INFORMATION SHEET

STRUCTURAL CONNECTIONS

NZ WOOD
For a better world

EPOXY GROUTED STEEL RODS
DESIGN DATA

The information provided below has been taken from the New Zealand Timber Design Guide 2007, published by the Timber Industry Federation and edited by Professor A H Buchanan. To purchase a copy of the Timber Design Guide, visit www.nztif.co.nz

MATERIAL SELECTION

Steel bars

Both deformed reinforcing and fully threaded steel bars can be used, but threaded steel bars are recommended.

Load transfer between steel rods and epoxy is mechanical, not adhesive, and threaded bars give greater pull-out strength than reinforcing bars for a given embedment length because they have lower stress peaks.

The surface deformations on the deformed bars produce more prying and wedging action than fully threaded bars, resulting in further splitting failures.

Grade 300 steel is most commonly used for the rods, however, high-strength threaded bars (AISI grade 4140, yield stress 680 MPa) may be used to reduce bar size.

Timber

The design equations were originally based on the use of No 1 framing grade radiata pine glulam timber, but can be used for design of GL8 or better grade glulam, or any grade of laminated veneer lumber (LVL).

It is recommended that wood containing pith, and concentrations of other potential defects, such as knots and finger joints, be avoided as far as possible in the region of the connection, and that the timber should be dry.

A reduction factor for timber moisture content greater than 15 percent but less than 22 percent is given below. The moisture content of the glulam must be kept below 22 percent to prevent loss of strength under long-term loads.

Epoxy

The following types of epoxy are recommended, having provided satisfactory test results and performance in service:

- Araldite 2005 (Nuplex Construction Products)
- Araldite K-80 (Nuplex Construction Products)
- West System ADR310/ADH26 (Adhesive Technologies Ltd)
- West System Z105/Z205 or Z105/Z206 (Adhesive Technologies Ltd)
- East 221 epoxy (Polymer Developments).

The West System ADR310 resin, which contains a white mineral filler, was used in the Sydney 2000 Olympic Games project.

The West System Z105 epoxy does not contain any filler and has a different appearance than the other epoxies, being more ‘glassy’, with a brittle appearance after failure.

The East 221 epoxy has demonstrated good performance in structures for periods of over 15 years.

All suitable epoxies have relatively low viscosity, which is important for ensuring the epoxy flows readily around the deformed or threaded steel bars, thereby providing complete encasement.

Use of epoxy that has a thick consistency can result in poor adhesion and some pull-out failures.

Test results have shown a significant difference in strength between the types of epoxy.

East 221 and Araldite 2005 have been found to give good results. The West System Z105 epoxy showed lower average strength than the others, but the difference was much less in high moisture content wood.

Typically, epoxy adhesives without mineral filler have better strength and stiffness than products with mineral filler because the filler disrupts the bonds linking the polymer molecules.

Geometry

The complex mechanics of load transfer from timber through an epoxy grout into a steel bar are not fully understood but experimental testing has highlighted significant geometrical factors that need to be considered in design.

Bar diameter

Bar diameters from 10 to 24 mm have been tested successfully. Any bar diameters in this range can be used, but best structural performance is usually achieved using a larger number of small bars, rather than a few large bars (this applies to most types of connections in timber structures).

Experimental testing has shown that the pull-out strength of a single bar is, among other factors, a function of the diameter of the bar.

Embedment length

Test results have shown that the failure load is almost directly proportional to the embedment length.

Tests of embedment length indicate a threshold embedment length of about 20 bar diameters, beyond which there is little increase in strength. This may reflect timber variability or defect location because it is expected that, with sufficient embedment length and appropriate edge distances, bar yielding can be achieved.

For design, longer embedment lengths are expected to provide additional safety against pull-out failure at little extra cost.

Edge distance

The average pull-out strength decreases with decreasing edge distance.

The effect of edge distance follows an empirical relationship whereby the strength is proportional to $r_e^{0.5}$ where r_e is the ratio of the edge distance to the bar diameter.

The edge distance, measured from the edge of the wood to the centre of the bar, should not be less than 1.5 bar diameters. This is the absolute minimum in the absence of any shear force.

In a typical situation, there will be some shear force, in which case the bolt hole edge distances from figure 4.1 in the New Zealand Timber Structures Standard NZS 3603 may be used.

Where high shear forces occur it is prudent to provide for joint shear transfer by means other that the epoxy grouted rods.

Some experts suggest a minimum edge distance of 2.5 bar diameters for axial loaded joints.

Hole diameter

Pull-out strength tends to increase as the hole diameter increases.

Larger holes result in more epoxy in the connection and a lower average shear stress at the wood/epoxy interface.

The preferred hole diameter is 1.25 bar diameters. Larger holes provide more tolerance but should not be greater than 1.50 bar diameters.

A disadvantage of large holes is that they reduce the net area of the wood at the end of the embedment, where brittle fractures can occur.

An assessment should be made of timber tension stress at bar termination points to avoid stress concentrations leading to premature failure.

Multiple bars

If several bars are used, they should have centre-to-centre spacing of at least 2 bar diameters. No more than three bars should be placed closely spaced in a single row.

Bars spaced closer than 75 mm to each other should have different embedment lengths so that the ends of the bars are staggered by at least 75 mm to reduce stress concentrations at the end of the embedment.

For connections using several layers of bars, it is recommended that the bars have staggered embedment lengths to reduce the potential for splitting along the line of the bar and to reduce stress concentrations where the bars end.

Diagram 1 shows alternative layouts of multiple bars in a glulam beam. Early designs used the arrangement shown at the bottom of the diagram, on the basis that the most highly stressed bar should have the greatest embedment length. However, joint tests have shown a splitting failure, as illustrated.

This failure may be caused by the extreme bar developing a load that exceeded the tension capacity of the outer laminations into which the bar was anchored.

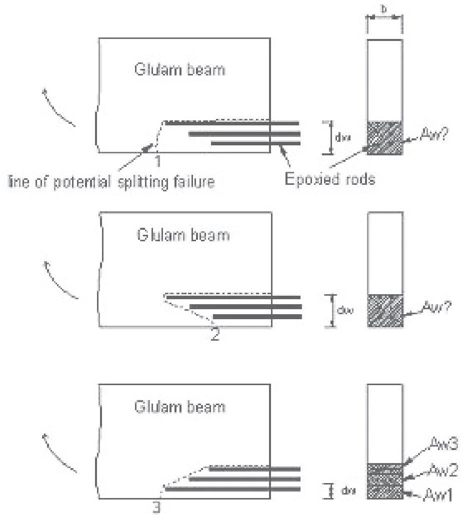
An alternative arrangement is shown at the top of diagram 1, which is considered to offer a lower possibility of failure if a shrinkage crack occurs at the end of the beam in one of the positions shown. Some experts, however, argue that the bottom layout in diagram 1 is preferred.

In either case, it is suggested that the wood stress be checked using the area A_w as shown and the force in all three pairs of bars.

An additional check is to ensure that the contributing cross-section area of wood into which each individual bar is anchored does not exceed the allowable tensile stress of the wood material.

Selecting a bar size that ensures bar yielding before the timber tension stress is exceeded minimises the risk of brittle failure.

Diagram 1: Multiple bars: alternative layouts in a glulam beam



Consideration needs to be made of the timber failure path for multiple rods under flexure.

In diagram 1, for the rods on the top, is failure line 1 likely, or is a progressive failure along line 2 the most likely, with the rods with the least embedment taking the most load?

Some experts believe that the bottom layout is a better solution, using the A_w associated with each level of rods.

llustration: Courtesy D Reid.

Testing by Korin et al has shown that a group of bars does not have the same strength as several separate bars of the same size and geometry.

For closely spaced bars, a group reduction factor k_g is proposed as shown in table 1.

Table 1: Strength reduction factor k_g for groups of bars

Number of bars in group	Reduction factor k_g
1, 2	1.0
3, 4	0.9
5, 6	0.8

Transverse reinforcement

The effect of possible shrinkage cracking should be considered if deep members are bolted to rigid supports, such as steel knee joints.

The best precaution for avoiding shrinkage is to use wood at a moisture content below the expected equilibrium moisture content.

Potential splits due to shrinkage or shear stresses can be prevented by careful placing of transverse reinforcing. One method is to provide threaded steel bars epoxied into drilled holes, crossing potential split lines, approximately 50 mm from the end of the member, as shown in diagram 2.

It is suggested that the reinforcing bars should have a cross-section area at least 1/25 of the area of the main bars.

Another form of transverse reinforcement is to use self-tapping screws near the epoxied rods to enhance their pull-out strength.

Diagram 2: Bar arrangement to prevent splitting due to shrinkage or shear stresses

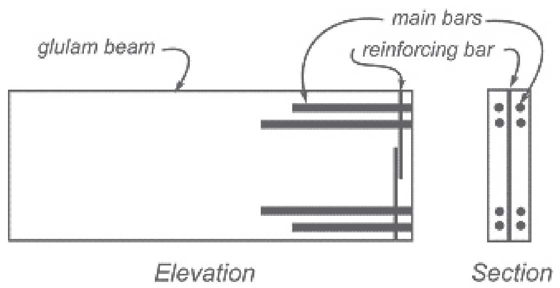


Illustration: Courtesy Timber Design Guide, 2007

DESIGN

An iterative process is necessary in design, whereby timber member sizes are arrived at, a geometric arrangement of rods is assumed and checked, and revisions are made to the geometric arrangement and member sizes as necessary.

The design equations proposed here are in the same form as that presented in the New Zealand Timber Structures Standard NZS 3603: 1993.

An epoxy bonded steel connection loaded in axial tension shall satisfy:

$$N^* \leq \phi\, Q_n \qquad [1]$$

where: N^* = design axial force produced by the strength limit state design loads (kN)
$\phi\, Q_n$ = design strength of the connection (kN); the least of the three values below
ϕ = strength reduction factor (from NZS 3603: 1993 or NZS 3404: 1992)
Q_n = nominal axial strength of the connection (kN).

The design axial strength considering steel yielding is:

$$(\phi\, Q_n)\text{steel} = \phi\text{steel}\ n\ A_s\ f_y \qquad [2]$$

where: ϕsteel = 0.8 (NZS 3404 for steel members in tension)
n = number of steel bars
A_s = cross-section area of each steel bar (mm^2/1,000) (for deformed reinforcing bars use the nominal bar area, for threaded steel rods use the tensile stress area at the thread)
f_y = characteristic yield strength of steel (MPa).

The design strength considering wood fracture at the end of the embedded bar is:

$$(\phi\, Q_n)\text{wood} = \phi\text{conn}\ k_1\ A_w\ f_t \qquad [3]$$

where: ϕconn = 0.7 (NZS 3603: 1992 for connections other than nails or bolts)
k_1 = duration of load factor (NZS 3603: 1993, table 2.4)
A_w = net area of wood cross section, excluding drilled holes (mm^2/1,000)
f_t = characteristic tensile strength (MPa)
(NZS 3603, Amendment 4 for MSG8 radiata pine, f_t = 6.0 MPa).

Note that the term A_w in equation 3 applies to members with axial tension loads, based on several tensile test specimens that failed with a brittle wood fracture at the end of the embedded steel bar.

Consideration needs to be given to the way in which the timber can fail. When applying equation 3 to flexural members, A_w should be taken as the notional net area of wood between the end of the longest bar and the tension face of the beam as shown in diagram 1.

The design axial capacity in tension considering bar pull out is:

$$(\phi\ Q_n)_{pull\ out} = \phi_{conn}\ k_1\ n\ k_g\ Q_k \qquad [4]$$

where: k_g = bar group reduction factor
(1.0 for 2 bars; 0.9 for 3 or 4 bars; 0.8 for 5 or 6 bars).

The characteristic strength Q_k is given by:

$$Q_k = 6.73\ k_b\ k_e\ k_m\ (l/d)^{0.86}\ (d/20)^{1.62}\ (h/d)^{0.5}\ (e/d)^{0.5} \qquad [5].$$

where:

- d = steel bar diameter ($12 \leq d \leq 24$ mm)
- l = embedment length ($5d \leq l \leq 20d$)
- h = hole diameter ($1.15d \leq h \leq 1.4d$)
- e = edge distance from centre of bar ($e \geq 2.5d$ recommended)
- k_b = bar type factor (threaded: 1.0; deformed: 0.8)
- k_e = epoxy factor (West System: 1.0; K-80: 1.0; Araldite 2005: 1.2)
- k_m = moisture factor (moisture content $<$ 15 percent: 1.0; 15 to 22 percent: 0.8).

SEISMIC DESIGN

Where the epoxied connection forms part of the seismic load resisting system, and the structure is designed as a nominally ductile structure (with displacement ductility factor $\mu > 1.25$ as defined in NZS 1170.5), the steel yielding strength from equation 2 multiplied by an appropriate over-strength factor must be less than the wood fracture or bar pull-out strength given by equations 3 and 4.

If ductility is required, the design philosophy must be clearly thought out. Wood members and epoxy adhesives usually exhibit sudden brittle fractures when loaded to failure in tension.

In compression, wood fails in a more ductile manner but this is not reversible. If ductility is required in glue laminated connections, it should preferably be provided in yielding of steel brackets located outside the member.

It is essential to use a capacity design approach to ensure that the selected weakest element yields before the capacity of the timber or epoxied connections are reached.