STRENGTH AND SERVICEABILITY OF CONTINUOUS STEEL FIBRE REINFORCED CONCRETE (SFRC) COMPOSITE SLABS WITH DEEP TRAPEZOIDAL STEEL DECKING

by

F.M. Abas, R.I. Gilbert, S.J. Foster and M.A. Bradford
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ABSTRACT
This report forms part of an on-going research project within the Centre for Infrastructure Engineering and Safety at the University of New South Wales on the strength and serviceability of steel fibre reinforced concrete (SFRC) composite slabs with deep trapezoidal steel decking. An experimental program is described involving load tests on two-span composite slabs fabricated with deep trapezoidal decks and steel fibre reinforced concrete. The aim was to study the effects of varying the steel fibre dosage on the cracking behaviour at the negative moment region, the redistribution of moments, the end slip between the decking and the concrete, and on the load carrying capacity of the slabs. In total, 8 two-span composite slab specimens were cast and moist cured for a period of 14 days and then loaded monotonically to failure at an age of at least 28 days. In addition to the steel decking, one of the specimens contained no reinforcing steel and no steel fibres, four of the specimens were reinforced only with steel fibres in the concrete (with nominal fibre contents of either 20, 30 and 40 kg/m$^3$). In the other three specimens, welded wire-mesh was included over the interior support, one with plain concrete and two with steel fibres in the concrete. The concrete properties, including compressive strength, tensile strength, modulus of elasticity and fracture energy, were measured on companion specimens for every test slab. Compared to the plain concrete composite slab and the slab containing SL62 welded wire mesh in the negative moment region over the interior support, the slabs containing steel fibres in excess of 20 kg/m$^3$ provided significant improvements in the slip load and the peak load. In addition, at service load levels the fibres provided crack control that was of similar effectiveness to that provided by the SL62 mesh.

KEY WORDS
Composite slabs; cracking; deflection; ductility; laboratory testing; moment redistribution; partial-interaction; steel fibre reinforced concrete (SFRC); strength; trapezoidal steel decking.
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1. INTRODUCTION

Composite steel-concrete slab floor systems are widely used in today's construction industry. Composite slabs with deep trapezoidal steel decks have a good reputation as load carrying members and are a very efficient form of construction. The steel-deck acts as the bottom, tensile reinforcement in the positive moment region of the slab. Conventional reinforcement in the form of bars or welded wire mesh is usually used as top reinforcement in the negative moment regions over the interior supports and also to control any cracking at the top surface of the slab due to restrained shrinkage or thermal strains. Construction would be simplified, and potentially become more economical, if the labour intensive task of reinforcement fixing and control of cover could be eliminated from the construction, and if a viable alternative means of crack control was available for continuous composite slabs.

The use of steel-fibres in concrete as an alternative (or supplement) to conventional reinforcement is now a mature technology after almost 40 years of experience and extensive research. The amount of fibres being produced and used by the construction industry is steadily growing and the range of application of steel fibre reinforced concrete (SFRC) is expanding. The inclusion of steel fibres in concrete has been shown to improve the post-cracking behaviour of a reinforced concrete member, in terms of crack control, ductility, punching shear (Maya et al., 2012) and fire resistance (Fike and Kodur, 2011). Therefore, the innovation of combining composite steel-concrete slabs and steel-fibres seems to be viable, with particular advantages for continuous slabs. Hitherto, there has been surprisingly little research reported on this topic in the open literature.

This report forms part of an on-going research project within the Centre for Infrastructure Engineering and Safety at the University of New South Wales on investigating the strength and serviceability of SFRC composite slabs with deep trapezoidal steel decks. Specifically, it describes a series of short-term static load tests on two-span composite slabs. The aim of the tests was to study the effects of varying the steel-fibre dosage on the cracking behaviour at the negative moment region and on the load carrying capacity of the slabs. In total, 8 two-span composite slab specimens were cast and moist cured for a period of 28 days and then loaded to failure. In addition to the steel decking, one of the specimens contained no reinforcing steel and no steel fibres, four of the specimens were reinforced only with steel fibres in the concrete (with nominal fibre contents of either 20, 30 and 40 kg/m³). In the other three specimens welded wire-mesh was included over the interior support, one with plain concrete and two with steel fibres in the concrete.

The experimental program was designed to gain a clearer insight into the effect of steel fibres on the mechanism of flexural cracking over the interior supports in composite slabs with deep trapezoidal steel decking. The location and width of cracks over the intermediate support for each specimen were carefully monitored. The effects of steel fibre dosage on the performance of each SFRC composite slab, in terms of maximum load, deflection, end slip and moment redistribution, were also studied. Each specimen was 7.0 m long, with two equal spans of 3.4 m and a 100 mm overhang at each end. The concrete properties, including compressive
strength, tensile strength, modulus of elasticity and fracture energy, were measured on companion specimens for every slab tested.

All slabs specimens were tested to failure using a deformation-controlled actuator applying load at the third points of each span through steel spreader beams. The load versus deflection response throughout the full loading range was recorded, together with the slip between the concrete and the steel decking measured at each end of the specimen, and the crack width and crack patterns in both the positive and negative moment regions.

The results indicate that the inclusion of steel fibres in excess of 20 kg/m³ provides significant advantages in terms of both the load at which slip between the concrete and the steel decking occurs and the peak load, as well as providing acceptable crack control at service loads.

2. BACKGROUND ON THE STRENGTH AND SERVICEABILITY OF SFRC COMPOSITE SLABS

The strength of a simply-supported composite steel-concrete slab depends on the longitudinal shear capacity between the deck and the concrete slab and this controls the effectiveness of the steel deck as positive moment reinforcement (Oehlers and Bradford, 1995; Petkevicius et al. 2010; Roberts-Wollman et al., 2004). Conventional reinforcing bars or welded wire mesh may be included for the control of cracking due to restrained shrinkage or for fire-resistance purposes. For continuous slabs, reinforcing bars or welded wire mesh are usually used to increase the strength of the section in the negative moment region over the interior supports and to control flexural cracking on the top surface of the slab.

While the deep trapezoidal decking alone provides some negative moment resistance at an interior support after the concrete has cracked, the absence of any top steel will result in a sudden redistribution of moments at first cracking and one or more wide unserviceable cracks over each interior support develop as the applied load increases.

As an alternative to reinforcing bars or mesh, steel fibres can be used to enhance the load-carrying capacity of the slabs and a number of investigations have been carried out over the past two decades. Ibrahim et al. (1994) investigated the effectiveness of steel fibres in composite slab systems. Their tests consisted of different specimens reinforced with wire-mesh and various quantities of steel-fibres ranging from 0.2% to 0.34% fibres by volume. The test specimens also had varying slab depths and different decking profiles. Similar tests were also conducted in Germany by Ackermann et al. (2008, 2009) and in the USA by Roberts-Wollman et al. (2004). The common objective of these studies was to find the suitability of steel fibres as a replacement to wire-mesh in composite slab construction. However, no conclusive design solutions pertaining to this application were proposed and it
was concluded that the use of steel fibres in negative moment regions in lieu of steel mesh still required substantial additional research.

This report describes an experimental program to quantify the strength and serviceability of continuous SFRC composite slabs with deep trapezoidal decking by varying the steel fibre dosage with and without the inclusion of wire-mesh. The effects on deflection, crack widths and moment redistribution at service loads are studied in detail and the bond-slip mechanism that controls the load carrying capacity of the slab is also considered, as are the strain hardening post-slip response, the strain softening post-peak response and the ductility of the slabs.

3. EXPERIMENTAL PROGRAM AND TEST SPECIMENS

3.1 Overview

Two-span continuous composite slabs fabricated using deep-trapezoidal steel sheeting (W-deck) and steel fibre reinforced concrete were cast and loaded to failure in the Heavy Structures Research Laboratory at the University of New South Wales in order to investigate the behaviour and characteristics of composite slabs containing steel fibres with and without conventional reinforcement in the negative bending region. The slab specimens were constructed from a ready-mix concrete containing various dosages of steel fibres ranging from 0 kg/m³ to 40 kg/m³ by volume. High strength steel, end hooked fibres (either 60 mm long Dramix RC80/60BN fibres or 35 mm long RC65/35BN fibres) were used. The desired volume of fibres was mixed into the concrete in the drum of the ready mixed concrete truck immediately before casting the test specimens.

In total, eight prismatic SFRC composite slabs with deep trapezoidal decking were cast and moist cured for a period of 14 days and then loaded to failure at ages greater than 28 days. A schematic diagram of the slabs and supports is shown in Figure 1.

![Diagram](image)

**Figure 1:** Dimensions and geometry of the two-span slabs specimen.
In addition to the steel decking, one of the specimens contained no reinforcing steel and no steel fibres, and four of the specimens were reinforced only with steel fibres in the concrete (with nominal fibre contents of either 20, 30 and 40 kg/m³). In the other three specimens welded wire-mesh was included over the interior support, one with plain concrete and two with steel fibres in the concrete. Each specimen was 7.0 m long by 700 mm wide, with two equal spans of 3.4 m, and a 100 mm overhang at each end and with cross-sectional. The thickness of each slab was between 142 mm and 152 mm (see Table 1). The concrete properties, including the compressive strength, direct tensile strength, residual tensile strength, stress-strain curve, modulus of rupture and fracture energy, were measured on companion specimens in the form of standard cylinders and prisms for every batch of concrete.

All slabs specimens were tested to failure using a deformation-controlled actuator applying load at the third points of each span through steel spreader beams. The load versus deflection response throughout the full loading range was recorded electronically, together with the slip between the concrete and the steel decking measured at each end of the specimen and the applied loads and the reactions at each support. The crack width and crack patterns in both the positive and negative moment regions were recorded manually throughout the tests.

The major objectives of the experimental program were to gain an understanding of the influence of various steel fibre dosages on:

(i) the extent and width of cracks at the intermediate support regions;

(ii) the deflection and degree of moment redistribution at service loads;

(iii) the load-carrying capacity and the end slip between the decking and the SFRC up to and beyond the peak load.

Another objective was to investigate the effects of combining steel fibres and welded wire mesh in the negative moment region on both the cracking behaviour and the load carrying capacity of the slabs.

3.2 Test Parameters

Table 1 provides details of each test specimen. In the specimen identifier, the first term is the specimen number (S1 to S8), the second term represents the nominal steel fibre dosage in kg/m³ and fibre type (L for 60 mm long fibres, S for 35 mm short fibres) and the third term represents the type of welded wire mesh, including wire size and spacing (e.g. 62 is mesh with 6 mm diameter wires at 200 mm centres).
Table 1: Details of test specimens.

<table>
<thead>
<tr>
<th>Slab Specimen</th>
<th>Actual slab depth, $D$ (mm)</th>
<th>Steel fibre length (mm)</th>
<th>Nominal steel fibre dosage (kg/m$^3$)</th>
<th>Measured steel fibre dosage (kg/m$^3$)</th>
<th>Mesh wire diameter/ wire spacing/ reinforcement ratio, $A_s/A_o$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1 - 0 - 00</td>
<td>142</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>S2 - 0 - 62</td>
<td>152</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>6 mm / 200 mm / 0.15%</td>
</tr>
<tr>
<td>S3 - 20L - 00</td>
<td>145</td>
<td>60</td>
<td>20</td>
<td>18.76</td>
<td></td>
</tr>
<tr>
<td>S4 - 30L - 00</td>
<td>142</td>
<td>60</td>
<td>30</td>
<td>27.92</td>
<td></td>
</tr>
<tr>
<td>S5 - 30L - 52</td>
<td>142</td>
<td>60</td>
<td>30</td>
<td>30.57</td>
<td>5 mm / 200 mm / 0.10%</td>
</tr>
<tr>
<td>S6 - 30L - 82</td>
<td>145</td>
<td>60</td>
<td>30</td>
<td>27.92</td>
<td>8 mm / 200 mm / 0.27%</td>
</tr>
<tr>
<td>S7 - 40L - 00</td>
<td>152</td>
<td>60</td>
<td>40</td>
<td>38.27</td>
<td></td>
</tr>
<tr>
<td>S8 - 40S - 00</td>
<td>146</td>
<td>35</td>
<td>40</td>
<td>40.21</td>
<td></td>
</tr>
</tbody>
</table>

An elevation and a section through a typical specimen indicating the location and extent of the welded wire mesh located in the top of the slab in the negative moment region over the interior support is shown schematically in Figure 2.

![Elevation and Section](image)

**Figure 2:** Location of welded wire mesh over the interior support of the two span specimens.

### 3.3 Construction of Specimens and Test Procedures

Figure 3(a) shows the formwork used in the construction of the specimens. Prior to casting each specimen, the inside surfaces of the timber side and end forms were cleaned and thinly coated with a concrete release agent to prevent adhesion of the concrete and to facilitate removal of the timber forms.
Commercially pre-mixed concrete was used in the manufacture of the test specimens. The concrete specified for the project had a maximum aggregate size of 10 mm and a slump of 140 mm. The relatively high slump was deliberately selected to ensure workability of the concrete after the inclusion of the steel fibres and to eliminate balling of the fibres that may occur should the concrete become too stiff at the time of mixing and placing. The concrete was placed in the formwork in equal layers and compacted using a hand vibrator. The top surface of each specimen was then finished by trowelling. Figure 3(b) shows the specimens just after the completion of concreting.

(a) Formwork prior to casting a typical slab.  (b) Slab specimens just after casting.

Figure 3  Fabrication of typical SFRC composite slab specimen.

Companion specimens were also cast at the same time as the slab specimens, including 150 mm diameter cylinders, for measuring the compressive stress-strain relationship and compressive strength, and 150mm x 150mm x 600mm prisms for measuring flexural tensile strength and fracture energy. Dog-bone shaped specimens were also cast to measure direct tensile strength and residual tensile strength (discussed further in Appendix 2). Upon the completion of the concreting, the specimens were covered with wet hessian and plastic sheets, as shown in Figure 4, and were moist cured for a period of 14 days after casting by hosing with water every day to facilitate hydration and minimise the loss of moisture. The specimens were left undisturbed in the formwork for two days prior to removal of the side and end forms.

Figure 4: Specimens were kept cured and moist under wet hessian.
The steel sheeting used for each composite slab was the deep trapezoidal steel deck (W-deck) supplied by Lysaght-Bluescope and the steel fibres were supplied by BOSFA. These testing materials were supplied as part of the ARC Linkage Project LP0991495, being conducted at UNSW, in partnership with Lysaght-Bluescope and BOSFA. The cross-sectional dimensions of the deck are shown in Figure 5 and section properties are given in Table 2. The thickness of the steel deck used throughout the experimental program was 1.00 mm.

One single sheet of W-deck 700 mm wide

Figure 5: Cross-section of a single sheet of W-deck trapezoidal steel decking.

Table 2: Properties of deep trapezoidal steel deck.

<table>
<thead>
<tr>
<th>Thickness $t_p$ (mm)</th>
<th>Area $A_{deck}$ (mm$^2$/m)</th>
<th>Weight (kg/m)</th>
<th>Width (mm)</th>
<th>Moment of Inertia $I_{ex}$ (mm$^4$/m) $\times 10^6$</th>
<th>Nominal yield strength $f_y$ (N/mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00</td>
<td>1414</td>
<td>8.14</td>
<td>700</td>
<td>119.9</td>
<td>550</td>
</tr>
</tbody>
</table>

Prior to concreting, headed shear studs were fixed at the intermediate support of each slab by welding directly to a 6 mm thick steel plate underneath the deck, as shown in Figure 6. The steel plate was used to replicate the top flange of the secondary steel beam that is typically used as an interior support in the construction of composite floor slabs. The shear studs were included in the test specimens because they are usually present on the top flange of secondary steel beams in typical composite floor slabs and may act as crack initiators.

Figure 6: Shear studs at intermediate support.
Two types of Dramix steel-fibres were used for the experimental program:

1. 60 mm long hooked end fibres (RC80/60BN) were used in the majority of the test specimens (S3 to S7); and

2. 35 mm hooked-end steel fibre (RC65/35BN) were used in one specimen (S8).

The properties of both types of steel fibres are given in Table 3. The steel fibres were mixed to the required dosages in the back of the concrete truck on-site within the laboratory. The quantities of the measured steel fibres added to the concrete for each test specimen were calculated and the exact dosage determined after mixing using standard wash-out tests. Data on fibre volumes for the concrete used in each slab was obtained from wash-out tests before, during and after casting and is given in Appendix 1.

**Table 3: Nominal properties of steel fibres used in the experimental program.**

<table>
<thead>
<tr>
<th>Steel-Fibre Designation</th>
<th>Diameter (mm)</th>
<th>Length (mm)</th>
<th>Aspect Ratio</th>
<th>Mean Tensile strength (MPa)</th>
<th>Modulus of Elasticity (GPa)</th>
<th>Strain at proportional limit (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC80/60BN (Long fibre)</td>
<td>0.75</td>
<td>60</td>
<td>80</td>
<td>1225</td>
<td>200</td>
<td>5500</td>
</tr>
<tr>
<td>RC65/35BN (Short fibre)</td>
<td>0.55</td>
<td>35</td>
<td>64</td>
<td>1345</td>
<td>200</td>
<td>5500</td>
</tr>
</tbody>
</table>

### 3.4 Instrumentation

The instrumentation used in the experimentation is indicated in Figure 7 and includes laser transducers, load cells, linear variable displacement transducers (LVDTs), mechanical dial gauges, concrete strain gauges, surface mounted strain gauges, demec gauges and a hand-held microscope.

Three laser transducers, each with an accuracy of 0.01 mm, were used to measure the deflection of the slabs at the mid-point of each span. The laser transducers can be seen in Figure 8.

Reactions were measured throughout the test using two load cells at each support as shown in Figure 9. Load cells LC1 and LC2 measured the reaction at the exterior support of the left span, LC3 and LC4 measured the reaction at the middle (or interior) support while LC6 and LC7 measured the reaction at the exterior support of the right span. An additional load cell (LC5) was located under the actuator to measure the load applied to the slab through the testing frame and spreader beam at all stages of the test.

A data logger was used to record the laser readings and load cell readings as loading progressed.
Figure 7 Typical instrumentation locations for test slabs.

Figure 8: Laser transducer position under mid-span of a typical slab.

Figure 9: Load cells positions (under slab).

The slip between the steel deck and the concrete was measured by using LVDTs, attached at both ends of each test specimens. Figure 10 show a typical arrangement of the LVDTs. The readings from the LVDTs were also recorded electronically by the data logger.
Concrete strain measurements at the top surface of the slabs over the intermediate support were recorded using both surface mounted strain gauges and Demec gauges. The strain gauge readings were electronically recorded by the data logger system, whereas the readings of the displacement between the Demec points were recorded manually using a Demec strain gauge over a gauge length of 200 mm. Figures 11 and 12 show the positions of the strain measuring devices.

The crack widths over the interior support in the negative moment region were directly measured using a hand-held microscope while the test was paused after each 5 kN load increment. The Demec gauge was also used to measure the crack widths by recording the deformation of the concrete between the Demec points on either side of the negative moment crack.

![Figure 10: LVDTs used to measure slip between steel deck and concrete slab.](image)

![Figure 11: Strain gauge points at top surface over interior support (plan view).](image)
3.5 Loading Procedure

Figures 13 and 14 show the test set-up, including the spreader beam arrangements. Load was applied at the third-span points in both spans. The initial load comprising the slab self-weight, the weight of the spreader beams and the weight of all packing used in each test was measured at the beginning of the test as the sum of the reactions measured by the load cells at each support.

The test procedure for all test specimens was identical. Deformation was applied continuously and slowly throughout the test in order to monitor the complete load response, including the post-peak load behaviour of each slab. The rate of deformation of the actuator ram was initially 0.3 mm/min up to the peak load and, subsequently, at a rate of 0.5 mm/min in order to accelerate the deformation in the post-peak range. Failure was deemed to have
occurred when one of the spans had deflected excessively and the load had dropped significantly (by more than 30%) below its peak value.

Data was recorded electronically continuously throughout the test. Initially, loads were applied in 5 kN increments, paused after each increment to inspect for cracking. After cracking was first detected (always over the interior support), the slab was unloaded. Load was then re-applied in 5 kN increments, pausing after each increment to take Demec and crack width readings, to inspect for new cracking and record crack locations. Deformation was gradually increased up to failure of the slab.

![Image](image_url)

**Figure 14: Testing set-up of the two-span SFRC slab.**

### 4.0 TEST RESULTS

The age and date of testing of each slab specimen is summarized in Table 4.

<table>
<thead>
<tr>
<th>Slab Specimen</th>
<th>Date of testing</th>
<th>Age at testing (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-0-00</td>
<td>19/5/11</td>
<td>28</td>
</tr>
<tr>
<td>S2-0-62</td>
<td>1/12/11</td>
<td>31</td>
</tr>
<tr>
<td>S3-20L-00</td>
<td>6/7/11</td>
<td>28</td>
</tr>
<tr>
<td>S4-30L-00</td>
<td>7/9/11</td>
<td>29</td>
</tr>
<tr>
<td>S5-30L-52</td>
<td>16/1/12</td>
<td>31</td>
</tr>
<tr>
<td>S6-30L-82</td>
<td>12/7/11</td>
<td>34</td>
</tr>
<tr>
<td>S7-40L-00</td>
<td>18/1/12</td>
<td>33</td>
</tr>
<tr>
<td>S8-40S-00</td>
<td>12/7/11</td>
<td>32</td>
</tr>
</tbody>
</table>

*Note: L = 60mm long fibre; S = 35mm long fibre*
4.1 Material Properties

Standard cylinders (150 mm diameter) and prisms (150 mm x 150 mm x 600 mm) were used to determine the compressive strength, modulus of elasticity, flexural tensile strength, direct tensile strength and the fracture energy of the test specimens. Direct tension tests using dog-bone shaped specimens were used to determine the residual tensile stress \( f_{15} \) at a crack opening displacement (COD) of 1.5 mm. Details of the dog-bone test specimens are provided in Appendix 2, together with graphs of the tensile stress versus crack opening displacements for the plain or fibre reinforced concrete used in each specimen. The results are summarised in Table 5.

Table 5: Material properties for each slab specimen at the time of testing.

<table>
<thead>
<tr>
<th>Slab Specimen</th>
<th>Average compressive strength (MPa)</th>
<th>Modulus of Elasticity (MPa)</th>
<th>Flexural Tensile Strength (MPa)</th>
<th>Direct Tensile Strength (MPa)</th>
<th>Residual tension ( f_{15} ) (MPa)</th>
<th>Fracture Energy (N/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1 - 0 - 00</td>
<td>49.1</td>
<td>30,100</td>
<td>4.44</td>
<td>3.65</td>
<td>-</td>
<td>129</td>
</tr>
<tr>
<td>S2 - 0 - 62</td>
<td>46.3</td>
<td>29,100</td>
<td>4.59</td>
<td>4.52</td>
<td>-</td>
<td>189</td>
</tr>
<tr>
<td>S3 - 20L - 00</td>
<td>43.5</td>
<td>27,500</td>
<td>3.90</td>
<td>3.55</td>
<td>0.55</td>
<td>3,470</td>
</tr>
<tr>
<td>S4 - 30L - 00</td>
<td>45.5</td>
<td>27,400</td>
<td>5.05</td>
<td>4.75</td>
<td>0.78</td>
<td>7,410</td>
</tr>
<tr>
<td>S5 - 30L - 52</td>
<td>45.0</td>
<td>30,400</td>
<td>5.06</td>
<td>3.33</td>
<td>0.78</td>
<td>4,260</td>
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<tr>
<td>S6 - 30L - 82</td>
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<td>27,400</td>
<td>5.05</td>
<td>4.75</td>
<td>0.78</td>
<td>7,410</td>
</tr>
<tr>
<td>S7 - 40L - 00</td>
<td>42.8</td>
<td>33,100</td>
<td>4.62</td>
<td>2.73</td>
<td>1.3</td>
<td>5,520</td>
</tr>
<tr>
<td>S8 - 40S - 00</td>
<td>57.8</td>
<td>31,700</td>
<td>5.15</td>
<td>3.93</td>
<td>0.63</td>
<td>3,250</td>
</tr>
</tbody>
</table>

4.2 Slab Specimen S1-0-00

Slab S1-0-00 contained no steel fibres or conventional steel reinforcement. This was the control slab that determined the lower bound values of the parameters under investigations. The measured self-weight of the slab, spreader beams and steel packing plates was 21.1 kN. Load from the actuator with the displacement control rate of 0.3 mm/min was applied and dispersed to the third points of both spans by the spreader beams. Failure was deemed to have occurred when one of the spans had deflected excessively and the load had dropped significantly (by more than 30%) below its peak value.

In this test, it was the left span (see Figure 15) that experienced the excessive deflection and triggered the failure. Figure 15 shows the left span deflecting excessively under load at failure, while the right span underwent significantly less deflection.

The load applied by the actuator \( P \) versus the deflection at the mid-point of the left span is shown in Figure 16. It can be seen that the load dropped slightly with the loss of stiffness
caused by the onset of cracking at the top surface of the slab at the interior support (at a load of about $P = 25$ kN). However, after cracking, the load increased up to positive moment cracking in the span and the initiation of slip between the sheeting and the concrete at a load of about $P = 41$ kN. As deformation increased, the load again increased gradually up to the peak value of $P = 61.8$ kN, before it dropped significantly in association with large deflection of the left span and large slip at the decking–concrete interface. The mid-span deflection of the left span corresponding to the maximum load was 49.5 mm.

The equivalent uniformly distributed superimposed load corresponding to the applied load ($P$ plus the weight of the spreader beams and packing) at the onset of cracking over the interior support is 7.59 kPa.

![Figure 15: Excessive deflection in left span of specimen S1-0-00.](image)

Crack widths were measured during the test by using a hand-held microscope with a magnification factor of 40x. First cracking occurred at the top surface of the slab directly over the interior support region at $P = 24.7$ kN. Due to the sudden loss of stiffness, the load dropped to $P = 16.0$ kN when the first crack occurred. In the absence of any reinforcement or fibres crossing the crack, the initial crack width immediately after cracking at $P = 16.0$ kN was 0.62 mm. This crack width is greater than the usual serviceability limit for maximum crack width (often taken to be 0.3 mm). A side, unserviceable crack was expected in this unreinforced region in negative bending. As loading progressed, the crack opened significantly. The crack width is plotted against applied load in Figure 17. The location of the crack (highlighted with a red marker) is shown in Figure 18. In essence a single wide crack developed over the support, gradually growing in width as the load increased.

Flexural cracking in the positive moment regions at mid-span occurred in the vicinity of the spreader beam as loading progressed but remained fine until the onset of slip between the steel sheeting and the concrete at $P = 41.0$ kN. At this point, the positive moment flexural cracks became noticeably wider. Just after slip, the maximum crack width over the interior support
was 1.7 mm and the load had dropped sharply to 28 kN. Under increasing deformation, the load began to increase again and continued to increase until the peak load was reached at $P = 61.8$ kN. As the load approached the peak load, with significant slip of the decking taking place, there was partial interaction between the decking and the concrete, rather than full interaction.

After peak loading, slab deflection increased significantly as the slab unloaded. The test was terminated after the slab unloaded by about 30% of peak load.

![Figure 16: Load-deflection curve for slab S1-0-00.](image)

![Figure 17: Crack width at interior support for slab S1-0-00.](image)
The end slip between the profiled steel deck and the concrete was measured using LVDTs located at two different points at each end of the specimen. However, significant slip was recorded only at the left end of the slab, i.e. at the end of the span that suffered excessive deflection and the span that controlled the failure. The end slip at the end of the right span remained very small throughout the test.

The end slip at the end of the left span of specimen S1-0-00 is plotted against applied load $P$ in Figure 19. Initially, up until the load $P = 41$ kN, the measured slip was less than 0.04 mm. After first cracking occurred in the top of the slab over the interior support and before the onset of slip, cracking was observed in the positive moment region of each span, but these cracks remained fine until the sudden onset of slip occurred at $P = 41$ kN, and the load dropped to $P = 28$ kN as deformation increased. The initial slip of 0.97 mm occurred suddenly and was far in excess of the 0.1 mm slip which is often taken as the point where the longitudinal shear resistance is deemed to begin to fail. Slip between the steel deck and the concrete continued to increase as the applied load increased towards its peak value ($P = 61.8$ kN). The slip corresponding to the peak load was 7.6 mm.

The bending moment distribution along the specimen was calculated from the measured support reactions. The bending moment values for the left span are plotted against the total load in Figure 20a. The total load in Figure 20a is the actuator load, $P$, plus the self-weight of the slab and the self weight of the spreader beams and packing plates used in the test. Also shown in Figure 20a are the calculated bending moment assuming uniform stiffness along the member and linear elastic material behaviour. The sagging (positive) moments shown in Figure 20 are the values directly under the outside point load and were calculated from statics using the measured reaction at the exterior support at the left end of the specimen. The hogging (negative) moment at the interior support was also calculated for the left span using the measured reaction at the exterior support of that span.
Figure 19: End slip versus applied load for slab S1-0-00.

The sudden change in the moment distribution when the crack occurred at the top surface of the slab at the interior support is evident. Just before cracking, the total load was 47.0 kN and after cracking the total load dropped to 38.3 kN. After cracking, the stiffness of the negative moment region dropped significantly, as too did the negative bending moment at the interior support and the ratio of the magnitudes of negative to positive bending moment decreased substantially (see Figure 20c).

At first cracking over the interior support, at a deflection of just 1.57 mm, the negative moment dropped from -10.5 kNm to -1.8 kNm, as can be seen in the graph of moments versus mid-span deflection in Figure 20b, while the positive moment changed relatively little. The drop off in load at this point was almost entirely associated with the sudden drop in negative moment. After cracking in the negative moment region, despite the absence of any steel reinforcement (or fibres) in the concrete tensile zone, the decking alone was able to carry significant negative moment because of the deep trapezoidal ribs.

A further sudden change in the moment distribution occurred when positive moment cracking and high bond stresses initiated bond slip at the steel-concrete interface when the total load first reached 63.2 kN (and the deflection reached 8.33mm). At this point, the positive moment dropped suddenly from 14.1 kNm to 11.0 kNm, but the negative moment remained essentially constant, as can be seen in the graph of moments versus mid-span deflection in Figure 20b. The drop off in load at this point was almost entirely associated with the sudden drop in positive moment.
(a) Bending moments versus total loads.

(b) Bending moments versus mid-span deflection.

(c) Moment ratio versus total load.

Figure 20: Bending moment in Slab S1-0-00.
4.3 Slab Specimen S2-0-62

Slab S2-0-62 contained no steel fibres but was reinforced with SL62 welded wire mesh in the negative moment region over the interior support. SL62 mesh is a square mesh manufactured from 6.0 mm diameter wires spaced 200 mm apart in each direction. Four longitudinal wires ($A_s = 113 \text{ mm}^2$) were located at a depth 30 mm below the top surface of the slab (measured to the centre of the wires). The ratio of steel area to concrete area was $A_s/A_c = 0.00151$ and this is considered to be typical for a composite slab of this depth and span for normal live load in building construction.

In this test, the left span experienced significantly higher deflection than the right span and it was the left span that initiated final failure. Figure 21 shows the left span deflecting excessively under load while the right span has relatively little deflection.

![Image of Slab S2-0-62](Image)

**Figure 21:** Slab S2-0-62 after peak loading.

The actuator load $P$ versus deflection curve for this slab is shown in Figure 22. The peak load applied by the actuator to the specimen was $P = 70.0 \text{ kN}$ and the initial self-weight of the slab, spreader beam and packing was measured to be 18.7 kN. The applied load dropped slightly at the onset of the cracking in the top surface of the slab at the interior support at $P = 14 \text{ kN}$. However, the load recovered as deformation increased and continued to increase until slip occurred between the concrete and the decking in the left span at an applied load of $P = 55.5 \text{ kN}$. At this point, the load dropped significantly to $P = 44.0 \text{ kN}$.

Flexural cracking in the positive moment region occurred as the slip load was approached, but remained very fine until slip occurred. Further deformation saw the loads again increase gradually up to the peak load before unloading in the post-peak range as deflections became large. The mid-span deflection corresponding to the peak load was 33.0 mm. At the peak load, one of the wires in the low ductility SL62 welded wire mesh fractured resulting in a slight but sudden drop in the applied load. Other wires fractured in the post-peak range.
The equivalent uniformly distributed superimposed load corresponding to the applied load ($P$ plus the weight of the spreader beams and packing) at the onset of the cracking over the interior support was 4.59 kPa.

Crack widths were measured during the test by using a hand-held microscope with a magnification factor of 40x. The crack width at the top surface of the slab immediately after first cracking was 0.16 mm. The width of the crack over the interior support is plotted against the applied load in Figure 23. The location of the crack is illustrated in Figure 24. As for the unreinforced slab, a single crack developed and continued to widen as the test progressed and the load increased.

Flexural cracking in the mid-span region occurred as the load approached the slip load, but the cracks remained very fine and did not induce a sudden change in shape of the load deflection curve. At $P = 56$ kN, separation and slip between the steel deck and the concrete started to occur. The load dropped to 44 kN before it again started to increase toward the peak load. During this stage, partial interaction between the deck and the concrete occurred.

The post peak stage was characterised by large deflection and a decrease in the applied load. The test was terminated when the load dropped 30% below the peak load.

The slip between the profiled steel deck and the concrete at the left end of the specimen is plotted against the actuator load $P$ in Figure 25. The end slip at the end of the right span remained very small throughout the test and was less than 0.8 mm at the end of the test. The onset of the slip occurred at $P = 55.5$ kN and initiated a sudden drop in the applied load to $P = 44.0$ kN. Slip between the steel deck and the concrete continued to increase as the applied load increased towards its peak value. The slip corresponding to the peak load was 5.3 mm.
The bending moment values for the left span determined from the measured reactions are plotted against the total load in Figure 26a and the absolute value of -ve moment to +ve moment versus total load is shown in Figure 26b. The total load in Figure 26 is the actuator load \( P \) plus the self-weight of the slab, spreader beams and packing plates used in the test (i.e. \( P + 18.7 \text{ kN} \)). Also shown in Figure 26a are the calculated bending moment assuming uniform stiffness along the member and linear elastic material behaviour. The sagging (positive) moments shown in Figure 26a are the values directly under the outside point load and were calculated from statics using the measured reaction at the exterior support. The hogging (negative) moment at the interior support was also calculated for the left span using the measured reaction at the exterior support of that span.

![Graph](image)

**Figure 23:** Crack width versus applied load for Slab S2-0-62.

![Crack pattern](image)

**Figure 24:** Crack pattern at the negative moment region over interior support (S2-0-62).
Figure 25: Slip at the left end of S2-0-62 versus applied actuator load, $P$.

Figure 26: Bending Moment versus load for Slab Specimen S2-0-62.
The sudden change in the moment distribution when the crack occurred at the top surface of the slab at the interior support is evident, but due to the presence of the steel reinforcement, the change was not as dramatic as for the unreinforced slab S1-0-00 and there was little drop off in load. A further change in the moment distribution occurred when positive moment cracking initiated bond slip at the steel concrete interface when the total load first reached 75.8 kN. This resulted in a significant drop in load, a reduction in the positive moment and an increase in the magnitude of the negative moment.

4.4 Slab Specimen S3-20L-00

Slab S3-20L-00 contained 18.76 kg/m$^3$ of 60 mm long steel fibres. The target volume of the steel fibres for this slab specimen was 20 kg/m$^3$. In this test, the left span eventually failed and controlled the strength of the two-span slab as shown in Figure 27. The initial self-weight of the slab, spreader beam and packing was measured at 18.7 kN.

![Figure 27: Slab S3-20L-00 after failure.](image)

The actuator load $P$ versus mid-span deflection curve for the left span is shown in Figure 28. As for the slab containing mesh, the load dropped only slightly at the onset of cracking at the top surface of the slab over the interior support (at $P = 22.0$ kN) and then increased until slip occurred between the decking and the concrete in the left span at an actuator load of $P = 65$ kN. Flexural cracking in the positive moment region occurred as $P$ approached the slip load at $P = 65$ kN. Slip between the steel decking and the concrete resulted in a sudden drop in load from $P = 65$ kN (at a deflection of 12.8 mm) to $P = 51.8$ kN (at a deflection of 15.6 mm).
Further increases in deformation saw the load increase to the peak load of \( P = 82.8 \text{ kN} \) at a mid-span deflection of 49.0 mm. As deformation increased into the post-peak range the load steadily decreased.

The equivalent uniformly distributed superimposed load corresponding to the applied load (\( P \) plus the weight of the spreader beams and packing) at the onset of cracking over the interior support is 6.41 kPa.

Crack widths were measured by using a hand-held microscope. Similar to the previous test, first cracking occurred at the top surface of the slab at intermediate support region. The load that caused the first crack was found to be \( P = 22 \text{ kN} \) and the load dropped to \( P = 20.8 \text{ kN} \). At this point, a hairline crack of width 0.06 mm was recorded over the interior support. As the load increased slightly, the crack became wider and, at \( P = 22 \text{ kN} \), the crack width was 0.26mm.

The width of the top crack over the interior support is plotted against the applied load in Figure 29 and the crack pattern over the interior support can be seen in Figure 30. A single crack occurred across the slab at the intermediate support region at a relatively low load. As the load increased, a second crack occurred in the same vicinity as shown in Figure 30.

Flexural cracks occurred in the left span near the point of maximum positive moment under the spreader beam as load increased, but these cracks remained very fine until, at \( P = 64 \text{ kN} \), slip and separation occurred between the steel deck and the concrete.
Figure 29: Crack width versus applied load Slab S3-20L-00.

Figure 30: Cracks over interior support (S3-20L-00).

The slip at the left end of the specimen is plotted against the actuator load in Figure 31. Slip at the right end remained small before the peak load, but increased to 1.3 mm by the end of the test. The initial slip of 1.1 mm at the left end of the slab occurred suddenly when $P = 64$ kN. Slip continued to increase as the load increased. The slip at peak load was 6.8 mm.
Figure 31: End Slip versus applied load for Slab S3-20L-00.

(a) Bending moments versus total loads.

(b) Moment ratio versus total load.

Figure 32: Bending moment versus total load Slab S3-20L-00.
The bending moments in the left span are plotted against the total load in Figure 32a. The total load in Figure 32 is the actuator load plus the self-weight of the slab, spreader beams and packing plates used in the test and equals $P + 18.7 \text{ kN}$. Also shown in Figure 32a are the calculated bending moment assuming uniform stiffness along the member and linear elastic material behaviour. The sudden change in the moment distribution when the first crack occurred at the top surface of the slab at the interior support at a total load of 41.7 kN is evident, but as for the slab containing welded wire mesh, the drop in load at this point was modest. At the point where slip first occurred (when the total load was 82.7 kN), there was a significant drop in load, but the ratio of negative to positive moment did not change appreciably (as can be seen in Figure 32b).

4.5 Slab Specimen S4-30L-00

Slab S4-30L-00 contained 27.9 kg/m$^3$ of 60 mm length steel fibres. The target volume of steel fibre was 30 kg/m$^3$. In this test, the right span showed substantial deflection at mid-span and triggered the failure. Figure 33 shows that the right span failed under load while the left span exhibited far less deflection.

The load versus deflection curve is shown in Figure 34. As in the previous slabs, first cracking occurred in the top of the slab over the interior support when the actuator load was $P = 18.3 \text{ kN}$. The load dropped as the first crack occurred to $P = 17.3 \text{ kN}$ and then continued to increase until slip occurred between the decking and the concrete in the positive moment region at $P = 62.0 \text{ kN}$. At this point, the load dropped sharply to $P = 51.8 \text{ kN}$. Subsequent deformation saw the load increase to the peak load of $P = 92.8 \text{ kN}$ at a mid-span deflection of 56.0 mm and then relatively rapid unloading in the post-peak period.

![Figure 33: Slab S4-30L-00 after failure.](image-url)
Figure 34: Load vs deflection for S4-30L-00.

The equivalent uniformly distributed superimposed load corresponding to the applied load ($P$ plus the weight of the spreader beams and packing) at the onset of cracking over the interior support was 5.39 kPa.

The maximum crack width over the interior support versus applied load is shown in Figure 35 and the single large crack over the interior support at the end of the test is shown in Figure 36. Immediately after first cracking, the maximum width of the crack was 0.17 mm and the crack width steadily increased as deformation and load increased.

Slip between the decking and the concrete occurred at $P = 61.8$ kN. Flexural cracking in the positive moment region occurred prior to the slippage of the deck.

Figure 35: Crack width at the interior support for slab S4-30L-00.
The slip between the decking and the concrete in the right span is plotted against the applied load \( P \) in Figure 37. The initial slip of 0.98 mm at the right end of the slab occurred suddenly when \( P = 62 \) kN. Slip continued to increase as the load increased. The slip at peak load was 6.9 mm. Slip at the left end was 1.7 mm at peak load and was 1.8 mm by the end of the test.

The bending moment values for the right span are plotted against the total load in Figure 38a. The total load in Figure 38 is the actuator load plus the self-weight of the slab, spreader beams and packing plates used in the test and equals \( P + 17.9 \) kN. Also shown in Figure 38a
are the calculated bending moment assuming uniform stiffness along the member and linear elastic material behaviour. The sudden change in the moment distribution when the first crack occurred at the top surface of the slab at the interior support at a total load of 37.0 kN is evident, but as for S2-0-62 and S3-20L-00, the drop in load at this point was modest. At the point where slip first occurred (when the total load was 81.9 kN), there was a significant drop in load, but the ratio of negative to positive moment did not change appreciably (as shown in Figure 38b).

![Graph](image)

(a) Bending moments versus total loads.

![Graph](image)

(b) Moment ratio versus total load.

**Figure 38**: Bending moment versus total load for right span of slab S4-30L-00.

### 4.6 Slab specimen S5-30L-52

Slab S5-30L-52 contained a combination of wire-mesh and steel fibres. The measured volume of steel fibres in the slab was 30.6 kg/m³ (while the target volume was 30 kg/m³). SL52 mesh consists of 5.0 mm diameter steel wires spaced 200 mm apart in each direction. The ratio of the area of the longitudinal wires to the concrete area is $A_d/A_c = 0.00105$.

In this test, failure was initiated in the right span, as shown in Figure 39.
Figure 39: Slab S5-30L-52 after failure.

The actuator load versus deflection of the right span is shown in Figure 40. When cracking occurred over the interior support at $P = 30.5$ kN, the load did not drop appreciably, although there was a distinct change in the slope of the load-deflection plot. At $P = 66.8$ kN, slip occurred between the decking and the concrete in the right span and the positive bending flexural cracks opened significantly. After slip, the load continued to increase with increasing deformation up to the peak load of $P = 88.8$ kN. The mid-span deflection corresponding to the maximum load is 52.0 mm.

The equivalent uniformly distributed superimposed load corresponding to the applied load ($P$ plus the weight of the spreader beams and packing) at the onset of cracking over the interior support is 8.53 kPa.

Figure 40: Load-Deflection curve Slab S5-30L-52.
Similarly to the previous tests, first cracking occurred at the top surface of the slab at the interior support (at $P = 30.5$ kN). The initial crack width observed was 0.22 mm and the crack width steadily increased as deformation and load increased. The maximum crack width over the interior support versus applied load is shown in Figure 41. The crack pattern over the interior support at the end of the test is illustrated in Figure 42. A single crack occurred across the slab at $P = 31$ kN, but as the load increased, more cracks occurred at the same vicinity.

Slip between the decking and the concrete occurred at $P = 66.8$ kN. Flexural cracks in the positive moment region occurred prior to the slippage of the deck.

![Graph showing crack width versus applied load for slab S5-30L-52.](image)

**Figure 41:** Crack width versus applied load for slab S5-30L-52.

![Image showing crack pattern for slab S5-30L-52.](image)

**Figure 42:** Crack pattern for slab S5-30L-52.
The slip between the decking and the concrete in the right span is plotted against the applied load $P$ in Figure 43. The initial slip of 1.02 mm at the right end of the slab occurred suddenly when $P = 66.8$ kN and the load dropped immediately to 54.8 kN. Slip continued to increase as the load increased. The slip at peak load was 7.1. Slip at the left end was 1.9 mm at peak load and was 2.0 mm by the end of the test.

![Graph showing the relationship between applied load and end slip.](image)

**Figure 43:** End slip versus applied load $P$ for S5-30L-52.

The bending moment values for the right span are plotted against the total load in Figure 44. The total load in Figure 44 is the actuator load plus the self-weight of the slab, spreader beams and packing plates used in the test and equals $P + 19.5$ kN.

![Graph showing bending moments versus total load.](image)

**Figure 44:** Bending moments versus total load for S5-30L-52.
Also shown in Figure 44 are the calculated bending moment assuming uniform stiffness along the member and linear elastic material behaviour. The small reduction in the negative moment occurred at first cracking at the top surface of the slab at the interior support at a total load of 50.5 kN. At the point where slip first occurred (when the total load was 89.3 kN), there was a significant drop in load, which resulted in a drop in the positive moment but little change in the negative moment.

4.7 Slab Specimen S6-30L-82

Slab S6-30-82 contained both welded wire-mesh and steel fibres. The measured volume of 60 mm hooked end steel fibres was 27.9 kg/m³. The target steel fibre volume was 30 kg/m³. The SL82 mesh in the top of the slab over the interior support consisted of 8 mm diameter wires spaced 200 mm apart. The ratio of the area of the longitudinal wires to the concrete area is $A_d/A_c = 0.00269$. In this test, the right span deflected significantly more than the left span (as shown in Figure 45) and triggered the failure.

The actuator load $P$ versus deflection of the right span is shown in Figure 46. When cracking first occurred over the interior support at $P = 15.0$ kN, the load did not drop appreciably, although there was a distinct change in the slope of the load-deflection plot. At $P = 79.3$ kN, slip occurred between the decking and the concrete in the right span and the positive bending flexural cracks opened significantly. After slip, the load continued to increase with increasing deformation up to the peak load of $P = 98.5$ kN. The mid-span deflection corresponding to the maximum load is 38.1 mm.

![Figure 45: Slab S6-30L-82 after failure.](image)

The equivalent uniformly distributed superimposed load corresponding to the applied load ($P$ plus the weight of the spreader beams and packing) at the onset of cracking over the interior support is 4.31 kPa. The equivalent uniformly distributed superimposed load corresponding when the maximum crack width reached 0.3 mm was 12.3 kPa.
Figure 46: Load-deflection curve Slab S6-30L-82.

Similar to the previous tests, the first cracking occurred at the top surface of the slab at the interior support (at $P = 15.0$ kN). The initial crack width observed was 0.06 mm and the crack width steadily increased as deformation and load increased. The maximum crack width over the interior support versus applied load is shown in Figure 47, where the maximum crack width remained less than 0.3 mm until $P$ reached 50.0 kN. Initially a single crack occurred across the slab at the intermediate support region, but as load increase a second crack occurred in the same vicinity as shown in Figure 48.

Figure 47: Maximum crack width at the interior support for Slab S6-30L-82.

Slip between the decking and the concrete occurred at $P = 79.3$ kN. As for the other slabs, flexural cracks in the positive moment region occurred prior to the slippage of the deck.
The slip between the decking and the concrete in the right span is plotted against the applied load $P$ in Figure 49. The initial slip of 1.03 mm at the right end of the slab occurred suddenly when $P = 79.3$ kN and the load dropped immediately to 67.5 kN. Slip continued to increase as the load increased. The slip at peak load was 5.2. Slip at the left end was 0.3 mm at peak load and was 0.4 mm by the end of the test.

![Figure 49: End slip versus applied load for S6-30L-82.](image)

The bending moment values for the right span are plotted against the total load in Figure 50. The total load is the actuator load plus the self-weight of the slab, spreader beams and packing plates used in the test and equals $P + 16.5$ kN. Also shown in Figure 50 are the calculated bending moment assuming uniform stiffness along the member and linear elastic material behaviour. No sudden reduction in the negative moment was observed at first cracking at the top surface of the slab at the interior support. At the point where slip first
occurred (when the total load was 89.3 kN), there was a significant drop in load, which resulted in a drop in the positive moment but little change in the negative moment.

![Graph showing bending moment versus total load for Slab S6-30L-82.](image)

**Figure 50**: Bending moment versus total load Slab S6-30L-82.

### 4.8 Slab Specimen S7-40L-00

Slab S7-40L-00 contained 38.3 kg/m$^3$ of 60 mm long hooked-end steel fibres. The target volume was 40 kg/m$^3$. In this test, the right span deflected significantly more than the left span (as shown in Figure 51) and triggered the failure.

![Image of Slab S7-40L-00 after testing.](image)

**Figure 51**: Slab S7-40L-00 after testing.
The actuator load $P$ versus deflection of the right span is shown in Figure 52. When cracking first occurred over the interior support at $P = 15.0$ kN, the load did not drop appreciably, although there was a distinct change in the slope of the load-deflection plot. Fine positive moment cracks occurred in both spans as the load approached the slip load. At $P = 57.3$ kN, slip occurred between the decking and the concrete in the right span and the positive bending flexural cracks opened significantly. After slip, the load continued to increase with increasing deformation up to the peak load of $P = 82.8$ kN. The mid-span deflection corresponding to the maximum load is 43.0 mm.

The equivalent uniformly distributed superimposed load corresponding to applied load ($P$ plus the weight of the spreader beams and packing) at the onset of cracking over the interior support is 5.08 kPa.

![Load-deflection curve for Slab S7-40L-00.](image)

Figure 52: Load-deflection curve for Slab S7-40L-00.

As for the previous tests, the first cracking occurred at the top surface of the slab at the interior support (at $P = 15.0$ kN). The initial crack width observed was 0.12 mm and the crack width steadily increased as deformation and load increased. The maximum crack width over the interior support versus applied load is shown in Figure 53. A single crack occurred across the slab at the intermediate support region as shown in Figure 54.

Slip between the decking and the concrete occurred at $P = 57.3$ kN. Flexural cracks in the positive moment region occurred prior to slippage of the deck.

The slip between the decking and the concrete in the right span is plotted against the applied load $P$ in Figure 55. The initial slip of 0.74 mm at the right end of the slab occurred suddenly when $P = 57.3$ kN and the load dropped immediately to 47.8 kN. Slip continued to increase as the load increased. The slip at peak load at the right hand end was 6.2 mm. Slip at the left end was 1.1 mm at peak load and was 1.2 mm by the end of the test.
Figure 53: Crack width versus applied load at the intermediate support for Slab S7-40L-00.

Figure 54: Crack pattern over interior support (S7-40L-00).

Figure 55: End slip versus applied load (S7-40L-00).
The bending moment values for the right span are plotted against the total load in Figure 56. The total load is the actuator load plus the self-weight of the slab, spreader beams and packing plates used in the test and equals \( P + 19.8 \) kN. Also shown in Figure 56 are the calculated bending moment assuming uniform stiffness along the member and linear elastic material behaviour. A reduction in the negative moment was observed at first cracking at the top surface of the slab at the interior support (when the total load was 34.8 kN). At the point where slip first occurred (when the total load was 77.1 kN), there was a significant drop in load, which resulted in a drop in the positive moment but little change in the negative moment.

![Figure 56: Bending moment versus total load (S7-40L-00).](image)

### 4.9 Slab Specimen S8-40S-00

Slab S8-40S-00 contained 40.2 kg/m\(^3\) of 35 mm long (short) hooked-end steel fibres. The target volume was 40 kg/m\(^3\). In this test, the left span deflected significantly more than the right span (as shown in Figure 57) and triggered the failure.

The actuator load \( P \) versus deflection of the right span is shown in Figure 58. When cracking first occurred over the interior support at \( P = 17.0 \) kN, the load did not drop appreciably, although there was a change in the slope of the load-deflection plot. Fine positive moment cracks occurred in both spans as the load approached the slip load. At \( P = 82.0 \) kN, slip occurred between the deck and the concrete in the left span and the positive bending flexural cracks opened significantly. After slip, the load decreased to \( P = 67.5 \) kN and then continued to increase with increasing deformation up to the peak load of \( P = 95.8 \) kN. The mid-span deflection corresponding to the maximum load is 45.0 mm.
The equivalent uniformly distributed superimposed load corresponding to the applied load ($P$ plus the weight of the spreader beams and packing) at the onset of cracking over the interior support is 5.29 kPa.

![Image of slab S8-40S-00 after test.](image1)

**Figure 57:** Slab S8-40S-00 after test.

As for the previous tests, the first cracking occurred at the top surface of the slab at the interior support (at $P = 17.0$ kN). The initial crack width observed was 0.16 mm and the crack width steadily increased as deformation and load increased. The maximum crack width over the interior support versus applied load is shown in Figure 59. A single crack occurred across the slab at the intermediate support region as shown in Figure 60.

Slip between the decking and the concrete occurred at $P = 82$ kN. Flexural cracks in the positive moment region occurred prior to slippage of the deck.

![Load-deflection curve S8-40S-00.](image2)

**Figure 58:** Load-deflection curve S8-40S-00.
Figure 59: Maximum crack width over interior support versus applied load (S8-40S-00).

Figure 60: Crack pattern over interior support (S8-40S-00).

The slip between the decking and the concrete in the left span is plotted against the applied load $P$ in Figure 61. The initial slip of 1.19 mm at the left end of the slab occurred suddenly when $P = 82.0$ kN and the load dropped immediately to 67.8 kN. Slip continued to increase as the load increased. The slip in the left span at peak load was 6.2 mm.

This specimen behaved somewhat differently to the others with regard to slip. A slip of about 0.2 mm suddenly occurred in the right span at a load of $P = 72$ kN (before any slip had occurred in the left failure-span), but this did not trigger an appreciable drop in load. Slip at the right hand end had increased to 0.55 mm when first slip occurred at the left hand (at $P = 82.0$ kN). At peak load the slip at the right end was 1.9 mm at peak load and was still 1.9 mm by the end of the test.
Figure 61: End slip versus applied load $P$ for S8-40S-00.

The bending moment values for the left span are plotted against the total load in Figure 62. The total load is the actuator load plus the self-weight of the slab, spreader beams and packing plates used in the test and equals $P + 18.7$ kN. Also shown in Figure 62 are the calculated bending moment assuming uniform stiffness along the member and linear elastic material behaviour. No sudden change in the negative moment was observed at first cracking at the top surface of the slab at the interior support. At the point where slip first occurred (when the total load was 101 kN), there was a significant drop in load, which resulted in a drop in the positive moment but little change in the negative moment.

Figure 62: Bending Moment versus total load slab S8-40S-00.
5.0 DISCUSSION

5.1 Load versus Deflection

All slabs tested exhibited similar load-deflection behaviour. Values of load and deflection at the mid-point of the span that failed are given in Table 6 at critical load levels. In Figure 63, the load-deflection response of the plain concrete composite slab (S1-0-00) is compared to that of the slab containing only SL62 mesh (S2-0-62). In Figure 64, the load-deflection response of S1-0-00 is compared to that of the slab containing 20 kg/m³ of 60 mm long end-hooked fibres (S3-20L-00).

Table 6: Key values of applied load (P) and mid-span deflection (Δ).

<table>
<thead>
<tr>
<th>Slab Specimen</th>
<th>At first cracking</th>
<th>At slip between decking and concrete</th>
<th>At Peak Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Just before P (kN)</td>
<td>Δ (mm)</td>
<td>Just after P (kN)</td>
</tr>
<tr>
<td>S1-0-00</td>
<td>24.7</td>
<td>1.2</td>
<td>16.0</td>
</tr>
<tr>
<td>S2-0-62</td>
<td>14.0</td>
<td>0.7</td>
<td>11.8</td>
</tr>
<tr>
<td>S3-20L-00</td>
<td>22.0</td>
<td>1.6</td>
<td>20.0</td>
</tr>
<tr>
<td>S4-30L-00</td>
<td>18.8</td>
<td>1.4</td>
<td>17.3</td>
</tr>
<tr>
<td>S5-30L-52</td>
<td>30.5</td>
<td>2.2</td>
<td>29.0</td>
</tr>
<tr>
<td>S6-30L-82</td>
<td>15.0</td>
<td>0.8</td>
<td>14.3</td>
</tr>
<tr>
<td>S7-40L-00</td>
<td>15.0</td>
<td>1.2</td>
<td>14.3</td>
</tr>
<tr>
<td>S8-40S-00</td>
<td>17.0</td>
<td>1.1</td>
<td>17.0</td>
</tr>
</tbody>
</table>

At first loading, the load increased linearly with deflection until the first crack occurred at the top surface of the slab over the interior support. At this point, the sudden reduction of stiffness resulted in a drop in the applied load and a change in the slope of the load deflection curve (Point A on each of the curves in Figures 63 and 64). The magnitude of the drop in load at first cracking depended on the amount of reinforcement or fibres in the negative moment region over the interior support. The plain concrete slab (S1-0-00) suffered a significant drop in load (from $P = 24.7$ kN to $P = 16.0$ kN), while for both S2-0-62 and S3-20L-00 the drop in load was much smaller.

As the tests continued, the load increased at a reduced stiffness towards point B on the curves in Figures 63 and 64. At this stage, flexural cracks developed in the positive moment regions in both spans, but due to the restraint provided by the decking, these bottom cracks remained fine and the loss of stiffness was relatively small (as is evidenced by the almost linear load-deflection response of all slabs from point A to point B on the curves in Figures 63 and 64). The slope of the curve from A to B for the slab containing mesh (S2-0-62) is significantly higher than that for slab S1-0-00 and also for the slab containing fibres (S3-20L-00).
Figure 63: Load-deflection curves for S1-0-00 and S2-0-62.

Figure 64: Load-deflection curves for S1-0-00 and S3-20L-00.

At point B on each curve in Figures 63 and 64, slip occurred between the concrete and the deck and the load dropped suddenly. From Figure 63, the load at which slip occurred was increased by the presence of the welded wire mesh over the interior support, but there was no significant change in the deflection at which slip first took place. However, from Figure 64,
the inclusion of just 20 kg/m³ of 60 mm fibres led to a larger increase in load and a significant increase in the deflection at which slip first takes place.

The peak load was significantly increased by the inclusion of both mesh and fibres, but the inclusion of 20 kg/m³ of 60mm fibres led to a larger increase in peak load than that provided by the SL62 mesh. Significantly, the inclusion of low-ductility welded wire mesh reduced the deflection at which the peak load occurred (compared to the slab without mesh) and also reduced the post-peak deformability. By contrast, the inclusion of fibres not only increased the peak load but also increased the deformability of the slab.

The load-deflection responses of the slabs containing various quantities of 60 mm end-hooked fibres (without welded wire mesh) are compared in Figure 65. The behaviour of the slabs with 30 kg/m³ and 40 kg/m³ of fibres are similar to that of the slab containing 20 kg/m³. Relatively little appears to be gained in terms of both strength and deformability by increasing the dosage of fibres above 20 kg/m³. All slabs containing fibres exhibited very significant increases in both slip load and peak load compared to the slab without fibres (S1-0-00) and all exhibited significantly more deformability.

![Figure 65: Load versus deflection for slabs with 60 mm fibres (0, 20, 30 and 40 kg/m³).](image)

The effect on the load-deflection response of increasing the amount of welded wire reinforcement over the interior support for the three slabs containing 30 kg/m³ of 60mm fibres is shown in Figure 66. Whilst the mesh significantly increases the post-cracking stiffness of the slab prior to slip, it does little to improve the strength of the slab (i.e. increase the peak load). Increasing the amount of low-ductility reinforcement over the interior support appears to decrease the deformability of the slab, and hence reduces its ductility.
Figure 66: Load versus deflection for slabs with both steel fibres and welded wire mesh.

A comparison between the load-deflection responses of S7-40L-00 and S8-40S-00 is shown in Figure 67. Whilst it is impossible to draw definitive conclusions from just two tests, the slab containing the 35mm (short) fibres had a significantly higher slip load and a significantly higher peak load than the slab containing the same quantity of 60 mm (long) fibres. The deflection at peak load and the deformability of each slab were similar.

Figure 67: Load versus deflection for slabs with 35mm and 60mm steel fibres.
5.2 Maximum Crack Width in Top of Slab over Interior Support

One of the aims of the test program was to investigate the suitability of steel fibres as a replacement for welded wire fabric in the negative moment region over the interior support and to assess the ability of steel fibres to effectively control cracking over the support by limiting the maximum crack width to an acceptable small value.

Figure 68 shows the effect of either SL62 mesh or 20 kg/m$^3$ of long fibres in reducing maximum crack widths. Without mesh (or fibres), the initial crack width in S1-0-00 was unacceptably large and the crack opened widely as deformation increased. With SL62 mesh (S1-0-62), the maximum crack width was much smaller, but the crack width became excessive (exceeding about 0.3 mm) at about 50% of the slip load. The fibres (S3-20L-00) provided a similar degree of crack control to the mesh at service load levels where maximum crack widths were acceptably small.

![Graph showing applied load vs. maximum crack width](image)

**Figure 68**: Maximum crack width versus applied load for S1-0-00, S2-0-62 and S3-20L-00.

A comparison of the maximum crack widths over the interior support for the specimens with various quantities of 60 mm end-hooked steel fibres (without any welded wire mesh) is shown in Figure 69. The addition of steel fibres reduced the maximum crack width significantly at typical service loads. However, it appears that the addition of 20 kg/m$^3$ is sufficient to improve crack control and little benefit is gained from higher doses of fibres. The slope of the load-crack width response for each dosage of fibres is approximately the same and the relationship is almost linear. As was the case for the slab with SL62 mesh and no fibres, the maximum crack for each of the slabs containing fibres started to become excessive (exceeding about 0.3 mm) at about 50% of the slip load.
Figure 69: Maximum crack width versus applied load for slabs with steel fibres.

A comparison of the maximum crack widths over the interior support for the specimens with various quantities of welded-wire fabric and containing 30 kg/m$^3$ of 60 mm end-hooked steel fibres is shown in Figure 70. When fibres are combined with mesh, even for very small quantities of reinforcing mesh (the ratio of steel area to concrete area for SL52 mesh is only $A_s/A_c = 0.00105$), the maximum crack width over the interior support remains less than 0.3 mm; well beyond normal service load levels. The initial crack widths are very small and maximum crack widths remain acceptably small at loads less than about 60% of the slip load of the slab.

Figure 70: Maximum crack width versus applied load for SFRC slabs with and without welded wire mesh.
The maximum crack widths over the interior support for the specimens with 40 kg/m³ of 60 mm end-hooked steel fibres (S7-40L-00) and 40 kg/m³ of 35 mm end-hooked steel fibres (S8-40S-00) are shown in Figure 71. There appears to be no significant difference in the effectiveness of the different types of fibres for controlling top surface cracks over the intermediate support.

![Graph showing maximum crack width versus applied load](image)

**Figure 71:** Maximum crack width versus applied load for slabs containing 40 kg/m³ of 35 mm (S) and 60 mm (L) steel fibres.

The applied load at first cracking over the interior support and the corresponding maximum crack width for each slab are shown in Table 7, together with the equivalent uniformly distributed live load. Also shown in Table 7 is the applied load when the maximum crack width reached 0.3 mm and the equivalent uniformly distributed live loads at this point and at the peak load.

**Table 7:** Equivalent uniformly distributed imposed loads at first cracking, when the maximum crack width reaches 0.3 mm and at the slip load.

<table>
<thead>
<tr>
<th>Slab ID</th>
<th>Weight of loading plates and spreader beam sb (kN)</th>
<th>Applied load at first crack $P + sb$ (kN)</th>
<th>Initial crack width (mm)</th>
<th>Equivalent uniform load at first cracking (kPa)</th>
<th>Applied load when max. crack width 0.3 mm $P + sb$ (kN)</th>
<th>Equivalent uniform load at 0.3 mm max. crack width (kPa)</th>
<th>Equivalent uniform load at the slip load (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1 - 0 - 00</td>
<td>8.5</td>
<td>33.2</td>
<td>0.62</td>
<td>7.59</td>
<td>-</td>
<td>-</td>
<td>11.3</td>
</tr>
<tr>
<td>S2 - 0 - 62</td>
<td>6.1</td>
<td>20.1</td>
<td>0.16</td>
<td>4.59</td>
<td>30.1</td>
<td>6.87</td>
<td>14.1</td>
</tr>
<tr>
<td>S3 - 20L - 00</td>
<td>6.1</td>
<td>28.1</td>
<td>0.06</td>
<td>6.41</td>
<td>29.1</td>
<td>6.64</td>
<td>16.2</td>
</tr>
<tr>
<td>S4 - 30L - 00</td>
<td>5.3</td>
<td>23.6</td>
<td>0.18</td>
<td>5.39</td>
<td>30.3</td>
<td>6.92</td>
<td>15.3</td>
</tr>
<tr>
<td>S5 - 30L - 52</td>
<td>6.9</td>
<td>37.4</td>
<td>0.22</td>
<td>8.53</td>
<td>44.9</td>
<td>10.2</td>
<td>16.8</td>
</tr>
<tr>
<td>S6 - 30L - 82</td>
<td>3.9</td>
<td>18.9</td>
<td>0.06</td>
<td>4.31</td>
<td>53.9</td>
<td>12.3</td>
<td>19.0</td>
</tr>
<tr>
<td>S7 - 40L - 00</td>
<td>7.2</td>
<td>22.2</td>
<td>0.12</td>
<td>5.08</td>
<td>28.2</td>
<td>6.45</td>
<td>14.7</td>
</tr>
<tr>
<td>S8 - 40S - 00</td>
<td>6.1</td>
<td>23.1</td>
<td>0.16</td>
<td>5.29</td>
<td>25.1</td>
<td>5.75</td>
<td>20.1</td>
</tr>
</tbody>
</table>
For all slabs containing fibres and/or mesh, cracks were controlled at floor live load levels typical of those found in building structures for most occupancies. No consideration has been given in this study to the likelihood (or otherwise) of an increase in crack widths with time due to long-term drying shrinkage.

5.3 Redistribution of Bending Moments

The bending moment values for the failure span of S1-0-00 plotted against the total load in Figure 20 are reproduced here in Figure 72a. The sudden change in the moment distribution when the first crack occurred at the top surface of the slab at the interior support is evident (at A in Figure 72). Just before cracking, the total load was 47.0 kN and after cracking the total load dropped to 38.3 kN, the stiffness of the negative moment region dropped significantly, as too did the negative bending moment at the interior support, and the ratio of the magnitudes of negative to positive bending moment decreased substantially (see Figure 72b).

(a) Bending moments versus total loads.

(b) Moment ratio versus total load.

Figure 72: Bending moments in Slab S1-0-00.
At first cracking over the interior support, the negative moment dropped from -10.5 kNm to -1.8 kNm, while the positive moment changed relatively little. The drop off in load at this point was almost entirely associated with the sudden drop in negative moment. After cracking in the negative moment region, despite the absence of any steel reinforcement (or fibres) in the concrete tensile zone, the decking alone was able to carry significant negative moment because of the deep trapezoidal ribs.

A further sudden change in the moment distribution occurred when sudden bond slip at the steel concrete interface suddenly took place when the total load first reached 63.2 kN. At this point (Point B in Figure 72), the positive moment dropped suddenly from 14.1 kNm to 11.0 kNm, but the negative moment remained essentially constant, as can be seen in Figure 72b. The drop off in load at this point was almost entirely associated with the sudden drop in positive moment.

The effect of including welded wire mesh on bending moments in the failure span of S2-0-62 can be seen by comparing the bending moments in Figure 73 with those in Figure 72.

![Graph](image_url)

(a) Bending moments versus total loads.

![Graph](image_url)

(b) Moment ratio versus total load.

**Figure 73**: Bending Moment versus load for Slab S2-0-62.
The sudden change in the moment distribution when the crack occurred at the top surface of the slab at the interior support is evident, but due to the presence of the steel reinforcement, the change was not as dramatic as for the unreinforced slab (S1-0-00) and there was little drop off in load. A further sudden change in the moment distribution occurred when positive moment cracking eventually initiated bond slip at the steel concrete interface when the total load first reached 75.8 kN.

The effects of including steel fibres on the redistribution of moments are shown in Figure 74 where the moment versus total load response of S3-20L-00 is shown. For this slab the total load is the actuator load plus the self-weight of the slab, spreader beams and packing plates used in the test and equals $P + 18.7$ kN. The sudden change in the moment distribution when the first crack occurred at the top surface of the slab at the interior support at a total load of 41.7 kN is evident, but as for the slab containing welded wire mesh, the drop in load at this point was modest. At the point where slip first occurred (when the total load was 82.7 kN), there was a significant drop in load, but the ratio of negative to positive moment did not change appreciably (as can be seen in Figure 74b). Similar results were obtained for S4-30L-00, S7-40L-00 and S3-40S-00.

![Graph](image)

(a) Bending moments versus total loads.

![Graph](image)

(b) Moment ratio versus total load.

**Figure 74:** Bending moment versus total load Slab S3-20L-00.
The effects on the redistribution of bending moments of the inclusion of both mesh and fibres is illustrated in Figure 75, where the moment versus total load response of S6-30L-82 is shown. In contrast with slabs containing either mesh or fibres, no sudden reduction in the negative moment was observed at first cracking at the top surface of the slab at the interior support. At the point where slip first occurred, the behaviour was similar to the other slabs with a significant drop in load and a significant drop in the positive moment.

![Graph showing bending moment versus total load for Slab S6-30L-82](image)

**Figure 75:** Bending moment versus total load Slab S6-30L-82.

### 6.0 OBSERVATIONS AND CONCLUSIONS

A series of short-term static load tests on two-span composite slabs has been described and the results have been presented. In total, eight two-span composite slab specimens were cast and moist cured for a period of 14 days and subsequently loaded to failure. In addition to the steel decking, one of the specimens contained no reinforcing steel and no steel fibres, four of the specimens were reinforced only with steel fibres in the concrete (with nominal fibre contents of either 20, 30 or 40 kg/m³). In the other three specimens welded wire mesh was included over the interior support, one with plain concrete and two with steel fibres in the concrete.

Based on the limited number of tests conducted, the following observations are made:

1. Compared to the slab containing SL62 mesh over the interior support (without fibres), all slabs containing steel fibres had a higher slip load and a higher peak load. In addition, the deflections at both the peak load and the slip load were greater for the slabs containing fibres.
2. The addition of 20 kg/m³ of 60 mm long end-hooked steel fibres increased the slip load from \( P = 41 \text{ kN} \) (for S1 - 0 - 00) to \( P = 65 \text{ kN} \) (for S3 - 20L - 00).

3. The addition of 20 kg/m³ of 60 mm long end-hooked steel fibres increased the peak load from \( P = 61.8 \text{ kN} \) (for S1 - 0 - 00) to \( P = 82.8 \text{ kN} \) (for S3 - 20L - 00).

4. The inclusion of 20 kg/m³ of 60 mm long end-hooked steel fibres increases the ratio of slip load to peak load, from 63% (for S1 - 0 - 00) to 75% (for S3 - 20L - 00).

5. Notwithstanding the above observations and allowing for variations in fibre distribution, a dosage of fibres of between 20 and 30 kg/m³ is recommended. In terms of both the slip load and the peak load, relatively little benefit appears to be gained by increasing the fibre content above 20 kg/m³.

6. The current practice of providing SL62 mesh over the negative moment region provided crack control (maximum crack widths of 0.3 mm) up to about 50% of the slip load. The same was true for the slabs containing only fibres. However, at loads above 50% of the slip load, the mesh was more effective at limiting the maximum crack width.

7. Crack control was most effective for the slabs containing both mesh and fibres.

8. In terms of maximum crack widths, relatively little benefit was gained by increasing the fibre content above 20 kg/m³.

9. The use of 35 mm long fibres (Dramix RC65/35BN), compared to a similar volume of 60 mm long fibres (Dramix RC80/60 BN), increased both the slip load and the peak load, but had little effect on the maximum crack width at similar load levels.

10. The inclusion of steel fibres (as an alternative to the use of SL62 mesh over the interior supports) is an effective measure to increase both the slip load and the peak load of continuous composite slabs. Under typical service loads, cracking over the interior support will be controlled for most applications.

Where a strong degree of crack control is required for the top surface of the slab, a potentially effective measure to gain the benefits of fibres without the use of reinforcing mesh would be to use a saw-cut over the interior support. This will promote the top crack to occur directly under the saw cut and will effectively hide the crack from view on the top surface. In this way, the benefits of fibres in increasing the slip load and the peak load are gained, and the problems of unsightly cracking are eliminated.

The results indicate that the inclusion of steel fibres in excess of 20 kg/m³ provides significant advantages in terms of both the load at which slip between the concrete and the steel decking occurs and the peak load, as well as providing acceptable crack control at service loads.
7.0 REFERENCES


APPENDIX I

Determination of Fibre Content:

The fibre content was determined for each concrete mix, before the slab was cast, during the concrete pour and/or after the concrete pour. The methodology adopted to determine the fibre content of each concrete mix was as follows:

1. A 300 mm high by 150 mm diameter cylinder was filled with fresh SFRC;
2. The contents of the cylinder was then emptied into a fine sieve;
3. The cement and fine particles were then “washed out” of the mix, leaving the fibres and coarse aggregate particles;
4. Using a magnet, the fibres were extracted;
5. A hand and visual check was made to ensure that all of the fibres were extracted;
6. The fibres were then dried and weighted using a precision scale;
7. The weight of fibres per cubic metre of concrete was then determined by dividing the measured weight of fibres (in kg) by the known volume of the cylinder (0.005301 m³).

Table A-1 provides the detailed results of the wash-out tests for each concrete mix.

<table>
<thead>
<tr>
<th>Slab Specimen</th>
<th>Test</th>
<th>Target Dosage kg/m³</th>
<th>Wt. of fibres per cyl grams</th>
<th>Stage of pour sample taken</th>
<th>Actual Dosage per Sample kg/m³</th>
<th>Actual Average per Sample kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-20L-00</td>
<td>S20-65/60</td>
<td>20</td>
<td>98.8</td>
<td>start</td>
<td>18.64</td>
<td>18.76</td>
</tr>
<tr>
<td></td>
<td>S20-65/60</td>
<td>20</td>
<td>112.5</td>
<td>start</td>
<td>21.21</td>
<td>18.76</td>
</tr>
<tr>
<td></td>
<td>S20-65/60</td>
<td>20</td>
<td>91.4</td>
<td>middle</td>
<td>17.24</td>
<td>18.76</td>
</tr>
<tr>
<td></td>
<td>S20-65/60</td>
<td>20</td>
<td>104.1</td>
<td>middle</td>
<td>19.64</td>
<td>18.76</td>
</tr>
<tr>
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<td>Wt. of fibres per cylinder grams</td>
<td>Stage of pour sample taken</td>
<td>Actual Dosage per Sample kg/m³</td>
<td>Actual Average per Sample kg/m³</td>
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</table>
APPENDIX 2

Direct Tensile Tests

Details of the specimen size and test set-up for the tensile stress versus crack opening displacement (COD) tests are shown in Figure A2-1. The specimen thickness was 30 mm. The specimens were cast horizontally in one pour into timber moulds. Several specimens were prepared for each different steel fibre concentration. Demoulding the specimens took place the day after casting and the specimens were then cured alongside the slab specimens until the day of testing.

The direct tension specimens were tested in an Instron Universal testing machine under displacement control and with the load measured via a 100 kN load cell. One end of the specimen was fixed-ended and the other pin-ended, with the load applied via a universal joint. The pinned-end was to remove any accidental eccentricity that may result from the alignment of the specimen in the testing machine. The raw test results for the SFRC specimens are plotted in Figures A2-2 to A2-5.

As the specimens are thin (30 mm) the boundary effect on the residual tensile strength of the SFRC is significant (Figure A2-5) and compensation must be made for the influence this effect. To convert the data to that for the general case where any fibre has an equal probability of being freely orientated in 3D space, the tensile stress for a given COD is calculated as:

\[ f_{COD} = k_b f_{residual} \]  \hspace{1cm} (A.1)

where \( f_{COD} \) is the residual tensile stress for a given COD, \( k_b \) is the boundary effect influence factor and \( f_{residual} \) is the measured post-cracking residual tensile stress for a given COD. For the Dramix 80/60 and 65/35 fibres used in this study, the boundary effect influence factors are calculated from Lee et al. (2011) as \( k_b = 0.685 \) and \( k_b = 0.758 \), respectively.

An idealized tensile stress versus COD is shown in Figure A2-6. The results obtained from the measured data, compensated for the boundary effect using Eq. A.1, are given in Table A2-1.
Table A2-1: Tensile strength of SFRC specimens.

<table>
<thead>
<tr>
<th>Weight of fibres (No. of Specimens) (kg/m³)</th>
<th>Fibre type</th>
<th>$f'_c$ (MPa)</th>
<th>$k_b$</th>
<th>$f_{cr}$ (MPa)</th>
<th>$f_{0.5}$ (MPa)</th>
<th>$f_{1.5}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (3)</td>
<td>-</td>
<td>47.7</td>
<td>-</td>
<td>4.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>20 (2)</td>
<td>80/60</td>
<td>43.5</td>
<td>0.685</td>
<td>3.5</td>
<td>0.44</td>
<td>0.55</td>
</tr>
<tr>
<td>30 (4)</td>
<td>80/60</td>
<td>45.3</td>
<td>0.685</td>
<td>3.8</td>
<td>0.88</td>
<td>0.78</td>
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<tr>
<td>40 (2)</td>
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<td>42.8</td>
<td>0.685</td>
<td>2.7</td>
<td>1.5</td>
<td>1.3</td>
</tr>
<tr>
<td>40 (2)</td>
<td>65/35</td>
<td>57.8</td>
<td>0.758</td>
<td>4.0</td>
<td>0.98</td>
<td>0.63</td>
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</table>

Figure A2-1: Uniaxial direct tensile test arrangement.
Figure A2-2: Tensile stress versus crack opening displacement for concrete with steel-fibre content 20kg/m³ (S3-20L-00).

Figure A2-3: Tensile stress versus crack opening displacement for concrete with steel-fibre content 30kg/m³ (Specimens 1 and 2: from S3-30L-00 and S6-30L-82; Specimens 3 and 4 from S5-30L-52).
Figure A2-4: Tensile stress versus crack opening displacement for concrete with steel-fibre content 40kg/m³ (S7-40L-00).

Figure A2-5: Tensile stress versus crack opening displacement for concrete with steel-fibre content 40kg/m³ (S8-40S-00).
Figure A2-5: Boundary influence on the post-cracking residual tensile strength of SFRC.

Figure A2-6: Key measurement points for uniaxial tensile stress versus crack opening displacement (COD) for SFRC.