Benefits of Top Strand and Steel Fibres in the Design and Manufacture of Hollow-core Precast Floor Slabs.

N. Brazzale & D.I. Kennett

Stahlton Engineered Concrete (a division of Fulton Hogan Limited), Christchurch.

J.B. Marshall

Formerly of Stahlton Engineered Concrete

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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 have shown 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 extruded or slip-formed process by which hollow-core floor slabs are produced make it impractical to cast in traditional shear 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 an increase in shear capacity as well as enhanced residual load carrying capacity after failure. The findings will be described in this paper and presented to show how Hollow-core units designed with controlled stress levels and reinforced with steel fibres better resist web shear and high gravity loads.

1 INTRODUCTION

Prestressed hollow-core slabs are the most common form of precast concrete flooring worldwide with around 50% of the total flooring market in Europe (Paine 1998). Hollow-core flooring has practical and economic advantages over other flooring systems.

The collapse of hollow-core 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 hollow-core pre-stressed flooring units in seismic regions. The aftermath of both these failures indicated signs of pre-cracked web shear initiated by inadequate concrete strength at release of pre-stress and/or inadequate attention to stresses at the ends of the units during the design phase.

Possible solutions are to manufacture highly pre-stressed hollow-core units with stressed top strands and to cast the hollow-core 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 extrusion or slip form machine process. Steel fibres were found to have the added benefit of increasing shear capacity of bare units by 20% with minimal dosages.

Research into the shear performance of hollow-core flooring units is not new. Testing was carried out by University of Nottingham in the UK by Paine (1998) and his results gave similar shear performance enhancement.

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

2 DESIGN CHECKS TO INCLUDE TOP STRAND AND AVOID WEB SHEAR

End stresses are calculated using internal stress blocks incorporating the total initial pre-stress at release, P_i, area of the section, A, eccentricity of the pre-stress 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 pre-stressed 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 manufacturers tend to adopt is 28MPa, so the limits are 2.64MPa in tension and 16.8MPa in compression.

Generally 150mm and 200mm deep hollow-core will not exceed stress limits. Depending on the number of strand and their pre-stress level the issue arises in 300mm and 400mm deep hollow-core units. Internal shear testing by Stahlton suggest there is a vertical shear capacity reduction of approximately 20% for bare units with web shear cracking induced by the high end stresses (Figure 1).

Figure 1. Web shear observed in this 400mm deep hollow-core unit

3 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 ($V_s=0$) and steel fibres (V_f). Accordingly, the shear capacity of a pre-stressed hollow core unit can be calculated as given below (NZS3101 2006):

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

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

The mean values of $f_{Rk,i}$ (the characteristic residual tensile strength of the steel fibre reinforced concrete at crack mouth opening level i) for each level of $CMOD_i$ are given by manufacturer specifications (Appendix A) and are summarized for the used steel fibre type is given in Table 1 (Values are given for 15 kg/m^3 steel fibre dosage rate).

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/m^3 fibre dosage

4 TEST SETUP

A single point loading test setup at the Stahlton Christchurch yard was used for the experimental tests. The loading distance from the nearest simple support was arranged such that the specimen would 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 2 and Figure 4 respectively.

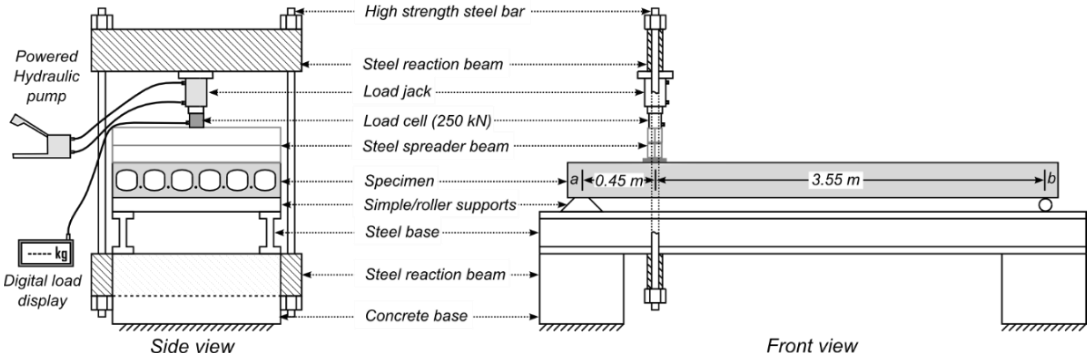

Figure 2. Test setup

5 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 to 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 3 with the only difference being the amount of steel fibres put into each specimen (Steel fibre type: Dramix 3D 80/60BG).

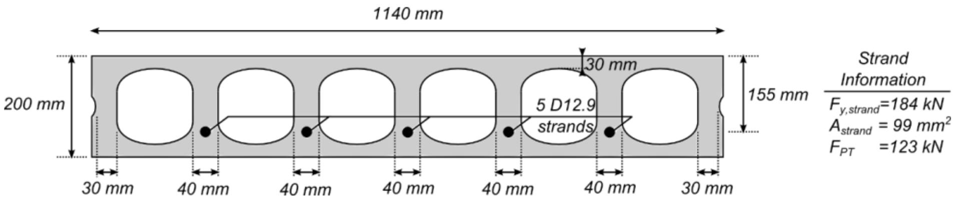

Figure 3. Cross section for the test specimens and pre-stress information

Figure 4. Test specimen placed within the setup

In total, there were six test specimens. Two of these specimens were as built control specimens without any steel fibres (Specimens A1, A2). Specimens S1 and S2 had 13.33 kg/m^3 steel fibres while specimens S3 and S4 contained 26.67 kg/m^3 steel fibres. Both ends of each specimen (end a, end b) were tested to shear failure, giving 12 tests in total. Concrete compressive strength, f'_c, was measured using a digital Schmidt hammer 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/m^3)*
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

6 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). A summary of the force readings taken during the tests is given in Table 3 and photographs of the damage sustained during testing of some of the units are shown in Figure 5.

Table 3. Summary of test observations (forces recalculated using g=9.81m/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
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
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 (V=0.8875F)

The results of specimen S4b have been neglected as the reaction frame showed significant out-of-plane deflection and the comparably high capacity observed in this test may not represent the real capacity value. Therefore, the results of S4b are excluded in the analysis of results section.

Figure 5. Photographs of observed damage (loads noted on units use g=10m/s^2)

7 ANALYSIS OF RESULTS

In order to analyse the results in a meaningful way, the average concrete strength of each specimen group is adopted. Since shear strength in concrete elements is directly proportional to the square root of the concrete strength, the observed experimental shear capacity values can be modified accordingly as shown in Table 4.

Table 4. Test results modified according to the average concrete strength values in each group of test specimens

#	f'_c (MPa)	Observed		$f'_{ave,c}$ (MPa)	Modified			
		F_{CMOD1} (kN)	F_{CMOD4} (kN)		$F_{M,CMOD1}$ (kN)	$F_{M,CMOD4}$ (kN)	$V_{M,CMOD1}$ (kN)	$V_{M,CMOD4}$ (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: $F_{M,CMODi} = \sqrt{\dfrac{f'_{ave,c}}{f'_c}} \times F_{CMODi}$, $\quad V_{CMODi} = F_{CMODi} \times 0.8875$ (from equilibrium)

Using these results, the comparison of all the specimens can be made as shown in Figure 6. 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.

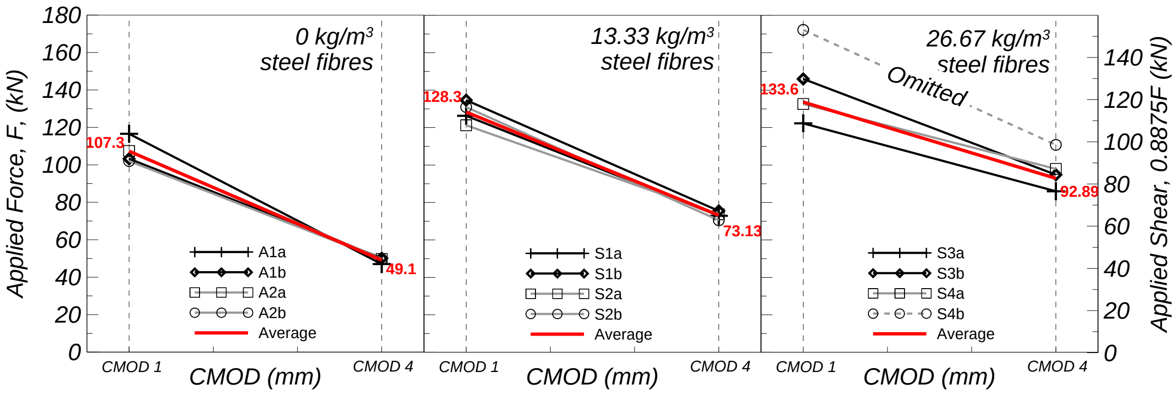

Figure 6. Test results modified using average concrete strength values: a) As built specimens without any steel fibres; b) Specimens with 13.33 kg/m3 steel fibres; c) Specimens with 26.67 kg/m3 steel fibres

Steel fibres appear to be more effective at increasing the residual capacity (i.e. at CMOD4 level) than the capacity at shear failure (i.e. CMOD1 level). This can be clearly seen when percentage of additional capacity over the as built capacity are plotted at CMOD1 and CMOD4 levels (Figure 7).

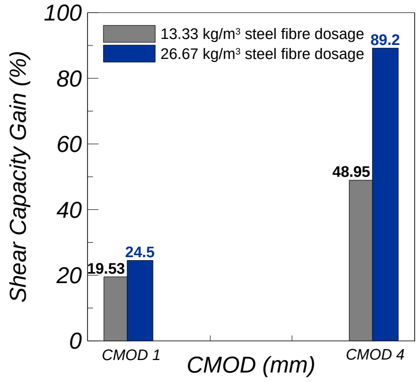

Figure 7. Percentage of shear capacity gain over the as built capacity

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 CMOD1 and CMOD4 respectively. Similarly, addition of 26.67 kg/m3 of steel fibres caused capacity gain of 24.5% and 89.2% at CMOD1 and CMOD4 respectively.

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.

8 CAPACITY CHECK ACCORDING TO NZS 3101

8.1 Concrete shear contribution (V_c)

Concrete shear contribution is calculated based on the following given parameters:

- b_w = 260 mm
- d = 155 mm
- A_{pt} = 99 mm2
- 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 4), will be used for comparison with theoretical capacity values. The resulting concrete shear contribution values, V_c, are given in Table 5.

8.2 Shear contribution due to steel fibres (Vf)

Ultimate capacity values due to the additional steel fibre content can be calculated using equations outlined in Part 2 appendix C5A of NZS 3101 (NZS3101 2006). 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 will result in an approximate 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 5.

Table 5. Comparison of calculated (V_T) and observed ($V_{M,CMOD1}$) ultimate shear shear capacities

#	$f'_{ave,c}$ (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 is calculated using the manufacturer's specifications for 15 kg/m^3 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 steel fibre content close to 15 kg/m^3, as is recommended 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. Notwithstanding this, increasing the dosage of steel fibres appears to increase the residual capacity to a greater extent than the ultimate capacity (refer Figure 7), which can be beneficial to ductility and life safety in the event of an ultimate failure of these elements.

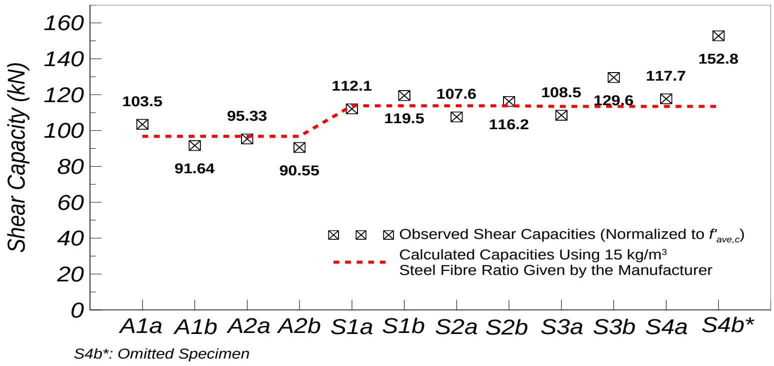

Figure 8. Comparison of experimental shear capacities (normalized to $f'_{ave,c}$) and calculated shear capacities

9 CONCLUSIONS

Theory and best practise supports the inclusion of top strand in 300mm and 400mm deep hollow-core units for improved shear capacity, better control of web shear cracking and for camber control.

Manufacturers are recommended to adopt top strand to satisfy end stress limits as common practise in their designs as well as to include web shear inspections as part of their QA processes.

Due to the limited number of tests undertaken to date, it is not possible to generalise the results. Comparison of these results with the more detailed research at the University of Nottingham (Paine 1998) adds weight to the conclusion that the addition of steel fibres has beneficial effects. Steel fibres are shown to add to the ultimate shear capacity of pre-stressed hollow-core units, in the order of 20%

in the work undertaken to date. At dosage rates higher than the manufacturer recommended 15 kg/m^3 the additional increase in ultimate capacity does not appear significant.

Higher steel fibre content does however appear to result in a higher residual strength of the considered element. This allows for some residual force resistance that may be important for life safety during such sudden failures. The quantification of the residual capacity for such scenarios is still an engineering challenge that 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.

The inclusion of minimal dosing of steel fibres in the hollow-core mix appears to 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.

10 FURTHER RESEARCH

Further research is required to conclusively prove and quantify the benefit of including steel fibre reinforcing in pre-stressed hollow-core flooring units in New Zealand. Stahlton Engineered Concrete has engaged with the University of Canterbury to commence discussions around further progressing research into the causes of and potential mitigation measures against web shear cracking.

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