 13-14).

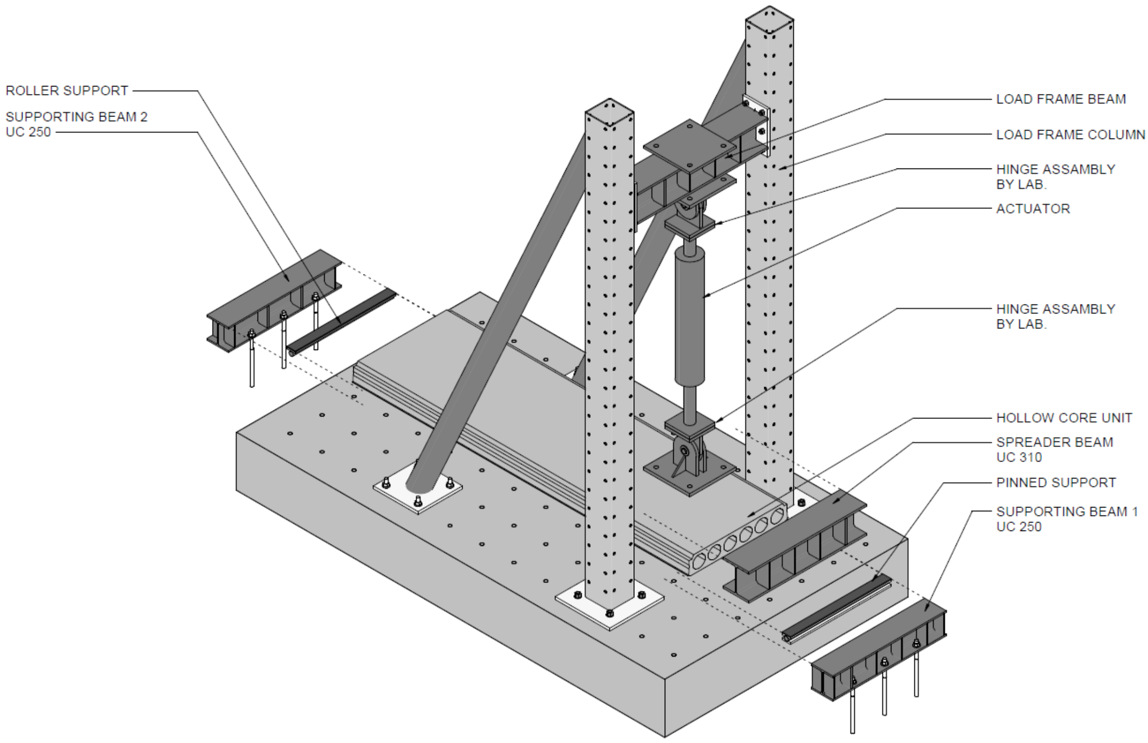

Figure 1 3D view of the test setup.

Table 1 Test setup dimensions.

h (mm)	Series	L (mm)	L_s (mm)	a (mm)	L_1 (mm)	L_2 (mm)
200	NC, FC1 & FC2	4020	3870	500	75	75
300	NC, FC1 & FC2	4500	3870	500	75	75
	Overhang	4500	2350	500	600	1050
Note: h = specimen height; L = specimen length; a = shear span; L_s = distance between supports; L_1 = Overhang at pinned support ; L_2 = Overhang at roller support.						

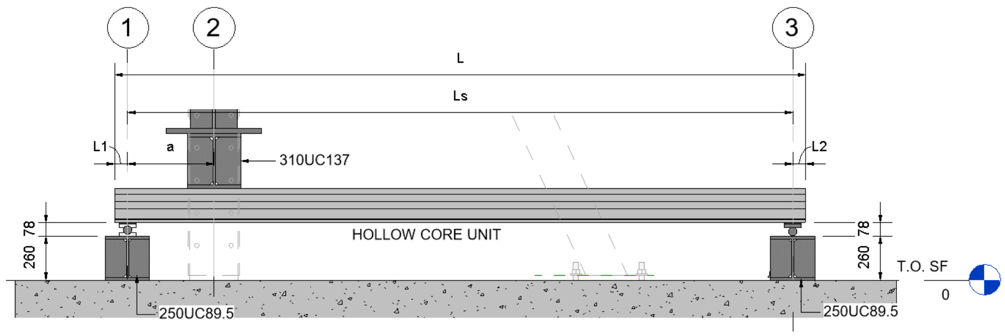

Figure 2 Elevation of the test setup.

Particle tracking velocimetry (PTV) technique was used to complete crack width measurements of PHCS. PTV is a technique that identifies individual particles frame to frame

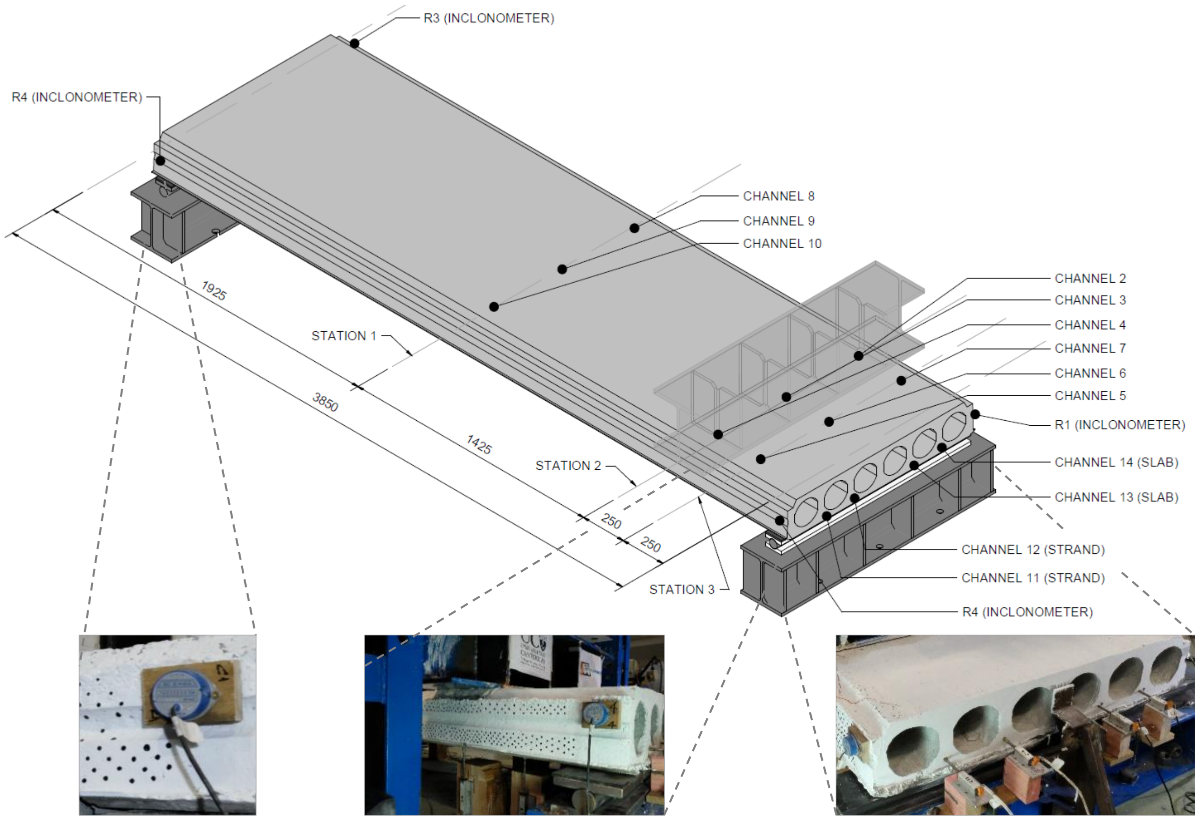

Figure 3 Specimen instrumentation

to establish a particle velocity estimate. The technique tracks the displacement of each particle during the test, simplifying the generation of crack width measurements and strain fields. The PTV analysis for this experimental programme was completed using *Streams 3.02*, a PTV software developed at the University of Canterbury (Nokes, 2019). Results from PTV analysis were contrasted with data collected from linear potentiometers.

TEST SPECIMENS

Two nominal heights, 200mm and 300mm, were investigated in this study. The nominal geometry and prestressing steel data of the specimens is shown in Figure 4 and Table 2, respectively. Specimens were identified according to their height, type of concrete, loading type and the test number, as shown in Table 3.

Table 2 Prestressing steel data

Specimen height (mm)	**N_s**	**φ_{ps} (mm)**	**A_{ps} (mm^2)**	**f_{pu} (MPa)**	**f_{pe} (MPa)**	**E_p (GPa)**
200	5	9.3	275	1860	1300	197
300	5	12.7	495	1860	1300	197

Note: h = specimen height; Ns= number of strands; ϕ_{ps} = nominal diameter; A_{ps} = Area of prestressing steel; f_{pu} = Ultimate tensile strength; f_{pe} = Effective tensile strength; E_p = Modulus of elasticity of prestressing steel.

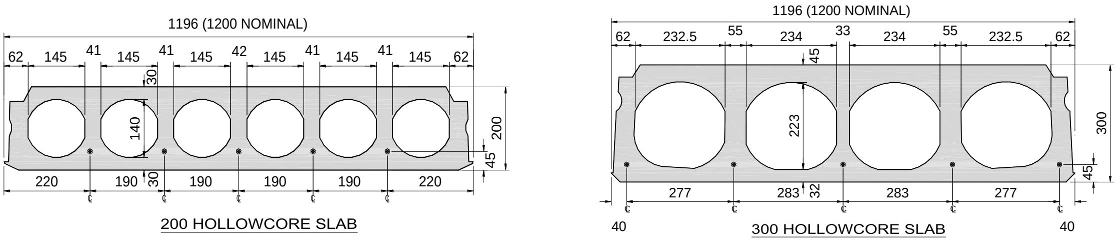

Figure 4. Hollowcore slabs nominal geometry (Stahlton Engineered Concrete)

Table 3 Specimen identification

Series	Specimen ID	h (mm)	D (mm)	a (mm)	a/d	f'$_c$ (MPa)	FD (kg/m^3)	Loading Type
Normal Concrete (NC)	200-NC-M-1	200	155	500	3.23	45	0	Monotonic
	200-NC-M-2	200	155	500	3.23	45	0	Monotonic
	200-NC-C-3	200	155	500	3.23	45	0	Cyclic
	200-NC-C-4	200	155	500	3.23	45	0	Cyclic
Fibre Type 1 (FC1)	200-FC1-M-1	200	155	500	3.23	45	13.33	Monotonic
	200-FC1-M-2	200	155	500	3.23	45	13.33	Monotonic
	200-FC1-C-3	200	155	500	3.23	45	13.33	Cyclic
	200-FC1-C-4	200	155	500	3.23	45	13.33	Cyclic
Fibre Type 2 (FC2)	200-FC2-C-1	200	155	500	3.23	45	26.67	Cyclic
	200-FC2-C-2	200	155	500	3.23	45	26.67	Cyclic
	200-FC2-M-3	200	155	500	3.23	45	26.67	Monotonic
	200-FC2-M-4	200	155	500	3.23	45	26.67	Monotonic
Normal Concrete (NC)	300-NC-M-1	300	255	500	1.96	45	0	Monotonic
	300-NC-C-2	300	255	500	1.96	45	0	Cyclic
	300-NC-M-3	300	255	500	1.96	45	0	Monotonic
	300-NC-C-4	300	255	500	1.96	45	0	Cyclic
Fibre Type 1 (FC1)	300-FC1-M-1	300	255	500	1.96	45	13.33	Monotonic
	300-FC1-M-2	300	255	500	1.96	45	13.33	Monotonic
	300-FC1-C-3	300	255	500	1.96	45	13.33	Cyclic
	300-FC1-C-4	300	255	500	1.96	45	13.33	Cyclic
Fibre Type 2 (FC1)	300-FC2-M-1	300	255	500	1.96	45	26.67	Monotonic
	300-FC2-C-2	300	255	500	1.96	45	26.67	Cyclic
	300-FC2-M-3	300	255	500	1.96	45	26.67	Monotonic
	300-FC2-C-4	300	255	500	1.96	45	26.67	Cyclic
Overhang (O)	300-FC2-C-O-1	300	255	500	1.96	45	26.67	Cyclic
	300-NC-C-O-2	300	255	500	1.96	45	26.67	Cyclic
Note: NC = normal concrete; FC1 and FC2, for steel fibre reinforced concrete type 1 and type 2 ; M = Monotonic test; C = Cyclic test; O = Overhang, h = specimen height; d = effective depth; a = shear span; f'$_c$ = concrete compressive strength; FD = Steel fibre Dosage.								

TEST RESULTS

Twelve full-scale shear tests were performed on 200 mm thick slabs and fourteen on 300 mm thick slabs. Specimens 200-FC1-C-4 and 200-FC2-M-4 were omitted from the data analysis owing to the flexural failure mode observed. The specimen 300-NC-M-1 was discarded due to premature failure. To evaluate the post-peak shear behaviour quantitatively, toughness indices at deflections of 10 mm (2% drift) and 20 mm (4% drift) were determined. Toughness index was calculated as the ratio of the area under the curve shear force vs displacement at a specific point divided by the area under the graph at the point of the peak load. Additionally, normalised shear ratios (V_{Test}/V_{NZS}), considering steel fibre contribution, were calculated at deflections of 10 mm (2% drift) and 20mm (4% drift).

Shear behaviour of 200 mm PHCS

Figure 5 shows the shear force vs deflection curves for 200 mm thick slabs. Three 200-NC PHCS specimens failed in web-shear mode, reaching a maximum shear force of 153.5±12.45 kN. These specimens exhibited a poor post-peak performance, more than 30% of their shear resistance dropped shortly after reach the peak force in a brittle behaviour. One 200-NC specimen exhibited a flexural-shear failure mode, as expected, a more ductile post-peak behaviour was observed, at 20 mm of deflection the specimen was able to resist about 100% of the design shear strength calculated according to NZS 3101.1&2:2006.

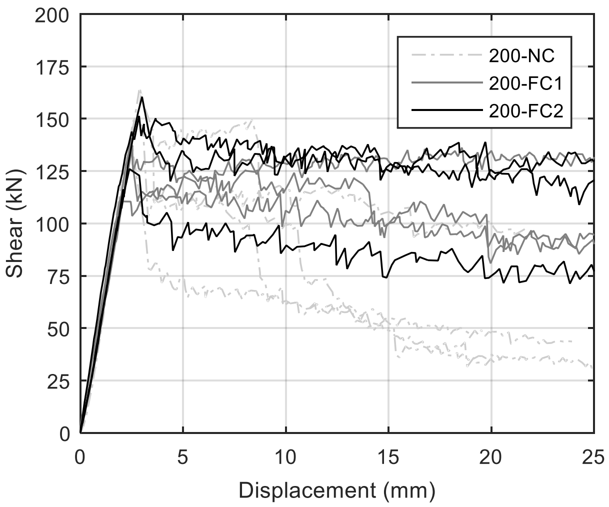

Figure 5 Shear force vs displacement at point load for 200 mm thick hollowcore slabs NC, FC1 and FC2 series. Note: a/d = 3.23.

Steel fibre reinforced concrete hollowcore specimens (FC1 and FC-2) exhibited a ductile post-peak behaviour, even specimens with a partial web-shear failure mode exhibited a moderate and gradual loss of strength. The extra shear resistance due to steel fibre contribution, predicted by code equations, was not detected at peak values. On the contrary, the average peak shear resistance decreased by about 9.9% and 5.1% for FC1 and FC2 series, compare to NC series (Figure 6). Nonetheless, average extra shear strengths of 38% and 37% (at 10 mm); and 92% and 97% (at 20 mm) were measured for FC1 and FC2 series, respectively. Toughness indices were improved, by 66% and 9% (at 10 mm), and by 107% and 32% (at 20 mm), for FC1 and FC2 series, respectively (Figure 6).

Shear behaviour of 300 mm PHCS

The shear force versus displacement behaviour for each 300 mm thick hollowcore slabs can be seen in Figure 7. Web-shear failure modes were observed in all tested specimens. The average peak shear forces were 235±9 kN, 235±12 kN and 225±11 kN, for NC, FC1 and FC2 series, respectively.

Normal concrete specimens showed poor post-peak performance, with a loss of resistance of about 57% immediately after reached the maximum shear force. A factor that influenced this behaviour was the development of a longitudinal crack in the web of all NC specimens. On the contrary, FC1 and FC2 hollowcore specimens exhibited a moderate loss of strength. At 10mm of deflection FC1 and FC2 series exhibited on average 53 kN (98%) and 49 kN (91%), higher shear strength than NC series. The average shear resistance of FC1 and FC2 specimens, at 20 mm of deflection, were up to 35 kN (83%) and 25 kN (60%) higher than NC specimens, respectively.

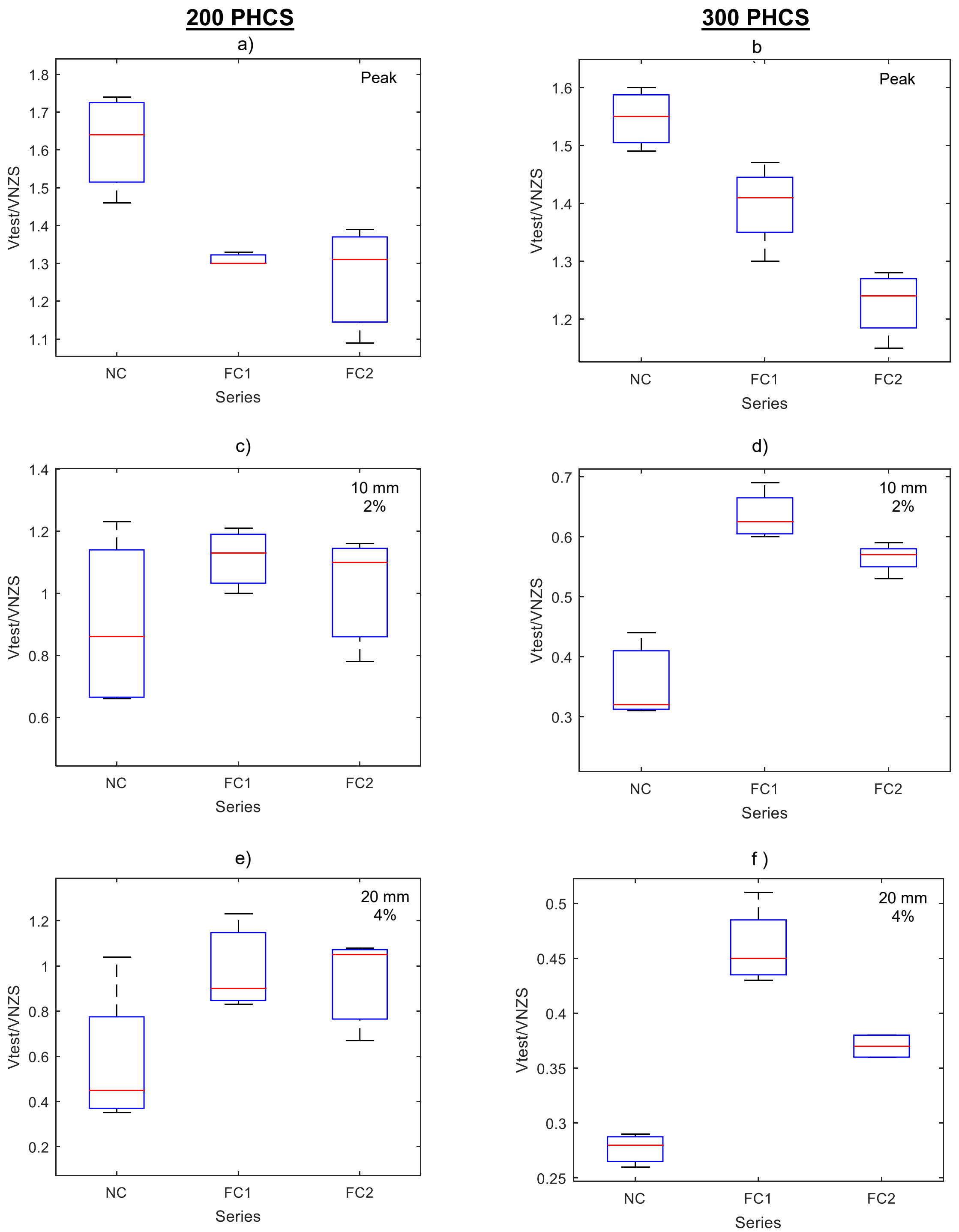

Figure 6 Box-plot for normalised shear force in 200 mm and 300 mm thick PHCS. a) & b) at peak shear; c) & d) at 10 mm (2%) of deflection; e) & f) at 20 mm of deflection .

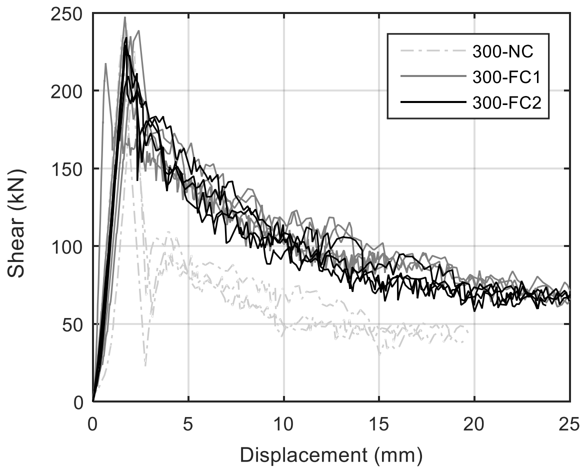

Figure 7 Shear force vs displacement at point load for 300 mm thick hollowcore slabs NC, FC1 and FC2 series, b) Overhang series. Note: a/d = 1.96.

The 300 overhang series consisted of two PHCS with fully developed transfer length ($50d_b$) at the beginning of the shear span (Figure 8). The shear behaviour of the specimens can be seen in Figure 9. Overhang series specimens, 300-NC-C-O and 300-FC2-C-O, exhibited a higher peak shear resistance than the average of the series 300 NC, FC1 and FC2. For 300-NC-C-O (normal concrete), a peak shear of 396 kN was registered. This is 69% higher than the average shear of 300 mm NC series. Specimen 300-FC2-C-O reach a peak shear of 491 kN, compare to the average of 300 FC2 series, this is an increment of 118%.

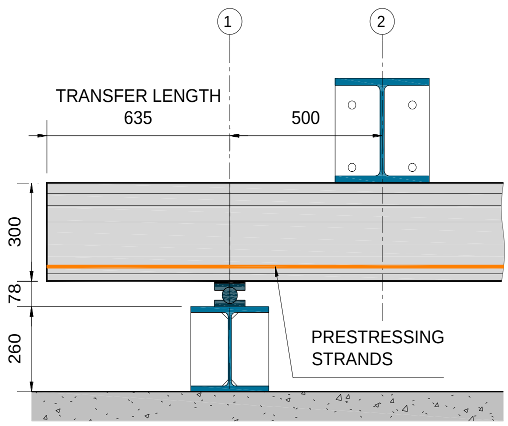

Figure 8 Test setup for 300 overhang series. Dimensions in mm

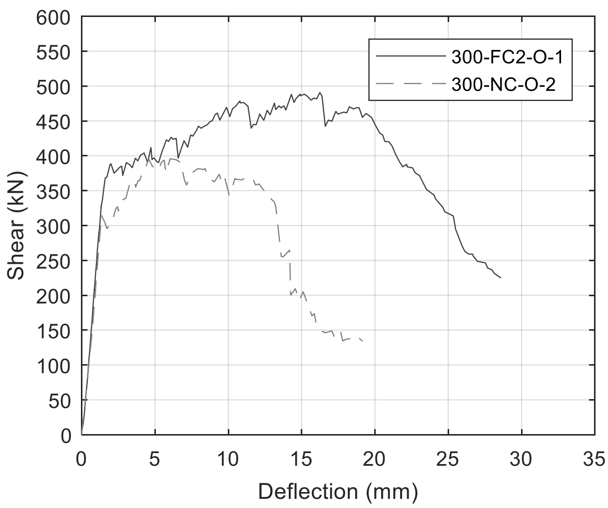

Figure 9. Shear force vs displacement at point load for 300 mm thick hollowcore slabs Overhang series.

<u>Crack width measurements with PTV</u>

Figure 10 and Figure 11 show the results from the crack width measurements for specimens 200-FC1-C-3 and 300-FC2-C-2, respectively. PTV analysis was verified through the vertical displacement of each specimen using data collected from potentiometers at stations 3. For specimen 200-FC1-C-3, errors of 3.73% and 2.21% were measured at peak load and 4% drift, respectively. Vertical displacement errors of 2.19% and 3.03%, in specimen 300-FC2-C-2, were determined at peak load and 3% drift, respectively. A possible source of these errors is the manual process in the particle coordinates transformation, completed to make the angle of the camera perpendicular specimen surface. Nonetheless, errors in vertical displacement were negligible; therefore, PTV analysis was considered satisfactory.

Figure 10a and Figure 11a show the crack width against drift behaviour for the 200-FC1-C-3 and 300-FC2-C-2 specimens. Both specimens presented first cracks are about 0.35% drift. The peak load was reached at 0.52% and 0.59% drift, 200mm and 300mm SFRC-PHCS, respectively. A linear behaviour was observed immediately after the first crack opened. At 2% drift (10mm), specimen 200-FC1-C-3 exhibited a crack width of 4.5 mm and shear resistance of 119kN (V_{max}/V_{NZS} = 1.13). At the same drift, crack width of 9.13 mm and shear strength 107kN (V_{max}/V_{NZS} = 0.59) were recorded for specimen 300-FC2-C-2.

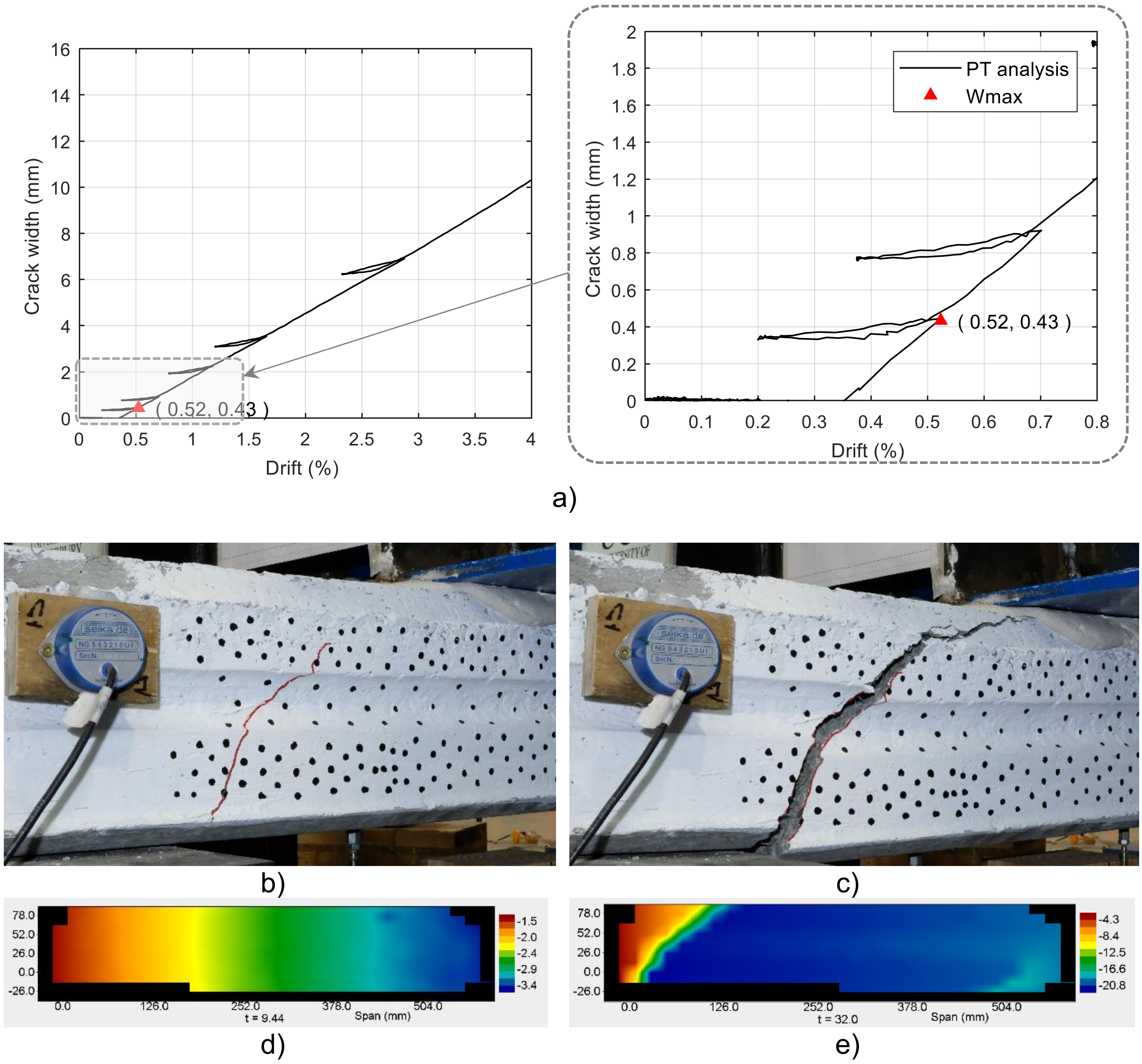

Figure 10. Particle tracking analysis for specimen 200-FC1-C-3. Crack width vs drift, b) at maximum shear force, b) Crack pattern at peak load, c) Crack pattern at 4% drift. d) Vertical displacement field at peak load e) Vertical displacement fields at 3 drift.

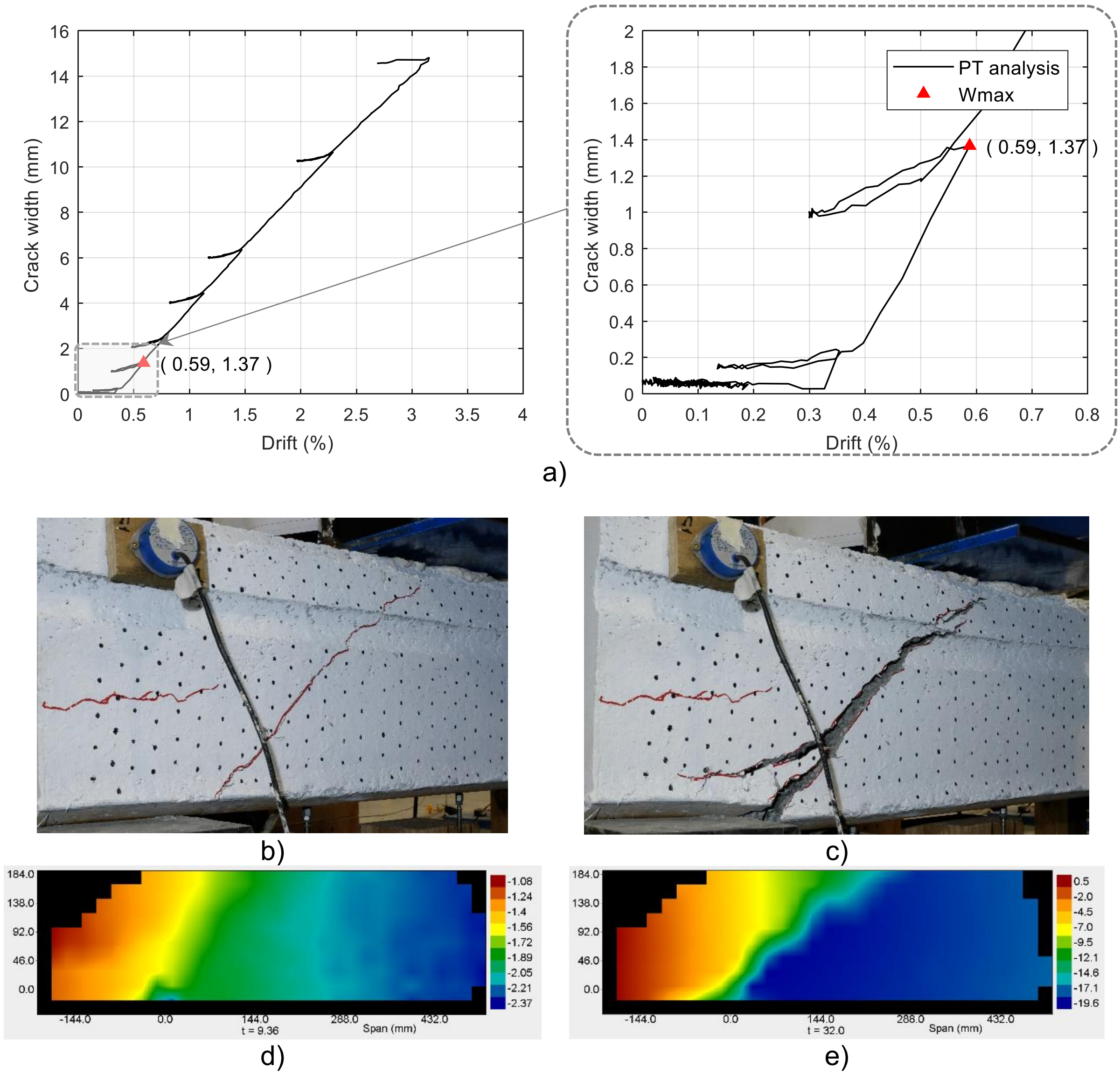

Figure 11. Particle tracking analysis for specimen 300-FC2-C-2. Crack width vs drift, b) at maximum shear force, b) Crack pattern at peak load, c) Crack pattern at 3% drift. d) Vertical displacement field at peak load e) Vertical displacement fields at 3 drift.

CONCLUSIONS

Steel fibre reinforced hollowcore slabs exhibited a ductile shear behaviour compared to conventional concrete hollowcore slabs. A gradual loss of resistance with higher residual shear strength, an essential aspect for life safety, was observed in SFRC-PHCS. For 200 mm thick specimens, residual shear improvements were 38% and 95%, at 10 mm (2% drift) and 20 mm (4% drift) of deflection, respectively, for both FC1 and FC2 series. For 300 mm thick hollowcore slabs, increases of 98% and 91% at 10mm; and 83% and 60 % at 20 mm, were measured for FC1 and FC2 series, respectively. Besides, higher toughness indices were obtained for all SFRC compared to NC specimens, which indicate a higher energy absorption capacity.

PHCS specimens presented a higher peak shear resistance than design shear strength calculated according to NZS 3101(NZS 3101.1&2:2006)(NZS 3101.1&2:2006). Series 200 showed V_{max}/V_{NZS} ratios of 1.62±0.13, 1.31±0.02, and 1.26±0.15; for NC, FC1 and FC2 series, respectively. For series 300, the ratios V_{max}/V_{NZS} were 1.54±0.06, 1.4±.0.07 and 1.23±0.06, for NC, FC1 and FC2 series, respectively. The normalised shear ratios at 20 mm (V_{20}/V_{NZS}), for series 200, were 0.57±0.32, 0.99±0.21 and 0.93±0.23; NC, FC1 and FC2, respectively. For series 300, V_{20}/V_{NZS} ratios of 0.28±0.02, 0.46±0.04, 0.37±0.01 were determined; for NC, FC1 and FC2, respectively.

The peak shear resistance of the 200 mm and 300 mm thick SFRC-PHCS was not increased, compared to NC-PHCS, as predicted by NZS 3101 equations. Possible explanations could be problems related to concrete compaction, which may have a detrimental effect on the shear resistance or increase the transfer length of the tendon, to some extent offsetting the beneficial role played by steel fibres.

Hollowcore specimens tested with an overhang length of 600 mm exhibited an enhanced performance compared to simple beam PHCS. Specimen 300-NC-C-O-2 showed an extra peak shear 69% higher than the average of 300 NC series. Specimen 300-FC2-C-1 reach a peak shear 118% higher than the average of 300 FC2 series. This behaviour was attributed to the improved transfer length, which increases the shear capacity due to the higher compressive force applied in the member.

Particle tracking technique showed accurate results when compared to data collected from linear potentiometers. High quality displacement and strain fields can be generated using this technique. Additionally, PTV offers advantages to capture discontinuous processes, e.g. crack growth, due to the tracking of individual particles.

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