STRENGTH AND DUCTILITY OF CORNER SUPPORTED TWO-WAY CONCRETE SLABS CONTAINING WELDED WIRE FABRIC

by

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UNICIV REPORT No. R-453 OCTOBER 2008
THE UNIVERSITY OF NEW SOUTH WALES
SYDNEY 2052 AUSTRALIA
http://www.civeng.unsw.edu.au

ISBN: 85841 420 1
Title:  Strength and Ductility of Corner Supported Two-way Concrete Slabs Containing Welded Wire Fabric

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Keywords: Corner-supported two-way slab; Ductility; Laboratory experiments; Low ductility reinforcement; Moment redistribution; Reinforced concrete; Slabs; Strain localization; Strength; Welded wire fabric.

Abstract:
This report forms part of an on-going research project at the University of New South Wales on the strength and ductility of reinforced concrete slabs containing low ductility reinforcement in the form of Class L welded wire fabric (WWF). A series of full range load tests is described on two-way, corner-supported reinforced concrete slab panels containing either Class L WWF or Class N deformed bars. Eleven slab panels, either square or rectangular, were subjected to transverse loads applied by a deformation controlled actuator in a stiff testing frame. The slabs were instrumented with load cells, strain gauges, linear variable displacement transducers, and lasers to measure applied forces and reactions, strains in the steel reinforcement and on the concrete surfaces, and deflections at all stages of loading. The results of the tests are presented and evaluated, with particular emphasis on the strength, ductility and failure mode of the slabs.

Slabs with Class L WWF failed in flexure in a brittle and sudden manner by fracture of the reinforcement in one direction with little plastic deformation and little prior warning of failure. By contrast, the slabs containing Class N deformed bar underwent very significant plastic deformation prior to gradual unloading under increasing deformation. The results confirm that corner-supported two-way slabs containing Class L reinforcement do not have the ductility necessary to justify the usual assumptions made in design and analysis of under-reinforced concrete slabs. The results also support the recent amendment to the Australian Standard for Concrete Structures, AS3600-2001 involving a reduction of the strength reduction factor by 20% when Class L reinforcement is used in suspended slabs (ie. $\phi = 0.64$ for Class L reinforcement and $\phi = 0.8$ for Class N reinforcement).
Strength and Ductility of Corner Supported Two-way Concrete Slabs containing Welded Wire Fabric

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Key Words

Corner-supported two-way slab, ductility, experiments, low-ductility reinforcement, moment redistribution, reinforced concrete, slabs, strain localization, welded wire fabric.
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1. Introduction

Ductility is a measure of the ability of a structural element or system to sustain plastic deformations before collapse without substantial loss of load resistance. Ductility is an essential property of concrete structures and a necessary requirement to justify the usual assumptions made in their analysis and design. Ductility allows for redistribution of internal actions from highly stressed areas to less stressed regions in statically indeterminate structures and allows the structure to develop the load path assumed in design. Ductile structures can develop the full strength of the critical sections considered in design, while brittle structures may not be able to do so. Ductile structures suffer relatively large deformations before failure and this provides warning of impending collapse. Ductility also provides robustness and helps in dissipating the internal energy generated by impact loading, such as earthquake actions and dynamic and blast loadings.

The trend in the construction industry to provide more cost effective materials has led to the use of higher strength reinforcing steel and concrete. Unfortunately, these materials have had an adverse impact on the ductility of reinforced concrete structures. In Australia, reinforcing steels are classified in AS/NZS4671 according to their ductility - either Class N (normal ductility) or Class L (low ductility). For each class of reinforcement, minimum limits are set for the strain at peak stress (or uniform elongation, $\varepsilon_{ru}$) and the ratio of tensile strength to yield stress ($f_{tu}/f_{sy}$). For Class L steel, $\varepsilon_{ru} \geq 1.5\%$ and $f_{tu}/f_{sy} \geq 1.03$. These limits are considerably lower than the corresponding limits set in Eurocode 2 for the lowest ductility steel permitted in Europe (Class A).

To date, little research has been directed at the strength and ductility of two-way slabs containing Class L reinforcement and no experimental test results have been reported in the literature.

This report forms part of an on-going research project at the University of New South Wales funded by the Australian Research Council on the strength and ductility of reinforced concrete slabs containing low ductility reinforcement in the form of Class L welded wire fabric (WWF). A series of full range load tests is described on two-way, corner-supported reinforced concrete slab panels containing either Class L WWF or Class N deformed bars. Eleven slab panels, either square or rectangular, were subjected to transverse loads applied by a deformation controlled actuator in a stiff testing frame. The slabs were instrumented with:

(i) load cells – to measure applied forces and reactions;
(ii) linear variable displacement transducers and lasers – to measure deflections; and
(iii) strain gauges – to measure strains in the steel reinforcement and on the concrete surfaces at all stages of loading.

The results of the tests are presented and evaluated, with particular emphasis on the strength, ductility and failure mode of the slabs.

Earlier experimental investigations on the strength and ductility of one-way slabs and the effect of support settlement have been described in several earlier reports (Sakka and Gilbert, 2008a, b and c).
2. Experimental Program

2.1 Test Specimens:

A total of eleven two-way, corner-supported, reinforced concrete slabs with different reinforcement ratios were cast and tested to investigate their behaviour under controlled deformation. Figure 2-1 shows a plan view of a typical slab, together with its boundary conditions and the coordinate axes. The slab specimens were either rectangular or square in plan. The square slabs had an overall side length of 2400 mm, while the length and width of the rectangular slabs were 3600 mm and 2400 mm, respectively. Table 2-1 summarizes the properties of the corner supported slabs. All slabs contained two layers of orthogonal bottom reinforcement. The steel bars or wires in the y-direction were placed closest to the soffit of the slab at an effective depth $d_y$ below the top surface, while the bars or wires in the x-direction had an effective depth $d_x = d_y -$ bar diameter, as shown in Figure 2-2. The reinforcement arrangements in the square slabs and in the rectangular slabs are shown in Figures 2-3 and 2-4, respectively. All the dimensions shown in these figures and throughout this report are in millimeters. Table 2-2 summarizes the properties of the concrete used in each slab and Table 2-3 lists the properties of the steel reinforcement.

![Figure 2-1: Plan view of corner-supported two-way slabs.](image)

![Figure 2-2: Section A-A (typical).](image)
Table 2-1: Properties of the corner-supported two-way slabs.

<table>
<thead>
<tr>
<th>Slab Designation</th>
<th>$L_y^a$ (mm)</th>
<th>$L_x^a$ (mm)</th>
<th>$D$ (mm)</th>
<th>Steel Class &amp; Type $^b$</th>
<th>Bar dia. (mm)</th>
<th>$n^c$</th>
<th>$A_{st}$ (mm$^2$)</th>
<th>$d_x$ (mm)</th>
<th>$p$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2S-1</td>
<td>2080</td>
<td>2080</td>
<td>103.4</td>
<td>L - SL62</td>
<td>6.0</td>
<td>12</td>
<td>339</td>
<td>78.7</td>
<td>0.18</td>
</tr>
<tr>
<td>S2S-2</td>
<td>2080</td>
<td>2080</td>
<td>101.4</td>
<td>L - SL82</td>
<td>7.6</td>
<td>12</td>
<td>544</td>
<td>76.2</td>
<td>0.30</td>
</tr>
<tr>
<td>S2S-3</td>
<td>2080</td>
<td>2080</td>
<td>100.1</td>
<td>L - SL102</td>
<td>9.5</td>
<td>12</td>
<td>851</td>
<td>76.2</td>
<td>0.47</td>
</tr>
<tr>
<td>S2S-4</td>
<td>2080</td>
<td>2080</td>
<td>100.0</td>
<td>N - N12</td>
<td>12.0</td>
<td>8</td>
<td>905</td>
<td>74.0</td>
<td>0.51</td>
</tr>
<tr>
<td>S2S-5</td>
<td>2080</td>
<td>2080</td>
<td>106.1</td>
<td>N - N10</td>
<td>10.0</td>
<td>12</td>
<td>942</td>
<td>75.5</td>
<td>0.52</td>
</tr>
<tr>
<td>S2S-6</td>
<td>2080</td>
<td>2080</td>
<td>100.0</td>
<td>N - N12</td>
<td>12.0</td>
<td>8</td>
<td>905</td>
<td>79.0</td>
<td>0.48</td>
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<tr>
<td>S2R-1</td>
<td>3280</td>
<td>2080</td>
<td>103.7</td>
<td>L - SL62</td>
<td>6.0</td>
<td>18</td>
<td>509</td>
<td>81.8</td>
<td>0.17</td>
</tr>
<tr>
<td>S2R-2</td>
<td>3280</td>
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<td>95.9</td>
<td>L - SL82</td>
<td>7.6</td>
<td>18</td>
<td>817</td>
<td>75.2</td>
<td>0.30</td>
</tr>
<tr>
<td>S2R-3</td>
<td>3280</td>
<td>2080</td>
<td>106.7</td>
<td>L - SL102</td>
<td>9.5</td>
<td>18</td>
<td>1276</td>
<td>84.6</td>
<td>0.42</td>
</tr>
<tr>
<td>S2R-4</td>
<td>3280</td>
<td>2080</td>
<td>100.0</td>
<td>N - N12</td>
<td>12.0</td>
<td>12</td>
<td>1357</td>
<td>74.0</td>
<td>0.51</td>
</tr>
<tr>
<td>S2R-5</td>
<td>3280</td>
<td>2080</td>
<td>101.6</td>
<td>N - N10</td>
<td>10.0</td>
<td>18</td>
<td>1414</td>
<td>72.0</td>
<td>0.55</td>
</tr>
</tbody>
</table>

$^a$ Centre to centre spans; $^b$ L - Class L; N - Class N; $^c$ n = number of bars or wires

Table 2-2: Concrete properties.

<table>
<thead>
<tr>
<th>Slab Designation</th>
<th>$f'_c$ (MPa)</th>
<th>$\varepsilon_c$ (%)</th>
<th>$f'_t$ (MPa)</th>
<th>$f'_{cy}$ (MPa)</th>
<th>$E_c$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2S-1</td>
<td>44.3</td>
<td>-</td>
<td>3.61</td>
<td>3.87</td>
<td>29.20</td>
</tr>
<tr>
<td>S2S-2</td>
<td>44.3</td>
<td>-</td>
<td>3.61</td>
<td>3.87</td>
<td>29.20</td>
</tr>
<tr>
<td>S2S-3</td>
<td>32.2</td>
<td>0.242</td>
<td>3.09</td>
<td>5.06</td>
<td>27.97</td>
</tr>
<tr>
<td>S2S-4</td>
<td>58.0</td>
<td>0.255</td>
<td>4.68</td>
<td>4.51</td>
<td>34.19</td>
</tr>
<tr>
<td>S2S-5</td>
<td>32.2</td>
<td>0.242</td>
<td>3.09</td>
<td>5.06</td>
<td>27.97</td>
</tr>
<tr>
<td>S2S-6</td>
<td>26.7</td>
<td>0.234</td>
<td>2.88</td>
<td>2.97</td>
<td>25.59</td>
</tr>
<tr>
<td>S2R-1</td>
<td>44.3</td>
<td>-</td>
<td>3.61</td>
<td>3.87</td>
<td>29.20</td>
</tr>
<tr>
<td>S2R-2</td>
<td>44.3</td>
<td>-</td>
<td>3.61</td>
<td>3.87</td>
<td>29.20</td>
</tr>
<tr>
<td>S2R-3</td>
<td>58.0</td>
<td>0.255</td>
<td>4.68</td>
<td>4.51</td>
<td>34.19</td>
</tr>
<tr>
<td>S2R-4</td>
<td>58.0</td>
<td>0.255</td>
<td>4.68</td>
<td>4.51</td>
<td>34.19</td>
</tr>
<tr>
<td>S2R-5</td>
<td>44.0</td>
<td>0.292</td>
<td>3.56</td>
<td>4.60</td>
<td>28.29</td>
</tr>
</tbody>
</table>

Table 2-3: Steel reinforcement properties.

<table>
<thead>
<tr>
<th>Slab Designation</th>
<th>Steel Type</th>
<th>Bar dia. (mm)</th>
<th>$E_s$ (MPa)</th>
<th>$f_{sy}$ (MPa)</th>
<th>$f_m$ (MPa)</th>
<th>$f_{sy} / f_m$</th>
<th>$\varepsilon_{sy}$ (%)</th>
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</thead>
<tbody>
<tr>
<td>S2S-1</td>
<td>SL62</td>
<td>6.0</td>
<td>$2 \times 10^5$</td>
<td>586</td>
<td>618</td>
<td>1.05</td>
<td>2.47</td>
</tr>
<tr>
<td>S2S-2</td>
<td>SL82</td>
<td>7.6</td>
<td>$2 \times 10^5$</td>
<td>600</td>
<td>641</td>
<td>1.07</td>
<td>2.11</td>
</tr>
<tr>
<td>S2S-3</td>
<td>SL102</td>
<td>9.5</td>
<td>$2 \times 10^5$</td>
<td>518</td>
<td>579</td>
<td>1.12</td>
<td>3.73</td>
</tr>
<tr>
<td>S2S-4</td>
<td>N12</td>
<td>12.0</td>
<td>$2 \times 10^5$</td>
<td>591</td>
<td>678</td>
<td>1.15</td>
<td>7.69</td>
</tr>
<tr>
<td>S2S-5</td>
<td>N10</td>
<td>10.0</td>
<td>$2 \times 10^5$</td>
<td>565</td>
<td>677</td>
<td>1.20</td>
<td>9.65</td>
</tr>
<tr>
<td>S2S-6</td>
<td>N12</td>
<td>12.0</td>
<td>$2 \times 10^5$</td>
<td>510</td>
<td>619</td>
<td>1.21</td>
<td>14.11</td>
</tr>
<tr>
<td>S2R-1</td>
<td>SL62</td>
<td>6.0</td>
<td>$2 \times 10^5$</td>
<td>590</td>
<td>627</td>
<td>1.06</td>
<td>2.46</td>
</tr>
<tr>
<td>S2R-2</td>
<td>SL82</td>
<td>7.6</td>
<td>$2 \times 10^5$</td>
<td>580</td>
<td>620</td>
<td>1.07</td>
<td>2.30</td>
</tr>
<tr>
<td>S2R-3</td>
<td>SL102</td>
<td>9.5</td>
<td>$2 \times 10^5$</td>
<td>580</td>
<td>620</td>
<td>1.07</td>
<td>3.09</td>
</tr>
<tr>
<td>S2R-4</td>
<td>N12</td>
<td>12.0</td>
<td>$2 \times 10^5$</td>
<td>591</td>
<td>678</td>
<td>1.15</td>
<td>7.69</td>
</tr>
<tr>
<td>S2R-5</td>
<td>N10</td>
<td>10.0</td>
<td>$2 \times 10^5$</td>
<td>565</td>
<td>677</td>
<td>1.20</td>
<td>9.65</td>
</tr>
</tbody>
</table>
Figure 2-3: Reinforcement arrangement for (a) S2S-1, 2, 3, and 5 and (b) S2S-4 and 6.

Figure 2-4: Reinforcement arrangement for (a) S2R-1, 2, 3, and 5 and (b) S2R-4.

2.2 Test Setup:

The slabs were effectively point supported at each of the four corners on a steel bearing plate (square in plan) as shown in Figure 2-1. Three of the four supports were roller supports and the fourth was a pin support. A typical support consisted of an adjustable steel stand, a load cell, a
cup-shaped steel cylinder, a steel sphere (at the roller supports only) and a steel bearing plate. The bearing plate used at each support was 120 x 120 x 20 mm. Figure 2-5 shows a typical roller support. The centre of each support was located at a distance of 160mm from each adjacent edge of the slab. The adjustable steel stand consisted of two steel plates (top and bottom) and four threaded steel rods. The height of each stand was adjustable to ensure that prior to testing full contact was obtained at each support and that the load due to self-weight was equally distributed to the four supports.

Two different arrangements were used for loading the slabs; either a single point load was applied to the slab at its mid-panel or four point loads were applied symmetrically about the mid-panel of the slab. Table 2-4 lists the different loading setups and the dimensions of the bearing plates under each point load. Figure 2-6 shows the locations of the loading points for each slab. Also given in Table 2-4 is the self-weight of each slab, the weight of the loading arrangement and packing plates and the rate at which deformation was applied to the slab throughout the test through the hydraulic actuator.

Photographs of various slabs under load, showing the different loading set-ups are provided in Figure 2-7.
Table 2-4: Loading details, self-weight and loading rates for the corner-supported slabs.

<table>
<thead>
<tr>
<th>Slab Designation</th>
<th>Loading points</th>
<th>Bearing plate dimensions (mm)</th>
<th>Slab self-weight (kN)</th>
<th>Weight of loading set-up and packing (kN)</th>
<th>Loading rate (mm/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2S-1</td>
<td>1</td>
<td>150 x 150 x 75</td>
<td>14.0</td>
<td>0.35</td>
<td>2.40</td>
</tr>
<tr>
<td>S2S-2</td>
<td>1</td>
<td>150 x 150 x 75</td>
<td>13.7</td>
<td>0.35</td>
<td>1.10</td>
</tr>
<tr>
<td>S2S-3</td>
<td>4</td>
<td>300 x 300 x 25</td>
<td>13.6</td>
<td>1.67</td>
<td>3.75</td>
</tr>
<tr>
<td>S2S-4</td>
<td>1</td>
<td>200 x 200 x 50</td>
<td>13.5</td>
<td>0.42</td>
<td>3.56</td>
</tr>
<tr>
<td>S2S-5</td>
<td>4</td>
<td>300 x 300 x 25</td>
<td>14.4</td>
<td>1.67</td>
<td>3.75</td>
</tr>
<tr>
<td>S2S-6</td>
<td>4</td>
<td>300 x 300 x 25</td>
<td>15.8</td>
<td>2.68</td>
<td>4.66</td>
</tr>
<tr>
<td>S2R-1</td>
<td>1</td>
<td>150 x 150 x 75</td>
<td>21.0</td>
<td>0.35</td>
<td>2.69</td>
</tr>
<tr>
<td>S2R-2</td>
<td>1</td>
<td>150 x 150 x 75</td>
<td>19.5</td>
<td>0.35</td>
<td>1.31</td>
</tr>
<tr>
<td>S2R-3</td>
<td>1</td>
<td>150 x 150 x 75</td>
<td>21.7</td>
<td>0.35</td>
<td>2.60</td>
</tr>
<tr>
<td>S2R-4</td>
<td>1</td>
<td>150 x 150 x 75</td>
<td>20.3</td>
<td>0.35</td>
<td>3.43</td>
</tr>
<tr>
<td>S2R-5</td>
<td>4</td>
<td>300 x 300 x 25</td>
<td>20.6</td>
<td>1.67</td>
<td>1.84</td>
</tr>
</tbody>
</table>

(a) Slabs S2S-1 & 2  
(b) Slab S2S-4  
(c) Slabs S2S-3, 5 & 6. 
(d) Slabs S2R-1, 2, 3 & 4. 
(e) Slab S2R-5.

Figure 2-6: Arrangement of loading plates for each slab.
Figure 2-7: Loading setup for the corner-supported two-way slabs (a) Slab S2S-1 (similar S2S-2, S2R-1, 2, 3, and 4), (b) Slabs S2S-5 (similar S2S-3, 6, and S2R-5) (c) slab S2S-4.
2.3 Instrumentation:

For each test slab, linear variable displacement transducers (LVDTs) and laser displacement sensors were used to measure deflections and displacements; strain gauges were used to monitor reinforcement and concrete surface strains; and load cells were used to measure the applied loads and the reactions at each support. Laser sensors were placed at locations where high deflections were expected because they have a larger range of measurement than the LVDTs.

Figure 2-8 shows the locations of the LVDTs and the laser sensors in the square slabs, while Figure 2-9 the locations of the strain gauges on the steel wires and on the concrete surface. Two orthogonal strain gauges were glued to the top surface of the concrete at the mid-panel of every slab. The location of the LVDTs, laser sensors, and strain gauges in the rectangular slabs are shown in Figure 2-10 and Figure 2-11. The load cells used to measure the reactions in the slabs are numbered and shown in Figure 2-6.

![Diagram of LVDTs and laser sensors](image1)

(a) Slabs S2S-1, 2 and 4.

![Diagram of strain gauges](image2)

(b) Slabs S2S-3, 5 & 6.

**Figure 2-8**: LVDT (LV) and laser (LS) locations in the square slabs.

![Diagram of strain gauges](image3)

(a) Slabs S2S-3 & 5

![Diagram of strain gauges](image4)

(b) Slab S2S-6

![Diagram of strain gauges](image5)

(c) All slabs

**Figure 2-9**: Tension and compression strain gauge locations in the square slabs.
Figure 2-10: LVDT (LV) and laser (LS) locations in the rectangular slabs.

Figure 2-11: Tension and compression strain gauge locations in the rectangular slabs.
2.4 Deflected Shape – Notation:

Figure 2-12a shows a typical corner supported two-way slab. Figure 2-12b shows the deflected shape of the slab caused by bending about the y-axis (i.e. due to loads being carried in the x-direction). The moments on the column lines in Figure 2-12b are larger than the moments in the mid-panel region and hence the deflection on the column line ($\Delta_{ey}$) is greater than the deflection at mid-panel ($\Delta_{my}$), as shown. The deflected shape and the relative magnitudes of $\Delta_{ex}$ and $\Delta_{mx}$ depend on the magnitude of curvature in each region and this in turn depends on the extent of cracking in each region, whether or not the steel in each region has yielded and so on. Figure 2-12c shows the deflected shape of the slab caused by bending about the x-axis (i.e. due to loads being carried in the y-direction).

Figure 2-12d shows the deflected shape of the corner-supported two-way slab panel carrying loads to the supports in both the x- and y-directions. In general, the deflection at the geometric centroid of the slab (i.e. at the mid-panel of the slab), shown as ($\Delta_{my} + \Delta_{mx}$) in Figure 2-12d, is the maximum deflection of the slab panel, but this may not necessarily be the case (particularly for rectangular slabs) depending on the extent of cracking of the concrete and whether or not the steel reinforcement in one-direction has yielded.

(a) Corner supported two-way slab.  
(b) Deflection due to bending in x-direction

(c) Deflection due to bending in y-direction  
(d) Deflected shape of corner-supported slab

Figure 2-12: Corner-supported two-way slab – deflected shape and notation.
3. Experimental Results

3.1 Detailed Results for Selected Slabs:

All the corner-supported two-way slab specimens were tested in displacement control mode. Applied loads, reactions, deflections, and strains were recorded throughout the whole test period for each slab up to the end of the test. Tables 3-1 and 3-2 list the reason why each test was terminated and the types of failure, if any, for the square and rectangular slabs, respectively.

Table 3-1: Termination of the square corner-supported slab tests and types of failures

<table>
<thead>
<tr>
<th>Slab Name</th>
<th>Reinf. Ductility Class</th>
<th>Termination point of test</th>
<th>Type of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2S-1</td>
<td>L</td>
<td>Sample failure -- sudden collapse</td>
<td>Fracture of reinforcing wires</td>
</tr>
<tr>
<td>S2S-2</td>
<td>L</td>
<td>Sample failure -- sudden collapse</td>
<td>Fracture of reinforcing wires</td>
</tr>
<tr>
<td>S2S-3</td>
<td>L</td>
<td>Sample failure -- sudden collapse</td>
<td>Fracture of reinforcing wires</td>
</tr>
<tr>
<td>S2S-4</td>
<td>N</td>
<td>Sample failure - local</td>
<td>Punching shear</td>
</tr>
<tr>
<td>S2S-5</td>
<td>N</td>
<td>Actuator deformation limit reached</td>
<td>Slab deflection was such that the actuator reached its maximum travel distance – Sample did not actually fail</td>
</tr>
<tr>
<td>S2S-6</td>
<td>N</td>
<td>Actuator deformation limit reached</td>
<td>Slab deflection was such that the actuator reached its maximum travel distance – Sample did not actually fail</td>
</tr>
</tbody>
</table>

Table 3-2: Termination of the rectangular corner-supported slab tests and types of failures

<table>
<thead>
<tr>
<th>Slab Name</th>
<th>Reinf. Ductility Class</th>
<th>Termination point of test</th>
<th>Type of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2R-1</td>
<td>L</td>
<td>Sample failure -- sudden collapse</td>
<td>Fracture of reinforcing wires</td>
</tr>
<tr>
<td>S2R-2</td>
<td>L</td>
<td>Sample failure -- sudden collapse</td>
<td>Fracture of reinforcing wires</td>
</tr>
<tr>
<td>S2R-3</td>
<td>L</td>
<td>Sample failure -- sudden collapse</td>
<td>Fracture of reinforcing wires</td>
</tr>
<tr>
<td>S2R-4</td>
<td>N</td>
<td>Slab soffit touched supporting frame</td>
<td>Slab deflection became so large that the slab touched the supporting steel frame – Sample did not actually fail</td>
</tr>
<tr>
<td>S2R-5</td>
<td>N</td>
<td>Actuator deformation limit reached</td>
<td>Slab deflection was such that the actuator reached its maximum travel distance – Sample did not actually fail</td>
</tr>
</tbody>
</table>

Before cracking, the deflected shape of all the square slabs was symmetric about both the x and y axes (with origin at the mid-panel). As the applied loads increased and the stress in the reinforcing steel approached the yield stress, the slabs started to show higher curvature in the weaker direction. The weaker direction in the square slabs (with equal quantities of steel in each orthogonal direction) is the direction of the reinforcement that has the smaller effective depth (i.e.
the x-direction). The weak direction in the rectangular slabs is the long direction (the y-direction) because the slabs had less total reinforcement in that direction and, because the span is longer, the applied moment is larger than in the orthogonal x-direction. In both the square and the rectangular slabs, after the formation of flexural cracks, the crack widths and crack spacing was found to depend on the steel type, the quantity and spacing of the reinforcement and the reinforcement ratio.

In this section, the detailed results for three of the square slabs (S2S-1, S2S-2 and S2S-3) and two of the rectangular slabs (S2R-2 and S2R-4) are presented. The detailed results for the remaining slabs are provided in the Appendix.

3.1.1 Slab S2S-1

The load versus mid-panel deflection (LVDT 1) curve is shown in Figure 3-1. The mid-panel deflection is the deflection at their geometric centroid of the slab (see Figure 2-12a). This slab was reinforced with 6.0mm diameter Class L WWF (SL62) and had very small reinforcement ratios in each direction, namely $p = 0.180$ and $0.167\%$ in the x and y directions, respectively. As can be seen in Figure 3-1, the ultimate strength of the slab was only just higher than the load required to cause first cracking (and as such had an unacceptable and impractically small amount of reinforcement). Prior to cracking the slope of the load deflection plot is proportional to the flexural rigidity of the uncracked slab and is indicated by the dashed line labeled ‘uncracked rigidity’. The rigidity of the fully cracked slab (obtained by ignoring the contribution of the tensile concrete) is proportional to the slope of the second dashed line labeled ‘cracked rigidity’. For this very lightly reinforced slab, the bulk of the total deflection is associated by the loss of stiffness due to cracking, with the steel yielding and deforming plastically immediately after first cracking.

![Figure 3-1: Load versus mid-panel deflection (S2S-1).](image)

The slab failed in a brittle mode by fracture of the reinforcing steel, resulting in sudden, catastrophic collapse. Failure was initiated when the first reinforcement wire in the x-direction fractured at a mid-panel deflection of $\Delta = 20.6$ mm ($L/\Delta = 101.0$). This was followed almost immediately by fracture of the remaining steel wires in the x-direction one after the other. As can
be seen in Figure 3-1, this occurred while the contribution of the cracked tensile concrete to the slab stiffness was still significant (and the load-deflection plot was well to the left of the dashed line representing the rigidity of the fully-cracked slab). The slab collapsed when the mid-panel deflection reached 23 mm, i.e. when the span to deflection ratio $L/\Delta = 90.4$. The concrete did not experience any signs of crushing in the compressive zone at any stage during the test.

When the total load reached 41.3 kN at a mid-panel deflection of 2.01 mm, two cracks appeared at mid-span. The first crack was parallel to the x-axis and the second crack was parallel to the y-axis. Prior to cracking, the slab curvatures about the x and y axes were similar and the slab deformation was symmetric about both axes. However, after the slab cracked and the mid-panel deflection increased significantly, the crack parallel to the y axis widened much more rapidly than the crack parallel to the x-axis. This was expected because the reinforcement crossing the y-axis crack had a significantly smaller effective depth ($d_e$) than the reinforcement in the other direction, and hence the second moment of area of the cracked slab in this direction was significantly smaller than for the cracked section in the orthogonal direction and the corresponding curvature was significantly greater. Figure 3-2 shows the two cracks just prior to failure, one in each orthogonal direction. The slab collapsed when the reinforcement wires crossing the y-direction crack (Figure 3-2b and Figure 3-2c) fractured (i.e. the reinforcement wires running in the x-direction).

Figure 3-2: Slab S2S-1 (a) crack parallel to x-axis, (b) crack parallel to y direction, (c) crack parallel to the y-axis just before collapse, (d) slab after collapse.
Figure 3-3 shows the relative deflections of the slab at each of the LVDT measurement points. The horizontal axis is the mid-panel deflection (LVDT 1) and the vertical axis is the deflection recorded at each LVDT location. After cracking, the slab was significantly stiffer in the y-direction than in the x-direction and, consequently, the slab deflection in the x direction was significantly larger than in the y-direction. The deflected shape of the slab more resembled that of a one-way slab, with the two portions of the slab on either side of the y-direction crack rotating about the axes through their point supports. The deflections at LVDT 2 and 3 started to increase at a higher rate than the deflections at LVDT 4 and 5, as can be seen in Figure 3-3.

![Graph showing LVDT readings versus mid-panel deflection (S2S-1).](image)

Figure 3-3: LVDT readings versus mid-panel deflection (S2S-1).

Figure 3-4 shows the load versus mid-panel deflection curve and the absorbed work versus deflection curve. The absorbed work at any particular deflection is represented by the area under the load-deflection curve from first loading to the deflection under consideration. As already mentioned, the cracking load is close to the peak load for this very lightly reinforced slab. The load is considered to plateau at $1/(f_{o} f_{y}) = 1/1.05$ of its peak load value. This means that the load started to plateau at a total load of 412 kN at a mid-panel deflection of $\Delta_1 = 7.5$ mm. The end of the plastic deformation is taken to be when the steel fractures or the load drops to less than 75% of its peak value, i.e. at a deflection of $\Delta_2 = 20.8$ mm. The total work absorbed by the slab between these two points is due to plastic deformation in the Class L wires and equals 563 kN.mm. The shape of the load-deflection plot for slab S2S-1 is unusual (compared to all other test slabs) in that the reinforcement quantity is so small that the steel yields immediately after first cracking and there is no post-cracking elastic deformation prior to reinforcement yielding.

Figure 3-5 shows the change in the four corner reactions with slab deflection. As expected, prior to cracking, the four reactions are very similar. After cracking, some relatively small differences in reactions began to occur between mid-panel deflections of 2.0 mm and 6.0 mm, probably due to loss of symmetry caused by cracking. At mid-panel deflections greater than about 10.0 mm, the two diagonal reactions at load cells 2 and 5 continued to increase, while the other two diagonal reactions, LC 3 and LC 4, started to decrease. This is due to the on-set of plastic deformation in some steel wires crossing the y-direction crack (probably those near the column lines) as the slab began to unload and fail.
Figure 3-4: Load and work versus mid-panel deflection for slab S2S-1.

Figure 3-5: Supports reaction-deflection curves for slab S2S-1.

The flexural capacity provided by the reinforcement wires in the x-direction ($A_{st} = 339 \text{ mm}^2$ at an effective depth $d_x = 78.7 \text{ mm}$) controlled the overall capacity of the slab, which failed in a similar manner to a one-way slab spanning in the x-direction by fracturing of the wires in that direction at the critical crack running along the y-axis at the slab mid-span. Figure 3-6 is a graph of the applied moment about the y-axis at the mid-span of the slab versus the mid-panel deflection. The figure also shows the calculated flexural capacity of the slab on this section. The maximum moment actually resisted by the slab at the critical crack was 18.4 kN.m, which is 14% higher than the calculated slab capacity of 16.1 kN.m. A discrepancy such as this was consistently observed throughout the tests involving Class L reinforcement and may be due to membrane action or a component of residual tension on the irregular cracked concrete surface (due to aggregate interlock/friction) or both.
Figure 3-6: Total applied moment about the y-axis for slab S2S-1.

The concrete compressive strains in the x and y directions on the top surface of the slab at the mid-panel are shown in Figure 3-7. The figure shows that strains in the x and y directions are similar at the beginning of the test. However, after first cracking, as the slab curvature starts to increase faster in the x direction, the concrete compressive strain in that direction also increases more than in the orthogonal direction. It is noted that at no stage does the concrete compressive strain in either direction exceed 500 x 10^-6 indicating that the concrete compressive stresses remain in the elastic range throughout the test.

Figure 3-7: Concrete surface compressive strains in x and y directions at the mid-panel S2S-1.

The deflection contours of the slab are shown in Figure 3-8. In each case, the deflection about each axis is assumed to be symmetric and the profiles have been determined from the LVDT readings measured on the x and y axes. The deflected slab contours before cracking at a total load of 30.0 kN and in the post-peak phase at a total load of 41.0 kN are shown in Figure 3-8a and Figure 3-8b, respectively. Prior to cracking, both the deflected shape and the magnitude of deflection in both the x and y directions are similar (as one would expect in a symmetrically loaded square corner supported slab). After cracking and as the steel in the x-
direction begins to deform plastically, the slab deflects predominantly in that direction. In fact, the deflected shape of the slab begins to look more like that of a one-way slab spanning in the x-direction rather than that of a symmetrically loaded two-way square slab.

![Graphs showing deflection contours](image)

(a) At a total load of 30kN prior to cracking  
(b) At a total load of 41kN in the post-peak range

**Figure 3-8:** Deflection contours of S2S-1.

### 3.1.2 Slab S2S-2

The load versus mid-panel deflection (LVDT 1) curve for S2S-2 is shown in Figure 3-9. This slab was reinforced with 7.6 mm diameter Class L WWF (SL82) and had reinforcement ratios of $p = 0.30\%$ and $0.27\%$ in the x and y directions, respectively. The lines representing the 'uncracked rigidity' and the 'cracked rigidity' are also shown on Figure 3-9. As can be seen, relatively little deflection occurred after the onset of yielding of the reinforcement (at a mid-panel deflection of about 22 mm) and hence relatively little deflection was associated with plastic deformation of the reinforcement. As was the case for slab S2S-1, the bulk of the total deflection is associated by the loss of stiffness due to cracking.

The slab started to crack at a total load of 38.2 kN when the mid-panel deflection was 2.2 mm. The applied load was gradually increased until the reinforcement in the x-direction began to yield at a load of about 60.8 kN when the mid-panel deflection reached 21.8 mm. The peak load of 65.0 kN was reached at a mid-panel deflection of 29.6mm. The slab then began to unload as the deformation was gradually increased. As for S2S-1, the slab failed in a brittle mode by fracture of the reinforcing steel resulting in sudden collapse. Fracturing of the low-ductility wires in their post-peak softening range commenced when the mid-panel deflection reached 34 mm ($L/\Delta = 61.2$), with the steel wires snapping one after the other in rapid succession. Throughout the entire test, the concrete did not experience any signs of crushing in the compressive zone.
Figure 3-9: Load versus mid-panel deflection (S2S-2).

As load increased above the first cracking load, several flexural cracks appeared running across the slab in both the x and y directions. The cracks spacing in each direction was initially approximately 200 mm. As the test progressed, additional cracks occurred parallel to x-axis in the mid-span region and the crack spacing reduced to about 100 mm. The cracks spacing in the orthogonal direction did not change, however, as the crack at the mid-span started widening. Figure 3-10a and Figure 3-10b show photographs of the slab under load with the cracks in both directions highlighted, while Figure 3-10c shows the slab after failure.

Figure 3-11 shows the relative deflections of the slab at each of the LVDT measurement points. At a mid-panel deflection of about 22 mm, the rate of increase in deflection at LVDT 4 and 5 started to drop, while the rate of increase in deflection at LVDT 2 and 3 started to increase. This is similar to the observation made on S2S-1, with the point at which the rate of deflection changes corresponding to the onset of yielding in the reinforcement wires. At this point, the slab is significantly stiffer in the y-direction than in the x-direction and, as a consequence, the slab deflection in the x direction becomes larger than in the y-direction. As deformation increases, the difference in deformation in each direction becomes more and more pronounced.

Figure 3-12 shows the load-deflection curve and the absorbed work-deflection curve for S2S-2. The load-deflection curve is considered to start to plateau at a total load equal to the peak load multiplied by 1/(f_{cu}/f_{y}), i.e. 65.0 x (1/1.07) = 60.8 kN. This occurs at a mid-panel deflection of $\Delta_1 = 21.8$ mm. The plastic plateau ended abruptly at a deflection of $\Delta_2 = 33.9$ mm. The total plastic work absorbed by the slab between these two points ($W_p$) is the area under the load-deflection curve and is equal to 772 kN.mm. The work from first loading to $\Delta_1$ is $W_0 = 1075$ kN.mm. The sudden drops in load at 33.9 mm and 34.8 mm are the effect of snapping of the steel wires in the x direction.

Figure 3-13 shows the change in the four corner reactions with slab deflection. For this slab, the four reactions are almost the same throughout the test until at a deflection of about 28 mm two diagonally opposite reactions begin to decrease, while the other two continue to increase. This effect commenced after the onset of plastic deformation in the steel and was also observed in S2S-1.
(a) Cracks parallel to x-axis spaced 100mm apart in the mid-span region

(b) Cracks parallel to the y-axis spaced 200mm apart in the mid-span region

(c) Slab after collapsing about the y-direction crack at mid-span

Figure 3-10: Slab S2S-2 during and after the test
Figure 3-11: LVDT readings versus mid-span deflection (S2S-2).

Figure 3-12: Load and work versus mid-panel deflection for slab S2S-2.

Figure 3-13: Supports reaction versus mid-panel deflection for slab S2S-2.
The failure mode for slab S2S-2 was exactly the same as that of S2S-1. Figure 3-14 shows the applied moment about the y-axis at mid-span versus mid-panel deflection. The figure also shows the slab flexural capacity about the same axis (i.e. the moment capacity provided by the reinforcement in the x-direction). The maximum moment resisted by the reinforcement in the x-direction was 29.7 kN.m and this is 14.7% higher than the calculated slab capacity of 25.9 kN.m. As mentioned previously, a similar discrepancy was observed in all tests involving Class L reinforcement.

![Figure 3-14: Total applied moment about the y axis for slab S2S-2.](image)

The variation of concrete compressive strains in the x and y directions at the mid-panel are shown in Figure 3-15. The figure shows that strain in the x direction is consistently higher than that in the y direction after first cracking. At the fracture of the first bar, the maximum concrete compressive strain was less than 800 x 10^-6.

![Figure 3-15: Concrete surface strains in the x and y directions at mid-panel (S2S-2).](image)
The deflection contours before cracking (at a load of 35 kN) and in the post peak region at a load of 64 kN are shown in Figure 3-16. The deflected shape of slab S2S-2 is similar to that observed in S2S-1. The deflection contours prior to cracking are essentially circular (symmetrical), becoming oval after yielding of the reinforcement, indicating higher deflection in the x direction due to the reduced stiffness in that direction.

![Deflection contours of slab S2S-2](image)

(a) At a load of 35 kN before cracking.  
(b) At a load of 64 kN in the post-peak phase.

**Figure 3-16**: Deflection contours of slab S2S-2.

### 3.1.3 Slab S2S-3

The load versus mid-panel deflection curve for S2S-3 is shown in Figure 3-17 (with deflection measured using LS 1). This slab was reinforced with 9.5 mm diameter Class L WWF (SL82) and had reinforcement ratios of $p = 0.47\%$ and $0.41\%$ in the x and y directions, respectively. The lines representing the ‘uncracked rigidity’ and the ‘cracked rigidity’ are also shown on Figure 3-17. For this slab, unlike S2S-1 and S2S-2, significant deflection occurred after the onset of yielding of the reinforcement. It is noted that the welded wire fabric in this slab was significantly more ductile than the reinforcement in S2S-1 and S2S-3, with $\varepsilon_{un} = 3.73\%$ and $f_{un}/f_{y} = 1.12$ (see Table 2-3), and as a consequence structural behaviour was considerably more ductile.

The slab started to crack at a total load of 43.2 kN when the mid-panel deflection was 3.1 mm. The applied load was gradually increased until the reinforcement in the x-direction began to yield at a load of 106.7 kN when the mid-panel deflection reached 52.0 mm. The peak load of 119.5 kN was reached at a mid-panel deflection of 83.9 mm. The slab then began to unload as the deformation was gradually increased. As for S2S-1 and S2S-2, the slab failed by fracture of the reinforcing steel, but unlike S2S-1 and S2S-2, the compressive concrete at the top surface of the slab above the critical crack did begin to crush prior to collapse (as can be see in Figure 3-18d).
Figure 3-17: Load versus mid-panel deflection (S2S-3).

Figure 3-18a is a photograph of the slab under load and Figure 3-18b is the soffit of the slab showing the critical crack running in the y-direction through the mid-span. An elevation of the slab showing the critical crack at mid-span just prior to failure is shown in Figure 3-18c and crushing of the compressive concrete on the top surface of the slab above the critical crack just before fracturing of the tensile reinforcement is shown in Figure 3-18d.

Figure 3-18: Slab S2S-3, (a) deflected slab just before failure, (b) slab soffit just before failure, (c) cracks parallel to the y-axis spaced at 200mm, and (d) concrete crushing in the compression zone at critical section.
Figure 3-19 shows the relative deflections of the slab at each of the measurement points. At a mid-panel deflection of about 60.0mm, the rate of increase in deflection at LVDT 2 on the column line in the x-direction started to increase appreciably, while the rate of increase in deflection at LVDT 3 on the column line in the y-direction started to decrease. This is similar to the observations made on S2S-1 and S2S-2, with the point at which the rate of deflection changes corresponding to the onset of yielding in the reinforcement wires.

![Figure 3-19: LVDT readings versus mid-span deflection (S2S-3).](image)

Figure 3-20 shows the load-deflection curve and the absorbed work-deflection curve for S2S-3. The load-deflection curve started to plateau at a total load equal to the peak load multiplied by $\frac{1}{f_{sw}/f_{yp}}$, i.e. $119.5 \times (1/1.12) = 106.7 \text{ kN}$. This occurs at a mid-panel deflection of $\Delta_1 = 52.0 \text{ mm}$. The load-deflection curve dropped significantly when the reinforcing wires fractured at $\Delta_2 = 102.8 \text{ mm}$. The total plastic work absorbed by the slab between $\Delta_1$ and $\Delta_2$ is $W_1 = 5800 \text{ kN.mm}$ (and is the area under the load-deflection curve between these two deflections), while the work from first loading to $\Delta_1$ is $W_0 = 3810 \text{ kN.mm}$.

![Figure 3-20: Load and work versus mid-panel deflection for slab S2S-3.](image)
Figure 3-21 shows the change in the four corner reactions with slab deflection. For this slab, the four reactions show similar trends throughout the test at all levels of loading.

![Reaction versus Mid-panel deflection for slab S2S-3](image)

**Figure 3-21:** Supports reaction versus mid-panel deflection for slab S2S-3.

The variation of steel reinforcement strains at the mid-span in both the x and y directions are shown in Figure 3-22. The figure shows that far more plastic deformation occurs in the wires in the x direction than in the wires in the y-direction (which are closest to the slab soffit) when the mid-panel deflection exceeded about 60 mm.

![Strain in steel wires running along x and y directions](image)

**Figure 3-22:** Strain in the steel wires running along the x and y directions in S2S-3.

The deflection contours before cracking (at a load of 41.2 kN) and in the post peak region at a load of 118.1 kN are shown in Figure 3-23.
3.1.4 Slab S2R-2

The load versus mid-panel deflection (LVDT 1) curve for S2S-3 is shown in Figure 3-24. This slab was reinforced with 7.6 mm diameter Class L WWF (SL82) and had reinforcement ratios of $p = 0.30\%$ and $0.27\%$ in the $x$ and $y$ directions, respectively. The lines representing the 'uncracked rigidity' and the 'cracked rigidity' are also shown on Figure 3-24. As for S2S-1 and S2S-2, relatively little deflection occurred after the onset of yielding of the reinforcement (at a mid-panel deflection of about 23 mm) and hence relatively little deflection was associated with
plastic deformation of the reinforcement. As was the case for slab S2S-1, the bulk of the total deflection is associated by the loss of stiffness due to cracking and the load-deflection curve terminates before the line representing the fully-cracked rigidity is reached.

The slab started to crack at a total load of 31.0 kN when the mid-panel deflection was 2.4 mm. The applied load was gradually increased until the reinforcement in the y-direction began to yield at a load of about 44.3 kN when the mid-panel deflection reached 22.6 mm. The peak load of 47.4 kN was reached at a mid-panel deflection of 34.7 mm. The slab then began to unload as the deformation was gradually increased. The slab failed by fracture of the reinforcing steel resulting in sudden catastrophic collapse. The maximum measured deflection prior to collapse was 48.5 mm.

Figure 3-25a is a photograph of the slab under load and Figure 3-25b is an elevation of the slab showing the critical crack at mid-span after failure. Figure 3-25c shows the top surface of the slab above the critical crack just after fracturing of the tensile reinforcement and collapse of the slab (onto the supporting test frame supports below).

![Slab S2R-2](image)

Figure 3-25: Slab S2R-2 (a) under load, (b) critical section after failure, and (b) top surface of failed slab.

Figure 3-26 shows the load-deflection curve and the absorbed work-deflection curve for S2R-2. The load-deflection curve started to plateau at a total load of Peak load/(f_m/f_y), i.e. 44.7 x (1/1.07) = 44.3 kN. This occurs at a mid-panel deflection of \( \Delta_1 = 22.6 \) mm. The load-deflection curve dropped significantly when the reinforcing wires fractured at \( \Delta_2 = 48.5 \) mm. The total plastic work absorbed by the slab between \( \Delta_1 \) and \( \Delta_2 \) is \( W_1 = 1180 \) kN.mm, while the work from first loading to \( \Delta_1 \) is \( W_0 = 830 \) kN.mm.
Figure 3-26: Load and work versus mid-panel deflection for slab S2R-2.

Figure 3-27 shows the change in the four corner reactions with slab deflection. As observed in S2S-1 and S2S-2, some relatively small differences in reactions began to occur after cracking with the diagonally reactions (LC2 and LC5) being somewhat smaller than the other two reactions (LC3 and LC 4). These differences are thought to be due to the loss of symmetry after cracking and variations in the deformational characteristics of the individual wires after first yield.

Figure 3-27: Supports reaction versus mid-panel deflection for slab S2R-2.

Figure 3-28 shows the applied moment about the x-axis at mid-span versus mid-panel deflection, together with the flexural capacity about the same axis (i.e. the moment capacity provided by the reinforcement in the y-direction).

The variation of concrete compressive strains in the x and y directions at the mid-panel are shown in Figure 3-29. Due to the significantly higher moment associated with the longer span, the strain in the y direction is consistently higher than that in the x direction throughout the
Figure 3-28: Total applied moment about the x-axis and moment capacity for slab S2R-2.

Test. At the peak load (when mid-panel deflection was 34.7 mm), the maximum concrete compressive strain was less than 2600 × 10\(^6\) and the concrete showed no sign of crushing. However, in the unloading phase after the peak load had been achieved, the compressive concrete strain in the y-direction increased substantially.

Figure 3-29: Concrete surface compressive strains in x and y directions at mid-panel of S2R-2.

3.1.5 Slab S2R-4

The load versus mid-panel deflection (LVDT 1) curve for S2R-4 is shown in Figure 3-30. This slab was reinforced with 12 mm diameter (Class N) deformed bar at 300 mm centres in each direction, i.e., twelve bars in the x-direction (\(A_s = 1357 \text{ mm}^2\) and \(p = 0.51\%\)) and eight bars in the y-direction (\(A_s = 905 \text{ mm}^2\) and \(p = 0.44\%\)). The lines representing the 'uncracked rigidity' and the 'cracked rigidity' are also shown on Figure 3-30. For this slab containing Class N deformed bars, the load-deflection curve is characterized by a large plastic plateau after first yield of the
reinforcement in the y-direction. With large plastic deformations without any loss of load
carrying capacity, structural behaviour was ductile. The test was terminated before the slab
reached its ultimate strength or its ultimate deformation.

The slab started to crack at a total load of 33.5 kN when the mid-panel deflection was 1.21
mm. The applied load was gradually increased until the reinforcement in the y-direction began to
yield at a load of about 62 kN when the mid-panel deflection reached 39.0 mm. After yielding,
the slab continued to deform and the applied load continued to increase until the test was
terminated at a mid-span deflection of 135.8 mm when the soffit of the deformed slab came in
contact with the testing frame (see Figure 3-31b). From Figure 3-30, it can be seen that the
contribution of the tensile concrete to the rigidity of the slab is still very significant at the onset of
yielding of the tensile reinforcement.

![Figure 3-30: Load versus mid-panel deflection for slab S2R-4.](image)

Figure 3-31a and Figure 3-31b are photographs of the slab under load just before the test was
terminated and Figure 3-31c is the soffit of the slab showing the flexural cracks at the mid-span
region just before the test was terminated.

Figure 3-32 shows the load-deflection curve and the absorbed work-deflection curve for S2R-
4. The load-deflection curve started to plateau at a total load value of about 62 kN at a mid-panel
deflection of $\Delta_1 = 39.0$ mm and the applied load very gradually increased to 67.1 kN when the
test was terminated at $\Delta_2 = 135.8$ mm. The total plastic work absorbed by the slab after first
yielding (i.e., between $\Delta_1$ and $\Delta_2$) is $W_1 = 6250$ kN.mm and the elastic work prior to yielding is
$W_0 = 1880$ kN.mm.

Figure 3-33 shows the change in the four corner reactions with slab deflection. For this slab,
the four reactions show similar trends throughout the test at all levels of loading.
Figure 3-31: Slab S2R-4 - (a) slab under load showing large plastic deformation; (b) test was terminated because slab touched the steel supporting frame; and (c) slab soffit showing cracks just prior to the termination of the test.

Figure 3-32: Load and work versus mid-panel deflection for slab S2R-4.
Figure 3-33: Supports reaction versus mid-panel deflection for slab S2R-4.

Figure 3-34 shows the applied moment about the x-axis at mid-span versus mid-panel deflection. The figure also shows the slab flexural capacity about the same axis (i.e. the moment capacity provided by the reinforcement in the y-direction). For this slab containing Class N bars, the maximum moment resisted by the reinforcement in the y-direction was only 94% of the calculated moment capacity (obtained assuming the stress in the reinforcement at peak capacity is \( f_{su} \)). However, the test was terminated before the moment capacity of the specimen was reached.

Figure 3-34: Moment about the x-axis versus mid-panel deflection and calculated moment capacity provided by the reinforcement in the y-direction of slab S2R-4.

The deflection contours of the slab at a mid-panel deflection of \( \Delta = 46.0 \) mm soon after first yield of the reinforcement in the y-direction are shown in Figure 3-35. It is noted that the deflected shape of the two-way slab resembles that of a one-way slab bending in the y-direction. On the column lines in the x-direction (i.e. at \( y = 0.0 \) and \( y = 3.28 \) m), the sag due to bending
about an axis in the y-direction is very small (< 3 mm), while on the line parallel to the x-axis through the mid-span (i.e. when y = 1.64 m), the deflection at the mid-panel is less than the deflection on the column lines. At the mid-panel (where x = 1.04 m and y = 1.64 m), the measured deflection was 46 mm, while on the column lines (where x = 0 and x = 2.08 m), the measured deflection was 50 mm.

![Diagram showing deflection contours for slab S2R-4 at a load of 62.7 kN when mid-panel deflection was 46.0 mm.]

**Figure 3-35:** Deflection contours for slab S2R-4 at a load of 62.7 kN when mid-panel deflection was 46.0 mm.

### 3.2 Summary and Discussion of Results

Figure 3-36 and Figure 3-37 show the load-deflection curves for the square and rectangular slabs, respectively. The x-axis in these figures, and all subsequent figures, represent the deflection of each slab at its geometric centroid (i.e. at the mid-panel of the slab). For each specimen reinforced with Class L welded wire fabric (S2S-1, S2S-2, S2S-3, S2R-1, S2R-2 and S2R-3), the load-deflection plot has a similar shape. The curve rises to a peak value (when the stress in the reinforcement reaches $f_{wu}$) and then immediately begins to decrease, with little evidence of the horizontal plastic plateau that characterizes the behaviour of under-reinforced slabs containing conventionally ductile reinforcement. When tested under displacement control, a relatively short unloading period of the specimen is observed after the peak load is reached followed by an abrupt termination of the curve when the reinforcement fractures and the slab collapses. Had the slabs
been tested under load control, collapse would have occurred when the peak load was first reached and the short post-peak unloading curve would not have been observed.

All slabs containing Class L reinforcement failed by fracture of the reinforcement and, with the exception of S2S-3, crushing of the compressive concrete on the critical section did not occur. It is noted that S2S-3 contained a relatively ductile welded wire fabric (with $\varepsilon_u = 3.73\%$ and $f_{sw}/f_{y} = 1.12$ - see Table 2-3).

In contrast, the slabs containing Class N bars (S2S-4, S2S-5, S2S-6, S2R-4, and S2R-5) showed a high capacity to absorb work and did not suddenly collapse. Only S2S-4 actually failed, and that was due to a punching shear failure under the applied load. The tests for the remaining Class N slabs were all terminated when either the maximum travel of the actuator was reached or the space available in the testing frame for deflection was exhausted.

**Figure 3-36:** Load-deflection curves for the square corner-supported two-way slabs.

**Figure 3-37:** Load-deflection curves for the rectangular corner-supported two-way slabs.
Table 3-3 summarizes the cracking and peak loads of the slabs, together with the corresponding mid-panel deflections.

<table>
<thead>
<tr>
<th>Slab</th>
<th>First Cracking</th>
<th>Peak</th>
<th>End of test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load (kN)</td>
<td>Load (kN)</td>
<td>Load (kN)</td>
</tr>
<tr>
<td>S2S-1</td>
<td>41.3</td>
<td>43.0</td>
<td>23.0</td>
</tr>
<tr>
<td>S2S-2</td>
<td>38.6</td>
<td>65.8</td>
<td>36.3</td>
</tr>
<tr>
<td>S2S-3</td>
<td>43.2</td>
<td>119.5</td>
<td>102.8</td>
</tr>
<tr>
<td>S2S-4</td>
<td>46.0</td>
<td>88.7</td>
<td>97.1</td>
</tr>
<tr>
<td>S2S-5</td>
<td>46.2</td>
<td>132.9</td>
<td>150.9</td>
</tr>
<tr>
<td>S2S-6</td>
<td>42.1</td>
<td>114.7</td>
<td>186.8</td>
</tr>
<tr>
<td>S2R-1</td>
<td>33.2</td>
<td>35.8</td>
<td>33.2</td>
</tr>
<tr>
<td>S2R-2</td>
<td>31.0</td>
<td>47.4</td>
<td>49.4</td>
</tr>
<tr>
<td>S2R-3</td>
<td>38.3</td>
<td>68.8</td>
<td>60.0</td>
</tr>
<tr>
<td>S2R-4</td>
<td>33.5</td>
<td>67.1</td>
<td>135.8</td>
</tr>
<tr>
<td>S2R-5</td>
<td>31.7</td>
<td>84.3</td>
<td>184.0</td>
</tr>
</tbody>
</table>

Table 3-4 lists the experimental and calculated peak moments (ultimate moments) for each test specimen and the % difference between them. The calculated value of ultimate moment was obtained by assuming:

(i) the strain diagram is linear (i.e. plane sections remain plane);

(ii) the stress in the reinforcement at peak load is \( f_m \);

(iii) the concrete in tension at the critical section carries no stress (i.e. no aggregate interlock on the irregular cracked surface); and

(iv) the concrete compressive stress block is rectangular with dimensions in accordance with AS3600-2001.

<table>
<thead>
<tr>
<th>Slab Name</th>
<th>Ultimate moment (kN.m)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Experimental</td>
<td>Calculated</td>
</tr>
<tr>
<td>S2S-1</td>
<td>18.4</td>
<td>16.3</td>
</tr>
<tr>
<td>S2S-2</td>
<td>29.7</td>
<td>25.9</td>
</tr>
<tr>
<td>S2S-3</td>
<td>45.2</td>
<td>35.6</td>
</tr>
<tr>
<td>S2S-4</td>
<td>43.8</td>
<td>43.8</td>
</tr>
<tr>
<td>S2S-5</td>
<td>50.7</td>
<td>46.1</td>
</tr>
<tr>
<td>S2S-6</td>
<td>44.1</td>
<td>41.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Slab Name</th>
<th>Ultimate moment (kN.m)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Experimental</td>
<td>Calculated</td>
</tr>
<tr>
<td>S2R-1</td>
<td>21.5</td>
<td>18.4</td>
</tr>
<tr>
<td>S2R-2</td>
<td>30.1</td>
<td>28.2</td>
</tr>
<tr>
<td>S2R-3</td>
<td>50.1</td>
<td>47.9</td>
</tr>
<tr>
<td>S2R-4</td>
<td>48.9</td>
<td>51.2</td>
</tr>
<tr>
<td>S2R-5</td>
<td>52.0</td>
<td>51.2</td>
</tr>
</tbody>
</table>

1 Test terminated when the sample failed in punching shear.
2 Test terminated because loading actuator reached its maximum travel distance.
3 Test terminated because slab touched the underneath supporting steel frame.
When applied to slabs containing Class L welded wire fabric, the first and second of these assumptions are reasonable, the third and fourth are not. Calculated ultimate capacities are significantly less than the measured values. It appears that a significant component of friction (aggregate interlock) may exist in the direction of the tensile reinforcement on the cracked region of the critical section, thereby providing a significant additional internal tensile force on the critical cross-section to carry a portion of the ultimate moment. In addition, the rectangular stress block for concrete in compression is not appropriate when the ultimate strength is controlled by fracture of the tensile reinforcement and not by crushing of the compressive concrete.

The slabs reinforced with Class L wires demonstrated low capacity to absorb energy in comparison with slabs reinforced with Class N bars. Table 3-5 lists the work done in the various stages of each slab test. The work is obtained by calculating the area under the load deflection curve. $W_0$ represents the work done by the applied load in deforming the slab in the elastic range from first loading up until first yielding of the reinforcement at a mid-panel deflection of $\Delta_1$. $W_1$ represents the work done in deforming the slab in the plastic range (i.e. the plastic plateau) between the deflection $\Delta_1$ and the deflection $\Delta_2$ at which point either the reinforcement fractures or the slab has unloaded to 75% of its peak value. A measure of the ductility of the slab is the ratio $W_1/W_0$. Another way to express the ductility of the slabs is by calculating the relative ductility, $(\Delta_2-\Delta_1)/\Delta_1$, and this is also given in Table 3-5, together with the uniform elongation ($\varepsilon_{\text{un}}$) of the reinforcement in each slab. The ductility measures are misleading in the case of S2S-1 and S2R-1, where the steel yields at first cracking, and are therefore not provided. The ductility measures are also not provided for S2S-4 which failed in punching shear and not in flexure.

Clearly, as the ductility of the reinforcement increases, the ability of the slab to absorb energy/work by deforming plastically increases significantly. The slabs containing Class L reinforcement are far less ductile than those containing Class N bars.

Table 3-5: Peak moments, deflections, and work done by the tested slabs.

<table>
<thead>
<tr>
<th>Slab Name</th>
<th>$M_{\text{max}}$ (kN.m)</th>
<th>$\Delta_1$</th>
<th>$\Delta_2$</th>
<th>$(\Delta_2-\Delta_1)$</th>
<th>Work (kN.mm)</th>
<th>Ductility Measures</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$W_0$</td>
<td>$W_1$</td>
<td>$W_1/W_0$</td>
<td>$(\Delta_2-\Delta_1)/\Delta_1$</td>
<td>$\varepsilon_{\text{un}}$ (%)</td>
</tr>
<tr>
<td>S2S-1</td>
<td>18.4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1075</td>
<td>0.72</td>
</tr>
<tr>
<td>S2S-2</td>
<td>29.7</td>
<td>21.8</td>
<td>33.9</td>
<td>12.1</td>
<td>772</td>
<td>1.52</td>
</tr>
<tr>
<td>S2S-3</td>
<td>45.2</td>
<td>52.0</td>
<td>102.8</td>
<td>50.8</td>
<td>5800</td>
<td>4.40</td>
</tr>
<tr>
<td>S2S-4</td>
<td>43.8</td>
<td>40.0</td>
<td>97.0</td>
<td>57.0</td>
<td>5060</td>
<td>3080</td>
</tr>
<tr>
<td>S2S-5</td>
<td>50.7</td>
<td>40.8</td>
<td>151</td>
<td>110</td>
<td>5600</td>
<td>3000</td>
</tr>
<tr>
<td>S2S-6</td>
<td>44.1</td>
<td>39.4</td>
<td>187</td>
<td>148</td>
<td>5060</td>
<td>3000</td>
</tr>
<tr>
<td>S2R-1</td>
<td>21.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S2R-2</td>
<td>30.1</td>
<td>22.6</td>
<td>48.5</td>
<td>25.9</td>
<td>830</td>
<td>1180</td>
</tr>
<tr>
<td>S2R-3</td>
<td>50.1</td>
<td>25.4</td>
<td>59.8</td>
<td>34.4</td>
<td>1260</td>
<td>2300</td>
</tr>
<tr>
<td>S2R-4</td>
<td>48.9</td>
<td>39.0</td>
<td>135.8</td>
<td>96.8</td>
<td>6250</td>
<td>1880</td>
</tr>
<tr>
<td>S2R-5</td>
<td>52.0</td>
<td>51.3</td>
<td>184</td>
<td>133</td>
<td>10570</td>
<td>2565</td>
</tr>
</tbody>
</table>

1. Test terminated when the sample failed in punching shear.
2. Test terminated because loading actuator reached its maximum travel distance.
3. Test terminated because slab touched the underneath supporting steel frame.
In the analysis and design of reinforced concrete structures, many assumptions are routinely made that are only appropriate and reasonable if the individual elements of the structure are ductile. For reinforced concrete flexural members, a ductility ratio $W_1/W_0$ of greater than 2.0 is usually considered to be necessary to justify the use of elastic analysis to determine the internal actions, or if the effect of support settlement is not to be considered. A significantly higher ductility ratio may be required if plastic analysis is used involving a significant level of moment redistribution. A still higher level is required, if the structure is to withstand dynamic forces, such as seismic, blast or impact loads.

When the ductility ratio for a flexural member is less than 2.0 (such as occurs for example when the reinforcement ratio exceeds 75% of the balanced reinforcement ratio), it is generally considered to be non-ductile. To compensate for the reduced ductility of such members, codes of practice such as AS3600-2001 reduce the strength reduction factor, thereby reducing the design strength of non-ductile members.

Figure 3-38 shows a plot of the ductility ratio $W_1/W_0$ versus the uniform elongation of the reinforcement $\varepsilon_{u,\ell}$ (%) for the slabs tested here. The trend is clear. The full line is the line of best fit for the values of $W_1/W_0$ listed in Table 3-5. However, the dashed line may be more appropriate as the values of $W_1$ for each of the slabs containing Class N bars is underestimated, because the tests were terminated before the specimen actually failed. All slabs containing welded wire fabric have ductility ratios $W_1/W_0$ less than 2.0 and all such slabs are therefore considered to be unsatisfactorily brittle. It would appear that the recent amendment to AS3600-2001, introducing a 20% penalty on the strength of slabs containing Class L reinforcement, is reasonable and appropriate. In effect this 20% penalty is equivalent to a reduction of the strength reduction factor for flexural members containing Class L reinforcement from 0.8 to 0.64.

![Figure 3-38: Ductility ratio (W_1/W_0) versus uniform elongation, \(\varepsilon_{u,\ell}\) (%).](image)

Figure 3-3738 also shows that the minimum uniform elongation specified for Class L reinforcement in the Australian Standards ($\varepsilon_{u,\ell} = 1.5\%$) results in ductility ratios significantly less than 1.0 and this is considered to be unsatisfactorily brittle. The use of such reinforcement in suspended reinforced concrete slabs is not recommended.
4. Conclusions and Summary

The following conclusions are drawn from the results of the experimental program presented in this report:

1. The two-way corner supported slabs reinforced with Class L welded wire fabric fail in a brittle mode by fracture of the tensile reinforcement and, generally, not by crushing of the compressive concrete.

2. Two-way corner-supported slabs containing Class L welded wire fabric are unable to undergo significant plastic deformation without a significance reduction in the applied load. This is true for both square and rectangular slabs.

3. All slabs containing Class L welded wire fabric had ductility ratios $W_1/W_0$ less than 2.0.

4. The current procedures for the design and analysis of reinforced concrete slabs at the ultimate limit state have been developed based on the assumption that the reinforcing steel is elastic-plastic with unlimited strain capacity. This is not the case when using Class L reinforcing steel and the usual procedures and the conventional understanding of the ultimate load behaviour of under-reinforced slabs are not applicable. For example, with the compressive concrete still in the elastic range at collapse, the use of Whitney’s equivalent rectangular stress block in the prediction of ultimate strength is not valid. With strain localization, significant tensile force is carried by the cracked concrete at the ultimate limits state and the assumption that the concrete carries no tensile force on a cross-section at ultimate is also not valid.

5. The brittle nature of the failure of the slabs containing Class L reinforcement supports the recent amendment to AS3600-2001, wherein the strength reduction factor for slabs is effectively and appropriately reduced from $\phi = 0.8$ for Class N steel to $\phi = 0.64$ for Class L reinforcement. Such a reduction in $\phi$ is consistent with the code approach for brittle members where the ductility ratio $W_1/W_0$ is less than 2.0.
5. References


6. Acknowledgements

This study was undertaken as part of an ARC Discovery project (DP0558370) in which the second author was awarded an Australian Professorial Fellowship. The support of the Australian Research Council is gratefully acknowledged.
Appendix A  Detailed Results for Slabs S2S-4, S2S-5 and S2S-6 and Slabs S2R-1, S2R-3 and S2R-5

A.1. Slab S2S-4

![Graph showing load versus mid-panel deflection for S2S-4.](image1)

**Figure A-1:** Load versus mid-panel deflection for S2S-4.

![Deflected slab before failure.](image2)

(a)

![Slab soffit showing punching shear failure.](image3)

(b)

**Figure A-2:** Slab S2S-4 (a) deflected slab before failure, and (b) slab soffit showing punching shear failure under point load at mid-panel.
Figure A-3: LVDT readings versus mid-panel deflection for S2S-4.

Figure A-4: Load and work versus mid-panel deflection for slab S2S-4.

Figure A-5: Supports reaction-deflection curves for slab S2S-4.
Figure A-6: Total applied moment about the y-axis and moment capacity for slab S2S-4.

(a) At a pre-cracking load of 45kN.  
(b) At a load of 86.9kN.

Figure A-7: Deflection contours for slab S2S-4.
A.2. Slab S2S-5

Figure A-8: Load versus mid-panel deflection for slab S2S-5.

Figure A-9: Slab S2S-5 (a) cracks at regular spacing along much of the span; (b) ductile behaviour - high deformation while maintaining much of the peak load (c) concrete crushing at critical section without reinforcement fracturing.
Figure A-10: LVDT and laser readings versus mid-panel deflection for slab S2S-5.

Figure A-11: Load and work versus mid-panel deflection for slab S2S-5.

Figure A-12: Supports reaction-deflection curves for slab S2S-5.
**Figure A-13:** Total applied moment about the y-axis and moment capacity for slab S2S-5.

**Figure A-14:** Strain in the steel wires running along the x and y directions in slab S2S-5.

(a) At a pre-cracking load of 40kN  
(b) At peak load of 132.8kN  

**Figure A-15:** Deflection contours for slab S2S-5
A.3. Slab S2S-6

Figure A-16: Load versus mid-panel deflection for S2S-6.

Figure A-17: Slab S2S-6 (a) high ductility under sustained load; (b) concrete crushing at critical section without reinforcement fracturing; (c) main crack at critical section under peak.
Figure A-18: LVDT and laser readings versus mid-panel deflection for slab S2S-6.

Figure A-19: Load and work versus mid-panel deflection for slab S2S-6.

Figure A-20: Supports reaction versus mid-panel deflection for slab S2S-6.
Figure A-21: Total applied moment about the y-axis and moment capacity for slab S2S-6.

Figure A-22: Strain in the steel wires running along the x and y directions in slab S2S-6.

(a) At pre-peak load of 50kN.  (b) At post-peak load of 90kN.

Figure A-23: Deflection contours for slab S2S-6.
A.4. Slab S2R-1

![Graph showing load versus mid-panel deflection for slab S2R-1.](image)

**Figure A-24:** Load versus mid-panel deflection for slab S2R-1.

![Images of slab S2R-1 after failure and close-up of cracks.](image)

**Figure A-25:** Slab S2R-1 (a) after failure, (b) side of slab showing critical crack at mid-span with very fine adjacent cracks, and (c) slab soffit after failure.
Figure A-26: Load and work versus deflection for slab S2R-1.

Figure A-27: Supports reaction versus mid-panel deflection for slab S2R-1.

Figure A-28 Total applied moment about the x axis and moment capacity for slab S2R-1.
Figure A-29: Concrete surface compressive strains along x and y directions at the origin of slab S2R-1.

Figure A-30: Deflection contours for slab S2R-1 at a post-peak load of 34.4kN.
A.5. Slab S2R-3

Figure A-31: Load versus deflection for slab S2R-3.

Figure A-32: Slab S2R-3 (a) failed slab, (b) side of slab showing crack locations just before failure, and (c) slab soffit at critical section just before failure.
Figure A-33: LVDT readings versus mid-panel deflection for slab S2R-3.

Figure A-34: Load and work versus mid-panel deflection for slab S2R-3.

Figure A-35: Supports reaction versus mid-panel deflection for slab S2R-3.
Figure A-36: Total applied moment about the x-axis and moment capacity for slab S2R-3.

Figure A-37: Deflection contours for slab S2R-3 at a post-peak load of 67.7kN.
A.6. Slab S2R-5

Figure A-38: Load versus mid-panel deflection for slab S2R-5.

Figure A-39: Slab S2R-5 (a) slab soffit with many cracks, (b) large ductility of slab without collapse, and (b) cracks spacing of 100mm along the long span.
Figure A-40: LVDT and laser readings versus mid-panel deflection for slab S2R-5.

Figure A-41: Load and work versus mid-panel deflection for slab S2R-5.

Figure A-42: Supports reaction versus mid-panel deflection for slab S2R-5.
Figure A-43: Total applied moment about the x axis and moment capacity for slab S2R-5.

Figure A-44: Strains in the steel wires in slab S2R-5.

Figure A-45: Deflection contours for slab S2R-5 at a load of 64.0kN