The Structural Performance of Non-metallic Timber Connections

submitted by

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of the

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Signature of Author ..........................................................

Andrew Thomson
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Abstract

Reducing the amount of metal used within a timber structure has many advantages, particularly when dealing with connections. Fire resistance and durability are commonly cited benefits. In addition the use of alternative connector materials minimises thermal bridging and can also provide a lighter weight structural solution.

Existing contemporary forms of non-metallic timber connections are commonly provided through the use of adhesives. However, these connections are reliant on a need for careful offsite, prefabricated construction. Traditional green oak carpentry connections provide a mechanically fastened non-metallic solution. However, carpentry connections are not widely compatible with contemporary architectural design or with the use of modern engineered timber products such as glulam.

Building upon research completed at the University of Bath, the aim of this thesis was to develop a mechanical, non-metallic connection system suitable for contemporary applications. Specific objectives were to investigate the structural performance of a defined connection system and to develop analysis methods to facilitate design.

A review of the literature demonstrated a lack of uptake and use of mechanical non-metallic connections. Guidance for the design of mechanical fasteners reflects the lack of innovation and research into the use of non-metallic materials.

Following an initial experimental investigation of non-metallic materials, an experimental testing programme was completed to investigate the use of glass fibre reinforced plastic (GFRP) dowels in conjunction with densified veneer wood (DVW) plates. The findings of the experimental study demonstrate that the use of these materials can provide a robust connection system for contemporary applications. The results of the experimental work provide guidance on dowel spacing requirements, connection response to load and connection failure modes. The failure modes of the proposed connection system were shown to be unique to the materials used and specific strength analysis methods have been developed to predict connection yield and ultimate strength. A method for predicting initial connection stiffness was also developed through the use of a beam on elastic foundation model.
Contents

Acknowledgements i

Abstract ii

List of figures x

List of tables xii

List of symbols xiii

1 Introduction 2

1.1 Timber connections ................................. 3
  1.1.1 Traditional carpentry connections ................. 4
  1.1.2 Metallic connections .......................... 7
  1.1.3 Non-metallic connection methods .................. 10
1.2 Rationale ........................................ 11
1.3 Scope of thesis .................................. 12
1.4 Objectives of study ............................... 14
1.5 Layout of thesis ................................ 15

2 Literature review 16

2.1 Analysis and design of metallic dowel and plate connections .... 17
  2.1.1 European yield model ........................... 17
  2.1.2 Brittle failure of structural members ............ 21
  2.1.3 Connection slip ............................... 27
2.2 GFRP dowel and plate connections ........................ 28
2.3 Pegged mortice and tenon connections ................... 31
  2.3.1 University of Wyoming research ................. 32
  2.3.2 Shanks ....................................... 35
2.4 Contemporary timber dowel connections .................. 38
  2.4.1 ETH Zurich .................................. 38
  2.4.2 Fukuyama et. al. ............................. 39
  2.4.3 University of Bath ............................. 40
2.5 Modified wood .................................... 41
2.5.1 Compressed wood ........................................... 41
2.5.2 Densified veneer wood ................................. 43
2.6 Concluding comments ................................. 48

3 Selection of dowel and plate materials 51
3.1 Dowel materials ........................................... 51
  3.1.1 Oak dowels ........................................... 51
  3.1.2 GFRP dowels ......................................... 52
  3.1.3 DVW and compressed wood dowels .................. 53
  3.1.4 Comparative testing ................................. 53
  3.1.5 Results and discussion ............................. 55
3.2 Plate materials ........................................... 60
3.3 GFRP-DVW connection ................................. 63
  3.3.1 Experimental investigation ......................... 63
  3.3.2 GFRP-DVW connection failure mode ............. 64
3.4 Concluding comments ................................. 66

4 Characterisation of connection components 67
4.1 Connection materials and fabrication .................. 67
4.2 Moisture content and density of LVL and Douglas fir glulam .... 69
4.3 Embedment strength and stiffness of Douglas fir glulam .... 69
4.4 GFRP-DVW bearing stiffness ........................... 71
4.5 Determination of E & G moduli for GFRP dowels ........ 74
  4.5.1 Graphical method .................................. 75
  4.5.2 Results and discussion ............................. 76
4.6 Effective bending resistance of GFRP dowels ........... 79
  4.6.1 Previous work ..................................... 82
  4.6.2 Novel testing method ................................ 83
  4.6.3 Testing programme .................................. 88
  4.6.4 Experimental results ............................... 90
4.7 Concluding comments ................................. 91

5 Experimental study of GFRP-DVW connections 92
5.1 Introduction ............................................. 92
5.2 Parallel to grain testing ............................... 94
  5.2.1 Dowel load share .................................. 95
  5.2.2 Reduced fastener spacings ......................... 96
  5.2.3 Influence of timber thickness ..................... 97
  5.2.4 Initiation of brittle failure ....................... 97
5.3 Results of parallel to grain tests ..................... 98
  5.3.1 Dowel load share .................................. 101
  5.3.2 Partial thickness shear plug ....................... 102
## List of Figures

1-1 A green oak carpentry frame under construction (Acknowledged to P. Gates) ........................................... 4

1-2 A single pegged mortice and tenon connection ................................. 6

1-3 The Golden Jubilee Room, Southampton .................................. 8

1-4 Konohana baseball practice dome, Miyazaki, Japan ................. 9

1-5 A joist repair using bonded in GFRP rods (Acknowledged to Rotafix) 11

1-6 An example of a connection made with multiple dowels and an in-plane flitch plate ........................................... 13

1-7 Connection slots being cut in timber members using a narrow gauge mortise tool ........................................... 14

2-1 Case 1 (left) and Case 2 (right) general dowel failure modes for built-in metallic dowels - $f_h = $ embedment strength (Larsen, 1973) ............ 18

2-2 Assumed material behaviour of metallic dowels in bending Larsen (1973) ........................................... 19

2-3 Assumed material behaviour for dowel embedment Larsen (1973) ... 19

2-4 EYM failure modes for metallic dowel-plate timber connections .......... 21

2-5 Brittle failure modes - a) net tension, b) group tear-out, c) plug shear, d) in-line splitting, e) perpendicular to grain splitting ............... 22

2-6 Definition of fastener spacings terminology .................................. 23

2-7 Perpendicular to grain splitting modes ........................................... 25

2-8 Connection failure response perpendicular to grain (Leijten and Van der Put, 2004) ........................................... 26

2-9 Type A (left) and Type C (right) failure modes ................................. 27

2-10 Three point bending test results used by Pedersen (2002) to determine $M_u$ and $V_u$ ........................................... 31

2-11 Possible mortice and tenon failure modes proposed by Miller et al. (2010) ........................................... 33

2-12 Three-plank, simulated connection test used by Shanks (2005) ...... 36

2-13 Resin cast mortice and tenon connections by Shanks (2005) ...... 38

2-14 Contemporary timber dowel plate connections (Stuerer, 2006) ...... 39

2-15 Connection test of oak dowel – birch plywood connection (Clarke, 2009) ........................................... 40
<table>
<thead>
<tr>
<th>Page</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-16</td>
<td>Partial shear plug failure in oak dowel – plywood plate connection (Chang et al., 2009)</td>
</tr>
<tr>
<td>2-17</td>
<td>Compressed wood dowels used at the Wood Utilization Centre, Miyazaki, Japan</td>
</tr>
<tr>
<td>2-18</td>
<td>In-plane tension and compression strength results for cross-wise DVW plate (Leijten, 1998)</td>
</tr>
<tr>
<td>2-19</td>
<td>Embedment data for DVW under short and long term loading (Leijten, 1998)</td>
</tr>
<tr>
<td>3-1</td>
<td>Double shear test specimens; parallel to grain (left) and perpendicular to grain (right)</td>
</tr>
<tr>
<td>3-2</td>
<td>5% offset analysis method used to determine connection yield</td>
</tr>
<tr>
<td>3-3</td>
<td>Typical load-slip plots for dowel material study</td>
</tr>
<tr>
<td>3-4</td>
<td>Failed parallel to grain specimens locked and dissected for inspection</td>
</tr>
<tr>
<td>3-5</td>
<td>Failed dowel-plate specimens</td>
</tr>
<tr>
<td>3-6</td>
<td>Load-displacement plots for plate material compatibility tests</td>
</tr>
<tr>
<td>3-7</td>
<td>Load displacement plot for GFRP-DVW connections</td>
</tr>
<tr>
<td>3-8</td>
<td>Failed GFRP-DVW connections (refer to Table 3.3 for specimen configurations)</td>
</tr>
<tr>
<td>4-1</td>
<td>Test setup for timber embedment strength and stiffness testing</td>
</tr>
<tr>
<td>4-2</td>
<td>Test setup used by Church and Tew (1997) for testing timber peg embedment strength</td>
</tr>
<tr>
<td>4-3</td>
<td>GFRP-DVW bearing stiffness test setup</td>
</tr>
<tr>
<td>4-4</td>
<td>Graphical interpretation of Timoshenko beam equations for the determination of E &amp; G moduli</td>
</tr>
<tr>
<td>4-5</td>
<td>Graphical method test results for 12 mm diameter GFRP dowels</td>
</tr>
<tr>
<td>4-6</td>
<td>Flexural modulus of pultruded GFRP rod for varying length (L) and diameter (d) ratios (Harvey et al., 2000)</td>
</tr>
<tr>
<td>4-7</td>
<td>EYM failure modes for timber dowel connections made with thick central steel plate</td>
</tr>
<tr>
<td>4-8</td>
<td>Close up of GFRP-DVW connection failure</td>
</tr>
<tr>
<td>4-9</td>
<td>EYM mode III failure for connection made with thick steel plate</td>
</tr>
<tr>
<td>4-10</td>
<td>Three point bending test on GFRP dowel</td>
</tr>
<tr>
<td>4-11</td>
<td>Development of novel GFRP dowel test method</td>
</tr>
<tr>
<td>4-12</td>
<td>Expected failure mode of metallic dowel compared with a GFRP dowel</td>
</tr>
<tr>
<td>4-13</td>
<td>Load-displacement response of GFRP dowel loaded in novel test setup (Points A, B, C correspond to diagrams of Figure 4-14)</td>
</tr>
<tr>
<td>4-14</td>
<td>Failure response of GFRP in novel test setup (Points A, B, C correspond to those on Figure 4-13)</td>
</tr>
<tr>
<td>4-15</td>
<td>Assumed plastic deformation of GFRP dowel</td>
</tr>
<tr>
<td>4-16</td>
<td>Test setup-effective bending capacity of GFRP dowels</td>
</tr>
</tbody>
</table>
4-17 Determination of $P_p$ from load-displacement plots ................. 90
5-1 Fabrication of specimen ........................................... 94
5-2 Test setup parallel to grain ....................................... 95
5-3 Section of test specimen showing equivalent shear areas of timber . 98
5-4 Cleavage failure of DVW plate ................................. 100
5-5 Net tension failure of DVW plate ............................... 100
5-6 Knot in side member of specimen from test group ‘a’ ............... 101
5-7 Partial shear plug detail (left) and progressive shear plug failure (right) 103
5-8 Typical load-slip plots for connections made with incrementally reduced in-line dowel spacing (12 mm diameter dowels) .......... 104
5-9 Specimens tested to investigate trigger of ultimate failure ....... 105
5-10 Influence of end and dowel spacing on ultimate failure capacity (12 mm diameter dowels) ........................................ 106
5-11 The limits of perpendicular to grain splitting on connection capacity 109
5-12 Test setup for perpendicular to grain connection tests .......... 110
5-13 Test setup shown for three dowel specimen (test group i) ....... 110
5-14 Load - slip response of perpendicular to grain specimens ....... 111
5-15 Splitting failure of single dowel specimen ..................... 113
5-16 Splitting failure of two dowel specimen ........................ 113
5-17 Splitting failure of three dowel specimen ...................... 114
5-18 Split in glue line of two dowel specimen ...................... 114
5-19 Connection components of disassembled specimen after testing ... 115

6-1 Specimen configuration for full scale GFRP-DVW connection with 12 No. 12 mm diameter GFRP dowels ................................ 119
6-2 Specimen configuration for full scale GFRP-DVW connection with 9 No. 12 mm diameter GFRP dowels ............................. 120
6-3 Specimen configuration for full scale metallic connection with 9 No. M12 stainless steel dowels ........................................ 121
6-4 Experimental setup for full scale connection tests ................. 123
6-5 Load-slip plots for full scale test specimens .................... 126
6-6 Load-slip plots for full scale GFRP-DVW specimens ............. 127
6-7 Typical load-slip plots for cyclically loaded GFRP-DVW specimens 128
6-8 Failed metallic specimens ........................................... 130
6-9 Failed GFRP-DVW specimens ...................................... 131
6-10 Glulam damage resulting from misaligned stainless steel dowel ... 133
6-11 DVW plate fabrication using handheld jigsaw .................... 134
6-12 Single locator dowel inserted through glulam and DVW plate ... 134
6-13 Single operation drilling of GFRP-DVW connection ............. 135
6-14 Completed GFRP-DVW connection .............................. 135
7-1 Assumed model for a GFRP-DVW beam on elastic foundation analysis 139
7-2 Schematic diagram of beam element on elastic foundation .......... 142
7-3 Nodal displacements (a), actions (b), and degrees of freedom (c) of a small beam element ........................................ 143
7-4 Schematic diagram of beam on elastic foundation for GFRP-DVW connection ....................................................... 148
7-5 Diagram of specimens for which stiffness has been analysed and predicted - letters d, a, i & h correspond with the test groups reported in Chapter 5 ........................................ 151
7-6 Graphical representation of stiffness model results parallel to grain (3 dowel specimens) ......................................... 154
7-7 Graphical representation of stiffness model results perpendicular to grain (3 dowel specimen) ........................ 154
8-1 GFRP-DVW connection failure modes ............................ 158
8-2 GFRP-DVW specimen failure modes ............................... 159
8-3 GFRP dowel failure .......................................................... 160
8-4 Experimental connection yield results plotted against predicted yield strength values ................................................. 164
8-5 Experimental test data values plotted with $\sqrt{GG_c/0.6}$ boundaries ................................. 167
8-6 Predicted splitting capacity shown against experimental load-slip plots (key relates to test group) .............................. 168
8-7 End view of failed specimen from test group ‘b’ (Chapter 5 ...... 172
8-8 End view of failed specimen from test group ‘c’ (Chapter 5 ...... 172
8-9 End view of failed specimen from test group ‘d’ (Chapter 5 ...... 173
8-10 End view of failed specimen from test group ‘e’ (Chapter 5 ...... 173
8-11 Dimensions used for analysis of partial plug shear failure ...... 174
8-12 Experimental test data values plotted with $p_l f_v, k$ values ........ 176
9-1 ‘Woodshed’ pavilion at the V&A (Danny Wicke) .................. 181
9-2 Internal view of the pavilion (Danny Wicke) ......................... 182
9-3 Softwood timber air drying prior to fabrication ........................ 183
9-4 Fabrication of a GFRP-DVW connection .............................. 183
9-5 Connection assembled in the workshop .............................. 184
9-6 Connections in the finished pavilion ................................. 184
A-1 Schematic diagram of beam on elastic foundation .................. 198
A-2 Shear deformation of small beam element ........................... 199
A-3 Equilibrium of vertical forces on beam element .................... 200
## List of Tables

2.1 Minimum spacings and end and edge distances for dowels (BS EN 1995, 2004) ................................................................. 23

3.1 Results summary - dowel material comparison for 12 mm diameter dowels ................................................................. 56

3.2 Non-metallic plate specimen configurations ......................................................... 61

3.3 Test specimen configurations ................................................................. 63

4.1 Moisture content and dry density of glulam and LVL .......................... 69

4.2 Embedment strength and foundation modulus of Douglas fir glulam 72

4.3 GFRP-DVW bearing stiffness ................................................................. 73

4.4 Span to depth ratios tested for graphical interpretation of E & G .......................... 76

4.5 $M_{eff}$ values for GFRP dowels ................................................................. 90

5.1 Test specimen configurations (dowel diameter, d = 12 mm for all tests) ................................................................. 96

5.2 Results summary for parallel to grain tests ................................................................. 99

5.3 Test specimen configurations ................................................................. 109

5.4 Results summary for perpendicular to grain tests ................................................................. 111

6.1 Test specimen descriptions ................................................................. 118

6.2 Results summary for full scale tests ................................................................. 125

7.1 Material properties used in stiffness analysis ................................................................. 150

7.2 Single dowel stiffness values for the determination of slip modulus, $k_{eq}$ 152

7.3 Connection stiffness model mean average results ................................................................. 152

7.4 Connection stiffness model fifth percentile results ................................................................. 152

8.1 Comparison of predicted connection strength with experimental values 162

8.2 Experimentally determined dowel spacings that ensure yield of GFRP-DVW connections prior to brittle timber failure parallel to grain ................................................................. 170

8.3 Comparison of predicted connection strength with experimental values 175
B.1 Test specimen configurations parallel to grain (dowel diameter, \( d = 12 \text{ mm} \) for all tests) ............................ 204
B.2 Test specimen configurations perpendicular to grain (dowel diameter, \( d = 12 \text{ mm} \) for all tests) ............................ 204
B.3 Experimental data for individual specimens ....................... 205
### List of Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Area of cross section</td>
</tr>
<tr>
<td>$a_1$</td>
<td>Fasteners spacing parallel to grain</td>
</tr>
<tr>
<td>$a_2$</td>
<td>Fasteners spacing perpendicular to grain</td>
</tr>
<tr>
<td>$a_{3,t}$</td>
<td>Loaded end distance</td>
</tr>
<tr>
<td>$a_{3,c}$</td>
<td>Unloaded end distance</td>
</tr>
<tr>
<td>$a_{4,t}$</td>
<td>Loaded edge distance</td>
</tr>
<tr>
<td>$a_{4,c}$</td>
<td>Unloaded edge distance</td>
</tr>
<tr>
<td>$b$</td>
<td>Total width of timber loaded in shear</td>
</tr>
<tr>
<td>$d$</td>
<td>Diameter</td>
</tr>
<tr>
<td>$E$</td>
<td>Flexural modulus</td>
</tr>
<tr>
<td>$e$</td>
<td>Eccentricity</td>
</tr>
<tr>
<td>$F_{\text{max}}$</td>
<td>Maximum recorded load resistance</td>
</tr>
<tr>
<td>$F_{90}$</td>
<td>Perpendicular to grain splitting capacity</td>
</tr>
<tr>
<td>$f_h$</td>
<td>Timber embedment resistance</td>
</tr>
<tr>
<td>$f_{v,k}$</td>
<td>Characteristic timber shear strength</td>
</tr>
<tr>
<td>$g$</td>
<td>Shear modulus</td>
</tr>
<tr>
<td>$G_c$</td>
<td>Fracture energy release rate</td>
</tr>
<tr>
<td>$h$</td>
<td>Member depth</td>
</tr>
<tr>
<td>$h_e$</td>
<td>Distance between loaded edge and most distant fastener</td>
</tr>
<tr>
<td>$I$</td>
<td>Second moment of area</td>
</tr>
<tr>
<td>$k$</td>
<td>Foundation modulus</td>
</tr>
<tr>
<td>$k_{\text{def}}$</td>
<td>Creep deformation factor</td>
</tr>
<tr>
<td>$k_{\text{mod}}$</td>
<td>Strength modification factor for load duration and moisture content</td>
</tr>
<tr>
<td>$k_{\text{ser}}$</td>
<td>Instantaneous slip modulus</td>
</tr>
<tr>
<td>$l$</td>
<td>Span length</td>
</tr>
<tr>
<td>$l_{\text{per}}$</td>
<td>Effective perimeter length of partial shear plug</td>
</tr>
<tr>
<td>$M$</td>
<td>Moment</td>
</tr>
<tr>
<td>$M_{\text{eff}}$</td>
<td>Effective plastic hinge capacity of GFRP dowel</td>
</tr>
<tr>
<td>$M_y$</td>
<td>Yield moment</td>
</tr>
<tr>
<td>$N$</td>
<td>Number of fasteners</td>
</tr>
<tr>
<td>$n_{\text{eff}}$</td>
<td>Effective number of fasteners in a row parallel to grain</td>
</tr>
<tr>
<td>$P$</td>
<td>Applied force</td>
</tr>
</tbody>
</table>
\( P_p \) Plastic load resistance
\( P_y \) Applied yield force
\( p_l \) Shear plug length for line of dowels
\( R \) Load resistance
\( R_{par} \) Load resistance parallel to grain
\( R_{perp} \) Load resistance perpendicular to grain
\( R_{bearing} \) Plastic bearing capacity of connection
\( R_{row \ shear} \) Row shear out capacity of multiple fasteners in line
\( R_{group \ tear-out} \) Block shear out capacity of multiple fastener group
\( R_{splitting} \) Splitting capacity of timber member
\( r \) Radius of dowel
\( s \) Deflection
\( s_b \) Deflection due to bending effect
\( s_v \) Deflection due to shear effect
\( t \) Thickness of timber side member
\( V \) Shear force

\( \alpha \) Angle of load to grain
\( \beta \) Constant used in the derivation of beam on elastic foundation solution
\( \delta \) Displacement
\( \gamma \) Shear strain
\( \gamma_m \) Partial factor for variation in material properties
\( \rho_m \) Mean density
\( \tau \) Shear stress
\( \theta \) Rotation at plastic hinge
\( \zeta \) Constant used in the derivation of beam on elastic foundation solution
\( \xi \) Constant used in the derivation of beam on elastic foundation solution
Chapter 1

Introduction

The use of structural timber in modern construction is increasingly commissioned as a response to concerns relating to the challenges of global climate change, depletion of fossil energy reserves, and an increasing rate of biodiversity loss. In regions where sustainable forestry practices are upheld timber offers a unique means of construction that simultaneously addresses the three challenges highlighted above. Responsibly managed forests and woodlands help to support biodiversity conservation, provide a significant global carbon sink, and produce structural materials with minimal use of fossil fuels (CEI-Bois, 2006). This thesis aims to broaden the knowledge base from which timber architecture is derived in order to further the use of this valuable resource.

The connections used in any structural system play a significant role in defining the form, proportion, and overall aesthetic of a building and this is particularly true of timber structures where member depths can often be determined by the size of a connection. Therefore within a timber structure the connections used to join structural elements are responsible for the successful execution of a particular design solution and also for a significant proportion of the structure’s cost. In this sense careful design of connections is required to keep financial costs to a minimum whilst also satisfying the architectural and engineering demands of a structure. In Europe codified connection design guidance is provided through the provisions of Eurocode 5 (EC5), which replaced the British Standard, BS 5268, in 2010. Unlike BS 5268, which was based on permissible stress design, Eurocode 5 is a limit states code where the limits are associated with either serviceability or ultimate loading of the structure. This shift towards a more flexible approach to design is a significant and positive step for timber engineering but, like BS 5268 before it, EC5 still only provides guidance for mechanical connections made from metallic components.

The lack of design guidance for alternative connector materials is a reflection of the ubiquitous nature of metal components within timber structures. Examples
of connections using non-metallic mechanical connections are principally only found in the field of traditional carpentry where generations of precedence and rules of thumb provide permissible design rules, often on the basis of little engineering rationale. However, the use of metallic components for the connections of a timber structure introduces problems associated with corrosion resistance, fire performance, thermal bridging potential and significant weight and cost considerations. In many situations these problems could be alleviated through the use of alternative non-metallic components. However, accepted practice and design guidance for mechanical connections of this type is restricted to the field of traditional carpentry where the lack of engineering analysis tools does not allow for significant innovation in design.

Currently, where a contemporary non corrosive connection type is required, the options available to the designer are to use prefabricated glued connections or mechanical stainless or galvanised steel fasteners. However, where glued connections provide corrosion resistance and improved fire performance (IStructE and TRADA, 2007) they require careful off-site fabrication and have restricted capacity perpendicular to grain. Metallic solutions have the significant advantage of being suitable for on-site assembly but require careful detailing for fire and costs are high when stainless steel fasteners are introduced. Evidently both glued and metallic systems have individual advantages but a non-metallic mechanical system can incorporate many of these into a single mainstream solution.

In order to address this gap in the knowledge of timber connection design this study aims to develop a means for providing a mechanical, metal free, connection system for contemporary timber structures. To avoid excessive offsite fabrication and to facilitate onsite erection of a structure an unglued connection system was considered essential. Additionally it was anticipated that the solution should be widely applicable to methods of fabrication and design currently in use. These considerations are discussed in greater detail in sections 1.2 and 1.3 within this chapter.

1.1 Timber connections

To outline the context of this thesis a short review of the current practice for making timber connections is included here. These methods range from the continued use of traditional carpentry techniques to the use of bonded in fibre reinforced rods. Within this range is a wealth of different metallic connection systems which have developed alongside innovations in the production of structural timber products.
1.1.1 Traditional carpentry connections

Present day options for unglued non-metallic connections are predominantly focused in the field of traditional carpentry. Many of these connections employ the use of timber peg connectors and can be found all over the world; notably green oak frames are used in the UK (Figure 1-1). The advantages of using traditional green oak connections are most regularly associated with corrosion resistance though the connections also provide sound fire resistance (Ross et al., 2007). Like many hardwoods oak has a high content of acidic extractives which attack ferrous materials and the use of traditional oak pegs removes this incompatibility of materials. The compatibility of materials in this sense is also a problem for the contemporary use of hardwood glulam such as sweet chestnut. In these instances galvanised or stainless steel components have to be used to avoid corrosion. In fire the use of timber pegs has been shown to not reduce the period of fire resistance but for the use of bolted or screwed joints with exposed heads conduction of heat into the centre of the timber and the loss of strength of the metallic fastener has to be protected against through the use of timber plugs (Ross et al., 2007). Additional advantages of green oak framing techniques are associated with:

- The accommodation of distortion due to shrinkage and movement by oak pegs which are less likely to induce splits in members than stiff metallic fasteners.
- The opportunity to fabricate frames entirely in house – negating the need to further subcontract.
- The suitability of the framing method for onsite fabrication.
In recent years a resurgence in the use of green oak and other similar carpentry techniques has been fueled by interest in using wholly natural structures and materials with low energy use in material conversion (Shanks, 2005). However, in spite of this wide use, framing techniques have remained constrained to traditional methods, where precedence provides the dominant basis for design. The continued practice of design by precedent, combined with a low level of knowledge surrounding the engineering rational and design of many carpentry connections, are likely reasons for timber dowelled connections remaining in the realm of traditional carpentry. The impact of this restriction is that traditional methods form a niche construction sector with few case studies of their application to modern architecture.

Prior to the industrial revolution, traditional methods of timber frame construction were widespread throughout the UK and the rest of Europe. European timber structures, which are typically recognised as being of a traditional construction form, date back to the medieval period (Ross et al., 2007). In Japan the vast availability of wood was the prerequisite for providing a building resource which has been of primary use since the Jomon Period (3500-300 BCE) (Herzog et al., 2004). The worldwide history of using timber to make structures means that there are many different variations of traditional carpentry connections. However, the general form of framing connections is frequently very similar. Commonly, variations are observed in the form of timber species, peg type, or geometry. In general traditional timber frame connections can be characterised in the following way (Ross et al., 2007):

- Connections are made up entirely of timber
- Mechanical connection, through bearing and interlock, provides the principal means of transferring load.
- Seasoned timber pegs are used to tighten and lock the connections. There are many variants of pegs, each of which has an influence on connection capacity.

The pegged mortice and tenon is one of the most commonly observed connections within traditional timber frames (Harris, 1978) and much research has been completed to provide means by which it can analysed (Brungraber, 1985; Schmidt and Mackay, 1997; Shanks, 2005). The general form of the connection is shown in Figure 1-2 where the constituent elements are labelled. Variations in the form of the mortice and tenon connection are not that pronounced but variations in timber species and pegs are seen geographically around the world. In the UK green oak is generally used for the framing members with seasoned cleft oak pegs, whilst Douglas fir framing members and turned white oak pegs are common to US
Japan framing uses square, seasoned, Japanese oak pegs within cedar framing members. In Taiwan round pegs are also used (Shanks et al., 2008).

The mortice and tenon connection is often conservatively assumed to transfer only compression or shear loads in analysis unless a connection is specifically designed to carry tension (Yeomans, 2003). This is partly a reflection of the application of engineering methods after construction for the purpose of restoration where the tensile resistance of the peg can no longer be relied upon. To a greater extent however this is a reflection of the structural systems used in traditional carpentry. Frames are typically designed as compressive structures due to the increased reliability of a structure which predominantly transfers loads through direct bearing of structural elements and not directly through the capacity of pegs. This could be attributed to the lack of engineering analysis methods during the development of the craft and thus an inability to reliably predict the tensile capacity of a pegged connection. However, although it is considered as a compression joint Shanks (2005) demonstrated that the tensile capacity of the mortice and tenon is significant. In particular an improvement in the strength and stiffness of braced frames was reported when a tension brace was included.

In the fabrication of a mortice and tenon connection an offset, or ‘draw-bore’, is typically used between the hole in the mortice wall and the tenon in order for the connection to be pulled tight as the peg is driven through. The draw-bore ensures that the faces of the connection are pulled tight against each other and provides a form of prestress. The prestress provided by the draw-bore then helps to keep a connection tight after cross grain shrinkage of the initially green mortice member. If this draw bore was excluded then the peg would begin to carry a proportion of
CHAPTER 1. INTRODUCTION

the compressive load upon shrinkage of the framing member. Therefore, shrinkage of a connection without draw-bore could cause the connection to open up and the frame stiffness would be adversely affected.

The use of prescriptive design rules derived for the use of heavy timber members coupled with considerations relating to shrinkage and distortion means that traditional carpentry provides a vernacular architectural style. Contemporary architecture often demands structures of high dimensional tolerance with minimal movement in order to accommodate brittle finishes such as glass. Additionally there is often a desire for clear uninterrupted spans which are now easily realised through the gluing of small section dimensionally stable timber to make glulam beams. Other new engineered products such as CLT (Cross Laminated Timber) can be used to resist racking loads which negates the necessity for braces. The result of using modern engineered timber products is that a contemporary aesthetic can be produced. However, in order for timber dowelled connections to be integrated into this contemporary field, reliable means of analysis must be provided and robustness must be proven. Additionally the section sizes of glulam are not restricted by that of a tree hence more efficient deep narrow beams can be used in timber structures but these would not always be suitable for connections such as the mortice and tenon. New connection methods are therefore required to allow use of non-metallic connections in contemporary applications. For the analysis of pegged mortice and tenon connections Shanks (2005) developed an empirical model for predicting the capacity of traditional connections but this model is not directly applicable to contemporary applications as it is dependent on the use of the traditional geometry of a mortice and tenon and uses empirical values for peg capacity, which are derived from simulated connection tests. Therefore in order to gain the advantages associated with the contemporary use of all-timber connections there is a need for more widely applicable analysis tools and a design solution that is suitable for application in contemporary architecture.

1.1.2 Metallic connections

The move away from traditional all-timber methods and towards the incorporation of metallic elements is reported by Harris (1978) as occurring in the eighteenth and nineteenth centuries when carpenters began to make use of pattern books instead of following the traditional practice of transferring knowledge from master to apprentice. These pattern books showed new and recommended designs which increasingly incorporated the use of bolts and straps in place of the more traditional mortice and tenon connection and so the ubiquitous inclusion of metallic connector components began.
Since timber construction began to move away from traditional methods the use of bolts, screws, and nails for connecting timber members has been commonplace. It was not until the end of the eighteenth century that iron was available in sufficient quantities to be used as a building material in its own right so bolts were also used to make up large mechanically jointed composite timber beams (Muller, 2000). The concept of mechanically jointed timber beams was later followed by the concept of glue lamination in the nineteenth century. The use of glues to join timber is one of the most significant developments in the history of timber engineering. The use of timber was revolutionised. Curved members of any length and optimised cross section could now be made using a simple jig and small cross-section timbers. The work of Otto Hetzer led to the refinement of the gluing technique and provided the basis for its practical application (Muller, 2000). Built in 1860, the Golden Jubilee room in Southampton is commonly acknowledged as being the first structure to be built using glue laminated timber (Figure 1-3) (Booth and Heywood, 1994).

![The Golden Jubilee Room, Southampton](image)

**Figure 1-3: The Golden Jubilee Room, Southampton**

The impact of the first and second world wars were both significant in the development of metallic fasteners in timber engineering. After the first world war there were significant timber shortages and a requirement for efficient use of the remaining resource. This stimulated the uptake and development of connectors such as the toothed dog and split ring connector. After the second world war it was shortages of steel which further drove the use and development of these connectors to provide large timber structures such as the airship hangars built
on the west coast of the US during the war (Yeomans, 1997). In the UK the British
Standard 5268 was originally based largely on the research and experience of
war time construction in Canada, the United States and Britain (Booth and
Reece, 1967). The code was prescriptive in its design guidance for the capacity
of mechanical connections and has since been replaced by Eurocode 5, which has
taken a more analytically based approach to design guidance so that the designer
can take into account variations in material properties.

Today many engineered timber products, such as laminated veneer lumber (LVL),
glulam, and sheet materials such as plywood and orientated strand board (OSB)
are readily available from sustainable sources. The development of steel connectors
has closely followed that of timber itself and has therefore had to accommodate
for these advances in engineered timber products. Greater understanding of load
transfer in timber, and the requirement to carry larger loads between massive
gineered members has produced connectors such as the fin-plate connector, and
in more recent years the multiple shear plate connector (Mischler et al., 2000). The
scale of these connections is often substantial as demonstrated in Figure 1-4.

Steel connectors now dominate the field of modern timber construction and come
in many forms, grades, and alloys. Nails are often used as a low cost form of
connection, as are pressed nail plates. Bolts and dowels are still readily used and
the recent introduction of self drilling dowels has been very successful. Self drilling
dowels are often used in conjunction with thin multiple shear plates to produce
high efficiency, close tolerance connections. However, despite metallic components providing the majority of design solutions in contemporary timber structures the use of non-metallic connectors may also be applicable in many cases.

The reasons suggested for the lack of uptake of non-metallic materials such as timber dowels are the absence of codified guidance for the analysis of non-metallic materials as well as the architectural restrictions posed by traditional carpentry techniques. Furthermore, the use and development of metallic connectors was often driven through the adversity of post war material shortages and not a full appraisal of possible connection means. These war time experiences of timber construction were then fed directly into design codes which formed the basis of the majority of new timber construction in and around Europe, the US and Canada. It is anticipated that recent research into timber pegged mortice and tenon connections (Shanks, 2005) as well as the use of new materials such as compressed wood and FRP materials will allow a broader approach to be adopted when dealing with appropriate connection design.

1.1.3 Non-metallic connection methods

Mechanical timber connections made without the use of metallic elements are rare, except where traditional carpentry techniques are employed. Nonetheless, the development and increased use of glues for joining timber has produced two commonly accepted, modern non-metallic connection techniques. The first is the use of bonded-in fibre reinforced polymer (FRP) rods and plates using high strength polymeric resins and the second is the use of full section finger joints. The method of bonding-in FRP rods has been most successful in the restoration and repair of historic timber structures (Figure 1-5). New-build use of the technique is generally restricted to prefabricated systems, as the use of many glues on site is not practical.

Restoration work generally uses ambient cure epoxy adhesives which are thixotropic so that repairs into the underside or side of timber members is possible (Ansell et al., 2010). Quality control is very hard on site due to the difficulty in the application of resins, joint assembly and ensuring the timber is of suitable moisture content to be glued. In addition to issues of on site fabrication, the long term performance of bonded in rod connections is also a concern in aggressive environments. It has been found that the effectiveness of the resin bond is reduced when the timber has a high moisture content, or if there is a cyclic variation in moisture content (Bainbridge and Mettem, 1998).

Although bonded in rod connections can be designed to provide a stiff, highly efficient connection, finger jointing at the interface between two members could be considered a more favourable technique in instances such as moment resisting
CHAPTER 1. INTRODUCTION

Figure 1-5: A joist repair using bonded in GFRP rods (Acknowledged to Rotafix)

portal frame haunch joints. Finger jointing requires no additional connectors and can provide a strength equal to that of the timber grade strength (Bainbridge and Mettem, 1998). However, as for bonded in rod and plate connections, there is still a necessity for off-site fabrication, and transportation of large prefabricated elements is not always feasible. Glued haunch joints of the types described above are often used in large portal buildings in agricultural environments or for the storage of grit salt. In these cases the exclusion of metallic components in the haunch connections removes the problem of corrosion associated with the building use but metallic elements are still required to mechanically connect the portal elements at the eaves and foundations. In these instances a mechanical metal-free connection would allow the structure to be wholly resistant to corrosion, which could potentially improve the lifespan of the structure and reduce the necessity for inspection and maintenance of the structure.

1.2 Rationale

Recognised methods of providing non-metallic connections within timber structures are specialist and for many cases this makes them inapplicable to mainstream contemporary applications. Within the market for non-metallic connections, bonded-in rod technology has been found to be best suited to remedial work in historic structures where few joints are made and the application of on-site adhesives is made by a specialist. Meanwhile, although finger jointing is a successful technique for providing high strength non-metallic connections, it is constrained heavily by the necessity for prefabrication.

There is significant potential for a non-metallic connection method suitable for on and off-site site applications, which is free from the use of adhesives. Unlike metallic connectors, the use of non-metallic elements to connect timber members
on site is not an issue in terms of staining of timber during frame erection and connector degradation through material incompatibility is also avoided. Within the scope of non-metallic dowelled connections there is significant precedent for the use of timber dowel connections in traditional joinery but a lack of contemporary application, due to the lack of analysis tools and design guidance in this area. Additionally the high strength of contemporary materials, such as fibre reinforced pultrusions and compressed timber products, offers further opportunities to develop new mechanical connection systems without the use of metallic components.

Several in depth studies into traditional pegged mortice and tenon joints (Brungraber, 1985; Schmidt and Mackay, 1997; Shanks, 2005) have been completed on the basis of an increased requirement for analysis of traditional structures, old and new. Outside of the field of traditional all-timber connections there exists a significant gap in knowledge regarding the use of timber dowels in a contemporary context. In his thesis on traditional oak frame connections, Shanks (2005) states that research is required to develop innovative joints based on all-timber solutions. This is particularly true for innovation in the use of flitch plate connections and softwood structural members.

The use of new strong materials such as fibre reinforced plastics for unglued connections is still in the early stages of development. Nonetheless, the feasibility of new strong materials in both non-metallic and metallic applications has been explored and significant precedence exists. This is discussed in detail within Chapter 2 through the review of the work on the use of GFRP dowels and compressed wood by Leijten (1998), Drake (2003) and Pedersen (2002).

1.3 Scope of thesis

In order to effectively bridge the gap between contemporary glued connections and metallic connections certain constraints were defined prior to investigating the development of a mechanical non-metallic connection. The first was that the system must not use glue as this causes significant problems for on-site assembly. Secondly the solution must be cost effective if it is to compete with the use of stainless steel and glued solutions. Finally ease of fabrication must be considered. For these reasons it was concluded that the most appropriate focus for this research project was the development of a multiple dowelled, slot-in-plate type connection such as that shown in Figure 1-6.

Metallic dowelled, flitch plate type connections are widely used by the timber construction industry and so represent minimum complexity in terms of adapting manufacture and fabrication to the use of non-metallic materials. The slots in
timber members are cut easily and efficiently using a narrow plunge mortise tool as shown in Figure 1-7. High dimensional tolerance holes for dowels are conventionally drilled prior to site assembly in order to accurately align the steel plates. Future development of the use of non-metallic plates may allow in-situ drilling of plates, providing the opportunity for reduced slip at structural connections. In addition to these points there is no requirement for glue within the assembly and the method of connecting structures in this way is proven through the use of dowel-plate type connection in modern structures.

Further to focussing on a dowel plate type connection, the main body of investigation reported within this thesis has been restricted to the development of a connection system that uses only one specific dowel material and one specific plate material. Several non-metallic materials with potential to provide structural connections are identified in Chapter 3 and based upon the appraisal of these materials, the rationale for defining the scope of this research is presented.
1.4 Objectives of study

The overriding aim of this study was to innovate and develop a means for providing a mechanical, non-metallic, timber connection for mainstream, contemporary, applications. This aim was satisfied through completion of the following objectives:

- Non-metallic materials have been investigated and critically assessed through experimental study and those best suited for use in contemporary timber connections have been identified.

- Strength and stiffness properties of the selected materials have been determined experimentally and are reported for analysis purposes.

- An experimental study of the chosen connection system was completed to develop understanding of connection load response in pull-out, connection failure modes and the influence of dowel spacing. The study was completed for parallel and perpendicular to grain load orientations.

- Full scale tests parallel to grain have been completed to investigate the newly proposed dowel spacing rules and to compare the connection system with a metallic counterpart.
CHAPTER 1. INTRODUCTION

- Predictive models have been developed and validated for the analysis of the strength and stiffness of the connection system.

- The connection system has been reviewed and discussed in the context of a real life application at the V&A museum in London.

1.5 Layout of thesis

This thesis is comprised of ten chapters and three appendices. In Chapter 2 the thesis begins by reviewing research to date and current design practices for dowel-plate type connections. Comparative testing and appraisal of non-metallic connection components is reported in Chapter 3. This chapter also defines the focus of the thesis through the selection of the most appropriate materials for further investigation. Chapter 4 presents the results of material characterisation of the materials taken forward from Chapter 3. Connection tests in pull-out and the subsequent development of the connection is reported in Chapter 5. Full scale connection tests are reported in Chapter 6 and connections made with metallic fasteners are also tested and reported for comparison. Methods for analysing the stiffness and strength of the connection system are presented and validated in Chapters 7 and 8 respectively. Through a collaboration with the Rural Studio the connection technique developed throughout this thesis was used in the ‘Woodshed’ timber pavilion for an exhibition at the Victoria and Albert Museum, London. Chapter 9 discusses design and fabrication recommendations for the connection system in the context of the Rural Studio pavilion. Finally, Chapter 10 concludes the thesis and outlines recommendations for future work. Appendix A presents an analytic derivation, which accompanies the connection stiffness analysis, Appendix B reports additional experimental data and Appendix C reports the dissemination of this work to date.
Chapter 2

Literature review

The literature reviewed in this chapter is principally concerned with the use and analysis of timber connections made with dowel type fasteners. Literature that reports the use of non-metallic connection components is limited and proposed analysis is often made in reference to accepted methods for metal fasteners. There is a great depth of research into the interaction and failure behaviour of metallic dowel type fasteners, but it cannot be assumed that this is generally applicable to the use of non-metallic materials.

This chapter begins by reviewing the current practice for the analysis and design of timber connections made with metallic dowel type fasteners and a slot-in plate. This is to provide context to the literature on non-metallic materials in addition to providing a basis for adoption, modification, and appraisal of the analysis methods proposed in this thesis. The section addresses three principal areas of concern for the successful design of a timber connection:

- Connection yield capacity
- Brittle splitting or timber shear failure
- Deformation of a connection

The latter sections describe the use of non-metallic materials in timber connections. Previous research into the structural performance of timber dowelled connections is presented, followed by research into the application of higher strength FRP and compressed wood materials.
2.1 Analysis and design of metallic dowel and plate connections

The design of timber connections made with metallic dowel-type fasteners addresses two distinct modes of failure. The failure modes considered are those which are ductile bearing failures and those which are brittle fracture or shear failures. The failure mode with the lowest estimated capacity will govern in design.

Ductile failure modes are described by the European Yield Model (EYM), an analytical model originally proposed in 1949 by Johansen. The EYM is a force equilibrium model, which balances the applied load with the embedment resistance of the connected structural member and the bending resistance of the fastener. Detailed discussion of the mechanics behind the EYM is given for dowel plate type connections in section 2.1.1.

The inherent assumption of the EYM is that a connection will fail in one of several ductile modes. In this sense estimated connection resistance for multiple dowels is directly proportional to the number of fasteners. However, in tests on multiple fastener connections, brittle fracture or shear failure modes are often observed at loads lower than those predicted by the EYM (Quenneville and Mohammad (2000) & Leijten and Van der Put (2004)). Eurocode 5 guidance on the subject of brittle connection failure is discussed in conjunction with wider research in section 2.1.2.

2.1.1 European yield model

Conventional dowel yield theory in the form of the EYM provides a simple and reliable means for calculating the load bearing capacity of metallic dowel type connections. The yield model was derived from observations made in the experimental realisation of the possible failure modes of metallic dowel type fasteners in single and double shear connections. The definition of failure modes was initially reported by Johansen (1949) who derived the first expressions for connection capacity using material properties and joint geometry. Johansen’s model was developed further by Larsen (1973) to incorporate the capacity for timber members of different embedment strengths to be evaluated.

Two defined effects characterise the behaviour of a metallic dowel within a connection (Johansen, 1949). The first is the dowel effect of the fastener, which is dependent on its resistance to bending and the resistance of the timber to crushing. The second is the tensile ‘rope’ effect of the fastener, which is dependent on fastener end restraint and friction between the fastener and the surrounding timber. The yield model is based upon the dowel effect. The tensile ‘rope’ effect is accounted for through an appropriate addition of strength to the yield model expressions.
To describe the failure modes of connections made in single or double shear with varying connecting materials two general cases of built-in dowel failure are presented by Larsen (1973) (Figure 2-1). Case 1 is the bearing failure of the timber under a stiff, stocky dowel and case 2 is the combined bending failure of the dowel in conjunction with bearing of the timber. In order to formulate the expressions for these two cases, material assumptions are made for the failure behaviour of the metallic dowel and the bearing timber. Figure 2-2 shows the typical moment rotation response of mild steel dowels where $M$ is the bending moment and $\theta$ is the measure of bending deformation. The bending behaviour of steel as shown in Figure 2-2 is typically approximated by assuming that the material is either perfectly elastic-plastic or more simply that it is stiff-plastic. In the formation of general expressions for case 1 and 2 failure modes Larsen (1973) adopts the latter approximation.

Embedment strength of timber under a metallic dowel can be determined using the general setup depicted in Figure 2-3. The corresponding load-displacement response of dowel bearing is also shown (where load, $P$ is force per unit depth of timber). The force per unit depth of timber is expressed as $f_h d$, where $d$ is the diameter of the fastener and $f_h$ is the mean stress under the dowel. $f_h$ is termed the embedment strength of the timber and carries the assumption that pressure...
beneath the dowel is uniformly distributed. In line with the material assumptions made for the bending behaviour of metallic dowels, bearing capacity is determined on the basis that the embedment response of a dowel in timber is stiff-plastic (Figure 2-3).

Case 1 connection failure, as presented by Larsen (1973), is shown in Figure 2-1 and addresses failure through rotation of the dowel about a point x within the thickness of the timber. It is assumed that the dowel is so stiff that only negligible elastic deformations occur. The load on the dowel is shown and is based upon the assumption of stiff-plastic embedment response. The internal shear and bending loads on the dowel are also shown. Simultaneous equations describing equilibrium of vertical forces and moments are solved to give the expression for $P_y$ shown below:
\[ P_y = \left[ \sqrt{(t + 2e)^2 + t^2} - (t + 2e) \right] f_h d \]  

[Case 1]  

where:  
\( P_y \) is the yield capacity of the connection when loaded at a lever arm of length \( e \)  
\( t \) is the thickness of the timber side member  
\( e \) is the eccentricity of load \( P_y \) on the dowel  
\( f_h \) is the embedment strength of the timber  
\( d \) is the dowel diameter

Considering the shear force shown in Figure 2-1 [case 1] the maximum moment in the dowel can be calculated as \( M_{max} = x^2 f_h d \). Where \( M_{max} \leq M_y \) equation 2.1 applies and \( M_y \) is the moment yield capacity of the dowel. However, where the yield capacity of the dowel is exceeded the dowel will form a plastic hinge along its length to give the failure mode shown in Figure 2-1 [case 2]. This mode of failure is the second generalised form of metallic dowel failure in timber. Ignoring the effects of friction along the dowel the yield resistance of the dowel connection can be calculated using the expression below:

\[ P_y = \left[ \sqrt{e^2 + \frac{2M_y}{f_h d} - e} \right] f_h d \]  

[Case 2]  

These two general cases of dowel failure form the basis of all EYM expressions for single and double shear dowel type connections in timber. Following the focus of this thesis, the failure modes for dowel-plate type connections are presented below with their respective EYM expressions (equations 2.9 and 2.10). The characteristic capacity of a connection (per shear plane) is taken as the minimum value obtained from the evaluation of the expressions for the three failure modes (I, II & III).

\[ R = f_h t d \]  

(Mode I)  

\[ R = f_h t d \left[ \sqrt{2 + \frac{4M_y}{f_h t d^2}} - 1 \right] \]  

(Mode II)  

\[ R = 2 \sqrt{M_y f_h d} \]  

(Mode III)  

where:  
\( R \) is the characteristic load carrying capacity per shear plane per fastener
$f_h$ is the characteristic embedment strength of the timber member

t is the thickness of the timber side member

d is the dowel diameter

$M_y$ is the characteristic fastener yield moment

Failure mode I is characterised by the plastic embedment of the dowel into the timber. Failure of this type signifies timber of low embedment resistance and a stiff, often stocky, dowel. In modes II and III timber bearing failure occurs simultaneously with the formation of plastic dowel yield points. These failure modes make the most efficient use of the fastener strength and are generally attributed to the use of slender fasteners.

2.1.2 Brittle failure of structural members

Mechanically fastened timber joints often fail in one of the brittle modes shown in Figure 2-5 when tested in a laboratory (Quenneville, 2009). In order to avoid these brittle modes of failure, rules based on laboratory connection tests and the experience of craftspeople are specified by most design codes (Schmid and Blass, 2002). These prescribed spacing rules are typically given as minimum dimensions and address the provision of end and edge distances, and the spacings between fasteners.

For traditional green oak carpentry connections, such as the mortice and tenon, a tenon end distance of only three times the dowel diameter is common (Shanks, 2005). This end distance is intended to prevent failure through tenon relish (plug shear) and the relatively small end distance reflects the strength capacity of the oak dowel and connecting tenon. For metallic fasteners, which are much stronger
and stiffer than timber dowels used in traditional carpentry, larger spacing rules are
prescribed. This is of significance for the development of non-metallic connections
where the strength and stiffness of dowel materials is typically lower than that of
metallic dowels.

Within the context of mitigating brittle connection failure EC5 minimum spacing
rules are included in table 2.1 for reference throughout this thesis. The terminology
used by EC5 is described in Figure 2-6 and is adopted throughout this thesis for
clarity and continuity.

The spacing rules in Table 2.1 are given with the intention of ensuring ductile
connection failure. However, where multiple fasteners are used brittle failure can
still occur at loads lower than those predicted by the EYM. The code states that
the load carrying capacity of a multiple fastener connection, consisting of fasteners
of the same type and dimension may be lower than the summation of the individual
load carrying capacities for each fastener (BS EN 1995, 2004). This must be
taken into account for components of load parallel and perpendicular to the grain
direction.
Parallel to the grain EC5 provides a reduction factor, $n_{eff}$, to determine an effective number of multiple fasteners from which the EYM capacity of a single fastener can be multiplied. This factor varies depending on the in-line spacing of fasteners, fastener diameter, and the number of bolts in-line. From inspection of the expression for $n_{eff}$ in EC5 it is evident that for a connection loaded parallel to grain through stiff metallic fasteners the effectiveness of the connection is improved through the use of large dowel spacings and small dowel diameters. In European timber construction there has been a noticeable shift towards the use of many small diameter fasteners instead of fewer large diameter fasteners. In this way ductile failure of the connections is more likely. The provision of $n_{eff}$ in EC5 is in many ways misleading as it does not give any insight into the likely failure mode of a connection with a low factor of effectiveness.

Greater insight into the mechanics and modes of connection failure parallel to grain can be gained from the work of Quenneville and Mohammad (2000). A large series of tests was completed to develop understanding of the failure modes and strength of steel bolted timber connections. Row shear out, group tear out, and splitting failure modes (Figure 2-5) were all observed but the main focus of the work was on the former two modes. Analysis of the results for these failure modes showed that
the longitudinal shear stress at failure is related to a factor which is a function of
the smaller distance (end/dowel spacing) and the member thickness. This observed
‘triggering’ of brittle failure suggests that when brittle shear failure governs there is
no structural advantage to having different end distance and dowel spacing values
since the smaller of the two dimensions would trigger failure (Quenneville and
Mohammad, 2000).

Splitting of timber when loaded perpendicular to grain poses particular restrictions
due to the comparatively low perpendicular to grain strength of timber compared
to the axial strength. However, connections at truss nodes and flitched beams mean
that it is difficult to avoid such loads. Much research has been completed on the
subject of perpendicular to grain tension failure of timber members (Foschi, 1973;
Ehlbeck et al., 1989; Van der Put and Leijten, 2000; Quenneville and Mohammad,
2001; Ballerini and Rizzi, 2007). Of particular note is the linear elastic fracture
mechanics model, which was developed by Van der Put and Leijten (2000) and
now forms the basis for design in EC5 (Jensen, 2005). The application of a fracture
mechanics model was originally proposed for evaluating the splitting of beams with
notches (Blass et al., 1995) and was later applied to the phenomenon of splitting
in connections (Van der Put and Leijten, 2000; Leijten and Van der Put, 2004).

The general energy release case depicted in Figure 2-7 forms the fundamental basis
for the fracture mechanics model used to describe perpendicular to grain crack
propagation in notched beams. Here a beam with a nominal longitudinal crack is
considered (Blass et al., 1995). The potential energy of the lower section of the
cracked beam under load, F, can be found and expressed in terms of the section
properties and the deflection of the section at the point of applied force. The
change in this potential energy during a small propagation of the crack tip can
then be obtained through differentiation. The resultant decrease in potential energy
from crack propagation corresponds to a positive energy release and simultaneous
increase of the fractured area. This energy release per unit of fracture area is
termed G (Griffith’s coefficient or strain energy release rate). At the point that
the load upon the beam is sufficiently large to cause propagation of the crack, G
has reached its critical value and is termed \( G_c \). The fracture load \( F_c \) can then be
expressed in terms of this fracture coefficient \( G_c \).

Application of the general fracture mechanics approach, to the specific geometry
of a split caused by a connection, provides the expression below for perpendicular
to grain splitting (Van der Put and Leijten, 2000). All corresponding geometry is
shown in Figure 2-7.
\[ \frac{V}{bαh} = \frac{\sqrt{\frac{GGc}{h}}}{\sqrt{0.6\alpha(1-\alpha)}} \]  

(2.6)

where:

\[ α = \frac{h_e}{h} \]

\( V \) is the maximum shear force on either side of the connection
\( b \) is the total thickness of timber loaded in shear
\( h \) is the timber member depth
\( h_e \) is the distance between loaded edge and centre of most distant fastener (mm)
\( G \) is shear modulus
\( G_c \) is the critical strain energy release rate

(a) General case of perpendicular to grain splitting
(b) Perpendicular to grain split caused by connection

Figure 2-7: Perpendicular to grain splitting modes

In order to derive a general design formula from equation 2.6 Leijten and Van der Put (2004) calibrated the unknown fracture parameter \( \sqrt{G/G_c} \) against published test results. To facilitate the calibration four different modes of failure were characterised, two of which result in splitting failure. These failure modes are illustrated in Figure 2-8 and the different modes of failure are characterised below.

A – The connection is much stronger than the splitting strength and embedment stresses under the fastener will be low. The connection is over designed.

B – Connection strength equals the splitting strength, embedment stresses are high. This is considered an optimally designed connection.

C – The connection causes splitting only after significant slip due to high embedment stresses and hardening of the timber after yield. This is characterised as an under designed connection.

D – This connection is under designed and no splitting will occur.
The characterisation of connection failure in this way is analogous to reinforced concrete design. The tensile reinforcement within steel reinforced concrete members must be under-designed to ensure the beam yields under ultimate loads. Over-design of the steel results in a brittle failure caused by concrete crushing. In the same way the capacity of mechanical fasteners in a timber connection should be under-designed to ensure a degree of connection ductility prior to ultimate splitting failure.

It was reported that high values for the apparent fracture parameter were found for type A connection failures and low values for type C (Leijten and Van der Put, 2004). This is because for type A failure the stresses that cause crack propagation develop over the full thickness of the timber member whilst for type C cracks will gradually form outwards from the connection interface, as illustrated in Figure 2-9. Therefore for design purposes a lower bound approach was adopted and the apparent fracture parameter relating to failure Type C was incorporated into equation 2.6. This is of significance for non metallic dowel materials as they are weaker than metallic counterparts and are therefore likely to fall into this lower bound category where brittle failure can only occur after significant fastener deformation and embedment.

For reference, in Chapter 5, the EC5 design expression (equation 2.7) for brittle failure is included here. The expression has been derived exclusively for softwoods and no guidance is provided for connections made in hardwoods. It is suggested
Figure 2-9: Type A (left) and Type C (right) failure modes

in the IStructE manual for the design of timber building structures (IStructE and TRADA, 2007) that a conservative enhancement of 20% may be applied to the splitting resistance. This guidance is informed by the BRE report ‘The strength properties of timber’ (Lavers, 1967). However, it is recommended that testing is completed for hardwood connections as no guidance is given by EC5. This study is focused on the use of softwood timber and so discussion of the fracture mechanics model, given in equation 2.7, is considered valid for results presented later in this thesis. The expression relates to the connection failure mode shown in Figure 2.7(b).

\[ F_{90} = 14b \sqrt{\frac{h_e}{(1 - \frac{h_e}{h})}} (N) \]  

(2.7)

where:
- \( F_{90} \) is the characteristic splitting capacity of the connection (N)
- \( b \) is the loaded member thickness (mm)
- \( h \) is the timber member depth (mm)
- \( h_e \) is the distance between loaded edge and centre of most distant fastener (mm)

### 2.1.3 Connection slip

The slip of timber connections under load must satisfy defined serviceability limit states. These limits are set to mitigate damage of brittle surface finishes and to ensure that the function of a building is not impeded during the service design life. BS EN 1995 (2004) provides methods for calculating the instantaneous slip of a connection based upon empirical relationships derived from many sets of test data (Blass et al., 1995). The long term creep deformation of connections is determined by applying a factor to the initial connection slip value. Final connection slip is therefore given as the sum of these two values.
CHAPTER 2. LITERATURE REVIEW

The application of these EC5 methods for the analysis of non-metallic connections is not considered suitable. This is due to the empirical basis of the expressions for calculating instantaneous slip modulus \( k_{ser} \) and the unknown basis of the creep deformation factor, \( k_{def} \). A similar approach to analysis may be adopted in the future but will require satisfactory test data for the derivation of equivalent empirical expressions.

2.2 GFRP dowel and plate connections

The use of glass fibre reinforced pultruded rod and plate materials was originally proposed by Drake (2003). The work examines the feasibility of the material for this application and the performance of connector groups before proposing a means for calculating connection capacity.

Double shear and dowel-plate type connections made between LVL (laminated veneer lumber) members were tested in tension and in bending. The tests were completed for a range of different fastener diameters and orientations. Initially, testing of eight different dowel group orientations was completed for double shear connections using 6 mm diameter dowels. The side and central timber members were each 39 mm thick and an average peak load resistance per dowel of approximately 6 kN was reported. Bending and tension tests were also completed for connections made with a central GFRP pultruded plate. However, the thickness of the timber members used is not considered to be wholly representative of that likely to be used in practice. The members were 51 mm thick with a central slot of 8 mm to accommodate the flitch plate, which leaves a side member thickness of only 21.5 mm. Nonetheless, Drake (2003) reported that, for the bending tests completed, a capacity of 42-45% of the unjointed section capacity was achieved using four 6 mm and 8 mm diameter dowels at EC5 minimum dowel spacings. Tensile tests were completed for similar connections to those loaded in bending, but only for connections made using four 8 mm diameter dowels. A mean peak connection capacity of 28 kN is reported and, from inspection of the presented load-slip plots, connection yield appears to have occurred at loads between 18-20 kN. No comparative tests using metallic fasteners are presented.

The use of GFRP as a dowel and plate material was examined further by Pedersen (2002) who tested single dowel and plate specimens as part of a larger body of work into metallic dowel type connections. The study reviews and develops methods for predicting connection capacity as the methods proposed by Drake are cited as being unclear mechanically. No further work was completed on further developing the scope in which GFRP can be applied as a non-metallic dowel material.
Pultruded materials are orthotropic and the failure behaviour of GFRP dowels is therefore significantly influenced by shear loads (Pedersen, 2002; Drake, 2003). The prediction of connection strength using the EYM relies upon the use of a plastic bending capacity for the fastener used. However, because GFRP dowels are sensitive to shear loading the characterisation of an appropriate $M_y$ value for use in the EYM was found to be problematic.

Drake and Ansell (2000) and Drake (2003) proposed two different methods of predicting the strength of connections made with GFRP dowels. The first method attempts to adapt the EYM through the use of EC5 expressions for calculating the bending resistance, $M_y$ of a metallic dowel type fastener. However, the mechanics of this first approach are unclear as an experimentally measured cross-fibre dowel shear strength is used as a direct substitution for a value of ultimate tensile strength (Drake and Ansell, 2000). This is not a reliable or mechanically correct method for the prediction of connection strength. The second proposed method focussed on the interlaminar shear strength of the dowels as failure of this type was observed in the dowels at the connection interface (Drake, 2003). In this case an interlaminar shear strength value was substituted into an expression for the maximum shear stress in a beam of solid circular section. However, this expression is derived for a beam subjected to pure bending and therefore takes no account of the significant shear loading of the dowel. Hence the reported strength predictions are all significantly lower than those measured experimentally.

Upon the basis that the strength analysis methods, proposed by Drake, were not reliable, Pedersen (2002) proposed an alternative approach. Pedersen completed a series of three point bending tests on GFRP dowels to give insight into the influence of shear loading on the maximum recorded bending resistance. In the tests, 12 mm diameter dowels were tested under spans ranging from 300 mm to 12.5 mm in increments corresponding to a halving of the span for each test. Larger 16 mm diameter dowels were also tested at spans ranging from 400 mm to 25 mm using the same increment reduction of halving the span each time. Based upon an observed reduction in recorded peak bending resistance Pedersen (2002) proposed the use of a linear dowel failure criterion to describe the relationship between shear and bending resistance in the GFRP dowel. The criterion used is given below:

$$\left(\frac{M}{M_u} + \frac{V}{V_u}\right) = 1$$ (2.8)

where:
- $M$ is the bending resistance provided by the dowel at connection failure
- $M_u$ is the bending strength in pure bending
- $V$ is the shear resistance provided by the dowel at connection failure
$V_u$ is the shear strength in pure shear

By substituting the yield moment of the dowel in the EYM expressions (equations 2.9-2.11 with $M$ from the failure criterion given in equation 2.8 Pedersen (2002) rewrote the EYM as follows:

$$R = f_h t d \ (Mode \ I) \quad (2.9)$$

$$R = f_h t d \left[2 + 4 \frac{M_u}{t^2 df_h} + 4 \frac{M_u}{tV_u} + 4 \left(\frac{M_u}{tV_u}\right)^2 - 1 - 2 \frac{M_u}{tV_u}\right] \ (Mode \ II) \quad (2.10)$$

$$R = \sqrt{4M_u df_h + \left(\frac{M_u df_h}{V_u}\right)^2 - \frac{M_u}{V_u} df_h} \ (Mode \ III) \quad (2.11)$$

To implement the modified EYM expressions a failure surface based on the linear failure criterion was fitted to experimental data. The experimental data were taken from the three point bend tests on GFRP dowels, which were completed for varying spans. The graphical representation of the failure surface reported by Pedersen (2002) is reproduced in Figure 2-10. It can be seen that significant extrapolation was required to derive a value for $V_u$, particularly in the case of the 16 mm diameter dowel. This may not be reliable and by no means verifies the use of a linear failure criterion. Additionally it is considered that the use of three point bend tests is questionable in terms of whether the values determined from these tests are directly applicable to the failure modes seen within the confines of a connection. On the whole the predicted capacities are not in good agreement with the connection yield capacities observed experimentally. The predictions for the yield capacity of connections made with varying timber thickness ranged from an under prediction of 13% for a side member of 21 mm and an over prediction of 32% for a side member thickness of 66 mm.

Pedersen (2002) and Drake (2003) provide limited discussion of connection stiffness. Drake (2003) presents findings from a finite element analysis of two double shear connection configurations. The first configuration was a full three dimensional model of a single dowel connection. The second was a two dimensional model of a connection made with two dowels. No attempt is made to derive a prediction of connection stiffness from the models and the discussion of the model is based upon qualitative comparison between a model made with steel dowels. The model did demonstrate that in the two dowel model the steel dowel closest to the applied load attracted a greater proportion of load than the other. For the GFRP dowel the load was more evenly distributed between the two dowels.
The discussion of connection stiffness by Pedersen (2002) is also limited. Reference to the use of a two dimensional numerical model is made. The analysis is stated to be based on a beam on elasto-plastic foundation model but no detail of the model is presented. A load displacement plot is presented as an output from the analysis but it is unclear from this plot what it was intended to show.

A clear response to the problem of analysing connection stiffness is currently lacking for GFRP dowelled connections. This may be due to difficulties associated with the complexity of finite element models or with the mechanics of beam on elastic foundation analysis.

2.3 Pegged mortice and tenon connections

The mortice and tenon connection plays a vital role in virtually all traditional timber frame structures and provides invaluable precedence for the development of modern timber dowelled connections (Harris, 1978). The form of the mortice and tenon is described in Chapter 1 (Figure 1-2) and common variations encountered are highlighted. In general the mortice and tenon is used as a method of transferring compressive loads within a frame but the connection also has a tensile capacity provided by one or more pegs. This tensile capacity is the dominant focus of research into mortice and tenon connections, both in terms of the influence on frame performance, and in terms of the analysis of joint capacity. The latter of these two areas is of particular relevance to this study. It provides an indication of connection strength as well as insight into the evaluation of timber dowelled connections loaded in double shear.
Prior to the work of Shanks (2005) at the University of Bath, the greater part of research into timber pegged mortice and tenon connections was carried out in North America. Brungraber (1985) tested a series of mortice and tenon connections as part of his thesis on traditional timber structures in Northern America. Bulleit et al. (1999) also published findings from a study into the analysis of traditional timber frames. However, the majority of work has been carried out at the University of Wyoming in the USA.

2.3.1 University of Wyoming research

Several studies have been completed at the University of Wyoming into the structural performance and analysis of North American mortice and tenon connections (Schmidt and Mackay, 1997; Schmidt and Daniels, 1999; Schmidt and Scholl, 2000; Miller et al., 2010). The first of these studies focused on the development of an analytical model for the prediction of mortice and tenon pull out strength. Later studies focussed more specifically on the performance of the whole connection. These subsequent reports presented findings related to design considerations for the connections (Schmidt and Daniels, 1999) and load duration and seasoning effects (Schmidt and Scholl, 2000). Most recently Miller et al. (2010) published a proposed new yield model for timber dowel connections, which addresses a unique dowel failure mode reported in the 1997 study.

The aim of the work completed by Schmidt and Mackay in 1997 was to apply and extend the European Yield Model (EYM) for the strength analysis of mortice and tenon connections. In this sense the model could be adopted into the North American National Design Specification (NDS) for Wood Construction. The four double shear failure modes set out in the European Yield Model (Figure 2-11) for use with metallic fasteners were assumed to apply. However, the experimental connection tests demonstrated that an additional dowel failure mode was common to timber pegged mortice and tenon connections. The proposed additional failure mode is termed mode V, in Figure 2-11 and is characterised by the combined bending and shear failure of the timber dowel (Schmidt and Mackay, 1997). Schmidt and Mackay proposed that this mode should be included in analysis as an extension to the EYM. The diagrams shown in Figure 2-11 are reproduced from the work of Miller et al. (2010) and the terminology is taken from this source for continuity.

For the analysis of mortice and tenon connections Miller et al. (2010) assume that the timber pegs yield plasticly in bending. Subsequently, modulus of rupture values obtained from experimental four point peg bending tests can be used in the EYM expressions relating to modes III and IV (Figure 2-11). The use of peg bending strength in this way is questionable because within a connection the
effective bending resistance of the orthotropic peg will be significantly reduced by shear loads (Shanks, 2005). In the study report by Schmidt and Mackay (1997) only one of these modes is reported to have been observed in full scale connection tests. The mode observed was mode III. A predicted capacity for this connection is not given. Instead it is suggested that this single hinge mode (III) is incompatible with the EYM. The basis for this is uncertain. Although mode III failure can be described diagrammatically in the form of two separate hinges within the thickness of the central member, the total hinge rotation is equal in both cases. It is possible
that the stated incompatibility more likely results from the assumption of plastic hinge formation. In reality significant tensile rupture of the timber fibres occurs at the position of the observed hinge (Shanks, 2005). Therefore beyond this failure the resistance of the dowel to rotation will be severely compromised.

The application of EYM expressions is not resolved by Schmidt and Mackay (1997) and the work of Shanks (2005) demonstrates that the use of the EYM for mortice and tenon connections is not appropriate. A method is presented for predicting the capacity of the newly observed failure mode V (Figure 2-11). The method is empirical in approach and was introduced in the 1997 report by Schmidt and Mackay. A full description of the proposed model was recently published by Miller et al. (2010). The method uses an expression for an effective peg shear strength which is defined using a failure surface. This failure surface relates experimentally determined connection capacities to the specific gravity of the peg and connection timber using the expression below:

\[ F_{vy} = 33440G_{PEG}G_{BASE}^{3/4} \]  

(2.12)

where:

- \( F_{vy} \) is the effective cross grain shear strength of the peg material
- \( G_{PEG} \) is the specific gravity of the peg
- \( G_{BASE} \) is the specific gravity of the timber base material

A Monte Carlo simulation is then run to determine the lower bound design strength value that corresponds to the tolerance in connection geometry and specific gravity values. The simulation gives equation 2.13 for the prediction of mode V capacity for a single peg.

\[ R = \frac{\pi D^2 F_{vy}}{2r_d} \]  

(2.13)

where:

- \( R \) is the load carrying capacity of the connection per fastener
- \( D \) is the peg diameter
- \( F_{vy} \) is the effective cross grain shear strength of the peg material
- \( r_d \) is a design reduction term (=3.5)
The work concludes that for timber pegged mortice and tenon connections the National Design Standard equations should be checked in association with the new mode V expression given above (Miller et al., 2010). However, no data is presented to confirm whether the NDS methods are reliable for timber pegs or in fact whether the failure modes have actually been observed experimentally. It is assumed that the new mode V failure mode is intended to consistently provide a lower bound strength prediction. This would negate the NDS expressions for such applications and thus makes their role redundant. Although this may be the case it does not address the single central mode III hinge failure, which is discussed in some detail within the 1997 report by Schmidt and Mackay. Here it is stated that the EYM expressions are not valid on the basis of a single hinge forming. This has already been discussed, and based purely on the EYM model it is mechanically correct, but only for the assumption of plastic hinge formation. Therefore although the new yield model presented by Miller et al. (2010) can be considered reliable for a mode V failure mode, the wider application of the NDS modes and the uncertainty surrounding the mode III failure leave the overall solution open to question.

2.3.2 Shanks

Shanks (2005) explored, in detail, the failure behaviour and analysis of the mortice and tenon connection commonly used throughout UK carpentry. The experimental programme encompassed a comprehensive experimental study of typical peg types, prior to full scale connection and frame testing. The connection tests investigated connection fit and progressive failure behaviour. Based upon this work an empirically based peg failure model was developed for the prediction of connection strength and recommendations are given for stiffness analysis and frame design. Further to this work Shanks et al. (2008) published findings from an experimental study into the mechanical performance of all-softwood, pegged mortice and tenon connections. This work investigated the performance of a simulated ‘three-plank’ mortice and tenon connection made in softwood with a wide range of different peg diameters and species.

The tensile resistance of a pegged mortice and tenon connection is almost entirely dependent upon the strength and integrity of the peg. In traditional carpentry timber pegs are typically cleft or die driven. The manufacture of these pegs involves splitting the oak along its grain direction using either a draw knife or, in the case of die driven dowels, a sharpened metal die. Machine made prismatic dowels are also used in timber carpentry connections. These are common in North America, Japan and Taiwan (Shanks et al., 2008) and are also used by Oakwrights framing company in the UK. In the UK, carpenters have been (and continue to be) critical of machined timber pegs. This is upon the basis that there is not the inbuilt
CHAPTER 2. LITERATURE REVIEW

quality control attributed with the manufacture of cleaved or die driven pegs. This manufacturing method splits the pegs along the grain and hence ensures continuity of wood fibres and straightness of grain. In manufactured pegs this may not be the case so care must be taken in peg selection.

Shanks’ (2005) investigation of mortice and tenon connections incorporated tests on cleft-tapered, die driven, and turned pegs to allow a clear comparison to be drawn between influential ultimate values such as bending strength, compression perpendicular to grain, tension parallel to grain, and shear parallel to grain. Results from the peg tests demonstrated a consistent 30% higher bending strength than clear material tests which is associated with the high quality of timber from which pegs are manufactured. On the comparison of peg type it was found that, in three point bending, the turned pegs were stronger than die driven dowels and stronger and stiffer than cleft, tapered pegs. However, the turned pegs were made from American White Oak whereas the cleft and die driven pegs were from European Oak, which is marginally weaker in bending (Lavers, 1967).

In order to establish the effect of peg orientation, connection fit, and peg behaviour within an oak mortice and tenon a ‘three-plank test’ was developed (Figure 2-12). The three plank test was developed as a variation of the test setup first proposed by Schmidt and Mackay (1997) which aimed to reliably and accurately simulate the loading conditions experienced in a typical connection. To investigate the influence of peg orientation pegs were tested in a radial orientation (perpendicular to growth ring direction) and in a tangential orientation (parallel to growth ring direction). Connection fit was investigated by reducing the thickness of the central plank from 40 mm to 28 mm in 4 mm increments whilst maintaining the central spacing at 40 mm thickness using oak blocks clamped between the planks.

Peg orientation was not found to affect connection strength but connection stiffness was found to be 10-50% higher when loaded radially compared to tangentially. The
difference in stiffness is believed to be attributed to the presence of ‘rays’ which run along the grain of oak in a radial manner Shanks (2005). Connection fit significantly affects pull-out stiffness and strength. The average strength and stiffness of a very poor fitting connection, with a 12 mm total gap, was found to be in the region of 40% lower than for a perfect fitting connection. This is because when the span is increased the peg is more flexible; bending becomes the more dominant influence on failure. In a better fitting connection, peg failure would be more influenced by cross grain shear and wedging of the peg within the connection Shanks (2005). The influence of shear span on bending failure stress was investigated experimentally for the three peg types. Shear span is defined as the length of the clear gap between the central and side members. It was clear that the effect of shear on the modulus of rupture was very significant. Shanks (2005) states that by altering the shear span from 38 mm to 5 mm the maximum bending stress is reduced by a factor of between 0.45 and 0.55.

Based upon the experimental investigation into connection fit and peg failure behaviour Shanks (2005) developed a method for assessing the ultimate strength of an oak mortice and tenon fixed with a seasoned oak peg. The failure modes defined by the European yield model were not deemed applicable to traditional oak pegged mortice and tenon connections as failure was seen to occur almost exclusively in the peg with very little damage to the bearing timber.

The proposed analysis method employs an energy approach and can be used as a basis for estimating the ultimate capacity of single connections and simple frames (Shanks and Walker, 2005; Hill et al., 2007; Shanks and Walker, 2009). The model was developed within the scope of several key assumptions and within the scope of prescribed traditional geometries. The key assumption made was that peg response is elasto-plastic because, although timber is considered brittle when loaded in tension or bending, the confinement provided by the connection produces an overall connection yielding behaviour due to peg ‘wedging’ Shanks (2005). Brittle failure of the connection can therefore only occur through tenon relish or mortice wall failure, where tenon relish is the parallel to grain plug shear of the timber bearing upon the peg. These failure modes can be controlled through the use of appropriate connection geometry derived from historic precedence.

From inspection of resin cast joints (Figure 2-13) pegs were seen to generally form either three or four hinges. In applying an energy equilibrium model to this failure mode by selecting a suitable spacing of the hinges and peg yield moment a good estimation of the connection capacity can be made. The spacing of the hinges was seen to vary between 0.75d and 1.5d where d is the peg diameter. Based upon peg tests at varying shear spans, Shanks (2005) derived an empirical relationship for the maximum bending stress as a function of shear span. This allowed values of yield moment, $M_y$, to be calculated for the appropriate shear span (shear span is
labelled as ‘a’ in Figure 2-13). Shear span can be selected based upon guidelines set out by Shanks (2005). These guidelines incorporate factors relating to connection fit, peg type, workmanship and confidence of knowledge.

Figure 2-13: Resin cast mortice and tenon connections by Shanks (2005)

2.4 Contemporary timber dowel connections

Relatively little research has been completed in the field of contemporary connections using timber dowels. One of the earliest references to the use of timber dowels and timber plates is at the ETH Zurich in the 1990’s (Stuerer, 2006). More recently the use of timber connection components has pulled a degree of focus in the field of Japanese timber research. In particular the use of compressed timber dowels and plates has been investigated. The advantages of material compatibility, fabrication ease, and fire resistance are often acknowledged (Jung et al., 2008; Fukuyama et al., 2008).

2.4.1 ETH Zurich

During the 1980’s the ETH Zurich, in Switzerland, completed a large amount of research into the development of steel multiple shear plate, dowel connections (Mischler, 1999). The work focussed on experimentally and theoretically understanding the effects of the various geometric parameters such as edge distances and dowel diameter and is recognised as significantly furthering progress in the development and use of multiple shear plate connections (Stuerer, 2006; Mischler et al., 2000).

The work at ETH Zurich was principally concerned with providing high efficiency steel connectors for truss connections (Stuerer, 2006; Mischler et al., 2000). Upon the basis of this work the use of hardwood dowels and plywood plates (Figure 2-14) was proposed for sensitive buildings such as radio antennas. The load bearing capacity of this connection type is reported to provide approximately two thirds the strength of its steel counterpart (Stuerer, 2006), though no scientific results are presented. Equally no other publications appear to have been made in reference
2.4.2 Fukuyama et. al.

The use of timber dowels loaded in single shear has been investigated by Fukuyama et al. (2008) at the University of Tokyo. The research project was completed as a collaboration between the University of Tokyo and Helsinki University of Technology. The specific aim of the research was to develop a non-metallic shear connector for use in stacked log construction, with low embodied energy and mitigation of condensation cited as the major advantages over steel dowels. From a technical point of view the study aimed to develop a standard theory of analysing timber dowels loaded in single shear.

A total of five different dowel materials were investigated: Finnish pine, white oak, Japanese cypress, Finnish birch, and compressed bamboo. Dowels of varying diameter and slenderness ratio were tested in single shear within Japanese cedar, Douglas fir and Finnish pine timber. Strength calculations were made by applying a modified version of the European Yield Model. Unlike Johansen’s original theory, which only considers the bearing yield of the timber member, Fukuyama et al. (2008) also considers bearing yield of the dowel. Hence, the proposed analysis is a direct application of the EYM but uses the minimum bearing resistance value of either the timber member or the dowel. The published results from the testing of 200 specimens in 30 different configurations correspond well, in most cases, with the ultimate load values calculated using the proposed methods. Additionally, stiffness values calculated using a beam on elastic foundation theory also agreed well with those determined experimentally.
2.4.3 University of Bath

Research into the use of white oak dowels in conjunction with plywood flitch plates has been carried out at the University of Bath by Chang et al. (2009) and Clarke (2009). Clarke (2009) tested scarf-type connections in bending with the aim of investigating the strength and stiffness of slot-in plate connections made with multiple white oak dowels. The aim of the work was to provide insight into the potential of the connection technique to resist bending loads with the intention that the connection could be a cost effective all-timber solution suitable for restoration projects. The experimental tests were limited in scope as only two different specimens were tested to failure; a single plate specimen and a double plate specimen. The single plate specimen is shown in Figure 2-15.

![Connection test of oak dowel – birch plywood connection (Clarke, 2009)](image)

Figure 2-15: Connection test of oak dowel – birch plywood connection (Clarke, 2009)

The double plate specimen was made using the same timber cross section (100 mm x 150 mm) and was weaker than the single plate specimen as a result of retaining an inadequate timber cross section. The single plate specimen, made with six dowels, provided a bending resistance of approximately 40% of the full section design capacity. There is potential for further innovation and development in the use of contemporary scarf type connections. Traditional carpentry connections usually only provide 15-30% of the full unjointed section capacity (Ross et al., 2007). However, they are often complex and thus considered too expensive for certain restoration projects. The use of steel or bonded-in rods as an alternative can lead to problems associated with corrosion and bond degradation. The use of engineered timber materials may therefore be more appropriate in certain instances.
The use of white oak dowels connected using plywood was further investigated by Chang et al. (2009). A large number of tests were completed for single plate connections loaded parallel to the member grain direction. Connections made with single dowels and two dowels in line were tested to investigate the spacing rules of the dowels. The plywood plates were 18 mm thick and the dowels were 16 mm in diameter. Three connection failure modes were observed in the testing:

- Three hinge dowel failure
- Partial shear plug of the timber member
- Net tension failure of the plywood

The three hinge dowel failure was similar to that observed by Shanks (2005) for mortice and tenon connections (Figure 2-13). Alternative dowel failure modes resulting from the use of plywood of lower thickness are presented in the paper by Thomson et al. (2009), which is included in the appendices of this thesis. The net tension failure of the plywood is undesirable as it results in the complete loss of connection resistance and limits post yield energy absorption. However, the tests completed in this study used relatively low quality plywood and high quality birch plywood would likely reduce the occurrence of this mode of failure.

The third observed failure mode was a partial thickness shear plug failure (Figure 2-16). This failure mode is significant in that it is not certain as to whether it has been reported in tests completed with metallic fasteners. The mode of failure is attributed to the low strength and stiffness of timber fasteners when compared with metallic equivalents. In contrast, full thickness shear plugs and splitting are reported for metallically fastened connections (Quenneville and Mohammad, 2000). Additionally, the partial shear plug failure has not been reported in the literature on traditional mortice and tenon connections due to the fact that the mortice walls are loaded perpendicular to grain and failure of the tenon occurs as a full thickness plug. Analysis of the partial shear plug mode can be considered important if the capacity of the plywood plate is to be safely designed.

2.5 Modified wood

2.5.1 Compressed wood

In Japan the use of compressed wood has been explored as a means of providing a replacement for steel connectors in timber structures. Compressed wood is made by compressing low density wood such as Japanese cedar in a hot press. Compressed
Figure 2-16: Partial shear plug failure in oak dowel – plywood plate connection (Chang et al., 2009)

wood with a density greater than 1000 kg/m$^3$ can be produced by compressing Japanese cedar (330 kg/m$^3$) for 30 minutes at a temperature of 130$^\circ$C (Jung et al., 2008). In the compression process clear wood specimens are placed between heated plates and compressed perpendicular to grain. The combined heat and pressure begins to soften the lignin within the cell walls of the wood. This allows the cells to drift and collapse, closing the open spaces within the wood cell structure. A rapid drop of temperature, whilst maintaining pressure, freezes the compression (Leijten, 1998).

Jung et al. (2008) tested full scale joints made from Japanese cedar glulam and connected with compressed wood plates and dowels as a substitute for steel counterparts. The results from testing beam to sill connections in cedar glulam demonstrated favourable capacities for compressed wood dowels. A connection made with a single 12 mm diameter dowel and loaded through two 15 mm compressed plates in pull out, provided a maximum load capacity of 18kN.

The potential for adding value and structural integrity to low strength timber such as Japanese Cedar is clearly demonstrated by this study. Concerns have been raised regarding the recovery of compressed wood when it comes into contact with moisture and hence the potential of increased risk of splitting. However, this phenomenon can be avoided if the correct compression rate is carefully controlled (Jung et al., 2008). Currently, constraints on the availability of clear, compressed wood are a limiting factor for its use, particularly in Europe where manufacturing facilities do not appear to exist. This is a reflection of the demand for compressed
wood, which is currently low. Therefore, for the reasons of availability and potential for mainstream application this study has not investigated the use of compressed clear wood. Nonetheless, in Europe the construction of electrical transformers utilises a form of compressed laminated wood which is available as a commercial product (Lignostone, 2010). This product was therefore considered to hold greater potential in terms of mainstream uptake and is discussed in more detail below.

2.5.2 Densified veneer wood

Densified veneer wood (DVW) is a compressed plywood material that is typically made using beech wood veneers and phenol formaldehyde resin (Leijten, 1998). The material is widely used to make electrical transformers where electrical insulation is required. Other uses of the material include bullet proof applications and most relevant to this study within structural timber connections (Lignostone, 2010).
The use of DVW in timber connections has been widely reported (Leijten, 1993; Guan and Rodd, 2001; Rodd and Leijten, 2003). In the development of a new connection system utilising hollow steel fasteners the material was used as reinforcement at the connection interface to provide a high level of connection ductility (Leijten, 1998, 1999). The DVW was glued to the connection interface, provided a high embedment resistance and prevented splitting of the structural members. Reinforcing connections in this way provided significant improvements in their strength capacity.

A study of the use of DVW dowels to provide non-metallic timber connections is reported by Ehlbeck and Eberhart (1989). The report demonstrated that EYM methods could be used for predicting strength, with the addition of a further dowel failure mode. However, Leijten (1998) states that long term loading studies showed that the dowels failed at a reduced capacity compared with short term tests. The failure behaviour was attributed to the use of fully resin impregnated DVW dowels.

Prior to the use of DVW for the reinforcement of connections Leijten (1998) reports that attempts were made to reinforce connections using steel plates and glass fibre material. However these were unsuccessful as the gluing of steel plate was a very complex procedure, whilst the glass fibre only provided marginal strength improvements. Therefore DVW was selected as the most appropriate material for this purpose. Leijten (1998) describes the following material advantages of using DVW:

- DVW is a commercially available material.
- The density of DVW is comparable with high density tropical hardwoods and therefore the drilling of holes requires similar equipment.
- Compared to ordinary timber the mechanical properties of DVW are less affected by load direction when using cross-wise layered veneers.
- Early tests showed an embedment strength of up to 160 MPa, which is about half that of steel.

Three different grain orientations of densified veneer wood are manufactured:

- Uni-directional – all of the veneers are orientated with the wood grain running parallel.
- Cross-wise – the veneers are alternately placed at 90° to each other.
- Tangential – the grain orientation of each veneer is at 45° or less to each other.
CHAPTER 2. LITERATURE REVIEW

The reason for producing the tangential material is to minimise in-plane orthotropy. However, Leijten (1998) states that the tangential orientation is more expensive than the other two as it creates more waste of veneer.

Manufacture of DVW uses the same compression principle as for compressed wood, which is outlined above. The difference between the two processes is in the initial stages. Compressed wood typically comprises clear, non-engineered wood, whereas DVW is made up from thin veneers of wood. The commercially available DVW in Europe is made by spreading phenol polymers onto the veneers and allowing them to dry. The veneers are then stacked dry and compressed. The heat and pressure of the process causes the resin to bond and partially impregnate the veneers (Leijten, 1998).

The performance of DVW is influenced by various production parameters. Leijten (1998) outlines the most significant of these parameters to be:

- Level of compression
- Compression temperature
- Time of compression
- Resin content

Compression temperature, and time of compression, influence the shrinkage and swelling properties of DVW material. Specifically for DVW a minimum temperature of 127°C is required to initiate polymerisation of the phenolic resin between the veneers (Leijten, 1998). Above 150°C the material’s capacity to absorb water is significantly reduced and hence the dimensional stability of this material is enhanced. Above temperatures of 150°C Leijten (1998) reports that the influence of compression time on water absorption is negligible though it is significant below this temperature. A compression time of 5 minutes gives a material with significantly higher water absorption capacity than a compression time of 45 minutes (Leijten, 1998).

The level of compression can be measured in terms of imposed stress to achieve a given volume reduction of the wood. In general it is observed that a higher level of compression will give improved mechanical properties in accordance with the increased density of the material. However, Leijten (1998) states that above a compression stress of 20 MPa the mechanical properties begin to reduce due to damage of the wood fibre structure.

The resin content of DVW material affects mechanical properties and dimensional stability. Fully resin-impregnated material is impenetrable to moisture and
therefore is very stable dimensionally. However, the modulus of elasticity, tension and bending strength of the material are reduced in fully resin-impregnated material. Therefore, partially resin-impregnated material is best suited for structural applications. Leijten (1998) discusses the application of DVW in timber structures in terms of EC5 service class. It is suggested that partially resin-impregnated DVW can be used in structural timber engineering for service class 1 and 2. For service class 3 conditions, appropriate measures to prevent moisture absorption are required or the use of fully resin-impregnated material may be considered. However, fully resin-impregnate material is more brittle and has lower mechanical strength properties (Leijten, 1998). For reference EC5 service classes are defined below:

- **Service class 1** – Moisture content in the materials corresponds to a temperature of 20°C and the relative humidity only exceeds 65% a few weeks a year. The average moisture content in most softwoods will not exceed 12%.

- **Service class 2** – Moisture content in the materials corresponds to a temperature of 20°C and the relative humidity only exceeds 85% a few weeks a year. The average moisture content in most softwoods will not exceed 20%.

- **Service class 3** – Climatic conditions are such that they lead to higher moisture contents than in service class 2; typically external, exposed conditions.

In-plane tension and compression strength and embedment resistance were investigated and reported by Leijten (1998). All of the tests were completed for cross-wise laminated material. Tension test results are reported for varying density and two different thicknesses of material (Figure 2.18(a)). Compression tests were completed for material conditioned to a moisture content which corresponded to service class 1 and service class 3 (Figure 2.18(b)).

Figure 2.18(a) shows that, for DVW in service class 1 conditions, the tensile strength of the material is influenced by the level of compression and hence density. For the material shown the intention was to manufacture two target densities of 1050 kg/m³ and 1300 kg/m³. However, this was not successfully achieved for the 8 mm thick material. Commercially manufactured, cross-wise laminated DVW is manufactured in three different target densities; low (900 - 1000 kg/m³), medium (1200 - 1300 kg/m³), high (1350 kg/m³) (Rochling, 2010).

The data presented in Figure 2.18(b) demonstrates how high moisture contents, associated with service class 3 conditions, influence the density and compressive strength of the material. The loss in density is associated with volumetric changes.
due to swelling. The importance of protecting DVW against moisture absorption in service class 3 conditions is evident in the loss of performance.

The investigation of embedment strength under short term and long term loading was also investigated experimentally by Leijten (1998). Figure 2.19(a) shows the results for the short term loading tests on beech cross-wise laminated DVW. The material was conditioned to service class 1 conditions prior to test. Tests were completed for tension and compression loading and in orthogonal directions and at

![Graph showing in-plane tension strength plotted against density for beech DVW](image1)

**Figure 2-18(a):** In-plane tension strength plotted against density for 8 and 15 mm DVW thickness

![Graph showing in-plane compression strength plotted against density for different species](image2)

**Figure 2-18(b):** In-plane compression strength plotted against density (data above dotted line is service class 1, below is service class 3)

Figure 2-18: In-plane tension and compression strength results for cross-wise DVW plate (Leijten, 1998)
45° orientation. The density of the material ranged between 1144 - 1253 kg/m³. The results showed that the variation between the type of loading (tension or compression) and the direction of load to grain angle were not significant. It was concluded that the embedment strength of DVW produced with Beech veneers is independent of the load orientation and type (Leijten, 1998).

Leijten (1998) presents data for the long term embedment resistance of beech DVW recorded over a period of over 1000 days. However, it is stated that the tests were still ongoing after four years with no observed failures. The data presented is particularly significant as prior to this study the only investigation of the long term performance of DVW was for fully resin impregnated dowels (Ehlbeck and Eberhart, 1989), which performed poorly under long term loading. The connections made with fully resin impregnated dowels were loaded at 45% of their short term capacity and failures were observed within several weeks Leijten (1998). The long term data presented by Leijten is for non-impregnated DVW.

The long term embedment tests were carried out at 40% of the short term embedment load capacity and the material was conditioned to service class 1 conditions (20 ± 2°C and 65 ± 2%). The creep deformation was taken as the change in distance between the dowel and the middle of the loaded end. Data was recorded every two weeks with a digital transducer that had an accuracy of 0.01 mm. The dowel diameter used in the tests was 18 mm. Results of the tests are plotted in Figure 2.19(b). There is a dip in the results at around 400 days and this was due to a breakdown in the conditioning equipment. From inspection of Figure 2.19(b) the total creep over 1000 days approximately 0.06 mm with no reported failures. This suggests that the long term load capacity of DVW that is not fully resin impregnated is acceptable for structural applications.

2.6 Concluding comments

At present, provisions for the specification and design of mechanically fastened, contemporary, non-metallic connections are significantly limited. This is partly due to the lack of design guidance in codes such as Eurocode 5. It is also a reflection of the level of research carried out into contemporary timber connections using non-metallic components.

Previous research into non-metallic dowelled connections has investigated the use of GFRP dowels, traditional timber pegs, and compressed wood dowels. In these instances, alternative methods of analysis are developed to predict connection strength and in some cases stiffness. Eurocode 5 methods were developed for the analysis of timber connections made with metallic dowel type fasteners. Therefore,
(a) Density plotted against embedment strength of DVW

(b) Long term creep data for DVW in service class 1 conditions

Figure 2-19: Embedment data for DVW under short and long term loading (Leijten, 1998)
the methods are not considered to be directly applicable to non-metallic materials, which are anisotropic and more sensitive to shear loading.

Research on the use of timber dowels has been mainly focussed in the field of traditional carpentry connections. The analysis methods proposed for mortice and tenon connections are therefore relatively specific to the traditional geometries used.

The use of GFRP dowels has been shown to have marked potential in terms of connection strength. However, the use of GFRP plate is questionable due to its unidirectional pultruded structure and embedment strength. The validity of analysis methods proposed for predicting strength are unconvincing.

Modification of wood through compression has been shown to produce high strength wood based materials. Commercially available sheet products, made from DVW, have potential to provide robust connection components. Research into the use of DVW sheet as a flitch plate within a timber connection has not previously been completed.
Chapter 3

Selection of dowel and plate materials

Presented within this chapter are the test results and observations from an initial series of experiments that investigated the use of different non-metallic dowel and plate materials. The objective of the study was to experimentally determine the comparative load response of connections made with different dowel materials prior to investigating the use of different plate materials. Based upon the findings of the study a single connection system is selected for further investigation and development.

3.1 Dowel materials

Three different non-metallic dowel materials were selected for comparative testing: European oak, glass fibre reinforced polymer (GFRP) and densified veneer wood (DVW). They were selected for their potential to provide robust connections that could realistically be adopted by the mainstream timber manufacturing and engineering community. For completeness stainless steel dowels were also tested to provide a reference in terms of connection stiffness, strength and dowel failure mode.

3.1.1 Oak dowels

Oak dowels have been used for hundreds of years in traditional timber structures. Shanks (2005) demonstrated that the UK green oak mortice and tenon connection has significant tensile capacity. This was particularly apparent in comparative tests of braced frames where the addition of a tensile brace (often ignored in
analysis) provided significant additional strength and frame racking resistance. Based upon the findings of this investigation and the development of analytical tools for the evaluation of oak dowelled connections it was felt that a contemporary engineered application of oak dowels was possible. The advantages of this type of connection would be associated with the pure architectural aesthetic of an all-timber connection, in addition to the use of low environmental impact, low cost materials.

For this investigation, machined, prismatic European oak dowels were selected for testing. The development of mechanical tools, to machine prismatic dowels from timber, has provided the opportunity for efficient production of timber dowels, and thus a wider mainstream uptake. Machined dowels are used frequently in Japan and Taiwan, whilst turned white oak pegs are used widely by Oakwrights green oak carpentry company in the UK. Traditional carpenters are often critical of machined dowels on the basis that there is potential for poor quality timber to be used, resulting in excessive cutting of grain. Traditional cleft pegs or die driven pegs have a degree of quality control in-built in their manufacture, which involves splitting along the grain in both cases. However, Shanks (2005) demonstrated that connections made with turned American white oak pegs were consistently stronger and stiffer than European oak cleft and die driven pegs. In part this can be attributed to the favourable mechanical properties of American white oak when compared to European oak. However, the increased stiffness can also be attributed to the complete confinement of machined pegs within the connection. An additional consideration in the use of machined dowels is that where straight grained timber is used for manufacturing dowels the total amount of continuous timber fibres within the dowel volume will be approximately equal to that of a cleft peg. Nonetheless, care is required when manufacturing timber dowels in order to ensure quality control. Therefore all dowels used in this test series were manufactured from straight grained European oak and any dowels which exhibited excessive cutting of grain were discarded.

3.1.2 GFRP dowels

The feasibility of using GFRP dowels to make timber connections was demonstrated by (Pedersen, 2002) and (Drake, 2003) as presented in Chapter 2. GFRP rod sections, which are readily available as an off-the-shelf product, can be easily cut to length and are of relatively low cost in comparison to metallic fasteners. The rods are manufactured as prismatic sections through the process of pultrusion. This process ensures efficient production, consistency of material and low costs. The pultrusion process consists of pulling resin impregnated glass fibres through a heated curing die at speeds of up to 3 metres/minute. Very high fibre contents,
of up to 70%, can be achieved through the use of pultrusion techniques. The high fibre content provides excellent mechanical properties and is achieved through the tight packing of fibres in the drawing process (Bakis et al., 2002). Resins used in the production of GFRP are based on products from the oil industry and are used for thermoset and thermoplastic matrices. The rods used in this study are an off-the-shelf product which use a polyester thermoset matrix and E-glass fibres. Thermoset resins are cured through heating in the production process to form cross-links and are not softened by reheating, which is an important consideration for fire resistance of connections.

### 3.1.3 DVW and compressed wood dowels

DVW dowels are machined from densified veneer wood sheets. They are commercially produced, high tolerance products, with favourable material properties. The earliest reported investigation on the use of DVW dowels to make timber connections was described by Ehlbeck and Eberhart (1989). The study proposed the use of the EYM strength analysis for the dowels, in addition to a new expression to describe a further unique failure mode. No further work was published on the use of these dowels. In the later stages of this study it was discovered that this was due to problems associated with the long term performance of the dowels. The work of Leijten (1998) on the use of DVW material, reports that fully resin impregnated DVW material can fail at significantly lower loads under long term loading than under short term loading. As this was not known until the later stages of the study, experimental testing of connections made with DVW dowels was completed as part of the initial study reported in this chapter. The results are included as they provide insight into the load response of connections made with particularly brittle dowels.

Studies into the use of clear compressed wood dowels have been completed in Japan (Jung et al., 2008; Fukuyama et al., 2008). However, the manufacture of compressed wood dowels is typically completed in small batches in the laboratory especially for research purposes. The use of DVW dowels was therefore favoured over compressed wood dowels for the practical reason of commercial availability. However, because it is now understood that fully resin-impregnated DVW dowels provide poor long term strength the use of clear compressed wood may be considered as the best area for future research focus on modified wood dowel materials.

### 3.1.4 Comparative testing

A simplified version of the method set out in BS EN 26891 (1991) was used to test the dowels in softwood double shear connections. The simplified method
uses monotonic loading and connection slip is measured using platen displacement and not LVDTs. The adopted test setup allowed specimens to be fabricated with relative ease and provides the capacity to easily repeat the tests.

A total of eight different double shear connection configurations were tested to incorporate two different timber bearing arrangements for each dowel material. The timber bearing arrangements were such that one set of tests were completed with the side member grain parallel to the load direction and the other with the grain perpendicular to the load direction. The two different joint configurations are shown in Figure 3-1. The height and depth of the stainless steel specimens were larger than those shown in order to mitigate splitting of the bearing timber prior to dowel yield. Kerto-S LVL (laminated veneer lumber) was selected for the bearing timber as it provided a more uniform, comparable medium than may be expected from sawn softwood timber. Kerto-S LVL has all of the veneers running in the same grain orientation.

![Figure 3-1: Double shear test specimens; parallel to grain (left) and perpendicular to grain (right)](image)

All tests used dowels with the same diameter so that variables such as the embedment resistance of the LVL members were kept constant and only the strength and stiffness properties of the dowels define the capacity of the connection. Dowels of 12 mm diameter were selected for testing and this was informed by the commercial availability of GFRP and DVW as well as being considered to be comparable to the diameter of metallic dowels commonly used in practice.

All specimens were tested using a displacement controlled loading rig and were loaded in compression at a rate of 1.5mm/minute. The loading rate was based upon inspection of load slip responses reported in the literature for oak and GFRP dowels and was intended to ensure connection failure within 300 seconds in accordance
with the methods set out in BS EN 26891 (1991). Load and platen displacement readings were logged throughout all the tests. Prior to testing all of the LVL components and oak dowels were stored in a climatically controlled store at 20\(\pm\)3\(^\circ\)C and 65\(\pm\)2% relative humidity until constant mass was reached. Five repeat tests were completed for each specimen configuration with the exception of the DVW dowels. Due to the limited availability of the DVW dowels for testing four repeat tests were completed for the load direction parallel to grain and two tests for the load direction perpendicular to grain.

### 3.1.5 Results and discussion

The mean average connection yield load, ultimate load and initial stiffness values are summarised in Table 3.1 for the eight different specimen configurations. Connection initial stiffness was defined as the gradient of the line that passes through the points corresponding to 10% and 40% of the ultimate load (BS EN 26891, 1991). Ultimate load was defined as the maximum load recorded prior to, or at, 15 mm connection slip. Yield strength was evaluated using the 5% offset method described in ASTM 5652-95. This method defines the yield load as the intercept between the load-slip curve and the line of initial stiffness offset by 5% of the dowel diameter (Figure 3-2).

![Figure 3-2: 5% offset analysis method used to determine connection yield](image)

The results of the tests are presented in Table 3.1 in rank order of yield capacity to help facilitate comparison. It is evident that the stainless steel dowels provided a significantly stronger and stiffer connection than the non-metallic counterparts. Nonetheless, the GFRP and DVW dowels provided a favourable connection yield capacity of approximately two thirds the value of the stainless steel dowel specimen. Additionally the ultimate capacity and stiffness values recorded for the GFRP
### Table 3.1: Results summary - dowel material comparison for 12 mm diameter dowels

<table>
<thead>
<tr>
<th>Dowel Material</th>
<th>Side member orientation</th>
<th>No. of tests</th>
<th>Yield load (kN)</th>
<th>Ultimate load (kN)</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean value</td>
<td>COV (%)</td>
<td>Range</td>
<td>Mean value</td>
</tr>
<tr>
<td>Oak</td>
<td>Parallel</td>
<td>5</td>
<td>4.8</td>
<td>17.3</td>
<td>3.7 – 5.6</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>5</td>
<td>4.6</td>
<td>8.1</td>
<td>4.3 – 5.1</td>
</tr>
<tr>
<td>GFRP</td>
<td>Parallel</td>
<td>5</td>
<td>10.9</td>
<td>4.3</td>
<td>9.8 – 11.0</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>5</td>
<td>9.3</td>
<td>8.3</td>
<td>13.3 – 16.8</td>
</tr>
<tr>
<td>DVW</td>
<td>Parallel</td>
<td>4</td>
<td>11.7</td>
<td>10.4</td>
<td>10.8 – 13.4</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>2</td>
<td>9.2</td>
<td>N/A</td>
<td>8.8 – 9.7</td>
</tr>
<tr>
<td>Stainless steel</td>
<td>Parallel</td>
<td>5</td>
<td>16.6</td>
<td>6.5</td>
<td>14.8 – 17.7</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>5</td>
<td>14.5</td>
<td>3.3</td>
<td>13.9 – 14.7</td>
</tr>
</tbody>
</table>
dowelled connections were approximately two thirds of the values recorded for the stainless steel dowels. The strength and stiffness values recorded for the oak dowels were significantly lower than those recorded for the other dowel materials. This was to be expected as timber dowels are a non-engineered product which are of relatively low strength. In practice a larger diameter of dowel would be used in a connection as a reflection of the dowel’s lower capacity and stiffness.

Coefficient of variation (COV) values are presented alongside the strength and stiffness data given in Table 3.1. Variation in results can generally be attributed to variation in the dowel materials and timber being tested. However, in certain cases significantly high COV values are presented. For the oak dowels large variation within the data was expected as the resistance of the oak dowels can be influenced by factors such as slope of grain, density, dowel orientation, and the presence of natural defects. Additionally, because the oak dowels are of relatively small diameter, the influence of these factors will be further enhanced. The level of variation recorded for the oak dowel specimens is in agreement with the test data for mortice and tenon connections, presented by Shanks (2005). Significant variation in the stiffness of the GFRP dowelled connections was also recorded. Data presented by Pedersen (2002) gave a COV value of 15% for the stiffness of 12 mm diameter dowel connections in glulam of 46 mm side thickness. These results were given for data recorded from ten tests and for connections made with central GFRP plates. Based on these results, the higher variation in GFRP connection stiffness given in Table 3.1 can be considered reliable as the data set is made up of fewer test results and was recorded for connections with a central member made from timber. The variation between the two data sets can therefore be attributed in part to inconsistency in fabrication and material variation.

Figures 3.3(a) and 3.3(b) present typical load slip responses for the connections loaded parallel and perpendicular to grain. Figure 3-4 shows photographs of specimens which were locked at the point of maximum slip and then dissected to allow inspection of the failed dowel. These figures provide a greater depth of insight into the load response and failure modes observed for the different dowel materials. Single, representative load slip plots were selected from the five available data sets for each specimen group in order to provide clarity in the presented results. The plots selected were considered to best represent typical connection responses in each case.

The load slip plots all display an initial linear response under load and beyond connection yield the oak, GFRP and stainless steel dowel specimens all displayed a degree of connection hardening under further loading. Inspection of the dissected specimens shown in Figure 3-4 suggest that for the GFRP and steel dowels this hardening is associated with further embedment of the dowel after the initial dowel yield whereas for the oak dowel the hardening is attributed to compression of the
CHAPTER 3. SELECTION OF DOWEL AND PLATE MATERIALS

Figure 3-3: Typical load-slip plots for dowel material study

(a) Typical load-slip plots for parallel to grain loading of specimens

(b) Typical load-slip plots for perpendicular to grain loading of specimens
dowel itself. It can be seen that for the DVW dowel a sharp stepped response in the load slip plots occurred at the point of connection yield, This was subsequently followed by a rapid loss of strength in the parallel to grain specimens. The stepped loss of connection capacity suggests a brittle dowel failure which is confirmed through inspection of the dissected DVW dowel specimen in Figure 3-4. In this specimen failure across the entire dowel cross section at four points can clearly be seen. A similar ultimate failure mode can also be seen for the oak dowels, however the connection load slip response suggests that this occurred only after significant crushing of the dowel, which was observed as connection hardening.

The influence of shear stiffness on the dowel failure modes is highlighted in Figure 3-4. A single central hinge can be observed in the stainless steel specimen whilst the non-metallic dowels all show four points of failure along their length. Stainless steel dowels have a flexural modulus which is an order of magnitude greater than that of the non-metallic dowels and so these modes of failure were not unexpected. The point of note in this instance is that although the stiffer stainless steel dowel provided a higher connection capacity, ultimate failure parallel to grain was often as a result of brittle splitting. This can be seen in the rapid loss of strength in the load slip response of Figure 3.3(a). The implication of this was that although non-metallic dowels are individually weaker than metallic counterparts it may be possible to place them at closer spacings than metallic dowels and still avoid
premature splitting. This would improve the load capacity per connected area of a timber member joined with non-metallic dowels. It should be noted that because these tests were completed in compression tensile failure perpendicular grain was not addressed but may also be a limiting factor in the use of metallic dowels.

The post yield performance and stiffness of the connections made with DVW dowels was poor in comparison to those made with GFRP dowels (3-3). It was therefore decided that further investigation of the use of DVW dowels would not be pursued. A limited investigation of the use of oak dowels in conjunction with birch plywood flitch plates was completed and this was reported previously by Thomson et al. (2009). The study showed that robust connections could be provided and designed using plywood plates and oak dowels, however the capacity of the connections was considered to pose a significant restriction on the mainstream use of the connection type. The further investigation and development of the use of GFRP dowels was therefore pursued as the most appropriate dowel material for mainstream contemporary connections.

3.2 Plate materials

Three different materials were selected as potential plate materials for use with GFRP dowels. The materials were birch plywood, low density crosswise DVW, and medium density crosswise DVW. All three of these materials have crosswise laminations so are capable of carrying loads in multiple directions and both use timber as their primary constituent material. In previous studies the use of GFRP plate with GFRP dowels has been investigated. However, this work has not pursued the further investigation of GFRP plate as a means of providing a shear plate connection. In main, this is due to concerns raised in relation to the unidirectional orientation of the material, which places restrictions on the direction in which the plate can be loaded as well as raising issues surrounding the continuity of fibres after drilling.

In order to determine the strength compatibility of the selected plate materials with the GFRP dowels, a limited series of preliminary tests were completed. The objective of these tests was to gain a qualitative understanding of how the different materials interact, and to determine which material had sufficient embedment resistance to cause failure of the dowel. Specimens were fabricated in the same manner as the double shear dowel tests with the only variance being that the central member was made from the appropriate plate material and not LVL. As for the double shear dowel tests the specimens were loaded in compression at a load rate of 1.5 mm/min. Tests were completed for single specimens only and the specimens were locked at the end of each test and dissected to inspect the
connection failure modes. Four different specimen configurations were tested and these are set out in Table 3.2 below. The labels assigned to the specimens in Table 3.2 coincide with those used in Figure 3-5.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Plate material</th>
<th>Plate thickness</th>
<th>Dowel diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Birch plywood</td>
<td>9 mm</td>
<td>12 mm</td>
</tr>
<tr>
<td>b</td>
<td>Birch plywood</td>
<td>9 mm</td>
<td>8 mm</td>
</tr>
<tr>
<td>c</td>
<td>Low density DVW</td>
<td>10 mm</td>
<td>12 mm</td>
</tr>
<tr>
<td>d</td>
<td>Medium density DVW</td>
<td>10 mm</td>
<td>12 mm</td>
</tr>
</tbody>
</table>

It is evident from the four specimens depicted in Figure 3-5 that birch plywood is not of compatible strength with GFRP dowels. The poor compatibility of plywood with GFRP dowels is due to the relatively low embedment strength of the plywood in comparison to the yield resistance of the GFRP dowel. A small level of interaction between the plywood and a more slender 8mm GFRP dowel was evident but this offered no clear strength advantage as it only occurred after significant embedment of the plywood plate. Low density DVW plate provided
good interaction with the GFRP dowel though significant embedment of the plate is evident in the dissected specimen. Medium density DVW plate material was found to be wholly compatible for use with GFRP dowels. It can clearly be seen in Figure 3-5 that the use of the DVW plate produced full dowel failure whilst itself remaining undamaged.

The load-displacement plots shown in Figure 3-6 for the four specimens demonstrate the significance of attaining full interaction of the dowel with the connector plate. The birch plywood plate connections perform poorly and the near flat post yield plateaus shown in the load slip plot is in agreement with the the plastic embedment failure observed in the dissected specimens. The low density DVW plate performs well in comparison to the birch plywood, however a significantly lower strength and stiffness response was recorded in comparison to the medium density plate. Therefore based upon these findings medium density crosswise DVW plate was selected as the most appropriate plate material for further investigation and development as part of a non-metallic connection system.

Figure 3-6: Load–displacement plots for plate material compatibility tests
3.3 GFRP-DVW connection

The experimental investigation of non-metallic dowel and plate materials presented in this chapter has informed the selection of GFRP dowels and medium density DVW plate for the development of a contemporary non-metallic timber connection system. Prior to beginning in-depth experimental programmes and material characterisation a series of small scale tests were completed to provide insight into the influence of side member thickness and dowel diameter on connection load response and failure mode.

3.3.1 Experimental investigation

Five different test configurations were completed. Details of the test specimen configurations can be found in Table 3.3. The objective of the tests was to investigate whether the GFRP dowel failure mode shown in Figure 3-5 was unique to this connection geometry or whether alternative failure modes would be observed for varying dowel slenderness ratios. Of the 19 tests completed four were locked at test completion and dissected so that the dowel failure mode could be seen in detail. Shown in Figure 3-8, the specimens dissected were 24, 36 and 48 mm thick side member specimens and a 16 mm diameter dowel specimen (the 48 mm side member specimen is repeated from Figure 3-5 for clarity). An 8 mm diameter dowel specimen was not locked and dissected but inspection of disassembled specimens after testing was completed to confirm failure modes.

Table 3.3: Test specimen configurations

<table>
<thead>
<tr>
<th>Specimen group</th>
<th>Side member thickness (mm)</th>
<th>Dowel diameter (mm)</th>
<th>Dowel slenderness (t/d)</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>24</td>
<td>12</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>b</td>
<td>36</td>
<td>12</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>c</td>
<td>48</td>
<td>12</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>d</td>
<td>48</td>
<td>8</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>e</td>
<td>48</td>
<td>16</td>
<td>3</td>
<td>5</td>
</tr>
</tbody>
</table>

All of the specimens exhibited an initial linear displacement response to load followed by post yield hardening of the connection, associated with embedment of the dowel. This can be seen in Figure 3-7. Connection capacity is strongly influenced by the dowel diameter used as would be expected for any dowel material. However, side member thickness had less of an influence on the load resistance and stiffness of the connections. From inspection of Figure 3-7 it can be seen that the yield of the connections made with 24 and 36 mm thick side members closely matched that of the specimens made with 48 mm thick side members. At increased connection slip a lower post yield strength was observed for the specimens with 24
3.3.2 GFRP-DVW connection failure mode

Inspection of disassembled specimens after testing, in association with the dissected specimens shown in Figure 3-8, showed that all of the specimens failed in the four hinge manner previously observed in Figure 3-5. This failure mode is similar to the four hinge failure mode characterised by the EYM and given as mode III in section 2.1.1 of Chapter 2. The basis for the expression of the mode III EYM dowel failure is given in the same chapter and the expression for predicting the connection capacity per shear plane for this failure mode is repeated here for reference:

\[ R = 2\sqrt{M_s f_k d} \]  

Equation (3.1)
This expression is independent of the thickness of the connected timber member. Hence, for a connection made with a dowel of equal diameter, where the same failure mode is observed an approximately equal yield capacity can be expected. For GFRP-DVW connections it was anticipated that the narrower side members may have failed in a manner more akin to modes I or II of the EYM. However, where the absence of a mode I mode can be attributed to the dowel being insufficiently stocky the absence of a mode II suggests the failure of the dowel itself is different to that of a metallic dowel. This is apparent from inspection of the failed dowels in Figure 3-8 where significant interlaminar failure of the dowel between the apparent hinges can be seen. The failure mode of GFRP dowels and their respective characterisation is presented in detail in section 4.6 of Chapter 4.
3.4 Concluding comments

Simulated double shear connection tests have been carried out to facilitate the selection of non-metallic dowel and plate materials suitable for making structural timber connections. In particular the combination of medium density DVW plate and GFRP dowels has been selected as a potential non-metallic connection system suitable for mainstream applications. Further conclusions that can be drawn from the work presented in this chapter are as follows:

- In a simple double shear connection GFRP and DVW dowels provided a yield capacity approximately two thirds of that recorded for equal diameter stainless steel dowels.

- Oak dowels provided a significantly lower load capacity than the other materials but could be considered for use in specialist architectural situations.

- DVW dowels provided poor connection stiffness and brittle post yield connection failure.

- For the double shear connections tested, four points of failure along the dowel were observed for non-metallic dowels and only a single hinge for the stainless steel dowel. It is suggested that this may be attributed to the comparatively low flexural stiffness of the non-metallic materials.

- DVW plate is compatible with GFRP dowels and does not exhibit any visible embedment damage at connection failure.

- The use of GFRP dowels and DVW plate was identified as a means of providing a robust, mechanical timber connection system suitable for contemporary applications.

- A single four hinge failure mode (comparable to mode III failure for metallic dowels) has been identified for double shear GFRP-DVW connections made with varying dowel diameters and side member thickness.
Chapter 4

Characterisation of connection components

The independent mechanical properties of the component materials that make up a GFRP-DVW connection directly define the response of the connection under load. Therefore in order for the strength and stiffness of a connection to be analysed it is necessary to have a sound knowledge of the behaviour and mechanical properties of the component materials.

This chapter presents the testing methodologies, rationale and results for the characterisation of the component parts of a GFRP-DVW connection. The reported material strength and stiffness values are used in Chapters 7 and 8 in order to analyse and predict connection stiffness and strength.

4.1 Connection materials and fabrication

The initial investigation of non-metallic connector materials presented in Chapter 3 used Kerto-S Laminated Veneer Lumber (LVL) throughout. The LVL timber was selected as a means of providing a basis for direct comparison between connections, made with different dowel materials, due to its engineered structure.

In a move away from LVL the test specimens reported in Chapters 5 and 6 were all made using UK grown Douglas fir glulam. The use of softwood glulam for these experimental tests was considered to best reflect the dominant material choice of the European timber engineering community. Therefore, this provided a broader applicability than could have been expected from using LVL. Douglas fir glulam is further suited for these tests as it is a species that is particularly prone to splitting (BS EN 1995, 2004) and therefore provides a lower bound resistance for
CHAPTER 4. CHARACTERISATION OF CONNECTION COMPONENTS

an investigation into the spacing of GFRP dowels. The glulam laminates were 46 mm thick and were machine graded to C24 strength.

The connection plates were made from medium density, cross laminated densified veneer wood (DVW). The selection of medium density plate was based upon the required embedment resistance for full dowel interaction. ‘Full dowel interaction’ is defined as a connection failure that occurs as a result of deformation of the dowel and bearing yield of the timber members as opposed to bearing failure of the connection flitch plate. This dowel interaction is described in further detail within Section 3.3 of Chapter 3. DVW material characteristics are discussed in detail in Chapter 2. In particular the influence of resin content is highlighted. DVW is manufactured to several different specifications including the level of resin impregnation. High resin impregnation of the material improves dimensional stability and reduces the absorptancy of the materials which are important characteristics in certain applications. However, Leijten (1998) reported that these attributes are at the cost of long term load capacity as fully impregnated DVW can fail unexpectedly under long term loads. This was not known prior to completing the parallel and perpendicular to grain tests, reported in Chapter 5, and resin impregnated material was used in these tests. The manufacturers grade of this plate was MII/2/30-E3. Nonetheless although the material has a high resin content and is therefore not suited to long term loading the short term loading of the test specimens is not significantly affected. For the full scale tests, reported in Chapter 6, low resin content material was used. The manufacturers grade for the material was MII/2-E3. The material was manufactured by Lignostone.

The Glass Fibre Reinforced Polymer (GFRP) rods used throughout this study are an off-the-shelf product. They are manufactured using an automated and continuous pultrusion process. In this process, dry fibres impregnated with a low viscosity, liquid, thermosetting resin are guided into a heated die where they are cured to form the desired section shape (Bank, 2006). The dowels used in this study are made using a polyester, thermoset matrix and E-glass fibres. E-glass is a borosilicate glass and is so named for its high electrical resistivity. This type of glass fibre is used in the vast majority of glass fibre FRP materials (Bank, 2006). An additional type of glass (S-glass) is also used as a reinforcement in FRP materials. However, this ‘structural’ S-glass is primarily used in specialist applications within the aviation industry due to cost. Polyester, thermoset resins are cured through heating in the production process and are not softened by reheating. This is an important consideration for the fire resistance of connections.
4.2 Moisture content and density of LVL and Douglas fir glulam

The moisture content and dry density of the timber used in this study was determined by oven drying specimens selected at random. Ten glulam and ten LVL specimens were cut from material that was stored in a climatically controlled room at $20 \pm 3 \degree C$ and $65 \pm 2\%$ relative humidity prior to testing. The samples were dried in an oven at a temperature of $105 \degree C$ until a constant mass was recorded. The moisture content was determined using equation 4.1, below:

$$MC = 100 \left( \frac{W_w - W_d}{W_d} \right)$$ (4.1)

Mean average values for the moisture content and dry density of the glulam and LVL are presented in Table 4.1. Characteristic dry density values are also provided and these were calculated in accordance with BS EN 14358 (2006).

<table>
<thead>
<tr>
<th>Material</th>
<th>Moisture Content (%)</th>
<th>Dry density $(kg/m^3)$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>COV</td>
</tr>
<tr>
<td>D. fir</td>
<td>13.1</td>
<td>3.8</td>
</tr>
<tr>
<td>Glulam</td>
<td>14.3</td>
<td></td>
</tr>
<tr>
<td>Kerto-S</td>
<td>12.8</td>
<td>6.6</td>
</tr>
<tr>
<td>LVL</td>
<td>14.3</td>
<td></td>
</tr>
</tbody>
</table>

4.3 Embedment strength and stiffness of Douglas fir glulam

The embedment strength and stiffness response of the Douglas fir glulam used for connection tests was determined using a simplified version of the method outlined within BS EN 383 (2003). The test setup described by BS EN 383 (2003) uses bespoke steel apparatus to load a dowel that passes through a hole in the timber specimen. However, the test method employed in this study used a simplified approach of loading the dowel in a half hole as described by Wilkinson (1991). The test setup is shown in Figure 4-1 for a perpendicular to grain specimen. Tests were completed for load orientations parallel and perpendicular to the timber grain direction.
Specimens were tested in a universal Dartec loading machine and load resistance and cross-head displacement were logged throughout the testing. Based upon initial exploratory tests a load rate of 0.3 mm/minute was used to ensure that failure was reached in approximately 300 seconds. Failure was defined as the point at which either maximum load resistance was reached or the displacement reached 5 mm. In accordance with BS EN 383 (2003) an initial cycle of load was applied between approximately $0.4F_{\text{max}}$ and $0.1F_{\text{max}}$ and the load was held for 30 seconds at each change of load direction. The parallel to grain specimen dimensions (width x height x thickness) were 72 x 80 x 50 mm and the perpendicular to grain specimen dimensions were 240 x 60 x 50 mm. The dowel used to embed into the timber was a 12 mm diameter stainless steel dowel. Five repeat tests were completed for each grain orientation.

The mean average and fifth percentile characteristic values for the glulam embedment strength and foundation modulus are summarised in Table 4.2. The characteristic values were calculated in accordance with (BS EN 14358, 2006). The presented values are specific to the use of a 12 mm diameter dowel and were calculated accordingly. The embedment strength was calculated as the maximum load resistance divided by the loading area of the dowel, where the loaded area is the dowel diameter multiplied by the timber thickness. In accordance with BS EN 383 (2003) where the maximum load was not reached before 5 mm displacement the load at 5 mm displacement was used instead. The foundation modulus is defined as the load resistance per unit displacement per unit dowel length (stiffness/unit dowel length) in the elastic loading range. Connection stiffness was determined as the gradient of the straight line that passed through the load slip plot at points corresponding to $0.1F_{\text{max}}$ and $0.4F_{\text{max}}$. For this case the stiffness was determined from the second application of load associated with the cyclic loading described.
previously.

### 4.4 GFRP-DVW bearing stiffness

Knowledge of the combined bearing stiffness of a GFRP dowel loaded by a DVW plate is necessary to predict the elastic slip response of a GFRP-DVW connection. Church and Tew (1997) presented a method for the determination of the bearing strength of timber carpentry connections using the test setup shown in Figure 4-2. This test setup was developed to overcome the difficulty of characterising the combined load resistance of a timber dowel bearing onto a timber foundation. A test method based upon this setup was adopted for the determination of a GFRP-DVW bearing stiffness value.

Figure 4-2: Test setup used by Church and Tew (1997) for testing timber peg embedment strength

The test setup and specimen orientation is shown in Figure 4-3. Testing was completed for 12 mm diameter GFRP dowels and 10 mm thick DVW plate. The DVW plate was fabricated from a single piece of material by drilling a central 12 mm diameter hole and cutting the sheet in half using a band saw. This ensured that there was minimum discontinuity between the top and bottom plates and the width of the saw cut provided the compression space in the specimen. The GFRP dowels were cut to a length of 50 mm and loaded at their mid point. Testing the
### Table 4.2: Embedment strength and foundation modulus of Douglas fir glulam

<table>
<thead>
<tr>
<th>Grain orientation to load</th>
<th>Embedment strength ($N/mm^2$)</th>
<th>Foundation modulus ($N/mm^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean avg.</td>
<td>Characteristic</td>
</tr>
<tr>
<td>Parallel</td>
<td>31.0</td>
<td>23.9</td>
</tr>
<tr>
<td>Perpendicular</td>
<td>25.1</td>
<td>22.2</td>
</tr>
</tbody>
</table>
dowels in this way allowed the influence of edge effects to be empirically included in the final stiffness value. Further testing is required to confirm the significance of edge effects on this testing method as well as the influence of using thicker DVW plate. Test specimens were loaded at a rate of 0.2 mm/minute until the gap in the DVW plate was closed. Five repeat specimens were tested.

Figure 4-3: GFRP-DVW bearing stiffness test setup

In a connection, the GFRP dowel resists a unidirectional load from the central DVW plate and the reaction to this load comes from the timber side members. Considering the unidirectional loading of a connection, a stiffness value suitable for connection slip analysis can be determined by modeling the total experimental system stiffness as two springs in series. Therefore the GFRP-DVW bearing stiffness for a single load direction is equal to twice the system stiffness measured experimentally.

The experimental stiffness was determined from the recorded load displacement response and was taken as the gradient of the line that passed through the plot at points corresponding to loads of $0.3F_{\text{max}}$ and $0.7F_{\text{max}}$. These points were selected to minimise the influence of initial fit in the test specimens. $F_{\text{max}}$ is the maximum load recorded at test termination, which corresponds to a total displacement of approximately 1 mm. The experimental results are summarised in Table 4.3. A mean average bearing stiffness of 40.0 kN/mm was determined for the five test specimens. In accordance with BS EN 14358 (2006) a 5% characteristic value of 30.0 kN/mm was calculated.

<table>
<thead>
<tr>
<th>Experimental system stiffness (kN/mm)</th>
<th>Unidirectional mean embedment stiffness (kN/mm)</th>
<th>Unidirectional characteristic embedment stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean  20.0</td>
<td>COV (%)  11.0</td>
<td>Range  16.9 – 21.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>40.0</td>
</tr>
</tbody>
</table>

72
CHAPTER 4. CHARACTERISATION OF CONNECTION COMPONENTS

4.5 Determination of E & G moduli for GFRP dowels

In order to predict the elastic deformation of a dowelled timber connection, knowledge of the dowel’s elastic moduli are required. For metallic materials it is typically only the flexural modulus which is of concern since the deformation of the dowel attributed to shear is generally considered negligible. However, unlike steel, which is an isotropic material, pultruded GFRP dowels are orthotropic and the inclusion of deformations due to shear becomes more pertinent.

In the case of anisotropic materials such as GFRP a degree of material flexural anisotropy can be defined to give insight into the influence of shear deformation. Where flexural anisotropy is equal to the ratio of the flexural Young’s modulus (E) to the shear modulus (G) (anisotropy ratio = E/G) it can be seen that as this ratio increases so too will the deformation due to shear. If a comparison is considered between mild steel, which has an anisotropy ratio of approximately 2.6 (Cob, 2004), and FRP pultruded sections, which have a ratio in the range of 18-30 (Bank, 1989), it becomes evident that this increased deformation will be significant for GFRP dowels.

In addition to the influence of shear deformation in GFRP dowels, the value of the flexural modulus, $E$, has been shown to be different from the longitudinal modulus determined through tensile tests (Bank, 1989). In this sense the flexural modulus of a GFRP dowel can be considered as a specific mechanical property and not a pure material property as assumed for metallic materials. Bank (1989) developed a technique for the simultaneous determination of the in-plane shear modulus and flexural modulus of FRP sections known as the graphical method. The method uses a graphical representation of the Timoshenko beam equations by expressing them in the form $y = mx + c$ and plotting the results for three point bending tests completed at different spans. The elastic constants $E$ and $G$ can then be determined from the gradient and intercept of the plot respectively. The GFRP dowels used in this study were tested using the graphical method. However, it is noted that for the determination of shear modulus, the method is sensitive to experimental error and variation in the gradient of the regression line plotted between data points. In this instance the shear modulus values of the GFRP dowels were found to be unreliable when compared with published values. Therefore, a value of $G$ taken from the literature was assumed. Nonetheless, the method and results of the graphical method tests are reported as the method does provide a reliable means of accurately determining the flexural modulus of the dowel material.
CHAPTER 4. CHARACTERISATION OF CONNECTION COMPONENTS

4.5.1 Graphical method

For a beam in three point bending the central deflection is calculated using equation 4.2 according to Timoshenko beam theory. It should be noted that a shear coefficient is not included in this case as it is incorporated into the shear modulus, which is considered a mechanical property of the dowel itself and not a general material property.

\[ s = \frac{Pl}{4} \left( \frac{l^2}{12EI} + \frac{1}{AG} \right) \]  (4.2)

where:

- \( s \) is the central displacement of the dowel
- \( P \) is the applied central load
- \( l \) is the span of the dowel
- \( E \) is the flexural modulus
- \( G \) is the shear modulus
- \( A \) is the cross sectional area

This equation can be rearranged into the linear form:

\[ \frac{4sA}{Pl} = \frac{1}{3E} \left( \frac{l}{r} \right)^2 + \frac{1}{G} \]  (4.3)

where the appropriate substitution for the section moment of inertia \((I = \pi r^4/4)\) and cross sectional area have been made.

Therefore, plotting \((4sA/Pl)\) on the y axis and \((l/r)^2\) on the x axis for multiple data points allows E and G to be determined from the straight regression line passing through the points. This is more clearly described in Figure 4-4.

From inspection of equation 4.3 it is apparent that, in theory, testing of only two different spans is required to determine values of E and G. However, in practice, due to the sensitivity of the method to variations in the gradient of the regression line, the testing of a greater number of spans is required to improve the reliability of the method. For the determination of E and G for the GFRP dowels used in this study, five different spans were tested to reduce potential error in the gradient of the regression line between the points. For each span three different dowels were selected at random for testing. The dowels were tested using a Dartec universal testing machine at a constant displacement rate of 0.5 mm/min. Load and displacement readings were logged throughout all of the tests.
4.5.2 Results and discussion

The results of testing the five different spans outlined in Table 4.4 are presented in Figure 4-5. The linear regression line, passing through the points is also shown with its equation. In Figure 4-5 all fifteen sets of results are plotted.

From the equation of the straight line given in Figure 4-5 the flexural modulus of the GFRP dowels can be calculated as 51.28 GPa and the shear modulus as 0.78 GPa. The corresponding anisotropy ratio of E/G therefore has a value of 65. This is significantly higher than values determined from published results (Bank, 1989; Mottram, 2004). At this point it is necessary to discuss the reliability of these results with regards to those published in literature and the shortcomings of the graphical method in terms of reliably determining a shear modulus value.

The value for the flexural modulus is considered to be reliable when compared to values presented in literature (Harvey et al., 2000; Mottram, 2004; Bank, 2006) and the use of the graphical method in determining this property is considered reliable in the literature (Mottram, 2004). It should however be noted that the
value presented here is higher than those often given by manufacturers, and this is attributed to the inclusion of shear deformation in the values reported in these sources. The influence of shear, on flexural modulus values reported with shear deformation included, is best described by Figure 4-6. This figure shows the results of three point bending tests completed on GFRP dowels by Harvey et al. (2000). The influence of shear deformations on the reported flexural modulus can clearly be seen for the short spans tested.

Unlike the flexural modulus, the shear modulus of FRP materials and the methods by which it is determined vary considerably. In his review of shear modulus
values for FRP pultrusions Mottram (2004) states that different test methods have reported shear moduli values in the range of 1.3 to 5.1 GPa (though in the original graphical method paper presented by Bank (1989) a modulus as low as 0.75 GPa is reported). Evidently the value of 0.78 GPa reported in this section for GFRP dowels lies significantly outside of the range given by Mottram (2004) and can only be considered valid within the context of the graphical method results reported by Bank (1989). For this reason, the reported shear modulus value of 0.78 GPa was not considered reliable or representative for analysis purposes. An alternative value was therefore selected based upon values reported in literature.

The apparent lack of consistency in the elastic shear modulus of pultruded sections is discussed in detail by Mottram (2004). Error arising from the use of the graphical method is attributed to sensitivity to small variations in the gradient of the regression line and additional, systematic error can be introduced through factors such as compression of the specimen under the loading points and inaccuracies in the test span. Systematic error such as compression of the dowel has the effect of raising the entire regression line, and hence lowering the value of G. The circular cross section of the dowels tested in this case suggests that compression of the specimen under the loading points will have been significant.

The experimental determination of in-plane shear modulus values for FRP materials is complex due to lack of agreement between published values and the test methods used. Beyond the graphical method, reported alternative methods include the Isopescu test method or torsional loading of coupons (Munjal, 1984). The Isopescu test method was designed for testing plate specimens and is therefore not suitable for dowel materials (Bank, 1990). In-plane shear modulus values are presented by Mottram (2004) for torsion tests and graphical method tests. Mottram (2004) states that there has been no reported incidents of the graphical method predicting G values higher than 3 GPa and that this is surprising given the sensitivity of the method to error. This is particularly significant given that G values reported from alternative methods show that although the material, in-plane, shear modulus does not possess a single value, they are in the range of 3 to 5 GPa. This suggests that the graphical method consistently under-predicts the shear modulus of FRP materials. Therefore, excluding graphical method values as unreliable, Mottram (2004) continues by stating that a value of 3GPa (corresponding to the lowest values reported from alternative test methods) is suitable for incorporation into design manuals as its application in design calculations will provide conservative values for deflections even before safety factors have been applied. Therefore for the stiffness analysis of GFRP-DVW connections in Chapter 7 the in-plane shear modulus will be taken as 3GPa. This provides an anisotropy value (E/G) of 17 for the experimentally determined flexural modulus of 51.28 GPa, which is in agreement with values presented in literature.
Future work on the determination of the in-plane shear modulus should focus on the use of torsion tests. The use of the graphical method can only be considered appropriate for finding the flexural modulus of dowel materials. The use of the Isopescu test is not suitable for dowel materials due to the required specimen dimensions for testing.

### 4.6 Effective bending resistance of GFRP dowels

This section presents a novel method for the characterisation of an effective bending resistance value for GFRP dowels. This material resistance property is subsequently used in the application of Eurocode 5 strength analysis methods in Chapter 8.

The strength analysis of timber connections made with metallic dowel type fasteners typically uses the method originally proposed by Johansen (1949). This method uses connection yield expressions, which now form the basis of the EYM strength assessment model given in BS EN 1995 (2004). The mechanics of the EYM expressions are presented in Chapter 2 together with the failure modes associated with connections made using dowels and a central flitch plate. Three ductile failure modes are defined and they are repeated in Figure 4-7 for reference.

![Figure 4-7: EYM failure modes for timber dowel connections made with thick central steel plate](image)

The failure characteristics of GFRP-DVV connections made with varying dowel slenderness are reported in Chapter 3. In each instance a single dowel failure mode
was observed. A close up of the dowel failure mode is shown in Figure 4-8 for a connection that was locked at maximum displacement and dissected.

![Figure 4-8: Close up of GFRP-DVW connection failure](image)

The GFRP dowel failure mode shown in Figure 4-8 most closely matches the four hinge EYM failure mode shown in Figure 4-7. Hence, the application of the EYM expression for this ‘mode III’ failure was considered practical. However, careful consideration of the mechanics used by the EYM are required for the successful application of this model. To provide understanding of the mechanics and assumptions of the EYM, a derivation for the expression of mode III failure is given below. The derivation is made in reference to Figure 4-9, which illustrates the mode III connection failure and also provides terminology for the symbols used in the derivation. Additionally, the same material load-response assumptions as those described in Chapter 2 are used for this derivation.

Taking moments about the dowel hinge marked as ‘A’ in Figure 4-9 gives:

$$2M_y = f_h d \frac{b^2}{2}$$

(4.4)

Rearranging equation 4.4 for ‘b’ gives:

$$b = 2 \sqrt{\frac{M_y}{f_h d}}$$

(4.5)
Figure 4-9: EYM mode III failure for connection made with thick steel plate

- $R$ = resistance per shear plane
- $f_h$ = embedment strength
- $M_y$ = dowel bending resistance
- $t$ = side member thickness
- $d$ = dowel diameter

Schematic of bearing stress under dowel:

$R = f_h \cdot db$
Hence, resolving vertically and substituting for ‘b’ gives an expression for connection capacity per shear plane:

\[ R = f_h db = 2\sqrt{M_y f_h d} \]  

(4.6)

The mechanics of the EYM expression, derived above, rely heavily on the assumption that plastic hinges of known moment capacity form at four points along the dowel length. Equating moments at the connection interface (Equation 4.4) allows simultaneous equations to be set up for the derivation of connection capacity. Knowledge of the dowel bending resistance is therefore required for the prediction of connection resistance. For metallic dowels the use of plastic hinges for analysis is well understood and the isotropic structure of metallic materials allows values of moment resistance to be reliably calculated from tensile yield strength characteristics. The use of anisotropic materials such as GFRP means that analysis, which incorporates bending, is not straightforward.

The mode in which a GFRP dowel fails will determine whether or not any degree of plasticity can be assumed. When tested at large spans, GFRP dowels fail in a very brittle manner as a result of interlaminar shear along the neutral axis (Figure 4-10). However, in timber connections GFRP dowels appear to form notional hinges at four points along their length (Figure 4-8). In this instance a degree of plastic bending resistance in the dowel is assumed on the basis of the ductile connection failure response reported in Chapter 3. This variance in dowel failure mode is due to the effect of shear loads on anisotropic materials such as FRP or timber (Bank, 1989; Shanks, 2005). In this instance the recorded bending strength at failure, and mode of failure, is sensitive to the span over which the material is loaded (Pedersen, 2002). Hence, if the EYM mode III failure expression described above is to be adapted for the analysis of GFRP-DVW connections a reliable means of characterising the bending resistance of a GFRP dowel is required.

4.6.1 Previous work

Previous work on the adaptation of the EYM for use with GFRP dowels is discussed in detail in Chapter 2. Initially Drake and Ansell (2000) used a direct substitution of \( M_y \) for a ‘cross breaking’ shear strength value. The cross breaking strength of the dowel was determined by shearing the dowel across the fibre direction with a steel guillotine test apparatus. However, there is no mechanical basis for substituting a bending resistance with a shear resistance value. Subsequently an alternative method was proposed that was based upon the interlaminar shear strength of the dowel. This method used an expression for the maximum shear stress in the dowel as a critical limit for predicting yield. However the derivation of the maximum
stress was based on pure bending of the GFRP rod which is not representative of
the loading in a timber connection.

Pedersen (2002) proposed the use of a linear relationship to derive a bending
capacity reduced by a factor related to the shear loading of the dowel. However,
as presented in Section 2.2 in Chapter 2 the assumed linear relationship required
significant extrapolation of test results, which makes the reliability of the model
questionable. Additionally, the model relied on data from three point bend tests
on GFRP dowels using various different support spans. Exploratory three point
bending tests, completed as part of this study were seen to fail as a result of
interlaminar shearing along the neutral axis plane. This failure mode is shown in
Figure 4-10 for one of the test specimens. It is evident from inspection of the GFRP
dowel failure mode shown in Figure 4-8 that the dowel failures in the three point
bending test vary considerably from the mode in the connection specimens. The
dowel in the timber connection shows a high level of interlaminar shear failure local
to the plate interface whilst the rest of the dowel appears undamaged. However the
dowel tested in three point bending has a single interlaminar shear failure along
half the dowel length. Hence the bending strength values determined, by Pedersen
(2002), from three point bending tests are based upon the limits of catastrophic
neutral axis failure. This is not considered valid for the strength analysis of the
dowel failure observed in the dissected GFRP-DVW connection.

Figure 4-10: Three point bending test on GFRP dowel

4.6.2 Novel testing method

For the specific loading conditions encountered in a timber connection, an
alternative test method was developed to characterise the effective bending
resistance of GFRP dowels. Initially three point bending tests were completed on 12 mm diameter GFRP dowels at a span/diameter ratio of 10. These tests consistently resulted in interlaminar shear failure of the dowel along the neutral axis (Figure 4-10). This failure mode is brittle and does not reflect that observed in the dissected specimens. Inspection of the dowels in the dissected test specimens showed a dowel embedment length of approximately 1.5d; where d is the dowel diameter. Virtual supports, or points of rotation, were therefore assumed to act at a maximum distance of 1.5d from the connection plate. Based upon this observation, testing of 12 mm diameter GFRP dowels over a shear span of 3d were completed.

Failure of the dowel occurred as a result of interlaminar shear failure in one half of the dowel, Figure 4.11(a). This interlaminar shear failure reflected the dowel failure
observed in the dissected connections, however, the asymmetric dowel failure shape did not. The asymmetric failure was attributed to variance in the support spacing and the absence of horizontal restraint in the central section of the dowel. A novel test method using a thick central steel plate was therefore developed. This test setup is shown in Figure 4.11(b). The shear span over which the dowel was tested was 1.5d and the dowel passed through a hole in the plate that was the same diameter as the dowel. The use of the plate provided a more uniform loading on the central portion of the dowel and the restraint provided by the plate caused failure in both sides of the dowel. The dowel failure mode closely reflects that observed in the dissected connection specimens (Figure 4.8).

It is important to highlight that the dowel failure shown in Figure 4.11(b) is an unconventional failure shape in comparison to the failure mode that might be expected of an isotropic metallic dowel (Figure 4-12). This is because interlaminar shear failure of the GFRP dowel defines the failure, as described in Figure 4-14. Metallic materials have comparably high shear stiffness and strength and so bending modes of failure are most common. If bending had dominated the failure then only two hinges would be visible and the ends of the dowel would have risen up to form a ‘v’ shape failure.

![Figure 4-12: Expected failure mode of metallic dowel compared with a GFRP dowel](image)

Three repeat tests were completed for the specimen setup shown in Figure 4.11(b). The load-displacement responses recorded from these tests are presented in Figure 4-13 and the dowel failure behaviour is illustrated in Figure 4-14. The initial load response of the dowels was elastic up to a load of approximately 5 kN. At this point a drop in load is observed followed by a similar drop at approximately 1.7 mm displacement. These two steps in the the load response are attributed to the interlaminar shear failure of the dowel either side of the central plate. At increased displacement the load capacity of the dowel plateaus at approximately 4.5 kN. The load response suggests that after the initial interlaminar shear failure of the dowel a degree of plastic load capacity is mobilised through bending resistance of the dowel.
CHAPTER 4. CHARACTERISATION OF CONNECTION COMPONENTS

at the four hinge positions and through friction between the delaminated sections of GFRP.

Figure 4-13: Load-displacement response of GFRP dowel loaded in novel test setup (Points A, B, C correspond to diagrams of Figure 4-14)

An empirical method, based upon an energy model, can be applied to the dowel failure shown in Figure 4.11(b). The energy approach allows the modes of energy dissipation within the failed dowel to be modeled as an effective bending resistance, $M_{eff}$. Assuming ideal elastic-plastic behaviour for the dowel load response the empirical bending capacity of the dowel can be determined by equating the internal energy dissipated (ED) through rotation of the dowel at the four hinges to the external work done (WD) (Figure 4-15).

The energy dissipated within the system is calculated as:

$$ ED = 4M_{eff} \theta $$  \hspace{1cm} (4.7)

The external work done by the dowel is calculated as the plastic load resistance, $P_p$, multiplied by the central deflection of the dowel, $\delta$. The definition of the plastic load resistance is described in detail in subsection 4.6.4.

$$ WD = P_p \delta $$  \hspace{1cm} (4.8)
Figure 4-14: Failure response of GFRP in novel test setup (Points A, B, C correspond to those on Figure 4-13)

A) Initial loading of dowel under combined shear and bending. The fibre-matrix interface is undamaged at this stage.

B) Interlaminar shear failure occurs between the loading plate and support.

C) Under continued load, work is done overcoming friction between the fibre shear planes and the collective bending resistance of the fibre bundle at each end of the failed zone.
Where, for small deflections, $\delta = x\theta$ and $x = 1.5d$ for the tests completed.

Therefore equating the work done on the system to the energy dissipated within the dowel gives the equation below:

$$M_{\text{eff}} = \frac{3P_p d}{8}$$  \hspace{1cm} (4.9)

This equation can be used to determine $M_{\text{eff}}$ for the four hinge GFRP failure mode observed in the dissected test specimens shown in Figure 3-8 in Chapter 3.

### 4.6.3 Testing programme

A testing programme was completed to determine the value of $M_{\text{eff}}$ for three diameters of GFRP dowels. The dowel diameters tested were 8, 12 and 16 mm. The test setup shown in Figure 4-16 was used in each instance and the dowels were loaded at a rate of 0.3 mm/minute so that failure of the dowels occurred at approximately 300±120 seconds from the start of test. The tests were completed using a universal Dartec loading machine and the displacement of the plate and load resistance of the dowel were logged throughout. Five repeat tests were completed for each dowel diameter.
CHAPTER 4. CHARACTERISATION OF CONNECTION COMPONENTS

(a) Test setup for characterisation of $M_{eff}$

(b) 10 mm diameter GFRP dowel test

Figure 4-16: Test setup - effective bending capacity of GFRP dowels
4.6.4 Experimental results

Values of $M_{eff}$ were calculated for each dowel diameter using equation 4.9 and are presented in Table 4.5. In order to do this mean average values of effective plastic load capacity were derived from the recorded load slip results. The method used to determine $P_p$ is set out in Figure 4-17 and is a modified approach proposed for this particular test. The method is a modification of the 5% offset method detailed previously in Chapter 3. The approach defines $P_p$ by defining a stiffness gradient as the line which passes through the load slip plot between points corresponding to $0.1F_{max}$ and $0.4F_{max}$. This line is then offset by 10% of the the dowel diameter and the intercept of this line with the load slip plot is taken as $P_p$. An offset of 10% was used to reduce the influence of the initial stepped yield response of the load slip plot on the recorded value of $P_p$. The stepped response of the load slip plot at this point is associated with the initial interlaminar shear failure of the GFRP dowel. Beyond this point the effective bending resistance of the dowel is assumed to have become mobilised.

<table>
<thead>
<tr>
<th>Dowel diameter (mm)</th>
<th>Plastic load resistance, $P_p$ (kN) Mean</th>
<th>COV (%)</th>
<th>Range</th>
<th>$M_{eff}$ (Nmm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>2.6</td>
<td>2.3</td>
<td>2.5 - 2.7</td>
<td>7728</td>
</tr>
<tr>
<td>12</td>
<td>4.7</td>
<td>4.3</td>
<td>4.4 - 4.9</td>
<td>21204</td>
</tr>
<tr>
<td>16</td>
<td>9.0</td>
<td>2.4</td>
<td>8.6 - 9.2</td>
<td>54084</td>
</tr>
</tbody>
</table>
4.7 Concluding comments

Material tests have been completed to characterise the properties of the component parts of a GFRP-DVW connection. In certain instances, novel methods of testing have been developed for the measurement of specific material characteristics. The main conclusions that can be drawn from the work presented in this chapter are as follows:

- GFRP dowels were observed to fail in a unique ‘flat-end’ manner.
- The failure of the dowel is characterised by interlaminar shear over the entire cross section of the dowel.
- Dowel failures observed in three-point bend tests are shown to be non-representative of the dowel failure mode observed within a connection.
- An effective dowel moment capacity has been determined for analysis using a novel testing method and plasticity analysis.
- Elastic properties of GFRP dowels were measured using a graphical interpretation of results. The method is highlighted as being unreliable for the determination of shear modulus values. A shear modulus value of 3 GPa is taken from literature for analysis purposes.
- The stiffness of DVW plate loading a GFRP dowel was determined using a novel test method and modeling the system as two springs in series.
Chapter 5

Experimental study of GFRP-DVW connections

5.1 Introduction

The performance of GFRP-DVW connections subject to pull-out loading parallel and perpendicular to grain was investigated experimentally. An extensive series of tests were completed to investigate the strength, stiffness, dowel load share and failure modes of connections to be understood. The test program also investigated the reduction of EC5 dowel spacing rules for parallel to grain loading in order to explore the potential for increasing connection load capacity per connected area of timber. An appropriate reduction of spacing rules parallel to grain is discussed as a result of these tests. Following exploratory calculations into the limits of perpendicular to grain timber splitting capacity, such a reduction was not investigated perpendicular to grain. Test results for connections using EC5 spacing rules are presented to provide insight into load response and perpendicular to grain splitting failure. To verify the performance of reduced spacing rules parallel to grain, full scale tests were completed for equivalent GFRP-DVW and metallic dowel-plate connections. These full scale tests are reported in Chapter 6.

Design of non-metallic connections made with GFRP and DVW is not addressed by Eurocode 5. Development of this connection type must therefore consider:

- Load share of multiple dowels
- Spacing of multiple dowels
- Characterisation of brittle failure modes
- Analysis of the connection strength and stiffness
CHAPTER 5. EXPERIMENTAL STUDY OF GFRP-DVW CONNECTIONS

The objective of the experimental investigation reported in this chapter was to provide clear insight into the first three points above. The test results provide a means of validation for the analysis methods presented in Chapters 7 and 8.

For clarity the experimental work reported in this chapter is set out in distinct sections. Sections 5.2 and 5.4 introduce the respective testing programmes designed to investigate dowel load share, spacing of dowels, and brittle failure modes. Sections 5.3 and 5.5 present results and strength and stiffness performance of the connections is discussed within the context of dowel load share.

Of the 50 specimens that are reported in this chapter only two contained knots within the volume of timber loaded by the GFRP dowels and in only one of these cases did the dowel pass through the knot (test group ‘a’, Table 1). Knots were intentionally excluded to reduce the variability of experimental results. No significant difference in failure loads was observed in the two specimens containing knots but a significant increase in connection stiffness was noted in the specimen from group ‘a’. This is discussed in detail in the results section for parallel to grain tests.

The fabrication methods used aimed to ensure that a high level of dimensional accuracy and consistency was achieved for each specimen. Therefore test results recorded for different configurations were comparable and variation between repeat tests was minimised. For the parallel and perpendicular to grain specimens the glulam side members were planed to the correct thickness for each test specimen using a planer-thicknesser and the dowel holes were drilled using a vertical axis pillar drill. To ensure accurate alignment of dowels, in the multiple fastener connections, a single location hole was drilled in the DVW plate and a single dowel installed. Subsequent holes were then drilled using the holes, previously drilled in the member, as guides. This process is illustrated in Figure 5-1 where the drilling of the plate and the soft headed hammer used to install the dowels can be seen. The ability to drill the plate in this way is a major advantage of this connection type. Metallic plates must be pre-drilled and achieving accurate alignment with the holes in the timber member can be difficult. For the full scale connections (reported in Chapter 6) the timber members were supplied with the plate slot pre-cut in the ends. Hence, for the non-metallic specimens the drilling of the specimens was completed using the process above. However, for the metallic specimens prefabrication of the steel flitch plates was necessary and significantly more complex than for the non metallic specimens. The fabrication of the full scale test specimen is discussed in detail in Chapter 6.
5.2 Parallel to grain testing

A total of 35 pull-out tests parallel to grain were completed to determine the strength, stiffness, and connection response to failure. Emphasis was placed on testing connections made with reduced EC5 spacing rules parallel to grain. Testing of GFRP dowels at reduced spacings was undertaken in recognition of their lower individual double shear capacity when directly compared with metallic dowels. The lower individual capacity of GFRP dowels means that under increased loading the dowel failure capacity can be more safely reached prior to timber splitting. Additionally, like a nailed connection, individual dowels are better able to move with the wood, which gives better load distribution within a multiple fastener connection. Under testing, connector groups made up of stiff metallic fasteners can often cause brittle failure through splitting the connected structural element prior to yielding of the fasteners (Quenneville, 2009). In some instances this can be attributed to the presence of a high point load under a fastener due to poor load distribution within a fastener group. In these instances the strength of the connection is limited by the timber and not the dowel, which represents an inefficient use of material as well as providing limited connection ductility.

The testing regime for the parallel to grain tests is set out in Table 5.1. Five specimens were tested for each group and the rationale for the testing regime is discussed below. The results and analysis of the experimental programme are presented in section 5.3.

The test setup used to load the specimens to failure is shown in Figure 5-2. All of the specimens were loaded in tension under a constant displacement controlled rate of 1mm/minute until ultimate failure of the specimen occurred. The loading rate was based on connection load response of the specimens reported in Chapter 3. In accordance with BS EN 26891 (1991) the load rate was selected with the intention of the majority of specimens exhibiting yield failure between 120-300 seconds after
the loading began. In general ultimate connection failure was reached in 3 to 10 minutes, equivalent to an instantaneous action in EC5 (BS EN 1995, 2004). Ultimate connection failure was defined as complete loss of load resistance or a connection slip in excess of 15 mm in correspondence with BS EN 26891 (1991). Slip of the plate relative to the side members was measured using a linear variable differential transformer (LVDT) attached to the specimen. Load was applied using a Dartec universal testing machine and both load and slip were recorded using strain smart data acquisition system 5000.

![Test setup parallel to grain](image)

**Figure 5-2: Test setup parallel to grain**

### 5.2.1 Dowel load share

Due to variation in hole tolerances, and the initiation of timber splitting prior to connection yield, the capacity of a connection made with multiple metal fasteners in line rarely equals the sum of the individual fastener capacities. However, connections made with GFRP dowels are less prone to splitting than metallic fasteners and DVW plate can be made with high dimensional tolerance. Thus load share for multiple GFRP fasteners in line was expected to offer an improvement over equivalent metallic fasteners.
### Table 5.1: Test specimen configurations (dowel diameter, d = 12 mm for all tests)

<table>
<thead>
<tr>
<th>Test Group</th>
<th>Side member cross section (b/t) (mm)</th>
<th>Number of dowels (N)</th>
<th>End distance ( (a_{3,t}) )</th>
<th>Dowel spacing ( (a_1) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>96 x 48</td>
<td>1</td>
<td>5d</td>
<td>N/A</td>
</tr>
<tr>
<td>b</td>
<td>96 x 48</td>
<td>3</td>
<td>5d</td>
<td>5d</td>
</tr>
<tr>
<td>c</td>
<td>96 x 48</td>
<td>3</td>
<td>4d</td>
<td>4d</td>
</tr>
<tr>
<td>d</td>
<td>96 x 48</td>
<td>3</td>
<td>3d</td>
<td>3d</td>
</tr>
<tr>
<td>e_i</td>
<td>96 x 48</td>
<td>3</td>
<td>5d</td>
<td>3d</td>
</tr>
<tr>
<td>e_ii</td>
<td>96 x 48</td>
<td>3</td>
<td>3d</td>
<td>5d</td>
</tr>
<tr>
<td>f</td>
<td>96 x 75</td>
<td>3</td>
<td>3d</td>
<td>3d</td>
</tr>
</tbody>
</table>

(EC5 values: \( a_{3,t} = 7d, a_1 = 5d \))

Test groups a and b (Table 5.1 were completed to investigate the extent that GFRP-DVW connections share load between multiple dowels in line. A multiple dowel configuration of three dowels in line was chosen for comparison with the single dowel tested in group ‘a’.

### 5.2.2 Reduced fastener spacings

Test groups ‘c’ and ‘d’ investigated the incremental reduction of fastener spacings parallel to grain. These tests were conducted to determine the minimum in line fastener spacing that would still provide connection ductility beyond yield. Test group ‘b’ represents a lower bound application of EC5 spacing rules, as it uses a reduced end distance of 5d as opposed to the 7d required by the code. The rationale for using equal end and dowel spacings was driven by reported observations of the ultimate failure modes of metallic dowel connections loaded parallel to grain. In their study of metallic bolted connections, Quenneville and Mohammad (2000) concluded that ultimate global connection failure of metallic bolts in line was initiated by a single full thickness shear plug, which then caused an unzipping of the remaining timber. Typically the plug length attributed to the ultimate failure was the lower of end distance or minimum dowel spacing for connections made in sawn timber. Therefore in testing reduced spacing rules both end and row spacing of test groups ‘b’, ‘c’ and ‘d’ were kept the same so that the area of timber mobilised at brittle failure would then be known. Testing aimed at understanding the extent to which an extended end distance influences the ultimate connection capacity of GFRP-DVW connections made with reduced spacings is discussed below in section 5.2.4.

When altering the spacing of connectors for parallel to grain loads it was important to consider brittle failure modes which are influenced by the net timber cross section retained after drilling the holes for fasteners. Two failure modes are directly related to the net cross section; net tension failure of the timber member and
fastener group tear out. EC5 spacing rules, for connectors in line, limit the spacing between lines of dowels to a minimum of 3d centre to centre and it was decided that this should be maintained if premature net tension or group tear out failures were to be avoided. The reduction of column spacings $a_{3,t}$ and $a_{1}$ (Figure 2-6) were therefore investigated with the aim of demonstrating improved efficiency of connector columns. Therefore, if a capacity equivalent to a connection made with metallic dowels is achieved, the net timber cross section retained is the same as that specified by EC5, thus not increasing the likelihood of brittle group failure or net tension failure.

5.2.3 Influence of timber thickness

Test group ‘f’ was developed to assess the influence of structural member thickness on the brittle failure mode capacity. To investigate this the specimens were designed with a small in-line dowel spacing but with a thickness that would provide a shear area of timber equivalent to a larger dowel spacing in a narrower member. The objective of this test was to confirm whether ultimate brittle capacity was more strongly influenced by in-line spacing or by the total plug shear area of connected timber in shear. The specimens of test group f therefore had a dowel and end spacing of 3d and a side member thickness of 75 mm. This increased thickness provided a full thickness shear area similar to a 48 mm thick side member (as provided for all other specimens) with a dowel and end spacing of 4.5d. This is shown in Figure 5-3. It was anticipated that the increased thickness may provide an improved ultimate resistance when compared with test group c, which had end and dowel spacings of 4d and a side member thickness of 48 mm.

5.2.4 Initiation of brittle failure

When considering the reduced spacing of GFRP fasteners it is important to consider the influence of the end distance ($a_{3,t}$) on mitigating the initiation of ultimate brittle failure. To reduce the chance of brittle failure modes prematurely initiating in connections loaded parallel to grain, EC5 minimum spacing rules prescribe an end distance ($a_{3,t}$) greater than the in-line dowel spacing ($a_{1}$). The larger end distance is provided in reference to the stress induced by the fasteners, which is typically higher in the end fastener (Jorissen, 1999). However, load share between fasteners is susceptible to variation along the line of fasteners due to natural differences in the tolerance of fastener holes, stiffness of the fasteners, and variations in timber embedment resistance. Therefore, it is not always the case that the timber under the end dowel is the most highly stressed. The result of this are the observations reported by Quenneville and Mohammad (2000) where ultimate failure is often
limited by the minimum shear capacity of either the end or in-line dowel spacing. Test groups ‘e_i’ and ‘e_{ii}’ (Table 5.1) were designed to test the extent that the ultimate failure load of a GFRP-DVW connection is influenced by the minimum end or dowel spacing. Test group b (5d end and dowel spacings) provided an upper bound and test group d (3d end and dowel spacings) a lower bound. Test group ‘e_i’ had an end distance of 5d and dowel spacing of 3d and test group e_{ii} had an end distance of 3d and dowel spacing of 5d. According to the observations of Quenneville and Mohammad (2000) it was expected that the ultimate capacity of test groups ‘e_i’ and ‘e_{ii}’ would be similar to that of test group ‘d’.

5.3 Results of parallel to grain tests

The mean average yield load, ultimate load and initial stiffness of the parallel to grain test specimens are summarised in Table 5.2. Initial connection stiffness is determined from the gradient of the line that passes through the points on the connection load slip plot, which correspond to 10% and 40% of the ultimate load. The method for determining the connection stiffness in this way is illustrated in Figure 3-2 in Chapter 3. The ultimate load is defined as the maximum load measured prior to failure up to 15 mm connection slip in accordance with BS EN
Yield strength was evaluated using the 5% offset method described in ASTM D 5652-95 (2007). This method defines the yield load as the intercept between the load-slip curve and the line of initial stiffness offset by 5% of the dowel diameter and is illustrated in Figure 3-2 in Chapter 3. Yield load, ultimate load and initial stiffness data for individual test specimens are presented in Appendix B for reference. The results in Appendix B are accompanied by a description of the post yield failure mode for the individual specimens.

<table>
<thead>
<tr>
<th>Test group</th>
<th>Yield load (kN)</th>
<th>Ultimate load (kN)</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>Mean value (%)</td>
<td>Range</td>
<td>Mean value (%)</td>
</tr>
<tr>
<td>a</td>
<td>13.2</td>
<td>10.2</td>
<td>12.0 – 15.6</td>
</tr>
<tr>
<td>b</td>
<td>39.1</td>
<td>11.7</td>
<td>35.2 – 46.8</td>
</tr>
<tr>
<td>c</td>
<td>38.9</td>
<td>5.9</td>
<td>36.5 – 41.4</td>
</tr>
<tr>
<td>d</td>
<td>32.6</td>
<td>4.5</td>
<td>30.6 – 34.4</td>
</tr>
<tr>
<td>e_i</td>
<td>33.0</td>
<td>7.3</td>
<td>28.9 – 34.9</td>
</tr>
<tr>
<td>e_ii</td>
<td>32.1</td>
<td>5.6</td>
<td>30.0 – 34.2</td>
</tr>
<tr>
<td>f</td>
<td>35.4</td>
<td>9.0</td>
<td>31.5 – 39.5</td>
</tr>
</tbody>
</table>

All of the specimens displayed a linear load-slip response up to yield of the GFRP dowel fasteners. Loading of the specimens was then continued until brittle failure of either the timber members or the DVW plate was observed. Specimens with a dowel spacing \(a_1\) of 4d or greater exhibited a linear response up to a mean average yield of 13kN per dowel. Further to the yield of the dowels, significant post yield load capacity was observed as part of an approximate bilinear load-slip response. In specimen groups ‘d’, ‘e_i’, ‘e_ii’, and ‘f’ post yield load resistance was limited by early brittle timber failure. The majority of ultimate brittle splitting failure occurred in the glulam side members, however, failure of the DVW plate was observed in three instances.

The three DVW plate failures were due to end cleavage failure (Figure 5-4) and net tension failure (Figure 5-5). End cleavage failure of the DVW plate occurred in a specimen in test group ‘a’ and this was attributed to an insufficient plate end distance of 3d (Figure 5-4). No cleavage failure occurred in subsequent tests, which all used an end distance of 4d. The cleavage failure initiated as a split in the end of the plate. The dowel then forced this split to open and the sides of the DVW plate failed under the combined effects of tension and bending loads (Figure 5-4).
tension failure of the DVW plate occurred prior to the failure of the timber side members in two specimens from test group ‘b’ (Figure 5-5). However, the loads at which these two failures occurred were significantly above the connection yield load due to the bilinear load-slip response of the connection. Nonetheless, for certain applications it will be necessary to specify plate of a higher density or increased thickness than that used in this study. Bearing failure of the plate due to dowel embedment was not observed in any of the tests.

Figure 5-4: Cleavage failure of DVW plate

Figure 5-5: Net tension failure of DVW plate

Variation in the recorded yield loads is low and the variation encountered is attributed to material inconsistencies. Greater variation was recorded for the ultimate failure loads of the specimens. This was expected, as the recorded failure capacities are significantly influenced by the mode of failure, which has many influencing factors. Similar to the recorded variation in connection yield
loads, ultimate failure capacity is influenced considerably by material variance, particularly in terms of timber ring width and orientation.

Marked variation in connection stiffness is reported in Table 5.2. In general the range of stiffness results are attributed to natural variation of the timber embedment resistance. However, specimen group ‘a’ shows a particularly high coefficient of variance for connection stiffness. This is attributed to the presence of a knot within one of the specimen side members, which resulted in the measurement of a much higher connection stiffness. The specimen is shown in Figure 5-6 where the high density of annual growth rings around the knot is clearly seen.

![Figure 5-6: Knot in side member of specimen from test group ‘a’](image)

5.3.1 Dowel load share

The mean average connection stiffness recorded for the three dowel specimens was approximately three times that recorded for the single fastener connections of test group ‘a’ (Table 5.2. This suggests that within the elastic loading range the load share between fasteners in multiple dowel connections is efficient.

In addition to load share within the elastic range, load resistance at yield was also found to be very efficient for multiple GFRP dowels in line. For connections made with three dowels and fastener spacings equal to or greater than 4d (test groups ‘b’ and ‘c’) an average yield capacity of 39 kN was recorded. This is three times the average 13 kN yield capacity recorded for the single dowel specimens tested in group ‘a’. Furthermore, significant post yield ductility was observed for these specimens (Figure 5-8).

The observed post yield ductility for specimen groups ‘a’, ‘b’ and ‘c’ is significant. By comparison the yield capacity of metal fasteners in line is limited by the factor
neff to account for splitting of the timber member prior to ductile yield of the fasteners (BS EN 1995, 2004). Unlike the observed behaviour for GFRP dowels, metallic dowels in line may therefore be designed for capacities that are defined by the splitting capacity of the timber and not ductile failure of the fasteners. Hence in certain cases an overloaded connection made with metal fasteners may fail in a brittle manner, which is an undesirable and often unpredictable failure mode. The connection load response observed in this study shows that brittle splitting of the timber member prior to fastener yield would be less likely for a GFRP-DVW connection.

5.3.2 Partial thickness shear plug

Partial thickness shear plug failure increases the ductility of the connection. Initial partial shear plug failure propagates through the glulam member under continued loading of a connection. It was expected that brittle, splitting failure of the glulam members would define the ultimate connection capacity in many cases. However, in contrast to this expectation ultimate failure was in the form of a partial thickness shear plug, which followed natural weaknesses in the timber member (Figure 5-7). This failure occurred in approximately 70% of cases. The exceptions to this mode were; failure of the DVW plate (within the connection or at the grips of the testing rig); no brittle failure prior to test termination; and mixed failure where it was unclear if splitting was the dominant mode. At the point of failure initial splitting of the specimens was observed at the connection interface during the formation of the shear plug. After the formation of the plug further splitting was often observed to propagate from the plug tip (as seen in Figure 5-7) or sudden brittle splitting would occur in the other side member as a result of load redistribution within the connection. Plug shear of the type shown in Figure 5-7 is uncommon in connections made with stiff metallic dowels and is attributed to deformation of the GFRP dowels local to the connection interface.

As a result of this failure mode ultimate connection capacity is not significantly improved by an increase in the thickness of the glulam side member. This can be seen in the failure response of specimen group f (Table 5.2). The specimens in this test group had end and dowel spacings of 3d and side member thicknesses of 75 mm. However, when compared to specimen group d, which had the same dowel spacings, but a side member thickness of only 48 mm, there was only a marginal increase of 13% ultimate load capacity for the 56% increase in shear area of timber. The lower than expected increase in capacity is attributed to the partial shear plug failure mode of the glulam. Initially plug shear local to the central plate occurred in the same way as in specimens with narrower side members. The initial failure was subsequently followed by a progression of plug shear failures across the thickness of
the member. This can be seen in the failed specimen from group 'f' shown in Figure 5-7. Although the ultimate capacity was not significantly improved through the increase of thickness, the mean average connection stiffness was approximately 24% higher than all of the other 3 dowel specimen groups (test groups 'b' – 'e ii'). This is attributed to the larger member thickness, which represents a larger foundation for the dowel to bear upon and hence suggests that there would be a lower embedment pressure underneath the dowel for equivalent loading.

Figure 5-7: Partial shear plug detail (left) and progressive shear plug failure (right)

5.3.3 Reduced fastener spacings

The load carrying capacity and the deformation behaviour of specimens made with incrementally reduced in-line dowel spacings are presented in Figure 5-8. Data are presented for the load slip response of specimens made with three dowels in line alongside that for a single dowel specimen. For clarity a single representative load slip plot was selected from the five available data sets for each specimen group. The strength and stiffness data recorded are also presented in Table 5.2 for each specimen group.

It is evident that no notable post yield ductility is provided by a connection made with an in-line dowel spacing and end distance of 3d. However for specimens made with spacings of 4d and 5d, a significant bilinear load deformation response was observed (Figure 5-8). The bilinear load-slip response is attributed to connection hardening resulting from the further embedment and deformation of the GFRP dowels at increased connection slip. Beyond the point of connection failure the stepped load-slip response of the connections was attributed to the progressive formation of partial shear plugs in the timber side members. This was ultimately followed by a complete loss of strength through splitting. The length of the
Figure 5-8: Typical load-slip plots for connections made with incrementally reduced in-line dowel spacing (12 mm diameter dowels)
shear plug, and hence the dowel spacing, clearly influences ultimate connection capacity. An in line spacing and end distance of 3d provides a low level of post yield resistance. However, for dowel spacings and end distance of 4d, significant post yield deformation and load resistance was observed and might therefore be considered as a minimum in-line spacing for GFRP dowels. The use of larger spacings improves the ultimate resistance parallel to grain as more timber must be mobilised prior to failure.

5.3.4 Initiation of brittle failure

The geometries of connections tested to investigate the initiation of brittle connection failure are shown in Figure 5-9. The connections shown on the left and right of Figure 5-9 are specimens taken from test groups ‘b’ and ‘d’ respectively. The geometries of these two specimens represent a connection that provides significant post yield load resistance at extended slip (specimen group ‘b’) and a connection that does not (specimen group ‘d’). This can be seen in the load slip plots presented in Figure 5-8.

Test groups ‘e\textsubscript{i}’ and ‘e\textsubscript{ii}’ are shown in the centre of Figure 5-9. They were designed with the intention of investigating whether an extended end distance can help mitigate brittle failure and whether the provision of an insufficient dowel spacing or end distance can initiate an ‘unzipping’ of a connection made with otherwise adequate spacings. The latter point is significant in terms of ultimate strength analysis as it confirms whether ultimate failure is the result of mobilising a single shear plug or whether the whole length fails simultaneously. The latter is assumed by EC5 Annex A for metallic connections (BS EN 1995, 2004).

![Figure 5-9: Specimens tested to investigate trigger of ultimate failure](image_url)
Figure 5-10: Influence of end and dowel spacing on ultimate failure capacity (12 mm diameter dowels)
CHAPTER 5. EXPERIMENTAL STUDY OF GFRP-DVW CONNECTIONS

The load deformation behaviour for specimen groups ‘b’, ‘e\text{i}’, ‘e\text{ii}’ and ‘d’ is shown in Figure 5-10. As for Figure 5-8 (described in subsection 5.3.3) single representative plots are displayed for each group to aid interpretation of the results.

It is evident from Figure 5-10 that the limiting effect of the 3d dowel and end distance spacing on specimen groups ‘e\text{i}’ and ‘e\text{ii}’ is significant. In both instances a rapid loss of connection capacity beyond connection yield was observed and this load slip behaviour reflected that of specimen group ‘d’, which had an end distance and dowel spacing of 3d. In spite of the significant limiting effect of a 3d dowel spacing on specimen group ‘e\text{i}’ the extended end distance did provide a 22% increase in the average ultimate connection capacity when compared to specimen group ‘d’.

This is in agreement with observed load share in metallic dowels which has shown that the end dowel is more highly stressed than subsequent dowels in line. Therefore it appears that for group ‘e\text{i}’ a higher level of stress can be resisted in the end dowel prior to the stress under subsequent dowels reaching a level that triggers plug shear and ‘unzipping’ of the connection.

The theory of a connection ‘unzipping’ as opposed to a single full length shear plug being mobilised is confirmed by specimen group ‘e\text{ii}’. In this instance the sum total of timber resisting plug shear is greater than for specimen group ‘e\text{i}’ yet the average ultimate load recorded for specimen group ‘e\text{ii}’ was 12% lower (Table 5.2). This suggests that the end plug more quickly became over stressed, limiting the capacity for distribution of load to the other dowels in line, and ultimately initiating the connection failure. Therefore, the partial plug shear failure length evidently limits ultimate load capacity in the same manner as that observed by Quenneville and Mohammad (2000) for metallic connections. Hence from these results it is suggested that large end or dowel spacings will not significantly delay the onset of ultimate brittle failure of GFRP-DVW connections if an insufficient dowel spacing or end distance of 3d is used. Accordingly, analysis aimed at determining the load at which ultimate brittle failure is initiated should focus on the resistance of a single plug and not the sum total of all loaded shear planes.

The results of this test series have demonstrated that although the initiation of brittle failure parallel to grain is restricted by the minimum length of dowel spacing or end distance, an extended end distance does allow better distribution of load along the line of dowels. This in turn can provide a more robust connection after the onset of brittle failure. Although not tested, the experimental results indicate that an in-line dowel spacing of 4d in conjunction with an end distance of 5d may be most appropriate for the use of GFRP-DVW connections loaded parallel to grain. This configuration is used in the full scale tests reported in Chapter 6.
5.4 Perpendicular to grain testing

Cantilever pull-out tests perpendicular to grain were completed to determine the strength, stiffness, and connection load response to failure. The tensile loading of timber connections in this orientation presents a particular problem of sudden brittle splitting failure. As described in Chapter 2, the brittle splitting capacity of connections loaded perpendicular to the grain direction is predicted using equation 2.7, a design equation based on linear elastic fracture mechanics. This design equation is given generally for all softwood timber species and provides a lower bound strength prediction based on the failure behaviour of the connection. The development and calibration of the model is discussed in detail in Chapter 2.

The mode of failure that defined the lower bound was found to be characteristic not of a stiff stocky dowel but of a slender dowel and the failure was characterised by a large degree of plastic deformation, dowel embedment, and hardening of the bearing timber (Leijten and Van der Put, 2004). This failure behaviour is also characteristic of that reported for GFRP dowels in Chapter 3. Therefore it was necessary to validate through testing whether the design expression given in EC5 may also provide a reliable means for predicting the brittle failure capacity of GFRP-DVW connections loaded perpendicular to grain.

Three different connection configurations were tested and they are summarised in Table 5.3. Five specimens were tested for each of the outlined configurations. These test configurations do not attempt to investigate the reduction of EC5 spacing rules on the basis of exploratory calculations into the likely perpendicular to grain splitting capacity of connections made with these spacings. The aim of the exploratory calculations was to determine the extent that splitting restricts the capacity of metal dowel connections and whether any significant improvement in the capacity of GFRP-DVW connections could be gained through the reduction of spacing rules. Hence, for minimum EC5 spacing rules, calculations for the predicted perpendicular to grain capacity and EYM capacity were compared for the connection configurations shown in Figure 5-11. The results of the calculations are also presented in Figure 5-11 and the parameters used were as follows: side member thickness = 48 mm; dowel diameter = 12 mm; embedment strength = 17.7 N/mm²; tensile strength of steel = 400 N/mm².

From inspection of Figure 5-11 it can be seen that the perpendicular to grain design splitting capacity of a metallic dowel-plate connection poses significant restrictions to the capacity of the connections. Of particular note is that the ductile dowel yield values are only calculated for a single column of dowels and any additional columns would therefore provide no significant improvement in ultimate connection strength.
CHAPTER 5. EXPERIMENTAL STUDY OF GFRP-DVW CONNECTIONS

For GFRP-DVW connections the initial testing reported in Chapter 3 indicated that, for a connection made with a 12 mm diameter dowel in softwood LVL, a connection yield capacity of approximately 10 kN followed by significant plastic deformation and hardening could reasonably be expected. With this in mind it is evident from Figure 5-11 that a reduction of EC5 minimum spacing rules would only serve to limit connection ductility arising from the observed plastic deformation. This is undesirable as it pushes the connection capacity, and hence the design capacity, towards a brittle mode. Upon this basis a reduction of EC5 spacing rules was not explored for this loading orientation.

**Table 5.3: Test specimen configurations**

<table>
<thead>
<tr>
<th>Test Group</th>
<th>Glulam cross section (mm)</th>
<th>Number of dowels (N)</th>
<th>End distance ($a_3, 1$)</th>
<th>Dowel spacing ($a_2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>g</td>
<td>84 x 48</td>
<td>1</td>
<td>5d</td>
<td>N/A</td>
</tr>
<tr>
<td>h</td>
<td>120 x 48</td>
<td>2</td>
<td>5d</td>
<td>3d</td>
</tr>
<tr>
<td>i</td>
<td>156 x 48</td>
<td>3</td>
<td>5d</td>
<td>3d</td>
</tr>
</tbody>
</table>

(EC5 minimum spacing rules were used in all cases)

Perpendicular to grain testing was completed using the test setup shown in Figure 5-12. The connections were tested as cantilevers to simulate shear loading of the glulam member at the connection location. In this way the connection strength and stiffness could be measured and any brittle splitting failure cracks, and their propagation, could be clearly examined through the entire thickness of the
connected timber at the end of the members. Load was applied at a constant displacement control rate of 1mm/min using a Dartec universal testing machine with the aim of reaching yield failure between 120-300 seconds after the loading began. In general ultimate failure was reached within 3 to 10 minutes. The slip of the connection was measured using a linear variable differential transformer (LVDT) mounted on either side of the specimen and both load and slip were recorded using strain smart data acquisition system 5000.

Figure 5-12: Test setup for perpendicular to grain connection tests

Figure 5-13: Test setup shown for three dowel specimen (test group i)
5.5 Results of perpendicular to grain tests

The results of the perpendicular to grain tests are summarised in Table 5.4 and the load slip plots for all three specimen groups are shown in Figure 5-14. Connection yield capacity, stiffness and ultimate capacity were all determined in the same manner as for the results reported in section 5.3 for parallel to grain specimens and an explanation of the methods used can be found in this section.

<table>
<thead>
<tr>
<th>Test group</th>
<th>Yield load (kN)</th>
<th>Ultimate load (kN)</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean value (%)</td>
<td>Mean value (%)</td>
<td>Mean value (%)</td>
</tr>
<tr>
<td></td>
<td>Range</td>
<td>Range</td>
<td>Range</td>
</tr>
<tr>
<td>g</td>
<td>10.8 3.8</td>
<td>10.5 - 11.4</td>
<td>11.8 - 14.5</td>
</tr>
<tr>
<td>h</td>
<td>19.3 2.8</td>
<td>18.8 - 20.2</td>
<td>21.7 - 26.5</td>
</tr>
<tr>
<td>i</td>
<td>27.6 5.1</td>
<td>25.1 - 28.5</td>
<td>29.2 - 36.7</td>
</tr>
</tbody>
</table>

From the results it can be seen that the load share was efficient within the elastic range. This is particularly true of the mean average stiffness values for the three specimen groups, which show a linear increase in connection stiffness with an
increasing number of fasteners. Significant variation in stiffness can be seen in the load slip plot of specimen group 'g' in addition to one apparently anomalous result. The anomalous load slip response was recorded as a result of the central roller restraint slipping part way through the test. The results of this test were therefore excluded from the data presented in Table 5.4. Roller slip was not observed in any other cases. The variation in the connection stiffness response of test group 'g' is partly attributed to flexural rotation of the cantilever portion of the beam under load. The rotation meant that the LVDT did not remain perpendicular to the tab on the plate at increased loads(Figure 5-12). Some discrepancy in the connection stiffness of test group 'g' is also attributed to natural variation in the glulam timber. Natural timber variation and member rotation are both more sensitive in single dowel connections.

The load slip responses presented in Figure 5-14 show that the primary failure mode was ductile for all specimens. The ductile connection yield was followed by a period of plastic deformation and hardening prior to splitting failure. The post yield plastic deformation and hardening of the connections is attributed to increased deformation and embedment of the dowels at the connection interface. This load response and failure mode was expected and is in agreement with the characteristic response outlined by Leijten and Van der Put (2004) for the perpendicular to grain splitting capacity of metallic dowel type connections.

Ultimate splitting failure modes for specimens made with one, two and three dowels are shown in Figures 5-15 to 5-18. In all instances ultimate loss of strength, and hence connection failure, was attributed to splitting of the member at the position of the lowest dowel. However, in one case a split also propagated at the position of the top dowel in a specimen from group h (Figure 5-16). The split propagated shortly before ultimate failure which was caused by splitting at the level of the lower dowels.

In two instances splitting occurred in the glue line of the structural member. This is shown in Figure 5-18 where the initial propagation of the split can also be seen. Both of the specimens were from group h and the data presented in Table 5.4 show that the ultimate failure loads were not significantly different from the other specimens. From Figure 5-18 a slight rotation of the right hand side member can be also be seen. This rotation was also seen in many of the other specimens and occurred as a result of using two separate timber members to simulate a slot type connection in a single section. In a full section this rotation would be restrained by internal stresses within the member. The rotation was not considered to have significantly influenced the ultimate failure capacities recorded.

No failure of the DVW plate was observed in any of the tests. Figure 5-19 shows the components of a disassembled connection that was previously loaded to failure.
It can be seen that no notable embedment damage occurred in the plate as a result of dowel bearing. However in the timber side members areas of crushing and splitting around the dowel holes are clearly visible. For the perpendicular to grain specimens the DVW plate was made with an end distance of 3d in accordance to EC5 minimum spacing rules for the structural member. For parallel to grain
specimens an end distance of this length was found to be susceptible to end cleavage failure for the 10 mm thick plate used. However, prior to failure of the structural member a high level of post yield hardening was observed for parallel to grain load orientations. Structural member failure perpendicular to grain occurs at a significantly lower load than when loaded axially so the level of stress in the DVW plate is therefore lower at failure for perpendicular to grain load orientations. Hence for this loading direction an end distance of 3d was observed to be sufficient for the
10 mm thick plate used. Further work is required to fully characterise this failure mode in DVW plates.

Figure 5-19: Connection components of disassembled specimen after testing

5.6 Concluding comments

Connection tests have been completed parallel and perpendicular to grain to investigate response to load and ultimate connection failure modes. The main conclusions that can be drawn from the experimental work presented in this chapter are as follows:

- The in-line spacing of dowels parallel to grain have been investigated. The experimental results demonstrate that GFRP dowels may be positioned closer together than metallic dowels set out to EC5 minimum spacing rules. This improves the efficiency of GFRP-DVW connections parallel to grain.

- Connections made with parallel to grain in-line spacings and end distance of four times the dowel diameter (centre to centre) displayed significant post yield ductility and energy absorption – spacings and end distance of three times the dowel diameter did not.
• Ultimate connection failure parallel to grain was triggered by partial plug shear in the timber members in the majority of cases. This failure mode appears to be unique to non-metallic dowel connections.

• The minimum length (end distance or in-line spacing) of partial shear plug was observed to trigger ultimate connection failure.

• Perpendicular to grain connections used EC5 minimum spacing rules. Significant post yield energy absorption was observed.

• Ultimate perpendicular to grain failure was observed to be splitting of the glulam members at the position of the lowest dowel.

• Brittle failure modes were observed for the DVW plates. Connection design must incorporate checks for these modes.

• No embedment failure of the DVW plate was observed.
Chapter 6

Full scale testing of GFRP-DVW connections

6.1 Introduction

Full scale slot-in-plate connections were tested parallel to grain to provide insight into the load response and failure modes of GFRP-DVW connections made with groups of fasteners. Parallel to grain loading of timber connections typically provides the highest capacity so testing the connections in this orientation also represented a critical loading situation for the DVW connector plates. In addition to investigating the load response of multiple dowelled connections the full scale tests allowed the spacing of GFRP dowels to be further tested. The tests also provided a platform for comparison with connections made with metallic components.

To further investigate the spacing of GFRP dowels, the GFRP-DVW connections used spacing rules that were based on the experimental findings reported in section 5.2. For comparison the GFRP-DVW connections were tested in conjunction with metallic connections. The metallic connections used EC5 minimum spacing rules. The full scale specimen configurations are shown in Figures 6-1, 6-2 and 6-3. An end distance \(a_{34}\) of 5d and an in line spacing \(a_{1}\) of 4d were used for the GFRP-DVW connections. All of the other dimensions were in accordance with EC5 minimum spacing rules. For reference throughout this section the specimens will be referred to using the descriptions given in Table 6.1.

Two different GFRP-DVW configurations were tested (Figures 6-1 and 6-2). The first configuration used the same connected area of timber as the metallic connections. This allowed a comparison between the connection efficiency of the two different connection types to be investigated. The second GFRP-DVW
connection used the same number of dowels as the metallic specimen but the dowels were positioned at the reduced spacings given above. Two repeat tests were completed for the GFRP-DVW connection with 12 dowels and one specimen was tested for the connection with 9 dowels. Three identical metallic specimens were tested.

Unlike the specimens previously reported in this chapter the GFRP-DVW connections used a 15 mm wide flitch plate, which was not fully resin impregnated (Manufacturers grade MII/2-E3). Based upon the manufacturers material properties a maximum load capacity of 160 kN was expected. However, stress concentration around the dowels was likely cause a reduction in this value. Nonetheless the plate capacity was considered sufficient to ensure yield of the GFRP fasteners. It was however acknowledged that the post yield hardening observed in previous tests may cause net tension of the plate prior to failure of the glulam member. For the 9 dowel GFRP-DVW connection plate failure was not expected prior to glulam failure.

The metallic connections were made using grade 303/1.4305 stainless steel dowels and a 10 mm thick steel flitch plate. In accordance with EC5 methods the predicted characteristic capacity of a single fastener was calculated as 23.8 kN (Mode II failure - Figure 2-4). For multiple dowels in line EC5 states that the capacity of a metallic connection is not linear to the number of dowels and must be multiplied by the partial factor $n_{eff}$ to account for the occurrence of brittle failure modes prior to EYM yield. In this instance the value of $n_{eff}$ is given as 2.12 (BS EN 1995, 2004), which means the overall connection design capacity is predicted as 133.2 kN. This is a design capacity as the characteristic value has also been factored by $\gamma_m = 1.3$ and $k_{mod} = 1.1$ in accordance with Eurocode 5 guidance. As this is a design strength the actual capacity of the connection was expected to be significantly higher.

### 6.2 Experimental test setup

The experimental setup used to test the full scale specimens is shown in Figure 6-4. The specimens were tested using a universal Dartec testing machine under displacement controlled tension loading. The specimens were loaded through the central flitch plates which were clamped in the jaws of the machine loading heads.
Figure 6-1: Specimen configuration for full scale GFRP-DVW connection with 12 No. 12 mm diameter GFRP dowels
Figure 6-2: Specimen configuration for full scale GFRP-DVW connection with 9 No. 12 mm diameter GFRP dowels
Figure 6.3: Specimen configuration for full scale metallic connection with 9 No. M12 stainless steel dowels.
CHAPTER 6. FULL SCALE TESTING OF GFRP-DVW CONNECTIONS

Based upon connection load response observed in previous testing a load rate of 0.4 mm/minute was used with the intention of initiating failure within 300 ± 120 seconds. This is in accordance with BS EN 26891 (1991) guidance.

Specimens GFRP-12-A and METAL-9-A were both tested under a monotonic load to failure. Based upon the recorded failure capacity of these specimens all subsequent tests were completed under cyclic loading. Two cycles of load were applied to each specimen between values approximately corresponding to 10% and 40% of the ultimate connection capacity. The load was held for 30 seconds at the peak and trough of each cycle and on the second cycle the specimen was loaded to failure.

6.3 Results

Load slip plots for each of the specimens are presented in Figure 6-5. The load-slip plots for the non-metallic specimens are shown in greater detail in Figure 6-6 and details of loading cycles applied to GFRP-DVW connections are shown in Figure 6-7. The slip of the connections was calculated as the mean average from the data recorded by the four LVDTs attached to each connection. No significant variation in this data was apparent, except at the onset of glulam failure. Distortion of the glulam member upon loading was not observed.

The yield load, ultimate load and initial stiffness results of the full scale connection testing are summarised in Table 6-5. The connection stiffness was determined from the gradient of the line that passes through points on the connection load slip plot, which correspond to 10% and 40% of the ultimate load. For the cyclically loaded specimens the stiffness was determined from the final loading cycle of the load slip plot. For this reason the stiffness values presented in Table 6-5 are higher for the cyclically loaded specimens. In accordance with BS EN 26891 (1991) the ultimate load is defined as the maximum load prior to failure up to 15 mm connection slip. Yield strength was evaluated using the 5% offset method described in ASTM D 5652-95 (2007). This method defines the yield load as the intercept between the load-slip curve and the line of initial stiffness offset by 5% of the dowel diameter and is illustrated in Figure 3-2 in Chapter 3.

The test setup used to load the specimens was such that failure of the glulam member, or DVW plate, initiated in the end of the member with the weaker connection. Upon the initiation of failure the other connection is effectively unloaded at the rate of capacity loss in the failed specimen. The interpretation of the load slip plots for the specimens could therefore be misleading without knowledge of which of plots correspond to ultimate connection failure. Hence, in
A displacement controlled tension load was applied through the cross-head of the testing machine.

Test specimen.

The connection flitch plates were clamped in the jaws of the testing machine.

A total of eight LVDT’s were used to record average connection slip.

Aluminium tabs were glued to the connector plates for the measurement of connection slip.

Figure 6-4: Experimental setup for full scale connection tests
reference to the load slip plots presented in Figures 6-5 and 6-6 a general mode of failure is given in Table 6.2 for the connections which failed. Connections that were not loaded to the point of ultimate failure are termed N/A.

In accordance with the spacing tests reported in section 5.3.3, the GFRP-DVW connections were made with closer in-line spacing rules than the metallic connections. Therefore the GFRP-DVW connections made with 12 dowels used the same connected area of timber as the metallic specimens, which were made with 9 dowels. This allows an approximate comparison to be made between the efficiency of the mean average yield capacities of these two connection configurations.

From Table 6-5 the mean average yield capacity of the GFRP-DVW connections made with 12 dowels was 129.8 kN and for the metallic connections (made with 9 dowels) it was 226.7 kN. Therefore in terms of load capacity per unit area of connected timber, the GFRP-DVW connections provided a yield capacity approximately 57% of that recorded for the metallic equivalent. The GFRP-DVW connections made with 9 dowels provided a mean average yield capacity of 92.2 kN which is 41% of the capacity provided by the metallic connections made with the same number of dowels, but at larger dowel spacings. The results show that the metallic connections gave a strength much higher than that calculated from EC5. The design strength for the metallic connection was calculated as 133.2 kN and with reference to this value the strength of the GFRP-DVW connections is high.

The same comparisons between connection types can be made for the recorded stiffness values. Comparing mean average stiffness values for the cyclically loaded connections shows that the stiffness of the 12 dowel GFRP-DVW connection was 55% of that recorded for the metallic connections. The recorded stiffness of the 9 dowel GFRP-DVW connections was 48% of the metallic connection stiffness. These comparisons are drawn between specimens that were, as far as possible, identical in terms of fabrication tolerance. Both the metallic and non-metallic connections were made with interference fit dowels and this is reflected in the load slip plots, which are all linear from the origin. However, although an interference fit is standard practice for dowelled connections, metallic bolted connections are made with a timber hole clearance to aid assembly. This clearance can be up to 1 mm for the timber member and up to 2 mm/0.1d (max) in steel plates (BS EN 1995, 2004). This tolerance must be included in the serviceability limit design of connections and therefore significantly increases the initial slip of a metallic connection. Therefore, in comparison to metallic bolted connections it is likely that GFRP-DVW connections would provide a more closely comparable slip modulus than that reported in these test results.

Photographs of failed metallic connections are shown in Figure 6-8. Figure 6.8(a) illustrates the typical mode of glulam member failure and Figure 6.8(b) shows
Table 6.2: Results summary for full scale tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load (kN)</th>
<th>Ultimate load (kN)</th>
<th>Stiffness (kN/mm)</th>
<th>Ultimate failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top</td>
<td>Bottom</td>
<td>Top</td>
<td>Bottom</td>
</tr>
<tr>
<td>GFRP-9</td>
<td>91.6</td>
<td>92.8</td>
<td>N/A</td>
<td>122.8</td>
</tr>
<tr>
<td>GFRP-12A*</td>
<td>131.0</td>
<td>128.0</td>
<td>N/A</td>
<td>141.3</td>
</tr>
<tr>
<td>GFRP-12B</td>
<td>136.0</td>
<td>124.0</td>
<td>139.2</td>
<td>N/A</td>
</tr>
<tr>
<td>METAL-9A*</td>
<td>248.8</td>
<td>248.0</td>
<td>N/A</td>
<td>249.5</td>
</tr>
<tr>
<td>METAL-9B</td>
<td>213.2</td>
<td>213.2</td>
<td>N/A</td>
<td>240.9</td>
</tr>
<tr>
<td>METAL-9C</td>
<td>238.5</td>
<td>198.5</td>
<td>257.5</td>
<td>N/A</td>
</tr>
</tbody>
</table>

*Not cyclically loaded
CHAPTER 6. FULL SCALE TESTING OF GFRP-DVW CONNECTIONS

Figure 6-5: Load-slip plots for full scale test specimens
CHAPTER 6. FULL SCALE TESTING OF GFRP-DVW CONNECTIONS

Figure 6-6: Load-slip plots for full scale GFRP-DVW specimens
Figure 6-7: Typical load-slip plots for cyclically loaded GFRP-DVW specimens.
the common dowel failure. Two types of glulam failure were observed. Timber splitting occurred along some lines of dowels and in other instances plug shear was observed. Both modes are shown in Figure 6.8(a) and no clear pattern was seen in the occurrence of the different failures. Dowel failure of the type shown in Figure 6.8(b) was common to all of the metallic connections though it was more apparent in the connections where the glulam had failed. As predicted by the EYM, it can be seen that the central line of dowels failed in a two hinge mode. However, the dowels either side remained relatively straight with no clear formation of hinges along their length. The yield of the central dowels is attributed to the lateral restraint of the side dowels upon the central portion of timber. Such restraint was not present on the sides of the glulam members and the resistance of the split timber was therefore not sufficient to cause dowel yield. The observed failure mode suggests that a more brittle connection failure may occur in specimens made with single or double lines of fasteners.

Figures 6.9(a) and 6.9(b) show the two different types of connection failure observed for the GFRP-DVW specimens. Figure 6.9(a) shows a DVW plate that failed in tension and Figure 6.9(b) shows glulam member failure. Tension failure of the DVW plate was observed in both specimens made with 12 GFRP dowels and glulam failure was observed for the specimen made with 9 GFRP dowels.

For the GFRP-DVW connection made with 9 dowels the capacity of the glulam resisting load was lower than the DVW net tension capacity. Hence, glulam failure was observed after a period of post yield connection hardening. The glulam failure was similar to that seen in the specimens previously tested in section 5.2. Partial thickness plug shear occurred initially and, under continued loading, this was followed by splitting. The progressive failure of the glulam in this way provided significant post yield energy absorption. Conversely, the connections made with 12 dowels provided poor post yield energy absorption due to the complete loss of plate capacity upon failure. Therefore, in spite of providing a lower yield capacity the 9 dowel specimens which caused failure of the glulam represent the most desirable failure mode for GFRP-DVW connections.

Tension failure of the DVW plate occurred at loads lower than may have been expected from the manufacturers data for clear tensile coupon tests. This is attributed to a concentration of stress around the dowel holes. Fibre cracking was heard shortly before the DVW plate failed, which supports the suggestion that failure was initiated as a result of stress concentrations around the dowel holes. Both plate failures were in line with the end line of dowels. This was expected as this is the point of highest stress in the plate.

It is considered that greater capacity could be gained from the use of multiple DVW shear plates. This has the potential to make more efficient use of the GFRP dowels,
Figure 6-8: Failed metallic specimens
CHAPTER 6. FULL SCALE TESTING OF GFRP-DVW CONNECTIONS

Figure 6-9: Failed GFRP-DVW specimens

(a) Non-metallic connection showing DVW plate failure

(b) Failed non-metallic connection showing partial plug shear
which fail close to the connection interface. The use of multiple metallic plates can be costly to fabricate due to the difficulty associated with alignment, however DVW plate can be drilled in a single operation so this is less likely to pose a significant economic barrier to the development of this connection type. Additionally the labour and costs associated with fabricating multiple slots in timber members may be balanced by the costs and time savings of using DVW plates.

6.4 Discussion

The results section of this study presented the comparative load response of metallic and GFRP-DVW connections loaded parallel to grain. Based only upon the the quantitative strength and stiffness results given, metallic connections may appear favourable over the use of GFRP-DVW connections. In certain highly loaded connections this will be true and the use of metallic systems may be unavoidable. Nonetheless, in many situations the use of GFRP-DVW connections could provide a significantly robust structural solution. In these situations qualitative attributes, such as fabrication, need to be considered as GFRP-DVW connections could provide a favourable solution when considering the selection of a connection system. An outline appraisal is given here based upon the experience of fabricating the comparative test specimens reported above. The intention of this appraisal is to provide insight into the fabrication methods adopted for the two connection types and the advantages and disadvantages of each.

The glulam billets used for testing were fabricated by Inwood Developments and were supplied with slots pre-cut ready to accommodate the connector plates. The assembly and fabrication of the connections and the respective components was completed in the university workshop. Two different methods of fabrication were adopted for the metallic and GFRP-DVW connections. The different methods were used in reflection of the different machinability of the connector materials.

Metallic plates were purchased at the required size for testing. The positions of the dowel holes were then marked, punched and drilled using three increments of drill bit diameter. This drilling operation was very time consuming and required the careful use of a specialist metalwork pillar drill. The stainless steel dowels were cut on a metalwork bandsaw and the ends of the dowels were beveled using a grinding stone. Again this took a considerable amount of time.

The holes for the stainless steel dowels were drilled using the steel plates as templates to ensure good alignment between the plate and glulam. This was completed using a 12 mm diameter drill bit to ensure an interference dowel fit in accordance with EC5 requirements (BS EN 1995, 2004). A drill guide was
used to help maintain a perpendicular drilling angle and the drilling operation was straightforward. However, difficulty arose when the connections were assembled. In a small number of holes the alignment between the plate and the glulam was out by approximately 0.5 mm. Subsequently driving the dowels through these holes was difficult as it essentially required the crushing of the surrounding timber. In some instances damage was caused to the surface of the glulam as a result of the misalignment. Figure 6-10 shows surface splitting damage caused by a misaligned dowel. The figure also shows the poor aesthetic quality of a misaligned dowel. It is likely that in practice oversized holes are used in the steel plate in order to accommodate fabrication tolerances.

For the non-metallic connections the DVW was supplied as a standard 2000 x 1000 mm sheet. The connector plates were cut to the required size using a standard handheld jigsaw (Figure 6-11). A single location hole of 12 mm diameter was then drilled in the DVW plate and in the glulam member. Incremental drilling was not required to drill the DVW plate and drilling was completed using a standard pillar drill for the plate. The glulam member was drilled using a hand drill and a drill guide.

After drilling the locator hole in the DVW plate and glulam a single dowel was inserted and subsequent holes were then drilled through the entire specimen in one operation (Figures 6-12 and 6-13). Drilling the glulam and DVW plate in one operation ensured accurate alignment of the plate and the glulam and no difficulty was encountered in the insertion of dowels. Prior to assembly, the GFRP dowels were cut using a water cooled diamond tipped saw and bevelled using a standard disc sander. This is a relatively quick process though rubber gloves were required to avoid getting any glass fibre splinters. A steel template was not used to position the drill for drilling holes in the GFRP-DVW connections. This made the process
CHAPTER 6. FULL SCALE TESTING OF GFRP-DVW CONNECTIONS

Figure 6-11: DVW plate fabrication using handheld jigsaw

Figure 6-12: Single locator dowel inserted through glulam and DVW plate
CHAPTER 6. FULL SCALE TESTING OF GFRP-DVW CONNECTIONS

Figure 6-13: Single operation drilling of GFRP-DVW connection

Figure 6-14: Completed GFRP-DVW connection
more time consuming as considerable time was spent setting up the drill position for each hole. It is therefore recommended that templates are used in the future to reduce the drilling time of GFRP-DVW connections and also to improve the accuracy of the drilling.

On balance the experience of fabricating the metallic connections was significantly more problematic than the fabrication of the GFRP-DVW connections. This was principally to do with the difficulty of alignment experienced with the metallic specimens. The use of metallic bolts and oversized holes would help to address this issue, though the initial slip of the connections would then be adversely affected. In addition to the problem of alignment, the fabrication of connector components was also considerably easier for the GFRP-DVW connections. The ability to be able to cut DVW sheet with handheld tools allows bespoke shapes and geometries to be easily made. Drilling of the DVW plate can also be completed in a single operation, which means prefabrication of the plate is not required in this sense. The drilling of the metallic plate was very time consuming due to the necessity to incrementally increase the drill bit size.

6.5 Concluding comments

Full scale connections have been tested in pull-out, parallel to grain. The tests allowed connection failure modes and load response to be understood for connections made with dowel groups. GFRP-DVW connections and metallic connections were tested to allow comparisons to be made. The main conclusions that can be drawn are as follows.

- Two different ultimate failure modes were observed for the GFRP-DVW connections; plate failure after dowel yield and glulam member failure after dowel yield.

- The failure mode of GFRP-DVW connections is controlled by the yield capacity of the dowel group and by the relative resistance of the glulam member and the DVW plate.

- Ultimate failure, as a result of glulam failure, is most desirable as this provided significant post yield ductility.

- Partial plug shear failure was observed in the glulam failure of the GFRP-DVW connection.

- DVW plate failure is undesirable as it is a brittle mode of failure.
• Where plate failure is avoided the use of 5d end distance and 4d in-line dowel spacing provided significant post yield ductility in the GFRP-DVW connection.

• In reference to EC5 spacing rules for metallic fasteners the use of reduced dowel spacings for GFRP-DVW connections allows the same connected area of timber to be mobilised but by a greater number of dowels.

• A GFRP-DVW connection, that loads the same area of timber as a metallic connection, was observed to provide a capacity equal to the EC5 design strength of the metallic connection and 50-60% of the strength experimentally measured for the metallic connection. Alternative connection configurations are likely to give different results.

• The fabrication of GFRP-DVW connections was found to be significantly easier than for metallic dowel-plate connections.
Chapter 7

Stiffness analysis of GFRP-DVW connections

7.1 Introduction

This chapter presents the findings from the application of a stiffness analysis method, which uses a beam on elastic foundation model loaded by a single spring. The model setup is introduced and the beam on elastic foundation stiffness matrix is derived. Application of the model is discussed and results are compared with experimental data. Future adoption of an EC5 analysis approach is also discussed.

The ability to predict the slip of a connection under load is necessary for the serviceability design of a timber structure. In the design of timber connections and members deformation behaviour is commonly a limiting factor over strength (Porteous and Kermani, 2009). It is therefore important to be able to provide design limits to ensure connections are not over or under designed. As far as possible methods of stiffness analysis should be practical if they are to be incorporated into mainstream engineering design. Within the field of engineering research and development, commercial finite element computer modeling packages are often employed to realise the task of modeling the elastic stiffness of structural systems. Shanks (2005) used finite element computer modeling in this way to investigate the elastic behaviour of pegged mortice and tenon connections. In this instance the application of the chosen approach is cited as being complex, specialised and time consuming. Thus this method was not wholly appropriate for the general engineering stiffness design of traditional mortice and tenon connections. Drake (2003) also made use of finite element computer modeling for the investigation of GFRP dowels loaded in double shear. However, no clear attempt to derive connection stiffness from the model is evident and no alternative approach is proposed.
CHAPTER 7. STIFFNESS ANALYSIS OF GFRP-DVW CONNECTIONS

This study has investigated the use of an alternative method of connection stiffness analysis. The method uses a combined stiffness model and was developed to provide an open, repeatable analysis technique. A beam on elastic foundation model is used to analyse the stiffness of the embedded portion of the dowel and direct stiffness measurements are used for the compressive stiffness of the dowel. The model setup is shown in Figure 7-1. It can be seen that the connection slip attributed to compression of the dowel under the loading of a DVW plate has been modeled as a spring connected in series with the foundation. The overall connection slip modulus is therefore calculated from equation 7.1 below.

\[
\frac{1}{k_{eq}} = \left( \frac{1}{k_1} + \frac{1}{k_2} \right)
\]  

(7.1)

where:

- \( k_{eq} \) is the system stiffness
- \( k_1 \) is the beam on elastic foundation stiffness
- \( k_2 \) is the dowel compression stiffness
From inspection of Figure 7-1 it can be seen that the assumption is made that in order to facilitate the beam on elastic foundation analysis the central section of the dowel within the plate thickness is not included and the load from the plate is assumed to act as a point load. The GFRP dowel is modeled as two elements of length $t$, where $t$ is the thickness of the timber side member. The stiffness matrices of these two elements form the global stiffness matrix that describes foundation deformation. The beam could be broken into further elements to gain understanding of the deformation along its length, however this produces a very large, complex global matrix and is not required to model central beam settlement (connection slip).

The advantage of using a beam on elastic foundation model is that the stiffness matrix required for analysis of dowel settlement can be quickly and readily computed using generic mathematics software. Additionally, only three material variables are required to evaluate the matrix. Therefore a stiffness value for connections of different geometries, foundation moduli, and dowel stiffness can be easily obtained once the matrix has been set up. In presenting the steps taken in the formation of the global stiffness matrix this method of analysis can also be repeated in practice or for further research purposes.

### 7.2 Beam on elastic foundation analysis

Beam on elastic foundation analysis is commonly used for the evaluation of design situations such as railway tracks and strip foundations in buildings. In these cases the influence of shear deformation in the steel railway track or concrete beam can be assumed to be negligible where the element is considered to be slender. This is because isotropic materials such as steel and reinforced concrete have a low ratio of shear stiffness to flexural stiffness. Therefore deflection due to shear deformation will generally be very small in comparison to those attributed to flexure. However, for cases of deep beams loaded at close intervals and for anisotropic materials, the influence of shear deformation can become significant (Bank, 1989; Aydogan, 1995). Hence, for the analysis of anisotropic, GFRP dowels, the beam on elastic foundation analysis presented in this chapter uses a general stiffness matrix formulation that incorporates shear effect.

The basis of the beam on elastic foundation analysis presented in this chapter is adopted from the method originally proposed by Hetenyi (1946) for classical beam analysis. In this method a winkler foundation is used, which means pressure between the foundation and the beam is assumed to be proportional to the settlement of the beam. Additionally Hetenyi (1946) assumes shear deformation to be negligible so a classical beam theory in the form of the Euler-Bernoulli bending
equation is used. As stated above this method can be used for simple analysis cases where foundation beams can be considered to be relatively slender between loading points and where the foundation beam is of a low anisotropy ratio. However, where this is not the case there is a requirement for the inclusion of shear deformation, which makes analysis more complex.

Previous models presented in literature have considered the inclusion of shear deformation through the application of Timoshenko beam theory. This considers the curvature of a beam due to shear and bending actions. However the analysis is very complex as both total rotation and deflection must be considered as individual degrees of freedom (Aydogan, 1995; Cheng and Pantelides, 1988). Therefore with the aim of providing a solution which is accessible and practical for potential adoption in practice an alternative analysis method was used. The selected, alternative, method was originally proposed by Aydogan (1995) for the formation of a global stiffness matrix suitable for application to civil engineering situations. The model includes shear effects through an appropriate approximation that considers only the deflection of the beam. This section describes the application of this method in order to generate a global stiffness matrix for the analysis of a GFRP dowel on a timber foundation. The derivation of the characteristic equation, which describes the beam deformation yields a fourth order differential equation. This is given below, and is used to form the global stiffness matrix. The derivation of the characteristic fourth order differential equation is presented in appendix A.

### 7.3 Formulation of global stiffness matrix

The formation of the global stiffness matrix is made by applying appropriate boundary conditions to the beam elements used to describe the GFRP dowel (Figure 7-1). The deformation of these beam elements is described by the general solution for a single beam element (Figure 7-2) and the derivation of this solution is given in appendix A. This general solution is for a beam on elastic foundation, which includes deformation of the beam due to shear displacements and yields the following characteristic differential equation.

\[
\frac{d^4 s}{dz^4} - 2\beta \left( \frac{d^2 s}{dz^2} \right) + \zeta s = 0
\]  

(7.2)

where:
\[
\beta = \frac{k}{2AG} \\
\zeta = \frac{k}{EI} \\
s \quad \text{is the vertical displacement of the beam element}
\]
This section describes the solution of this characteristic equation and the application of boundary conditions to form a global matrix. It is assumed that the timber foundation is modeled as a linear elastic winkler foundation. Hence the contact pressure at any point under the dowel is assumed to be proportional to the settlement of the foundation (Wang et al., 2005). For this model the settlement relationship is described as the foundation modulus, \( k \), which is defined as load per unit settlement per unit length of the dowel.

The characteristic equation 7.2 is a linear homogenous differential equation and so \( e^{az} \) can be considered as a solution (Stroud and Booth, 2001). Substituting this in, equation 7.2 becomes:

\[
a^4 - 2\beta a^2 + \zeta = 0
\]  

(7.3)

where the roots of the equation are

\[
a_{1,2} = \pm \sqrt{\beta + i\xi}
\]  

(7.4)

\[
a_{3,4} = \pm \sqrt{\beta - i\xi}
\]  

(7.5)

and

\[
\xi = \sqrt{\zeta - \beta^2}
\]  

(7.6)
Therefore the general solution to equation 7.2 can be expressed as:

\[ s = \sum_{i=1}^{4} C_i e^{a_i z} \]  

(7.7)

where \( C_i \) are the unknown coefficients, which relate to the boundary conditions shown in Figure 7-3.

Expressed in matrix form the general solution can be written as:

\[ s(x) = Z \mathbf{c} \]  

(7.8)

where

\[ Z = \begin{vmatrix} e^{a_1 z} & e^{a_2 z} & e^{a_3 z} & e^{a_4 z} \end{vmatrix} \]  

(7.9)

and

\[ \mathbf{c} = \begin{pmatrix} C_1 \\ C_2 \\ C_3 \\ C_4 \end{pmatrix} \]  

(7.10)
CHAPTER 7. STIFFNESS ANALYSIS OF GFRP-DVW CONNECTIONS

The nodal displacements shown in Figure 7-3 (a) can be written as below for the terminology given in the figure.

\[ D_1 = s(0) = s_0 \]  
(7.11)

\[ D_2 = \left| \frac{d s_b}{dz} \right|_{z=0} = \theta_0 \]  
(7.12)

\[ D_3 = s(l) = s_l \]  
(7.13)

\[ D_4 = \left| \frac{d s_b}{dz} \right|_{z=l} = \theta_l \]  
(7.14)

where:

- \( D_1 \) is the vertical displacement of the beam element at node \( z=0 \)
- \( D_2 \) is the rotation of the beam element at node \( z=0 \)
- \( D_3 \) is the vertical displacement of the beam element at node \( z=l \)
- \( D_4 \) is the rotation of the beam element at node \( z=l \)
- \( s_b \) is the vertical displacement of the beam element due to bending
- \( \theta \) is the rotation of the beam element

In vector form these nodal displacements can be expressed as:

\[ \mathbf{d} = \begin{bmatrix} D_1 \\ D_2 \\ D_3 \\ D_4 \end{bmatrix} \]  
(7.15)

Note that for this specific theory of bending with shear effect the boundary conditions must be taken as given below in equation 7.16. This is because rotation of the dowel due to shear effects is not included as a separate degree of freedom.

\[ \theta = \frac{ds_b}{dz} = \frac{ds}{dz} - \frac{ds_v}{dz} \]  
(7.16)

where:

- \( s_v \) is the vertical displacement of the beam element due to shear deformation

The unknown coefficients \( (C_{1,2,3,4}) \) of the general solution (equation 7.8) are obtained from the boundary conditions described in Figure 7-3 and equations 7.11 to 7.14 for a small beam element. In matrix form they can be written as below:
CHAPTER 7. STIFFNESS ANALYSIS OF GFRP-DVW CONNECTIONS

\[
\mathbf{d} = \mathbf{Bc} \rightarrow \mathbf{c} = \mathbf{B}^{-1}\mathbf{d}
\]

(7.17)

Matrix \( \mathbf{B} \), which relates the boundary conditions to the general solution is found as follows:

\[
\begin{bmatrix}
D_1 & s_0 \\
D_2 & \theta_0 \\
D_3 & s_l \\
D_4 & \theta_l
\end{bmatrix} =
\begin{bmatrix}
B_{1,1} & B_{1,2} & B_{1,3} & B_{1,4} \\
B_{2,1} & B_{2,2} & B_{2,3} & B_{2,4} \\
B_{3,1} & B_{3,2} & B_{3,3} & B_{3,4} \\
B_{4,1} & B_{4,2} & B_{4,3} & B_{4,4}
\end{bmatrix} C
\]

(7.18)

For the formation of matrix \( \mathbf{B} \) it is necessary to write an expression for \( \frac{ds_v}{dz} \) in terms of \( \frac{ds}{dz} \). This can be done by substituting equation 7.19 into 7.20 (these equations are derived in appendix A) and rearranging to make \( \frac{ds_v}{dz} \) the subject.

\[
\frac{ds_v}{dz} = \frac{T}{GA}
\]

(7.19)

\[
\frac{d^3s}{dz^3} = -\left( \frac{1}{EI} \right) \frac{dM}{dz} + \left( \frac{k}{AG} \right) \frac{ds}{dz} = -\frac{T}{EI} + \left( \frac{k}{AG} \right) \frac{ds}{dz}
\]

(7.20)

\[
\frac{ds_v}{dz} = \left( \frac{EI}{AG} \right) \left\{ -\left( \frac{d^3s}{dz^3} \right) + \left( \frac{k}{AG} \right) \frac{ds}{dz} \right\}
\]

(7.21)

Row 1 of matrix \( \mathbf{B} \) is found using the general solution \( s(x) = e^{at_i}C_i \) for \( z=0 \), as below:

\[
s_0 = e^{a_t(0)}C_i \rightarrow s_0 = C_i
\]

(7.22)

The second row of matrix \( \mathbf{B} \) is found using a substitution of equation 7.21 into equation 7.12 as follows:
\[
\theta_0 = \left[ \frac{d s}{d z} + \left( EI \right) \left[ \frac{1}{AG} \left( \frac{d s}{d z} - \frac{k}{AG} \right) \right] \right] (7.23)
\]

Considering \( s = e^{a_i z} C_i \), and \( z = 0 \), then \( \frac{d s}{d z} = a_i C_i \) and \( \frac{d^3 s}{d z^3} = a^3 C_i \). Additionally using \( \beta = k/2AG \) and taking \( \varphi = EI/AG \), equation 7.23 can be simplified to the form given below:

\[
\theta_0 = [a_i - \varphi(-a_i^3 + 2\beta a)]]C_i
\]

(7.24)

\[
\theta_0 = [a_i[1 + \varphi(a_i^2 - 2\beta a)]]]C_i
\]

(7.25)

let \( d_i = (a_i^2 - 2\beta) \) to yield:

\[
\theta_0 = [a_i(1 + \varphi d_i)]C_i
\]

(7.26)

The third and fourth rows of matrix \( B \) are found in the same manner as above except that \( z = l \). The complete matrix is given here:

\[
B = \begin{vmatrix}
1 & 1 & 1 & 1 \\
\frac{1}{a_1 e^{a_1 l}} & \frac{1}{a_2 e^{a_2 l}} & \frac{1}{a_3 e^{a_3 l}} & \frac{1}{a_4 e^{a_4 l}} \\
\frac{1}{a_1 e^{a_1 l}} & \frac{1}{a_2 e^{a_2 l}} & \frac{1}{a_3 e^{a_3 l}} & \frac{1}{a_4 e^{a_4 l}} \\
\end{vmatrix}
\]

(7.27)

Equation 7.28, given below, is a rearrangement of equation A.10 originally given in Appendix A:

\[
M = EI \left[ \frac{d^2 s}{d z^2} - \left( \frac{1}{AG} \right) (p - ks) \right]
\]

(7.28)

Using this equation, the force vector, \( p \), acting on the beam element can be written in correspondence to the four degrees of freedom in Figure 7-3. This is done by assuming the beam carries no distributed loads and uses the relation, shear force, \( T = d M/d z \):
CHAPTER 7. STIFFNESS ANALYSIS OF GFRP-DVW CONNECTIONS

\[
\mathbf{p} = \begin{bmatrix}
P_1 \\ P_2 \\ P_3 \\ P_4 
\end{bmatrix} = \begin{bmatrix}
EI \left[ \left( \frac{d^3 s}{dz^3} \right) - 2\beta \left( \frac{ds}{dz} \right) \right]_{z=0} \\
EI \left[ \left( \frac{d^2 s}{dz^2} \right) - 2\beta s \right]_{z=0} \\
-EI \left[ \left( \frac{d^3 s}{dz^3} \right) - 2\beta \left( \frac{ds}{dz} \right) \right]_{z=l} \\
EI \left[ \left( \frac{d^2 s}{dz^2} \right) - 2\beta s \right]_{z=0}
\end{bmatrix}
\] (7.29)

Differentiation of the general solution given in equation 7.8 and substitution into the force vector \( \mathbf{p} \) (equation 7.29) gives:

\[
\mathbf{p} = \mathbf{Ec}
\] (7.30)

Where \( \mathbf{E} \) is:

\[
\mathbf{E} = EI \begin{bmatrix}
a_1d_1 & a_2d_2 & a_3d_3 & a_4d_4 \\
-d_1 & -d_2 & -d_3 & -d_4 \\
-a_1d_1e^{a_1l} & -a_2d_2e^{a_2l} & -a_3d_3e^{a_3l} & -a_4d_4e^{a_4l} \\
d_1e^{a_1l} & d_2e^{a_2l} & d_3e^{a_3l} & d_4e^{a_4l}
\end{bmatrix}
\] (7.31)

Substitution of \( \mathbf{c} = \mathbf{B}^{-1}\mathbf{d} \) (equation 7.17) into \( \mathbf{p} = \mathbf{Ec} \) (equation 7.30) gives

\[
\mathbf{p} = \mathbf{EB}^{-1}\mathbf{d} = \mathbf{Sd}
\] (7.32)

Where \( \mathbf{S} \) is is a 4x4 stiffness matrix corresponding to the nodal degrees of freedom (\( \mathbf{d} \)) of the beam element considered. Therefore \( \mathbf{S} \) for a beam element is:

\[
\mathbf{S} = \mathbf{EB}^{-1}
\] (7.33)

and displacement of the beam element nodes is found from:

\[
\mathbf{d} = \mathbf{pS}^{-1}
\] (7.34)
CHAPTER 7. STIFFNESS ANALYSIS OF GFRP-DVW CONNECTIONS

The beam model described in Figure 7-1 for the GFRP dowel is repeated here with appropriate terminology included for the formation of the global 6x6 stiffness matrix.

Two 4x4 stiffness matrices must be derived for beam segments A and B (shown in Figure 7-4). $S_A$ and $S_B$ below represent the elemental stiffness matrices for beam elements A and B respectively.

$$S_A = \begin{bmatrix} S_{A,1,1} & S_{A,1,2} & S_{A,1,3} & S_{A,1,4} \\ S_{A,2,1} & S_{A,2,2} & S_{A,2,3} & S_{A,2,4} \\ S_{A,3,1} & S_{A,3,2} & S_{A,3,3} & S_{A,3,4} \\ S_{A,4,1} & S_{A,4,2} & S_{A,4,3} & S_{A,4,4} \end{bmatrix}$$

(7.35)

$$S_B = \begin{bmatrix} S_{B,1,1} & S_{B,1,2} & S_{B,1,3} & S_{B,1,4} \\ S_{B,2,1} & S_{B,2,2} & S_{B,2,3} & S_{B,2,4} \\ S_{B,3,1} & S_{B,3,2} & S_{B,3,3} & S_{B,3,4} \\ S_{B,4,1} & S_{B,4,2} & S_{B,4,3} & S_{B,4,4} \end{bmatrix}$$

(7.36)

These are then combined to yield the global stiffness matrix below:
7.4 Application of model

This section presents the application of the stiffness model to determine slip moduli predictions for four of the connection configurations reported in Chapter 5. The results from the model are presented in tabular form (Tables 7.3 and 7.4) and also as load slip plots alongside the corresponding connection test data (Figures 7-6 and 7-7).

The proposed stiffness model for GFRP-DVW connections (Figure 7-1) is made up of two elements; the slip attributed to the GFRP dowel bearing on the timber foundation and the compression of the dowel under the central plate. The latter element was measured empirically (Section 4.4, Chapter 4) and incorporated into the model as a spring connected in series. It was acknowledged that deformation arising from this portion of a connection would be widely repeated throughout a structure and an empirical value can therefore be considered appropriate. However, the embedded dowel length and foundation modulus will change for each variance of member thickness and load direction. The beam on elastic foundation model allows these variables to be readily adjusted.

The empirical determination of the dowel compression stiffness under a DVW plate is presented in Chapter 4 for a 10 mm thick plate. A mean value of 40.0 kN/mm was recorded and a 5% characteristic value of 30 kN/mm was calculated from the experimental data in accordance with BS EN 14358 (2006). These values are both used here in the application of the model.

The beam on elastic foundation model was evaluated using Maple 13, which is a generic mathematics program suitable for the input and manipulation of matrices. Maple 13 was chosen for its user friendly interface but it would be equally possible to construct the global matrix solution in matlab or similar programs. To evaluate the beam on elastic foundation matrix solution, and determine the deformation of the GFRP dowel, five variables must be input into the program. These variables are

\[
\begin{bmatrix}
S_{A,1} & S_{A,2} & S_{A,3} & S_{A,4} \\
S_{A,2} & S_{A,3} & S_{A,4} & \\
S_{A,3} & S_{A,4} & S_{A,1} + S_B & S_{A,1} \\
S_{A,4} & S_{A,1} + S_B & S_{A,2} & S_{A,1} \
\end{bmatrix}
\]

\[
S_{Global} =
\begin{bmatrix}
S_{A,1} & S_{A,2} & S_{A,3} & S_{A,4} \\
S_{A,2} & S_{A,3} & S_{A,4} & \\
S_{A,3} & S_{A,4} & S_{A,1} + S_B & S_{A,1} \\
S_{A,4} & S_{A,1} + S_B & S_{A,2} & S_{A,1} \
\end{bmatrix}
\]

(7.37)
CHAPTER 7. STIFFNESS ANALYSIS OF GFRP-DVW CONNECTIONS

- Flexural modulus, $E$
- Shear modulus, $G$
- Foundation modulus, $k$
- Dowel diameter, $d$
- Side member thickness, $t_{1,2}$

For the materials used in this study the first three of these variables are presented and discussed in Chapter 4. Due to the difficulty in accurately measuring the shear modulus of the GFRP dowels a published value of 3 GPa is used throughout the application of this analysis. The value was selected upon the basis of data reviewed in literature by Mottram (2004). This is discussed in detail in Chapter 4. The flexural modulus and foundation modulus were both determined directly from material tests and their values are given in Table 7.1.

<table>
<thead>
<tr>
<th>Orientation</th>
<th>Value</th>
<th>Foundation modulus, $K$ ($N/mm^2$)</th>
<th>Flexural modulus ($N/mm^2$)</th>
<th>Shear modulus ($N/mm^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parallel</td>
<td>Mean</td>
<td>1245.3</td>
<td>51282</td>
<td>3000</td>
</tr>
<tr>
<td></td>
<td>5%</td>
<td>883.4</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Perpendicular</td>
<td>Mean</td>
<td>321.3</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>5%</td>
<td>235.2</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

The above variables are used to determine a slip modulus for four different connection configurations; single dowel connections loaded parallel and perpendicular to grain and connections made with three dowels in line loaded parallel and perpendicular to grain. The objective of applying the model to these connection configurations was firstly to investigate the degree of sensitivity to the input of different foundation moduli and secondly to determine the reliability of the model for multiple dowel applications.

Based upon the efficient load share exhibited in the experimental connection test data (presented in Chapter 5) the slip modulus for multiple dowel connections was calculated as springs acting in parallel. This is also in agreement with EC5 guidance for determining the slip of multiple fastener connections (BS EN 1995, 2004).

7.5 Results and discussion

Mean average and characteristic fifth percentile slip moduli have been determined for each of the connection configurations shown in Figure 7-5. The slip moduli are
CHAPTER 7. STIFFNESS ANALYSIS OF GFRP-DVW CONNECTIONS

Figure 7-5: Diagram of specimens for which stiffness has been analysed and predicted - letters d, a, i & h correspond with the test groups reported in Chapter 5 displayed graphically against experimental load slip plots in Figures 7-6 and 7-7. Numerical results are given in Tables 7.3 and 7.4.

The slip moduli values were calculated using equation 7.1. The stiffness values calculated from the beam on elastic foundation analysis are given below in Table 7.2 alongside stiffness values for a 12 mm GFRP dowel under a 10 mm thickness, DVW plate loading. For the beam on elastic foundation analysis the side member thickness was input as 48 mm and the dowel diameter was 12 mm as per the experimental specimens.

\[ \frac{1}{k_{eq}} = \left( \frac{1}{k_{bef}} + \frac{1}{k_{comp}} \right) \]  \hspace{1cm} (7.38)

where:

- \( k_{eq} \) is the connection stiffness per fastener
- \( k_{bef} \) is the beam on elastic foundation stiffness
- \( k_{comp} \) is the stiffness of a central portion of the GFRP dowel loaded by a DVW plate

Mean connection stiffness predictions are in reasonable agreement with experimental results. In all instances the model gave an over-prediction of mean connection stiffness when compared to the recorded data. An over-prediction of connection stiffness can be attributed to a range of factors. The experimentally measured
Table 7.2: Single dowel stiffness values for the determination of slip modulus, $k_{eq}$

<table>
<thead>
<tr>
<th>Orientation</th>
<th>$k_{bef}$ (kN/mm)</th>
<th>$k_{comp}$ (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>5% characteristic</td>
<td>Mean 5% characteristic</td>
</tr>
<tr>
<td>Parallel</td>
<td>58.5</td>
<td>40.0</td>
</tr>
<tr>
<td></td>
<td>43.9</td>
<td>30.0</td>
</tr>
<tr>
<td>Perpendicular</td>
<td>19.2</td>
<td>40.0</td>
</tr>
<tr>
<td></td>
<td>14.93</td>
<td>30.0</td>
</tr>
</tbody>
</table>

Table 7.3: Connection stiffness model mean average results

<table>
<thead>
<tr>
<th>Orientation</th>
<th>No. of Dowels</th>
<th>Predicted stiffness (kN/mm)</th>
<th>Experimental stiffness (kN/mm)</th>
<th>Experimental range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parallel</td>
<td>1</td>
<td>23.8</td>
<td>15.0</td>
<td>9.4 - 30.6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>71.4</td>
<td>45.3</td>
<td>18.5 - 53.9</td>
</tr>
<tr>
<td>Perpendicular</td>
<td>1</td>
<td>13.0</td>
<td>8.4</td>
<td>5.2 - 12.1</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>39.0</td>
<td>24.9</td>
<td>23.5 - 27.1</td>
</tr>
</tbody>
</table>

Table 7.4: Connection stiffness model fifth percentile results

<table>
<thead>
<tr>
<th>Orientation</th>
<th>No. of Dowels</th>
<th>Predicted stiffness (kN/mm)</th>
<th>Experimental stiffness (kN/mm)</th>
<th>Experimental range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parallel</td>
<td>1</td>
<td>17.8</td>
<td>4.83</td>
<td>9.4 - 30.6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>53.4</td>
<td>31.6</td>
<td>18.5 - 53.9</td>
</tr>
<tr>
<td>Perpendicular</td>
<td>1</td>
<td>10.0</td>
<td>3.1</td>
<td>5.2 - 12.1</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>30.0</td>
<td>21.7</td>
<td>23.5 - 27.1</td>
</tr>
</tbody>
</table>

For the connections reported in this chapter, monotonic loading was completed in all but one instance. A specimen from test group ‘d’ was subject to a single unloading-reloading cycle. The load-slip plot for this specimen is labelled in Figure 7-6 and is highlighted in blue. It can be seen from inspection of this figure that the initial connection stiffness is improved for the second cycle of load. This is because the dowels have been able to ‘bed-in’ under the first loading cycle and as a result of this process, the stiffness values for the test connections will be subject to error associated with fabrication inaccuracies in the connection, material variability and distortion of the specimens under load.

The effect of material variability, fabrication tolerances and experimental error is evident in the range of experimental data given in Tables 7.3 and 7.4 and from inspection of the load-slip plots shown in Figures 7-6 and 7-7. Here, significant variation in the recorded results can be seen. Inspection of the model predictions plotted with the experimental load-slip data provides a clearer comparison between predicted and experimental results than can be gained from the numerical data. Experimentally recorded connection stiffness is most noticeably over predicted for the three dowel, perpendicular to grain specimens. Better agreement is achieved for the other connection configurations.
The fit (and hence stiffness) of the connection is improved. In the instance of the specimen shown in Figure 7-6 the load-slip response of the second load cycle is in good agreement with the predicted plot.

The significance of the observed connection response to cyclic loads is that in the monotonically loaded specimens, the initial uptake of fabrication tolerances may have given reduced experimental stiffness values. The results from the full scale tests (reported in Chapter 6) reinforce this observation. The recorded connection stiffness for the monotonically loaded full scale specimens was significantly lower than for the specimens subjected to two unload-reload cycles (Table 6-5).

Characteristic stiffness predictions and experimental values are presented in Table 7.4. The characteristic predicted values were determined by using characteristic foundation modulus values and a characteristic stiffness value for the compressive stiffness value of the GFRP dowel. All characteristic data was calculated using statistical methods given in BS EN 14358 (2006). The intention of calculating characteristic stiffness data was to investigate values that could be considered for design purposes. The values were also intended to provide insight into whether the model would provide significantly conservative predictions. From Table 7.4 it can be seen that the predicted values are notably higher than those determined from experimental data. Additionally, the values do not provide conservative predictions when compared to the experimental data range (Table 7.4).

The variation between the characteristic predicted results and the lower bound of the experimental range and be attributed to material variation, fabrication tolerance and experimental error in the same manner as for the mean prediction results. The more notable variation between the predicted values and the characteristic experimental values is due to the relatively small number of data sets for each connection configuration. Calculating characteristic values for small data sets is sensitive to slight variation. Therefore these predictions should only be viewed as informative values, specific to the presented experimental data. Further testing would be expected to give a higher characteristic value for the experimental connection data, as the significance of variation is reduced.

From inspection of the analysis results presented in Tables 7.2, 7.3 and 7.4, an understanding of the sensitivity of the stiffness model can be gained. Comparing results for parallel and perpendicular to grain load orientations provides insight into the influence of timber embedment stiffness on the overall connection stiffness. The mean timber foundation modulus parallel to grain was determined experimentally as 1245.3 kN/mm² for the glulam used in this study. Perpendicular to grain the mean modulus was only 25% of this value. The result of this was that, perpendicular to grain, the beam on elastic foundation stiffness prediction was only 33% of that found for a parallel to grain orientation. The foundation modulus
CHAPTER 7. STIFFNESS ANALYSIS OF GFRP-DVW CONNECTIONS

Figure 7-6: Graphical representation of stiffness model results parallel to grain (3 dowel specimens)

Figure 7-7: Graphical representation of stiffness model results perpendicular to grain (3 dowel specimen)
appears to significantly influence the stiffness prediction of the beam on elastic foundation model. However, this stiffness relates only to the built in portion of the dowel.

Overall the mean predicted connection stiffness perpendicular to grain was 55% of the parallel to grain stiffness of 23.8 kN/mm. This indicates that the sensitivity of the beam on elastic foundation analysis to the value of foundation modulus used, significantly influences the overall connection stiffness. The leveling effect on the connection stiffness from the bearing stiffness of the GFRP dowel under the loading of the DVW plate is relatively slight. However, in assuming that the compressive stiffness of the GFRP dowel acts as a spring load upon a beam resting on an elastic foundation the proposed stiffness model will always be limited by the stiffness value used for the spring load.

For this study the stiffness, $k_{\text{comp}}$, was determined using a novel test setup (described in Chapter 4). The assumption was made that the total system stiffness, determined from symmetrically loading the dowel, could be modeled as two springs acting in series to give the unidirectional stiffness, $k_{\text{comp}}$. This value acts as an upper bound to the model due to the assumed spring setup. Therefore although the stiffness predictions presented are in reasonable agreement with experimental values further investigation of the characterisation of $k_{\text{comp}}$ may be required to verify the testing method adopted and to give further confidence in the value proposed.

### 7.6 Eurocode methods

Eurocode 5 provides expressions for calculating slip modulus values associated with the initial deformation of timber connections under load. The expressions are derived empirically from experimental test data (Blass et al., 1995) and are simple to apply in the sense that they only require knowledge of the fastener diameter and timber density. The expression for calculating the slip modulus of metallic dowel connections is given below in equation 7.39. No distinction is made for the orientation of load to grain or member thickness. Therefore it is assumed that slip modulus values calculated with this expression must be conservative for many situations.

$$k_{\text{ser}} = \frac{\rho_m^{1.5} d}{23}$$  \hspace{1cm} (7.39)

The mean dry density of the glulam used in this study was 533.1 kg/m$^3$ and the dowel diameter was 12 mm. Therefore, using the expression above, EC5
provides a slip modulus value of 6.4 kN/mm per fastener per shear plane. For a timber connection made with a central steel plate a slip modulus of 12.8 kN/mm is determined for a single dowel. This value appears to be compatible with experimental values of GFRP-DVW connection stiffness measured parallel to grain (Table 7.3). However it is significantly higher than experimental stiffness values perpendicular to grain.

If further connection testing is completed in the future then a modified expression similar to that in equation 7.39 could be proposed for GFRP-DVW connections. However, at present insufficient data is available to reliably calibrate the expression.

### 7.7 Concluding comments

A method of stiffness analysis for GFRP-DVW connections has been developed. The method uses a beam on elastic foundation model to analyse the built in portion of the dowel and includes the influence of shear deformation in the dowel. The derivation of the global stiffness matrix for this analysis is presented to facilitate future use and development of the model. The main conclusions from this chapter are as follows:

- The use of a beam on elastic foundation analysis in conjunction with a single spring load can be used to predict GFRP-DVW connection stiffness.
- The analysis allows slip modulus values to be determined for different load orientations and dowel diameters.
- Use of the analysis is rapid once the global matrix has been entered into appropriate mathematics software.
- Variation between predicted and experimental mean values may be attributed to initial dowel ‘bed in’ and hence lower recorded stiffness values for monotonically loaded specimens.
- Adoption of the simple EC5 method of slip analysis is discussed but currently this approach is not applicable to GFRP-DVW connections due to limited availability of experimental data.
- The experimental determination of further material parameters, such as timber embedment strength for different timber species and dowel compression stiffness for different diameters, could be used to allow a range of theoretical stiffness values to be generated. These values could be used to calibrate an analysis method similar to that provided by EC5, which is based upon dowel diameter and timber density.
Chapter 8

Strength Analysis of GFRP-DVW connections

This chapter presents methods for predicting the strength and failure mode of GFRP-DVW type connections. Possible ductile and brittle failure modes are described and methods for their analysis are proposed. Ductile connection capacity is associated with GFRP fastener yield and is predicted using a novel adaptation of the European Yield Model (EYM). Strength values for ductile connection capacity can subsequently be used to assess the ultimate brittle failure mode of the connection.

8.1 Failure modes

Connection failure modes encountered in the experimental investigation of GFRP-DVW connections are presented in Chapters 5 and 6. From the tests completed in this investigation four different connection failure modes can be classified for GFRP-DVW connections. These failure modes are described below and illustrated in Figure 8-1.

- Type A – Fastener yield followed by connection hardening and brittle splitting/plug shear of the timber member
- Type B – Fastener yield followed by connection hardening that results in brittle DVW plate failure
- Type C – Brittle plate failure prior to GFRP fastener yield
- Type D – Brittle timber member failure prior to GFRP fastener yield
CHAPTER 8. STRENGTH ANALYSIS OF GFRP-DVW CONNECTIONS

Figure 8-1: GFRP-DVW connection failure modes

Failure types ‘A’ and ‘B’ were observed for connections tested in a parallel to grain load orientation. Type ‘A’ was also observed for perpendicular to grain loading. These failure modes are shown in Figure 8-2. Failure of the timber member prior to yield failure of the GFRP fasteners was not observed in any of the tests. For components of load parallel to grain, failure type ‘D’ can be controlled through the use of appropriate spacing rules. For load components perpendicular to grain, premature timber splitting can be controlled through appropriate connection design.

The load slip plots presented in Figure 8-2 show the significant post yield energy absorption that can be obtained from a type ‘A’ failure mode. GFRP-DVW connections which exhibit failure type ‘A’ can be considered to be well designed due to their superior energy absorption and post yield robustness.

In order to predict the likely failure mode of a GFRP-DVW connection the design must check the following aspects of connection resistance:

- Ductile yield resistance of the GFRP fasteners
- DVW plate capacity
- Timber member failure for components of load perpendicular and parallel to grain
Figure 8-2: GFRP-DVW specimen failure modes

(a) Failure mode - Type B/C
(b) Failure mode - Type B
(c) Failure mode - Type A
(d) Failure mode - Type A
CHAPTER 8. STRENGTH ANALYSIS OF GFRP-DVW CONNECTIONS

The connection failure modes can then be characterised as below:

**Type A** – Yield capacity < Timber member capacity < DVW plate capacity

**Type B** – Yield capacity < DVW plate capacity < Timber member capacity

**Type C** – DVW plate capacity < Yield capacity & Timber member capacity

**Type D** – Timber member capacity < Yield capacity & DVW plate capacity

8.2 Ductile dowel failure

The European Yield Model given in EC5 is widely used to predict the yield capacity of timber connections made with metal dowel type fasteners. Multiple modes of dowel failure are given and the three that correspond to dowelled timber connections made with a central plate are described in Chapter 2.

A single EYM failure mode has been observed for GFRP dowels in GFRP-DVW connections. This mode corresponds to the four hinge dowel failure given by the EYM (Mode III in Figure 2-4). This mode was observed in specimens which were ‘locked’ and dissected after testing and also in the tests reported in Chapters 5 and 6. These dowel failures are shown in Figure 8.3(a) and 8.3(b).

![Figure 8-3: GFRP dowel failure](image)

Previous studies have attempted to modify the use of EYM expressions for use with GFRP dowels. However, the assumptions made in these proposed methods are not considered reliable. This is discussed in detail in Chapter 4.

A new novel method is proposed for the use of the EYM ‘mode III’ expression to predict the yield capacity of GFRP-DVW connections. The method uses the mechanics of the original Johansen (1949) expression for the observed mode of
failure. However, the expression is used in association with an effective plastic bending capacity \( M_{eff} \) for the GFRP dowel. The characterisation of \( M_{eff} \) is presented in Chapter 4.

The expression used to calculate the capacity of a mode III dowel failure for a single shear plane is given in equation 8.1 below. Derivation of the EYM expression is outlined in Chapter 4. In BS EN 1995 (2004) the expression has a factor of 2.3 at the beginning of the equation instead of the value of 2, which is obtained through pure derivation. It is uncertain what this additional factor relates to and it is assumed that it is an empirical adjustment made in reference to experimental test data. The analysis of GFRP-DVW connections will not use the value of 2.3 as it is likely to have been derived from metallic dowel test data.

\[
R_k = 2\sqrt{M_{eff}f_{h,k}d} \quad (8.1)
\]

where:
- \( R_k \) is the characteristic load carrying capacity per shear plane per fastener
- \( f_{h,k} \) is the characteristic embedment strength of the timber member
- \( d \) is the dowel diameter
- \( M_{eff} \) is the effective yield moment of the GFRP dowel

Predicted connection yield strength values are compared with experimental values in Table 8.1. The experimental data presented in the table is taken from tests completed for single dowel connections. Specimen characteristics such as dowel diameter and grain orientation are described alongside the results. Yield strength values for specimens made with LVL are taken from the results presented in Figure 3-7 of Chapter 3. The yield values were determined using the 5% offset method described in Chapters 3 and 5. Values for specimens made with glulam were taken from Tables 5.2 and 5.4 in Chapter 5.

The predicted capacities were calculated using characteristic embedment strength values. For the glulam timber this value is given in Table 4.2. For the LVL timber a characteristic embedment strength of 31.4 N/mm\(^2\) was used. This was calculated using Eurocode methods for the characteristic dry density of the LVL material, which is given in Table 4.1. Mean average values used for \( M_{eff} \) are given in Table 4.5.

The results presented in Table 8.1 are in reasonable agreement with those measured experimentally. Discrepancy between predicted strength values and the experimental values can partly be attributed to factors associated with the definition of experimental connection yield and natural variation in the timber.

160
Chapter 8. Strength Analysis of GFRP-DVW Connections

Table 8.1: Comparison of predicted connection strength with experimental values

<table>
<thead>
<tr>
<th>Dowel diameter (mm)</th>
<th>Timber type</th>
<th>Grain orientation</th>
<th>Predicted yield capacity (kN)</th>
<th>Mean avg. exp. yield capacity (kN)</th>
<th>Experimental range (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>LVL</td>
<td>Par.</td>
<td>6.4</td>
<td>5.5</td>
<td>5.0 - 6.4</td>
</tr>
<tr>
<td>12</td>
<td>LVL</td>
<td>Par.</td>
<td>13.0</td>
<td>12.3</td>
<td>11.8 - 13.1</td>
</tr>
<tr>
<td>16</td>
<td>LVL</td>
<td>Par.</td>
<td>24.0</td>
<td>21.8</td>
<td>20.3 - 22.7</td>
</tr>
<tr>
<td>12</td>
<td>Glulam</td>
<td>Par.</td>
<td>11.3</td>
<td>13.2</td>
<td>12.0 - 15.6</td>
</tr>
<tr>
<td>12</td>
<td>Glulam</td>
<td>Perp.</td>
<td>10.9</td>
<td>10.8</td>
<td>10.5 - 11.4</td>
</tr>
</tbody>
</table>

Various methods exist for the definition of a connection yield point from experimental results. Munoz et al. (2008) describes six common analysis methods that are used in North America, Europe, Japan and Australia. These methods use various points of reference for the estimation of the yield point from experimental load slip plots. A comparison of experimental results by Munoz et al. (2008) showed that the methods give different connection yield values for a single load slip result. The comparison included the 5% offset method used in this study. In reference to a mean average value, the estimation of yield load with the 5% method gave reliable results. Therefore although this method can be considered suitable for this study, the exact definition of connection yield is not resolved (Munoz et al., 2008) and can hence lead to a degree of discrepancy when yield values are compared to predicted strength.

The material properties used in this analysis were determined in various different ways. In all cases mean average values of $M_{eff}$ were used. These values were determined using the novel testing method set out in Chapter 4. It is possible that a degree of error in the predicted connection strength may have arisen from these values. Further testing is required to develop the testing method and to provide characteristic values.

The timber embedment strength values used for the analysis were determined using two different methods. The LVL embedment strength was calculated using the empirical expression given in equation 8.32 in BS EN 1995 (2004). This method is based upon the density of the timber in question and the dowel diameter. The glulam embedment strength was determined directly from experimental testing and characteristic values were calculated for the test data using BS EN 14358 (2006). In both instances the embedment strength of the timber may have been susceptible to error in the calculation of characteristic values as the sample size was small. Additionally in the instance of the glulam timber, knots were deliberately excluded in embedment testing to provide a lower bound value. In the test connections some areas of tight grain were encountered where knots were close to dowels and this is likely to have provided a higher embedment resistance in some instances.
The results presented in Table 8.1 provide insight into the strength analysis of GFRP-DVW connections made with a single fastener. However, most timber connections are made up of multiple fasteners. For metallic fasteners in line the EYM uses a partial factor to reduce the predicted load capacity of connections carrying load parallel to grain. This is to safeguard against brittle splitting failure which can occur prior to the full EYM capacity being reached. For GFRP-DVW connections made with dowels in line and loaded parallel to grain such a reduction was not necessary for dowel spacings greater than four times the dowel diameter. Therefore, where minimum spacing rules are adhered to, a linear relationship can be used for the prediction of connections made with multiple GFRP fasteners.

The strength analysis results for GFRP-DVW connections made with single and multiple fasteners are shown graphically in Figures 8-4. In these Figures the predicted values are plotted against experimental results reported in Chapters 5 and 6. In general the predicted results agree well with those measured experimentally and an $R^2$ value of 0.986 was calculated for the presented data. This positive correlation between the predicted yield strength values and the experimental values is encouraging. However in certain instances the model overpredicts the connection strength and this could lead to an unsafe design. EC5 accounts for this through the requirement that values calculated with the EYM are factored to take account of material variation and load duration. Factored strengths can then be used in design.

To calculate the design strength values from the EYM, characteristic values are divided by $\gamma_m$ and multiplied by $k_{mod}$. These factors correspond to material variation and load duration respectively. Therefore to illustrate the use of these factors in the strength analysis of GFRP-DVW connections, factored ‘design’ values are also plotted in Figure 8-4. These values are factored for medium term loading ($k_{mod}=0.8$) which corresponds to actions such as imposed floor loading. For timber connections BS EN 1995 (2004) gives a value of 1.3 for $\gamma_m$.

The design values plotted in Figure 8-4 all fall below the corresponding experimental values and would therefore be considered safe for the design of GFRP-DVW connections. However, this study has not completed long term load tests on GFRP-DVW connections. Therefore, although the $k_{mod}$ factors provided by EC5 appear valid, further work is required to verify their use with different materials and for longer term loading.

### 8.3 Perpendicular to grain timber failure

It is necessary to be able to predict the perpendicular to grain splitting capacity of GFRP-DVW connections to avoid brittle failure in the mode described as ‘Type C’
CHAPTER 8. STRENGTH ANALYSIS OF GFRP-DVW CONNECTIONS

Figure 8-4: Experimental connection yield results plotted against predicted yield strength values

(a) Experimental results plotted against predicted values (including full scale tests)

(b) Experimental results plotted against predicted values (close up of lower capacity connections)
in Figure 8-1. The EC5 design method for evaluating the perpendicular to grain failure uses a single expression that was derived from a fracture mechanics model. The design expression provided by EC5 is given in equation 8.2 for reference. This equation is used for the calculation of perpendicular to grain connection splitting capacity made with all types of metal dowel type fasteners. This is because an original fracture mechanics model was calibrated to experimental test results and provides an inclusive lower bound prediction of connection capacity (Leijten and Van der Put, 2004).

\[
F_{90} = 14b \sqrt{\frac{h_e}{(1 - \frac{h_e}{h})}} \quad (N)
\]  \hspace{1cm} (8.2)

where:
\(F_{90}\) is the characteristic splitting capacity of the connection (N)
\(b\) is the loaded member thickness (mm)
\(h\) is the timber member depth (mm)
\(h_e\) is the distance between loaded edge and centre of most distant fastener (mm)

In reference to the initiation of perpendicular to grain failure, two modes of dowel failure are reported by Leijten and Van der Put (2004). The modes are shown in Figure 2-9 and relate to a failure caused by a stiff, stocky dowel or a slender, flexible dowel. The stocky dowel failure was reported to provide a higher load resistance than the more slender dowels (Leijten and Van der Put, 2004). Therefore the slender dowel failure mode forms the lower bound of the calibrated model for metallic dowels. The failure of GFRP dowels is similar to that of a slender metallic dowel as can be seen in Figure 8-3. Hence, if the fracture mechanics approach used by EC5 is to be adopted for GFRP-DVW connections it is necessary to determine whether the observed experimental splitting loads fall within the bounds of the failure criterion.

The equation given by Leijten and Van der Put (2004) for the evaluation of perpendicular to grain splitting is given in equation 8.3. Discussion of the derivation of this equation is given in Chapter 2 for the general case of a cracked timber member loaded perpendicular to the crack direction. Leijten and Van der Put (2004) present a calibration of equation 8.3 to experimental data reported in literature for metallic dowel type connections loaded perpendicular to grain. In their study a constant of 15.5 N/mm\(^{1.5}\) is reported as a suitable starting point for a structural design code. Evidently from inspection of equation 8.2 the EC5 expression uses a lower empirical constant of 14. From equation 8.3 it can be seen that the empirical constant used is equal to \(\sqrt{G_G_c}/0.6\).
\[ \frac{V}{b\sqrt{h}} = \sqrt{\frac{GG_c}{0.6}} \sqrt{\frac{\alpha}{(1 - \alpha)}} \]  

(8.3)

where:

- \( \alpha = \frac{h_e}{h} \)
- \( V \) is the maximum shear force on either side of the connection
- \( b \) is the total thickness of timber loaded in shear
- \( h \) is the timber member depth
- \( h_e \) is the distance between loaded edge and centre of most distant fastener (mm)
- \( G \) is shear modulus
- \( G_c \) is the apparent fracture energy release rate

To calibrate the fracture mechanics expression given in equation 8.3, for use with GFRP-DVW connections, the experimental results reported in Section 5.4 of Chapter 5 can be used. Plotting \( V/b\sqrt{h} \) on the ordinate against \( h_e/h \) on the abscissa of a cartesian chart allows experimental results to be displayed alongside lines corresponding to different values of constant \( \sqrt{GG_c/0.6} \). This is shown in Figure 8-5 for the experimental splitting capacity of connections reported in Chapter 5. The experimental results are labelled with the reference letter that corresponds with those used in Chapter 5. All of the test specimens were made to EC5 minimum spacing rules and where multiple fasteners were used they were positioned in a single column. The number of dowels used in each specimen is given below for the corresponding test group:

- Test group ‘g’ - single dowel
- Test group ‘h’ - two dowels
- Test group ‘g’ - three dowels

Lines corresponding to different values of \( \sqrt{GG_c/0.6} \) can be seen on the plot shown in Figure 8-5. The upper most of these plots (\( \sqrt{GG_c/0.6} = 14 \)) corresponds to the constant used in the EC5 expression given above. It can be seen that in terms of mean connection strength the plot fits well. However, a significant number of the experimental values fall below the line, which would result in an under prediction of connection splitting strength. Therefore a reduction of the factor to a value of 12 could be considered for the design of GFRP-DVW connections.

It is acknowledged that two of the test results for single dowel specimens (test group ‘g’) fall below the boundary line corresponding to \( \sqrt{GG_c/0.6} = 12 \). However the
Figure 8-5: Experimental test data values plotted with $\sqrt{G_c/0.6}$ boundaries
use of this factor provides a predicted splitting capacity of 12.2 kN, which is above all of the experimentally recorded connection yield capacities reported in Chapter 5. It should also be noted that the timber section size used in this test specimen is very unlikely to be used in practice as it was dimensioned to the minimum EC5 spacing rules (84 mm deep). If such a section was used in practice the connection could only become unsafe if two dowels were positioned along the length of the member and this would not satisfy the design criteria for perpendicular to grain loading. For example the perpendicular to grain design capacities given above in section 8.2 give a value of 13.4 kN for two dowels which is greater than the predicted splitting capacity of 12.2 kN.

For \( \sqrt{G G_c/0.6} = 12 \) the perpendicular to grain splitting strength of the GFRP-DVW connections tested in this study can be predicted with equation 8.4 below. In Figure 8-6 the predicted splitting capacities are shown in conjunction with their respective experimental load slip plots. The connection geometry used in the analysis are given in Section 5.4 of Chapter 5.

\[
F_{90, GFRP-DVW} = 12b \sqrt{\frac{h_e}{1 - \frac{b}{h_e}}} (N) \tag{8.4}
\]

Figure 8-6: Predicted splitting capacity shown against experimental load-slip plots (key relates to test group)
CHAPTER 8. STRENGTH ANALYSIS OF GFRP-DVW CONNECTIONS

For the tests completed in this study Figure 8-6 shows a good prediction of splitting strength for design purposes. Upon the basis of these results the use of a fracture mechanics model to predict splitting capacity can be assumed valid. However, further work is required to investigate the influence of dowel groups on the failure capacity of timber members loaded in this orientation. The tests completed in this study have only investigated the failure of single fastener columns at the end of timber members. Future tests are therefore required to verify the calibration of the model for connections made along the length of a member and also for fastener groups.

Additional testing is also necessary to investigate the influence of timber species on splitting resistance. The proposed model can currently only be assumed as valid for softwood timber. Tests on connections made with hardwood and softwood timbers would allow the prediction of perpendicular to grain splitting to then be made in relation to connection geometry and as a function of timber density.

8.4 Parallel to grain timber failure

The ultimate parallel to grain failure modes of metallic dowel type connections are set out in Chapter 2. The four different modes described are:

- Plug Shear - shear of a timber plug loaded by a fastener
- Block shear - shear of a block of timber loaded by a fastener group
- Net tension - Failure of the net timber cross section in tension
- Splitting - Failure of the connected timber by splitting along a line of fasteners

For metallic fasteners designed using EC5 these modes of failure are controlled through the use of minimum spacing rules and the factor $n_{eff}$ which reduces the design capacity of multiple fasteners in line.

Brittle timber failure of GFRP-DVW connections can also be controlled through the use of minimum spacing rules. For tests completed using the minimum spacing rules given in Table 8.2, brittle failure parallel to grain was not observed prior to connection yield. These spacing rules use a lower value for the end distance ($a_{3,t}$) and in line dowel spacing ($a_{1}$) than given by BS EN 1995 (2004) for metal dowels. The reduction in spacing rules is made in response to the lower individual capacity of GFRP dowels when compared with metal dowels. This was investigated in the experimental programme of parallel to grain tests reported in Chapter 5.
Table 8.2: Experimentally determined dowel spacings that ensure yield of GFRP-DVW connections prior to brittle timber failure parallel to grain

<table>
<thead>
<tr>
<th>Spacing and end/edge distances</th>
<th>Minimum spacing or edge/end distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a_1)</td>
<td>(4d)</td>
</tr>
<tr>
<td>(a_2)</td>
<td>(3d)</td>
</tr>
<tr>
<td>(a_{3,t})</td>
<td>(5d)</td>
</tr>
<tr>
<td>(a_4)</td>
<td>(3d)</td>
</tr>
</tbody>
</table>

Spacing terminology is made in reference to Figure 2-6

The mitigation of timber failure prior to fastener yield allows a parallel to grain design capacity for GFRP-DVW connections to be reliably predicted using the methods set out in section 8.2. However, for GFRP-DVW connections it is necessary to be able to calculate the ultimate post yield capacity of the timber member in order to mitigate failure of the DVW plate. As demonstrated in Figure 8-2, the failure of DVW plate prior to the ultimate failure of the timber is an undesirable mode of ultimate connection failure due to the complete and sudden loss of load capacity. This failure mode should hence be avoided. Through appropriate design a connection that fails in a favourable Type A mode can be provided (Figure 8-1). For this failure type the following criteria must be satisfied:

Yield capacity < Timber member capacity < DVW plate capacity

The parallel to grain ultimate failure modes of GFRP-DVW connections are reported in Chapters 5 and 6. Two principle modes of failure were observed. These were failure of the DVW plate and partial thickness plug shear of the timber member. Partial thickness shear plugs were seen to form as initial splits at the connection interface, which then propagated to shear plugs. After the formation of the shear plug, full thickness splitting of the timber member was commonly observed under continued loading. In a small number of specimens, full thickness splitting of the timber appeared to occur at the same instance as partial plug shear. This mixed mode failure occurred at similar loads to those observed for failures that were initiated as partial shear plugs.

Prediction of the ultimate resistance of a partial thickness shear plug is more complex than for a full thickness shear plug. The analysis of full thickness plug failure can be made using an appropriate shear strength value and the area of timber loaded in shear (Quenneville and Mohammad, 2000; BS EN 1995, 2004). In this case the area is the length of the plug multiplied by the timber thickness. However, for a partial thickness shear plug the area of timber mobilised at failure is not certain. Figures 8-9 to 8-10 show the end view of a selection of failed GFRP-DVW connections that have been loaded parallel to grain. It can be seen that significant variation can occur in the size of the shear plugs. It can also be seen
that the failure appears to follow natural weaknesses in the glulam. This suggests that there is likely to be significant variation in the shear strength of each specimen also.

An analysis method is proposed to define an effective plug shear perimeter for use in conjunction with the characteristic shear strength of the connected timber member. The effective shear perimeter was defined by plotting failure boundaries against experimental results. These failure boundaries were based upon calculated connection resistance values for a range of failure perimeters and plug lengths. In order to implement the proposed analysis method it was necessary to define the way in which the shear area of timber was calculated.

The dimensions used for the analysis are shown in Figure 8-11. The total length of timber loaded in shear was taken as the minimum distance of $a_1$ or $a_{3,t}$ (Figure 2-6) multiplied by the number of fasteners in line. The distances $a_1$ and $a_{3,t}$ are the in line dowel spacing and end distance dimensions respectively. The use of the minimum of these two dimensions is in response to the findings presented by Quenneville and Mohammad (2000) and the experimental work reported in Chapter 5. In both cases it was shown that the minimum of the end or dowel spacing triggered ultimate plug shear failure. This limited the total connection resistance to the capacity of a connection made entirely with spacings equal to the minimum dimension. Therefore, for analysis purposes it is assumed that the total length of timber loaded at the point of failure is equal to the number of dowels multiplied by the minimum of the end or dowel spacing.

$$p_l = \min[a_{3,t}, a_1]N \quad (8.5)$$

where:

- $p_l$ is the total plug shear length for a line of dowels
- $N$ is the number of dowels in line
- $a_{3,t}$ is the end distance used for the line of dowels
- $a_1$ is the in line dowel spacing

The shear strength used in this analysis was taken from the timber grade strength values given in BS EN 383 (2007). The glulam used in this study was mechanically graded as C24 and the characteristic shear strength value for C24 timber is 2.5 N/mm$^2$.

To determine the effective perimeter of the observed partial shear plugs, ultimate connection strength was plotted on the ordinate against the total plug shear length
(1) Failure initiated as partial thickness shear plug followed by split propagation
(2) Redistribution of load after shear plug failure caused full thickness splitting of other member
Plug perimeter = 46 mm

Figure 8-7: End view of failed specimen from test group ‘b’ (Chapter 5

(1) Failure initiated as partial thickness shear plug followed by split propagation
(2) Redistribution of load after partial shear plug failure caused a full thickness shear plug to gradually form in other member
Plug perimeter = 72 mm

Figure 8-8: End view of failed specimen from test group ‘c’ (Chapter 5
(1) Failure initiated as partial thickness shear plug followed by split propagation
(2) Redistribution of load after shear plug failure caused full thickness splitting of other member
Plug perimeter = 56 mm

Figure 8-9: End view of failed specimen from test group ‘d’ (Chapter 5

(1) Failure initiated as partial thickness shear plug followed by split propagation
(2) Redistribution of load after partial shear plug failure caused full thickness shear plug to form in other member
Plug perimeter = 42 mm

Figure 8-10: End view of failed specimen from test group ‘eii’ (Chapter 5
Figure 8-11: Dimensions used for analysis of partial plug shear failure

multiplied by the timber shear strength on the abscissa. Experimental results were then plotted against theoretical values for a range of shear plug perimeter values. The theoretical connection strength values were calculated using equation 8.6:

$$P_{ult} = p_l l_{per} f_{v,k}$$  \hspace{1cm} (8.6)

where:

- $P_{ult}$ is the ultimate connection resistance
- $p_l$ is the total plug shear length for a line of dowels
- $l_{per}$ is the effective perimeter length of the partial shear plug
CHAPTER 8. STRENGTH ANALYSIS OF GFRP-DVW CONNECTIONS

\( f_{v,k} \) is the characteristic timber shear strength

Figure 8-12 shows the chart of experimental results plotted with theoretical strength predictions. The experimental data is taken from the tests presented in Chapters 5 and 6. The results of specimens that didn’t fail or failed as a result of DVW failure are not included. The data point on the far right of the chart is taken from the full scale tests.

The data presented in Figure 8-12 shows that an effective plug perimeter of 70 mm provides a suitable basis for the prediction of mean ultimate connection strength. Numerical values for predicted capacities calculated for a 70 mm plug perimeter are presented in Table 8.3. The experimental values presented in the table are for the same specimens plotted in Figure 8-12. The predicted strength values used a characteristic shear strength of 2.5 N/mm\(^2\).

For the design of DVW plate an upper bound prediction of the plug shear capacity of a connection is desirable. However, the data presented in Figure 8-12 shows a significant spread of experimental values for connections made with a minimum dowel or end distance of 3d. This large spread of data made it difficult to provide a consistent upper bound prediction of connection capacity. Therefore, although the analysis, given in Table 8.3, provides a reliable means for predicting mean capacity in most cases, consideration must be given to the design of the DVW plate. This is discussed in greater detail below.

<table>
<thead>
<tr>
<th>Experimental test group</th>
<th>Predicted strength (kN)</th>
<th>Mean average exp. ultimate load (kN)</th>
<th>Experimental range (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>21.0</td>
<td>20.7</td>
<td>20.1 - 21.2</td>
</tr>
<tr>
<td>b</td>
<td>63.0</td>
<td>61.6</td>
<td>61.0 - 62.4</td>
</tr>
<tr>
<td>c</td>
<td>50.4</td>
<td>52.82</td>
<td>47.3 - 59.7</td>
</tr>
<tr>
<td>d</td>
<td>37.8</td>
<td>35.66</td>
<td>31.1 - 43.0</td>
</tr>
<tr>
<td>e_i</td>
<td>37.8</td>
<td>43.9</td>
<td>37.1 - 50</td>
</tr>
<tr>
<td>e_{ii}</td>
<td>37.8</td>
<td>36.6</td>
<td>32.8 - 40.3</td>
</tr>
<tr>
<td>f</td>
<td>37.8</td>
<td>40.4</td>
<td>34.2 - 47.6</td>
</tr>
<tr>
<td>GFRP-9</td>
<td>151.2</td>
<td>122.8</td>
<td>single test</td>
</tr>
</tbody>
</table>

Except for the full scale connection test, all of the experimental results presented were from connections made with a single line of dowels. The spread in the data from these tests suggests that the orientation of timber growth rings has a significant influence on the maximum load capacity of the connection. This is because, in a single line of dowels, each dowel will be loading the same growth ring orientation and if the orientation is particularly strong the global connection strength will reflect this. However, in a connection made with multiple lines of
Figure 8.12: Experimental test data values plotted with $p_l$, $f_v$, $k$ values.
dowels (such as GFRP-9) it is not possible for each line of dowels to load the same growth ring orientation. Therefore, it is more likely that failure of the connection will initiate as a result of plug shear in a weak grain orientation. Further testing of fastener groups may therefore provide a more reliable basis from which an upper bound prediction of ultimate connection strength could be made. If this was so then a design factor could be considered for the strength prediction of a single line of dowels.

In addition to the influence of fastener groups on failure load, further testing is required to calibrate the effective perimeter value for different dowel diameters and particularly for different timber grades and species. This would allow the value of $l_{\text{per}}$ to be given as a function of dowel diameter and timber density. Further to this an effective number factor could be used for connections with multiple columns of dowels. This would account for the influence of connection geometry. In the case of partial shear plug failure the effective number factor would apply to the number of dowel columns; reducing total capacity as the number of columns increases. This would account for the increased probability of a column of dowels loading a weak grain orientation as discussed previously.

8.5 Plate failure

The design of DVW plate is necessary to ensure that if a connection is significantly overloaded ultimate failure occurs in the timber member and not in the DVW plate. This is primarily a concern for connections that resist a high proportion of load parallel to grain since connection capacity in this orientation is greater than perpendicular to grain. Failure of a glulam member parallel to grain can also provide significant energy dissipation under continued loading so ensuring the DVW plate does not fail prior to this is advantageous (Figure 8-2). Connections loaded perpendicular to grain were observed to be limited by the brittle splitting resistance of the timber member. This occurred at relatively low loads in comparison to the plate capacity. Nonetheless the plate should still be designed to have a greater resistance than the capacity of the timber member. This ensures that any available post yield connection ductility is mobilised in a situation where the connection is heavily overloaded. Adequate plate capacity is easier to provide in a perpendicular to grain load orientation due to the lower capacity of perpendicular to grain connections.

Two modes of DVW plate failure were observed in the experimental study of GFRP-DVW connections; end cleavage failure and net tension failure. End cleavage failure of the plate is shown in Figure 5-4. This failure mode was only observed in one instance and was attributed to an insufficient end distance being used. All
subsequent tests used an end distance of 4d and cleavage failure was not observed in any of these tests.

Tension failure of DVW plate causes a complete and sudden loss of connection capacity (Figure 8-2) and should therefore be designed against. This mode of failure was observed in the specimens tested in test group ‘b’ (Chapter 5) and in the full scale connection tests made with 12 GFRP dowels (Chapter 6). The specimens tested in test group ‘b’ were made with resin impregnated DVW and this can negatively influence the load carrying capacity of DVW material under long term loading Leijten (1998). At the time of testing this was not known and under the short term test loading the effect of this is likely to have been minimal. Nonetheless, a partially resin impregnated material was used for the full-scale connections. The tensile capacity of partially resin impregnated material is not reported to be effected by long term loading. The discussion in this section is primarily made in reference to the plate failure observed in the full scale tests. This is because the failure loads of the resin impregnated material are less valid for future connection design.

Using the analysis methods given above, a prediction can be made for the resistance of a C24 glulam member to partial shear plug failure or splitting failure perpendicular to grain. Based upon the predicted load capacity a suitable DVW plate cross section can be designed for a higher load capacity. However, an exact design strength value for DVW plate has not been fully resolved. In Chapter 6 the expected failure modes of the test specimens are discussed and failure of the DVW plate is highlighted as a likely mode for the connections made with 12 dowels. The manufacturers data sheet gave a tensile strength of 100 N/mm$^2$ for the medium density, partially resin impregnated material used. Based upon the net cross section of the plate this was predicted to provide a connection capacity of 160 kN. However, the results shown in Figure 8-2 shows that in the two instances of plate failure the maximum load resistance was 140 kN. This apparent loss in capacity is attributed to stress concentrations in the plate occurring around the dowel holes.

To fully understand the effect of stress concentrations around the dowel holes in a DVW plate requires a significant amount of experimental testing. Factors such as plate thickness, dowel spacing, and the effect of dowel groups all need to be understood before a widely applicable design method can be introduced for DVW plate. This depth of study was outside the scope of this investigation and so instead the test data from the full scale tests can be used as a basis for preliminary connection design. In cases where the capacity of the plate appears to be borderline specific tests may be necessary to confirm the capacity of the plate.

The full scale tests were made using the minimum spacing rules set out in Table 8.2 and therefore can be considered to provide a lower bound basis in terms of plate capacity. The mean average failure load of the plate was 140 kN, which is equal
8.6 Conclusions

The possible failure modes of GFRP-DVW connections have been characterised and are described. Analysis methods are presented to allow the failure capacity, and failure mode, of a connection to be predicted. The main conclusions from this chapter are as follows:

- The EYM expression for a four hinge dowel-plate connection failure can be applied to GFRP-DVW connections. In conjunction with an appropriate value for the dowels moment of resistance, this analysis allows the connection yield capacity to be predicted.

- The moment of resistance of a dowel can be determined through the novel test method and energy approach set out in Chapter 4.

- The perpendicular to grain splitting strength of a GFRP-DVW connection can be predicted using a fracture mechanics model, calibrated to experimental test results.

- Parallel to grain, partial shear plug, failure loads can be predicted by deriving an effective plug failure perimeter from connection test results. This effective plug failure perimeter can be used to calculate the area of timber resisting shear and, in conjunction with an appropriate timber shear strength value, the partial shear plug capacity can be predicted.

- Prediction of DVW plate net-tension failure load is required to mitigate brittle failure modes. An effective tensile strength of the plate can be approximated from observed connection failures. Further testing is required for detailed design.
Chapter 9

Discussion and application of study

This chapter presents a summary of the findings that pertain to the practical use of GFRP-DVW connections. During the course of completing the experimental work described in this thesis, a good depth of understanding has been gained regarding many aspects of GFRP-DVW connection behaviour and fabrication. Additionally, in the summer of 2010 GFRP-DVW connections were used within an installation at the V&A museum in London. Using this project as a reference an appraisal of the connection technique is made and suggestions given for future development.

9.1  Woodshed - Rural Studio Pavilion

GFRP-DVW connections were used in the Rural Studio pavilion for the Victoria and Albert Museum’s 2010 Architectural Exhibition; ‘Architects Build Small Spaces’. For the exhibition the V&A commissioned a series of international architects to design and build six 1:1 scale structures throughout the Museum. The design driver for the experimental projects was to provide a small enclosed space that represented an escape from busy urban life. One of the central aims of the V&A was to provide an exhibition which moved away from explaining architecture through drawings and models and instead allowed the visitor to experience the architecture itself.

Entitled ‘Woodshed’ the Rural Studio pavilion was one of six winning designs commissioned for the V&A exhibition. The architectural design ethos of Rural Studio and previous collaboration with the University of Bath led to the innovative use of GFRP-DVW connections in the pavilion. Unlike all of the other exhibiting architects Rural Studio was the only organisation that was not part of a professional
practice. Instead the Rural Studio is a design and build architecture degree program run within Auburn University, Alabama. The studio was founded in 1993 by architects Samuel Mockbee and Dennis Ruth with the aim of making architecture students aware of the important role that their profession plays in shaping everyday lives. Each year architecture students working within the studio design and build several projects, which range from domestic houses to large civic buildings (Oppenheimer and Hursley, 2002). More than 80 projects have been completed since the studio was founded.

The concept for the ‘Woodshed’ installation at the V&A was based around a desire to make a structure from locally sourced softwood forest thinnings. Often this timber is either sold to make low value products such as fence posts or left standing due to the financial costs of thinning. However, the Rural Studio design intended to demonstrate that this low value timber can be used to provide aesthetically stimulating architecture. In doing so greater awareness of the material, and its potential value are expressed. Photographs of the installed pavilion are shown in Figure 9-1 and 9-2.

![Figure 9-1: ‘Woodshed’ pavilion at the V&A (Danny Wicke)](image)

The design of the pavilion was intended to embrace innovation and so the use of GFRP-DVW connections was seen to be particularly appropriate. Additionally the weight of the structure was reduced through the replacement of metallic elements and the connections were of relatively low cost. The structure itself was made up of 49 individual portals that were held together using threaded rod. Each portal was made with timber members measuring 150 x 150 mm cross section (Figure 9-3).
The frame connections were made using a single, central DVW plate and two 12 mm diameter GFRP dowels in each member (Figure 9-4). The pavilion was fabricated by a green oak carpentry company so the use of these materials was not common to the carpenters. This allowed a unique insight into how practical construction issues were dealt with and how the fabrication of GFRP-DVW connections could be improved in the future.

9.2 Connection design

Strength design methods for GFRP-DVW connections are presented in detail within Chapter 8. At the time of designing the Rural Studio pavilion connections
CHAPTER 9. DISCUSSION AND APPLICATION OF STUDY

Figure 9-3: Softwood timber air drying prior to fabrication

Figure 9-4: Fabrication of a GFRP-DVW connection
Figure 9-5: Connection assembled in the workshop

Figure 9-6: Connections in the finished pavilion
CHAPTER 9. DISCUSSION AND APPLICATION OF STUDY

these methods were not formally finalised and the connection strength design was based on data determined through experimental connection testing. Nonetheless the process of designing the connections highlighted the necessity for knowledge of dowel spacing rules as well as consideration of the DVW plate capacity. The pavilion connections used minimum end and dowel spacings of 4d parallel to grain. Perpendicular to grain, the same 4d spacings were used for the edge and dowel spacings. These spacing rules were chosen upon the basis of experimental tests reported in Chapter 5. Parallel to grain the spacing rules are lower than those prescribed by EC5 for metallic dowels.

Parallel to grain spacing rules for GFRP-DVW connections were of particular focus in this study. This was in response to the lower individual capacity of the GFRP fasteners in comparison to metallic counterparts. The advantage of reducing the spacing rules provided by EC5 for metallic dowels is that more efficient connection strength and stiffness can be gained. Based upon the experimental findings presented in Chapters 5 and 6, it is recommended that for parallel to grain loads a minimum end distance of 5d and a minimum in-line dowel spacing of 4d is used.

The analysis of connection slip under load is discussed in detail in Chapter 7. In particular the practical implications for design are highlighted. Accurate analytical modeling of connection slip is always complex. For the pavilion connection slip predictions were made based upon experimental data. In EC5 connection slip is dealt with through the provision of an empirical lower bound expression, which is derived from many sets of test data (Blass et al., 1995). For GFRP-DVW connections a beam on elastic foundation analysis was developed as part of this study to provide an analytical solution for predicting the slip of GFRP-DVW connections under load. This approach provided results that were in good agreement with those observed experimentally. However, for practical purposes it is more likely that the experimental data given in this thesis will provide the basis for stiffness design in future projects. If further connection tests are completed using different densities of timber then in the future either a more general empirical approach may be derived for GFRP-DVW connections or a tabulated data set may be generated using the beam on elastic foundation model.

9.3 Connection fabrication

Beyond the distinct advantage of corrosion resistance, the fabrication methods associated with GFRP-DVW connections could be considered their biggest advantage. To this end a qualitative comparison between the fabrication of metallic connections and GFRP-DVW connections is made in Chapter 6. The observations
reported in the discussion were made first hand and the fabrication methods used were based on experience of previously fabricating many test specimens. However, in meeting the carpenters who were fabricating the Rural Studio pavilion several key points were brought to light which are important to consider in the fabrication of these connections.

Firstly GFRP-DVW connections should be approached with the mindset of making a connection with materials that are vastly more workable than steel but still far harder than timber components. In this sense both the DVW plate and GFRP dowels can be readily handled and machined in-house; lowering costs and removing the necessity to employ an external contractor. However, it should not be assumed that the materials can be machined using carpentry techniques appropriate to green oak carpentry. The carpenters who were fabricating the connections initially treated the connections as you would a traditional carpentry connection and found fabrication of the DVW plate difficult due the assumption that conventional carpentry tools could be used. In particular drilling of the DVW plate should be made using an HSS drill bit suitable for metal and timber materials. It was found that the use of drill bits designed for use solely with timber were unsuitable and the holes were thus difficult to drill.

Due to the nature of the pavilion project the carpenters and museum staff were keen for all drilling to be completed prior to the arrival of the pavilion on site. This is contrary to the connection fabrication method discussed in Chapter 6. In this instance the plate and the member were drilled in one operation to ensure good alignment. It was therefore envisaged that for less sensitive applications structural members could be delivered to site with the DVW plate installed in one of the member ends. The plate could then be located on site and drilled in situ, thus allowing a tight tolerance fit to be made. Nonetheless, the experience of prefabricating the elements of the pavilion was positive and difficulty with alignment was not a problem. To an extent this could be attributed to the more pliable nature of GFRP dowels, which allows the accommodation of minor inaccuracies in alignment.

For the pavilion the carpenters wanted to use draw bore within the connections. Draw-bore is a technique used widely in traditional carpentry to tighten and pre-stress a green connection. The peg hole in the tenon member is offset from the hole in the mortice member by a set distance so that when the peg is driven through the two members draw tight together. The desire to use this technique was therefore in line with the careful design of green oak frames which must account for significant shrinkage of the timber upon drying. The rural studio pavilion was made using timber which was felled only 5-6 months prior to installation and so the moisture content was still relatively high upon installation. For this reason the carpenters were keen to use a hole offset. Experimental proof loading of a
connection demonstrated that this did not effect the connection strength and upon shrinkage the draw bore load on the dowels would relax making deconstruction of the pavilion easier at the end of the exhibition. In contemporary applications it is not anticipated that draw bore will be used. Principally this is because the timber is expected to be of a stable moisture content at the time of manufacture and because the potential to drill the plate onsite is removed.

The final observation relating to connection fabrication relates to dowel manufacture. This can be unpleasant if gloves are not worn due to small glass fibre splinters that can occur during the cutting of dowels. Therefore it is recommended that gloves are worn. Thin surgical gloves have been found to be very effective in this instance. Cutting of the dowels is best made using a water cooled diamond tipped saw, however a hacksaw is also adequate if a dust mask is worn. The ends of the dowels should be appropriately rounded using a circular sander or file to ease installation of the dowel into the interference fit holes.

9.4 Concluding comments

The main conclusions from this chapter are as follows:

- GFRP-DVW connections were used successfully in the Rural Studio ‘Woodshed’ structure at the V & A museum in London.
- Fabrication of DVW plate and GFRP dowels should be made with the mindset of using materials significantly more workable than steel but much harder than timber components.
- Complete pre-fabrication (including all drilling) was successful in the ‘Woodshed’ project. However, on-site drilling may also be considered for less sensitive structures.
- The nature of the ‘Woodshed’ installation meant that it was necessary to disassemble the structure at the end of the exhibition period. This was achieved without significant difficulty and demonstrates that GFRP-DVW connections can also be used in temporary, demountable applications.
Chapter 10

Conclusions and future work

10.1 Introduction

The main aim of this study was to innovate and develop a new type of mechanical, non-metallic timber connection. The proposed connection was to be suitable for mainstream, contemporary applications. This aim has been satisfied, though further research needs to be completed if the use and design of GFRP-DVW connections is to become commonplace.

Previous research completed at the University of Bath, studied traditional mortice and tenon connections and the use of GFRP dowels in timber connections. These studies, and the wider literature reviewed in Chapter 2, signaled a lack of knowledge and a requirement for research into providing a defined, non-metallic, connection system for contemporary applications. Building on previous research, a connection system, using GFRP dowels and DVW plates, has been defined, developed, and investigated experimentally. New methods of analysis are also proposed in recognition of the current lack of guidance for non-metallic timber connections.

This thesis provides an in depth introduction to the research, and design of contemporary non-metallic connections. It is anticipated that the study will act as a foundation and catalyst for further work. The focus of the work has been on experimental investigation of connection behaviour, under load and at failure. In line with this focus, analysis methods are proposed to predict connection behaviour. Main conclusions from the study are presented below in addition to recommendations for further research.
10.2 Conclusions

10.2.1 Non-metallic materials

Oak, GFRP and DVW dowels were investigated experimentally for their suitability to make dowel-plate type connections. DVW dowels are not recommended for structural applications due to their brittle mode of failure. Oak dowels used in conjunction with Birch plywood plates can be considered for low stress applications. GFRP dowels used in conjunction with DVW plates provide connections with high load capacity and post yield ductility under test loading.

10.2.2 Connection testing

Significant post yield ductility can be achieved, parallel to grain, by GFRP-DVW connections made with dowel spacings that are lower than those given by EC5 for metallic dowels. The use of EC5 minimum spacings for perpendicular to grain loads also provides significant post yield ductility.

Brittle connection failure can occur as a result of perpendicular to grain splitting of a timber member or as a result of tension failure of a DVW plate. These modes can be mitigated through appropriate design.

Locking and dissecting GFRP-DVW connections allowed the failure of GFRP dowels to be closely inspected. The dissected connections showed that a four hinge dowel failure is common to the connection type, irrespective of timber thickness. No embedment failure of the DVW plate was observed.

Parallel to grain loading of GFRP-DVW connections, using reduced dowel spacings, provided a capacity equal to the EC5 design capacity for a metallic equivalent and 50-60% of the experimental capacity of a metallic equivalent.

10.2.3 Stiffness modelling

Beam on elastic foundation analysis can be used as a means of modeling the stiffness of GFRP-DVW connections. However, for mainstream engineering design the method is relatively complex. An empirical design approach akin to that given by EC5 for metallic dowels would be more appropriate but requires further test data.
10.2.4 Strength modelling

The yield strength of a GFRP-DVW connection can be predicted using an EYM method of analysis. This analysis requires that appropriate values are used for the moment resistance of GFRP dowels.

Methods for predicting perpendicular to grain splitting failure and parallel to grain, partial shear plug, failure have been developed. This allows desirable, ductile post yield failure modes to be designed for.

10.2.5 Connection design and fabrication

GFRP-DVW connections were found to be significantly easier to fabricate than metallic dowel-plate connections.

DVW plate can be drilled using conventional HSS drill bits and hand operated drills. This allows the plate to be drilled in its installed position within a connection. Therefore, high dimensional accuracy and alignment can be achieved, facilitating simple assembly.

Both GFRP dowels and DVW plates can be machined using non-specialist machinery. This provides opportunities for in-house connection manufacture and on-site fabrication.

10.2.6 Overall conclusions

The work reported in this thesis clearly demonstrates the structural performance of GFRP-DVW, non-metallic connections.

The connection system provides a contemporary jointing system, which makes use of cost effective, commercially available materials.

The successful application of the connection system in the rural studio ‘Woodshed’ pavilion, at the V & A, demonstrates the systems potential.

10.3 Continuation of research

This thesis presents the development and experimental investigation of a new, non-metallic, connection system that uses non-conventional materials. The connection system has been shown to provide high load resistance and robustness under short
term, destructive testing loads. Further research is now required to investigate the long term performance of the connection.

Long term embedment creep tests on DVW plates are described in the literature in Chapter 2. These show encouraging results, however, long term creep tests for complete connection assemblies are required for the design of serviceability limit states. Additional consideration must be given to the environment in which the connection system will be used. The literature reviewed in Chapter 2 states that DVW plate can be reliably used for structural applications in service class 1 and 2 environments. However, for service class 3 applications, surface treatment of the DVW plate must be considered to prevent excessive moisture absorption. Long term performance must therefore also consider the climatic conditions in which to test.

The benefit of improved fire resistance is often cited for non-metallic connections. Non-metallic materials will not conduct heat into the middle of a connection in the same manner as metallic components, reducing charring around fasteners. Additionally, GFRP dowels are expected to retain their strength at elevated temperatures and ‘char’ at their ends. This is unlike metallic dowels, which lose strength at elevated temperatures and must be protected by timber plugs. In order to confirm the performance of GFRP-DVW connections, fire tests on loaded specimens are required.

Testing of full structural frame assemblies is required to further validate proposed analysis methods. These tests should also incorporate bending and shear testing of connections to allow design guidance to be developed further.

The consideration of variable load actions should be incorporated within a study of long term load performance. Limited cyclic loading tests are reported in this thesis for short term loading. The results show reliable elastic response but further work must investigate the performance under the action of many repeat load cycles.
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REFERENCES


REFERENCES


Appendix A

Beam on elastic foundation –
General solution

The derivation of the general solution for a beam on an elastic foundation with shear effects included is given in this appendix. The solution was originally proposed by Aydogan (1995) and includes the shear effect by an approximation that considers the deflection of the beam on a winkler foundation. A winkler foundation is a linear elastic model where contact pressure from the beam at any point along its length is assumed to be proportional to the settlement of the foundation (Wang et al., 2005). For this derivation the winkler foundation modulus is termed, $k$, and is defined as load per unit deflection per unit of dowel length.

![Figure A-1: Schematic diagram of beam on elastic foundation](image)

The Euler-Bernoulli differential equation for deflection, $s$, of a beam of constant cross section on a winkler elastic foundation can be expressed as below:

$$EI \frac{d^4 s}{dz^4} + ks = p$$  \hspace{1cm} (A.1)
APPENDIX A. BEAM ON ELASTIC FOUNDATION – GENERAL SOLUTION

where:

- $EI$ is the flexural rigidity of the beam
- $s$ is the deflection of the beam
- $k$ is the foundation modulus
- $p$ is the lateral load on the beam

The deflection $s$ consists of two parts:

$$s = s_b + s_v \quad \text{(A.2)}$$

where:

- $s_b$ is the deflection due to bending
- $s_v$ is the deflection due to shear

Differentiating equation A.2 twice with respect to $z$ yields:

$$\frac{d^2 s}{dz^2} = \frac{d^2 s_b}{dz^2} + \frac{d^2 s_v}{dz^2} \quad \text{(A.3)}$$

From beam bending theory $M = -EI \frac{d^2 s_b}{dz^2}$. Therefore the first term of equation A.3 can be expressed as below:

$$\frac{d^2 s_b}{dz^2} = -\frac{M}{EI} \quad \text{(A.4)}$$

Figure A-2: Shear deformation of small beam element
APPENDIX A. BEAM ON ELASTIC FOUNDATION – GENERAL SOLUTION

Shear deformation for a small segment of the beam is shown in Figure A-2. From the kinematic situation of the beam and Hooke’s law we can write:

\[ \tan \gamma \approx \gamma = \frac{ds_v}{dz} = \tau G \]  \hspace{1cm} (A.5)

where:
- \( \gamma \) is the shear strain
- \( \tau \) is the shear stress

The shear stress, \( \tau \), is equal to the shear force on the beam, \( T \), divided by its cross-sectional area, \( A \). Therefore, as \( z \to 0 \) equation A.5 can be rewritten as:

\[ \frac{ds_v}{dz} = \frac{T}{GA} \]  \hspace{1cm} (A.6)

Differentiating equation A.6 with respect to \( z \) yields:

\[ \frac{d^2 s_v}{dz^2} = \frac{1}{GA} \frac{dT}{dz} \]  \hspace{1cm} (A.7)

Looking at vertical equilibrium of the small beam element shown in Figure A-3:

\[ \frac{dT}{dz} = -(p - ks) \]  \hspace{1cm} (A.8)

Substituting equation A.8 into equation A.7 gives:
APPENDIX A. BEAM ON ELASTIC FOUNDATION – GENERAL SOLUTION

\[
\frac{d^2 s_v}{dz^2} = -\frac{1}{GA}(p - ks) \tag{A.9}
\]

Using equation A.4 and equation A.9 in equation A.3 the following equation can be written:

\[
\frac{d^2 s}{dz^2} = \frac{M}{EI} - \left(\frac{1}{GA}\right)(p - ks) \tag{A.10}
\]

The shear force, \(T\), in the beam is equal to the rate of change of moment, or \(\frac{dM}{dz}\). Using this relationship, and assuming that the beam has no distributed loads \((p = 0)\), substituting in for the shear force and differentiating equation A.10 twice yields:

\[
\frac{d^3 s}{dz^3} = -\left(\frac{1}{EI}\right) \frac{dM}{dz} + \left(\frac{k}{AG}\right) \frac{ds}{dz} = -\frac{T}{EI} + \left(\frac{k}{AG}\right) \frac{ds}{dz} \tag{A.11}
\]

\[
\frac{d^4 s}{dz^4} = \left(\frac{1}{EI}\right) \frac{dT}{dz} + \left(\frac{k}{AG}\right) \frac{d^2 s}{dz^2} = -\left(\frac{ks}{EI}\right) + \left(\frac{k}{AG}\right) \frac{d^2 s}{dz^2} \tag{A.12}
\]

Therefore:

\[
\frac{d^4 s}{dz^4} - \left(\frac{k}{AG}\right) \frac{d^2 s}{dz^2} + \frac{ks}{EI} = 0 \tag{A.13}
\]

Equation A.13 is the differential equation for a beam on an elastic foundation with shear effect included. It can more simply be expressed as:

\[
\frac{d^4 s}{dz^4} - 2\beta \left(\frac{d^2 s}{dz^2}\right) + \zeta s = 0 \tag{A.14}
\]

Where:

\[
\beta = \frac{k}{2AG} \tag{A.15}
\]


APPENDIX A. BEAM ON ELASTIC FOUNDATION – GENERAL SOLUTION

\[ \zeta = \frac{k}{EI} \]  (A.16)

This characteristic equation is a linear homogenous differential equation and so \( e^{az} \) can be considered as a solution. The characteristic equation A.14 therefore becomes:

\[ a^4 - 2\beta a^2 + \zeta \]  (A.17)

Consider that \( \zeta \geq \beta^2 \), which is the usual case, the roots of equation A.17 can found as below:

\[ a^2 = \frac{2\beta \pm \sqrt{4\beta^2 - 4\zeta}}{2} \]  (A.18)

\[ a^2 = \beta \pm \sqrt{\beta^2 - \zeta} \]  (A.19)

\[ a^2 = \beta \pm i\xi \]  (A.20)

Where:

\[ \xi = \sqrt{\zeta - \beta^2} \]  (A.21)

and

\[ i = \sqrt{-1} \]  (A.22)

Therefore the roots of the characteristic equation are:

\[ a_{1,2,3,4} = \pm \sqrt{\beta \pm i\xi} \]  (A.23)

The general solution to equation A.14 can therefore be expressed as below:
\[ s = \sum_{i=1}^{4} C_i e^{a_i z} \]  
(A.24)
Appendix B

Experimental data

This appendix presents experimental test data for the 50 specimens reported in Chapter 5. In addition to providing detailed experimental data the individual connection failure modes are reported. A summary of mean average results is given in Chapter 5. The connection configurations are given below and the data for individual specimens is given in Table B.

Table B.1: Test specimen configurations parallel to grain (dowel diameter, \( d = 12 \) mm for all tests)

<table>
<thead>
<tr>
<th>Test Group</th>
<th>Side member cross section ((b/t)) (mm)</th>
<th>Number of dowels (N)</th>
<th>End distance ((a_{3,\ell}))</th>
<th>Dowel spacing ((a_1))</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>96 x 48</td>
<td>1</td>
<td>5d</td>
<td>N/A</td>
</tr>
<tr>
<td>b</td>
<td>96 x 48</td>
<td>3</td>
<td>5d</td>
<td>5d</td>
</tr>
<tr>
<td>c</td>
<td>96 x 48</td>
<td>3</td>
<td>4d</td>
<td>4d</td>
</tr>
<tr>
<td>d</td>
<td>96 x 48</td>
<td>3</td>
<td>3d</td>
<td>3d</td>
</tr>
<tr>
<td>e_i</td>
<td>96 x 48</td>
<td>3</td>
<td>5d</td>
<td>3d</td>
</tr>
<tr>
<td>e_ii</td>
<td>96 x 48</td>
<td>3</td>
<td>3d</td>
<td>5d</td>
</tr>
<tr>
<td>f</td>
<td>96 x 75</td>
<td>3</td>
<td>3d</td>
<td>3d</td>
</tr>
</tbody>
</table>

(EC5 values: \( a_{3,\ell} = 7d, a_1=5d \))

Table B.2: Test specimen configurations perpendicular to grain (dowel diameter, \( d = 12 \) mm for all tests)

<table>
<thead>
<tr>
<th>Test Group</th>
<th>Side member cross section ((d/t)) (mm)</th>
<th>Number of dowels (N)</th>
<th>End distance ((a_{3,\ell}))</th>
<th>Dowel spacing ((a_1))</th>
</tr>
</thead>
<tbody>
<tr>
<td>g</td>
<td>84 x 48</td>
<td>1</td>
<td>5d</td>
<td>N/A</td>
</tr>
<tr>
<td>h</td>
<td>120 x 48</td>
<td>2</td>
<td>5d</td>
<td>5d</td>
</tr>
<tr>
<td>i</td>
<td>156 x 48</td>
<td>3</td>
<td>4d</td>
<td>4d</td>
</tr>
</tbody>
</table>

(EC5 minimum spacing rules were used in all cases)
Table B.3: Experimental data for individual specimens

<table>
<thead>
<tr>
<th>Specimen name</th>
<th>Yield load (kN)</th>
<th>Ultimate load (kN)</th>
<th>Stiffness (kN/mm)</th>
<th>Post yield failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>a-1</td>
<td>13.9</td>
<td>23.0</td>
<td>10.6</td>
<td>Cleavage failure of DVW plate</td>
</tr>
<tr>
<td>a-2</td>
<td>12.1</td>
<td>20.1</td>
<td>14.5</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>a-3</td>
<td>12.0</td>
<td>20.1</td>
<td>12.9</td>
<td>No brittle failure observed</td>
</tr>
<tr>
<td>a-4</td>
<td>12.9</td>
<td>21.0</td>
<td>30.6</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>a-5</td>
<td>12.9</td>
<td>21.4</td>
<td>9.4</td>
<td>No brittle failure observed</td>
</tr>
<tr>
<td>b-1</td>
<td>46.8</td>
<td>57.4</td>
<td>18.5</td>
<td>Tension failure in DVW plate</td>
</tr>
<tr>
<td>b-2</td>
<td>37.5</td>
<td>61.0</td>
<td>52.8</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>b-3</td>
<td>39.5</td>
<td>61.4</td>
<td>46.8</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>b-4</td>
<td>35.2</td>
<td>58.4</td>
<td>42.3</td>
<td>Tension failure of DVW plate</td>
</tr>
<tr>
<td>b-5</td>
<td>36.6</td>
<td>62.4</td>
<td>45.1</td>
<td>Splitting/Partial shear plug failure</td>
</tr>
<tr>
<td>c-1</td>
<td>36.9</td>
<td>47.3</td>
<td>45.2</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>c-2</td>
<td>41.4</td>
<td>51.1</td>
<td>48.7</td>
<td>Splitting/Partial shear plug failure</td>
</tr>
<tr>
<td>c-3</td>
<td>41.2</td>
<td>52.2</td>
<td>53.9</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>c-4</td>
<td>36.5</td>
<td>59.7</td>
<td>37.3</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>c-5</td>
<td>38.7</td>
<td>53.8</td>
<td>41.3</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>d-1</td>
<td>30.6</td>
<td>31.1</td>
<td>26.2</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>d-2</td>
<td>33.2</td>
<td>34.4</td>
<td>29.1</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>d-3</td>
<td>33.0</td>
<td>43.0</td>
<td>36.2</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>d-4</td>
<td>34.4</td>
<td>37.3</td>
<td>41.4</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>d-5</td>
<td>31.7</td>
<td>32.5</td>
<td>49.5</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
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<td>34.5</td>
<td>46.6</td>
<td>30.8</td>
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</tr>
<tr>
<td>e-2</td>
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<td>37.1</td>
<td>32.3</td>
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</tr>
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<td>32.8</td>
<td>47.2</td>
<td>38.3</td>
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</tr>
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<td>e-4</td>
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<td>50.0</td>
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<td>Partial shear plug failure</td>
</tr>
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<td>28.9</td>
<td>38.7</td>
<td>34.1</td>
<td>Partial shear plug failure</td>
</tr>
<tr>
<td>Specimen name</td>
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<td>Ultimate load (kN)</td>
<td>Stiffness (kN/mm)</td>
<td>Failure mode</td>
</tr>
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<td>----------------</td>
<td>--------------------</td>
<td>-------------------</td>
<td>-------------------------------------</td>
</tr>
<tr>
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</tr>
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</tr>
<tr>
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<td>48.9</td>
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</tr>
<tr>
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<td>13.3</td>
<td>5.3</td>
<td>Perp to grain split in line with dowel</td>
</tr>
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<td>14.5</td>
<td>5.4</td>
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<td>10.5</td>
<td>Perp to grain split in line with dowel</td>
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<td>20.2</td>
<td>22.2</td>
<td>17.6</td>
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<td>36.7</td>
<td>25.0</td>
<td>No brittle failure observed</td>
</tr>
<tr>
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<td>34.2</td>
<td>24.0</td>
<td>Perp to grain split in line with lowest dowel</td>
</tr>
<tr>
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<td>28.0</td>
<td>35.6</td>
<td>27.1</td>
<td>Perp to grain split in line with lowest dowel</td>
</tr>
<tr>
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<td>32.1</td>
<td>23.5</td>
<td>Perp to grain split in line with lowest dowel</td>
</tr>
<tr>
<td>i-5</td>
<td>28.0</td>
<td>29.2</td>
<td>24.8</td>
<td>Perp to grain split in line with lowest dowel</td>
</tr>
</tbody>
</table>
Appendix C

Published work

Contemporary metal free dowel connections for timber structures
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Proceedings of the 11th International Conference on Non-conventional Materials and Technologies, 2009

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Experimental performance of mechanical metal free timber connections loaded parallel to grain
Thomson, A., Harris, R., Ansell, M., Walker, P.
The Structural Engineer 88 (17), September 2010