Moment-resisting connections in glulam beams
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ABSTRACT
This paper describes an experimental investigation into the use of steel bars bonded into glulam timber beams with epoxy resins in order to provide moment-resisting joints between glulam beams and steel supports. The objective of this research was to provide information about the bending and shear strength of such connections.

Past research has investigated the use of steel rods epoxy bonded into glulam beams in order to study the tensile behaviour of such joints and to a limited extent has also studied the moment resistance of this type of joint. In the case of the tensile behaviour, past research at the University of Canterbury has investigated a number of parameters that affect joint strength.

This project has investigated both the moment and associated shear resistance of the joints when subjected to monotonic or cyclic loadings. The results of the experiments show that the main factor limiting the bending strength of steel rod connections into glulam members is the longitudinal splitting caused by the shear forces acting on the steel rods at, say, a beam to column interface. The strength was enhanced by providing transverse steel rods to act as a mechanical anchorage.

This technology requires further refinement before being recommended for widespread use, but the results to date show excellent potential.

INTRODUCTION
Over the last 20 years, there has been an increasing interest in the use of steel threaded rod epoxied into glulam timber members in order to fabricate moment-resisting joints. The use of epoxied steel rod technology for timber connections began in Denmark about 1980. Since then many studies have been carried out in Europe to investigate the reliability of this jointing method and a number of formulae have been presented to aid engineers in designing this type of timber connection (Riberholt 1988).

This connection method has been studied at the University of Canterbury with the aim of testing the European concept in New Zealand conditions. Townsend (1990) and Deng (1997) tested the tensile strength of single steel rods bonded into glulam using epoxy resins while Korin et al (1999) tested the tensile strength of single and multiple rods. In a study investigating the ductility of these joints in earthquake situations, Fairweather (1992) carried out tests on several moment-resisting connections and obtained hysteresis loops that showed good ductility and energy dissipation properties. However, one common feature of this research on tensile and bending capacity was the shear splitting failure of the glulam. Until now, no research appears to have been carried out to investigate the combined bending and shear behaviour of such timber connections. Structural engineers designing these connections have to provide either a large reserve capacity against shear failure or some additional steel rods to carry the expected shear force.

The objective of this study was to develop a high strength moment-resisting connection having reliable shear strength and significant ductility that could perform well in seismic conditions. To achieve this goal, pull-out and direct shear tests were carried out in addition to monotonic and cyclic moment tests on typical moment-resisting joints suitable for use in a glulam frame structure.

TYPICAL MOMENT-RESISTING CONNECTIONS
The particular connection tested comprised eight threaded bars epoxy bonded into a 1.36 m length of nominally 495 x 135

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mm cross-section glulam timber. The bars were screwed into couplers that were also bonded into the glulam. This enabled bolts to be screwed into the couplers to fasten the glulam member to a steel support. Three proposed arrangements of the threaded bars were tested using two specimens for each arrangement. The three arrangements tested were as shown in Figure 1. Each arrangement comprised four threaded bars grouped together at each side, the differences between the arrangements being:

1. for the centre bar specimens, two additional bars were placed at the mid-depth in order to resist the shear force.
2. for the angle bar specimens the two groups of bars were aligned at 20° to the longitudinal axis.
3. for the tie bar specimens a transverse mild steel tie bar was provided near the support to aid in resisting any shear cracking.

Figure 1  Epoxy bonded threaded bar arrangements used for the test specimens

The test arrangement used is shown in Figure 2. The steel frame was designed to match up with the holding down bolts in the strong floor.
MONOTONIC LOADING TESTS

The aim of these tests was to test the strength of the glulam. The threaded bars used in these tests were of high tensile steel having a yield strength of 680 MPa while the commercial couplers had an ultimate strength of about 150 kN. The bar arrangement that gave the best performance was then used for the cyclic test specimens.

A summary of test results is given in Table 1. The angle bar specimens both failed in shear as did centre bar specimen CB300-1. Centre bar specimen CB300-2 failed in tension. In the tie bar specimens TB300-1a and 1b, it was found that “failure” was caused by fracture of the couplers on the tension side of the connection. As a result of this, the tie bar tests were repeated (TB300-2a and 2b) using couplers fabricated from high tensile steel and having an ultimate strength of about 190 kN. Specimen TB300-2a “failed” due to yielding in the support frame while TB300-2b failed in tension. On the basis of the New Zealand timber code, the allowable shear stress on the glulam section is 3.35 MPa. From Table 1 it can be seen that only the improved tie bar specimens (TB300-2a and 2b) achieved shear stresses above the maximum allowable code shear strength.
Table 1  Stress distribution in the specimens in the monotonic tests

<table>
<thead>
<tr>
<th>Type</th>
<th>Test number</th>
<th>Lateral force, F</th>
<th>Reduced moment M\textsubscript{1}</th>
<th>Maximum shear stress</th>
<th>Timber stress at M\textsubscript{1}</th>
<th>Ave force in bars</th>
<th>Ave stress in bars</th>
<th>Tension in timber $^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle bar</td>
<td>AB300-1</td>
<td>93.2</td>
<td>91.8</td>
<td>2.07</td>
<td>16.3</td>
<td>90.0</td>
<td>354</td>
<td>23.6</td>
</tr>
<tr>
<td>Angle bar</td>
<td>AB300-2</td>
<td>92.1</td>
<td>90.7</td>
<td>2.06</td>
<td>16.2</td>
<td>88.9</td>
<td>350</td>
<td>23.3</td>
</tr>
<tr>
<td>Centre bar</td>
<td>CB300-1</td>
<td>120.1</td>
<td>118</td>
<td>2.67</td>
<td>21.0</td>
<td>116</td>
<td>476</td>
<td>30.6</td>
</tr>
<tr>
<td>Centre bar</td>
<td>CB300-2</td>
<td>138.5</td>
<td>136</td>
<td>3.10</td>
<td>24.4</td>
<td>134</td>
<td>549</td>
<td>35.5</td>
</tr>
<tr>
<td>Tie bar</td>
<td>TB300-1a</td>
<td>101.9</td>
<td>100</td>
<td>2.27</td>
<td>18.0</td>
<td>99.5</td>
<td>406</td>
<td>26.1</td>
</tr>
<tr>
<td>Tie bar</td>
<td>TB300-1b</td>
<td>124.7</td>
<td>122</td>
<td>2.78</td>
<td>22.0</td>
<td>121</td>
<td>497</td>
<td>31.9</td>
</tr>
<tr>
<td>Tie bar</td>
<td>TB300-2a</td>
<td>157.0</td>
<td>154</td>
<td>3.50</td>
<td>27.6</td>
<td>152</td>
<td>624</td>
<td>40.1</td>
</tr>
<tr>
<td>Tie bar</td>
<td>TB300-2b</td>
<td>153.7</td>
<td>151</td>
<td>3.42</td>
<td>27.0</td>
<td>149</td>
<td>611</td>
<td>39.3</td>
</tr>
</tbody>
</table>

Notes:  
1 Moment at the level of the ends of the threaded bars (= F * 0.985).  
2 Based on the cross-sectional area of the breadth (135 mm) and the depth of timber from the outer edge to the inner coupler (113 mm).

CYCLIC LOADING TESTS

The strongest threaded bar arrangement from the monotonic loading tests, i.e. the tie bar arrangement, was used for these tests. Four specimens were tested, two specimens in stage one and a further two specimens using a modified arrangement based on the results of the pull-out tests. The threaded bars used in these tests were threaded reinforcing bars of grade 300 steel. The purpose of these tests was to investigate the ductility of the connection to see if it could satisfy the ductility demand that could arise in seismic conditions, with yielding occurring in the threaded bar. While it is better to have ductility outside the glulam beams (e.g. Fairweather, 1992) in order to allow inspection and replacement, this project looked at the possibility of obtaining ductility within the beams.

On the basis of the monotonic testing, the tie bar specimens should have been able to resist a load of about 150 kN but both specimens failed at loads of about 120 kN. Both specimens failed during testing to a ductility of 4. During both tests, shear cracks were observed to propagate upwards from the bottom during the displacement cycling. The reduction of ultimate shear resistance was probably due to this crack formation on both sides of the specimens reducing the effective timber cross-section available to resist the shear.

Based on the first two tests and the results of the pull-out tests, the following modifications were made to the next test specimens:
1 the tie bar diameter was increased from 12 mm to 16 mm in order to provide greater resistance to the development of shear cracks.
2 a 50 mm length of the embedded bars near the couplers was wrapped in insulation tape in order to provide a region of the threaded bar over which yield propagation could occur and thus improve the ductility of the connection.

Table 2  Summary of test results for the cyclic tests

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Maximum lateral force</th>
<th>Maximum Shear stress</th>
<th>Maximum beam deflection</th>
<th>Yield deflection</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$kN$</td>
<td>$MPa$</td>
<td>$mm$</td>
<td>$mm$</td>
<td></td>
</tr>
<tr>
<td>TB300-4</td>
<td>117</td>
<td>2.63</td>
<td>70</td>
<td>22.8</td>
<td>Shear crack propagation</td>
</tr>
<tr>
<td>TB300-5</td>
<td>121</td>
<td>2.72</td>
<td>70</td>
<td>19.4</td>
<td>Tensile failure in timber</td>
</tr>
<tr>
<td>TB350-6</td>
<td>104</td>
<td>2.33</td>
<td>81.5</td>
<td>25.7</td>
<td>Test specimen tilted sideways and eventually testing was stopped</td>
</tr>
<tr>
<td>TB350-7</td>
<td>105</td>
<td>2.36</td>
<td>91</td>
<td>24.2</td>
<td>Large shear crack formed</td>
</tr>
</tbody>
</table>
A summary of the test results is given in Table 2 and the load-displacement hysteresis plots are shown in Figure 3. The pinched loops were due to the couplers punching through the baseplate when they were under compression.

During the cyclic testing of specimen TB350-6, the specimen tilted sideways as the displacement ductility was increased from 2 to 4. This tilting was caused by progressive crushing of the timber on one of the specimen and resulted in the test being stopped when the specimen was no longer sufficiently stable to be safe. However, it was observed that the peak loading recorded had dropped from a maximum load of 108 kN to 81 kN. As this drop in maximum load was more than 20%, the connection was considered to have failed. When testing specimen TB350-7, the specimen was propped laterally in an attempt to prevent sideways tilting, and as well, two complete cycles of loading to a ductility of 3 were added. The specimen resisted the displacement demand at ductility 3 but when the testing continued to a ductility of 4, a large shear crack formed that propagated from the bottom to top of the specimen to give a complete shear failure.

From the test results, the reliable ductility of the connection was 3. After introducing a 50 mm yield propagating region in the threaded bar, as well as increasing the size of the tie bar, the number of shear cracks formed during the cyclic testing were reduced in number and size. The modifications could be considered to be effective in that the timber cracking was reduced, the ductility level reached remained the same – albeit with larger overall deformation – though the strengths were slightly reduced.

**PULL-OUT TESTS**

The aim of these tests was to investigate the following:

- The location of yield in the threaded bars within the timber specimen,
- The possibility of improving the ductility of the threaded bar and coupler connection through the introduction of a yield propagation region.

The pull-out tests carried out were:
1. (PO-1) Two threaded reinforcing bars were fitted into a common coupler and tested in tension to determine the ultimate tensile strength and the likely yield location.

2. (PO-2) A length of glulam of size 90 x 90 x 750 mm connected by threaded bars with strain gauges attached was tested in tension.

3. (PO-3) A length of glulam having the same size as for the PO-2 tests but with the first 50 mm of the threaded bar near the coupler having insulation tape applied in order to provide a debonded region over which yield propagation could occur without constraint from the surrounding timber. The bonded length was the same as for the PO-2 tests.

The average results for the three sets of tests carried out are given in Table 3.

<table>
<thead>
<tr>
<th>Test</th>
<th>Yield strength MPa</th>
<th>Ultimate strength MPa</th>
<th>Yield displacement mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>PO-1</td>
<td>285</td>
<td>350</td>
<td>34</td>
</tr>
<tr>
<td>PO-2</td>
<td>308</td>
<td>431</td>
<td>14</td>
</tr>
<tr>
<td>PO-3</td>
<td>332</td>
<td>460</td>
<td>24</td>
</tr>
</tbody>
</table>

Observations
1. The strength of the connection was not reduced by the inclusion of the debonded region.
2. When the threaded bar was bonded to the timber members using (rigid) epoxy, the yield displacement was reduced to about 1/3rd of that when it was tested on its own (PO-1). However, with a 50 mm debonded region near the coupler, the yield displacement of the connection increased to about 2/3rd of that of the threaded bar itself. Thus the provision of the debonded region greatly improved the ductility of the tension specimens.

DIRECT SHEAR TESTS

The aim of the direct shear tests was to investigate the strength of the connection in pure shear. Six test specimens were prepared and tested under direct shear force. The specimens all had a length of 580 mm. Since the width of the timber specimens varied, three specimens had two bolts and three had only one bolt for the connection at each end, all bolts being placed at the mid-depth of the specimens. The test specimens were tested in either one-point loading at the centre of the span or two-point loading near each end support. In the one-point tests, the load was applied over a bearing length of 200 mm while in the two-point tests the load was applied over a bearing length of 100 mm at each end. A summary of the test results is given in Table 4, other details can be found in Wong (2000).

<table>
<thead>
<tr>
<th>Test description</th>
<th>Section properties</th>
<th>No. of bolts at each end</th>
<th>Ultimate Shear force kN</th>
<th>Shear force Per bolt kN</th>
<th>Shear strength (Johansen) kN</th>
<th>Timber Shear stress MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>width mm</td>
<td>depth mm</td>
<td>2</td>
<td>2</td>
<td>170</td>
<td>42.5</td>
</tr>
<tr>
<td>1</td>
<td>2 point</td>
<td>130</td>
<td>242</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1 point</td>
<td>130</td>
<td>243</td>
<td>2</td>
<td>168</td>
<td>42.1</td>
</tr>
<tr>
<td>3</td>
<td>1 point</td>
<td>133</td>
<td>286</td>
<td>2</td>
<td>183</td>
<td>45.7</td>
</tr>
<tr>
<td>4</td>
<td>1 point</td>
<td>90</td>
<td>268</td>
<td>1</td>
<td>101</td>
<td>50.4</td>
</tr>
<tr>
<td>5</td>
<td>2 point</td>
<td>90</td>
<td>269</td>
<td>1</td>
<td>88</td>
<td>44.0</td>
</tr>
<tr>
<td>6</td>
<td>1 point</td>
<td>90</td>
<td>174</td>
<td>1</td>
<td>82</td>
<td>41.0</td>
</tr>
</tbody>
</table>

The equation of Johansen used for determining the shear strength in Table 4, assumes that failure occurs by crushing of the timber under the bar and coupler. An alternative method would be to assume that all the shear force is resisted by bearing on the coupler alone. In this case, using the characteristic strength for compression perpendicular to the grain from NZS 3603 the shear strength resisted by each bar is 34.7 kN which is close to the value used in Table 4.

The tests showed little difference between the two-point and one-point tests in terms of strength. The assumption that all the shear force is resisted by bearing between the wood and the coupler gave a good (conservative) estimate of the shear
strength. The main difference between the one-point and two-point shear tests is in their failure crack patterns. Comparing the results of the direct shear tests with the monotonic and cyclic loading tests, the ultimate shear strengths were similar to those from the tests on the tie bar specimens.

**DISCUSSION**

In order to explain the difference in performance between the three bar arrangements used in the monotonic tests, the load paths in the specimen need to be investigated.

In the case of the tie bar specimens, Figure 4(a) shows the assumed internal load path if the shear force is resisted entirely in the compression region of the support while Figure 4(b) shows the possible load path if the shear force is resisted by the tension bars. In the former case, there is an effective compression strut running from the point of application of the lateral force down to the steel bars in the compression region of the beam. In this case the tie bar does not carry any force but does help to prevent any shear crack formulation. If on the other hand, the shear force is resisted solely at the tension side of the beam, then in order to provide force equilibrium in the timber near the support, the tie bar must transfer the shear force back to the compression face. Therefore the tie bar is essential for internal force equilibrium. Thus the strength of the tie bar required varies from 0-100% of the shear force depending on the load path depending on how the connection was detailed to resist the shear force.

In the case of the central bar specimens (Figure 4c and d), show possible load paths. In (c), the two compression struts are connected by tension forces acting perpendicular to the grain. However, if the internal load path were to be as in (d), the shear strength of the connection may be weaker than expected as tension perpendicular to the grain stresses are developed near the support, on the compression side of the beam.

The location where the shear force is resisted depends on the geometry of the holes and bars. In the tests carried out in this project, it was not possible to determine which bars resisted the shear force because all the holes in the base support were drilled oversize. In this situation, it would seem prudent to design for option (b) of Figure 4.
In the case of the angle bar specimens, a high stress zone may occur near the ends of the threaded bars where the shear stress is no longer distributed over the full cross-section of the specimen but becomes concentrated in a small volume of wood leading to a shear crack being propagated along the grain as shown in Figure 5.

**SUMMARY AND CONCLUSIONS**

On the basis of the tests described in this paper, the following conclusions can be drawn:

- A high strength moment connection can be developed by using a threaded bar – epoxy resin connection.
- A reliable shear resisting, threaded bar connection can be developed by using a small diameter threaded bar as a tie bar near the support. The tie bar provided should have a minimum strength of 25% of the design shear force pending further research.
- A ductility of 3 was achieved in the epoxy-bar connection having a tie-bar near the support, the ductility being limited either by shear failure of the glulam or by pull-out of the bars.
- Without the tie-bars near the supports, shear failure was the commonest failure mode of the test specimens.
- For the specimens with threaded bars arranged at an angle to the grain, premature failure of the specimens occurred.
- On the basis of the direct shear tests, the provision of dowels at the mid-depth of the section provides good shear transfer between glulam beams and steel supports.
- Precise fabrication of the steel support and placement of the bars in the glulam is required.
- The steel couplers worked well.

**REFERENCES**


