Ductile moment-resisting connections in glulam beams

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ABSTRACT: This paper describes an experimental investigation into the use of epoxied steel bar connections in glulam beams. The tests confirm that high strength moment connections can be achieved with threaded steel rods epoxied into the end-grain of glulam timber, bolted to steel brackets. For ductile seismic design with the epoxied steel rods designed to yield in tension, a displacement ductility of 3 was achieved, but brittle failures occurred at larger displacements. Ductile behaviour of glulam frames is better designed with yielding of the steel connecting brackets rather than yielding of the epoxied rods.

The main factor limiting the strength and ductility of these connections is longitudinal splitting caused by shear forces near the beam-to-bracket interface. Shear strength can be enhanced by providing transverse steel rods epoxied through the full depth of the beam. Threaded bars arranged at an angle to the grain are not recommended, because they result in premature shear failures.

1 INTRODUCTION

The overall objective of this research programme is to develop a high strength moment-resisting connection having reliable shear strength and significant ductility. The intended application is in seismic design of moment-resisting glulam frame structures where the connections include epoxied steel rods and steel brackets.

The use of this technology began in Denmark about 1980. A number of design formulae have been presented in Europe (Riberholt 1988, Johansen 1995). In New Zealand, Townsend (1990) and Deng (1997) tested the tensile strength of single steel rods while Korin et al (1999) tested multiple rods. New Zealand design recommendations are given by Buchanan and Fairweather (1993) and Buchanan and Moss (1999).

For non-seismic conditions, timber structures are typically designed with strength to resist the factored load combinations, with stresses generally remaining in the elastic range. If moment-resisting glulam frame structures are to be designed for ductile response to seismic forces, it is essential that the steel components in the connections be able to provide sufficient ductility without premature failure elsewhere, including the following failure modes:

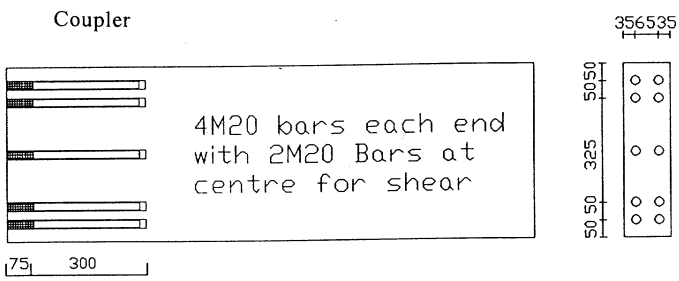
- Tensile yielding of the steel connecting bracket
- Tensile yielding of the steel rods
- Pullout failure of the epoxy
- Tensile failure of the threaded couplers
- Compression punching of the couplers into the steel connecting bracket
- Tensile failure of the timber near the end of the epoxied rods
- Shear failure of the timber between the two groups of rods

Fairweather (1992) carried out cyclic tests on moment-resisting connections obtaining good ductility and energy dissipation. Best results were obtained with the glulam members bolted to ductile steel connecting brackets. Ductile damage to the brackets allows inspection and replacement after an earthquake.

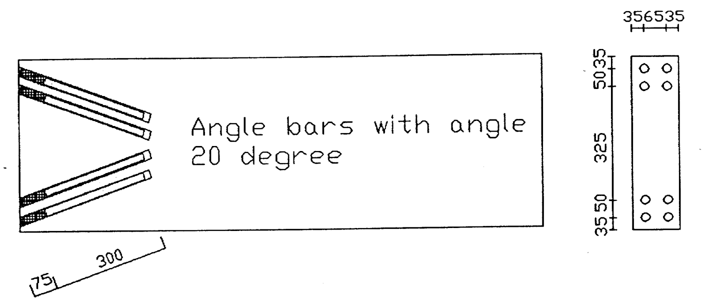
The present study investigates whether adequate ductility can be achieved from yielding of the epoxied steel rods, rather than yielding of the steel brackets. Several tests have resulted in a brittle shear failure in the timber before reaching the design level of flexural strength or ductility. A contributing factor can be tension perpendicular-to-grain stresses resulting from shrinkage of the glulam, if the beams are bolted to rigid steel brackets at high moisture content, then allowed to dry out while restrained. Structural engineers must provide a large reserve capacity against shear failure or some additional transverse epoxied steel rods to carry the expected shear force.

In typical details, the steel rods protrude 50mm from the end of the glulam, for fixing with steel nuts, but a better detail is to use slightly shorter rods with an internally threaded steel coupler. The couplers protect against damage in transit, and allow yielding of the bars or brackets in compression as well as in tension. This detail has been used in the Sydney 2000 Olympics buildings and elsewhere.

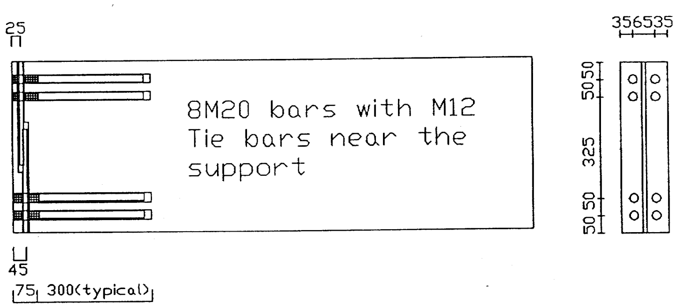
The objectives of the present experimental study were to investigate whether adequate ductility can be achieved with yielding of the epoxied steel rods within the glulam, rather than yielding of the steel brackets and to compare arrangements of reinforcing within the glulam to prevent premature shear failures.



Centre bar specimens arrangement



Angle bar specimens arrangement



Tie bar specimens arrangement

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

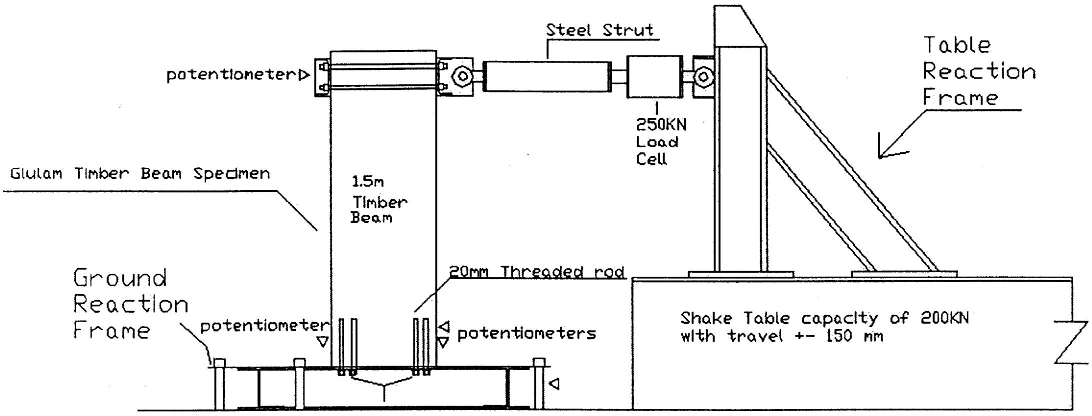


Figure 2 Elevation of the test frame setup

2 MOMENT-RESISTING CONNECTIONS

All the tested beams consisted of eight threaded bars epoxied into a 1.36 m length of 495 x 135 mm glulam timber. The bars were screwed into couplers before being epoxied into the glulam. The eight main bars were designed to resist the bending moment. Three arrangements of bars were tested to investigate shear resistance, as shown in Figure 1.

- For the *centre bar specimens*, two additional bars were placed at the mid-depth.
- For the *angle bar specimens* the two groups of main bars were aligned at 20°.
- For the *tie bar specimens* a transverse mild steel tie bar was provided near the support.

The test arrangement used is shown in Figure 2. The H-shaped ground reaction frame was designed to suit the holding down bolt locations in the strong-floor.

3 MONOTONIC LOADING TESTS

The aim of these tests was to test the strength of the glulam in flexure and shear, for the three different bar arrangements. The threaded bars used in these tests were AISI Grade 4140 high tensile steel having a yield strength of 680 MPa.

Table 1 Stress distribution in the specimens in the monotonic tests

Type of bar arrangement	Test number	Lateral force, F	Reduced moment M_1[1]	Maximum shear stress	Timber stress at M_1	Avg force in bars	Avg stress in bars	Tension stress in timber[2]
		kN	*kNm*	*MPa*	*MPa*	*kN*	*MPa*	*MPa*
Angle bar	AB-1	93.2	91.8	2.07	16.3	90.0	354	23.6
Angle bar	AB-2	92.1	90.7	2.06	16.2	88.9	350	23.3
Centre bar	CB-1	120.1	118	2.67	21.0	116	476	30.6
Centre bar	CB-2	138.5	136	3.10	24.4	134	549	35.5
Tie bar	TB-1a	101.9	100	2.27	18.0	99.5	406	26.1
Tie bar	TB-1b	124.7	122	2.78	22.0	121	497	31.9
Tie bar	TB-2a	157.0	154	3.50	27.6	152	624	40.1
Tie bar	TB-2b	153.7	151	3.42	27.0	149	611	39.3

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).

A summary of test results is given in Table 1. The angle bar specimens both failed in shear with a longitudinal crack down the centre of the beam, starting between the ends of the angled bars. The centre bar specimen CB-1 also failed in shear. Centre bar specimen CB-2 failed in tension. In the tie bar specimens TB-1a and 1b, failure was caused by fracture of the couplers on the tension side of the connection. A separate test showed that the commercial "Vemo" couplers had an ultimate strength of about 150 kN. As a result of this, the tie bar tests were repeated (TB-2a and 2b) using new high tensile steel couplers. Specimen TB-2a "failed" due to yielding in the support frame while TB-2b failed in tension.

The characteristic shear strength in the New Zealand timber code (NZS 3603:1993) is 3.8 MPa. From Table 1 it can be seen that only the improved tie bar specimens (TB-2a and 2b) achieved shear stresses close to the code level of shear strength.

4 CYCLIC LOADING TESTS

The strongest bar arrangement from the monotonic tests (i.e. tie bars) was used for the cyclic tests. The bars in these tests were Grade 300 steel reinforcing bars with threaded ends. The purpose was to investigate the ductility of the connection with yielding occurring in the bars.

A summary of the test results is given in Table 2 and the load-displacement hysteresis plots are shown in Figure 3. On the basis of the monotonic testing, the tie bar specimens should have been able to resist a load of about 150 kN without a shear or tension failure in the timber, but both specimens failed at loads of about 120 kN. Both specimens failed after several cycles of cyclic loading, before reaching a ductility of 4. During both tests, shear cracks were observed to propagate from the end of the beam. The reduction of ultimate shear resistance was probably due to these cracks reducing the effective timber cross-section available to resist the shear force.

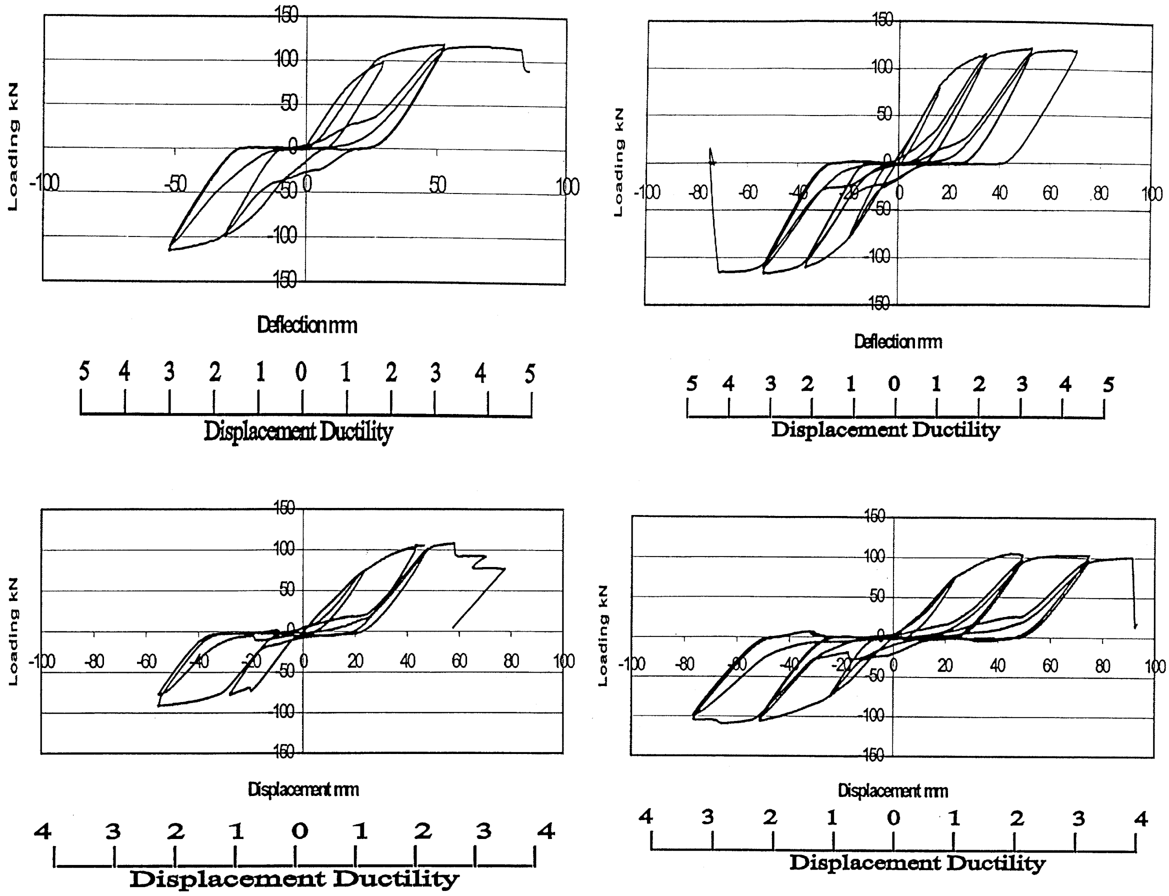


Figure 3 Load-displacement hysteresis loops for the cyclic load tests

The pinched hysteresis loops were due to the couplers punching through over-size holes in the base-plate when they were in compression. This undesirable behaviour could be prevented in the future by ensuring that the holes in the steel plate are not much larger than the diameter of the steel rods, but still allowing sufficient tolerance for construction. Punching of the couplers does not cause total failure, but it does result in undesirable pinching of the hysteresis loops.

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

During the cyclic testing of specimen TB-6, the specimen tilted sideways as the displacement ductility was increased from 2 to 4. This tilting was caused by lack of bracing, accompanied by progressive crushing of the timber on one side of the specimen. The test was stopped when the specimen was not sufficiently stable to be safe, and the load had dropped from a maximum of 108 kN to 81 kN. Specimen TB-7 was propped laterally to prevent sideways tilting, and two complete cycles of loading to a ductility of 3 were added. The specimen achieved ductility of 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 beam.

Table 2 Summary of test results for the cyclic tests

Test number	Maximum lateral force	Maximum shear stress	Maximum beam deflection	Deflection at first yield	Failure mode
	kN	*MPa*	*mm*	*mm*	
TB-4	117	2.63	70	22.8	Shear failure
TB-5	121	2.72	70	19.4	Tensile failure in timber
TB-6	104	2.33	81	25.7	Specimen tilted sideways
TB-7	105	2.36	91	24.2	Shear failure

From the all the cyclic test results, the reliable ductility of the connection was found to be not greater than 3. The “improvements” of introducing a 50 mm unbonded yield region in the steel bar, and increasing the size of the tie bar, caused the number of shear cracks formed during the cyclic testing to be reduced in number and size, but there was no significant improvement in overall performance.

Comparison of Tables 1 and 2 shows lower failure loads in the cyclic tests, as expected because of the lower strength steel rods. Two of the tests failed prematurely due to poor quality wood (tensile failure) and instability, but the other two exhibited unexpected shear failures with shear stresses lower than those obtained in the final two monotonic tests. This suggests that the yielding of the steel rods inside the timber is creating internal damage leading to shear failures.

The test series was terminated at this point for operational reasons, but more testing is necessary to resolve the questions raised. In particular the base-plate should be made with closer tolerances to prevent the couplers punching through in compression, and suitable quality timber should be used to prevent premature timber failures.

5 TENSION TESTS

Two axial tension tests were carried out to investigate the location of yielding in the threaded bars within the timber specimen, and the possibility of improving ductility with the introduction of an unbonded yield propagation region. A threaded bar instrumented with strain gauges was epoxied into a piece of glulam timber 90 x 90 x 750 mm. The assembly was tested in tension. In the second test, the first 50 mm of the threaded bar near the coupler had insulation tape applied to provide a debonded region. The average results for the three sets of tests are given in Table 3.

Table 3 Summary of the tension test results

Test	Steel stress at yield	Steel stress at ultimate load	Yield displacement
	MPa	MPa	mm
Fully epoxied	308	431	14
50mm debonded	332	460	24

Table 3 shows that the strength of the connection was not reduced by the inclusion of the debonded region, but the yield displacement increased 71% from 14mm to 24mm.

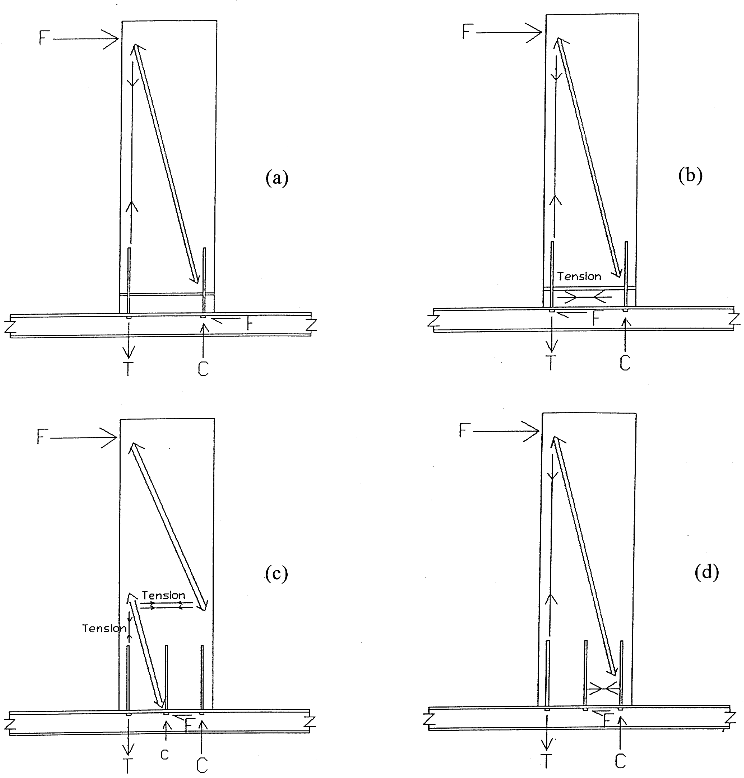


Figure 4 Possible load paths based on strut and tie models for the monotonic test specimens.
(a) and (b) tie bar specimens
(c) and (d) centre bare specimens

6 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.

For 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 a 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 compression region of the beam. In this case the tie bar does not carry any force but can help to prevent any shear cracks. If on the other hand, the shear force is resisted solely at the tension side of the beam, then the tie bar must transfer the shear force back to the compression face, and the tie bar is essential for internal equilibrium. Thus the strength of the tie bar required varies from 0-100% of the shear force depending on the load path, which in turn depends on how the connection is detailed and constructed and how much tolerance there is in the bolt holes.

The intention in design of the central bar specimens is for the outer bars to resist all of the bending moment and the central bars to resist all of the shear force. Figures 4(c) and (d), show possible load paths. In (c) the two compression struts are connected by tension forces acting perpendicular to the grain, in the body of the beam. 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. Cracking in this region could then weaken the beam when the load is reversed and the damaged region comes into tension.

For both the tie bar arrangement and the central bar arrangement, the location where the shear force is resisted depends on the relative locations and sizes 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, which was not satisfactory. In the case of the angle bar specimens, the test results show that a high stress zone will 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. A suggestion for eliminating this stress concentration is to extend the bars to pass each other in an X-configuration, but there are problems of accuracy of drilling and congestion inside the glulam.

Considering the test results and this discussion, the preferred option for resisting shear forces is that shown in Figure 4(b), but allowing for the timber to resist some of the tensile force in tension perpendicular to the grain. It is suggested that the tie bar should have a minimum strength of 25% of the design shear force, pending further research.

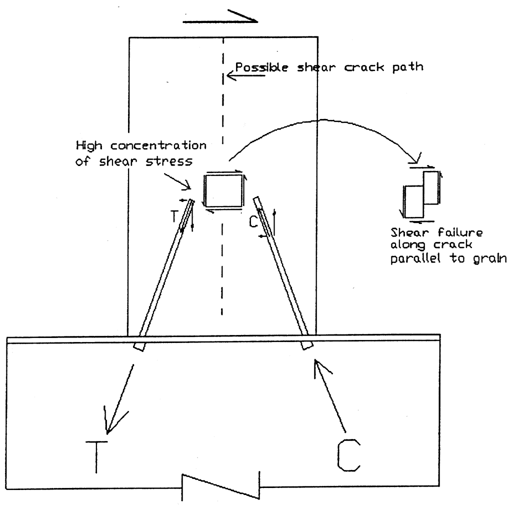


Figure 5 Predicted load paths for the angle bar monotonic test specimens

7 CONCLUSIONS

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

- High strength moment connections between glulam beams and steel connecting brackets can be achieved with threaded steel rods epoxied into the end-grain of glulam timber.
- These connections are most suitable for gravity loads and non-ductile seismic design.
- For ductile seismic design, yielding of steel connecting brackets is preferred to yielding of the epoxied rods.
- In connections where the epoxied steel rods are designed to yield in tension, a displacement

ductility of 3 can be achieved. This type of yielding is not currently recommended for ductile design because of brittle failures which occur at larger displacements.

- It is recommended that all high strength moment connections (seismic and non-seismic) be reinforced in shear with a small diameter threaded tie bar near the support, across the full depth of the beam. The tie bar should have a minimum strength of 25% of the design shear force, pending further research.
- Threaded bars arranged at an angle to the grain are not recommended, because they cause premature shear failure in the timber between the ends of the bars.
- "Vemo" steel couplers work well with Grade 300 steel rods. They cannot develop the full strength of AISI Grade 4140 high strength steel.
- Compression punching of the couplers into the steel connecting bracket should be prevented by ensuring that the drilled holes in the bracket are not much larger than the bar diameter.

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