This thesis has been submitted in fulfilment of the requirements for a postgraduate degree (e.g. PhD, MPhil, DClinPsychol) at the University of Edinburgh. Please note the following terms and conditions of use:

This work is protected by copyright and other intellectual property rights, which are retained by the thesis author, unless otherwise stated.
A copy can be downloaded for personal non-commercial research or study, without prior permission or charge.
This thesis cannot be reproduced or quoted extensively from without first obtaining permission in writing from the author.
The content must not be changed in any way or sold commercially in any format or medium without the formal permission of the author.
When referring to this work, full bibliographic details including the author, title, awarding institution and date of the thesis must be given.
The Fire Performance of Restrained Polymer-Fibre-Reinforced Concrete Composite Slabs

David Christopher Alexander Fox

A thesis presented for the degree of
Doctor of Philosophy

BRE Centre for Fire Safety Engineering
School of Engineering
Institute for Infrastructure and Environment
The University of Edinburgh
2013
The work outlined in this thesis has been carried out solely by myself at the BRE Centre for Fire Safety Engineering, University of Edinburgh under the supervision of Dr. Tim Stratford and Dr. Martin Gillie. Where external sources were used, appropriate reference is made.

David Fox, 2013
Abstract

Composite slab flooring systems for steel-framed buildings consist of a profiled steel deck and a cast in-situ slab. The slab traditionally includes a layer of light gauge steel mesh reinforcement. This mesh is placed near the surface, which controls the early-age cracking caused by concrete drying and shrinkage. The steel mesh also performs a vital structural role at high temperatures. Structural fire tests and numerical investigations over the last 15 years have established that the mesh can provide enhanced fire resistance. A load-carrying mechanism occurs in fire with the mesh acting as a tensile catenary, spanning between perimeter supports. This structural mechanism is currently utilised regularly in the performance-based fire engineering design of steel-framed buildings.

In a recent development, this mesh can be removed by using concrete with dispersed polymer fibre reinforcement to form the composite slab. The polymer-fibre-reinforced concrete (PFRC) is poured onto the deck as normal, and the fibres resist early crack development. For developers this technique has several advantages over traditional reinforcing mesh, such as lower steel costs, easier site operations and faster construction.
However, to date the fire resistance of such slabs has been demonstrated only to a limited extent. Single element furnace tests with permissible deflection criteria have formed the basis for the fire design of such slabs. But these have not captured the full fire response of a structurally restrained fibre-reinforced slab in a continuous frame. The polymer fibres dispersed throughout the slab have a melting point of 160°C, and it is unclear how they contribute to overall fire resistance. In particular, there has been no explanation of how such slabs interact with the structural perimeter to maintain robustness at high deflections.

This project was designed to investigate the structural fire behaviour of restrained polymer-fibre-reinforced composite slabs. An experimental series of six slab experiments was designed to investigate the effects of fibre reinforcement and boundary restraint. A testing rig capable of recording the actions generated by the heat-affected slab was developed and constructed. Model-scale slab specimens were tested with different reinforcement and perimeter support conditions, to establish the contributions to fire resistance of the polymer fibres and applied structural restraint.
Publications

Conference papers


D. Fox, A. S. Usmani, D. Lange: An Analysis of the “Compressive Ring” in Reinforced Concrete Slabs in Fire; 7th International Congress – Concrete: Construction’s Sustainable Option, Dundee, July 2008

D. Fox, T. Stratford: The Fire Performance of Polymer Fibre Reinforced Composite Concrete Slabs; 6th International Conference Structures in Fire, University of Michigan, June 2010
Acknowledgements

Funding for this project has been provided by Arup Fire, Grace Construction Products, Richard Lees Steel Decking and the EPSRC.

Sincere thanks to my supervisor Dr. Tim Stratford for his patient guidance and experience throughout this project. I am also grateful for the help, advice and encouragement from Prof. José Torero. I am especially thankful for the co-operation and expertise of Dr. Barbara Lane and Dr. Graeme Flint of Arup Fire. Acknowledgement is also made to Adrian Shepherd of Richard Lees Steel Decking, and Dr. Klaus-Alexander Rieder and Richard Hoare from Grace for their information and services.

For assistance in the fabrication and operation of the testing rig, and much more besides, my gratitude is given to Derek Jardine, Jim Hutcheson and Michal Krajcovic.

Finally, thank you to Margaret Rose, to Andrew, and to my other family and friends for their patience and unwavering support.
Contents

Declaration iii
Abstract iv
Publications vi
Acknowledgements vii

Chapter 1 Introduction 1

1.1 Project Background 2
1.2 Project Aims 4
1.3 Thesis Outline 4

Chapter 2 Fibre Reinforcement of Concrete 7

2.1 Development and Applications of Fibre Reinforced Concrete 8
2.1.1 Introduction to Fibre Reinforced Concrete 8
2.1.2 Historical and Industrial Development 9
2.1.3 Scientific Research into FRC 11
    Initial Investigations 11
    Fabrication and Dosage 14
2.1.4 Development of Other Fibre Materials 16

Synthetic Polymer Fibre Reinforced Concrete 16
Glass Fibres 17
Natural Fibres 18
High Modulus Synthetic Fibres 18

2.1.5 Current Applications of Fibre Reinforced Concrete 19

Slab-on-Ground Construction and Paving 19
Shotcrete 19
Thermal Spalling Mitigation 20
Ultra High Performance Concrete 21
Fibre Reinforced Concrete Composite Slabs 22

2.1.6 Classification of Fibres for Concrete Reinforcement 23

2.2 Effects of Fibre Reinforcement and Mechanical Behaviour of FRC at Ambient Temperature 25

2.2.1 Plain Concrete in Tension 25

2.2.2 Cracking of FRC Elements and Load-Deflection Curves 26

2.2.3 Function of Reinforcing Fibres 27

2.2.4 Fibre Pull-Out, Fibre Breakage and Critical Length 28

2.2.5 Fibre Reinforcement in Fresh Concrete Before Hardening 30

2.3 Testing Procedures for Fibre Reinforced Concrete 31

2.3.1 Flexural Strength and Toughness Tests 31

2.3.2 Issues with Standardised Toughness Test Methods 34

2.3.3 The Wedge Splitting Test 35

2.3.4 Further Flexural FRC Tests 37

2.4 The Strux 90/40 Fibre 39

2.4.1 Introduction, Shape and Properties 39

2.4.2 Fibre Strength at Elevated Temperatures 40

2.4.3 Properties of Strux 90/40 Concrete from Flexural Tests and...
Wedge Splitting Tests 41

2.4.4 First Laboratory Investigations into PFRC Material 44
  Test 1 - Effect of Different Water-Cement Ratios 44
  Test 2 - Effect on Strength due to Incorporation of Fibres 46

Chapter 2 Summary 47

Chapter 3 Behaviour of Composite Slabs in Fire 48

3.1 Introduction 49
  3.1.1 Composite Steel-Concrete Slabs 49
  3.1.2 Steel Material Behaviour at High Temperatures 51
  3.1.3 Concrete Material Behaviour at High Temperatures 53
  3.1.4 Fire Behaviour 56
    Design Fires 57
  3.1.5 Composite Slab Design and Fire Rating 58

3.2 Fire Resistance Testing of Composite Slabs 60
  3.2.1 Test Construction and Features 60
    Conditioning 61
    Furnace 61
    Loading 61
    Test Instrumentation 62
  3.2.2 Performance Criteria 62
  3.2.3 Testing and Boundary Conditions 64
  3.2.4 Observed Fire Behaviour and the Cardington Experiments 66

3.3 Slab Behaviour and Performance-Based Design 69
  3.3.1 Membrane Action in Composite Slabs at High Temperatures 69
  3.3.2 Performance-Based Design and Fire Protection 71

3.4 Polymer Fibre Reinforced Concrete Composite Slabs 73
  3.4.1 Design and Construction 73
Chapter 3 Summary

Chapter 4 Experimental Design and Methodology

4.1 General Arrangement and Test Sequence
4.1.1 General Overview of the Experimental Scheme
4.1.2 Features of the Experimental Design to Address the Project Aims
4.1.3 Experimental Sequence and Test Parameters

4.2 Detailed Design of Experimental Systems
4.2.1 Slab Specimen Design and Fabrication
   Slab Dimensions and Features
   Fibre Reinforced Concrete Mix Design
   Casting and Specimen Preparation
4.2.2 Construction and Design of Restraining Frame
   Portal Frames
   Edge Restraint Arrangement
4.2.3 Radiant Panel Installation
4.2.4 Load Application System
4.2.5 Measurement of Slab Deflections

4.3 Reaction Force Measurement with Strain Gauges
4.3.1 Introduction
4.3.2 Arrangement and Design
   Perimeter Gauges
   Column Gauges
   Bracing Gauges
4.3.3 Calibration of Gauges

4.4 A Priori Numerical Analysis
Chapter 4 Summary

Chapter 5 Experimental Results

5.1 Results from Experiment 1: Ambient Load Test of a Fully Restrained PFRC Slab

5.1.1 Experiment 1 Failure Mode and Load-Deflection Results
5.1.2 Experiment 1 Reaction Force Results
5.1.3 Experiment 1 Reaction Moment Results
5.1.4 Experiment 1 Discussion
5.1.5 Experiment 1 Summary

5.2 Results from Experiment 2: First Heated Test of a Fully Restrained PFRC slab

5.2.1 Experiment 2A Heating and Cooling Test Results
5.2.1.1 Experiment 2A Heating and Cooling Temperature Results
5.2.1.2 Experiment 2A Heating and Cooling Deflection Results
5.2.1.3 Experiment 2A Heating and Cooling Reaction Force Results
5.2.1.4 Experiment 2A Heating and Cooling Reaction Moment Results
5.2.1.5 Experiment 2A Heating and Cooling Discussion
5.2.2 Experiment 2B Re-loading Test Results
5.2.2.1 Experiment 2B Re-loading Load-Deflection Results
5.2.2.2 Experiment 2B Re-loading Reaction Force Results
5.2.2.3 Experiment 2B Re-loading Reaction Moment Results
5.2.2.4 Experiment 2B Re-loading Discussion
5.2.3 Experiment 2 Summary

5.3 Results from Experiment 3: Second Heated Test of
Fully Restrained PFRC Slab  

5.3.1 Experiment 3 Temperature Results  
5.3.2 Experiment 3 Failure Mode and Deflection Results  
5.3.3 Experiment 3 Reaction Force Results  
5.3.4 Experiment 3 Discussion  
5.3.5 Experiment 3 Summary  

5.4 Results from Experiment 4: Heated Test of Fully Restrained Steel Mesh Reinforced Concrete Slab  

5.4.1 Experiment 4A Heating and Cooling Test Results  
  5.4.1.1 Experiment 4A Heating and Cooling Temperature Results  
  5.4.1.2 Experiment 4A Heating and Cooling Deflection Results  
  5.4.1.3 Experiment 4A Heating and Cooling Reaction Force Results  
  5.4.1.4 Experiment 4A Heating and Cooling Reaction Moment Results  
  5.4.1.5 Experiment 4A Heating and Cooling Discussion  
5.4.2 Experiment 4B Ambient Re-loading Test Results  
  5.4.2.1 Experiment 4B Re-loading Load-Deflection Results  
  5.4.2.2 Experiment 4B Re-loading Reaction Force Results  
  5.4.2.3 Experiment 4B Re-loading Reaction Moment Results  
  5.4.2.4 Experiment 4B Re-loading Discussion  
5.4.3 Experiment 4 Summary  

5.5 Results from Experiment 5: Heated Test of a Fully Restrained Plain Concrete Slab  

5.5.1 Experiment 5 Temperature Results  
5.5.2 Experiment 5 Failure Mode and Deflection Results  
5.5.3 Experiment 5 Reaction Force Results  
5.5.4 Experiment 5 Reaction Moment Results  
5.5.5 Experiment 5 Discussion  
5.5.6 Experiment 5 Summary  

5.6 Results from Experiment 6: Heated Test of PFRC Slab
Without Horizontal or Rotational Restraint 185
5.6.1 Experiment 6 Temperature Results 185
5.6.2 Experiment 6 Failure Mode and Deflection Results 187
5.6.3 Experiment 6 Reaction Force Results 190
5.6.4 Experiment 6 Reaction Moment Results 190
5.6.5 Experiment 6 Discussion 191
5.6.6 Experiment 6 Summary 192

5.7 Strength and Moisture Content Results from Slab Concrete Specimens 193
5.7.1 Compressive and Tensile Strength of Slab Concrete from Sample Specimens 193
5.7.2 Moisture Content of Slab Concrete from Sample Cylinder Specimens 195

Chapter 5 Summary 197

Chapter 6 Analysis and Discussion 198

6.1 Thermal Behaviour 199
6.1.1 Temperature Evolution through Slab 199
6.1.2 Thermal Profiles during Heated Tests 202

6.2 Slab Deflection Behaviour 204
6.2.1 Applied Load during Heated Tests 204
6.2.2 Slab Deflections during Heated Tests 205
6.2.3 Load-Deflection Behaviour during Heated Tests 206
6.2.4 Load-Deflection Behaviour of Ambient Loading Tests 208
6.2.5 Summary of Deflection Behaviour 211

6.3 Comparison of Reaction Frame Results 212
6.3.1 Frame Reaction Force during Heated Tests 212
6.3.2 Frame Reaction Moment during Heated and Ambient Tests 215
6.3.2.1 Heated Tests 215
6.3.2.2 Ambient Tests 216
6.3.3 Summary of Reaction Behaviour 217
6.3.4 Summary and Shortcomings of Experimental Results 218

6.4 Model Refinement based on Experimental Data 219
6.4.1 Thermal Input 219
6.4.2 Boundary Conditions and Applied Load 220
6.4.3 PFRC Material Modelling 221

Chapter 6 Summary 222

Chapter 7 Conclusions and Further Work 223

7.1 Conclusions and Recommendations 224

7.2 Further Work 227

References 228

Appendix 236
A.1 Measurement of Frame Stiffness 237
A.2 Strain Gauge Response to High Temperatures 240
A.3 Calibration of Strain Gauges 243
## List of Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>Composite steel and concrete slab</td>
<td>2</td>
</tr>
<tr>
<td>1.2</td>
<td>Mesh and fibre reinforced composite slabs: trapezoidal and re-entrant</td>
<td>3</td>
</tr>
<tr>
<td>2.1</td>
<td>Designs of steel and polymer fibre used as concrete reinforcement</td>
<td>8</td>
</tr>
<tr>
<td>2.2</td>
<td>Early systems of fibrous concrete reinforcement</td>
<td>10</td>
</tr>
<tr>
<td>2.3</td>
<td>Plots showing inverse relationship between tensile cracking stress and wire spacing in a reinforced composite</td>
<td>12</td>
</tr>
<tr>
<td>2.4</td>
<td>Relationship between composite stiffness and orientation of fibre</td>
<td>13</td>
</tr>
<tr>
<td>2.5</td>
<td>Maximum fibre content versus coarse aggregate content</td>
<td>14</td>
</tr>
<tr>
<td>2.6</td>
<td>M.O.R. ratio versus effective spacing</td>
<td>14</td>
</tr>
<tr>
<td>2.7</td>
<td>Typical uniaxial stress-displacement relations</td>
<td>22</td>
</tr>
<tr>
<td>2.8</td>
<td>Fracture process in plain concrete</td>
<td>25</td>
</tr>
<tr>
<td>2.9</td>
<td>Typical load-deflection curves of plain and steel fibre reinforced concrete beams</td>
<td>27</td>
</tr>
<tr>
<td>2.10</td>
<td>Fibres bridging across crack</td>
<td>28</td>
</tr>
<tr>
<td>2.11</td>
<td>Fibre and matrix pullout system</td>
<td>29</td>
</tr>
<tr>
<td>2.12</td>
<td>Setup of JCI-SF4 and ASTM C 1018 test for fibre reinforced concrete</td>
<td>31</td>
</tr>
<tr>
<td>2.13a</td>
<td>Evaluation of residual flexural strength ratio</td>
<td>32</td>
</tr>
<tr>
<td>2.13b</td>
<td>Evaluation of the fracture energy</td>
<td>32</td>
</tr>
<tr>
<td>Section</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>2.14a</td>
<td>Load-deflection graph and derivation of toughness indices</td>
<td>33</td>
</tr>
<tr>
<td>2.14b</td>
<td>Load-deflection graph and calculation of residual flexural strengths</td>
<td>33</td>
</tr>
<tr>
<td>2.15</td>
<td>Illustration of wedge splitting test specimen</td>
<td>36</td>
</tr>
<tr>
<td>2.16</td>
<td>Representative Load-CMOD curves from wedge splitting test</td>
<td>37</td>
</tr>
<tr>
<td>2.17</td>
<td>Strux 90/40 polymer macro fibres and typical steel fibres</td>
<td>39</td>
</tr>
<tr>
<td>2.18</td>
<td>Temperature versus tensile strength of polymer fibre</td>
<td>41</td>
</tr>
<tr>
<td>2.19</td>
<td>Flexural strength and equivalent flexural strength of polymer FRC at elevated temperatures</td>
<td>42</td>
</tr>
<tr>
<td>2.20</td>
<td>Cumulative Fracture Energy measured up to CMOD of 2mm, 3mm, 5mm, 10mm and 20mm at elevated temperatures</td>
<td>43</td>
</tr>
<tr>
<td>2.21</td>
<td>Reduction in $f_{e,3}$ and $G_{f,10}$ of Strux 90/40 concrete with temperature</td>
<td>44</td>
</tr>
<tr>
<td>2.22a</td>
<td>Polymer fibre reinforced concrete cube specimen</td>
<td>45</td>
</tr>
<tr>
<td>2.22b</td>
<td>Shear stresses in concrete cube failure mechanism</td>
<td>45</td>
</tr>
<tr>
<td>3.1</td>
<td>Laying steel mesh on profiled deck</td>
<td>49</td>
</tr>
<tr>
<td>3.2</td>
<td>Slab panel diagram</td>
<td>50</td>
</tr>
<tr>
<td>3.3</td>
<td>Reduction factors for yield strength, Young's modulus and limit of proportionality of structural steel</td>
<td>51</td>
</tr>
<tr>
<td>3.4</td>
<td>Reduction factors for yield strength, Young's modulus and limit of proportionality of reinforcing steel</td>
<td>51</td>
</tr>
<tr>
<td>3.5</td>
<td>Conductivity of steel at high temperatures</td>
<td>52</td>
</tr>
<tr>
<td>3.6</td>
<td>Specific heat of steel at high temperatures</td>
<td>52</td>
</tr>
<tr>
<td>3.7</td>
<td>Thermal elongation of steel at high temperatures</td>
<td>52</td>
</tr>
<tr>
<td>3.8</td>
<td>Compressive strength reduction factors for NWC, LWC, and concrete with calcareous aggregates</td>
<td>54</td>
</tr>
<tr>
<td>3.9</td>
<td>Conductivity of NWC and LWC</td>
<td>55</td>
</tr>
<tr>
<td>3.10</td>
<td>Specific heat of NWC and LWC</td>
<td>55</td>
</tr>
<tr>
<td>3.11</td>
<td>Thermal elongation of NWC, LWC and calcareous aggregate concrete</td>
<td>55</td>
</tr>
<tr>
<td>3.12</td>
<td>Heat release rate of compartment fire over time</td>
<td>56</td>
</tr>
<tr>
<td>3.13</td>
<td>Standard and parametric fire curves for design</td>
<td>58</td>
</tr>
<tr>
<td>3.14</td>
<td>Slab specimen and furnace construction</td>
<td>61</td>
</tr>
</tbody>
</table>
4.10 Radiant panel assembly within restraint frame 97
4.11 Photograph of insulation applied to upper bracing channels 98
4.12 Diagram of board insulation applied to frame edges 98
4.13 Loading and measurement system 99
4.14 Underside of test specimen showing instrumentation 100
4.15 Direction of frame actions on heated and loaded slab 101
4.16a Arrangement of the perimeter gauges - elevation 103
4.16b Arrangement of the perimeter gauges - plan 103
4.17a Identical gauge response due to pure slab axial force 104
4.17b Opposite gauge response due to slab applied load 104
4.18 Circuit diagram of perimeter gauges, showing temperature-compensated active and passive pair 105
4.19 Arrangement of column gauges 106
4.20a Shear force diagram of column due to applied slab force and moment 106
4.20b Bending moment diagram of column due to applied slab force and moment 106
4.21 Setup of calibration procedure 109
4.22 Evaluating calibration factors for 1A, 1B, 2A, 2B 109
4.23 Evaluating calibration factors for 1C, 2C 110
4.24 Post-crack stress-displacement material model for PFRC in tension 112
4.25 Model assembly for a priori numerical simulations 113
4.26 Temperature evolution within slab used in a priori analysis 114
4.27 Deflection records of steel mesh, polymer fibre and unreinforced concrete slabs, a priori simulation 115
4.28 Comparative deflection records of steel mesh, polymer fibre and unreinforced concrete slabs 115

5.1 Failure mode and crack pattern, Exp. 1 121
5.2 Load versus central deflection, Exp. 1 121
5.3 Slab cracking on unloaded surface as seen from the South-East corner of the frame, Exp. 1 122
5.4 Deflected profile of slab in the N-S direction, Exp. 1 123
5.5 Deflected profile of slab in the W-E direction, Exp. 1 124
5.6 Photograph of N-S deflected shape from West side 124
5.7a Applied load versus reaction force for 1A, 1B and net (Column 1) 126
5.7b Applied load versus reaction force for 2A, 2B and net (Column 2) 126
5.7c Applied load versus reaction force for 3A, 3B and net (Column 3) 127
5.7d Applied load versus reaction force for 4A, 4B and net (Column 4) 127
5.7e Applied load versus reaction force for 5A, 5B and net (Column 5) 128
5.7f Applied load versus reaction force for 6A, 6B and net (Column 6) 128
5.7g Applied load versus reaction force for 7A, 7B and net (Column 7) 129
5.7h Applied load versus reaction force for 8A, 8B and net (Column 8) 129
5.8 Collated net reaction force results from perimeter gauges, Exp. 1 130
5.9 Applied slab load versus axial reaction force, recorded by columns 131
5.10 Averaged column and perimeter frame reactions 131
5.11 Applied load versus calculated reaction moment, Columns 1 to 6 132
5.12 Applied load versus calculated reaction moment, Columns 7 and 8 132
5.13 Flexural load carrying mechanism pre-cracking 134
5.14 Compressive load carrying mechanism post-cracking 134
5.15 Temperatures recorded by corner and quarter point thermocouples, Exp. 2A 137
5.16 Temperatures recorded at TC4 thermocouple tree, Exp. 2A 137
5.17 Temperatures recorded at TC5 thermocouple tree, Exp. 2A 138
5.18 Deflection of slab during heating and cooling, Exp. 2A 139
5.19 Photograph from South-West corner of crack pattern in heated face after 25 minutes (5 minutes of cooling), Exp. 2A 140
5.20 Diagram of crack pattern in heated face after 24 minutes (4 minutes of cooling), Exp. 2A 140
5.21 Deflected profile of slab in the N-S direction, Exp. 2A 141
5.22 Deflected profile of slab in the W-E direction, Exp. 2A 141
5.23 Reaction force records for 1A and 1B during heating and cooling, showing net reaction force; Exp. 2A (Column 1 gauges) 143
5.24 Average net reaction forces recorded by gauges in the North-South and West-East directions during heating and cooling, Exp. 2A 143
5.25 Average recorded reaction moments in the North-South and West-East directions, Exp. 2A 144
5.26 Deflected shape in West-East direction post-cracking, Exp. 2A 145
5.27 Deflected shape in North-South direction post-cracking, Exp. 2A 146
5.28 Photograph of loaded face, post failure; Exp. 2B re-loading test 147
5.29 Load-deflection behaviour of slab, Exp. 2B 148
5.30 Deflected profiles of slab in N-S direction, Exp. 2B 149
5.31 Deflected profiles of slab in W-E direction, Exp. 2B 149
5.32 Average reaction forces recorded above and below slab including net reaction, Exp. 2B 150
5.33 Reaction moments recorded during re-loading, Exp. 2B 151
5.34 Frame reaction to re-loading test in North-South direction, Exp. 2B 152
5.35 Temperatures recorded by corner and quarter point thermocouples, Exp. 3 154
5.36 Temperatures recorded at TC4 thermocouple tree, Exp. 3 154
5.37 Temperatures recorded at TC5 thermocouple tree, Exp. 3 155
5.38 Diagram of crack pattern, Exp. 3 156
5.39 Photograph of cracked slab post-failure, Exp. 3 156
5.40 Deflection of slab during heating and loading, Exp. 3 157
5.41 Deflected profiles of slab in North-South direction, Exp. 3 158
5.42 Deflected profiles of slab in West-East direction, Exp. 3 158
5.43 Average reaction force measured by column gauges, Exp. 3 159
5.44 Deflected profile and reactions of slab during heating, Exp. 3 160
5.45 Temperatures recorded by corner and quarter point thermocouples, Exp. 4A 163
5.46 Temperatures recorded at TC4 thermocouple tree, Exp. 4A 163
5.47 Temperatures recorded at TC5 thermocouple tree, Exp. 4A 164
5.48 Deflection of slab during loading, heating and cooling, Exp. 4A 165
5.49 Deflected profiles of slab in North-South direction, Exp. 4A 165
5.50 Deflected profiles of slab in West-East direction, Exp. 4A 166
5.51 Average reaction forces measured by perimeter gauges, Exp. 4A 167
5.52 Average reaction moments from frame along: a) West, East and
| 5.53 | Illustration of frame reactions after 2h 30mins of heating, Exp. 4A | 168 |
| 5.54 | Photo of slab shear failure along North side, Exp. 4B | 170 |
| 5.55 | Load-deflection response of slab, Exp. 4B | 171 |
| 5.56 | Deflected profiles of slab in North-South direction, Exp. 4B | 172 |
| 5.57 | Deflected profiles of slab in West-East direction, Exp. 4B | 172 |
| 5.58 | Applied load versus reaction force, Exp. 4B | 173 |
| 5.59 | Applied load versus reaction moment, gauges 1, 2, 5-8 (E, W, S side) | 174 |
| 5.60 | Applied load versus reaction moment, gauges 3 and 4 (North side) | 174 |
| 5.61 | Frame reaction and slab shape at failure, Exp. 4B | 175 |
| 5.62 | Temperatures recorded by corner and quarter point thermocouples, Exp. 5 | 177 |
| 5.63 | Temperatures recorded at TC4 thermocouple tree, Exp. 5 | 178 |
| 5.64 | Temperatures recorded at TC5 thermocouple tree, Exp. 5 | 178 |
| 5.65 | Crack pattern in slab at failure, Exp. 5 | 179 |
| 5.66 | Photograph of slab after failure seen from South-West corner, Exp. 5 | 179 |
| 5.67 | Deflection of slab during heating, Exp. 5 | 180 |
| 5.68 | Deflected profiles of slab in North-South direction, Exp. 5 | 181 |
| 5.69 | Deflected profiles of slab in West-East direction, Exp. 5 | 181 |
| 5.70 | Average reaction forces recorded by frame during heating, Exp. 5 | 182 |
| 5.71 | Average reaction moments recorded by frame during heating, Exp. 5 | 183 |
| 5.72 | Failure mode, crack development and frame response, Exp. 5 | 184 |
| 5.73 | Temperatures recorded by corner and quarter point thermocouples, Exp. 6 | 185 |
| 5.74 | Temperatures recorded at TC4 thermocouple tree, Exp. 6 | 186 |
| 5.75 | Temperatures recorded at TC5 thermocouple tree, Exp. 6 | 186 |
| 5.76 | Crack pattern in unrestrained slab | 187 |
| 5.77 | Photograph of unheated face at failure, Exp. 6 | 187 |
| 5.78 | Deflection of slab during heating to failure, Exp. 6 | 188 |
| 5.79 | Deflected profiles of slab in North-South direction, Exp. 6 | 189 |
| 5.80 | Deflected profiles of slab in West-East direction, Exp. 6 | 189 |
| 5.81 | Reaction forces measured by perimeter and column gauges, Exp. 6 | 190 |
5.82 Average recorded reaction moment, Exp. 6
5.83 Failure mode of Exp. 6 with no force response from frame
5.84 Compressive strength of representative slab cube specimens
5.85 Split tensile strength of representative slab cylinder specimens
5.86 Foil-wrapped cylinder specimens for moisture measurement
5.87 Pre-drying mass, post-drying mass and moisture content of slab cylinder specimens

6.1 Comparison of temperatures records from centre thermocouple tree (TC5) in each heated experiment
6.2 Slab temperatures recorded by quarter point thermocouples
6.3 Slab temperatures recorded by corner thermocouples
6.4 Slab temperature profiles during heated experiments
6.5 Applied load versus time for each heated experiment
6.6 Slab central deflection versus time for each heated experiment
6.7 Applied load versus deflection for each heated experiment
6.8 Applied load versus deflection (Range 0mm-2mm)
6.9 Applied load versus deflection for three ambient load tests
6.10 PFRC ambient test, Exp. 1 post-failure showing failure mechanism
6.11 PFRC re-loading test, Exp. 2B post-failure showing failure mech.
6.12 Steel reinf’d slab re-loading test post-failure, with uncracked interior
6.13 Comparison of perimeter gauge reactions during heated tests
6.14 Comparison of column gauge reactions during heated tests
6.15 Average recorded reaction moment during heated tests
6.16 Comparison of applied load-moment behaviour during ambient tests
6.17 Temperatures through height for heated slab experiment simulations

A.1 Arrangement of frame stiffness test
A.2 Frame column stiffness in East-West direction
A.3 Frame column stiffness in North-South direction
A.4 Photograph of load jack arrangement across frame
A.5 Photograph of deflection gauges on columns 7 and 8, South side
<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>A.6</td>
<td>Panel operation showing gauge test insulation arrangement</td>
<td>240</td>
</tr>
<tr>
<td>A.7</td>
<td>Temperature sensitivity of North side perimeter gauges</td>
<td>241</td>
</tr>
<tr>
<td>A.8</td>
<td>Temperature sensitivity of East side perimeter gauges</td>
<td>241</td>
</tr>
<tr>
<td>A.9</td>
<td>Temperature sensitivity of South side perimeter gauges</td>
<td>242</td>
</tr>
<tr>
<td>A.10</td>
<td>Evaluating calibration factors for 1A, 1B, 2A, 2B</td>
<td>243</td>
</tr>
<tr>
<td>A.11</td>
<td>Evaluating calibration factors for 1C, 2C</td>
<td>243</td>
</tr>
<tr>
<td>A.12</td>
<td>Evaluating calibration factors for 3A, 3B, 4A, 4B</td>
<td>244</td>
</tr>
<tr>
<td>A.13</td>
<td>Evaluating calibration factors for 3C, 4C</td>
<td>244</td>
</tr>
<tr>
<td>A.14</td>
<td>Evaluating calibration factors for 5A, 5B, 6A, 6B</td>
<td>245</td>
</tr>
<tr>
<td>A.15</td>
<td>Evaluating calibration factors for 5C, 6C</td>
<td>245</td>
</tr>
<tr>
<td>A.16</td>
<td>Evaluating calibration factors for 7A, 7B, 8A, 8B</td>
<td>246</td>
</tr>
<tr>
<td>A.17</td>
<td>Evaluating calibration factors for 7C, 8C</td>
<td>246</td>
</tr>
</tbody>
</table>
# List of Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>FRC breakdown and usage</td>
<td>19</td>
</tr>
<tr>
<td>2.2</td>
<td>Fibre properties</td>
<td>40</td>
</tr>
<tr>
<td>2.3</td>
<td>Constituents of Strux FRC in beam and wedge splitting tests</td>
<td>41</td>
</tr>
<tr>
<td>3.1</td>
<td>Mix design for deck polymer fibre reinforced concrete</td>
<td>74</td>
</tr>
<tr>
<td>4.1</td>
<td>Experiment sequence</td>
<td>84</td>
</tr>
<tr>
<td>4.2</td>
<td>Concrete mix constituents</td>
<td>88</td>
</tr>
<tr>
<td>4.3</td>
<td>Calibration factors for strain gauges</td>
<td>110</td>
</tr>
<tr>
<td>5.1</td>
<td>Summary of experiments</td>
<td>119</td>
</tr>
<tr>
<td>5.2</td>
<td>Moisture content analysis and results</td>
<td>196</td>
</tr>
<tr>
<td>6.1</td>
<td>Temperature data from experimental results for numerical modelling</td>
<td>220</td>
</tr>
<tr>
<td>A.1</td>
<td>Recorded stiffnesses of columns</td>
<td>238</td>
</tr>
<tr>
<td>A.2</td>
<td>Calibration factors for strain gauges</td>
<td>247</td>
</tr>
</tbody>
</table>
Chapter 1

Introduction
1.1 Project Background

Composite steel and concrete slabs consist of a profiled steel deck and a poured concrete topping, and are the preferred method of floor construction in multi-storey, steel-framed structures. The deck acts as the main tensile reinforcement for the slab section once the concrete has hardened. Nominal secondary reinforcement is usually provided by a light steel mesh placed within the concrete section. This offers resistance to surface cracking, caused by concrete drying and shrinkage. Figure 1.1 shows an illustration of a typical trapezoidal profile composite slab.

Research and analysis into the behaviour of in-situ composite slabs in fire has shown that they possess greater resistance to high temperatures than simple design and furnace testing procedures would predict. Two factors play an important role in providing this extra resistance. For a continuous slab, the fire-affected region is restrained from thermal expansion by the cooler, stiffer surrounding structure. Also, the reinforcing mesh allows stable load-carrying structural mechanisms to develop at high deflections, through membrane action. This high-temperature structural behaviour is well understood and is incorporated into the modern performance-based fire design of steel-framed structures.

Polymer-fibre-reinforced concrete (PFRC) composite slabs are now available to designers of steel framed buildings, as an alternative to mesh reinforced composite slabs. In PFRC slabs, the concrete topping poured over the deck contains dispersed polymer fibres, 40mm in length. These fibres perform the anti-cracking function of the mesh by distributing tensile stresses at the surface. Fibres also provide

Figure 1.1: Composite steel and concrete slab (RLSD, Ribdeck E60)
reinforcement throughout the concrete volume as opposed to at a specified height above the deck. A comparison between mesh and PFRC slabs is shown in Figure 1.2, for both trapezoidal and re-entrant profiles. Construction using PFRC slabs has several practical advantages over using traditional slabs, including cost savings from the reduced steel requirement, a faster construction speed, a safer working area for the concreting operation and no need to store or lift bundles of mesh around the site.

The fire resistance of PFRC slabs has been demonstrated using standardised furnace tests on representative slab elements. These suggest that PFRC slabs can achieve a fire rating of 1 to 2 hours, depending on the slab depth and shape of deck profile used. However, in a full structural context, the load-carrying mechanisms by which continuous PFRC slabs exhibit fire resistance are not clear. The polymer fibre material has a melting point of 160ºC, and it is not apparent to what extent the stiffness of the surrounding structure is relied upon to provide structural stability at high temperatures.

This PhD project addresses the need for further understanding of the behaviour of restrained PFRC slabs in fire. An experimental test series was proposed, using model-scale slab specimens to investigate the effects of fibre reinforcement and boundary conditions on the fire resistance of composite slabs. A testing rig capable of recording the forces and moments transmitted by the slab due to restrained thermal expansion was constructed, which provided a full history of the slab’s responses in fire.
1.2 Project Aims

This research aims to provide:

- A series of model-scale experiments investigating the properties and behaviour of polymer-fibre-reinforced composite slabs in fire, including measurement of the boundary interactions at the slab perimeter;
- Further understanding of the effects that structural restraint to thermal expansion has on the stability of composite slabs exposed to fire;
- Experimental evidence of the role structural polymer fibres can have as concrete reinforcement in slabs at high temperatures; and
- A numerical analysis of the experiment series, using the data from the tests as a benchmark for the production of a material model for polymer-fibre-reinforced concrete.

1.3 Thesis Outline

This thesis is divided into the following chapters.

Chapter 2 Fibre Reinforcement of Concrete

In this chapter, fibre reinforced concrete is introduced. The development of the material is outlined along with a list of applications forming the current state of the art, including use in composite deck slabs. The mechanics of fibre reinforcement and properties of PFRC are then explained. This is followed by a description of the specific test methods used to evaluate the performance of PFRC. Finally, the macro polymer fibre used throughout this research is fully described, along with the properties of the resulting fibre reinforced concrete. Finally, the results and conclusions from preliminary load tests on PFRC specimens are described.
Chapter 3  Behaviour of Composite Slabs in Fire

This chapter introduces composite construction and performance of slabs in fire. The material properties of steel and concrete, compartment fire behaviour and performance of composite slabs are each introduced. The standard test procedure for evaluating fire resistance of composite slabs is described. Limitations of isolated standard testing are discussed, providing an introduction to the full-scale fire experiments performed at Cardington. Techniques of performance-based fire design for more efficient structures based on the Cardington outcomes are then explained, utilising slab membrane action and computational analysis. The PFRC slab construction method under investigation is then described in detail. The reasons for concern about the fire resistance of such slabs are then explained.

Chapter 4  Experimental Design and Methodology

In this chapter, the design of the experiment series is outlined. An overview of the test arrangement is provided, with an explanation of how this meets the test requirements. A detailed description is then given for each of the experimental features and systems, including the slab specimens, construction of the reaction frame, heating panel installation, loading method, instrumentation and use of strain gauges to measure the restraining actions. Finally this chapter outlines the results from a series of numerical simulations of the test procedure, performed before the tests had been carried out. The Finite Element Analysis package ABAQUS was used to simulate three heated tests incorporating representative temperature behaviour, boundary conditions and loading arrangements. The aim of these analyses was to verify that the experiment would produce viable data, in terms of slab deflections and temperatures achieved, so that useful conclusions may be drawn as per the project aims.
Chapter 5 Experimental Results

This chapter presents the results from the series of experiments. Each experiment is covered in detail sequentially. The temperature, deflection and reaction results are presented, followed by observations and an explanation of the specimen behaviour. The final section presents results from tests on sample specimens, establishing uniformity of strength and moisture content across the range of slabs.

Chapter 6 Analysis and Discussion

In this chapter, the relative performance of the slab specimens is compared and analysed. The temperatures, deflections and reactions from each test are used to illustrate the differences and similarities in behaviour between the slab specimens. Finally, test parameters recorded from the experiment series (temperatures achieved, boundary conditions, load applied) are discussed. This is followed with an explanation of how to achieve more realistic numerical simulations, using the data directly recorded from the experiments.

Chapter 7 Conclusions and Further Work

The conclusions and recommendations provided by this research are listed. Specific suggestions are also made for topics of research which would provide further insight into the fire behaviour of PFRC slabs.
Chapter 2

Fibre Reinforcement of Concrete

This chapter introduces fibre reinforced concrete. The following subjects are covered:

- The development of fibre reinforced concrete from initial concepts to the state of the art, including the current application in steel-concrete composite slabs;
- The properties of fibre reinforced concretes and the parameters which affect composite behaviour;
- The material testing standards and procedures used for classifying the various designs of fibre and assessing composite performance;
- The characteristics of the polymer fibre used in this research.
2.1 Development and Applications of Fibre Reinforced Concrete

This section introduces fibre reinforced concrete and provides a brief description of its development through history to the present day. Early scientific research on the material is then explained, followed by a description of the various kinds of reinforcing fibre available and the engineering applications for which they are used.

2.1.1 Introduction to Fibre Reinforced Concrete

Fibre reinforced concrete (FRC) is a term for any cementitious or mortar-based multi-phase construction material, with short, discrete reinforcing fibres dispersed uniformly throughout to improve aspects of performance. Fibre reinforcement is distinct from traditional steel bar or mesh reinforcement. It is also not associated with fibre reinforced polymer (FRP) reinforcing systems, whether these consist of embedded bars or adhered sheets or wraps. Figure 2.1 shows common steel and synthetic fibres used for reinforcement of concrete. Steel fibres often use deformed ends to improve anchorage, while synthetic fibres rely on bond and friction.

Figure 2.1: Designs of steel and polymer fibre used as concrete reinforcement
Chapter 2 - Fibre Reinforcement of Concrete

The fibres in FRC are added to the regular concrete constituents during the mixing stage, and therefore the design, handling and placement of such mixes must take the presence of fibres into account. Steel and synthetic polymers are the two most common materials used for reinforcing fibres. A wide range of shapes and sizes exist using these materials, to be used depending on the required FRC composite properties. Other fibres made from glass or natural fibres such as jute, sisal or coir are available. FRC has been developed from simple initial concepts into a well-understood and efficient composite material used internationally in construction and architecture.

2.1.2 Historical and Industrial Development

The concept of reinforcing brittle building materials with dispersed fibres dates back to antiquity. Adobe and cob, the ancient construction materials developed in Egypt and Mesopotamia from 3500BC are the earliest surviving examples. These materials consisted of a wet clay, sand or mud mixture, reinforced with straw or other fibrous material (Schaffer, 2001). Cuboid moulds were filled and left to dry to form bricks, or a wet render was applied to existing structures. The straw improved the cohesion of the bricks while forming, and reduced brittleness after hardening.

These building materials pre-date the ancient Greeks, who used simple lime-based mortars in their construction methods. The Romans were the first to truly develop cementitious material technology. Naturally occurring silica-rich ash (pozzolan) was an abundant hydraulic binding material. It was found to dramatically improve the strength of hardened lime mortar and also possessed the ability to set underwater. As before, straw, horsehair and other natural fibres like reeds or grass were often added during production to improve durability and reduce brittleness and cracking.

Concrete technology was neglected after the Roman period, as timber and masonry construction became prevalent. In the second half of the 19th century, the initial concepts of bar-reinforced concrete were developed through the ferrocement work of Joseph-Louis Lambot, Joseph Monier and François Coignet. Reinforced concrete
became a viable construction material, and concrete including short dispersed fibres was investigated by inventors seeking to further optimise their concrete properties.

From the 1870’s to the 1950’s, a series of patents were issued for various fibrous concrete reinforcement techniques. The earliest patents involved using waste products of industrial processes, such as waste "iron gas retorts" (Bérard, 1874) or cut nails (Porter, 1910). These may have provided only modest improvements as the fibre shapes were not designed specifically for improving concrete. However, later fibres developed through testing and trial and error had more tortuous shapes and were better suited. Selected systems of this type are illustrated in Figure 2.2 (Graham, 1911; Weakley, 1912; Meischke-Smith, 1920; Etheridge, 1931). These systems used iron or steel fibres, or shaped wires dispersed through the bulk or at the surface of a concrete element. Early steel fibres provided improved abrasion resistance, impact strength and toughness to concrete. It was well understood that the quality of bond obtained between the fibres and concrete was critical. Of these early inventions, a reinforcing system of helical steel fibres providing longer bonded

![Figure 2.2: Early systems of fibrous concrete reinforcement. 1: Graham’s dispersed steel (1911) 2: Weakley’s continuous twisted wire (1912) 3: Meischke-Smith’s wide twisted fibre (1920) 4: Etheridge’s iron rings (1931)](image-url)
lengths and greater pull-out resistance demonstrated the reinforcing mechanics of steel fibres best. This design significantly enhanced concrete element performance compared to other fibre types (Constantinesco, 1943). Throughout this period asbestos fibre was also a common addition to concrete structures, providing enhanced strength and fire resistance. Once the detrimental health consequences of asbestos were discovered, engineers looked to create substitute materials with similar characteristics, and research into FRC intensified.

2.1.3 Scientific Research into FRC

Through experimentation by developers, the mechanics of FRC were well investigated by 1960. However, there was no published scientific work, and FRC as a material was only slowly finding practical use in concrete roads and slab-on-ground construction in the USA.

Initial Investigations

The pioneering scientific work on steel fibre reinforced concrete (SFRC) started in the early 1960s. J. P. Romualdi is credited by many authors with the first theoretical investigations into the reinforcing behaviour of steel fibres in concrete (Zollo, 1997; Swamy, 1975; Labib and Eden, 2006; Beddar, 2008; Darwish et al., 2008). Initially, research was restricted to the tensile response of a cementitious composite with long, parallel, closely spaced wires in uniaxial tension. Romualdi demonstrated theoretically that in this case the cracking stress of the composite material increases with closer spacing of the fibres (Romualdi and Batson, 1963). Figure 2.3 shows their relationship between tensile cracking stress and wire spacing, for three different reinforcement ratios (2.5%, 5%, 7%). Later, dispersed discrete fibres in a matrix were investigated and Romualdi developed equations that described their spacing in terms of their dimensions and dosage (Romualdi and Mandel, 1964).

The stiffness and strength of composite materials reinforced with short fibres was investigated in terms of a linear law of mixtures (Garkhail et al., 2000). The stiffness
of a general reinforced composite material with aligned wires \( E_c \) depended only on the volume fraction of fibre present. Cox had previously expanded on this by introducing a fibre inefficiency factor \( \eta_{LE} \) into the stiffness equation, to account for incomplete shear bond along the fibre length (Cox, 1952):

\[
E_c = \eta_{LE} E_f V_f + E_m V_m
\]

Eq. 1

where,

\( E_f \) = Young’s modulus of the fibre;
\( V_f \) = Volume fraction of the fibre;
\( E_m \) = Young’s modulus of the matrix;
\( V_m \) = Volume fraction of the matrix.

The fibre inefficiency factor \( \eta_{LE} \) was computed from the fibre dimensions and spacing.

To calculate the stiffness of a fibre reinforced concrete composite with short, randomly oriented fibres, Krenchel developed a fibre orientation factor \( \eta_0 \) and incorporated it into the stiffness equation (Krenchel, 1964). Therefore:

\[
E_c = \eta_0 \eta_{LE} E_f V_f + E_m V_m
\]

Eq.2
with the fibre orientation factor:

\[ \eta_0 = \sum a_n \cos^4 \theta_n \]  
Eq. 3

where:

\( a_n = \) the fraction of fibres oriented at the angle \( \theta \) with respect to the direction of load.

Equations 2 and 3 allowed the stiffness of composites to be calculated with fibres oriented at different angles. Figure 2.4 shows the relative stiffness of a composite material, with its reinforcing fibres oriented at increasing angles away from the direction of loading. Equation 3 allows for fractions of reinforcing fibres at different orientations within the composite, and the orientation factor is calculated from the proportions of fibres acting at different angles. The equation gives an orientation factor (\( \eta_0 \)) of 0.2 for short fibres randomly distributed throughout a reinforced composite (Garkhail et al., 2000).

![Figure 2.4: Relationship between composite stiffness and orientation of fibre (Krenchel, 1964)](image-url)
Fabrication and Dosage

The work of Romualdi was expanded upon specifically by experimental and theoretical investigations into the properties of concrete with various coarse aggregate contents and fibre dosages (Swamy and Mangat, 1974). In this research, inch long needle-like steel fibres with an aspect ratio (length / diameter) of 100 were used. The maximum volume of fibres it was possible to include in a concrete mix was shown to depend on the coarse aggregate content. High aggregate contents reduced the available volume of cement paste through which the fibres could disperse. Above a critical volume, the fibres would tangle and not spread throughout the concrete, a phenomenon known as 'balling up'. Balling up was recognised to be detrimental to composite strength as it allowed the formation of regions of unreinforced concrete and created problems with workability and compaction. Figure 2.5 shows a decreasing linear relationship between maximum fibre content and coarse aggregate content. Higher aggregate contents were also shown to reduce composite strength for given fibre dosages.

The fibre dosage was subsequently analysed through the concept of effective fibre spacing. Previously, Romualdi developed a relationship between cracking stress and

![Figure 2.5: Maximum fibre content versus coarse aggregate content (Swamy and Mangat, 1974)](image1)

![Figure 2.6: M.O.R. ratio versus effective spacing (Swamy and Mangat, 1974)](image2)
fibre spacing: it was later shown that this relationship did not take aggregate content into account. The ‘effective spacing’ concept was proposed, which removed the aggregate sensitivity. A series of flexure tests on FRC specimens was performed. Figure 2.6 above compares the observed relationship between the modulus of rupture ratio and the effective spacing, to theoretical curves incorporating the new effective spacing factor. This amendment to the theory provided much better agreement with recorded test results.

A later study investigated the effect that the methods of fabrication and compaction had on the behaviour of hardened SFRC (Swamy and Stavrides, 1976). An early focus was on establishing even dispersal and random orientation of the fibres, as this was critical to the concept of an isotropically reinforced composite. Previous strength tests on SFRC prisms compacted with external vibration had indicated anisotropic behaviour relative to the plane of vibration. So, compressive and flexural tests were carried out on prisms with different compaction methods and with different load orientations relative to the casting direction. It was found that external vibration was the most efficient method of compacting SFRC, as rodding was ineffective due to the fibres becoming distorted and misshapen.

Another finding was that in flexural tests, beams cast parallel to the axis of bending possessed greater utilisation of fibres than beams cast perpendicular. The steel fibres tended to settle horizontally under vibrational compaction, therefore the majority of fibres in beams cast “on-end” had nearly vertical orientations relative to the beam axis. This made them ineffective as tensile flexural reinforcement. Fibre counts at the cracked surfaces were greater in the parallel-cast beams, and predictably these beams exhibited much greater toughness and energy absorption capacity under load. Fibre orientation was established as being critical to the performance of the composite member. Efforts were made to find ways of assessing the direction of fibres in a non-destructive manner. An early attempt using x-ray radiographic imaging was shown to be successful in this regard, with individual, aligned steel fibres being clearly visible in the processed image (Swamy, 1975).
Fibre Pull-Out and Cracking

More general theories describing the stress-strain response and failure behaviour of FRC were developed during the same period. Previous theoretical work on the load response of fibre reinforced composites had been completed, illustrating the physics of fibre pull-out from a matrix for bonded or unbonded fibres. Theoretical principles of fracture, crack propagation and fibre bridging had also been developed (Kelly, 1970). These were originally proposed for the case of metal fibre reinforced metallic or resin matrices, where the behaviour of both matrix and fibre phases were ideally brittle.

Equations were derived characterising the strength and failure mechanisms of weak, low failure strain matrices reinforced with more ductile fibres. Early research was conducted on the micromechanics of fibre debonding and the conditions determining whether fibres failed by fracture or pull-out from the matrix (Aveston, 1973). A further, theoretical study on this phenomenon was produced (Rajagopalan, 1975), and equations were developed determining whether composite failure would occur with fibre pull-out or breakage. This highlighted the concept of fibre critical length.

2.1.4 Development of Other Fibre Materials

This section describes the development of FRC using polymer, glass, natural fibres and high strength synthetic materials.

Synthetic Polymer Fibre Reinforced Concrete

The first mention of synthetic polymer fibre reinforced concrete dates from 1963, on the basis of experimental work by American Military Laboratories. It was recognised that a synthetic fibre reinforced concrete was suitable for the construction of bomb shelters, as this produced better impact and blast resistance compared to designs consisting of glass or asbestos fibre. This was due to the lower elastic modulus of the polymer compared to other materials. A subsequent patent (Goldfein, 1967) derived
from this research outlines several key issues with the use of polymer fibres, in this case including “nylon, polypropylene, polyvinylene chloride and polyethylene”. Goldfein performed flexure tests and impact tests on a variety of cements and mortars. The flexure tests investigated reference cement and mortar mixes, and mixes reinforced with 1 inch steel fibres, 1 inch glass resin fibres coated in sand, and 1 and 3 inch nylon fibres. The 1 inch nylon fibres outperformed the steel fibres, in terms of strength obtained and toughening observed. The toughening was increased further with the 3 inch nylon fibres, however longer fibres were harder to include in the mix due to fibre balling. At this time it was acknowledged that due to their low Young's modulus, synthetic fibres were unsuitable as reinforcement to improve strength in purely tensile or flexural loading. However, this property was noted to give significant improvements in ductility and especially impact and blast resistance to the composite.

**Glass Fibres**

The use of glass fibres as a structural reinforcement material began in 1942, not for reinforcing concrete, but in glass fibre reinforced polyester resin (GFRP). The process of bundling then twisting individual filaments into strands created glass fibres with high stiffness and strength. The incorporation of these into thermosetting polymers produced increased toughness and ductility in the composite. Borosilicate E-glass and soda-lime A-glass fibres were usually used, due to their high strength and durability. Glass fibre reinforced concrete (GFRC) was first investigated in the Soviet Union in the late 1950s (Shah, 1992). In early trials it was found that the common E-glass and A-glass fibres were unsuitable for use in cement. The alkali environment severely degraded the fibres, as OH⁻ ions from the cement paste attacked the molecular silicon oxide bonds at the glass surface. This reduced the fibre strength and shear bond, and with time the fibres would naturally degrade. This issue was eventually resolved with the development of alkali-resistant, 16-20% zirconia-based Z-glass fibres. Subsequently, a concerted research effort into the suitability of GFRC as a building material was undertaken at Building Research Establishment (BRE) in England (Majumdar, 1970).
Glass fibre reinforcement was limited in its suitability for reinforcing concrete members, and instead was mainly employed for secondary reinforcement and crack control in cladding and sandwich panel systems for building exteriors. In this regard it was moderately successful, until architectural trends changed so that aluminium and metallic cladding were used more frequently. One problem was that large sandwich panels were exposed to high thermal gradients in winter months, and the GFRC was not durable and cracked under thermal stresses.

**Natural Fibres**

Throughout the developing world, the use of natural fibres in concrete and mortar plays a role in cost-effective construction and attracts significant research interest. Natural organic fibres used to reinforce concrete include coir, sisal, palm fibre and elephant grass. Locally sourced natural fibres differ in performance from country to country. Mechanisms of fibre bond and durability within the alkaline environment are key considerations for natural FRC. Sisal reinforced concrete was investigated as a possible replacement for asbestos material in India. While ultimately the material had only 60-80% the strength of asbestos concrete, the economic benefits and health advantages were sufficient to continue research into the material (Saxena et al., 1992).

**High Modulus Synthetic Fibres**

Aside from steel, polymer and glass, synthetic fibres made from carbon and aramid have been investigated for their practicality as distributed reinforcement of concrete and cement. These fibres typically have a very high modulus, but their cost makes them inefficient for bulk reinforcement of large volumes. Carbon fibre reinforced concrete research began in Japan around 1986, and research effort has since moved to the USA. Carbon fibres of around 5mm length at a dosage of 0.189% volume have been shown to increase flexural strength by 85%, but the material cost of the associated fibres and mixing agents increased by 39% (Chung, 1992).
2.1.5 Current Applications of Fibre Reinforced Concrete

This section summarises recent and advanced applications of fibre reinforced concrete.

**Slab-on-Ground Construction and Paving**

Slab-on-ground construction is one of the most well-established applications of FRC. One of the drivers of modern FRC development was that steel fibres increased the abrasion resistance of hard-wearing concrete roads and industrial floors. This made it a useful material for slabs with onerous service conditions like frequent usage by heavy duty or caterpillar-tracked vehicles. Large cast concrete areas exposed to the elements also face difficult curing conditions. With the onset of drying shrinkage and settlement cracking, these surfaces experienced accelerated degradation. The addition of steel fibres provided better crack control in the early stages, and enhanced toughness once the slab was in service. Recently this technology has been applied to individual paving slabs suitable for pavements, patios and architectural installations. It has been demonstrated in practice that fibre reinforced paving slabs can experience longer cycles of loading without cracking compared to conventional concrete. Table 2.1 shows the usage of the approximately 100 million m$^3$ of FRC produced annually.

**Shotcrete**

Steel fibres are an important component of shotcrete, where a concrete mix is delivered to a surface through a high-pressure nozzle. This casting technique is used to create tunnel linings, and to support areas of irregular geometry where traditional reinforcement would be difficult to place. It is also used to create vaulted roofs, or

<table>
<thead>
<tr>
<th>Application</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab-on-ground</td>
<td>60</td>
</tr>
<tr>
<td>Shotcrete</td>
<td>25</td>
</tr>
<tr>
<td>Precast members</td>
<td>5</td>
</tr>
<tr>
<td>Miscellaneous use</td>
<td>10</td>
</tr>
</tbody>
</table>

Table 2.1: FRC breakdown and usage (Mindess, 2008)
domestic vessels like tanks or swimming pools. It can also be used to shore up the sides of loose channel walls and stabilise against rockfalls. Shotcrete has several properties differentiating it from regular cast reinforced concrete, such as higher cement contents, smaller aggregates, less uniform distribution of constituents and a unique sensitivity to water-cement ratio. The use of fibres in shotcrete, usually steel, provides reduced brittleness and limits cracking widths. Also, removing the requirement for cover to reinforcement can reduce the required section depth.

Cohesion and rebound of particles and fibres are key considerations in shotcreting. Adhesion to the surface and the gradual build-up of a section depend on the energy absorption characteristics of the substrate and developing wet shotcrete face, as well as the nozzle velocity and distance. The type of fibres used, the water content and presence of any other admixtures all affect the ease with which sections are built up.

An advantage that fibre-reinforced shotcrete has over bulk FRC is preferential fibre orientation. The impact force of placement means fibres are unlikely to end up normal to the receiving surface. This orients the fibres into the plane of the element, enhancing the reinforcing effect. In standard FRC the fibres are assumed to have a random isotropic three-dimensional orientation, unless another system is in place to manipulate the fibres. This means that for any direction of principal tensile stress, the majority of the fibres will tend to directions perpendicular to the stress and will be ineffective.

**Thermal Spalling Mitigation**

The ability of polypropylene fibres to provide resistance to thermal spalling was first established in the early 1980’s (Atkinson, 2004). Thermal spalling occurs when material is ejected from the surface of a concrete element due to heat exposure. The mechanisms controlling this phenomenon are complex and not yet fully understood; recent work has attempted to predict the nature of concrete spalling through development of a full thermo-hydro-mechanical numerical model (Koury, 2008a). Spalling behaviour is sensitive to many parameters related to the strength, age,
density and particularly the moisture content of the concrete. Spalling causes loss of section and can reduce cover to steel reinforcement. Therefore, it is a significant concern in fire safety engineering. The use of 2kg/m$^3$ polymer fibres is accepted in Eurocode 2 (BS EN 1992-1-2:2004) as an effective method of spalling prevention for high strength concretes.

**Ultra High Performance Concrete**

In recent years there have been attempts made to augment the behaviour of the strongest possible cementitious composites with the inclusion of fibres. Ultra high performance concretes (UHPCs) can achieve compressive strengths in excess of 250MPa. These materials are composed of the stiffest and smallest possible aggregates with a sophisticated blend of cementitious and cement replacement materials. The hydration requirements are strict, with water/cement ratios lower than 0.2 being common. These mixtures have higher homogeneity and density due to the reduction in pore size. As the strength and stiffness of cementitious composites increase, their stress-strain behaviour becomes more brittle. Failure occurs at increasingly lower strains as the tolerance for flaws and microcracks reduces. Therefore, to remain viable as construction materials some ductility must be introduced and this is usually done by adding small fibres to the mix.

RPC (Reactive Powder Concrete) is a type of UHPC with a maximum aggregate size of 600µm, and can easily achieve a compressive strength of 200MPa (Bonneau *et al.*, 1996). The optimized grain size distribution increases density, while curing under pressure along with heat treating in the early stages of hardening both reduce the presence of microflaws in the matrix. The presence of steel fibres does not disturb the packing of the granules as the fibre diameter of 200µm is roughly on the same scale as the aggregate (Bonneau *et al.*, 2000).

Another UHPC application is the family of materials classed as ECC (Engineered Cementitious Composite). The constituents of these materials are selected to maximise ductility and deformation capacity, and to maintain load-carrying capacity.
at high deflections. This material possesses excellent ability to distribute strain and prevent crack localisation. This is achieved through high dosages of polymer microfibres and very close control of the aggregate particle size distribution. Figure 2.7 (Li, 2008) shows typical uniaxial stress-deflection curves for plain concrete, fibre reinforced concrete and an ECC (named HPFRCC, "High Performance Fibre Reinforced Cementitious Composite"). Plain concrete (1) possesses little deflection capacity after initial cracking. General fibre reinforced concretes (2) demonstrate tension-softening, where crack opening is accompanied by a lower permissible stress. The major property of ECC (3) is that multiple microcracking permits increasing tensile stress capacity beyond the initial elastic range.

**Fibre Reinforced Concrete Composite Slabs**

In traditional steel frame building construction, the floor slabs use profiled steel deck and a poured concrete topping. The concrete and steel deck develops a shear bond which permits the slab to carry load in flexure. In addition, a light gauge steel mesh is placed within the concrete section. This mesh is included to resist the onset of

![Figure 2.7: Typical uniaxial stress-displacement relations for: (1) plain concrete, (2) fibre reinforced concrete and (3) HPFRCC, engineered cementitious composite (Li, 2008).](image-url)
plastic shrinkage and drying cracks, and can help to carry loads over supports.

New composite slabs are available where the light steel mesh is omitted, and instead the concrete is reinforced with either steel or polymer fibres, or a blend of both. The fibres provide resistance against cracking associated with dehydration and hardening of the concrete. The omission of the steel mesh has many practical benefits to the construction schedule:

- the steel cost is reduced;
- no need to raise or store bundles of mesh on site saving crane time and space;
- time is saved from not having to place and fix mesh;
- avoids mesh overlap or misplacement which reduces concrete cover;
- the reinforcement is uniformly dispersed throughout the concrete; and
- the deck concreting operation is easier and safer without mesh underfoot.

The use of fibre reinforced concrete does not provide any extra strength to the composite slab; the fibres perform the anti-crack function of the mesh only. Proprietary fibre reinforced composite slab systems have been tested individually by manufacturers for shear stud performance, transverse shear resistance and fire resistance. A fire engineering design approach has been used to evaluate the results of the fire resistance tests and design tables have been developed incorporating a range of slab spans, depths and fire resistance periods (RLSD, 2009). A more thorough description of PFRC slab construction is provided in Chapter 3.

### 2.1.6 Classification of Fibres for Concrete Reinforcement

Steel fibres are separated into five groups based on manufacturing technique. These are:

- **Group I** Cold-drawn wire
- **Group II** Cut sheet
- **Group III** Melt extract
- **Group IV** Shaved cold drawn wire
- **Group V** Milled from blocks

Polymer fibres are divided into three classes dependent on size and structure. These are:

- **Class Ia** Micro fibres: < 0.3mm diameter, monofilament
- **Class Ib** Micro fibres: < 0.3mm diameter, fibrillated
- **Class II** Macro fibres: > 0.3mm diameter

Monofilament microfibres are single fibre strands, which usually occur in ‘bundles’ which disperse under agitation. Fibrillated fibres are similar but contain partial cross-linking between individual strands, and have more resistance to dispersal.

A summary of all the current guidance and application-specific information on the use of steel and macro-polymer FRC was commissioned for the Concrete Society in 2007. The resulting technical reports, TR 63 and TR 65, provide qualitative design guidance, performance data and classification techniques for steel and macro-polymer fibres in concrete (Concrete Society 2007a, 2007b).
2.2 Effects of Fibre Reinforcement and Mechanical Behaviour of FRC at Ambient Temperature

In this section, the material properties and behaviour of FRC is discussed. A brief overview of the mechanics of plain concrete is given, followed by sections describing the effect that fibre reinforcement has on various concrete properties.

2.2.1 Plain Concrete in Tension

Concrete is assumed by engineers to have a tensile stress capacity approximately 8-15% of the compressive stress capacity. The low tensile strength of concrete is a result of its microstructure. Concrete elements can be assumed to contain a pervasive network of flaws, or microcracks. These occur throughout the hardened cement paste, and at the aggregate/paste boundary. When areas of concrete are subjected to increasing tensile stress, these microcracks propagate and coalesce, until eventually enough microcracks join together and a full crack is formed.

Plain concrete also has low toughness. This is a measure of how resistant the material is to further crack propagation, once the cracking stress has been reached. The resistance to further applied stress comes from various mechanisms in the region just ahead of the crack tip, called the fracture process zone. This cracking mechanism is illustrated in Figure 2.8.

![Figure 2.8: Fracture process in plain concrete](image)
The fracture process zone mechanisms involved in resisting crack extension in plain concrete include:

- additional plastic microcracking ahead of the crack tip;
- friction between crack faces, due to rough concrete surfaces separating;
- crack path deviation; where the presence of an aggregate particle in the crack path forces the crack tip to move around it, absorbing more energy; and
- aggregate bridging; where large aggregate particles span across the crack faces. These particles need more energy to break the shear bond and overcome friction before they are pulled out.

In plain concrete, the stress capacity of these mechanisms in the fracture process zone is low compared to the stress required to initiate cracking.

### 2.2.2 Cracking of FRC Elements and Load-Deflection Curves

A comparison of the load-deflection behaviour of plain and fibre reinforced concrete beams is given in Figure 2.9. This figure illustrates how FRC beams possess greater load resistance after crack initiation. The characteristics of the descending branch are used to assess the effectiveness of different fibres at providing toughness to concrete. This is covered in greater detail in Section 2.3, when FRC test methods are discussed.

Fibre reinforced concrete has improved toughness and ductility, and a reason for this is that the fibres reduce crack localisation. In an ideal scenario, the fibres are able to transmit the stresses across the cracked section into the concrete on either side. If sufficient stress is transmitted in this way, multiple cracking can occur on the tensile face of a beam. This principle is only practical in high performance fibre reinforced composites, but it can generate extremely high levels of ductility and energy absorption capacity.
2.2.3 Function of Reinforcing Fibres

When a fibre reinforced concrete element undergoes tensile stress, the pre-cracking stage is similar to an unreinforced element. Fibres do not provide significant increases in the uniaxial tensile or compressive strength of concrete. The main role of the fibres is to alter the post-crack behaviour. Once a crack is developed, the fibres increase the resistance to further crack propagation. This is illustrated in Figure 2.10. In the early stages of crack formation, the fibres will bridge across the full length of the crack. Each fibre adds to the work required to extend the crack, due to the energy used in debonding and pulling the fibre out from the concrete surface.

Therefore, the fibres are said to increase the size and crack resistance of the fracture process zone mechanisms. The quantity of extra energy consumption associated with fibre pullout is dependent on the fibre shape, length and quality of bond with the matrix. By transmitting stress between the two faces of the crack, the fibres resist further separation and crack extension. Due to the fibre action, larger crack openings and deformations are possible beyond the peak cracking stress than with unreinforced concrete.
2.2.4 Fibre Pull-Out, Fibre Breakage and Critical Length

The bond characteristics between a fibre and the matrix are the most critical factor determining the ability of a fibre to be effective in resisting crack separation. Depending on the bond strength between the matrix and the fibre, one of two outcomes are possible;

- If the bond is strong, the fibre will break before it can be pulled out of the matrix;
- If the bond is weak, the fibre will be drawn out through a mechanism of elastic debonding from the matrix followed by frictional pullout.

As the objective of fibre reinforcement is to improve toughness and deformation capacity of a composite, then pullout of fibres from the matrix is the preferable mode of failure. A fibre–matrix system is shown in Figure 2.11. The fibre has half its length \( l \) embedded in the matrix. As the axial load \( P \) on the fibre is increased, the interface between the fibre and the matrix undergoes a reactionary shear stress \( \tau_f \). This shear stress acts along the bonded surface area of the fibre, which for a circular cross-section fibre is \( l\pi r \), where \( r \) is the fibre radius.
Defining the tensile stress capacity of the fibre as $\sigma_f$, the maximum tensile stress and shear stress can be equated;

$$\sigma_f \pi r^2 = \tau_f l \pi r$$

Eq. 4

And re-arranging for $l$:

$$l_c = \frac{\sigma_f r}{\tau_f}$$

Eq. 5

This is the definition of fibre critical length (Mindess et al., 2003). The critical length is the length below which a fibre will pull out from a matrix with a given maximum shear stress, and above which, the fibre will break. This measure is useful in evaluating whether or not certain fibres will provide toughening to a matrix with known shear bond properties.

Ouyang et al. (1994) conducted a study on the load-slip behaviour of fibres embedded in a cementitious matrix. The fibres in this study were pulled out at various angles, as well as normal to the crack face. The force required was plotted against the amount of slip. The relationship between load and slip was discovered to be non-linear. The effect of pullout at angles away from normal to the face was to
increase the peak load supported by the fibre; as well as breaking the fibre bond and subsequent pullout, work was done in crushing the concrete at the face.

It was observed that the distribution of shear force along the fibre is non-linear. Also, low modulus circular fibres are susceptible to Poisson’s effects. As the fibre is stretched, the reduction in cross-sectional area applies a direct tensile stress on the matrix at the interface. This reduces the shear transfer capacity and can cause pullout at lower loads.

2.2.5 Fibre Reinforcement in Fresh Concrete Before Hardening

For fibres to be effective as reinforcement throughout hardened concrete elements, they require certain properties during the mixing and plastic stages of the concrete. While increasing the dosage of fibres increases the ability to withstand stress after cracking, it is difficult to introduce high volumes of fibres without the fibres clumping together or balling up, as described above in Section 2.1.3. Balling up can occur if the fibres are either too stiff (shear interlock) or too slender (entanglement), particularly at high dosages. Modern polymer fibres are designed for use-specific flexibility criteria which ensure they remain highly dispersible.

Fibres also have an adverse effect on concrete workability. Friction and contact between the fibres increases the viscosity and energy required for compaction. Because adding more water to maintain workability levels would affect the strength properties of the hardened concrete, mix designs for FRC usually include plasticisers or water-reducing admixtures. These agents lubricate the cement paste phase of the wet concrete; this compensates for the workability without affecting the water-cement ratio. High-level water reducing agents or superplasticisers are recommended for use in concretes with high dosages of steel or polymer macro fibres. These are also suitable when concrete has to be pumped to high floors, as drag in the pipe is reduced and lower pump pressures are required.
2.3 Testing Procedures for Fibre Reinforced Concrete

This section discusses methods for testing FRC and quantifying the improvements in toughness that result. It goes on to examine the classification techniques for the different fibres that are available. Efforts to characterise and classify different types of FRC have increased in the past 10 years, as it is perceived by designers that a lack of approved design codes and standards is inhibiting widespread usage of FRC in engineering.

2.3.1 Flexural Strength and Toughness Tests

As of the late 1970’s, the testing of concrete was based mostly on strength and durability. Test methods at the time lacked the capability to assess different concretes based on post-cracking toughness and post-peak stress behaviour. New test methods had to be devised with new toughness parameters able to describe the post-peak behaviour. An outline and discussion of 12 such tests from Europe, Asia and America is given by Gopalaratnam (1995). Of these, the Japanese Concrete Institute (JCI) test method JCI-SF4 (JCI, 1984) and the American Society for Testing and Materials (ASTM) C 1018 test (ASTM C 1018, 1992) were two of the most

![Figure 2.12: Setup of JCI-SF4 and ASTM C 1018 test for fibre reinforced concrete](image-url)
successful methods. These were widely adopted, in the UK and internationally (Concrete Society, 2007a). They share a basic setup of a simply supported concrete beam loaded at the third points. This is illustrated in Figure 2.12. The two tests both record a measure of flexural strength and various parameters relating to the load resistance of the specimen after cracking.

The JCI-SF4 standard specifies two tests for the performance of FRC beams. In the first, the flexural strength of the beam ($\sigma_b$) and a residual flexural strength ratio ($\alpha_b$) are evaluated. The residual flexural stress ratio is an index describing the load-carrying capacity of the fibre reinforcement, as a ratio of the pre-cracked and cracked flexural strength ($P_0/P$). The procedure is illustrated in Figure 2.13a. The second test specified by the standard evaluates the flexural toughness ($T_b$). This represents the energy absorbed by the beam throughout loading and is calculated as the area under the load-deflection plot up to a deflection $\delta_{ib}$, equal to $L/150$. This is shown in Figure 2.13b.

The ASTM C 1018 test recorded the load-deflection curve and introduced the concept of toughness indices ($I_5$, $I_{10}$ and $I_{20}$). These indices were measures of the post-crack load capacity of the concrete and are calculated as shown in Figure 2.14a.
The peak load is given by $P$ and the deflection at peak load is given by $\delta$. The toughness indices were ratios of the energy absorbed by the beam up to a specified deflection (e.g. $\text{Area}_{OACD}$ for $3\delta$), to the energy absorbed by the beam up to the peak load $P$ ($\text{Area}_{OAB}$). While the test was suitable for evaluating the performance of different concretes, the determination of the deflection at first crack $\delta$ was subjective.

![Load-deflection graph and derivation of toughness indices](image1)

$I_5 = \frac{\text{Area}_{OACD}}{\text{Area}_{OAB}}$

$I_{10} = \frac{\text{Area}_{OAEF}}{\text{Area}_{OAB}}$

$I_{20} = \frac{\text{Area}_{OAGH}}{\text{Area}_{OAB}}$

Figure 2.14a: Load-deflection graph and derivation of toughness indices (ASTM C 1018)

![Load-deflection graph and calculation of residual flexural strengths](image2)

$f_{150,0.75} = \frac{P_{150,0.75}}{bd^2}$

$f_{150,3} = \frac{P_{150,3}}{bd^2}$

Figure 2.14b: Load-deflection graph and calculation of residual flexural strengths (ASTM C 1609)
in practice. This led to unacceptably wide scatter in the toughness indices reported from different laboratories. Furthermore, the results from the test were highly dependent on specimen geometry and the loading conditions. The measurement of toughness indices dependent on the first crack deflection $\delta$ was retired in 2006 with the publication of ASTM C 1609. A new test method in this standard is based on the same load-deflection procedure. However, the quantification of flexural performance is based on residual flexural strengths ($f$) calculated at beam deflections of $l/600$ and $l/150$ (0.75 and 3mm). This is shown in Figure 2.14b. ASTM C 1609 also allows for the toughness of the beam ($T_{150, 3}$) to be calculated from the area under the load-deflection plot up to a deflection of 3mm.

The three test methods outlined above all require a stiff, servo-controlled testing apparatus capable of applying a constant low rate of deflection. This is to ensure steady recording of the applied load as the high energy release rate associated with cracking occurs. The ASTM C 1399 test (2000) removes this requirement and as a result can be conducted in a greater number of laboratories. The experimental setup is similar to ASTM C 1609, except that a 12mm thick steel plate is attached to the bottom of the beam. The beam and plate are loaded beyond initial cracking, to a deflection of 0.5mm. The beam is then unloaded and the steel plate is removed. The remaining cracked beam undergoes a second, residual load test and the average residual strength (ARS) over a range of deflections is calculated. This ARS value can be used to compare the effectiveness of different fibres, dosages and concrete mixes at sustaining load after cracking.

### 2.3.2 Issues with Standardised Toughness Test Methods

All of the test methods above have been found to exhibit sensitivity to issues such as loading rate, specimen size and operator variability. This is in addition to the fact that there is no single method or parameter in use which provides a full description of the strain-softening behaviour of FRC, suitable for incorporation into design codes. Mindess *et al.* (2003) provided a list of 5 criteria which any toughness parameter suitable for the quality control of FRC must possess. Briefly, these are;
Chapter 2 - Fibre Reinforcement of Concrete

- a physical meaning which is readily understandable;
- calculation methods for toughness values must represent application-specific serviceability conditions;
- low variability across a range of concrete properties;
- quantifies FRC behaviour on a strength, toughness or crack resistance basis, reflecting properties of the load vs. deflection behaviour; and
- independent of specimen size and geometry.

In addition to these criteria, there are several requirements pertaining to the fracture performance of the specimen which must be met. The fracture surfaces must be sufficiently large in relation to the specimen size to capture all the energy absorbing mechanisms in the fracture process zone, without making the specimen too large that testing is practically difficult. This is important in FRC, where the size of the fracture process zone is increased relative to normal concrete. Stable crack progression is also necessary to capture the full softening behaviour.

2.3.3 The Wedge Splitting Test

The wedge splitting test is an alternative test method for quasi-brittle cementitious composites, with several advantages over beam flexure tests in assessing the fracture properties of fibre reinforced concretes. The test provides stable crack propagation over a large area, and a low ratio of specimen volume to fracture surface. The procedure is based on the compact tension test and has been modified for use with FRC (Rieder, 2002). Cuboid specimens are formed with vertical and side notches as shown in Figure 2.15.
The side notches are 15mm deep and the vertical starter notch is 25mm deep, providing an intact crack area 175mm long \((B)\) by 155mm high \((W)\). The notches are cut by a saw after the moulded specimen has set. This ensures that the bulk concrete fracture properties are tested, avoiding boundary effects in the fibre and aggregate distribution. The wedge is driven into the specimen at a constant crosshead speed, generating a horizontal splitting load \(F_H\). This load separates the fracture surfaces, and the deformation is measured by the crack mouth opening displacement \((CMOD)\).

Typical Load-\(CMOD\) curves for normal and high strength FRC are shown in Figure 2.16. By recording the \(F_H\) and \(CMOD\) relationship, the following fracture properties can be obtained:
2.3.4 Further Flexural FRC Tests

Other test methods for assessing the properties of FRC have been developed. RILEM’s Technical Committee 162 – TDF (Test and design methods for steel fibre reinforced concrete) proposed a notched FRC beam test (RILEM, 2000). The test used three point loading on a beam with dimensions of 150x150x550 and a central 25mm notch sawn into the tension face. Again, a stiff deflection-controlled testing apparatus was recommended to conduct the test and the following concrete properties were measured:

- Fracture energy $G_F$ (N/m)
- Notch tensile strength $\sigma_{nts}$ (MPa), a factor which combines the tensile and flexural stress applied to the specimen, 
- Critical stress intensity factor $K_{IC}$ (MPam$^{0.5}$), which incorporates the specimen geometry and maximum load, 
- Critical energy release rate $G_{IC}$ (N/m), which measures the energy required for initiation of the crack.
• Limit of Proportionality (LOP); representing the bending stress at which irreversible deformation of the beam first occurred;
• two measures of equivalent flexural strength, at deflections 0.65mm and 2.65mm greater than the deflection at LOP; and
• optional measurement of the CMOD, allowing for the relationship between the beam deflection and crack width to be evaluated.

A further flexural test for FRC is the centrally loaded round panel arrangement proposed by ASTM C 1550-12 (ASTM, 2012). A disc-shaped FRC specimen is supported at the edge by three symmetrically arranged support pivots and loaded centrally. The energy absorption capacity of the FRC is evaluated as the work done in reaching a prescribed deflection. Due to the symmetrical nature of the test arrangement and reliability of the cracking pattern, it is considered a better method of evaluating the performance of FRC than flexural beam tests. It has also been used to test shotcrete panel specimens, formed using a vertically oriented disc mould sprayed and built up horizontally (Bernard, 2003).

As described before, the inclusion of fibres affects the workability of concrete in the plastic state and on the impact strength of hardened concrete. Specific test methods have been adopted to assess these particular characteristics, although there are several concrete properties for which the conventional testing methods remain valid. These include compressive strength, air content, shrinkage, freeze-thaw resistance and creep (ACI 544.2R-89, 1989).
2.4 The Strux 90/40 Fibre

The polymer fibre used in the current project is Strux 90/40, made by Grace Construction Products (Grace, 2004). Its dimensions and properties are explained as well as its performance at high temperatures. The results of flexural and wedge splitting tests, conducted by the fibre manufacturer, are presented. These show the detrimental effect of elevated temperatures on polymer FRC. The results of a basic material investigation into Strux-reinforced concrete are then also outlined.

2.4.1 Introduction, Shape and Properties

The fibre is a polyolefin mono-filament macro fibre with dimensions of 40 x 1.4 x 0.105mm. The exact composition of the fibre material is proprietary blend of polyethylene and polypropylene. The fibres are shown in Figure 2.17, next to some common designs of steel fibre. It is categorised as a Class II polymer fibre according to BS EN 14889-2, as the equivalent fibre diameter is calculated as 0.443mm. This equivalent diameter gives it an aspect ratio of 90. The rectangular cross-section is designed to provide a high surface area to volume ratio, maximising the available area for shear bond with concrete. The cross section also gives the fibre increased flexibility compared to a round fibre; this is beneficial during handling and mixing as

Figure 2.17: Strux 90/40 polymer macro fibres (left) and typical steel fibres (right)
it enables the fibres to bend past each other without entanglement. This also improves the finish obtained after floating, as flexible fibres have a lower tendency to emerge from the surface.

Table 2.2 shows physical properties of the fibre, from the manufacturer’s literature (Grace, 2004). Multiplying the tensile strength by the cross-sectional area gives a nominal fibre breakage load of 91.14N. Assuming that 20mm of fibre is embedded in a matrix, the minimum bond strength required for fibre breakage instead of pull-out is 1.63MPa.

### 2.4.2 Fibre Strength at Elevated Temperatures

At normal temperatures, the fibres are stiff and have a high Young's modulus (9.5GPa). This is due to the melt-extrusion manufacturing process. The molten polymer material is stretched during formation of the fibres such that, when cooled, the molecules in the fibre possess a preferential orientation in the axis of the fibre. However, this strength reduces at elevated temperatures. Figure 2.18 shows the reduction in fibre strength as the temperature of the fibre increases, as tested by the fibre manufacturers (Rieder, 2004a). At 120°C, the fibre has lost approximately half its strength. The loss of strength continues until the melting point is reached at 160°C. Softening due to heating reduces the fibre’s ability to prevent crack faces from opening further wider. The shear bond capacity between the fibre and the matrix also loses strength as the fibre weakens, which tends to increase the occurrence of pull-out as the failure mechanism. This tendency is offset however, as the reduction in strength reduced the critical length and increases the chance of fibre breakage.
2.4.3 Properties of Strux 90/40 Reinforced Concrete from Flexural Tests and Wedge Splitting Tests

This thesis investigates the fire resistance of FRC composite deck slabs, reinforced with 5.3kg/m$^3$ (0.58% by volume) of polymer fibre. This section presents the results from beam tests and wedge splitting tests conducted by the fibre manufacturer on a representative composite deck concrete mix, at temperatures from 20°C to 150°C (Rieder, 2004b). The concrete mix is given in Table 2.3. The test method was an adapted ASTM C 1018 test to provide values of flexural and equivalent flexural strength.

Table 2.3: Constituents of Strux FRC in beam and wedge splitting tests (Rieder, 2004b)

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Mass per m$^3$</th>
<th>Specific gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>1055</td>
<td>2.94</td>
</tr>
<tr>
<td>Sand</td>
<td>900</td>
<td>2.76</td>
</tr>
<tr>
<td>Cement</td>
<td>325</td>
<td>3.15</td>
</tr>
<tr>
<td>Water (W/C ratio 0.585)</td>
<td>190.2</td>
<td>1</td>
</tr>
<tr>
<td>Air</td>
<td>-</td>
<td>2%</td>
</tr>
<tr>
<td>Strux 90/40</td>
<td>5.3</td>
<td>1.05</td>
</tr>
<tr>
<td>Adva (superplasticiser)</td>
<td>1.78</td>
<td>0.92</td>
</tr>
<tr>
<td>Total Unit Weight</td>
<td>2472kg/m$^3$</td>
<td></td>
</tr>
</tbody>
</table>

Figure 2.18: Temperature versus tensile strength of polymer fibre (Rieder, 2004a)
Figure 2.19 shows the flexural test results. The solid line ($\sigma_{\text{flex}}$) shows the variation in peak flexural strength of the beams with increasing temperature. The dashed line ($f_{e,3}$) represents the equivalent flexural strength at a post-peak deflection of 3mm. The peak flexural strength shows non-linear behaviour as the temperature increases, rising after 100ºC. This value is solely dependent on matrix strength at this fibre dosage though, and has little relation to the effectiveness of the fibres. No conclusion was available to explain the non-linear behaviour of the flexural strength (Rieder, 2004b). The dashed line shows that the post-crack toughening due to the Strux fibres does reduce consistently with increasing temperature. The equivalent flexural strength approaches zero as the temperature reaches the fibre melting point, due to the fibres softening and becoming unable to resist further crack opening. The reduction in toughening caused by fibre softening at elevated temperatures is therefore evident.

Figure 2.20 shows data from the wedge splitting tests. The fracture energy $G_f$ required to increase the CMOD to 2mm, 3mm, 5mm, 10mm and 20mm is plotted cumulatively for a range of temperatures. The $G_f$ values were calculated by integrating the $F_H$-CMOD relationship up to each given CMOD value.
The energy required to separate the crack faces of the wedge splitting specimen is shown to decrease with rising temperature. The reason for this is the reduction in energy absorption ability of the fibres as they get hotter. The toughening ability of the fibres in the 20°C specimen is clear; in opening the CMOD from 2mm to 20mm, the energy absorbed increased from 882N/m to 5066N/m, an extra requirement of 4184N/m. The specimen tested at 140°C however, recorded only a small increase of 328N/m to achieve the same rise in CMOD. This effect is due to the loss of strength of the fibres with increasing temperature.

The previous two figures quantify the way in which concrete temperatures approaching the melting point of Strux fibres negate the toughening effect. As reported by the fibre manufacturer, “At 150°C STRUX 90/40 reinforced concrete loses its ability to provide any crack bridging effect due to the fact that the tensile strength of STRUX 90/40 at this temperature approaches zero” (Rieder, 2004b).

Figure 2.21 shows the relative reduction in equivalent flexural strength $f_{e,3}$ from the beam tests and the reduction in cumulative fracture energy at CMOD of 10mm from the wedge splitting tests. The lines from the different concrete tests show a similar trend, representing the loss of fibre effectiveness in concrete approaching 160°C.
2.4.4 First Laboratory Investigations into PFRC Material

The project work at Edinburgh began with two simple material investigations into Strux 90/40 reinforced concrete. These were intended firstly as familiarisation exercises, in order to achieve some level of experience in the handling and testing of Strux 90/40 FRC before casting the main test slab specimens.

Test 1: Effect of Different Water-Cement Ratios

An initial trial batch of polymer fibre reinforced concrete was cast to mix design similar to the one shown in Table 2.3, used in the flexural and wedge splitting tests. Six 102mm cube specimens and six 102mm by 204mm cylinder specimens were produced and tested for compressive strength (BS EN 12390-3, 2002) and split tensile strength (BS EN 12390-6, 2002) at seven days. The full water content was added to the mix in addition to the aggregate-bound water, leading to an increased water-cement ratio in excess of 0.65. This resulted in a weaker than expected average cube compressive strength of 33.4MPa (σ = 0.89MPa) and tensile strength of

Figure 2.21: Reduction in $f_{e,3}$ and $G_{f,10}$ of Strux 90/40 concrete with temperature
2.86MPa ($\sigma = 0.21$MPa). The failure of the cube and cylinder specimens was different to the explosive, sudden failure of plain concrete. Peak strength was accompanied by a slow crushing and plastic deformation, as the unconfined sides of the cube were pressed outward. Figure 2.22a shows a failed specimen with the loose material at the sides broken off, leaving an hourglass shape and exposed fibres visible. The failure mechanism of an unconfined FRC cube which accounts for this observation is also illustrated in Figure 2.22b. The vertical load induces 45° shear stresses in the cube, which tends to push the sides outward. The shear planes along which these stresses act are intersected by fibres, which provide an internal arresting force against the lateral expansion. When the cube strength is reached, the release of energy associated with failure is dissipated due to the energy required to pull out the fibres, over and above the energy required for brittle failure of the matrix and aggregates.

A second trial batch of Strux concrete was produced, with the water and plasticiser content controlled to maintain the required workability at a lower water-cement ratio of 0.48. Twelve cubes and four cylinders were fabricated using the amended, dryer mix and the cylinders and nine of the cubes were tested at seven days. The average compressive strength of the cubes was increased to 57.1MPa ($\sigma = 3.06$MPa) and the tensile splitting strength had increased to 3.7MPa ($\sigma = 0.12$MPa). The failures of
these higher strength specimens were explosive, as with traditional concrete, and there was little evidence of fibres exposed on the failure surfaces. This is likely to be because of the increased matrix strength. A stronger shear bond, caused by the stiffer, stronger matrix, increased the likelihood of fibre failure coming from breakage rather than pull-out. When this occurred, the energy absorption properties linked to the frictional pullout of fibres were negated and the fibres had little effect.

**Test 2: Effect on Strength due to Incorporation of Fibres**

To determine the effect Strux fibres had on the strength of concrete, two batches were prepared; one batch consisted of the recommended Strux mix, and the second was identical except the fibres and plasticiser were omitted. Twelve cube specimens and six cylinder specimens of each mix were prepared and tested as in the previous investigation. It was assumed beforehand that the fibres would have little effect on the compressive strength, and would slightly increase the tensile strength. The fibre reinforced cubes had an average strength of 37.9MPa (σ = 0.50MPa) while the unreinforced cubes had an average strength of 32.5MPa (σ = 0.77MPa). Also, the Strux cylinders had an average tensile strength of 2.3MPa (σ = 0.06MPa) and the plain cylinders had an average strength of 2.8MPa (σ = 0.11MPa). Therefore, the addition of fibres increased the compressive strength as measured by BS EN 12390-3 by 14.2%, and decreased the tensile strength as measured by BS EN 12390-6 by 17.9%.

The second investigation highlighted how important the particular testing method used is in assessing properties of FRC, and how individual tests should be reviewed for their effectiveness. The increase in cube strength with the addition of fibres makes sense considering the extra energy needed not just to separate the internal crack faces but also to strip out the fibres. At this cube strength the fibres were again able to control the rate of energy release associated with failure, and pull-out was observed.

The reduction in tensile splitting strength caused by the presence of fibres can possibly be explained. In the splitting cylinder test, there is only one crack plane. The tensile
strength of concrete measured this way is primarily influenced by the matrix strength across this crack plane. Fibres crossing the crack plane, at the dosage used, would not have had the capability to increase the crack strength of the matrix significantly. Also, assuming a random distribution of fibres in the cylinder specimen, there may have been fibres aligned along the crack plane. For each fibre aligned in this way, the fracture area of the matrix is reduced by the area of one fibre, $56\text{mm}^2$. A plurality of fibres aligned along the crack face in this way may have significantly lowered the effective matrix crack area, reducing the splitting tensile capacity of the cylinders.

**Chapter 2 Summary**

The characteristics and performance of fibre reinforced concrete have been introduced. The improvements in fibre technology leading to the current state of the art have been outlined. Physical properties of fibres and FRC, and the methods used to test their effectiveness as reinforcement, have been explained. The properties and reinforcing capabilities of the Strux 90/40 polymer fibre, used throughout this research, have also been described.
This chapter describes the behaviour and testing of composite steel and concrete slabs exposed to fire. The topics covered include:

- Material properties of steel and concrete;
- Design and fire testing of composite slabs, including the Cardington tests;
- Modern, performance-based fire safety engineering of composite slabs;
- The design of polymer-fibre-reinforced composite slabs, and implications for their fire resistance.
3.1 Introduction

This section introduces composite construction and describes the material and thermal behaviour of both concrete and steel. This allows understanding of the response of structures to high temperatures, and the relationships are used later in numerical structural analysis. It then introduces compartment and design fires, and explains slab design and fire rating.

3.1.1 Composite Steel-Concrete Slabs

Modern composite steel-concrete slabs consist of a profiled steel deck and a poured concrete topping. Sheets of steel decking (trapezoidal or re-entrant) are laid spanning between the supporting beams. This acts as both permanent formwork during casting, and tensile reinforcement for the slab section after hardening. Indentations and dovetail segments along the ribs provide shear strength and mechanical interlock with the poured concrete, and this bond gives moment resistance to the hardened slab. Composite behaviour with the supporting steel frame is achieved through shear studs, welded through the deck onto the supporting beam flanges. These studs fix the hardened slab to the steel frame, which contributes to the stiffness and load-bearing capacity of the structure. Additional reinforcement is usually provided by a light steel mesh placed within the concrete, which controls cracking and shrinkage. Figure 3.1 shows mesh being laid over a profiled deck prior to casting.

Figure 3.1: Laying steel mesh on profiled deck (One New Change, London)
Rules for the design of composite floors for buildings are explained in Eurocode 4 Part 1-1 (BS EN 1994-1-1:2004). Continuous floor slabs are usually designed as a series of simply supported spans, provided that nominal reinforcement is placed over intermediate beam supports. This is commonly adopted and the design ambient load capacity is controlled by the flexural resistance of the composite slab section.

Composite floors in structures can be divided into slab panels, which are supported at the perimeter by primary beams. A generic example of a 9 metre panel is illustrated in Figure 3.2. The primary beams span between vertical supports; either columns or a structural core. Secondary beams support the slab between the primary beams. Composite slab spans (across the secondary beams) are usually 3 to 4m, dependent on the decking profile and imposed load.

Decking sheets have either trapezoidal or re-entrant profiles, with depths of 45-80mm and sheet thicknesses of 0.9-1.2mm. Decking is usually rolled from grade S350 steel, galvanised with 275g/m$^2$ zinc to prevent corrosion (MCRMA, 2009). Reinforcing mesh for composite slabs usually has bars at 200mm centres, with diameters from 6-10mm. These diameters correspond to the mesh designations A142 to A393, as explained in BS 4483 (BS 4483:2005). Supporting beams can either be solid or cellular rolled UB sections, or lightweight trusses.

![Figure 3.2: Slab panel diagram](image)
3.1.2 Steel Material Behaviour at High Temperatures

Design guidance on the material behaviour of structural and reinforcing steel at high temperatures is presented in Eurocode 3 Part 1-2 (BS EN 1993-1-2:2005) and Eurocode 4 Part 1-2 (BS EN 1994-1-2:2005). As temperature increases, the yield stress, Young's modulus and the limit of proportionality of steel reduce from ambient ($T=20^\circ\text{C}$) values. These reductions occur at different rates; the reduction factors for structural and reinforcing steel are shown in Figure 3.3 and Figure 3.4 respectively. Eurocodes 3 and 4 also provide data for the conductivity, specific heat capacity and thermal expansion of steel, used to calculate design temperatures. These relationships are shown in Figures 3.5, 3.6 and 3.7 respectively.

![Figure 3.3: Reduction factors for i) yield strength, ii) Young's modulus and iii) limit of proportionality of structural steel (Eurocode 4, 2005)](image)

![Figure 3.4: Reduction factors for i) yield strength, ii) Young's modulus and iii) limit of proportionality of reinforcing steel (Eurocode 4, 2005)](image)
Figure 3.5: Conductivity of steel at high temperatures (Eurocode 4, 2005)

Figure 3.6: Specific heat of steel at high temperatures (Eurocode 4, 2005)

Figure 3.7: Thermal elongation of steel at high temperatures (Eurocode 4, 2005)
Steel loses over 50% of its strength when it reaches a temperature of 600°C, and loses around 80% at 700°C. Because of this, steel beams and columns in buildings are often designed to reach a critical temperature of 550 °C to 620°C at the end of the fire resistance period.

In Figure 3.6, the spike in specific heat capacity at 735°C is modelled due to a phase change in the crystalline steel structure. The atomic arrangement of the alloy changes form, from a face-centred to body-centred cubic arrangement. This requires the atoms to move further apart (Kodur, 2010). The transition requires extra heat energy, temporarily raising the specific heat of the material. In practice this results in a decrease in the heating rates of steel at these temperatures. The thermal elongation of steel is also affected by this phase change, with a delay in expansion caused by the transition as shown in Figure 3.7.

Eurocodes 3 and 4 also both provide a unique design stress-strain relationship for structural and reinforcing steel, dependent on the properties illustrated in Figures 3.3 and 3.4. This relationship can be amended to include the effect of strain hardening, when advanced calculation models are used. Modelling this phenomenon allows the maximum stress to be increased during ductile straining, for temperatures <400°C.

3.1.3 Concrete Material Behaviour at High Temperatures

The thermal response of concrete is complex due to the free water, aggregates and hardened cement paste phases each contributing differently to the overall performance. Accurate modelling of the material behaviour and temperature increase due to fire requires a highly coupled thermal, hydraulic and mechanical analysis. Thermal degradation of the aggregate and breakdown of cement paste cause an overall strength reduction, and in particular the moisture content influences the heating rate. However, in most cases of structural analysis, a simplified calculation model is considered adequate (Khoury, 2008b).
Chapter 3 - Behaviour of Composite Slabs in Fire

The material properties for concrete design described by Eurocodes 2 and 4 (BS EN 1992-1-2:2004, BS EN 1994-1-2:2005) take into account NWC (normal weight concrete), LWC (lightweight concrete), and siliceous or calcareous aggregates. For calculating the specific heat capacity, the moisture content is considered. Figure 3.8 shows the reduction factors for concrete compressive strength with increasing temperature, for NWC, LWC and calcareous concrete.

The tensile strength of concrete is conservatively ignored in design. If it is taken into account, the strength reduces linearly with temperature from the ambient value to zero between 100ºC and 600ºC (EN1992-1-2:2004).

Figures 3.9, 3.10 and 3.11 show the conductivity, specific heat capacity and conductivity properties of concrete for design according to Eurocodes 2 and 4 (Eurocode 2, 2004; Eurocode 4, 2005). The density and type of aggregate used govern the conductivity and thermal expansion behaviour. The specific heat of concrete is roughly 1kJ/kg.K, except for a peak between 100ºC and 200ºC. This peak in heat capacity represents the endothermic effect of vaporisation of water within the concrete. The height of the spike is controlled by the moisture content. This peak in specific heat capacity is how the hydral component of the concrete response is incorporated into the Eurocode material model.

Figure 3.8: Compressive strength reduction factors for NWC, LWC, and concrete with calcareous aggregates (Eurocode 2, 2004; Eurocode 4, 2005)
Figure 3.9: Conductivity of NWC and LWC (Eurocode 4, 2005)

Figure 3.10: Specific heat of NWC and LWC (Eurocode 2, 2004)

Figure 3.11: Thermal elongation of NWC, LWC and calcareous aggregate concrete (Eurocode 2, 2004; Eurocode 4, 2005)
Chapter 3 - Behaviour of Composite Slabs in Fire

The section above described the material and thermal properties of concrete and steel, over the range of temperatures commonly experienced in fires. The differing reactions of concrete and steel to heat are key to understanding the fire behaviour of composite slabs. The next section briefly describes the behaviour of fires in buildings, and the rates of temperature increase for which structures are designed.

3.1.4 Fire Behaviour

The general fire design scenario for a composite slab is a fire in the compartment below. Compartment fire behaviour can be characterised as having three distinct phases (Drysdale, 2005). These are illustrated in Figure 3.12 and explained below;

- the growth period, where the fire is localised with low heat release rates;
- the fully developed phase, when the fire has grown to involve almost all the combustible materials in the compartment; and
- the decay period, after the peak heat release rate and the fire begins to run out of fuel or oxygen.

The transition between growth and full development is usually sudden. This phenomenon is called flashover, and can be caused by the heated compartment boundaries re-radiating heat back towards the other combustible materials present.

Figure 3.12: Heat release rate of compartment fire over time (Drysdale, 2005)
This accelerates ignition of fuel throughout the compartment and causes a rapid rise in heat release rate, smoke production and temperature. The fully developed fire then burns until all the available fuel or oxygen in the compartment is consumed. The heat release rate then begins to decrease, leading to the decay phase and flame extinction.

**Design Fires**

In structural fire design of composite slabs, the soffit and supporting steelwork are subjected to a particular time-temperature relationship representing the temperature rise in the compartment below. These relationships (fire curves) define the growth rate, peak temperature and fire duration. Fire curves for use in structural design are described in Eurocode 1 Part 1-2 (BS EN 1991-1-2:2002). These include:

- the standard temperature-time curve, used historically to represent the effect of a post-flashover fire,
- parametric fire curves, which simulate more natural fires and incorporate the effect of compartment ventilation, materials and fuel load,
- specific curves for external or rapid-burning hydrocarbon fuel fires, and;
- rules for advanced fire models, taking into account gas phase properties, mass exchange/smoke development and burning characteristics of the fuel.

Figure 3.13 shows the standard fire curve for one hour, and a parametric fire curve incorporating a decay phase. The standard curve can applied to simple design scenarios, and is still used in fire resistance testing of structural elements. Parametric fires take into account ventilation effects, fuel load and compartment materials. They are used regularly in the fire design of common steel-framed buildings with simple floor plans. However, their application is limited to spaces up to 500m$^2$ in area and a maximum height of 4m. Many modern composite structures have features exceeding these criteria such as atriums and open floor plates. Parametric fires are also inapplicable for areas with unusually dense fire loads. Such situations require design fire conditions assessed individually, on a probabilistic basis.
3.1.5 Composite Slab Design and Fire Rating

In conventional design, the composite slab spans across the supporting secondary beams utilising its flexural resistance (Figure 3.2). When affected by fire from below, the slab section and beams heat up. This starts to lower the slab moment resistance, until after certain time the slab cannot support the incident load, and failure occurs (failure in this instance meaning a certain critical deflection has been reached). The fire rating of the slab is the length of time for which it will fulfil its structural role before failure. Early fire design guides stated that an unprotected composite slab would provide a fire rating of at least 30 minutes, based on testing (ECCS, 1993).

If a higher fire rating was required, or a slab failed to reach 30 minutes, two options were available. Firstly, insulation material could be applied to the supporting beams, slowing their temperature increase and the deflection rate of the slab. Secondly, additional reinforcing bars could be placed in the troughs of the decking profile. This increased the slab moment resistance, providing extra flexural capacity for high temperatures. In this case, concrete cover to the extra rebar was an important consideration, as their effectiveness was reduced at high temperatures.
Design on this basis was widely adopted. It was considered to be conservative, as the beneficial effects of structural continuity were well-known but ignored. Modern fire design of composite structures uses the real behaviour observed from full-scale structural fire testing, which is discussed in later sections.
3.2 Fire Resistance Testing of Composite Slabs

This section introduces the standardised furnace tests used to assess the fire performance of composite slabs. The test construction, features and performance criteria are described. It is then explained how the fire behaviour observed under test conditions is not exactly representative of the fire performance of slabs within continuous steel frame structures, which was apparent during the full-scale fire tests on a composite steel-framed building, carried out at Cardington in 1995. This gives background on the design and aims of the current experimental investigation, which was designed to provide information on slab behaviour which is not captured by standardised tests.

3.2.1 Test Construction and Features

The rules governing fire tests for composite slabs are laid out in BS EN 1363 Part 1 (BS EN 1363-1:1999), supplemented by BS EN 1365 Part 2 (BS EN 1365-2:2000). The size of slab specimen tested is usually determined by the area of furnace available to perform the test; however tested spans of 3 to 4 metres are most common.

The slab is fixed in place above a test furnace “in a special test frame designed to reproduce the required or the design boundary and support conditions” (BS EN 1363-1:1999). Recently in practice, the most common restraint provision is for the slab to be fixed to support beams, spanning in one direction across an intermediate beam. A diagram of a generalised test specimen and furnace construction is shown in Figure 3.14.

This configuration creates a negative moment over the intermediate support to represent the effect of structural continuity, but the ends of the specimen are free to rotate. A discussion of boundary conditions and how they relate to the applicability of the test results is given below in section 3.2.3.
Chapter 3 - Behaviour of Composite Slabs in Fire

Conditioning

The strength and moisture content of a tested slab must reflect service conditions, at a time when hydral equilibrium has been reached. While concrete usually requires 28 days to achieve its characteristic strength, a test slab takes four to five months to dry out. This is necessary to demonstrate representative thermal behaviour. Fresh specimens with high moisture contents will exhibit altered heating rates and higher spalling risk due to the energy balance involving evaporation of water, as discussed in Section 3.1.3. The moisture content must be measured prior to testing with either a direct meter or oven dried core sample. Acceptable levels of moisture are between 1 to 5% by mass, according to BS EN 1363 Annexe F: “Guidance on Conditioning”.

Furnace

The furnaces used in composite slab tests are instrumented with thermocouples to monitor the temperature increase. BS EN 1363 requires that the temperature in the furnace compartment follows the standard fire curve (Figure 3.13), within a defined level of tolerance. This tolerance is 15% during the initial, rapid heating part of the curve, reducing to 2.5% after one hour when the rate of temperature rise is slower.

Loading

The test load applied to the slab specimen before and throughout heating is determined by the organisers, and must be related to the ultimate load of the specimen (BS EN 1363). The load applied is critical if the test is used as a realistic
basis for further analysis. BS EN 1994-1-2 provides guidance for selecting the load level, recommending a fire limit state load which is 65-70% of the ultimate load. In practice, 6.7kN/m$^2$ is the most common applied load in fire tests to represent fire service conditions (see Figure 3.15). A hydraulic mechanism is typically used to apply pressure through steel plates on the slab surface. These plates allow continuous pressure to be provided while following the deformed shape of the slab.

**Test Instrumentation**

The slab temperature evolution is measured with thermocouples at regular intervals through the depth, and across the unheated surface. These allow measurement of the thermal profile through the slab, and can check that the furnace is heating the slab evenly. Deflection is measured by one or more gauges placed on the unheated face at the centre of the main span.

**3.2.2 Performance Criteria**

The fire performance of composite slabs in standard tests is evaluated using three criteria. These are:

- Integrity,
- Insulation, and
- Loadbearing capacity.

The measures of integrity and insulation relate to the ability of the slab to function as a separating element between two storeys in a structure. The integrity criterion is breached by sustained flaming through the slab, excessive crack widths or ignition of cotton pads applied to the unheated face. The presence of steel decking is usually considered sufficient in practice to maintain integrity. The insulation criterion limits the average temperatures achieved at the unheated face to 140°C, or 180°C at any single location. However, as Figure 3.15 shows, in each of the of slab tests conducted in the UK between 1983 and 1991 the unheated surface did not exceed 140°C after 90 minutes. (SCI 1991a).
Chapter 3 - Behaviour of Composite Slabs in Fire

The loadbearing capacity criterion is determined by the deflection and deflection rate of the slab. The test reaches failure at a deflection of $L^2/400d$, where $L$ is the unsupported span (mm) and $d$ is the depth of the slab (distance from extreme fibres of design compressive and tensile zones, mm). For common slab assemblies this deflection limit equates to 200-220mm. The deflection rate limit is set to $L^2/9000d$, which for common tests equates to 6-10mm/min.

The report from a standard test states the temperatures achieved, describes the extent of cracking and the fire resistance period achieved by the specimen. This is measured from ignition to the time when one of the above criteria are met. Further analysis of the data is often made to draw up design tables for individual profiles.

However, the results from such experiments have limited application to real fires affecting composite slabs in steel framed buildings. The test only reflects the ability of a width of slab to maintain flexural capacity under a prescribed and unrealistic temperature exposure. While structural continuity is often simulated with the intermediate beam, this is only in the longitudinal direction and ignores the contribution to behaviour made by a surrounding slab. Despite this, the single element fire test is still assumed to be a conservative basis for design.

Figure 3.15: Full summary of 18 composite slab fire tests in U.K. 1983-1991 (SCI, 1991a)

<table>
<thead>
<tr>
<th>PROFILE</th>
<th>CONCRETE TYPE</th>
<th>SLAB DEPTH (mm)</th>
<th>SPAN (m)</th>
<th>IMPOSED LOAD (kN/m²)</th>
<th>REINFORCEMENT</th>
<th>SURFACE TEMP. (°C)</th>
<th>TEST PERIOD (min)</th>
<th>TEST REF.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Robertson OLS9</td>
<td>LWC</td>
<td>130</td>
<td>3.0x</td>
<td>6.7</td>
<td>A142 mesh</td>
<td>73</td>
<td>1-1.5 h</td>
<td>60</td>
</tr>
<tr>
<td>Robertson OLS9</td>
<td>LWC</td>
<td>130</td>
<td>3.0*</td>
<td>6.7</td>
<td>A142 mesh</td>
<td>70</td>
<td>1-1.5 h</td>
<td>104</td>
</tr>
<tr>
<td>Robertson OLS9</td>
<td>LWC</td>
<td>130</td>
<td>3.0c</td>
<td>6.7</td>
<td>A142 mesh</td>
<td>95</td>
<td>1-1.5 h</td>
<td>109</td>
</tr>
<tr>
<td>Halocure (UK)</td>
<td>LWC</td>
<td>120</td>
<td>3.0c*</td>
<td>6.7</td>
<td>A142 mesh</td>
<td>60</td>
<td>1-1.5 h</td>
<td>109</td>
</tr>
<tr>
<td>Hafin (UK)</td>
<td>LWC</td>
<td>110</td>
<td>3.0c</td>
<td>5.26</td>
<td>Y6 @ 225 as mesh</td>
<td>110</td>
<td>1-1.5 h</td>
<td>109</td>
</tr>
<tr>
<td>HYF C146</td>
<td>NWC</td>
<td>100</td>
<td>3.0c</td>
<td>5.75</td>
<td>Y6 @ 150 as mesh</td>
<td>90</td>
<td>1-1.5 h</td>
<td>109</td>
</tr>
<tr>
<td>Preload OLS9</td>
<td>NWC</td>
<td>140</td>
<td>3.6c*</td>
<td>6.7</td>
<td>A153 mesh</td>
<td>66</td>
<td>1-1.5 h</td>
<td>90</td>
</tr>
<tr>
<td>Metaplex A55</td>
<td>NWC</td>
<td>140</td>
<td>3.6*</td>
<td>6.7</td>
<td>A193 mesh</td>
<td>65</td>
<td>1-1.5 h</td>
<td>90</td>
</tr>
<tr>
<td>Halocure (UK)</td>
<td>LWC</td>
<td>150</td>
<td>3.0c*</td>
<td>10.0</td>
<td>A183 mesh</td>
<td>46</td>
<td>1-1.5 h</td>
<td>120</td>
</tr>
<tr>
<td>Ribdeck 60</td>
<td>LWC</td>
<td>140</td>
<td>3.0c*</td>
<td>10.0</td>
<td>A183 mesh</td>
<td>46</td>
<td>1-1.5 h</td>
<td>120</td>
</tr>
<tr>
<td>Alphablock</td>
<td>LWC</td>
<td>140</td>
<td>3.0c*</td>
<td>8.5</td>
<td>A252 mesh</td>
<td>56</td>
<td>1-1.5 h</td>
<td>120</td>
</tr>
<tr>
<td>SMD 1551</td>
<td>NWG</td>
<td>140</td>
<td>3.6*</td>
<td>6.7</td>
<td>A153 mesh</td>
<td>96</td>
<td>1-1.5 h</td>
<td>135 SMD 1</td>
</tr>
<tr>
<td>Oulkspan 95</td>
<td>NWG</td>
<td>140</td>
<td>3.6*</td>
<td>5.0</td>
<td>A142 mesh</td>
<td>96</td>
<td>1-1.5 h</td>
<td>135 SMD 1</td>
</tr>
<tr>
<td>Oulkspan 405</td>
<td>NWG</td>
<td>150</td>
<td>3.6*</td>
<td>7.0</td>
<td>A252 mesh</td>
<td>74</td>
<td>1-1.5 h</td>
<td>135 SMD 1</td>
</tr>
<tr>
<td>Multideck 60</td>
<td>NWG</td>
<td>150</td>
<td>6.0c*</td>
<td>6.7</td>
<td>A252 mesh</td>
<td>68</td>
<td>1-1.5 h</td>
<td>135 SMD 1</td>
</tr>
<tr>
<td>Multideck 8D</td>
<td>NWG</td>
<td>150</td>
<td>4.0c*</td>
<td>6.7</td>
<td>A252 mesh</td>
<td>68</td>
<td>1-1.5 h</td>
<td>92</td>
</tr>
</tbody>
</table>

63
3.2.3 Testing and Boundary Conditions

A rapid increase in the use of composite construction occurred in the UK throughout the 1980s. Until 1982 however, the majority of fire testing on composite slab specimens was performed with simple support conditions, and only achieved fire resistance periods of under one hour, unless additional reinforcing bars were placed in the deck troughs. These tests resulted in uneconomical design rules for composite slabs, because they ignored the beneficial effects of structural continuity (Cooke, 1988, CIRIA, 1985).

The lack of test data applicable to continuous slabs was first addressed between 1983 and 1986. A series of six furnace tests with more representative support conditions was conducted by the Construction Industry Research and Information Council (CIRIA), and three tests were performed by the British Steel Corporation (BSC). These tests are summarised in the first 9 entries in Figure 3.15.

The CIRIA experiments included one simply supported test, and four unequal-span tests using an intermediate support beam to simulate continuity (Figure 3.14). These tests had main spans \( L \) of 3.0m and 3.6m; the setup and the resulting bending moment profile are shown in Figure 3.16. A larger test was also conducted on a 4m wide slab with two equal spans of 3m. This test used a sliding support along one edge, which permitted lateral movement. This attempted to model the behaviour of an 8m slab, using a furnace only 4m wide. A diagram of this test construction is shown in Figure 3.17.

The BSC tests were similar to the large CIRIA test. These attempted to simulate a corner of a building, with two equal spans of 3m and a further cantilever of 1m. A diagram of this test layout is shown in Figure 3.18. Again, 8m width was simulated by using a sliding joint mechanism. It was concluded from these tests that a fire resistance period of 90 minutes or more was possible, without additional bar reinforcement, if structural continuity could be developed (CIRIA 1985). Further tests were conducted by different manufacturers with their own decking profiles up.
Chapter 3 - Behaviour of Composite Slabs in Fire

Figure 3.16: Test arrangement and moment diagram for CIRIA unequal span furnace tests (CIRIA, 1985)

Figure 3.17: Test arrangement for CIRIA large panel equal span test, incorporating sliding top edge to simulate 8m width (CIRIA 1985)

Figure 3.18: BSC fire test arrangement modelling a corner of a building, also incorporating a sliding edge (SCI, 1991a)
to 1991 (Figure 3.15). This level of complexity in applied boundary conditions for slab furnace tests did not become commonplace, however, as speed and economy took priority. Continuous setups such as the unequal span arrangement in Figure 3.16 became standard. This permitted calculation of design tables based on simulated continuity, yet the guidance produced was still highly conservative compared to real behaviour.

3.2.4 Observed Fire Behaviour and the Cardington Experiments

In June 1990, a fire occurred at the Broadgate Phase 8 project in London. This was a newly constructed steel frame building undergoing fit-out; full insulation had not yet been applied to beams and columns. Despite this, the structure suffered only minor damage during the 4.5 hour fire, which for 2 hours reached temperatures over 1000ºC. The fire resistance observed was well in excess of the design values for individual members. Structural repairs were a minor component of the refurbishment work, representing less than £2M out of the £25M total cost (SCI, 1991b).

A similar 1991 fire at Churchill Plaza, Basingstoke demonstrated similar levels of fire resistance. The building experienced a 4 hour fire, with complete burnout of two floors, suffering only superficial structural damage (BCSA, 2005). These events changed the focus from adequate performance of separate structural components, to consideration of the stability and load-carrying mechanisms of the overall frame.

To investigate whole-frame behaviour in fire, a series of tests were conducted on an eight-storey purpose-built composite structure at Building Research Establishment (BRE), Cardington in 1996. This addressed the limitations of standard furnace testing by subjecting slabs with realistic boundary conditions to natural fires. The test structure complied with the design codes of the time (BS 5950 Part 4, 1994; BS 5950 Part 8, 1990). Figure 3.19 shows the floor layout of the test structure, with the names and locations of the six tests over the various floors. Figure 3.20 shows the deformation of the heated floor slab in the Large Compartment test (British Steel, 1999).
The Corner and Large Compartment tests were designed to demonstrate the performance of the floor slab. Only the columns and primary beams had board insulation applied. Despite the fire causing high deflections (up to 550mm), the slabs were able to resist the fires without collapse, reflecting the robust structural performance observed in the Broadgate and Basingstoke fires.
The Cardington results were rigorously analysed by several parties (BRE, University of Sheffield, University of Edinburgh) to improve the level of understanding of how real structures responded to fire (SCI, 2006; Usmani et al., 2000). The robustness of the floor slabs was attributed to the highly redundant nature of the frame and slab assembly, coupled with the actions arising from restrained thermal expansion. The slab was able to adopt secondary load-carrying mechanisms not accounted for in the design procedure. In fire the slab acted as a two-dimensional membrane, directing the applied load to the protected structure, bypassing the weakened secondary beams. A combination of compressive and tensile membrane action within the slab allowed it to support greater loads in fire than those predicted by normal design (British Steel, 1999, Wang, 2005).

The Cardington investigation, and subsequent analysis work, generated much greater understanding of how in-plane membrane action contributes to the fire resistance of steel framed structures. This mechanism and the advanced design approaches permitted by utilising it are discussed in the next section.
3.3 Slab Behaviour and Performance-Based Design

This section explains how improved understanding of the behaviour of composite slabs in fire has enabled more efficient structural designs to be adopted, and describes mechanisms that will be investigated by this thesis.

3.3.1 Membrane Action in Composite Slabs at High Temperatures

The Cardington experiments established that structural response to fire is dependent on the forces and deflections arising from restrained thermal expansion of the slab and beams. It was demonstrated that the composite slab experiences a combination of compressive and tensile membrane action as the fire progresses, and that tensile membrane action in particular is a significant mechanism for maintaining structural stability (SCI, 2006).

Figure 3.21 illustrates the development of compressive and tensile membrane action in a slab exposed to fire. Figure 3.21a shows the slab in normal working conditions, spanning across two secondary beams. The solid line within the slab represents the reinforcing mesh. At this stage the load is carried by the flexural capacity of the slab, with the deck between the secondary beams acting in tension (T). Compressive membrane action occurs at low deflections as the flexural stiffness of the floor and beams begins to reduce. This is illustrated in Figure 3.21b. Due to thermal expansion, a large in-plane compressive force is generated within the slab. The magnitude of force is dependent on the stiffness of the surrounding structure. This force braces the slab across the frame, which maintains stability as the flexural resistance declines. However, the slab will continue to deflect with thermal bowing and further weakening of the heated supporting beams.

As the compressive force begins to drop off, the sagging slab eventually starts to pull in on the surrounding frame, as shown in Figure 3.21c. The tensile strength of the reinforcing mesh is used to transmit load to the slab perimeter, i.e. tensile membrane action. The temperature rise of the mesh is limited by the insulating effect of the slab, however its tensile stiffness controls the rate of deflection.
Compressive membrane action is dependent on the surrounding structure being stiff enough to resist the thermal expansion force. However, tensile membrane action can be developed in the slab without any in-plane restraint. A ring of compressive stress forms around the perimeter of the sagging region, which can support the tensile action at the centre. In this way, as the deflection of the slab increases, the load carrying capacity also increases. This is due to compatibility constraints of the two-way spanning deflected shape. Figure 3.22 shows a diagram illustrating this mechanism. Tensile membrane action for improving fire resistance depends on bi-axial bending and two-way action of the slab panel (Bailey, 2002). If this is not provided, slab failure will occur with a regular crack pattern from yield line analysis.
3.3.2 Performance-Based Design and Fire Protection

Up until the Cardington tests it had been standard practice to provide fire protection to all the steel members in a structural frame. This was done to limit the temperature rise in fire until the required fire resistance period was obtained. The Cardington tests demonstrated that in fire, tensile membrane action could be used to support the slab at high deflections. This result attracted attention due to the potential to design safe composite structures using less insulation on structural members.

Bailey proposed a simple method of fire design which utilised the flexural resistance of the slab and supporting beams, including an enhancement factor due to membrane action (Bailey, 2002). The size of the enhancement factor depended on the aspect ratio of the slab, and the maximum allowable displacement. Subsequent tests on composite slabs had demonstrated that ultimate failure of the slab panel occurs due to yielding of the mesh, in the longitudinal direction (Bailey et al., 2000). This placed a limit on the allowable displacement of a slab in fire, dependent on the failure strain of the reinforcing steel.

The proposed design method utilised the composite slab membrane capacity, but nevertheless included several conservative assumptions and structural limitations. The design was based on the floor plan being divided into rectangular floor design.
zones, bounded by protected primary beams and columns. An illustration of a floor divided into square panels is shown in Figure 3.23. The beams within the protected panels are left unprotected. The slab panels are also considered to simply supported, with no lateral restraint at the perimeter. This is conservative as internal floor slabs have a high degree of restraint, which enhances the load capacity through compressive membrane action as described in the previous section.

The Cardington tests provided valuable information on full-frame fire performance. The results allowed benchmarking and optimisation of computational structural models, using commercially available software packages. Finite element analysis (FEA) of structures in fire has allowed design based on predicted building performance in relevant fire scenarios. This has been successfully and widely adopted over the last ten years, again with the aim of providing more economical designs by removing redundant fire protection. Lamont et al. (2006) published a case study for design of a steel framed office building in London. This presented the improvements in fire protection efficiency possible with performance-based design over traditional prescriptive design methods. Global and local finite element analyses were conducted to demonstrate that a structure with partial protection exhibited sufficient fire resistance to comply with code requirements.

The fire performance of composite slabs is well understood, and the mesh is a key component. The next section describes the fire resistance of PFRC slabs, which have distributed polymer fibres in place of a steel mesh.
3.4 Polymer Fibre Reinforced Concrete Composite Slabs

This section describes the construction of PFRC composite slabs, explains their advantages over steel mesh reinforced slabs and discusses their fire resistance testing to date. The areas where further understanding is required of their fire behaviour within continuous steel structures are also explained.

3.4.1 Design and Construction

In the PFRC composite slabs under investigation, the concrete is dosed (5.3kg/m³) with the 40mm rectangular polymer fibres described in the previous chapter. The fibres are added with the regular concrete constituents and the floor is poured, compacted and finished as normal. The reinforcing steel mesh is therefore omitted, because in normal conditions the fibres perform its anti-cracking function. The fibre reinforced concrete maintains the shear bond and composite behaviour required of a floor slab. Load capacity is based on the flexural resistance of the hardened composite section. PFRC slabs are suggested in design guidance and have been available to developers in the UK since 2004, used throughout the retail, education, leisure and residential building sectors (RLSD, 2006; Concrete Society, 2008; MCRMA, 2009).

Figure 3.24: Casting of PFRC slab with no mesh reinforcement (RLSD, 2006)
Table 3.1: Mix design for deck polymer fibre reinforced concrete

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Mass, Volume (kg, l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>360-400kg</td>
</tr>
<tr>
<td>Gravel (2:1 ratio 20mm and 10mm)</td>
<td>1000kg</td>
</tr>
<tr>
<td>Sand</td>
<td>800kg</td>
</tr>
<tr>
<td>Water</td>
<td>125l added</td>
</tr>
<tr>
<td>Fibres</td>
<td>5.3kg</td>
</tr>
<tr>
<td>Superplasticiser</td>
<td>1.4l</td>
</tr>
<tr>
<td>Water/cement ratio</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Small amounts of bar reinforcement are still recommended around the structural perimeter, if edge beams are designed to act compositely. Also, short lengths of mesh are recommended to tie across construction joints.

PFRC mixes for deck concrete are similar to that outlined in Table 3.1. To maintain workability a superplasticiser is added to the mix, which allows the concrete to be pumped to the required level while maintaining the final strength. The exact amounts of water and superplasticiser used in forming slabs are dependent on the moisture condition of the aggregate used, and are best determined experimentally (RLSD, 2009).

### 3.4.2 Benefits of Polymer Fibre Reinforcement

The advantages of fibre reinforcement were mentioned in Section 2.1.5. Eliminating the requirement for reinforcing steel mesh lowers the project cost but also avoids associated site operations. These include delivery, storage, lifting and placement of mesh. The absence of mesh also provides a safer underfoot environment for the concreting operation. The fibre reinforcement is distributed throughout the entire slab, providing more uniform crack control than a single layer of mesh.

Other FRC composite slab technologies are available. Both steel fibre and hybrid fibre reinforced composite slabs are promoted by different manufacturers (Bekaert, 2012; Tata Steel, 2012). Polymer fibres have a higher strength-to-weight ratio than steel, and the dosage of steel fibres required for composite decking is 20-40kg/m³. The increased stiffness of steel fibres also creates more wear on pumping machinery.
3.4.3 Fire Testing of PFRC Slabs to Date

Standardised fire testing has been conducted on PFRC slab specimens, following the procedure outlined in section 3.2.2 above. The test arrangement was a 3.6m main span, with a 0.92m adjacent span over a protected intermediate beam. A diagram of this setup is shown in Figure 3.25. The furnace temperature followed the standard fire curve (Figure 3.13). A PFRC slab with trapezoidal deck demonstrated a fire resistance of 1 hour, while a re-entrant deck slab demonstrated 2 hours. Design tables were produced for each decking profile based on these results. The temperature profiles from the trapezoidal test are shown in Figure 3.26. The slab was 165mm deep, with thermocouples placed at regular 20mm intervals (WarringtonFire, 2006).

![Diagram of test setup for PFRC slab furnace tests](image)

Figure 3.25: Test setup for PFRC slab furnace tests, with protected intermediate beam (dimensions in mm)

![Temperature profiles in trapezoidal slab](image)

Figure 3.26: Temperature profiles in trapezoidal slab (WarringtonFire, 2006)
The vertical dashed line at 160°C (fibre melting point) is intersected by the thermal profiles. After 70 minutes, Figure 3.26 shows that the lower 95mm of the slab will have lost its fibre reinforcement. Above this depth, the fibres will have begun to soften, according to the strength-temperature relationship of the fibres presented from experimental data in Figure 2.18. Despite the loss of strength, the slab did not fail until 65 minutes of heating, when it reached a vertical deflection of 196mm. The exact behaviour of the slab at failure is not clear. The deck had reached a temperature of over 700°C. It is possible the weak, hot deck and cooler concrete at the face were supporting the applied load through flexure. However, it is likely that a component of the slab resistance came from axial compression, due to the restraint to thermal expansion provided by the test arrangement (Figure 3.25).

3.4.4 Fire Resistance and Load Carrying Mechanisms

Despite the standard fire testing of PFRC slabs, there still remains a lack of data on how such slabs would behave in a continuous structural frame. The robust fire behaviour observed during the Cardington tests led to a detailed understanding of the load-carrying mechanisms present in traditional mesh reinforced slabs. These mechanisms could not be captured by slab furnace testing, highlighting a need for further investigation of PFRC slabs in fire. The stability of such slabs at high deflections in fire remains undemonstrated.

It seems reasonable to assume that the occurrence of compressive membrane action at the onset of a fire is unaffected by the absence of reinforcing mesh. The concrete slab will still experience a compressive thrust if lateral restraint is available to prevent thermal expansion. However, as the slab continues to deflect the only element preventing collapse of the slab will be the heated steel deck. Large cracks will appear at the panel perimeter with no tensile mechanism to arrest the deflection or carry load. This outcome is illustrated in Figure 3.27.
The remaining issues relating to the fire resistance of PFRC composite slabs in fire are:

- which load-carrying mechanisms develop at high temperatures which allow PFRC slabs to maintain stability?
- is any contribution to fire resistance made by the presence of fibres?
- is it possible to develop tensile membrane action in a composite PFRC slab, and if not, how does failure occur?

If the PFRC slab does not demonstrate the same robustness in fire as a steel mesh reinforced slab, this will affect the efficiency of possible designs. It would again be necessary to provide full fire protection to each of the secondary beams supporting a slab panel, since the fibrous slab does not possess the same structural redundancy. This is illustrated in Figure 3.28: the deflection of the slab in fire is limited by the protected beams, which maintain their supporting function for a longer period of fire duration. The absent mesh is not required as the slab does not develop high deformation. In this fire design case, the cost savings associated with using polymer fibre reinforcement would be set against the expense of applying full fire protection to the secondary beams to achieve a satisfactory level of structural fire resistance.
Chapter 3 Summary

This chapter has described the fire behaviour of composite slabs. The methods used to assess fire resistance and the structural mechanics arising from high temperatures are explained. The characteristics of PFRC composite slabs are outlined, along with the areas where extra research is required to guarantee fire resistance.
This chapter explains the development of the experimental investigation into the behaviour of polymer-fibre-reinforced concrete slabs in fire. A general outline of the test design and procedure is presented first, which explains how the experiments were designed to meet the project brief. This is followed by a section covering the detailed design of the separate test components, including preparation of the slab specimens, frame construction, and the heating and loading systems. The final section explains the use of strain gauges on the restraining frame to quantify the forces imparted by the loaded and heated slab.
4.1 General Arrangement and Test Sequence

This section outlines the general test arrangement and experimental procedures. It explains how the experiments were designed to meet the requirements of the project brief, test programme and how this addressed the research objectives.

4.1.1 General Overview of the Experimental Scheme

Figure 4.1 shows a schematic profile of the frame and slab arrangement used in the experimental series. The test frame was designed for two-way slab action, with a second portal frame perpendicular to the one shown in the figure. Square, flat slab specimens were tested in a two-way spanning arrangement, with resistance to all degrees of freedom applied at the slab edges. The slabs were flat, with no profiled composite steel decking, following discussion with the Industrial Partners (see section 4.1.2).

The flat PFRC slab specimens were supported above and below by steel square hollow sections along the perimeter. These hollow sections were connected to the steel columns by an edge support arrangement (a channel and two 90° angles) providing resistance to horizontal movement. The columns were connected to the Lab Strong Floor at the base, and bracing channels were connected above the slab spanning between the columns. The slab was positioned so that it was at the mid-height within the frame.

The slabs were heated from above using gas radiant panels, and loaded from beneath using hydraulic jacks. This is opposite to the normal position of the slabs and contrary to standardised slab fire tests in which the slab is positioned over a furnace and load is applied from above. This inverted arrangement was adopted because:

- it removed the risk of broken slab material falling and damaging the delicate heating equipment;
Figure 4.1: Schematic of general test arrangement
it allowed heating via radiation down from the panels to the slab surface, with convection away from the specimen and frame, hence allowing the heating to be defined and to a reasonable extent avoiding unwanted heating of the test frame and support arrangements;

it allowed the load jacks to react against the strong floor;

the structural mechanics and load-carrying mechanisms being investigated did not depend on direction of loading; and

it eased the installation of instrumentation, for example deflection gauges were mounted on a separate scaffold frame beneath the slab specimen.

A typical fire resistance test using this arrangement involved first applying an initial service load. The radiant panels were then turned on, generating a thermal profile through the slab depth. The slab behaviour was recorded by measuring slab deflections, temperature, crack development and the actions on the restraining frame. The restraint actions were due to (a) axial force due to restrained thermal expansion and membrane action, and (b) reaction moment due to restrained thermal curvature and flexural action. These actions on the frame were recorded using strain gauges applied to the test frame columns and edge support arrangements, as explained fully in Section 4.3.

4.1.2 Features of the Experimental Design to Address the Project Aims

The experiments were specifically designed (Figure 4.1) to investigate the effects of temperature and restraint upon concrete slabs. The frame design permitted:

- slab specimens to be subjected to repeatable elevated temperature conditions;
- slabs to be tested with different kinds of reinforcement;
- the applied restraint to be changed by amending the edge support arrangement; and
- measurement of the edge restraint.
These features allowed the experimental investigation into PFRC-reinforced concrete slabs, and comparison with similar slabs with steel or no reinforcement. In particular, the test arrangement allowed the effects of boundary restraint to be separated from the contribution made by the fibres to load carrying at elevated temperatures. As described earlier, the slab specimens tested were flat with no steel sheeting. This avoided the 1-way directional effects of ribbing and the possibility of deck separation due to differential thermal expansion. Doing so greatly simplified the experiment design and subsequent interpretation of test results. This was especially true of the thermal profiles generated, which became ideally a simple distribution through the depth. Omitting these slab features was justified because the focus of the investigation was on the performance of polymer fibre reinforcement and boundary restraint in fire. These aspects of behaviour were taken as independent of realistic, trapezoidal slab profiles and the presence of steel deck.

### 4.1.3 Experimental Series and Test Parameters

This section outlines the six different experiments that were conducted to investigate the behaviour of structurally-restrained polymer fibre-reinforced concrete slabs in fire. They are summarised in Table 4.1, with a more detailed explanation of the individual test procedure for each slab given at the start of Chapter 5.

**Experiment 1: Ambient Load Test of a Fully Restrainted PFRC Slab**

The first experiment was a simple ambient load test on a fully built-in Strux slab specimen. This test established the strength, load-deflection behaviour and crack pattern of the fibre-reinforced slab. This was necessary to establish the load level to apply to the heated tests.

**Experiment 2: First Heated Test of a Fully Restrainted PFRC Slab**

The second experiment was designed to test the fire resistance of a fully restrained PFRC slab. A load representative of that applied in the fire limit state according to Eurocode 1 Part 1-2 (BS EN 1991-1-2:2002) was applied to this test, based upon the ambient capacity determined by Experiment 1. Details of how this load was calculated are given in Section 4.2.4 below.
Experiment 3: Second Heated Test of a Fully Restrained PFRC Slab
Experiment 3 was a repeat of Experiment 2, intended to provide repeatability in the test programme and to allow for any mistakes or problems that arose during testing.

Experiment 4: Heated Test of a Fully Restrained Steel Mesh Reinforced Concrete Slab
Experiment 4 was the same as Experiments 2 and 3, but with light gauge steel mesh reinforcement instead of polymer fibre reinforcement. Details on the mesh size selected are given in Section 4.2.1. This specimen was tested in order to provide a benchmark against which the PFRC slab behaviour could be compared.

Experiment 5: Heated Test of a Fully Restrained Plain Concrete Slab
A second benchmark was provided by Experiment 5, which tested a plain concrete slab with no reinforcement. This would show what beneficial effect the fibres had, if any.

Experiment 6: High Heated Test of a of a PFRC Slab Without Horizontal or Rotational Restraint
The final test investigated the importance of edge restraint for supporting the PFRC concrete slabs in fire. A PFRC slab was heated in the same manner as Experiment 2, but with no axial or moment edge restraint. The applied load was reduced to half the value used in previous tests, to account for the change in ambient collapse mechanism due to removal of the rotational edge restraint.

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Load applied</th>
<th>Reinforcement</th>
<th>Boundary conditions</th>
<th>Fire Exposure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Loaded to failure</td>
<td>Polymer fibres</td>
<td>Full Restraint</td>
<td>None</td>
</tr>
<tr>
<td>2</td>
<td>Fire limit state load</td>
<td>Polymer fibres</td>
<td>Full Restraint</td>
<td>✓</td>
</tr>
<tr>
<td>3</td>
<td>Fire limit state load</td>
<td>Polymer fibres</td>
<td>Full Restraint</td>
<td>✓</td>
</tr>
<tr>
<td>4</td>
<td>Fire limit state load</td>
<td>Steel mesh</td>
<td>Full Restraint</td>
<td>✓</td>
</tr>
<tr>
<td>5</td>
<td>Fire limit state load</td>
<td>No reinforcement</td>
<td>Full Restraint</td>
<td>✓</td>
</tr>
<tr>
<td>6</td>
<td>50% Fire limit state load</td>
<td>Polymer fibres</td>
<td>Free to expand and rotate at edges</td>
<td>✓</td>
</tr>
</tbody>
</table>
4.2 Detailed Design of Experimental Systems

This section details the individual test components and their combination into the test apparatus. It covers the design and construction of the slab specimens, construction of the restraint frame, installation of the radiant panel system and the loading system.

4.2.1 Slab Specimen Design and Fabrication

The layout of the slab specimens is shown in Figure 4.2. This shows the slab dimensions, perimeter bolt holes, thermocouple layout and loading pad arrangement, each of which is described below.

Slab Dimensions and Features

The dimensions of the slab specimens were governed by:

- The need to create a slab that was as slender as practicable, so that membrane load-carrying mechanisms could develop (as discussed in Chapter 3).
- The need to generate a realistic thermal gradient through the depth of the slab using the heat flux obtainable from the gas radiant panels.
- Limits on the size of restraint frame and the amount of concrete that could realistically be handled for this project.
- The minimum slab thickness of 40mm was governed by the 40mm length of the fibre and 10mm aggregate size.

The resulting slab specimens were 40mm deep by 1400mm square. There was a 100mm support strip along each edge, giving an unsupported span of 1200mm. Whilst the span to depth ratio of 30 was stocky for composite slab panels, it was a compromise between realistic model dimensions and the practicability of construction. A ratio of 60 is more realistic, representing a panel 9m square with a slab depth of 150mm.
The slab edges needed to be fixed into the testing frame to provide restraint against both in-plan expansion and rotation. The aim was to achieve restraint that was as close to fully fixed as possible, although there was inevitably some flexibility in the frame (which was measured prior to testing). The edges of the slab butted up against the test frame to provide compressive restraint (Figure 4.1), whilst tensile and moment restraint were provided by clamping the edge of the slab between 100mm square box sections (Figure 4.1). Seven 30mm diameter steel bolts at 150mm centres were used to clamp the box sections onto the slab. These passed through holes cast into the slab, the centres of which were 60mm from the edge. Figure 4.2 shows a plan of the test specimens. The blue dashed lines show the extent of the square hollow section supports above and below the slab. The ends of these sections were cut off at 45° so that restraint was provided into the corners.
Chapter 4 - Experimental Design and Methodology

The four hatched areas of Figure 4.2 show the location of the loading pads on the underside of the slab. Ideally, the fire resistance tests would be conducted under the action of a uniformly distributed load across the surface. However, due to the presence of slab instrumentation the loading had to be applied through separate patches. This slightly altered the bending moment distribution.

The crosses on Figure 4.2 show the 9 thermocouple locations on the surface of each slab. The locations were arranged symmetrically and designated TC1 through to TC9. TC 1, 2, 8 and 9 were near the corners, 200mm from the slab edge. These are called corner thermocouples throughout the results presented in Chapter 5 of this thesis. TC 3, 4, 6 and 7 were all 300mm from the slab centre in the NSEW directions. These are called quarter point thermocouples throughout the results. TC5 was located at the centre.

At TC4 and TC5, thermocouples trees were embedded in the concrete to record the variation in temperature through the depth of the slab. At TC4, four thermocouples were cast into the slab at nominal distances 5mm, 10mm, 20mm and 30mm from the heated face (TC4-1 to TC4-4). At TC5, six thermocouples were positioned at nominal distances of 1mm, 5mm, 10mm, 15mm, 20mm and 30mm from the heated face (TC5-1 to TC5-6). At the remaining locations a single thermocouple was positioned within the slab at 10mm from the heated face to record the variation in temperatures across the slab.

**Fibre Reinforced Concrete Mix Design**

The mix design for the slab specimens was based upon the concrete mix recommended by the Industrial Partners for use in full-scale polymer-fibre-reinforced composite slab construction. However, a 10mm maximum aggregate size was required for the 40mm deep slabs, whereas the standard mix included aggregates up to 20mm in diameter. Consequently the free water content had to be adjusted to compensate for the increase in surface area due to the greater proportion of fine material.
The constituents of each concrete mix are compared in Table 4.2. A polymer fibre-reinforced concrete was used for the specimens used in Experiments 1, 2, 3 and 6. The fibre dosage was held at 5.3kg/m$^3$, which is the recommended level for composite deck applications as described in Section 3.4.1. Experiments 4 and 5 tested a steel-mesh reinforced slab specimen and a monolithic plain concrete slab specimen. To cast these specimens, a control concrete was used with the same constituents except for the fibres. The fibre-reinforced mix included a compatible superplasticiser to compensate for the loss of workability caused by the fibres.

### Table 4.2: Concrete mix constituents

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Recommended Strux mix</th>
<th>Fibre reinforced test concrete</th>
<th>Plain test concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Usage</td>
<td>RLSD Composite slabs</td>
<td>Slabs 1, 2, 3 and 6</td>
<td>Slabs 4 and 5</td>
</tr>
<tr>
<td>Cement (kg)</td>
<td>400</td>
<td>455</td>
<td>455</td>
</tr>
<tr>
<td>Water (l)</td>
<td>180</td>
<td>205</td>
<td>205</td>
</tr>
<tr>
<td>Fine aggregate (kg)</td>
<td>810</td>
<td>946</td>
<td>946</td>
</tr>
<tr>
<td>20mm Coarse aggregate (kg)</td>
<td>660</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>10mm Coarse aggregate (kg)</td>
<td>330</td>
<td>774</td>
<td>774</td>
</tr>
<tr>
<td>Polymer fibre (kg)</td>
<td>5.3</td>
<td>5.3</td>
<td>-</td>
</tr>
<tr>
<td>Superplastisicer (l)</td>
<td>2.16</td>
<td>2.16</td>
<td>-</td>
</tr>
<tr>
<td>Water / Cement ratio</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
</tr>
</tbody>
</table>

The constituents of each concrete mix are compared in Table 4.2. A polymer fibre-reinforced concrete was used for the specimens used in Experiments 1, 2, 3 and 6. The fibre dosage was held at 5.3kg/m$^3$, which is the recommended level for composite deck applications as described in Section 3.4.1. Experiments 4 and 5 tested a steel-mesh reinforced slab specimen and a monolithic plain concrete slab specimen. To cast these specimens, a control concrete was used with the same constituents except for the fibres. The fibre-reinforced mix included a compatible superplasticiser to compensate for the loss of workability caused by the fibres.

### Casting and Specimen Preparation

A collapsible wooden formwork was made to cast the slab specimens, with interior dimensions of 41x1400x1400mm. The base of the form was covered with a 1mm thick sheet of steel to provide a smooth finish to the heated face and reduce moisture ingress to the wooden base. The form incorporated the 28 steel tubes that formed the restraint bolt holes around the perimeter of the slab, which were located using aluminium studs bolted through the base of the form.

The light steel reinforcing mesh used in Slab 4 (see Table 4.1) had 3mm diameter mild steel wires at 50mm spacing, providing a reinforcement ratio of 0.35%. This ratio is typical of composite slabs (0.2-0.4%). The mesh was inserted after the form.
had been half filled with concrete, with bars fitted behind the boltholes at the perimeter to ensure reinforcement continuity into the restraining frame.

The concrete mixes in Table 4.2 were prepared using a 40 litre capacity rotary pan mixer. The volume of each slab was approximately 78 litres, and cube and cylinder specimens were taken from each batch for consistency testing of strength and moisture content (Section 5.7). Therefore, three batches of concrete were required for each slab. The moisture content of the aggregates was carefully monitored so that the batches were similar. The concrete from each batch was placed in the form and consolidated using a vibrating rod. Levelling was achieved by skimming a steel box section over the wet concrete.

After levelling, the thermocouples were inserted by pushing them down to the correct depth, supported off 5 steel bars held 12mm above the concrete surface, with the correct length of wire protruding down into the concrete (Figure 4.3). This method resulted in only a few of the thermocouples across all the slabs being misplaced from their design depths by more than 2mm. The actual depths of each thermocouple were measured after testing and incorporated into the results reported in the next chapter.

The slab specimens were allowed to cure for a period of two years before testing, the

Figure 4.3: Slab after casting showing thermocouple insertion bars
first six months of which were spent in a dehumidified environment. The sample specimens were kept in a curing tank for a month, before being stored with the slabs.

The slabs were painted white before testing to highlight the appearance of cracks. A finished specimen is shown in Figure 4.4. The thermocouple wires are seen trailing from the surface. The squares drawn on the slab show the arrangement of the loading pads, and the lines of crosses along the central axes were to guide the placement of deflection gauges. The exact locations of the deflection gauges along the centrelines were different across the experiments. The locations of each were recorded and are presented along with the deflection results in Chapter 5.

The specimens were inverted before being inserted into the frame, and the back sheet of steel was removed presenting a smooth concrete surface to the heating panels.

![Slab specimen ready for test frame installation](image)

**Figure 4.4: Slab specimen ready for test frame installation**

### 4.2.2 Construction and Design of Restraining Frame

An overview of the frame is shown in Figure 4.5, looking from the South-East. This photograph shows the portal frames (composed of columns and bracing channels), and the slab edge arrangement. The radiant panel and supply valves can be seen in the centre. The detailed arrangement of the frame is described below.
Figure 4.5: Photograph of restraining frame from South-East
Portal Frames

The slab was supported at mid-height by two pairs of portal frames; two oriented North-South, and two oriented East-West (Figure 4.5). The portal frames were constructed using the university’s steel C-channel sections (305×102×46). The north-south and east-west portal frames were independent of each other, with no connection between them except through the slab and the laboratory strong floor. The test frame is shown in plan at a scale of 1:20 in Figure 4.6, and in elevations in Figure 4.7. An isometric drawing on one side of the frame is shown in Figure 4.8. This shows how each side was assembled; the circles represent bolt locations. The frame was comprised of four of these sides arranged so that the square hollow sections fitted around the perimeter of the square slab specimens.

The eight channel columns around the perimeter of the slab were labelled 1 to 8, clockwise from the West side as illustrated in Figure 4.6.

The bracing channels at the top of the frames had to be offset, with the East-West channels above the North-South channels (Figures 4.7a and 4.7b). This enabled the East-West channels to be supported while the West side was removed during slab installation.

The portal frames were intended to provide rigid fixity to the edge of the slabs, but inevitably had some finite stiffness. The stiffness of the test frame was measured before the experimental procedure (see Appendix A.1). The columns in the East-West direction had an average stiffness of 65.5kN/mm and the columns in the North-South direction had an average stiffness of 70.1kN/mm. This was a 6% difference, but this effect was not seen to influence the recorded test data.
Edge Restraint Arrangement

The slab specimens were placed at the mid-height of the frame, defined as the top of the North-South cross brace (or the bottom of the East-West brace), as shown in Figure 4.7. This shows the location and design of the edge restraint arrangement, which connected the slab to the columns and resisted lateral slab movement. These had three different components; the square hollow sections that held the slab, a perimeter channel section, and two angle sections above and below. The edge restraint and bolt provision is shown in detail with dimensions in Figure 4.9a.
Figure 4.7a: Test frame South elevation

Figure 4.7b: Test frame West elevation
Figure 4.8: Isometric drawing of one side of frame showing location of bolt connections
Experiments 1 to 5 required rigid axial rotational edge restraint, so these slabs were clamped using the 30mm diameter edge bolts cut from threaded bars (Figure 4.9a). Experiment six, however, required an edge arrangement with no lateral or rotational support. This was achieved by moving the edge channel outwards to allow the slab to freely expand, and by fitting 10mm diameter steel roller bars between the slab and the SHS, as shown in Figure 4.9b.

### 4.2.3 Radiant Panel Installation

The radiant panel assembly was constructed from six cast-iron propane burners mounted horizontally onto a steel frame. This is shown in-situ within the test rig in Figures 4.1 and 4.10. The full heating area measured 100cm by 110cm. The assembly was suspended from the North-South bracing channels by four threaded...
rods. Propane and air were fed into mixer valves mounted on the rear of each panel. The air was supplied to the heaters by 6 high-speed electric fans. The panels each had an individual propane line, including a flame arrest, a pressure regulator and a flow meter. The flow of propane to each panel was set to ten litres per minute. Lighting the panel involved igniting each individual burner sequentially. Measurement of the heat flux at slab level gave an average of 55.1kW/m$^2$.

Figures 4.11 and 4.12 show the insulation applied to the frame to prevent excessive heating, which could have resulted in thermal expansion. Figure 4.11 is a photograph showing the fibre insulation provided around the columns and bracing channels. Figure 4.12 shows (in red) the boards applied around the heating panels and slab perimeter. Ceramic board insulation was erected around the radiant panels to protect the gas and air supply arrangement. A 30mm thickness of ceramic insulation was placed around the upper steel hollow sections which clamped the slab. Also, ceramic board insulation was propped on top of the steel hollow sections to protect the columns. This arrangement created a channel for the escape of hot gases.
Figure 4.11: Photograph of insulation applied to upper bracing channels

Figure 4.12: Diagram of board insulation applied to frame edges
4.2.4 Load Application System

Loading was applied by two 5 tonne hydraulic jacks from beneath the slab. Each was fitted with a square section spreader bar connected to two square steel pads. These pads had sides of 200mm, were 25mm thick for adequate stiffness, and were bedded onto the slab surface with a layer of mortar to ensure even contact and avoid stress concentrations. A ball junction allowed the pads to follow the rotation of the slab. The loading arrangement is shown in Figures 4.13 and 4.14.

The load was applied and controlled with the university’s Losenhausen hydraulic test machine. A third, 10 tonne hydraulic jack was connected in parallel with the loading jacks and placed inside a self-reacting frame with a load cell so that the applied load could be recorded.

The Fire Limit State (FLS) load applied to the slabs during the heated tests was calculated from the ambient failure load from the first experiment. A FLS load reduction factor ($\eta_{fl}$) of 0.65, as recommended by BS EN 1994-1-2, was used with the ambient failure load and applied to the heated slabs.

Figure 4.13: Loading and measurement system
4.2.5 Measurement of Slab Deflections

The deflection of the slab was measured by 14 linear potentiometer deflection gauges, which can be seen in Figure 4.14. These were supported off a tubular scaffold frame that was independent of the loading frame. The gauges were in contact with the slab at several positions along the centrelines. As explained in Section 4.2.1, the exact positions of the gauges along the centrelines changed between tests as the instrumentation was re-assembled. However, a consistent pattern was followed for each test. Short-travel gauges (<30mm) were placed close to the slab boundary, where deflections were low. Longer travel gauges (<100mm) were placed near the centre where slab deflections were highest. Short steel extender rods (100mm) were attached to the ends of the deflection gauges. These rods extended their reach so they could be set further from the slab, while maintaining the same travel distance. Doing so kept them below the level of the safety mesh, and reduced the risk of high temperatures from the heated slab damaging the equipment. A fall arrest mesh was positioned beneath the slab to prevent parts of the slab and the loading pads from falling on to the loading equipment and the deflection gauges in the event of slab collapse. The mesh was hung from the lower square hollow sections, as shown in Figure 4.14.

Figure 4.14: Underside of test specimen showing instrumentation
4.3 Reaction Force Measurement Using Strain Gauges

A key requirement for these restrained slab tests was to quantify the edge reactions, so as to help understand the effects of the perimeter restraint upon the slab load-carrying mechanisms in fire. This section explains how strain gauges were arranged on the restraining frame to measure the reaction forces imparted by the tested slab specimens.

4.3.1 Introduction

Strain gauges were attached to the frame to measure the reaction forces and moments caused by load and temperature on the slabs. There were two principal components to these reactions, illustrated in Figure 4.15:

- an axial thrust in the plane of the slab, caused by restrained thermal expansion or in-plane load carrying mechanisms; and
- a reaction moment, arising from the flexural stiffness of the frame in restraining the slab from rotating at the perimeter.

The axial thrust and moment were required on each of the four sides of the slab.

Figure 4.15: Direction of frame actions on heated and loaded slab
4.3.2 Arrangement and Design

Three different sets of gauges were installed on the frame to measure the reactions:

- perimeter gauges, placed on the edge restraint system at the slab boundary;
- column gauges, placed on the columns midway between the column base and the height of the slab; and
- bracing gauges, placed on the cross braces above the slab.

The use of multiple sets of strain gauges provided a degree of redundancy in the measurement system, which was necessary because of uncertainties in the way that the strain gauges would perform, particularly at elevated temperature. Each set of strain gauges is described in more detail below.

Note that these gauges were not intended to measure meaningful values of strain. They acted as load cells, and their output was calibrated against a known applied force prior to the experiments, as described in Section 4.3.3 and the Appendix (A.3).

Perimeter Gauges

The perimeter gauges measured the force transmitted through the edge restraint, above and below the slab level. These were placed on the angle sections adjacent to each of the 8 columns, shown in elevation in Figure 4.16a and from above in Figure 4.16b. A pair of gauges (1 active, 1 passive) was installed at each location where force was measured. The reason for using an active and passive pair at each location was for temperature compensation, which is described below. Perimeter gauges were therefore installed at 16 locations in total (one pair above the slab level and one pair below the slab level, at each column).

The edge gauges were designated after the column (1 to 8) they were adjacent to, and whether they were above (A) or below (B) the slab. For example, the gauge placed on the slab perimeter above the slab, at Column 1, was designated Gauge 1A. This
A pure axial reaction force in the plane of the slab resulted in the strain gauges above and below the slab recording the same force. A pure moment, on the other hand, resulted in forces of the same magnitude, but the opposite sign. This principle is illustrated in Figure 4.17a and Figure 4.17b, with the equations used to evaluate the force and reaction moments.

To calculate the net axial force imparted by the slab at a column, the forces recorded by both gauges above and below the slab were added together. For example, if a purely in-plane force $P$ was applied by the slab at column 1, then the forces recorded by 1A and 1B (FA and FB) would both equal $-P/2$. To calculate the reaction moment, the difference between the two recorded forces was multiplied by half the distance $z$ between them (0.16m).
Net axial force = $F_A + F_B$ \hspace{0.5cm} \text{Eq. 6}

Moment = $(z/2)(F_A - F_B)$ \hspace{0.5cm} \text{Eq. 7}

Figure 4.17a: Identical gauge response due to pure slab axial force ($F_A = F_B$)

Figure 4.17b: Opposite gauge response due to slab applied load ($F_A = -F_B$)

The angle gauges were close to the radiant panel, although protected by insulation. However, they would also heat up due to conduction through the slab, and consequently every active gauge was paired with a passive gauge that was wired as a temperature compensating half bridge as shown in Figure 4.18. The passive gauge needed to experience the same temperature as the active gauge, but experience zero strain; it was hence bonded very close to the edge of the angle section, where no load would be transferred through the steel (Figure 4.16a). In this way, any strain induced in the active gauge by the increase in temperature would be equalled and cancelled out by the identical temperature-induced strain in the passive gauge. The success of this temperature compensation arrangement was later confirmed during a heated test in which no load was applied (see Appendix A.2), and also during Experiment 6 (in which the slab was not restrained).
Column Gauges

The second set of strain gauges was applied to the columns to measure the shear force and moment in each of the 8 columns, at a height 61cm above the base. There were two types of gauge applied:

- Column web rosettes; two gauges were bonded onto the web of the column at +/-45° to the column axis, to measure the local shear force.
- Column flange pairs. Two gauges were bonded onto opposing flanges of the column to measure the local moment.

The arrangement of column gauges is shown in Figure 4.19. The column gauges were designated 1C to 8C, after the number of the column to which they were attached. Both the column web rosettes and the flange pairs were wired as differential half bridges, and hence did not require additional temperature compensation.

The shear force and bending moment diagrams for the columns under the slab reaction (axial load and moment) are shown in Figures 4.20a and 4.20b.
Figure 4.19: Arrangement of column gauges

Figure 4.20a: Shear force diagram of column due to applied slab force and moment

Figure 4.20b: Bending moment diagram of column due to applied slab force and moment
To obtain the axial and force and moment reactions from the slab, free body analysis was required to de-couple the actions on the column. The equations developed to conduct this are given below where:

\[ P_S = \text{the in-plane reaction force from the slab} \]
\[ M_S = \text{the moment reaction from the slab} \]
\[ P_G = \text{the in-plane force reaction measured by the gauge} \]
\[ M_G = \text{the moment reaction measured by the gauge} \]
\[ L = \text{length of the column} \]
\[ a = \text{distance from base to gauge height} \]

\[ P_G = \frac{P_S}{2} - \frac{M_S}{L} \quad \text{Eq. 8} \]

\[ M_G = \left( \frac{P_S}{2} - \frac{M_S}{L} \right) a \quad \text{Eq. 9} \]

Obtaining accurate values of the slab perimeter reaction forces \((P_S \text{ and } M_S)\) depended on both the shear force and moment in the columns \((P_G \text{ and } M_G)\) being recorded accurately. In practice however, the resolution of data from the flange pairs was poor. Consequently, the column gauge data obtained was not reliable, and the perimeter gauges were the principal measure of the reaction moment and axial force from the slab. The data from the rosette gauges was of good quality, however, and were used as a limited check upon the in-plane slab force response.

**Bracing gauges**

Gauges were bonded to the bracing channels at the top of the portal frame. These were intended to measure the axial load in the bracing members; however, they were not successful. The portal frame was too stiff, resulting in very small strains that could not meaningfully be measured, and the gauges were susceptible to temperature instability, due to their position above the heaters. The results from these gauges
were not, however, required, due to the redundancy provided by also installing angle gauges and leg gauges on the frame.

4.3.3 Calibration of Gauges

As stated earlier, the gauges were not designed to measure strain, but instead were used to measure the force acting through the frame. This section describes the procedure used to convert the strain output from the gauges into measurements of force, using a gauge calibration factor. The calibration factor was found by applying a force to the frame and analysing the gauge reactions for their applied force/strain relationship. The gauges on the West side of the frame are used as an example (1A, 1B, 1C, 2A, 2B, 2C).

A known calibration force $P$ was applied across the frame, as shown in Figure 4.23. As the applied force was increased, the strain gauges recorded a linear (elastic) increase in strain, proportional to the load.

When these applied force/strain records are plotted, the gradient of each line represents the calibration factor ($P/\varepsilon$) for that gauge. These relationships for the perimeter gauges on the West frame edge (1A, 1B, 2A, 2B) are shown in Figure 4.22, with applied gauge load on the y-axis and recorded strain on the x-axis. These factors allowed for back-calculation of the force through each gauge by multiplying by the recorded strain. Figure 4.23 shows the applied force/strain relationship for gauges 1C and 2C. The full calibration data for all gauges are shown in Appendix A.3. Table 4.3 shows the calibration factors for each of the gauges on the frame.

The relationships between applied force and recorded strain in Figures 4.20 and 4.21 show the linear, elastic response of the frame. However, the plotted lines do not pass through the origin. This was due to local changes in stiffness at low load levels; slippage of bolted connections occurred before the full elastic strength of the frame was utilised.
Figure 4.21: Setup of calibration procedure

Figure 4.22: Evaluating calibration factors for 1A, 1B, 2A, 2B
Figure 4.23: Evaluating calibration factors for 1C, 2C

Table 4.3: Calibration factors for strain gauges

<table>
<thead>
<tr>
<th>Gauge</th>
<th>Cali. Factor (kN)</th>
<th>Gauge</th>
<th>Cali. Factor (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>-0.057</td>
<td>5A</td>
<td>-0.071</td>
</tr>
<tr>
<td>1B</td>
<td>-0.127</td>
<td>5B</td>
<td>-0.110</td>
</tr>
<tr>
<td>1C</td>
<td>0.571</td>
<td>5C</td>
<td>0.521</td>
</tr>
<tr>
<td>2A</td>
<td>-0.062</td>
<td>6A</td>
<td>-0.049</td>
</tr>
<tr>
<td>2B</td>
<td>-0.058</td>
<td>6B</td>
<td>-0.066</td>
</tr>
<tr>
<td>2C</td>
<td>0.418</td>
<td>6C</td>
<td>0.616</td>
</tr>
<tr>
<td>3A</td>
<td>-0.098</td>
<td>7A</td>
<td>-0.054</td>
</tr>
<tr>
<td>3B</td>
<td>-0.082</td>
<td>7B</td>
<td>-0.165</td>
</tr>
<tr>
<td>3C</td>
<td>-0.477</td>
<td>7C</td>
<td>0.513</td>
</tr>
<tr>
<td>4A</td>
<td>0.085</td>
<td>8A</td>
<td>-0.109</td>
</tr>
<tr>
<td>4B</td>
<td>-0.090</td>
<td>8B</td>
<td>-0.050</td>
</tr>
<tr>
<td>4C</td>
<td>0.572</td>
<td>8C</td>
<td>0.450</td>
</tr>
</tbody>
</table>
4.4 **A Priori Numerical Analyses**

### 4.4.1 Introduction

This section describes the numerical modelling of three of the tests in the experiment series, conducted beforehand to verify the experimental design. These three were the heated tests on the PFRC slab, the steel mesh reinforced slab and the plain concrete slab (Experiments 3, 4 and 5). The primary objective of this early simulation work was to verify the frame and slab design, so that the tests would provide useful data on the performance of PFRC slab specimens in fire. A secondary objective was for these analyses to form a basis for later, more rigorous numerical simulations of the test programme once the experiments had been completed. The *a priori* polymer fibre reinforced concrete material model, slab restraint condition and thermal exposure are presented and explained. Slab deflection results from these analyses are then presented, with a discussion of their behaviour and performance. The Finite Element Analysis (FEA) package Abaqus (Dassault Systèmes, 2010) was used to perform the numerical analyses.

### 4.4.2 Model Development

This section describes the experiment model used in the *a priori* analyses. It covers the PFRC material model, the frame model assembly used to obtain representative boundary conditions for the slab, and the thermal input obtained to represent the temperature increase due to fire.

A material model was developed for the polymer fibre reinforced concrete used in the PFRC slab tests. The compressive properties came from the recommendations for concrete in Eurocode 4 (BS EN 1994-1-2:2005). The tensile properties of the concrete were altered from those of plain concrete: a temperature-dependent stress-displacement relationship was specified. This relationship was derived from wedge splitting test results on PFRC samples (Rieder, 2004b), and is covered in Fox (2010).
Figure 4.24: Post-crack stress-displacement material model for PFRC in tension (Range 0mm to 2mm)

Figure 4.24 shows the stress-post crack displacement model developed over the range 0-2mm. For temperatures below 140°C, the fibres provide large increases in ductility and displacement capacity over regular concrete. This ability to sustain deflection without brittle cracking represents the fibres’ crack bridging and ability to transmit load across small crack widths. This ductility decreases as temperatures approach the fibre melting point, reflecting the experimental results (Rieder, 2004b).

At temperatures over 200°C, the concrete properties revert to those used in regular concrete as the influence of the fibres is lost. No attempt was made in these *a priori* analyses to simulate the effect of fibre voids in heated polymer fibre reinforced concrete.

Figure 4.25 shows the model assembly used. The dimensions and sizes of the sections used came from the experiment design (Section 4.2.2). Symmetry in the experimental design was exploited, and a quarter-model was developed incorporating the test frame. The frame consisted of 1-D beam elements with custom profiles attributed to represent the frame components; the square hollow sections and perimeter, column and bracing channel C-sections were included. These profiles are
activated purely for visualisation in Figure 4.25: the sections were analytically one-dimensional.

The lower square hollow sections below the slab were omitted as they caused over-constraint issues in the analysis, and were redundant in providing vertical restraint to the slab section. Nodes between connecting elements were simply tied together. Load was applied to the slab as a UDL of 5kPA over the free area of the underside (interior 600x600mm). This model arrangement matched the experimental design as laid out in Section 4.2 above.

Figure 4.26 shows the thermal gradients to which the slab was subjected during heating. These were derived from a heat transfer analysis on a representative 40mm slab section, exposed to a one-hour BS 476 time-temperature curve (Figure 3.13). To represent cooling, the temperatures decreased linearly back to ambient conditions (20°C) over the course of another hour.
The thermal properties of the concrete used in the heat transfer analysis were as described in Figures 3.5-3.7, as the thermal characteristics of PFRC are practically identical to those of plain concrete. Figure 4.26 includes vertical lines showing the region 140°C to 160°C. In this analysis, the fibres were still effective in lower regions of the PFRC slab after an hour’s heating. The temperature profile was applied across the entire slab surface, with no in-plane variation towards the edges.

### 4.4.3 Modelling Deflection Results

Figures 4.27 and 4.28 show the deflection records of the PFRC slab, the steel mesh reinforced slab and the unreinforced slab. Figure 4.27 shows the full deflection history over an hour of heating, and the hour during which the slab cooled. Figure 4.28 focuses on low-deflection behaviour observed in the first 18 minutes of heating.

Figure 4.27 shows that the unreinforced slab experienced runaway failure at approximately 50 minutes. The mesh and PFRC slabs both tracked close together,
Figure 4.27: Deflection records of steel mesh, polymer fibre and unreinforced concrete slabs, *a priori* simulation

Figure 4.28: Comparative deflection records of steel mesh, polymer fibre and unreinforced concrete slabs (first 18 minutes)

with the PFRC slab reaching 41.8mm and the steel mesh slab reaching 37.2mm after 1 hour. During cooling these deflections roughly remained the same. This Figure demonstrates the differences in response between the three slabs, caused solely by the different reinforcements and material properties used. The mesh reinforced slab
is predicted to be the stiffest, which is reasonable due to the contribution to strength provided by the steel. The difference in response between the unreinforced slab and the PFRC slab show a significant increase in fire resistance with the adoption of the stress-displacement material model for PFRC (Figure 4.24). Thus, it could be concluded from these *a priori* models that the use of polymer fibres can have an effect on the behaviour in fire of concrete slabs.

Figure 4.28 shows that in early stages of heating, the mesh reinforced slab was more resistant to deflection from the start. It also had a slightly lower initial deflection caused the initial 5kPa applied load. After approximately 10 minutes, both the unreinforced and steel mesh reinforced slabs experienced a jump in deflection of approximately 0.8mm. This can be attributed to the due to the low cracking strain of plain concrete. However, the PFRC slab did not experience this jump. This indicated that the slab was acting in a more ductile manner. This result demonstrates the extra ductility provided by the PFRC material model.

In these numerical analyses, the slab deflections during the cooling regime did not revert significantly. However, the realistic behaviour of a cooling, deflected slab experiencing restraint is complex to model accurately. A key factor in this complexity is the irreversibility of the fibre melting process. Any accurate material model of PFRC suitable for both heating and cooling must incorporate different mechanical properties after cooling to ambient temperature. An attempt to capture the loss of ductility after cooling caused by loss of the fibres was not attempted in these *a priori* models. Another key factor in the difficulty of modelling such slabs is the friction generated at opened crack faces. As the slab deflects and cracks open, friction and aggregate interlock between the faces prevent the slab from reverting to its original shape. Chapter 6 below (Section 6.4: Model Refinement Based on Experimental Data) provides data and information on numerical modelling techniques to support further, advanced simulation of this experiment series. However, the initial *a priori* modelling presented above succeeded in demonstrating the viability of the experimental design, and that the fire resistance of the slab specimens is significantly dependent on the means of reinforcement.
Chapter 4 Summary

This chapter has presented the aims and objectives of the experimental investigation. A description of each test in the series was given, with an explanation of how the test sequence achieves the project aims. The individual experimental systems were then described in detail, covering the slab specimens, restraining frame, radiant panel and load application systems. The use of strain gauges on the restraining frame to measure the reactions caused by the heated and loaded slab is explained. The arrangement and design of the various gauges was covered, along with the calibration procedure used to convert the strain data into meaningful reaction force and moment measurements. Finally, an overview was given of the numerical modelling conducted before the experiment series. This established that the proposed frame and specimen dimensions would result in measurable slab deflections, with obvious differences in behaviour between the three types of slab tested.
This chapter presents the results of the experimental investigation into the fire resistance of polymer fibre reinforced concrete slabs. Sections 5.1 to 5.6 cover the experiments on each test specimen in sequence. The data presented include:

- slab temperature data;
- slab deflection histories;
- slab deflected profiles;
- applied load-deflection behaviour;
- axial reaction forces transferred by the slab to the restraining frame; and
- slab edge moments recorded by the restraining frame.

The parameters for each experiment in the series, and a summary of the outcomes, are presented in Table 5.1. Finally, Section 5.7 presents the results from strength and moisture content tests on representative cube specimens, which were formed and cured alongside the slabs.
### Table 5.1: Summary of experiments

<table>
<thead>
<tr>
<th></th>
<th>Experiment 1</th>
<th>Experiment 2</th>
<th>Experiment 3</th>
<th>Experiment 4</th>
<th>Experiment 5</th>
<th>Experiment 6</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Reinforcement type</strong></td>
<td>A: Heating, B: Ambient</td>
<td>A: Heating, B: Ambient</td>
<td>A: Heating, B: Ambient</td>
<td>None</td>
<td>None</td>
<td>5.3 kg/m³ polymer fibre</td>
</tr>
<tr>
<td><strong>Boundary conditions</strong></td>
<td>Full vertical, lateral and rotational support</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Loading and rate</strong></td>
<td>Increased load until slab failure ~1kN/min</td>
<td>Load held at 21.5kN</td>
<td>Increased load until slab failure ~2kN/min</td>
<td>Load held at 25kN</td>
<td>Increased load until slab failure ~2kN/min</td>
<td>Load held at 24kN</td>
</tr>
<tr>
<td><strong>Heat flux</strong></td>
<td>None</td>
<td>None</td>
<td>None</td>
<td>55 kW/m²</td>
<td>None</td>
<td>55 kW/m²</td>
</tr>
<tr>
<td><strong>Experiment Summary</strong></td>
<td>Flexural slab cracking occurred at 36.8kN. Ultimate load of 40.4kN via compressive membrane action.</td>
<td>Radiant panel failed during test: only 20 minutes heating. Deflection reached 12.1 mm, rising further during cooling.</td>
<td>Re-loading test on ambient temperature, cracked slab.</td>
<td>Deflection reached 35mm without significant cracking. Load on heated slab increased to failure at 33.4kN.</td>
<td>Deflection limited to 21.2mm without cracking. Load increased to 34.8kN without failure, test discontinued.</td>
<td>Re-loading test on ambient mesh reinforced slab. Shear failure along North edge at 76.7kN.</td>
</tr>
<tr>
<td><strong>Heating time</strong></td>
<td>-</td>
<td>20 mins</td>
<td>-</td>
<td>2h 30 mins</td>
<td>2h 30 mins</td>
<td>-</td>
</tr>
<tr>
<td><strong>Max. temp.</strong></td>
<td>-</td>
<td>268°C</td>
<td>-</td>
<td>685°C</td>
<td>589°C</td>
<td>-</td>
</tr>
<tr>
<td><strong>Max. load</strong></td>
<td>40.4kN</td>
<td>21.5kN</td>
<td>30.0kN</td>
<td>33.4kN</td>
<td>34.6kN</td>
<td>75.7kN</td>
</tr>
</tbody>
</table>
5.1 Results from Experiment 1: Ambient Load Test of a Fully Restrained PFRC Slab

This section presents the results from the first experiment, which was the ambient (approximately 20ºC) load test on a fully restrained PFRC slab specimen. The aim of this test was to assess the ambient load capacity of the fully restrained slab, and to determine the load level to be applied during the heated tests.

5.1.1 Experiment 1 Failure Mode and Load-Deflection Results

A photograph of the slab from beneath the specimen, showing the failure mode post-failure, is shown in Figure 5.1. A roughly symmetrical pattern of cracking developed. Full depth hogging cracks were formed around the perimeter of the slab, with interior sagging cracks forming a square around the centre. Cracks at 45º extended towards the slab corners from the centre. Edge cracking did not extend fully into the slab corners; instead, the perimeter cracks curved around the corners due to the high degree of rotational fixity. This crack pattern indicated a failure mode in which four trapezoidal slab facets rotated up in the direction of loading, while the central square region rose vertically without significant rotation. This square region was caused by the fixity provided by the loading pads, which were bonded to the slab.

Figure 5.2 shows the relationship between applied load and central deflection of the slab. This relationship illustrates two different modes to the slab behaviour, corresponding to the uncracked (O-A) and cracked (B-D) slab states. There was a stiff initial response until cracking at Point A (36.8kN, 3.0mm). This was followed by a secondary, post-cracking mode of behaviour starting at Point B (36.8kN, 17.6mm). During this mode the slab reached its ultimate strength at Point C (40.4kN, 35.1mm). The deflection and crack development on the top (unloaded) face due to the applied load is shown sequentially in Figures 5.3a, 5.3b and 5.3c. Figure 5.3a shows no significant cracking at a load of 35kN. At 36.8kN (Figure 5.3b), the slab developed sagging cracks and the central deflection jumped from 3.0mm to 17.6mm.
Chapter 5 - Experimental Results

Figure 5.1: Failure mode and crack pattern, Exp. 1

Figure 5.2: Load versus central deflection, Exp. 1
Figure 5.3a, b, c: Slab cracking on unloaded surface as seen from the South-East corner of the frame, Exp. 1

Figure 5.3a
35kN – No visible cracking, 2.6mm deflection

Figure 5.3b
36.8kN – Cracked unloaded face, 17.6mm deflection

Figure 5.3c
40.4kN – Slab failure
Chapter 5 - Experimental Results

The slab post-failure is shown in Figure 5.3c, in which the cracks have widened and the deformed shape of the slab is visible.

Figure 5.4 and 5.5 show how the slab deflected profile developed during loading, along the North-South axis and the West-East axis respectively. A profile is plotted for each of the points A, B, C, and D throughout the test as described in Figure 5.2. The vertical grey dashed lines in each of these figures indicate the measured position of the cracks forming the central square portion of the cracked slab (see Figure 5.1). These lines were measured at 375mm from the North edge, 325mm from the South edge, 350mm from the West edge and 350mm from the East edge. Combining the deflection data with the known crack positions, the deflected profiles of the slab at each point are plotted by the black lines.

As a comparison to Figure 5.4, Figure 5.6 shows a photograph of the slab post-failure. The slab is viewed from the West side, providing a visual profile along the North-South axis. The rest of the image has been darkened for clarity. The deformed shape reflects the peak in deflection profile towards the North side from Figure 5.4.

![Figure 5.4: Deflected profile of slab in the N-S direction, Exp. 1](image-url)
Chapter 5 - Experimental Results

Figure 5.5: Deflected profile of slab in the W-E direction, Exp. 1

Figure 5.6: Photograph of N-S deflected shape from West side
5.1.2 Experiment 1 Reaction Force Results

This section presents the reaction force data recorded by the frame during loading of the slab specimen. The data from the perimeter gauges (above and below the slab, see Figure 4.14) are presented first, followed by the reaction force data recorded by the column gauges.

For this experiment, the data from individual gauges are presented. This is to demonstrate the process by which the in-plane slab reaction forces and reaction moments are calculated. For the subsequent experiments, only the averaged data from the perimeter and columns are presented (which describes the full slab behaviour, but greatly simplifies presentation of the results) except where individual gauge responses are required, for instance to explain the effects of localised cracking.

Figures 5.7a to 5.7h show the recorded relationships between the applied slab load and frame reaction force during Experiment 1 at each Column 1 to 8. These were recorded by the perimeter gauges above and below the slab. For example, Figure 5.7a shows the reaction forces recorded by individual Gauges 1A and 1B located adjacent to Column 1, above (A) and below (B) the slab. A positive frame reaction force indicates compression, from restrained slab expansion. A negative reaction force indicates a tensile frame response, with the slab pulling in. Each of these Figures also shows the sum of the two recorded forces, representing the net axial force at that location, as described in Section 4.3.2.

As the applied load increased from zero, the gauges each recorded a linear increase in the magnitude of reaction force. Each gauge above the slab (1A, 2A, ... 8A) recorded a compressive reaction to the applied load. Each gauge below the slab (1B, 2B, ... 8B) recorded a tensile reaction to the applied load. Due to these opposite reactions, the net reaction forces recorded up to the cracking load of 36.8kN were either zero (as in Figures 5.7a, e, f, and g), or less than 6kN, due to unbalanced reaction forces (Figures 5.7b, c, d, and h).
Chapter 5 - Experimental Results

Figure 5.7a: Applied load versus reaction force for 1A, 1B and net (Column 1)

Figure 5.7b: Applied load versus reaction force for 2A, 2B and net (Column 2)
Chapter 5 - Experimental Results

Figure 5.7c: Applied load versus reaction force for 3A, 3B and net (Column 3)

Figure 5.7d: Applied load versus reaction force for 4A, 4B and net (Column 4)
Figure 5.7e: Applied load versus reaction force for 5A, 5B and net (Column 5)

Figure 5.7f: Applied load versus reaction force for 6A, 6B and net (Column 6)
Figure 5.7g: Applied load versus reaction force for 7A, 7B and net (Column 7)

Figure 5.7h: Applied load versus reaction force for 8A, 8B and net (Column 8)
At the cracking load, every gauge except 4B (due to localised damage) recorded a sudden increase in compressive reaction force of between 6kN and 8kN. The compressive reactions continued to increase with further applied load, until the ultimate load of the slab was reached (40.4kN). Figures 5.7a to 5.7h show that the reaction force response to loading describes two distinct modes to the slab behaviour, corresponding to the pre-cracked and post-cracked two-stage response seen in the deflection data (Figure 5.2).

In Figures 5.7a to 5.7f the results recorded are similar. Figures 5.7g and 5.7h show a slightly different tensile response, related to the moment and cracking behaviour, which is explained below in Section 5.1.3.

Figure 5.8 collates the net axial forces recorded from the perimeter gauges. This shows the sets of gauges where the reactions were balanced (1, 5, 6 and 7), with the sets of gauges where imbalanced reactions caused a small but non-zero net reaction force (2, 3, 4 and 8). At the cracking load, the increase in compressive thrust of 12kN to 16kN was recorded (consistent with double the increase recorded by the individual gauges, of 6kN to 8kN). Overall, the reaction behaviour across the full perimeter was shown to be consistent with a two-stage response.

![Figure 5.8: Collated net reaction force results from perimeter gauges, Exp. 1](image_url)
Reaction Force Results from Column Gauges

Figure 5.9 shows the relationship between applied load and axial reaction force, recorded by each of the column gauges (as shown in Figure 4.17). These results describe the same overall reaction behaviour. The axial thrust from the slab is low until the cracking load is reached, where a compressive reaction of 12 to 16kN was generated. The net axial responses recorded by the perimeter gauges and column gauges are compared in Figure 5.10. These show reasonable agreement, indicating that both sets of gauges were measuring the frame response reliably.

Figure 5.9: Applied slab load versus axial reaction force, recorded by columns

Figure 5.10: Averaged column and perimeter frame reactions
5.1.3 Experiment 1 Reaction Moment Results

Figure 5.11 shows the relationship between the applied load and the slab reaction moment, as recorded by the perimeter gauges at Columns 1 to 6. Figure 5.12 shows the applied load-reaction moment relationship at Columns 7 and 8, on the South side of the frame (Figure 4.6).

Figure 5.11: Applied load versus calculated reaction moment, Columns 1 to 6

Figure 5.12: Applied load versus calculated reaction moment, Columns 7 and 8
Each gauge on the frame recorded a linear increase in reaction moment with applied load, reaching between 1.1kNm and 2.5kNm at the slab cracking load (36.8kN). This range indicates that the magnitude of the reaction moment was not identical along each side of the slab boundary. No pattern to the reaction moment distribution is discernable, so this must have been caused by the local rotational stiffness of the slab.

Along the South edge, the recorded reaction moment began to decline after a load of 33kN (Figure 5.12). This flexural relaxation of the frame indicates that cracking initiated along this edge.

As the load increased post-cracking, the moment reactions along the perimeter generally show an associated increase until failure. This is explained by the rise in the line of thrust acting on the frame, and the slab exerting a combination of both an in-plane compressive reaction force, and a flexural reaction moment.

### 5.1.4 Experiment 1 Discussion

As the load was applied, the central deflection of the slab increased linearly at approximately 20kN/mm (Figure 5.2). The restraining frame recorded a linear increase in reaction moment during loading around the entire perimeter (Figures 5.11 and 5.12). At this stage there was no significant in-plane reaction force recorded between the slab and the frame (Figure 5.8, Figure 5.9). This indicated that the slab was resisting the applied load due to its flexural resistance. This mechanism is illustrated in Figure 5.13. The applied load $P$ caused a compressive reaction above the slab, and a tensile reaction below the slab.

However, this flexural mechanism had a limit, due to the low tensile strength of the slab concrete. At a load of 33kN, a hogging crack began to form along the South side of the loaded face. The recorded reaction moment along this edge began to decline, and at a load of 36.8kN, a crack pattern developed as seen in Figure 5.3b. This caused a jump in central deflection from 3.0mm to 17.6mm, and the slab exerted a
thrust of approximately 26kN on each side of the frame. Despite the development of this crack mechanism, further loading up to 40.4kN was resisted by the slab acting in compression across the frame. This behaviour was apparent from the additional loading causing increased horizontal reaction forces from the frame. This reaction was recorded by both the perimeter gauges (Figure 5.8) and the column gauges (Figure 5.9). The compressive slab mechanism is illustrated in Figure 5.14.

![Figure 5.13: Flexural load carrying mechanism pre-cracking](image)

![Figure 5.14: Compressive load carrying mechanism post-cracking](image)
5.1.5 Experiment 1 Summary

In Experiment 1 a fully restrained, polymer fibre reinforced concrete slab specimen was loaded to failure. The slab exhibited a two-stage (pre-crack and post-crack) reaction to the load; initially the slab resisted the load through flexure, and after cracking, the slab resisted load through compression across the stiff restraining frame. At failure, the slab deflection rose in an uncontrolled manner and the slab broke into several facets (Figure 5.1).
5.2 Results from Experiment 2: First Heated Test of Fully Restrained PFRC Slab

This section presents the results from the first heated test on a polymer fibre reinforced concrete slab. The slab had full restraint applied, the same boundary condition as provided in Experiment 1. A Fire Limit State load was applied to the slab, based on the cracking load recorded in Experiment 1 (see Section 4.2.4). This load should have been 24kN, equal to 0.65*36.8kN, but due to a miscalibrated load cell the actual applied load was 21.5kN. The radiant panel was then lit. Twenty minutes into heating, however, the radiant panel malfunctioned and had to be turned off. Data logging continued while the slab began to cool down. Once the slab had cooled to below 50ºC, the load was removed and the test ended. Following this, an ambient load test was conducted on the cool slab to assess its residual load capacity and structural behaviour. Therefore, this section is split into two sub-sections:

- 5.2.1 Experiment 2A Heating and Cooling Test Results; and
- 5.2.2 Experiment 2B Ambient Re-loading Test Results.

5.2.1 Experiment 2A Heating and Cooling Test Results

This section outlines the results recorded from the first test, where the slab underwent 20 minutes of heating, followed by subsequent cooling. Temperature, deflection and reaction force/moment results are presented, followed by a discussion.

5.2.1.1 Experiment 2A Heating and Cooling Temperature Results

The locations of the thermocouples used to monitor the slab temperature were as reported in Section 4.2.1 and Figure 4.2. Figure 5.15 shows the temperatures measured by the corner and quarter point thermocouples. Ignition of the panel was at \( t = 0 \) minutes, from which the slab temperature started to rise. The panel was turned off at 20 minutes, but the temperatures continued to rise for another 2 to 3 minutes. Although they were all placed at the same depth in the slab (10mm), the quarter point thermocouples recorded higher heating rates and peak temperatures than the corner thermocouples, due to being closer to the centre of the heated area.
Figure 5.16 shows the temperatures recorded through the depth of the slab by the TC4 thermocouple tree. Figure 5.17 shows the temperatures recorded by the TC5 thermocouple tree, at the centre of the slab. Due to an error while casting, the TC5 tree had two thermocouples placed 5mm from the heated face.

Figure 5.15: Temperatures recorded by corner and quarter point thermocouples, Exp. 2A

Figure 5.16: Temperatures recorded at TC4 thermocouple tree, Exp. 2A
Figure 5.17: Temperatures recorded at TC5 thermocouple tree, Exp. 2A

Figure 5.16 and Figure 5.17 show similar heating and cooling behaviour through the depth of the slab, with the rate of temperature increase and peak temperature dependent on the distance from the heated face. The maximum temperature recorded was 288ºC. When the radiant panel was turned off, the heated face cooled rapidly, but the inner slab continued to rise in temperature briefly as heat conducted through the depth. After 38 minutes, the slab temperature had become uniform through the depth and the slab continued to cool naturally. During heating above 100ºC, slight changes in the heating rate were observed. This effect was most obvious from thermocouples at depths furthest from the heated face, where the heating rate was slowest. This was due to the evaporation of water from the concrete, as discussed in Section 3.1.3.

5.2.1.2 Experiment 2A Heating and Cooling Deflection Results

Figure 5.18 shows the deflection of the slab throughout the test. The central deflection is shown, as are the deflections recorded at points 15mm and 300mm from the South edge along the North-South axis.
At Point O, the initial load of 21.5kN was applied. This caused a central deflection of 1.2mm. At $t=0$ mins, the radiant panel was lit and the slab began to deflect. Point A marks the point when the panel was turned off (20 minutes, 12.1mm). Deflections immediately began to reduce.

At Point B (25 minutes, 10.7mm), however, a crack pattern developed in the heated face of the slab. A photograph of the crack pattern is shown in Figure 5.19, with a diagram for clarification in Figure 5.20. After cracking occurred, the slab deflection began to rise again despite ongoing cooling. The deflection continued to rise past Point C (60 minutes, 15.7mm), eventually reaching Point D (178 minutes, 21.5mm) when the slab had cooled to 50ºC. At this point the load was removed, and the deflection dropped back to 14mm.

The deflected profiles of the slab during heating and cooling are shown in Figure 5.21 and 5.22, along the North-South and West-East axes respectively. Similarly to the deflection profile data presented for Experiment 1, a slab profile is plotted for each Point O, A, B, C and D as described in Figure 5.18. The dashed lines in Figure 5.22 complete the profiles, since failure of a deflection gauge caused a gap in the data.
Figure 5.19: Photograph from South-West corner of crack pattern in heated face after 25 minutes (5 minutes of cooling), Exp. 2A

Figure 5.20: Diagram of crack pattern in heated face after 24 minutes (4 minutes of cooling), Exp. 2A
Chapter 5 - Experimental Results

Figure 5.21: Deflected profile of slab in the N-S direction, Exp. 2A

Figure 5.22: Deflected profile of slab in the W-E direction, Exp. 2A
Chapter 5 - Experimental Results

The increase in deflection between Point O and Point A, and the decrease between Point A and Point B were caused by thermal bowing, then contraction of the slab. This caused the slab profiles at these times to have a curved shape. This was demonstrated in both the North-South and West-East directions for Points A and B (Figures 5.21 and 5.22). However, when the crack formed down the centreline in the North-South direction, the deflected profile in the West-East axis became pointed. This is shown in the profiles C and D of Figure 5.22, showing the two segments of the slab meeting at a peak. The corresponding profiles in the North-South direction still show a rough curvature with flattening at the centre, caused by the three segments of the slab (short triangular segments at the ends joined by the North-South crack). This asymmetric shape had an effect on the reaction force behaviour, which is described below.

5.2.1.3 Experiment 2A Heating and Cooling Reaction Force Results

This section presents reaction force data from the heating and cooling phases of Experiment 2. Figure 5.23 shows the reaction forces recorded by gauges 1A and 1B, including the net reaction force. This is included to clarify how the average net reaction forces were calculated. During initial loading, before \( t = 0 \) mins, Gauge 1A recorded a compressive reaction force and 1B recorded a tensile reaction of approximately the same magnitude. The reactions were balanced, and resulted in a near-zero net reaction force. This behaviour was seen during loading in Experiment 1 (Figure 5.7a-h). As heating began, the restrained thermal expansion of the slab generated a large compressive thrust. The net reaction force rose to 18.5kN by the time the panel was turned off at 20 minutes. As the slab cooled, this reaction declined until the load was removed after 178 minutes.

Figure 5.24 shows the average reaction forces from the perimeter gauges during the test, in the North-South (Columns 3, 4, 7 and 8) and West-East (Columns 1, 2, 5 and 6) directions. At Point O, there was no significant thrust from the slab to the frame as a result of load application. However, once the panel was lit at \( t = 0 \) mins a compressive reaction was recorded around the entire perimeter. The thermal expansion of the slab was restrained by the frame, in an approximately symmetrical
manner. The reaction force records in the two directions up to point A (20 minutes) are approximately equal. The reaction forces continued after the onset of cooling, between points A and B (20 minutes and 25 minutes).

Figure 5.23: Reaction force records for 1A and 1B during heating and cooling, showing net reaction force; Exp. 2A (Column 1 gauges)

Figure 5.24: Average net reaction forces recorded by gauges in the North-South and West-East directions during heating and cooling, Exp. 2A
Chapter 5 - Experimental Results

At point B (25 minutes) in Figure 5.24, the crack pattern appeared on the slab surface. After 30 minutes, the West-East reaction forces continued to decline, while the North-South reaction forces started to increase again. The reactions stabilised after 120 minutes, as the rate of cooling decreased. At point D, the applied load was removed; the average West-East reactions dropped to 0kN, but the average North-South reactions dropped from 15.0kN to 6.2kN.

5.2.1.4 Experiment 2A Heating and Cooling Reaction Moment Results

The average reaction moments recorded by the frame are shown in Figure 5.25, for the North-South and West-East directions. During loading and heating (up to 20 minutes), the average reaction moments in both directions followed the same pattern of increase due to applied load ($t < 0\text{mins}$) and during heating ($0\text{mins} > t > 20\text{mins}$). When the radiant panel was turned off at 20 minutes, the moments began to diverge. This was similar to the deflection and reaction force behaviour, except the change in behaviour deflection and reaction force was caused by the crack development at $t = 25\text{min}$. The reaction moments in both directions remained steady from 60 minutes to 178 minutes, before falling when the load was removed.

![Figure 5.25: Average recorded reaction moments in the North-South and West-East directions, Exp. 2A](image-url)
Chapter 5 - Experimental Results

5.2.1.5 Experiment 2A Heating and Cooling Discussion

This section has presented the results for the first heated test of a fully restrained PFRC slab. The slab was exposed to 20 minutes of heating before the radiant panel was turned off due to a fault (see Figures 5.14, 5.15 and 5.16). The slab resisted the initial applied load of 21.5kN in flexure, with little in-plane reaction force between the slab and frame (Figure 5.22). At the onset of heating, the frame restrained the thermal expansion of the slab generating a compressive reaction along the entire perimeter (Figure 5.23). Slab deflection rose due to thermal bowing until the panel failure, when the deflection began to reduce (Figure 5.17). After 5 minutes of cooling, the crack pattern shown in Figures 5.18 and 5.19 developed in the heated surface. This asymmetric pattern caused the slab to behave differently in the north-South and West-East directions. Discussion of this asymmetric behaviour is presented below.

Figure 5.25 shows the slab profile in the West-East direction, post-cracking. At this point the upper face of the slab was cooling rapidly. Thermal contraction of the two slab segments caused the central crack to propagate. This weakened the slab, which slowly rose in deflection $\delta$ due to the constant applied load. The reaction force from the frame in this direction decreased during cooling. This was attributed to the thermal contraction also, reducing the in-plane thrust of the slab.

![Figure 5.26: Deflected shape in West-East direction post-cracking, Exp. 2A](image)

Figure 5.26: Deflected shape in West-East direction post-cracking, Exp. 2A
Slab cooling and edge cracking also reduced the reaction moment in the frame in the West-East direction.

Figure 5.27 shows the slab profile in the North-South direction, post-cracking. The lengths of the segments are taken from Figure 5.20. The reaction forces increased despite the slab cooling (Figure 5.24), and the reaction moment remained high after the panel malfunction (Figure 5.25). This behaviour was accounted for by the short length of the small triangular slab facets at the boundary (Figure 5.20). Due to the central segment having a uniform deflection, the triangular facets had to rotate through a greater angle to maintain compatibility. This resulted in both an increased in-plane reaction force, and an increased reaction moment as the slab deflection rose. Also, their small size of these facets meant that thermal contraction would have little influence on reducing the in-plane reaction force generated by the slab restraint. It was not clear what the fibres contributed to the overall behaviour of the slab; they were still present around the slab perimeter and at the unheated face after heating.

This section has discussed the behaviour and reactions of the slab in the first heated test. As described above, the slab was later subjected to an ambient re-loading test once it had cooled down. The results from this test are presented below.

Figure 5.27: Deflected shape in North-South direction post-cracking, Exp. 2A
5.2.2 Experiment 2B Ambient Re-loading Test Results

This section outlines the results from the subsequent ambient load test, conducted on the slab after the heated test to determine its residual strength. Deflection, reaction force and reaction moment results are presented from the slab, which was cracked from the cooling phase of the heated experiment. This test was conducted in an identical manner to experiment 1, as the slab was simply loaded to failure.

5.2.2.1 Experiment 2B Re-loading Load-Deflection Results

This section presents the failure and load-deflection behaviour from the ambient re-loading test. Figure 5.28 shows a photograph of the loaded face of the slab, post-failure. Under loading, the slab further propagated the crack pattern which developed on the heated face during cooling (Figure 5.19, Figure 5.20). The slab was split into two large trapezoidal fragments in the West-East direction, and into two small triangular facets in the North-South direction. Cracks also occurred at 45º around the corners, due to the two-way rotational fixity in those regions. Figure 5.29 shows the load-deflection behaviour of the cracked slab, at three locations on the slab face.

Figure 5.28: Photograph of loaded face, post failure; Exp. 2 re-loading test
The deflection rose linearly with load up to 20kN, after which the slab began to increase the rate of deflection until failure. The maximum load resisted by the slab was 30.0kN, at a deflection of 25.6mm immediately before failure. At this point the deflection rose in an uncontrolled manner.

Figure 5.30 shows the deflected profiles of the slab in the North-South direction during re-loading, with the measured crack locations from Figure 5.20. Profiles are plotted at increasing load increments of 5kN, starting from 10kN. Figure 5.31 shows the profiles in the West-East direction, and the position of the central crack.
Figure 5.30: Deflected profiles of slab in N-S direction, Exp. 2B

Figure 5.31: Deflected profiles of slab in W-E direction, Exp. 2B
5.2.2.2 Experiment 2 Re-loading Reaction Force Results

Figure 5.32 shows the average measured reaction force results from the perimeter gauges. The reactions above and below the slab (‘A’ and ‘B’) are shown, along with the average net reaction force. The average net reaction rose linearly with applied load until the failure load of 30.0kN was approached. At this point the reaction force increased proportionally, until failure. Because the slab was cracked, it had no resistance in tension to the load applied. Therefore, no tensile reaction force was recorded by the frame.

The reaction forces presented indicate that during loading of the cracked slab, only a compressive reaction was recorded. There was no change in the mode of behaviour, unlike in experiment 1. The slab resisted load by solely acting in compression, utilising the stiffness of the restraining frame to prevent deflection. Once the deflection had reached 25.6mm, the slab could no longer sustain a compressive thrust and failure occurred.

![Graph showing reaction forces vs load](image)

Figure 5.32: Average reaction forces recorded above and below slab including net reaction, Exp. 2B
5.2.2.3 Experiment 2 Re-loading Reaction Moment Results

Figure 5.33 shows the average reaction moment recorded by each gauge on the frame. As explained above, there was no flexural resistance to the applied load due to the slab cracking. However, the moment recorded increased linearly with the applied load until the failure load was approached, reaching 1.5kNm at 29kN. This moment was generated by the high line of compressive thrust in the slab, at the boundary. A greater compressive reaction acted through the upper gauges than the lower gauges.

![Graph showing reaction moments recorded during re-loading, Exp. 2B](image)

**Figure 5.33: Reaction moments recorded during re-loading, Exp. 2B**

5.2.2.4 Experiment 2 Re-loading Discussion

This section has presented the results from the ambient reloading test in experiment 2. A simple load-deflection relationship was recorded by the cracked slab, which had only one mode of behaviour. The applied load was transferred to the frame horizontally, with the slab acting in compression. This is illustrated in Figure 5.34, using the North-South direction between Columns 3 and 8 as an example. The reactions above and below are illustrated by the size of the arrows. The frame
Chapter 5 - Experimental Results

restrained the lateral expansion of the slab, until the deflection reached a point when this mode of action became untenable. The slab during re-loading developed a similar axial compressive mechanism to the slab in Experiment 1, during its post-crack phase of behaviour (Figure 5.14).

5.2.3 Experiment 2 Summary

Experiment 2 was the first experiment using the radiant panel to investigate the fire performance of PFRC slabs. Due to the panel failure after 20 minutes of heating, the Experiment was comprised of two individual tests; the heating and cooling test (Experiment 2A) and the ambient re-loading test (experiment 2B).

During cooling in Experiment 2A, a crack pattern formed in the heated face of the slab. This caused different structural behaviour (reaction force and moment) in the two axis directions, which has been explained and presented. This crack also caused the increasing deflection, despite cooling and thermal contraction.

The subsequent ambient loading test demonstrated that the cracked slab could only resist load though acting as a compressive membrane. The lateral restraint provided by the frame resisted the load-induced expansion of the slab, until the deflection became too great and the slab shape could not support compressive action.
5.3 Results from Experiment 3: Second Heated Test of Fully Restrained PFRC Slab

The procedure for experiment 3 was identical to experiment 2; a heated test of a fully restrained polymer fibre reinforced slab. In this experiment, the Fire Limit State load applied before heating was 25kN. The slab resisted 2 hours and 30 minutes of heating under the applied load, and the slab reached a deflection of 35mm without any significant cracking appearing on the heated face. After 2 hours and 30 minutes of heating, the applied load was increased until failure of the slab. The peak load supported by the heated slab was 33.4kN immediately before failure. After slab failure at 2 hours 35 minutes, the radiant panel was turned off.

An electrical fault with the computer running the frame reaction data collector meant that only a small amount of reaction data was usable. This is covered below in Section 5.3.3.

5.3.1 Experiment 3 Temperature Results

This section presents the temperatures recorded by the slab thermocouples during the heating and cooling in experiment 3. The data collection continued after the panel was turned off at 155 minutes, following loading of the heated slab to failure. Presentation of the temperature data is done in the same manner as for experiment 2A. Figure 5.35 shows the temperatures recorded at the slab corners and quarter points. Heating lasted for 155 minutes (2 hours, 35 minutes) as explained above. Again, the quarter points recorded faster temperature rise due to being closer to the centre of the heated area. Figures 5.36 and 5.37 show the temperatures recorded through the depth of the slab at the TC 4 and TC 5 thermocouple trees. The peak temperature recorded by the slab was 609ºC. The same pattern of thermal behaviour was seen through the slab, as deeper thermocouples recorded lower heating rates and lower peak temperatures.
Figure 5.35: Temperatures recorded by corner and quarter point thermocouples, Exp. 3

Figure 5.36: Temperatures recorded at TC4 thermocouple tree, Exp. 3
5.3.2 Experiment 3 Failure Mode and Deflection Results

Figure 5.38 shows a diagram of the crack pattern that developed in the heated slab at failure. The crack pattern was asymmetric, with a crack in the West-East direction 300mm from the North edge. Figure 5.39 shows a photograph of the slab during removal from the frame. In this photograph the West segment of the slab has been removed, showing the final deflection profile in the North-South direction.

The deflection history of the slab is shown in Figure 5.40. The deflection of three locations on the slab face is presented (central deflection, 210mm from the centre and 15mm from the edge). At point O, the applied load of 25kN caused a central deflection of 1.4mm. At \( t = 0 \) min, the radiant panel was lit. The slab began to deflect, in a non-linear manner between 0 and 30 minutes. The points A (11 minutes, 8.1mm), B (10 minutes, 9.5mm) C (35 minutes, 13.8mm) and D (2 hours 30 minutes, 35.1mm) define approximately where the rates of deflection changed. After point D, the rise in deflection was caused by the load increasing at a steady rate from 25kN to the failure load of 33.4kN.
Chapter 5 - Experimental Results

Figure 5.38: Diagram of crack pattern, Exp. 3

Figure 5.39: Photograph of cracked slab post-failure, Exp. 3
The deflected profiles caused by heating are shown in Figure 5.41 and Figure 5.42 for the N-S and W-E axes respectively. Profiles are plotted for each of the points O, A, B, C and D as described above. The non-uniform rate of slab deflection is shown by the relative deflections between each profile from O to C. The recorded profiles describe a curved slab surface, with the maximum deflection at the centre. This would be expected from thermal bowing across a heated area.

The profile at D represents the surface of the slab after 2 hours and 30 minutes of heating, where the central deflection reached 35.1mm. At this point no significant cracking had occurred on the heated face, and the curved profile down the N-S axis in Figure 5.41 illustrates that observation. The peaked profile D in the W-E axis in Figure 5.42 was due to failure of several deflection gauges, caused by heat radiating from the unheated face of the slab. The crack pattern established in Figure 5.38 developed suddenly at the failure load, so no discernible crack lines could be shown in the deflection profiles presented.
Chapter 5 - Experimental Results

Figure 5.41: Deflected profiles of slab in North-South direction, Exp. 3

Figure 5.42: Deflected profiles of slab in West-East direction, Exp. 3
5.3.3 Experiment 3 Reaction Force Results

As explained above, only a partial record of the reaction force data from the restraining frame was obtained, due to an electrical fault. Figure 5.43 shows the average reaction force recorded by each column, from 4 minutes to 2 hours 33 minutes. The reaction data for initial loading, and for loading to failure at the end of the test, were not recorded.

The data show a rising compressive reaction force during heating, with a shape that closely resembles that of the mid-slab temperature rise (TC 5-4, TC 5-5) from Figure 5.37. This similar pattern illustrated how the thermal expansion of the concrete linked the slab temperature and the magnitude of the reaction force acting on the slab.

![Figure 5.43: Average reaction force measured by column gauges, Exp. 3](image)
5.3.4 Experiment 3 Discussion

Figure 5.44 shows a diagram of the slab behaviour and reactions during heating in Experiment 3. The restrained slab exhibited both thermal bowing and a compressive thermal reaction force during heating. The bowing was caused by the development of a thermal gradient through the depth of the slab. The interaction between the thermal gradient and restraint to thermal expansion was the cause of the non-linear slab deflection in the first 30 minutes of heating. The difference in temperature between the top and bottom of the slab was approximately constant after 30 minutes. Therefore, thermal bowing did not have a significant effect on deflections after this time. The increasing deflection after 30 minutes was instead caused by a combination of restrained slab expansion, and strength and stiffness degradation of the slab under load.

The restrained thermal expansion caused a compressive load-carrying mechanism. The slab adopted a curved profile without cracking, even at a deflection of 35.1mm.
5.3.5 Experiment 3 Summary

Experiment 3 investigated the behaviour of a fully restrained polymer fibre reinforced concrete slab exposed to high temperatures. An initial load of 25kN was applied before the radian panel was lit. After 2 hours 30 minutes of heating the slab reached a peak temperature of 608°C. The heating caused significant thermal bowing and expansion, which permitted the 40mm deep slab to deflect up to 35.1mm at the centre without cracking of the heated face. There was no indication that the slab was close to failure after two hours 30 minutes of heating, so the applied load was increased. The slab eventually failed at an applied load of 33.4kN.

The heated slab produced an in-plane thrust on the restraining frame, which increased in magnitude following the rate of temperature increase.
5.4 Results from Experiment 4: Heated Test of Fully Restrained Steel Mesh Reinforced Concrete Slab

This section presents the results from Experiment 4, in which a steel mesh reinforced slab specimen was subjected to the same loading, heating, and restraint conditions as in the previous experiments. This allowed a comparison to be made between the behaviour in fire of a PFRC slab and a traditional steel reinforced slab.

A Fire Limit State load of 25kN was again applied to the slab, before heating commenced. The slab again survived 2 hours 30 minutes of heating, so the load was gradually increased as in Experiment 3. The load reached 34.6kN without any significant cracking in the slab, so the load was removed and the panels turned off.

After cooling, a subsequent ambient load test to failure on the slab was conducted in the same manner as experiment 2. Therefore, the results from Experiment 4 are presented in two sections:

- 5.4.1 Experiment 4A Heating and Cooling Test Results; and
- 5.4.2 Experiment 4B Ambient Re-loading Test Results.

5.4.1 Experiment 4A Heating and Cooling Test Results

This section presents the results from the heated test on a steel mesh reinforced slab specimen. The full response to heating and cooling is presented; after 2 hours and 30 minutes of heating, the slab was allowed to cool naturally while data continued recording.

5.4.1.1 Experiment 4A Heating and Cooling Temperature Results

Figures 5.45, 5.46 and 5.47 present the temperature data recorded by the slab thermocouples throughout heating and cooling, in the same manner as in the previous sections. Figure 5.47 again shows how the slab thermal gradient through the depth had developed within the first 30 minutes, and stayed steady until the end of heating. The peak temperature recorded was 583°C at 167mins (2 hours 47mins).
Chapter 5 - Experimental Results

Figure 5.45: Temperatures recorded by corner and quarter point thermocouples, Exp. 4A

Figure 5.46: Temperatures recorded at TC4 thermocouple tree, Exp. 4A
5.4.1.2 Experiment 4A Heating and Cooling Deflection Results

During heating and cooling, the slab did not form a distinct crack pattern. The central deflection was limited to 21.2mm, as illustrated in Figure 5.48, which shows the deflection behaviour of the slab during heating and cooling. The initial load of 24kN caused a deflection of 1.0mm, before the radiant panel was lit. As in experiment 3, the first 30 minutes of heated produced a non-linear rate of deflection, before the slab followed a steady decrease in rate between 30 minutes and 2 hours 30 minutes. The letters marking out times in Figure 5.48 are defined as follows:

- O: \( t = 0\) min, deflection = 1.0mm  
  Start of heating
- A: \( t = 10\) mins, deflection = 7.3mm  
  Non-linear deflection
- B: \( t = 25\) mins, deflection = 9.0mm  
  Non-linear deflection
- C: \( t = 2\) h 30mins, deflection = 21.2mm  
  End of heating
- D: \( t = 2\) h 42mins, deflection = 22.5mm  
  Loaded to 34kN
- E: \( t = 2\) h 48mins, deflection \( \approx \) 20mm  
  Load removed
Chapter 5 - Experimental Results

The deflection profiles of the slab throughout heating are shown in Figure 5.49 and Figure 5.50, in the North-South direction and the West-East direction respectively. The slab adopted a smooth, curved profile in both directions due to thermal bowing. The sharp appearance of the profiles in Figure 5.50 (West-East direction) was due to this axis having fewer deflection gauges installed in that direction.

Figure 5.48: Deflection of slab during loading, heating and cooling, Exp. 4A

Figure 5.49: Deflected profiles of slab in North-South direction, Exp. 4A
5.4.1.3 Experiment 4A Heating and Cooling Reaction Force Results

This section presents the reaction force results from the experiment 4 heating and cooling test. Figure 5.51 shows the average forces recorded by the perimeter gauges, and the net reaction forces. The points O-E marked on the time axis are as described above, in Figure 5.48. During initial heating, the gauges above the slab (A) recorded a compressive reaction and the gauges below the slab (B) recorded a tensile reaction, as seen in the previous experiments. When the radiant panel was lit, a rapid rise in compressive reaction was recorded as the frame resisted the thermal expansion of the slab. This net reaction force held steady after approximately 30 minutes, despite the slab temperatures continuing to rise slowly. After 30 minutes the gauges below the slab started to record a reduction in compressive force. The loading and unloading of the slab occurring after point C caused an increase in compressive reaction from the A gauges, and a decrease from the B gauges. This was reversed when the load was removed at point D, which also caused a small reduction of 4kN in the recorded net reaction force. Turning off the radiant panels at point E caused the slab to contract, reducing the reaction force in the frame.
5.4.1.4 Experiment 4A Heating and Cooling Reaction Moment Results

Figure 5.52 illustrates the average reaction moments recorded by the frame. The North edge of the frame recorded a very different pattern of reaction moment behaviour to the other edge (West, East and South). The reaction moment at each edge caused by initial loading (Point O) was in the range of 0.9 to 1.1kNm. However, when heating began, the reaction moment recorded by the West, East and South edges began to rise when the moment recorded by the North edge reduced. After 30 minutes the North edge moment began to rise, reaching 1.7kNm at the end of heating. After 150 minutes the recorded moments react similarly to the increased load, and subsequent unloading.
5.4.1.5 Experiment 4 Heating and Cooling Discussion

This section has presented the results from the heated test of a fully restrained steel mesh reinforced slab specimen. The initial load of 25kN was resisted in flexure, similar to the previous tests. When heating began, the slab started to deflect with a bowl-shaped profile, in the direction of loading. This was due to a combination of thermal bowing due to the induced temperature profile, and thermal expansion forcing the deflection to rise to accommodate the change in length. A net compressive reaction force was recorded by the frame, in response to the slabs restrained thermal expansion.

After 2 hours and 30 minutes of heating, the slab had no visible cracks on the heated face. The central deflection at this time was limited to 21.2mm, and the peak temperature had reached 579ºC. The applied load was increased gradually to 34.6kN, and the deflection increased slightly to 22.4mm. This increase in load caused an increase in the reaction moment along the entire perimeter (see Figure 5.52, point C-D). Figure 5.53 shows an illustration of the frame reactions acting on the slab after heating, with the slab adopting a shallow deflected profile.
As heating progressed, the moment reaction recorded on the North side of the frame was different to that recorded along the other three sides. This non-symmetric behaviour indicates localised damage, and the North side moment response is covered in the following analysis of the residual ambient strength test, experiment 4B.

Figure 5.53: Illustration of frame reactions after 2h 30mins of heating, Exp. 4A
5.4.2 Experiment 4B Ambient Re-loading Test Results

This section presents the results from the ambient load test carried out on the fully restrained steel mesh reinforced slab, after the heated test described above. The slab had no visible damage, other than perimeter hairline cracks on the loaded face and discolouration from the heat.

5.4.2.1 Experiment 4 Re-loading Load-Deflection Results

The mesh reinforced slab failed in shear along the North side. A photograph from underneath the slab is shown in Figure 5.54. The ruptured reinforcing bars protruding from the crack surfaces can be seen. The mesh prevented widening of cracks in the central region of the slab, and an overall crack mechanism did not occur. Figure 5.55 shows the load-deflection behaviour of the slab. Until approximately 58kN, the response was linear. At 58kN, the shear crack began to develop. This caused a change in the stiffness and the slab began to deflect further with the applied load.

![Photo of slab shear failure along North side, Exp. 4B](image)
As the peak load was approached, the individual reinforcing bars began to rupture. This started with a bar at the centre, corresponding to the point of highest reinforcement strain. Rupture of bars away from the centre occurred sequentially as the applied load passed 70kN up to a maximum supported load of 76.7kN.

Figures 5.56 and 5.57 show the deflected profiles of the slab along the North-South and West-east axes respectively. The approximately equal distances between the profiles at 20kN, 40kN and 60kN indicate the linear response to load before cracking of the North edge occurred. The discontinuity in the slab caused by the crack was not captured specifically in these profiles due to the low spatial resolution of the gauge positions.
Figure 5.56: Deflected profiles of slab in North-South direction, Exp. 4B

Figure 5.57: Deflected profiles of slab in West-East direction, Exp. 4B
5.4.2.2  Experiment 4B Re-loading Reaction Force Results

Figure 5.58 shows the relationship between applied load and average recorded reaction force. The 'A' gauges above the slab recorded a linear increase in compressive reaction with applied load, and the 'B' gauges recorded a small tensile reaction. The tensile reaction at high applied loads was due to the capacity of the steel mesh, as plain concrete would have cracked. The net reaction was an increasing compressive force, principally due to the low magnitude of the tensile reaction. This indicated that the slab was acting in a combined flexural-axial mechanism.

After 58kN the tensile reaction from the 'B' gauges began to reduce. This was coincident with the reduction in stiffness associated with formation of the North perimeter shear crack. This was caused by relaxation of the frame on the North side, and the effect this had on the other three sides.

![Figure 5.58: Applied load versus reaction force, Exp. 4B](image)
5.4.2.3 Experiment 4B Re-loading Reaction Moment Results

Figures 5.59 and 5.60 contrast the reaction moment behaviour along the East, South and West sides of the frame with that of the North side.

Figure 5.59: Applied load versus reaction moment, gauges 1, 2, 5-8 (E, W, S side)

Figure 5.60: Applied load versus reaction moment, gauges 3 and 4 (North side)
Figure 5.59 presents the reaction moment response at Columns 1, 2 and 5 to 8 corresponding to the W, E and S sides. An approximately linear increase in moment was recorded in response to the applied load for each of the 6 gauges. However, in Figure 5.60 the reaction moments recorded on the North side began to decline at approximately 60kN to 65kN. This explicitly shows the relaxation of the North side as the slab starts to fail along that edge.

5.4.2.4 Experiment 4B Re-loading Discussion

Figure 5.61 shows the deflected shape and frame reaction at failure of the mesh-reinforced slab in experiment 4B (ambient reloading). The central deflection was limited due to the tensile strength of the mesh. The slab resisted the load using a combination of flexural capacity and compressive membrane action; a high net in-plane thrust was recorded by the frame, but there was also a tensile component indicating an effective reaction moment response.

Failure of the specimen was caused by reinforcement rupture, which started at the centre of the North side (Figure 5.54). From there the increasing load caused the rupture of reinforcement to spread towards the North side corners until the supported load began to decline.

Figure 5.61: Frame reaction and slab shape at failure, Exp. 4B
5.4.3 Experiment 4 Summary

Experiment 4 demonstrated the effect of including a reinforcing mesh in the slab. During the heated test, the deflection was limited to 21.2mm after 2 hours 30 minutes. The slab experienced compression from restrained thermal expansion, and was stiff when further load was applied to the heated slab. No surface cracking was observed under the combined heating and loading.

A subsequent ambient load test was conducted. The mesh changed the failure mode of the slab from a yield line crack pattern to reinforcement rupture followed by shearing along the North edge.
Chapter 5 - Experimental Results

5.5 Results from Experiment 5: Heated Test of Fully Restrained Plain Concrete Slab

This section presents the data from the heated test conducted on the plain concrete slab, without fibres or mesh reinforcement. The same boundary conditions, loading and heat exposure were applied to the slab as in the previous heated tests. This experiment was designed to demonstrate what contribution, if any, was provided by the polymer fibres to the slab fire resistance.

In this test, the slab again supported the applied load of 24kN for over 2 hours 30 minutes of heating, without any significant cracking appearing on the heated face. Further load was applied to the slab, which eventually failed at 36.5kN.

5.5.1 Experiment 5 Temperature Results

As in previous sections, the temperatures recorded by the slab thermocouples are shown in Figures 5.62, 5.63 and 5.64 below. The thermal response was similar to that of each previous experiment, with the slab reaching a peak temperature of 605°C.

![Temperature graph](image)

Figure 5.62: Temperatures recorded by corner and quarter point thermocouples, Exp. 5
Chapter 5 - Experimental Results

Figure 5.63: Temperatures recorded at TC4 thermocouple tree, Exp. 5

Figure 5.64: Temperatures recorded at TC5 thermocouple tree, Exp. 5
5.5.2 Experiment 5 Failure Mode and Deflection Results

A diagram of the crack pattern which developed in the unreinforced slab at failure is shown in Figure 5.65. The crack pattern was similar to the one which occurred in experiment 1 (Figure 5.1). Figure 5.66 shows the failure shape of the slab, as seen from the South-West corner. The rotation of the South and West trapezoidal facets and the rise of the square central slab portion are shown.

Figure 5.65: Crack pattern in slab at failure, Exp. 5

Figure 5.66: Photograph of slab after failure seen from South-West corner, Exp. 5
Figure 5.67 shows the deflection of the slab during heating. At Point O at \((t = 0\text{mins})\) the central deflection caused by the 24kN applied load was 1.0mm. During heating, the slab deflected in the same non-linear manner as in each previous heated experiment, passing through Points A (15mins, 9.2mm), B (60mins, 20.3mm), C (120mins, 28.2mm) and D (150mins, 30.0mm). The deflection then rose to 33.0mm as the applied load was increased from 24kN to the failure load of 36.5kN.

Figures 5.68 and 5.69 show the deflection profiles along the N-S and W-E axes. The non-uniform deflection rate is illustrated by the uneven time steps represented.

![Figure 5.67: Deflection of slab during heating, Exp. 5](image-url)
Figure 5.68: Deflected profiles of slab in North-South direction, Exp. 5

Figure 5.69: Deflected profiles of slab in West-East direction, Exp. 5
5.5.3 Experiment 5 Reaction Force Results

Figure 5.70 shows the average reaction force recorded by the frame perimeter gauges before and during heating. The initial load caused a compressive and tensile moment couple to form, although the magnitudes of the forces were unbalanced resulting in a significant net compressive force. Heating caused thermal expansion of the slab, which generated an increase in compressive reaction.

![Graph showing average reaction forces recorded by frame during heating, Exp. 5](image)

5.5.4 Experiment 5 Reaction Moment Results

Figure 5.71 shows the average reaction moment recorded by the perimeter gauges. Before heating, the response was similar along each edge. After heating began, the East, South and West sides of the frame recorded a rise in reaction moment from 1kNm to approximately 2kNm. The North side recorded a different pattern of behaviour, decreasing in reaction moment until 30 minutes had elapsed. After then,
the moment began to increase until reaching the same value as the other sides after 2 hours 30 minutes.

Figure 5.71: Average reaction moments recorded by frame during heating, Exp. 5

5.5.5 Experiment 5 Discussion

Prior to heating, the slab reached a deflection of 1.0mm under the FLS load of 24kN. The slab behaved in a flexural manner in response to the initial load. As heating commenced, the slab deflection began to rise in a non-linear manner. The frame recorded an average compressive response due to restrained thermal expansion. The failure mode and frame reaction is illustrated in Figure 5.72. The rate of deflection declined between 40mins to 2 hours 30mins, at which point no visible cracking was observed on the heated face. Therefore, the load was increased on the heated slab. The heated slab resisted an increased of load of 36.5kN, at which point sudden failure occurred and a symmetrical crack pattern developed on the unloaded face (see Figure 5.65).
This section has presented the results of a heated test on a fully restrained concrete slab, containing no reinforcement. The slab behaved similarly to the polymer fibre reinforced slab specimen from Experiment 3, but had a central deflection 5mm lower after the 2 hour 30 minute heating period.
5.6 Results from Experiment 6: Heated Test of PFRC Slab Without Horizontal or Rotational Restraint

Experiment 6 investigated the effect of removing lateral and rotational restraint from a PFRC heated slab. The reaction frame was amended by placing 10mm diameter steel rollers between the slab and the frame perimeter, as described in Section 4.2.2. The radiant heat flux was identical to the previous experiments. Since the edge fixity condition had changed so that the slab was approximately simply supported, the applied load during heating was reduced by 50% to 12kN. This ensured that the applied load remained the same proportion of the Fire Limit State (65%) as in previous experiments. After ignition of the panels, collapse of the slab occurred after around 10 minutes.

5.6.1 Experiment 6 Temperature Results

The temperatures recorded by the slab thermocouples are presented in Figures 5.73, 5.74 and 5.75. The test only ran for approximately 10 minutes, and the slab reached a peak temperature of only 207ºC.

![Figure 5.73: Temperatures recorded by corner and quarter point thermocouples, Exp. 6](image-url)
Figure 5.74: Temperatures recorded at TC4 thermocouple tree, Exp. 6

Figure 5.75: Temperatures recorded at TC5 thermocouple tree, Exp. 6
5.6.2 Experiment 6 Deflection Results

Figure 5.76 shows the crack pattern that developed in the slab at 9 minutes 38 seconds. As the slab had no rotational restraint along the perimeter, no hogging cracks formed around the perimeter. Figure 5.77 shows a photograph of the loaded

Figure 5.76: Crack pattern in unrestrained slab

Figure 5.77: Photograph of unheated face at failure, Exp. 6
face of the slab at failure, highlighting a junction of 3 central cracks in the white circle.

Figure 5.78 shows the deflection history of the unrestrained slab. The behaviour was simple with a low rate of deflection under initial heating, until the development of the crack mechanism shown in Figure 5.76. Rapid uncontrolled deflection accompanied failure of the slab and ended the test.

The deflected profiles along the North-South and West-East axes are shown in Figure 5.79 and 5.80.

![Figure 5.78: Deflection of slab during heating to failure, Exp. 6](image-url)
Figure 5.79: Deflected profiles of slab in North-South direction, Exp. 6

Figure 5.80: Deflected profiles of slab in West-East direction, Exp. 6
5.6.3 Experiment 6 Reaction Force Results

The reaction force recorded by both the perimeter gauges and the columns are shown in Figure 5.81. No axial force was recorded due to the slab expansion being allowed for by the rollers.

![Figure 5.81: Reaction forces measured by perimeter and column gauges, Exp. 6](image)

5.6.4 Experiment 6 Reaction Moment Results

A small reaction moment of 0.45kNm was recorded by the frame during initial loading and throughout heating. This was caused by the vertical restraint applied by the frame slightly inhibiting the free rotation of the slab. This moment was reduced to 0kNm at failure as the frame relaxed.
5.6.5 Experiment 6 Discussion

The load placed on the slab before heating was reduced to 12kN to compensate for the loss of rotational restraint. When heating commenced, the slab began to deflect slowly until 9 minutes 38 seconds had elapsed. At that time the slab failed as the deflection ran away in an uncontrolled manner. The maximum temperature in the slab at this time was 207°C, and some fibres will have started to melt in the centre.

No reaction force was recorded by the frame, as no in-plane compression was generated through the slab to provide stability. A small reaction moment was recorded due to the frame reaction to load application, which reduced to zero after the slab failure and removal of the load. The loss of rotational restraint meant that the crack profile developed did not have hogging perimeter cracks around the sides of the slab.
5.6.6 Experiment 6 Summary

In Experiment 6 the slab resisted only 10 minutes of heating before failure. The lack of restraint to thermal expansion meant that the slab experienced no in-plane compression; the fire resistance of the slab was controlled simply by its flexural resistance. This meant that due to the low tensile strength of concrete, cracks had no resistance to opening other than that provided by the fibres. It is not immediately clear what contribution the fibres made to the fire resistance period of the unrestrained slab.
Chapter 5 - Experimental Results

5.7 Strength and Moisture Content Results from Slab Concrete Specimens

This section provides strength and moisture content data from the representative sample concrete specimens that were cast alongside each slab. The sample specimens were placed in a curing tank for a month, at a pH of 9.7. After this, they were stored in a dehumidified tent along with the main slab specimens for over a year. From this point on, all the specimens were stored in normal conditions until testing, on average 24 months after casting.

5.7.1 Compressive and Tensile Strength of Slab Concrete from Sample Specimens

Each slab's representative strength specimens were tested on the same day as the main experiment. The cube and split cylinder testing was conducted according to BS EN 12390-3 and BS EN 12390-6. Figure 5.84 shows the compressive strengths of the cube specimens and Figure 5.85 shows the split tensile strengths of the cylinder specimens. The average strengths of each slab's specimens are also given.

![Figure 5.84: Compressive strength of representative slab cube specimens](image-url)
The average cube compressive strength of the slabs tested was 52.0MPa, with a range of 45.1MPa (Slab 4) to 57.2MPa (Slab 5). The cube strengths are in line with the expected strength range of c35 concrete after two years of curing, which is in the region of 50MPa. Failure of the PFRC cube specimens was ductile and controlled; the fibres were able to provide internal frictional resistance to the cube deformation. This indicated that the cement matrix was not sufficiently strong that breakage was the dominant fibre failure mechanism. The plain concrete cubes tested (slabs 4 and 5) demonstrated common brittle failure. However, there was a difference in strength of 12MPa between Slab 4 and Slab 5 concrete. This was despite the mixes having the same design strength. The moisture content of the aggregates in the concrete was closely controlled, so this discrepancy is attributed to the statistically small number of cube samples tested. The compressive strengths of the PFRC mixes are all within these bounds, so the mixing and batching of the slabs was considered to be sufficiently uniform to conduct a fair series of experiments.

The average tensile strength of the slab concrete was 3.9MPa, with a range of 3.4MPa (slab 2) to 4.4MPa (slab 5). The calculated tensile strength of the concrete is consistent with the standard 8-12% $\ell_{cu}$ value, across all of the samples tested.
5.7.2 Moisture Content of Slab Concrete from Sample Cylinder Specimens

Three 100mm by 200mm sample cylinders from each slab were used to calculate the moisture content of the cured concrete. This was done to ensure that sufficient time for curing had been allowed, as the high moisture content of early-age concrete can detrimentally affect the temperature propagation. Too high a moisture level exaggerates the endothermic effect of water evaporation at 100ºC and causes thermal gradients which may not be representative of service concrete (Section 3.1.3).

The cylinders were weighed before and after a week in an oven exposed to temperature of 110ºC. The cylinders were wrapped in foil so that any material coming loose would be retained, as the calculated percentage of moisture was sensitive to any changes in mass. The oven used for drying is shown in Figure 5.86. Figure 5.87 shows the pre-drying mass, post-drying mass and moisture content of each of the tested samples. The pre- and post-drying masses are shown as bars and measured off the left vertical axis. The calculated moisture content of each sample is read off the right vertical axis. The data for each slab are summarised in Table 5.2. The average moisture content of the slabs was 2.97%, with the range from 2.84% to 3.23%. This is within the recommended value of 1-5% (Section 3.2.1).

Figure 5.86: Foil-wrapped cylinder specimens for moisture measurement
Figure 5.87: Pre-drying mass, post-drying mass and moisture content of slab cylinder specimens

Table 5.2: Moisture content analysis and results

<table>
<thead>
<tr>
<th>Slab</th>
<th>Pre drying mass (g)</th>
<th>Post-drying mass (g)</th>
<th>Change (g)</th>
<th>Moisture content (%)</th>
<th>Slab average moisture content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3321.9</td>
<td>3217.0</td>
<td>104.9</td>
<td>3.16</td>
<td>3.23</td>
</tr>
<tr>
<td></td>
<td>3498.9</td>
<td>3376.7</td>
<td>122.2</td>
<td>3.49</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3510.2</td>
<td>3403.4</td>
<td>106.8</td>
<td>3.04</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>3579.5</td>
<td>3469.9</td>
<td>109.6</td>
<td>3.06</td>
<td>3.03</td>
</tr>
<tr>
<td></td>
<td>3571.8</td>
<td>3466.3</td>
<td>105.5</td>
<td>2.95</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3537.5</td>
<td>3428.7</td>
<td>108.8</td>
<td>3.08</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>3598.5</td>
<td>3497.3</td>
<td>101.2</td>
<td>2.81</td>
<td>2.84</td>
</tr>
<tr>
<td></td>
<td>3534.7</td>
<td>3428.5</td>
<td>106.2</td>
<td>3.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3583.0</td>
<td>3486.5</td>
<td>96.5</td>
<td>2.69</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>3580.3</td>
<td>3474.6</td>
<td>105.7</td>
<td>2.95</td>
<td>2.94</td>
</tr>
<tr>
<td></td>
<td>3607.8</td>
<td>3500.5</td>
<td>107.3</td>
<td>2.97</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3615.7</td>
<td>3511.3</td>
<td>104.4</td>
<td>2.89</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>3585.9</td>
<td>3480.5</td>
<td>105.4</td>
<td>2.94</td>
<td>2.88</td>
</tr>
<tr>
<td></td>
<td>3608.3</td>
<td>3506.2</td>
<td>102.1</td>
<td>2.83</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3630.8</td>
<td>3526.6</td>
<td>104.2</td>
<td>2.87</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>3596.4</td>
<td>3492.5</td>
<td>103.9</td>
<td>2.89</td>
<td>2.90</td>
</tr>
<tr>
<td></td>
<td>3660.0</td>
<td>3554.1</td>
<td>105.9</td>
<td>2.89</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3666.4</td>
<td>3559.6</td>
<td>106.8</td>
<td>2.91</td>
<td></td>
</tr>
</tbody>
</table>
Chapter 5 Summary

This chapter has presented the results of the experimental investigation into the fire performance of restrained, polymer fibre reinforced concrete slabs. The performance and outcomes of each of the six experiments in the series has been explained, and the data presented. Experiments two and four were reported in two sections each, due to these tests ending prematurely and the slabs undergoing subsequent ambient load testing.

The results from testing the representative concrete samples from each slab have also been presented. These show that the strengths and moisture contents of the concrete used in the slab specimens were within applicable bounds, and that the tests were not affected by defects in the slab construction.
This chapter presents a comparative analysis of the results outlined for each experiment in the previous chapter. This discusses the differences in behaviour across the slabs in the test series, and consequently develops insight into the role that polymer fibres play in concrete during fire. The temperature evolution, deflection behaviour and structural reactions are compared for each test to analyse and explain the performance of each slab.
6.1 Thermal Behaviour

This section analyses the temperature response from each of the heated slabs. The similar temperature evolutions recorded in each experiment are compared and discussed. The panel ignition sequence, fuel flow and heat flux were kept constant across the experiments to provide a consistent thermal exposure condition.

6.1.1 Temperature Evolution through Slabs

Temperature data from each of the thermocouples in the central trees (TC5) are compared in Figure 6.1, from each heated experiment. Figure 6.1 illustrates how the heating periods of Experiment 2 and Experiment 6 were shortened: heating in Experiment 2 lasted for 20 minutes before heating panel failure, and heating in Experiment 6 lasted only 10 minutes before slab collapse. During cooling, the slab temperature profile through the depth became uniform, as shown by the descending lines grouping together.

Figure 6.1 also shows the similarity of the temperatures recorded in the slabs during experiments 3, 4A and 5. The temperatures recorded at identical points within the slab depth tracked within ±20ºC throughout each test. In Experiment 4A, the peak temperature reached at the heated face was 583ºC after 2 hours 47 minutes (167mins). In experiments 3 and 5, the peak temperatures achieved were higher, at 609ºC and 605ºC respectively after approximately 2 hours 36 minutes (156mins). The reason for the lower peak temperature of experiment 4 was because of the lower slab deflection at the end of the test (see next Section, 6.2), which meant the slab was further away from the heating panels. The reason for Experiment 4A having a longer heating duration was because the slab resisted the increased loading for 11 minutes longer. This was because in Experiment 4A, the heated slab survived 18 minutes of increased loading (25kN to 34.6kN). In Experiments 3 and 5, the heated slabs both failed after only 6 minutes of increased loading. The loading regimes from each test are compared in detail in Section 6.2.2.
Figure 6.1: Comparison of temperature records from centre thermocouple tree (TC5) in each heated experiment
Figures 6.2 and 6.3 show the temperatures recorded in each of the heated tests at the quarter points and corner locations respectively. These figures both show that the slab temperature rise was consistent at each location, with the quarter points (see Figure 4.2) reaching peak temperatures in the range 500°C to 510°C and the corners reaching peak temperatures in the range 340°C to 360°C.

![Figure 6.2: Slab temperatures recorded by quarter point thermocouples](image1)

![Figure 6.3: Slab temperatures recorded by corner thermocouples](image2)
6.1.2 Thermal Profiles during Heated Tests

Figure 6.4 shows the thermal profiles within each of the heated slabs at defined time intervals (ignition, 10mins, 20mins, 40mins, 60mins and 150mins). The profiles for Experiment 2 end at 20 minutes due to the heating panel failure. The profile for Experiment 6 was taken at 10 minutes, and records higher temperatures than at 10 minutes in the other experiments. Otherwise, the profiles recorded in Experiments 3, 4 and 5 show closely similar behaviour up to 150 minutes. These indicate that despite the different reinforcement types used, each slab heated up in a similar manner.

These thermal profiles also describe the pattern of thermal bowing in the slab, which contributed to the deflection increases observed during each heated test. The profiles at 20 minutes show the most curvature, indicating that this was the point when thermal bowing was contributing most to the slab deflection. The similar thermal gradients from 40 minutes indicate that thermal bowing due to variation of the thermal gradient would have ceased.

Figures 6.1 to 6.4 show that the addition of polymer fibres did not significantly affect the heat transfer properties or temperature development of the tested concrete slabs, over the temperature ranges experienced.

![Figure 6.4: Slab temperature profiles during heated experiments](image-url)
A key outcome shown by Figure 6.4 is that the fibres in the centre of the slab will have melted throughout the entire 40mm depth between 20 and 40 minutes. After this point the fibres were unable to contribute to the stability of the slab. The line at 160°C shows the fibre melting temperature. Prior to this, the fibres will have gradually lost strength throughout the slab according to the strength versus temperature relationship shown in Figure 2.18 (page 41).

This indicates that the PFRC slab had no effective fibre reinforcement for approximately the latter two hours of heating. As is discussed below, the fire resistance of the PFRC slab was solely due to an in-plane compressive force bracing the slab across the frame. This compressive force was generated by the slab’s thermal expansion being prevented by the stiff surrounding structure.
6.2 Slab Deflection Behaviour

This section discusses the deflection behaviour recorded by each slab under the effects of loading and heating. The loading histories of the heated slabs are compared first. The deflection behaviour recorded during, and at the end, of the heated tests is then explained, including a discussion of the load capacity of each heated specimen. Finally, a comparison is made between the ambient strength test (Exp. 1) and the residual strength tests on a PFRC and steel reinforced slab (Exp. 2B and 4B).

6.2.1 Applied Load during Heated Tests

Figure 6.5 shows the load levels applied to the slabs during each of the heated tests. Ignition of the panels is at $t=0$ mins. Prior to ignition, the load applied to each slab was set to approximately the Fire Limit State load of 24kN (12kN for exp. 6, unrestrained). Experiment 2A had a lower load of 21.5kN caused by a calibration error (see Section 5.2) while Experiment 3 and 4A (steel mesh and unreinforced) had constant loads of 25kN. Figure 6.6 also shows how the load was increased in Experiments 3, 4A and 5 after 150 minutes of heating had elapsed. The PFRC and unreinforced slabs (3 and 5) failed at loads of 33.4kN and 36.5kN respectively. The mesh reinforced slab in Experiment 4A survived increased loading up to 34.6kN with no extra visible damage, so the reloading in that experiment was halted.

![Figure 6.5: Applied load versus time for each heated experiment](image-url)
6.2.2 Slab Deflections during Heated Tests

Figure 6.6 shows the central deflections recorded during the heating phases in Experiments 2A to 6. Again, ignition is at $t=0\text{mins}$, prior to which the slabs had deflected between 1.0mm and 1.4mm due to the applied Fire Limit State loads. The data for Experiment 2A (PFRC slab, heating and cooling) shows the small peak of 12.1mm at 20mins due to failure of the panel, and is not a fair comparison with the other records due to the cooling which caused its increasing deflection. The PFRC (Experiment 3), steel reinforced (Experiment 4A) and unreinforced (Experiment 5) all showed a similar pattern of behaviour (non-linear rise), but arrived at different values of deflection after 2 hours and 30mins of heating. The PFRC slab reached 35.1mm at that point, in contrast to 30.0mm for the unreinforced slab and 21.2mm for the steel mesh reinforced slab. Two key findings from this comparison are that;

- the steel mesh reinforced slab had lower deflections than the others from approximately 10 minutes into heating, indicating that the mesh provided an overall increase in stiffness, and
- the PFRC slab started to deflect further than the unreinforced slab after approximately 40 minutes.

Figure 6.6: Slab central deflection versus time for each heated experiment
According to Figure 6.4, at 40 minutes the entire depth of each of the slabs had reached over 200ºC, indicating that the fibres would have largely melted (melting point 160ºC) from the middle of the PFRC slab.

The lack of fire resistance of the unrestrained PFRC slab (exp. 6) is illustrated in Figure 6.6. It deflected at a uniform rate, until at 9 minutes 40 seconds, total collapse of the slab occurred with an uncontrolled rise in deflection and breaking.

The observations indicate that the applied boundary restraint was responsible for the slabs’ fire resistance, and that the deflection of each slab was dependent on their reinforcement provision. The voids left by melted fibres in the PFRC slab would cause a lower stiffness than the monolithic unreinforced slab under the same conditions, causing the greater deflection observed (5.1mm, 145% greater then the steel reinforced slab).

### 6.2.3 Load-Deflection Behaviour during Heated Tests

Figure 6.7 shows the applied load versus deflection behaviour during the same series of tests. Figure 6.7 has no time axis; the horizontal sections of each line represent the deflection caused due to heating, while maintaining a constant load. In Experiment 2A, the 21.5kN load applied remained constant during heating and cooling, before being removed with a reduction in deflection from 21.5mm to 12.5mm.

The PFRC slab (Experiment 3), mesh reinforced (Experiment 4A) and unreinforced (Experiment 5) tests all show the same pattern of loading, heating, further loading and load removal. The final deflections due to heating for Experiment 3, 4A and 5 were 35.1mm, 21.2mm and 30.0mm respectively. The loads on each heated slab were then increased, with the PFRC and unreinforced slabs in Experiments 3 and 5 reaching failure and the mesh reinforced slab in Experiment 4A increasing its deflection by 1.1mm. The stiffnesses of each heated slab during further loading were 3.4kN/mm, 8.7kN/mm and 4.1kN/mm for Experiments 3, 4A and 5 respectively. The steel mesh reinforced slab demonstrated over twice the stiffness of the PFRC and unreinforced slabs. In Experiment 6, the unrestrained PFRC slab failed due to the
heat exposure without any change in the applied load of 12kN.

Figure 6.7 also shows that the fully restrained slabs (all expect Experiment 6) had a similar stiffness under the initial applied load. Figure 6.8 shows the same data as Figure 6.7, but shows only the deflection range 0mm-2mm to provide extra detail. The two PFRC slabs demonstrated slightly lower stiffness during the initial loading (18.6kN/mm and 15.5kN/mm) than the steel mesh and unreinforced slabs (19.6kN/mm and 22.8kN/mm). The unrestrained slab had a stiffness of approximately half these values (9.1kN/mm), due to the change in support condition (see Section 5.6).

The reduced deflection range in Figure 6.8 shows up the very stiff response recorded at very low load levels, near the graph origin. Experiments 2A, 3 and 4A only begin recording linear deflections at an initial load in the range 4 to 5kN, with Experiments 5 and 6 doing so at a range of 2 to 2.5kN.
6.2.4 Load-Deflection Behaviour of Ambient Loading Tests

Figure 6.9 shows the load-deflection data for the ambient loading tests in Experiment 1, Experiment 2B and Experiment 4B. In Experiment 1, the fully restrained PFRC slab exhibited two stages of behaviour; there was a stiff, flexural response leading to cracking of the slab at 36.8kN, followed by a compressive axial response up to the ultimate load of 40.4kN. The cracked, restrained slab in Experiment 2B only exhibited the compressive axial response and failed at a load of 30.0kN. The slab gradually lost stiffness as loading increased and deflection began to run away.

The restrained steel mesh reinforced slab from Experiment 4B had a linear load-deflection response up to a load of 58kN. The stiffness up to this load was approximately the same as the initial stiffness observed in Experiment 2B, despite the presence of steel mesh reinforcement and lack of severe cracking. After 58kN, the mesh reinforced slab lost some stiffness as the reinforcement began to fail along the North edge, reaching a peak load of 76.7kN.
The presence of steel mesh reinforcement gave the slab in Experiment 4B a much higher strength, and also changed the slab failure mode. Figures 6.10 and 6.11 show the post-failure loaded faces of the slabs in Experiments 1 and 2B. The crack patterns are visible, showing the locations of full-depth cracks forming the slab failure mechanisms. Figure 6.12 shows the post-failure loaded face of the mesh reinforced slab in Experiment 4B. The full depth crack is along the North side, where the broken reinforcement can be seen protruding from the crack surfaces. As the broken slab lifted, cracks developed along the East and West sides but the reinforcement along these regions remained intact. There was no severe cracking in the interior, with no comparable slab failure mechanism.
Figure 6.10: PFRC ambient test, Exp. 1 post-failure showing failure mechanism

Figure 6.11: PFRC re-loading test, Exp. 2B post-failure showing failure mechanism

Figure 6.12: Steel reinf’d slab re-loading test post-failure, with uncracked interior
6.2.5 Summary of Deflection Behaviour

This section summarises the deflection behaviour observed during the test programme as described above.

- In the heated tests with full applied boundary restraint, the PFRC, unreinforced and steel mesh reinforced slabs exhibited similar patterns of deflection.
- Under the initial applied load pre-heating, the PFRC slabs were slightly less stiff than the unreinforced or mesh reinforced slabs.
- After 2 hours 30 minutes of heating, the PFRC slab had deflected 35.1mm, the unreinforced slab 30.0mm and the mesh reinforced slab 21.2mm.
- The differences in deflection are accounted for by the stiffness of each slab, due to the reinforcement in each. The mesh reinforced slab was stiffer than the others during loading and heating. The PFRC slab became less stiff than the unreinforced slab at temperatures over 200ºC.
- The collapse of the unrestrained slab (Exp. 6) after 10 minutes was a result of low flexural strength and no effective interaction with the surrounding frame.

The inability of unrestrained PFRC slabs to sustain load in fire is of key significance. The test series demonstrated that a level of structural resistance to thermal expansion is essential for a PFRC slab to develop a load-carrying mechanism in fire. This has critical implications for the design of composite structures using polymer fibre reinforced concrete.
6.3 Comparison of Reaction Frame Results

This section compares the axial forces and moments recorded during heating and loading of the slab specimens, across the experiment series. This is used to assess the performance of each slab in terms of their structural load-carrying mechanisms.

6.3.1 Axial Reaction Force during Heated Tests

Figure 6.13 compares the axial forces recorded by the perimeter gauges during each heated test. Data for Experiment 3 (second heated PFRC test) are missing due to the failure of the data logging equipment. The failure of the heating panel 20 minutes into the PFRC test (Experiment 2A) caused the rising axial reaction to revert, and cracking of the slab at 30 minutes caused the reaction force to stay steady during cooling. The heating phases of the PFRC and steel mesh reinforced tests (Experiment 2A and Experiment 4A) recorded very similar patterns of axial reaction, indicating the steel mesh reinforcement did not affect the compressive axial response. The recorded reaction for the unreinforced slab test (Experiment 5) starts at a higher level, due to a small (≈4kN) axial reaction to the initial applied load pre-heating. This was caused by an imbalance in the compressive and tensile reactions to the load.

![Figure 6.13: Comparison of perimeter gauge reactions during heated tests](image-url)
Otherwise, the axial reaction during heating in the unreinforced slab test is similar to those of the mesh reinforced and PFRC slab tests, before 20 minutes. The pattern of axial reaction recorded by the perimeter gauges shows a rapid rise to approximately 20kN over the first 30 minutes of heating. The mesh reinforced slab held a steady reaction till the end of the test, while the unreinforced slab recorded a slow increase up to 26.7kN after two hours of heating.

In the unrestrained PFRC test (Experiment 6), there was no significant axial response recorded by the perimeter gauges, either due to the applied load or heating. The applied load of 12kN caused a slightly negative average reaction force of -0.5kN from the frame. This reaction was not affected by the temperature increase, over the 10-minute heating duration. The negative 0.5kN reaction reverted back up to zero with load removal the end of the test. This lack of axial reaction force was the cause of the unrestrained slab’s limited fire resistance.

Figure 6.14 shows the average axial forces recorded by the column gauges. This shows a similar dataset to Figure 6.13, but the column reactions were available from Experiment 3 (second PFRC test) so it can be included and compared this way.

![Figure 6.14: Comparison of column gauge reactions during heated tests](image-url)
The first PFRC test and the unrestrained PFRC test (Experiments 2A and 6) show the same patterns of behaviour as in Figure 6.13, with the first PFRC test recording a peak at 20 minutes due to failure of the heating panel, and the unrestrained test failing to record any significant reaction. The remaining tests recorded very similar patterns of axial compressive reaction. After 2 hours and 30 minutes of heating, the PFRC slab in Experiment 3 had reached an average reaction of 30.1kN, compared to 35.1kN and 37.7kN for the steel mesh and unreinforced slabs respectively.

The patterns of reaction force from the column gauges in Figure 6.14 show a similarity with the shape of the time-temperature curves for the slabs during heating (see Figure 6.1). This demonstrates the link between temperature increase and restrained expansion force. In Figure 6.13, the reaction plots from the perimeter gauges have a more bi-linear shape, with the gradients changing at 20 minutes. The reason for the difference is that the column results incorporated an effect from the applied and thermal moment reaction at the slab perimeter.

Despite these, the axial reactions from the restraining frame demonstrated the compressive reaction of the heated slabs, and the reduction in reaction force due to slab cooling. The roller bar edge arrangement for the unrestrained slab was effective in representing a condition of no in-plane slab restraint, as neither the perimeter or column gauges recorded any significant reaction force in Experiment 6.

From the data presented above, it can be concluded that the provision of perimeter restraint to thermal expansion has more effect on a slab’s fire resistance than using polymer fibre reinforcement. Each of the restrained heated slabs behaved in a similar fashion regardless of the reinforcement provision; they resisted 2 hours 30 minutes of fire exposure with no severe cracking on the heated face. But the PFRC unrestrained slab failed after only 10 minutes of the same heat exposure. Removing the in-plane restraint meant that the PFRC slab could not interact with the surrounding structure, and no secondary load-carrying mechanism was developed. The ability of the polymer fibre reinforcement to delay collapse of the heated, unrestrained slab was seen to be negligible during Experiment 6.
6.3.2 Frame Reaction Moment during Heated and Ambient Tests

This section describes the moment reactions recorded by the frame during the heated and ambient experiments. This is used to provide a general comparison of the behaviour of each of the slabs tested. Averaged moment reaction data is provided per test; discussion of individual moment gauge response, including the differences in behaviour between individual columns within each, is given in Chapter 5.

6.3.2.1 Heated Tests

Figure 6.15 shows the average recorded moment response for the heated tests; again, strain data for Experiment 3 (second PFRC test) was unavailable. Each test recorded a rise in moment due to the initial applied load. These moments pre-heating were 0.79kNm, 0.86kNm, 1.06kNm and 0.43kNm for the first PFRC test, the mesh reinforced test, the unreinforced test and the unrestrained test respectively. The roller bar support condition for the unrestrained slab test (Experiment 6) still generated a boundary reaction moment due to the shear reaction present at the edge. This reaction was not affected by the rise in slab temperature or deflection, and reverted to zero at slab failure and removal of the load.

![Figure 6.15: Average recorded reaction moment during heated tests](image-url)
Chapter 6 - Analysis and Discussion

The three restrained tests all demonstrated an increase in reaction moment, albeit at different rates. The PFRC test (Experiment 2A) rose in reaction moment until failure of the heating panel, reaching a peak of 1.53kNm before levelling out as the slab began to cool.

The steel mesh and unreinforced slabs both show a 30 minute delay in the rise of reaction moment. The reason for this delay in average response is the same in both cases. While certain gauges on the frame were recording an increase in moment, others were recording a similar decrease. The best explanation for some of the gauges recording a decrease in moment was a lack of fixity around the frame/slab junction, causing some time before the slab became fully engaged with rotational restraint within the frame.

Evidence for this is that after 30 minutes, the average gauge response began to rise and record the contribution from the thermal moment effect on the restrained slabs. The records from the steel mesh and unreinforced slabs track closely together, rising from approximately initial values to 2.1kNm between 30mins and 2 hours 30mins of heating. Both also show a rise in moment proportional to the further applied load at the end of each test. In the steel mesh test (Experiment 4A) the load was increased from 25kN to 34.6kN, with the average moment reaction peaking at 2.3kNm. In the unreinforced test (Experiment 5) the load was increased to 36.5kN, reaching slab failure, with the average moment peaking at 2.6kNm.

6.3.2.2 Ambient Tests

Figure 6.16 shows the linear relationship recorded between applied load and average reaction moment in the ambient PFRC slab load test (Experiment 1), the reloading of a damaged PFRC slab test (Experiment 2B) and the reloading test of the steel mesh reinforced slab (Experiment 4B). The records from the ambient load test and the damaged PFRC reloading test show a very similar relationship, up until their respective peak loads. This outcome is expected as the reaction moment across the slab boundary is a function of the load applied and the test geometry, irrespective of
the slab type or reinforcement. However, as slab failure is approached, the recorded moment began to increase; this was due to the transition between flexural and axial modes of behaviour, and cracking of the slab on the loaded side. The steel mesh reinforced reloading test recorded a slightly higher reaction moment in response to the same applied loads, and maintained a lower gradient up to its failure load of 76.7kN.

### 6.3.3 Summary of Reaction Behaviour

The overall pattern of reaction behaviour was similar for all the restrained heated tests; as the slabs rose in temperature and deflected, an in-plane compressive reaction was generated which rose fastest during the first 30 minutes. This in-plane force contributed to the fire resistance of each tested slab; the unrestrained PFRC slab in Experiment 6 could not develop an in-plane compressive force and lasted only 10 minutes under heating.
6.3.4 Summary and Shortcomings of Experimental Results

The experimental data generated during this project reveal how essential the property of in-plane perimeter restraint is to the fire resistance of polymer-fibre-reinforced concrete composite slabs. The slab behaviour captured in the deflection and reaction force data will be of use in the validation of structural modelling tools and as a benchmark for further, complex numerical analyses. The slab temperature data also provides a benchmark for thermal or heat conduction analyses involving fibre and steel reinforced concrete. The data and mechanisms recorded by the reaction frame comprise a key contribution to knowledge and allow the practical understanding of slab-frame interaction to be taken further. This is a significant consideration in a regulatory environment which relies on unrepresentative, single element furnace tests to determine the performance of full structural assemblies.

The principal shortcoming of the data collected was the failure to observe a tensile membrane mechanism of structural fire resistance. The axial reaction results of Experiment 4A demonstrated that the frame could record an in-plane tensile reaction, which in that case arose from thermal contraction during slab cooling (see Figure 5.51, after 180 minutes). However, due to the slab dimensions used across the tests, the specimens were too stocky to ‘snap through’ into a load carrying mechanism which utilised the tensile strength of the slab. This may be demonstrable with the use of a thinner slab and a lighter reinforcing steel rod or wire mesh.
Chapter 6 - Analysis and Discussion

6.4 Model Refinement based on Experimental Data

An aim of the project was to conduct a numerical analysis of the experiment series, leading to a robust material model to describe the PFRC behaviour in fire. This could then be used to model the use of PFRC slabs in full-scale structures. The \textit{a priori} modelling described above achieved its goal of verifying the general experiment design.

The \textit{a priori} model, however, contained a number of assumptions, which were made because of various unknown parameters before the experiments were carried out. These were:

- the thermal input provided by the radiant panels to the slab;
- the ambient capacity of the slab, and hence the load to be applied during heated tests;
- the stiffness of the boundary restraint provided by the frame; and
- the details of the PFRC material model.

The following sections explain how the thermal input, boundary conditions and load application can be addressed in the model after the experiments. This would allow the details of the PFRC material model and hence the behaviour of PFRC concrete slabs to be better understood. This work, however, proved to be more involved and complex than had been anticipated, and has not been completed as part of this thesis; it is included here to provide a starting point for future research to be carried out.

6.4.1 Thermal Input

The \textit{a priori} model considered a 1 hour exposure to the standard fire curve. The experiment series used a radiant panel which subjected the slabs to $55\text{ kW/m}^2$. As seen in Section 6.1, the temperature evolutions were similar across each of the long-duration heated tests. A representation of the slab temperature evolution for use in a numerical simulation is shown in Figure 6.17, with the data in Table 6.1 splitting the heating phase into 5 discrete time steps.
Table 6.1: Temperature data (°C) from experimental results for numerical modelling

<table>
<thead>
<tr>
<th>Depth</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>30</th>
<th>60</th>
<th>Step time</th>
</tr>
</thead>
<tbody>
<tr>
<td>36mm</td>
<td>143</td>
<td>329</td>
<td>469</td>
<td>533</td>
<td>580</td>
<td>Total time</td>
</tr>
<tr>
<td>28mm</td>
<td>111</td>
<td>268</td>
<td>411</td>
<td>481</td>
<td>530</td>
<td></td>
</tr>
<tr>
<td>20mm</td>
<td>80</td>
<td>211</td>
<td>355</td>
<td>430</td>
<td>481</td>
<td></td>
</tr>
<tr>
<td>12mm</td>
<td>60</td>
<td>178</td>
<td>320</td>
<td>393</td>
<td>442</td>
<td></td>
</tr>
<tr>
<td>4mm</td>
<td>52</td>
<td>160</td>
<td>293</td>
<td>361</td>
<td>407</td>
<td></td>
</tr>
</tbody>
</table>

6.4.2 Boundary Conditions and Applied Load

The provision of boundary restraint to the slab was simulated by modelling the test frame in the *a priori* analyses. During the experiment series, the frame was tested for its lateral stiffness in the plane of the slab (see Section 4.2.2, Portal Frames). The results are presented in Appendix A.1, with the lateral stiffnesses being between 65 and 70kN/mm.
The load applied during the tests was transmitted through four 200mm square steel plates, as a more uniformly distributed load was impractical due to the instrumentation. This had an effect on the eventual shape of the slab failure mechanisms, as seen in Figure 5.1 etc. A more realistic numerical simulation would use the recorded loads (~25kN during heating) and the loading pad geometry from Figure 4.2.

6.4.3 PFRC Material Modelling

The information provided above allows a numerical simulation of the experimental behaviour to be used to compute a robust material model for PFRC, based on the test results. Wedge splitting test results were the starting point for the stress-displacement material model used in the \textit{a priori} testing. A stress-displacement relationship was adopted to avoid the convergence and element size issues common in FE analysis of unreinforced brittle materials.

Characterising the loss of concrete ductility that gradually occurs as the fibres approach their melting point at 160°C is a key step to a robust material model. Other approaches to inelastic behaviour, like a temperature-dependant fracture energy criterion could be adopted depending on the analysis type used.
Chapter 6 Summary

This chapter has presented further analysis of the results presented in Chapter 5. A comparison of the performance of the slabs across the experiment series has been provided and explained. The thermal exposure across the heated tests was shown to be uniform. The differences in slab behaviour (deflection rates and perimeter reactions) have been discussed and explained.

The lack of fire resistance observed in the unrestrained, polymer fibre reinforced slab test (Experiment 6) has also been discussed, in terms of the integrity of the polymer fibres in a heated slab and the reaction force data recorded by the restraining frame. The fibres melted early in the tests relative to the duration of fire exposure and were unable to play a role in maintaining the integrity of the slab. The fire resistance of the slabs tested has been shown to be due to structural interaction with the restraining frame, arising from restrained thermal expansion.

Polymer fibre reinforced concrete composite slabs can therefore only exhibit fire resistance if the surrounding structure is sufficiently stiff to generate a compressive reaction strong enough to support the slab. This is posited as a reason for the adequate performance of PFRC slabs in isolated fire resistance tests. Ensuring the boundary stiffness of PFRC slab panels is therefore critical in the design process. PFRC slabs should be used with caution around the perimeter edges of buildings where sufficient restraint may not be available.

The data show convincingly that applied restraint permits longer periods of fire resistance to concrete slabs, and that the presence of polymer fibres has no discernible beneficial effect on the fire resistance of a concrete slab with in-plane restraint to thermal expansion.

This chapter concluded by presenting formatted temperature data and modelling techniques to be used in a robust \textit{a posteriori} numerical simulation project, to yield further understanding from the experiment series.
In this thesis, an experimental investigation into the fire performance of polymer fibre reinforced composite concrete slabs has been presented. This chapter summarises the conclusions from the experiment series, and suggests areas for work which would provide further useful information about the fire resistance and design of such structures.
Chapter 7 - Conclusions and Further Work

7.1 Conclusions and Recommendations

This section presents the conclusions from the experimental and numerical studies described in the previous chapters. The three principal conclusions are as follows.

- The ambient load test on a restrained PFRC slab specimen (Experiment 1) captured a clear transition between two modes of structural response. An initially stiff, flexural perimeter moment response changed when the slab cracked, to a load-carrying mechanism characterised by high in-plane compressive forces bracing the cracked slab across the stiff boundary.

- The similar behaviour of each of the restrained tests shows that despite the reinforcement provision, the key parameter controlling fire resistance in this series of experiments was the ability of the slabs to react in compression against the rigid test frame. The restraint allowed load to be carried even though all fibres had melted.

- The unrestrained, heated PFRC slab did not record any significant interaction with the restraining frame. The slab deflection rose steadily until a pattern of cracks appeared just before 10 minutes had elapsed. With no restraint to thermal expansion, the crack propagation was not halted and total slab collapse occurred at 10 minutes. This further demonstrates that PFRC slabs only carry load in fire due to restraint.

The fire resistance testing previously performed on polymer fibre reinforced composite concrete slabs, and currently used as a basis for design, does not give a realistic representation of their service conditions. The standardised single element furnace tests completed to date are unable to characterise the complex structural interactions which dominate the response of steel framed structures in fire. Furthermore, an unknown degree of restraint is present during standard approval tests, and this thesis shows that the degree of restraint is critical to the slab behaviour.
Consequently, there is a lack of applicable, full-scale test data on which to base the performance-based fire design of PFRC structures. There exists a critical lack of understanding of how the use of PFRC slabs affects full-frame behaviour and load carrying mechanisms in fire, relative to a traditional steel mesh reinforced composite slab.

This thesis has presented an investigation into the structural response of PFRC slabs at high temperatures, with a focus on the perimeter restraint and detailed understanding of thermal and mechanical mechanisms. Conducting this research involved the design and construction of a custom testing frame capable of accommodating the test equipment, and also of direct measurement of the slab’s structural response to loading and heating. Use of calibrated strain gauges to quantify the force and moment reactions of a restrained slab in detail has, to the author’s knowledge, never been attempted in slab fire tests whether restrained or unrestrained. This technique generates a greater understanding of the full structural response of slabs to fire.

- The ambient load test on a restrained PFRC slab specimen (Experiment 1) captured a clear transition between two modes of structural response. An initially stiff, flexural perimeter moment response changed when the slab cracked, to a load-carrying mechanism characterised by high in-plane compressive forces bracing the cracked slab across the stiff boundary.

- Three fully restrained fire tests were conducted on PFRC, plain concrete and mesh reinforced concrete slab specimens, and were directly comparable. All three performed similarly, but the mesh reinforced slab exhibited the greatest fire resistance by deflecting the least and possessing the highest residual strength.

- The PFRC and unreinforced slabs’ deflections tracked similarly up to a time (40mins) when the polymer fibres would have melted through the entire depth of the PFRC slab. After this had occurred, the PFRC slab was less stiff and
began to deflect further than the unreinforced slab. Therefore, the fibres could not have carried any load and another mechanism must have acted in support of the slab.

- A common feature of the three restrained heated tests was that there was no significant cracking on the tensile, heated face. Thermal bowing was responsible for the early increases in deflection seen in the tests; the compressive thrust through the slab and the increase in length due to thermal expansion meant that tensile cracking stresses did not occur in the slabs during heating.

- That the unrestrained PFRC slab recorded no significant axial reaction force is proof that firstly, the roller bar support arrangement used to provide simple support with room for lateral movement was successful. Secondly, this result also shows that the strain gauges on the frame were effectively insulated from the heat, which did not interfere with the strain measurement.

PFRC slabs are unlikely to adopt a high-deflection load-carrying mechanism in fire, due to the lack of continuous reinforcement spanning through the slab. It is recommended that sufficient fire protection/insulation is applied to the primary and secondary beams in a PFRC composite panel. In this regard, the cost savings attributed to using PFRC slabs over traditional mesh reinforced composite slabs will be affected by the need for full fire protection.

The PFRC slabs tested in this investigation developed full depth cracks and broke into separate facets at failure. The role of the steel deck in preventing collapse has not been addressed, and the results herein are thus a worst case assessment of the behaviour of PFRC slabs. Nevertheless, the tests have given valuable understanding that it is important to recognise in design.
7.2 Further Work

This thesis has presented a model-scale experimental investigation into the fire performance of polymer fibre reinforced composite concrete slabs. The following is a list of suggestions on how to take this research further, or address questions on the subject of PFRC slabs in fire which were not covered by this thesis.

- A pair of further heated tests, conducted on mesh reinforced and unreinforced slab specimens in the unrestrained arrangement (Experiment 6) would allow for a more meaningful comparison of the fire resistance capacity of composite slabs without applied restraint.

- Equipment failures during the tests (heating panel in Experiment 2, strain data collector in Experiment 3) highlighted the necessity of redundancy in the test programme. Repeated tests on the same slab configurations as tested would provide a greater level of certainty about the behaviours and interactions observed.

- An expansion of the test frame, such that it becomes capable of testing composite slabs with more representative span/depth ratios would be of significant benefit. Tensile membrane action was not observed directly in this series of experiments, due to the scale of equipment available. Testing longer spans would allow the frame to capture the outward compressive force and subsequent tensile pull that characterises slabs developing tensile membrane action.

- A more detailed assessment of the change in material properties at temperatures around the fibre melting point would permit the development of a more robust numerical model to describe the PFRC. This model could then be used to complete the numerical simulation of the experiment series as explained in Chapter 6. It could also play a role in analysing the structural behaviour of full scale slab panels in performance-based design of steel structures.


British Steel, 1999: *The Behaviour of Multi-Storey Steel Framed Buildings in Fire*, British Steel plc, Swinden Technology Centre 1999


References


Dassault Systèmes, 2010: Abaqus 6.10 Finite Element Analysis Package, Dassault Systèmes, Simulia Corp 2010. Providence, RI, USA


Etheridge, 1933: Concrete Construction. US patent 1913707, Etheridge, H., granted 13 June, 1933

Fox, 2010: The Fire Performance of Polymer Fibre Reinforced Composite Concrete Slabs. Fox, D., Stratford, T. 6th International Conference Structures in Fire, University of Michigan, June 2010


JCI-SF4, 1984: Method of Tests for Flexural Strength and Flexural Toughness of Fiber Reinforced Concrete. JCI Standard SF-4, Japan Concrete Institute June 1984


Krenchel, 1964: Fibre Reinforcement, Krenchel,H. Akademisk Forlag, Copenhagen 1964

Labib, 2006: An investigation into the use of fibres in concrete industrial ground-floor slabs. Labib, W., Eden, N. School of the Built Environment, Liverpool John Moores University, Liverpool
References


Mindess, 2008: Fibrous Concrete Reinforcement: Developments in the Formulation and reinforcement of concrete: Ch. 7, pp. 154-166. Woodhead Publishing Ltd


References


Tata Steel, 2012: *ComFlor® Composite Floor Decking*. Product Literature, Tata Steel, 2012


A.1 Measurement of Frame Stiffness

The lateral stiffness of the test frame had to be evaluated prior to the experiment series, to show that it would provide sufficient restraint to the thermal expansion of the slab specimens. To assess the lateral stiffness, a load jack was attached between the slab edge restraints at the sides of the frame. This is illustrated in Figure A.1. The load was increased and the deflection of each of the columns was recorded. This was done across the East-West axis, then again for the North-South axis.

The results for the East-West test (Columns 1, 2, 5, and 6) are presented in Figure A.2 and the results for the North-South (Columns 3, 4, 7, and 8) and presented in Figure A.3. Photographs of the load jack arrangement and the deflection gauges on the South side of the frame are shown in Figure A.4 and A.5 respectively.
Appendix - A.1: Measurement of Frame Stiffness

Table A.1: Recorded stiffnesses of columns

<table>
<thead>
<tr>
<th>Column</th>
<th>Stiffness (kN/mm)</th>
<th>Column</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>62.4</td>
<td>5</td>
<td>72.3</td>
</tr>
<tr>
<td>2</td>
<td>89.0</td>
<td>6</td>
<td>96.0</td>
</tr>
<tr>
<td>3</td>
<td>73.5</td>
<td>7</td>
<td>83.1</td>
</tr>
<tr>
<td>4</td>
<td>328.6</td>
<td>8</td>
<td>71.5</td>
</tr>
</tbody>
</table>

Figure A.2: Frame column stiffness in East-West direction

Figure A.3: Frame column stiffness in North-South direction
Figure A.4: Photograph of load jack arrangement across frame

Figure A.5: Photograph of deflection gauges on Columns 7 and 8, South side
A.2 Strain Gauge Reaction to High Temperatures

This section presents a check conducted on the vulnerability of the perimeter strain gauges to high temperatures. Despite the strain gauges being wired as temperature compensated pairs, there remained concern that excessive heat could reach the gauges by convection of hot gas or by conduction through the frame during the tests. This would affect the force measurements and accuracy of the results. A diagnostic test was proposed to run the heating panel under experimental conditions and check to what extent the gauges were affected by high temperatures.

A 1.5 inch thick mat of insulating material was placed within the restraining frame to represent the slab. Thinner (~0.5 inch) board insulation was propped around the sides of the frame, protecting the perimeter gauges. This insulation arrangement is illustrated in Figure A.6, showing the panels in operation with one side of the frame removed.

The radiant panel was lit and the force response from each of the perimeter strain gauges was recorded. The force results for the perimeter gauges on the North side (3A, 3B, 4A, 4B), East side (5A, 5B, 6A, 6B) and South side (7A, 7B, 8A, 8B) are shown in Figures A.7, A.8 and A.9 respectively.

Figure A.6: Panel operation showing gauge test insulation arrangement
Figure A.7: Temperature sensitivity of North side perimeter gauges

Figure A.8: Temperature sensitivity of East side perimeter gauges
With two exceptions (5A and 7A) the gauges responses due to the heating panel operation were within the range $\pm 2\text{kN}$. The 25 minutes elapsed in this test were sufficient for the panel faces to attain a steady temperature, colour and heat flux. It was concluded from this test that the strain gauges were not critically susceptible to temperature-induced variation in measuring force, assuming an accuracy of $\pm 2\text{kN}$.

During the heated tests, additional insulation was provided to the frame perimeter to further alleviate the risk of temperature-induced errors in measuring the frame response. 1.5inch thick insulation mats were placed along the exposed sides (top and front) of the square hollow sections, to reduce the heat conduction rate to the perimeter gauges.

The success of the insulation and temperature compensation arrangement was shown by the results of the unrestrained slab test (exp. 6). No reaction force was recorded by the perimeter gauges during heating, indicating that neither the unrestrained slab nor the operation of the heating panels were generating a structural response from the frame.
A.3 Calibration of Strain Gauges

This section presents the full data from the calibration procedure used to convert strain readings from the perimeter and column gauges into measurements of force for structural analysis. Figures A.10 to A.17 show the applied load versus strain data for each gauge. The calibration procedure was explained in Section 4.3.3.

Figure A.10: Evaluating calibration factors for 1A, 1B, 2A, 2B

Figure A.11: Evaluating calibration factors for 1C, 2C
Appendix - A.3: Calibration of Strain Gauges

Figure A.12: Evaluating calibration factors for 3A, 3B, 4A, 4B

Figure A.13: Evaluating calibration factors for 3C, 4C
Figure A.14: Evaluating calibration factors for 5A, 5B, 6A, 6B

Figure A.15: Evaluating calibration factors for 5C, 6C
Table A.2 summarises the calibration factors found for the gauges.
### Table A2: Calibration factors for strain gauges

<table>
<thead>
<tr>
<th>Gauge</th>
<th>Cali. Factor (kN)</th>
<th>Gauge</th>
<th>Cali. Factor (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>-0.057</td>
<td>5A</td>
<td>-0.071</td>
</tr>
<tr>
<td>1B</td>
<td>-0.127</td>
<td>5B</td>
<td>-0.110</td>
</tr>
<tr>
<td>1C</td>
<td>0.571</td>
<td>5C</td>
<td>0.521</td>
</tr>
<tr>
<td>2A</td>
<td>-0.062</td>
<td>6A</td>
<td>-0.049</td>
</tr>
<tr>
<td>2B</td>
<td>-0.058</td>
<td>6B</td>
<td>-0.066</td>
</tr>
<tr>
<td>2C</td>
<td>0.418</td>
<td>6C</td>
<td>0.616</td>
</tr>
<tr>
<td>3A</td>
<td>-0.098</td>
<td>7A</td>
<td>-0.054</td>
</tr>
<tr>
<td>3B</td>
<td>-0.082</td>
<td>7B</td>
<td>-0.165</td>
</tr>
<tr>
<td>3C</td>
<td>-0.477</td>
<td>7C</td>
<td>0.513</td>
</tr>
<tr>
<td>4A</td>
<td>0.085</td>
<td>8A</td>
<td>-0.109</td>
</tr>
<tr>
<td>4B</td>
<td>-0.090</td>
<td>8B</td>
<td>-0.050</td>
</tr>
<tr>
<td>4C</td>
<td>0.572</td>
<td>8C</td>
<td>0.450</td>
</tr>
</tbody>
</table>