BENEFITS OF TOP STRAND AND STEEL FIBRES IN THE DESIGN AND MANUFACTURE OF HOLLOWCORE PRECAST FLOOR SLABS

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ABSTRACT

Pre-stressed hollow-core floor slabs are efficient, light weight flooring elements that can cover long span lengths. Even though these elements have adequate bending capacity, site observations showed that without careful consideration to initial concrete strength and high levels of eccentric pre-stress during design, these units may be susceptible to web shear failures.

The automated process by which hollow-core floor slabs are produced make it impossible to cast in stirrup reinforcement to resist web shear as well as shear demand induced by higher gravity loads. The brittle nature of failure has been a concern for many researchers, engineers and manufacturers. Manufacturers conduct regular shear testing in accordance with the PCI Manual to verify the shear capacity of bare hollow-core floor units.

Stahlton Engineered Concrete engaged the University of Canterbury Quake Centre to conduct shear testing on hollow-core slabs that had various doses of steel fibres added to the concrete mix. The results showed reasonable increase in shear capacity as well as enhanced residual load carrying capacity after failure. The detailed findings will be described in the paper and presented to show how Hollow-core units designed with controlled stress levels and reinforced with steel fibres will provide robustness to resisting web shear and high gravity loads.

INTRODUCTION



Prestressed hollowcore slabs are the most common form of precast concrete flooring worldwide with around 50% of the total flooring market in Europe (Paine & Andrews 1998). Hollowcore flooring has advantages over other flooring systems such as:

- Thinnest floor for longer spans usually un-propped during construction.
- Efficient and economical to manufacture.
- Production capacity exists for larger quantities to be made quicker.
- Efficient at spreading concentrated loads.
- Spaced hollowcore provides a lighter floor solution.

The collapse of hollowcore flooring during the Northridge Earthquake (Norton et al 1994) as well as Jeff Matthews experiment at University of Canterbury's Department of Civil Engineering (Matthews et al 2004) prompted concerns for engineers on the performance of hollowcore prestressed flooring units in seismic regions. The aftermath of both these failures indicated signs of pre-cracked web shear initiated by inadequate prestress release concrete strength and/or inadequate attention to stresses at the ends of the units during the design phase.

Figure 1
a) Hollowcore damage Northridge Earthquake b) Failure during Matthews experiment 2004.

Solutions are to manufacture highly prestressed hollowcore units with stressed top strands and to cast the hollowcore units with steel fibres to provide reinforcement across the potential web shear cracks. Stirrups are not an option to be cast in the webs of hollow-core flooring due to the extruded or slip forming machine process. Steel fibres were found to have the added benefit of increasing shear capacity of bare units by 20% with minimal dosages.

Research in the shear performance of hollowcore flooring units is not new. Testing was carried out by University of Nottingham in the UK by Paine & Andrews et al (1998) and their results gave similar shear performance enhancement.

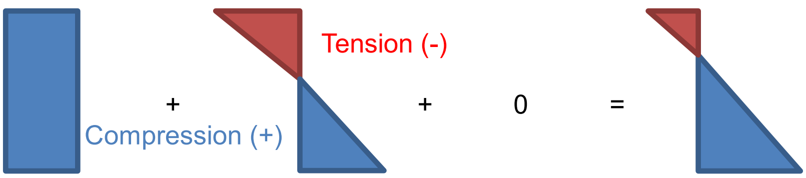
It is important manufacturers include quality assurance inspections for web shear as part of their processes. One common complaint of hollowcore amongst contractors is the large hogging cambers of the bare units. Top strand also provides manufactures with improved camber control.

DESIGN CHECKS TO INCLUDE TOP STRAND AND AVOID WEB SHEAR

End stresses are calculated using internal stress blocks incorporating the total initial prestress at release, P_i, area of the section, A, eccentricity of the prestress relative to the neutral axis of the section, e, and the section modulus about the top of the section, Z_t. The tensile stress limit for prestressed concrete in accordance with NZS3101:Part1:2006 cl.19.3.3 is 0.5 times the square root of the concrete strength at the time of pre-stress release, f_{ci}. For compression the limit is 0.6 times the release strength. The minimum release strength manufactures tend to adopt is 28MPa so the limits are 2.64MPa in tension (-) and 16.8MPa in compression (+).

Top of section

Bottom of section



$$\frac{P_i}{A} + \frac{P_i * e}{Z} + \frac{M}{Z} = 0 \text{ kNm AT ENDS}$$

Example: For a 300mm deep Hollowcore unit with 10x12.9mm diameter strands stressed to 67% of 184kN centred 45mm up from the soffit of the unit:

P_i = 1196kN A = 0.205m^2 e = 0.102m Z_t = 0.0141m^3 Z_b = 0.0147 m^3

Transfer stresses at ends are:

$\frac{P_i}{A} = +5.84\text{MPa} + \frac{P_i{*}e}{Z_t} = -8.65\text{MPa}$ Top stress = - 2.81MPa < - 2.64MPa NG!
need top strand

$\frac{P_i}{A} = +5.84\text{MPa} + \frac{P_i{*}e}{Z_b} = +8.30\text{MPa}$ Bottom stress = + 14.14MPa < 16.8MPa OK!

A maximum of 7x12.9dia strand in 150mm and 200mm deep hollowcore does not exceed the stress limits. Depending on the number of strand and their prestress level the issue arises in 300mm and 400mm deep hollowcore units. From the example we see 10x12.9 strand and more need top strand in 300mm hollowcore and it is the same for 400mm deep. Internal shear testing by Stahlton suggest there is a vertical shear capacity reduction of approximately 20% for bare units with web shear.

Figure 1 c) Web shear observed in this 400mm deep hollowcore unit.

BACKGROUND THEORY ON SHEAR CAPACITY CALCULATIONS WITH STEEL FIBRES

The total shear capacity, V_T, of a pre-stressed hollow core unit with steel fibres can be calculated by the shear contribution from concrete (V_c), shear reinforcement (V_s) and steel fibres (V_f). In the reported specimens, there were no shear reinforcement so its contribution is taken as zero (V_S=0). Accordingly, the shear capacity of a pre-stressed hollow core unit can be calculated as given below (NZS3101 2006):

$$V_T = V_c + V_f \tag{1}$$

Concrete shear contribution can be calculated using equations 2 and 3:

$$V_c = \left(1 + \frac{K \cdot N_{pt}}{b_w h \cdot f'_c}\right) v_b \cdot b_w d \tag{2}$$

$$v_b = (0.07 + 10\frac{\Sigma A_{pt}}{b_w d}) \cdot \sqrt{f'_c}\,,\ 0.08\sqrt{f'_c} \le v_b \le 0.2\sqrt{f'_c} \tag{3}$$

where, K = 3 for N > 0 (compression), 12 for N < 0 (tension)
N_{pt} = Axial force imposed by pre-stressing -in N (allow for losses)
b_w = The total width of the web
h = The section height
d = The effective depth of the strand
A_{pt} = Area of the strand (accounting for dowel action)
f'_c = Concrete compressive strength

Shear contribution given by the steel fibres can be calculated as outlined in Part 2 appendix C5A of NZS 3101 (NZS3101 2006):

$$V_f = 0.7 k_f k_1 \tau_{fd} b_w d \text{ (in } mm \text{ and } MPa\text{)} \tag{4}$$

$$k_f = 1 + n\left(\frac{h_f}{b_w}\right)\cdot\left(\frac{h_f}{d}\right) \le 1.5 \text{ (in } mm\text{)} \tag{5}$$

$$n = \frac{b_f - b_w}{h_f},\; n \le 3 \text{ and } n \le \frac{3b_w}{h_f} \text{ (in } mm\text{)} \tag{6}$$

where, b_f = Width of the flange
h_f = Thickness of the flange

$$k_1 = 1 + \sqrt{\frac{200}{d}} \le 2 \text{ (in } mm\text{)} \tag{7}$$

$$\tau_{fd} = 0.12 f_{Rk,4} \text{ (Design value)} \tag{8}$$

where, $f_{Rk,4}$ = The characteristic residual tensile strength of the steel fibre reinforced concrete at crack mouth opening level 4, i.e. at $CMOD_4$ =3.5 *mm*. (For ultimate shear strength analysis, $f_{Rk,1}$ was used in this report since it gives the highest contribution)

The mean values of $f_{Rk,i}$ for each level of $CMOD_i$ are given by the manufacturer specifications (Appendix A) and are summarized for the used steel fibre type is given in Table 1 (Values are for 15 kg/m^3).

Table 1 Manufacturer's specification for $f_{Rk,i}$ values (Dramix 3D 80/60BG)

$CMOD_i$	$f_{Rk,i}$ (*MPa*)
$CMOD_1$ = 0.5 mm	2.4
$CMOD_2$ = 1.5 mm	2.3
$CMOD_3$ = 2.5 mm	2.1
$CMOD_4$ = 3.5 mm	2.0

CMOD: Crack mouth opening displacement
The values given above are for Dramix 3D 80/60BG type steel fibres with 15kg/m³ fibre dosage

TEST SETUP

In the tests, the one point loading test setup at Stahlton was used. The loading distance from the nearest simple support was arranged such that the specimen could fail in shear. The load was applied monotonically using a powered hydraulic pump. The loads were measured using a 250kN capacity load cell placed between the loading jack and spreader beam. The readings were taken by using a digital load display showing the *kg* values of the loading. The detailed schematics and photo of the test setup is shown in Figure and Figure 1c respectively.

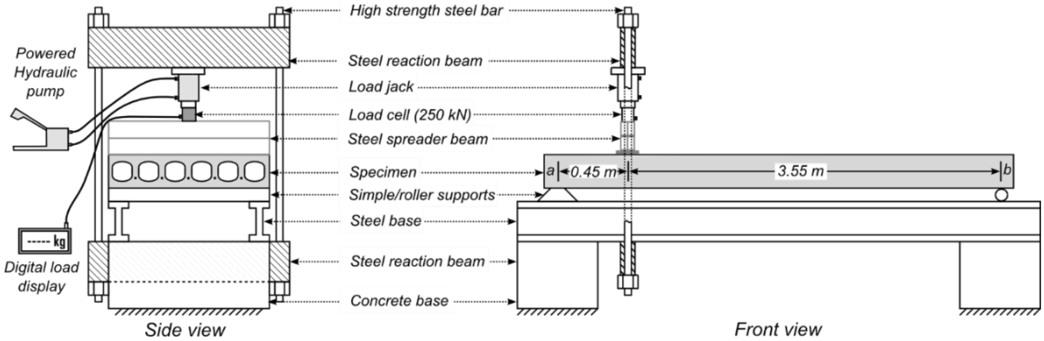


Figure 2 The test setup

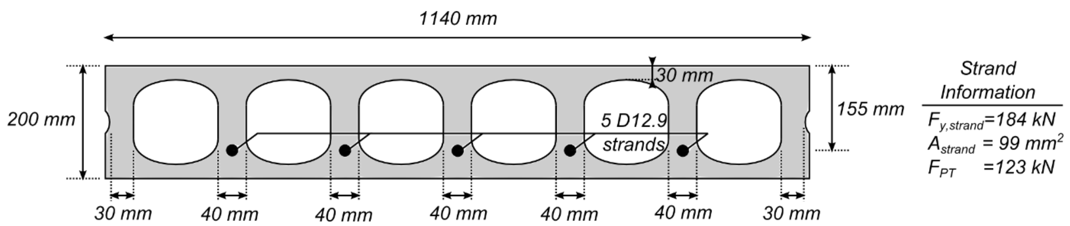
Limitations of the Test Setup

The test setup was good enough to observe the load carrying capacity, but it limited the possible observations and data that could have been made in a laboratory condition:

- The reaction frame formed by top and bottom steel beams connected by high strength steel bars were not rigid and confined enough to prevent out of plane deformations. This might have affected the expected experimental response of the specimen to a degree.
- The load was applied by using a powered hydraulic pump and the loading rate could not be controlled slowly enough to observe displacements. Because of this, no displacement measurements could be taken in the setup.
- All readings, measurements were taken manually. No real time data logging was used in this on-site test setup. As a result of this, the measurements are not as accurate as a test carried out in laboratory conditions.
- There were no concrete cylinder specimens present in the testing grounds. Therefore, the concrete strength measurements were taken using a digital Schmidt hammer. 10 measurements per specimen were taken and averaged. Resulting measurements are only rough approximations for concrete strength, which will be given for each test specimen.

TEST SPECIMENS

Each specimen was pre-stressed with 5 high strength strands of 100 mm^2 nominal area. The yield force of each strand is given as 184 *kN* in the Mill Test Certificate provided by the manufacturer (Appendix B). The specimens were pre-stressed at 67% of their yielding force (123 kN). Potential pre-stressing losses were in the order of 15% for these specimens. The cross sectional detail for each specimen is as shown in Figure 1a with the only difference being the amount of steel fibres put into each specimen (Steel fibre type: Dramix 3D 80/60BG).



a)

b) c)

Figure 1 a) Cross section for the test specimens and pre-stress information; b) Digital Schmidt hammer; c) Test specimen placed within the setup

In total, there were only six test specimens. 2 of these specimens were as built control specimens without any steel fibres (Specimens A1, A2). On the other hand specimens S1, S2 had 13.33 kg/m^3 steel fibres while specimens S3, S4 contained 26.67 kg/m^3 steel fibres. Both ends of each specimen (end a, end b) were tested till shear failure, summing up to 12 tests in total. Due to the lack of concrete cylinder samples, the concrete compressive strength, f'_c, was measured using a digital Schmidt hammer (Figure 1b) by averaging 10 hammer readings at each end of the specimens. The properties of the test specimens are summarized in Table 2.

Table 2 Test specimens

	Specimen Number	*Specimen Detail*	*Concrete Strength f'_c (MPa)*	*Steel Fibre Density (kg/m³)*
As built specimens Steel fibre dosage 0	A1a	As built specimen 1 end a	70.5	0
	A1b	As built specimen 1 end b	62.0	0
	A2a	As built specimen 2 end a	63.0	0
	A2b	As built specimen 2 end b	63.5	0
Steel fibre dosage 13.33	S1a	Steel fibre specimen 1 end a	64.5	13.33
	S1b	Steel fibre specimen 1 end b	58.5	13.33
	S2a	Steel fibre specimen 2 end a	70.0	13.33
	S2b	Steel fibre specimen 2 end b	60.0	13.33
Steel fibre dosage 26.67	S3a	Steel fibre specimen 3 end a	65.0	26.67
	S3b	Steel fibre specimen 3 end b	66.0	26.67
	S4a	Steel fibre specimen 4 end a	57.0	26.67
	S4b	Steel fibre specimen 4 end b	58.5	26.67

TEST RESULTS

The specimens were monotonically loaded until shear crack formation occurred, which approximately corresponds to *CMOD*1 (~0.5 mm deflection). After this state, the specimens were pushed further until a level crack widening occurred, which approximately corresponded to *CMOD*4 (~3.5 mm deflection). In all of the tests, brittle shear failure was observed. However, the specimens with steel fibres had slightly higher capacity both at *CMOD1* (Ultimate capacity point) and *CMOD4* (Residual capacity point). The results reported here are analyzed further in the analysis of results section. While doing the tests, manual load readings were taken using the digital display connected to the load cell, which gave the load values in terms of mass (i.e. *kg*). For practicality, the observed force values were written on the test specimens using g=10 m/s^2 during the tests. However, as it is given in the following text, these values have been recalculated using g=9.81 m/s^2 for a better accuracy in analyses of results.

Test Results of As Built Specimens: A1a, A1b, A2a and A2b

The summary of the force readings taken during the tests is given in Table 3 and Figure 2. Damage photos for the as built specimens, without any steel fibres, are shown from Figure 3 to Figure 6.

Table 3 Summary of test observations (Forces have been recalculated using g=9.81 m/s^2)

Specimen	*f'_c (MPa)*	*F_{CMOD1} (kN)*	*V_{CMOD1} (kN)*	*F_{CMOD4} (kN)*	*V_{CMOD4} (kN)*
A1a	70.5	121.64	107.96	49.05	43.53
A1b	62.0	101.04	89.68	49.05	43.53
A2a	63.0	105.95	94.03	49.05	43.53
A2b	63.5	101.04	89.68	49.05	43.53

CMOD 1 corresponds to the shear failure value
CMOD 4 corresponds to the residual capacity after shear crack widens
F: applied load value
V: shear force resulting from the applied load (V=0.8875F)

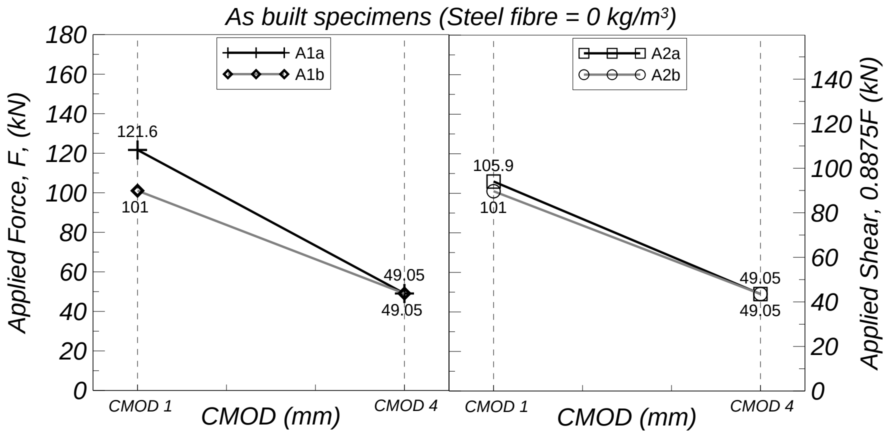


Figure 2 Applied force vs. corresponding *CMOD* values for as built specimens A1a, A1b, A2a, A2b

Figure 3 Test specimen A1a

Figure 4 Test specimen A1b

Figure 5 Test specimen A2a

Figure 6 Test specimen A2b

Test Results of Specimens with 13.33 kg/m³ steel fibre density: S1a, S1b, S2a and S2b

The summary of the force readings taken during the tests is given in Table 4 and Figure 7. Damage photos for these specimens with steel fibres, are shown from Figure 8 to Figure 11.

Table 4 Summary of test observations (Forces have been recalculated using g=9.81 m/s^2)

Specimen	*f'$_c$ (MPa)*	*F_{CMOD1} (kN)*	*V_{CMOD1} (kN)*	*F_{CMOD4} (kN)*	*V_{CMOD4} (kN)*
S1a	64.5	127.53	113.18	73.58	65.30
S1b	58.5	129.49	114.92	72.59	64.43
S2a	70.0	127.53	113.18	77.50	68.78
S2b	60.0	127.53	113.18	68.67	60.95

CMOD 1 corresponds to the shear failure value
CMOD 4 corresponds to the residual capacity after shear crack widens
F: applied load value
V: shear force resulting from the applied load (V=0.8875F)

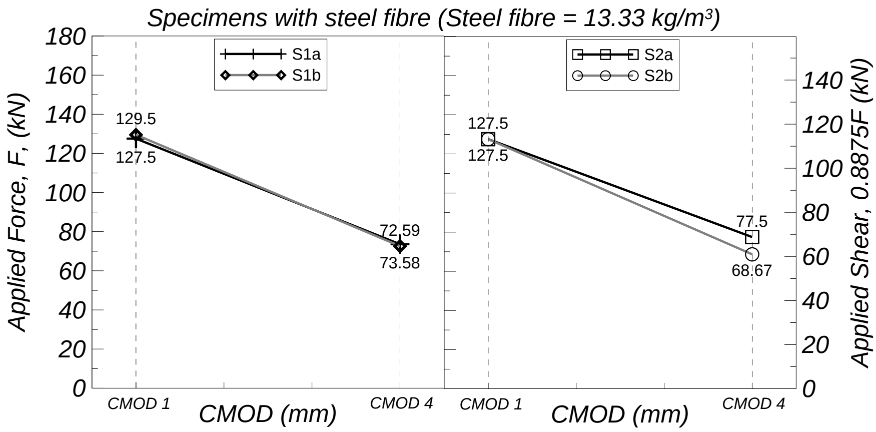


Figure 7 Applied force vs. corresponding *CMOD* values for specimens with steel fibre density 13.33 kg/m³ S1a, S1b, S2a, S2b

Figure 8 Test specimen S1a with visible steel fibres within cracks (13.33 kg/m³ steel fibre content)

Figure 9 Test specimen S1b with visible steel fibres within the cracks (13.33 kg/m^3 steel fibre content)

Figure 10 Test specimen S2a (13.33 kg/m^3 steel fibre content)

Figure 11 Test specimen S2b (13.33 kg/m^3 steel fibre content)

Test Results of Specimens with 26.67 kg/m3 steel fibre density: S3a, S3b, S4a and S4b

The summary of the force readings taken during the tests is given in

Table 5 and Figure 12. Damage photos for these specimens with steel fibres, are shown from Figure 13 to Figure 16. The results of specimen S4b have been neglected since the reaction frame showed significant out-of-plane deflection in this test and the significantly high capacity value observed in this test may not represent the real capacity value. Therefore, the results of S4b are not considered in the analysis of results section.

Table 5 Summary of test observations (Forces have been recalculated using g=9.81 m/s^2)

Specimen	*f'_c (MPa)*	*F_{CMOD1} (kN)*	*V_{CMOD1} (kN)*	*F_{CMOD4} (kN)*	*V_{CMOD4} (kN)*
S3a	65.0	125.57	111.44	88.29	78.36
S3b	66.0	151.07	134.08	98.10	87.06
S4a	57.0	127.53	113.18	94.18	83.58
S4b	58.5	167.75	148.88	107.91	95.77

CMOD 1 corresponds to the shear failure value
CMOD 4 corresponds to the residual capacity after shear crack widens
F: applied load value
V: shear force resulting from the applied load (V=0.8875F)

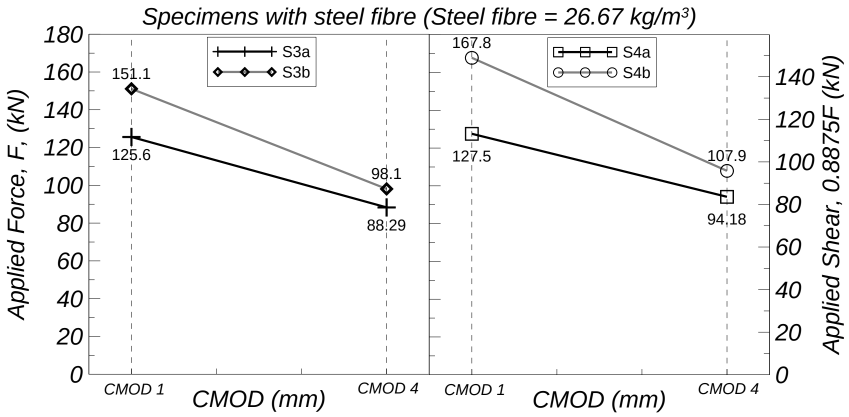


Figure 12 Applied force vs. corresponding CMOD values for specimens with steel fibre density 26.67 kg/m³ S1a, S1b, S2a, S2b

Figure 13 Test specimen S3a (26.67 kg/m³ steel fibre content)

Figure 14 Test specimen S3b (26.67 kg/m³ steel fibre content)

Figure 15 Test specimen S4a (26.67 kg/m³ steel fibre content)

Figure 16 Test specimen S4b (26.67 kg/m^3 steel fibre content)

ANALYSIS OF RESULTS

Since all specimens had slightly different concrete compression strength due to Schmidt hammer measurements, the comparison of the results may not be very accurate. However, it is possible to average the concrete strength in each respective specimen groups and represent each group with a single concrete strength value (i.e. for as built specimen group and steel fibre reinforced specimen group). Since shear strength in concrete elements is directly proportional to the square root of the concrete strength, as given in Equation 3, the observed experimental shear capacity values can be modified accordingly to facilitate a more meaningful comparison. This process is given in Table 6.

Table 6 Test results modified according to the average concrete strength values in each group of test specimens

#	f'_c (MPa)	Observed F_{CMOD1} (kN)	Observed F_{CMOD4} (kN)	$f'_{ave,c}$ (MPa)	Modified $F_{M,CMOD1}$ (kN)	Modified $F_{M,CMOD4}$ (kN)	Modified $V_{M,CMOD1}$ (kN)	Modified $V_{M,CMOD4}$ (kN)
A1a	70.5	121.64	49.05	64.75	116.58	47.00	103.46	41.72
A1b	62.0	101.04	49.05		103.26	50.13	91.64	44.49
A2a	63.0	105.95	49.05		107.41	49.73	95.33	44.13
A2b	63.5	101.04	49.05		102.03	49.53	90.55	43.96
S1a	64.5	127.53	73.58	63.25	126.29	72.86	112.08	64.66
S1b	58.5	129.49	72.59		134.65	75.48	119.50	67.00
S2a	70.0	127.53	77.50		121.23	73.67	107.59	65.38
S2b	60.0	127.53	68.67		130.94	70.51	116.21	62.57
S3a	65.0	125.57	88.29	61.63	122.27	85.97	108.51	76.30
S3b	66.0	151.07	98.10		145.98	94.79	129.56	84.13
S4a	57.0	127.53	94.18		132.60	97.92	117.69	86.91
S4b	58.5	167.75	107.91		172.17	110.75	152.80	98.29

Note: $F_{M,CMODi} = \sqrt{\frac{f'_{ave,c}}{f'_c}} \times F_{CMODi}$, $V_{CMODi} = F_{CMODi} \times 0.8875$ (from equilibrium)

Using these results, the comparison of all the specimens can be made as shown in Figure 17. In this plot, it can be seen that the added doses of steel fibres in concrete causes an increased capacity at both *CMOD* 1 and *CMOD* 4 levels.

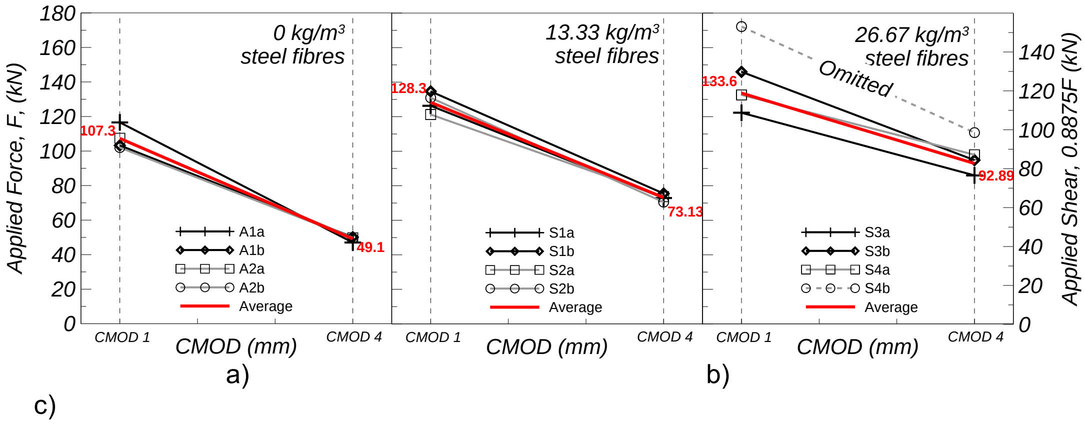


Figure 17 Test results modified using average concrete strength values: a) As built specimens without any steel fibres; b) Specimens with 13.33 kg/m³ steel fibres; c) Specimens with 26.67 kg/m³ steel fibres

On the other hand, the steel fibres seem to be more effectively increasing the residual capacity (i.e. at *CMOD*4 level) rather than the capacity at shear failure (i.e. *CMOD*1 level). This can be clearly seen when percentage of additional capacity over the as built capacity are plotted at *CMOD*1 and *CMOD*4 levels (Figure 18). In this figure, it can be seen that addition of 13.33 kg/m³ to the as built specimen resulted in capacity gain of 19.53% and 48.95% at *CMOD*1 and *CMOD*4 respectively. Similarly, addition of 26.67 kg/m³ of steel fibres caused capacity gain of 24.5% and 89.2% at *CMOD*1 and *CMOD*4 respectively.

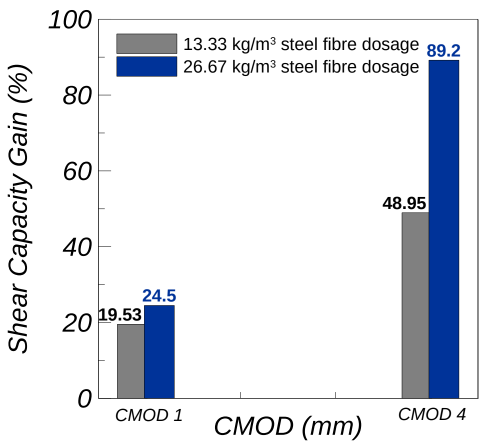


Figure 18 Percentage of shear capacity gain over the as built capacity

Considering the limited number of tests in this particular inspection, it is not possible to conclude a generalized result for the amount of added capacity by different steel fibre dosages. However, it can safely be said that additional steel fibres in concrete is beneficial for increasing the shear capacity of a hollow core section. This also adds much needed residual resistance that can be a lifesaving factor in the case of a shear failure.

Capacity Check According to NZS 3101

Concrete shear contribution (V_c)

Concrete shear contribution can be calculated using equation 2 and 3 as follows (b_w=260 *mm*, d=155 *mm*, A_{pt}=99 *mm*^{2}, N_{pt}=5×123 *kN*):≈

Due to the variances in each test, the average concrete strength values, $f'_{ave,c}$, and the observed shear capacities normalized accordingly, $V_{M,CMOD1}$ (previously given in Table 6), will be used for the comparison with theoretical capacity values. The resulting concrete shear contribution values, V_c, are given in Table 7.

Shear contribution due to steel fibres (V_f):

Ultimate capacity values due to the additional steel fibre content can be calculated using equations 4-8. For these calculations, $f_{Rk,i}$ values given in the manufacturer's steel fibre specification (Appendix A) are required. However, it should be noted that the provided values are only valid for steel fibre dosage of 15 kg/m^3. In the reported test specimens, the steel fibre dosage levels were 13.33 kg/m^3 and 26.67 kg/m^3. Therefore, the numerical capacity calculation given by NZS 3101 (equations 4-8) will result in an average shear capacity rather than an exact estimate. This estimation may give reasonable results for the specimens with 13.33 kg/m^3 steel fibre content whilst the result may show a degree of deviation for the specimens with 26.67 kg/m^3 steel fibres. The shear capacity contribution can be calculated as given below. The results are summarized in Table 7.

Table 7 Comparison of calculated ultimate shear capacity, V_T, and the observed shear capacities of the test specimens ($V_{M,CMOD1}$)

#	$f'_{ave,c}$ (MPa)	$V_{M,CMOD1}$ (kN)	V_c (kN)	V_f (kN)	V_T (kN)	Difference (%)
A1a	64.75	103.46	96.78	0	96.78	-6.5
A1b		91.64		0	96.78	+5.6
A2a		95.33		0	96.78	+1.5
A2b		90.55		0	96.78	+6.9
S1a	63.25	112.08	96.46	17.34	113.80	+1.5
S1b		119.50		17.34	113.80	-4.8
S2a		107.59		17.34	113.80	+5.8
S2b		116.21		17.34	113.80	-2.1
S3a	61.63	108.51	96.11	17.34	113.45	+4.6
S3b		129.56		17.34	113.45	-12.4
S4a		117.69		17.34	113.45	-3.6
S4b*		152.80		17.34	113.45	-25.8

** Omitted specimen*

V_f values are calculated using the manufacturer's specifications for 15 kg/m³ steel fibre content

When these capacities are plotted, it can be seen that the calculated shear capacities correlate well with the experimental results. This is also valid for the specimens with steel fibres. Due to the lack of information for high dosages of steel fibres, the results are more accurate for a steel fibre content about 15 kg/m^3, as suggested by the manufacturer. Moreover, the capacity gain from 13.33 kg/m^3 to 26.67 kg/m^3 is negligible in the reported experimental results. However, as stated previously, increasing doses of steel fibres seem to be increasing the residual capacity more efficiently than the ultimate capacity (Shown in Figure 18), which can be beneficial to ductility and life safety of these elements.

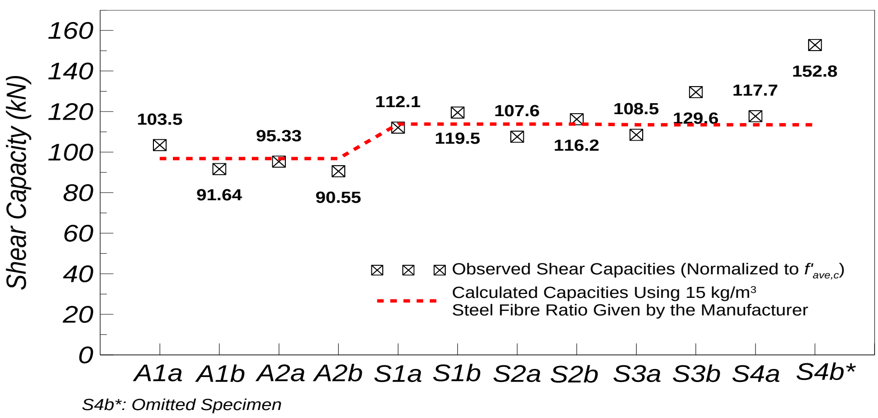


Figure 19 Comparison of experimental shear capacities (normalized to $f'_{ave,c}$) and calculated shear capacities according to NZS 3101

CONCLUSIONS

The theory and best practise supports the benefits in casting top stand in 300mm and 400mm deep hollowcore:

- Improved shear capacity.
- Better control of web shear cracking.
- Better camber control for the customer.

Manufactures are recommended to adopt top strand to satisfy end stress limits as common practise in their design as well as include web shear inspections as part of their QA processes, if they are not already, to regain confidence levels of structural engineers.

Considering the limited number of tests reported herein, it is not possible to generalise the results. However, comparing these results with more detailed research at the University of Nottingham (Paine, Andrew 1998) it can be stated that addition of steel fibres into concrete, in general, has beneficial effects. Although they add to the ultimate shear capacities of the pre-stressed hollow core elements (~20% in the reported work), this additional strength is not too significant above 15 kg/m^3 dosage of steel fibres. On the other hand, higher steel fibre content may contribute more significantly to residual strength of the considered element. This allows some residual force resistance that may be important for life safety during such sudden failures. The quantification of the residual capacity for such scenarios are still an engineering challenge that still needs further research (Al-Ani et al. 2008; NZS3101 2006). Nonetheless, even the low dose of 13.33 kg/m^3 steel fibre content resulted in an ultimate capacity gain of approximately 20% while it caused approximately 50% residual strength gain. 26.67 kg/m^3 dose of steel fibre caused capacity gains of 25% for ultimate and 90% for residual.

It would appear the inclusion of minimal dosing of steel fibres in the hollowcore mix may provide an element of robustness to what is traditionally seen as non-shear steel reinforced brittle product to guard against web shear while in service in buildings dominated by earthquake demand.

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