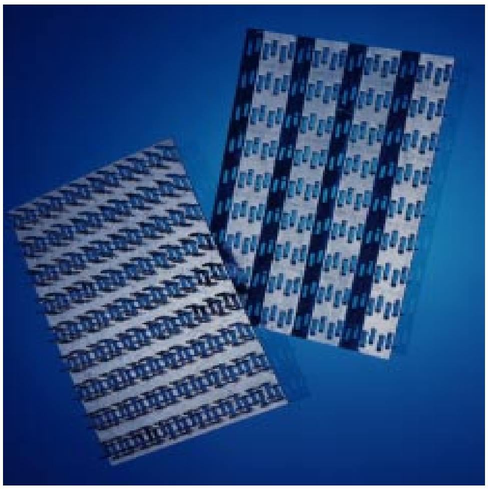
Gang-Nail Connectors - How They Work

A Gang-Nail connector is a steel plate with a collection of spikes or nails projecting from one face (See diagram). The spikes, or teeth, are formed by punching slots in steel but leaving one end of the 'plug' connected to the sheet. The teeth are then formed so they project at right angles to the plate. During this process the teeth are shaped to produce a rigid projection. When the teeth of a connector plate are pressed into timber laid end-to-end, the plate 'welds' them together by forming a Gang-Nail joint. Connectors are always used in pairs with identical plates pressed into both faces of the joint.



The concept is simple but the design of efficient Gang-Nail connectors requires careful balancing of tooth shape and density, connector plate thickness and ductility. An ongoing commitment to research and development ensures that MiTek's licensed truss fabricators have the most efficient truss system at their disposal.

Performance criteria for Gang-Nail connectors

It is not economical to have a single connector that gives optimum performance under all loading conditions, for all of Australia's wide range of commercial timbers. MiTek Australia Ltd. has developed a complementary range of connector plates of varying plate thickness (gauge), tooth layout and tooth profile. These are:

- GQ – 20 gauge (1.0 mm thick) galvanised steel. General purpose connector. Many short, sharp teeth 128 teeth in a 100 mm x 100 mm area.
- GE - 18 gauge (1.2mm thick) galvanized steel. Similar to GQ. For use when additional steel strength is required.
- G8S – 18 gauge (1.2 mm thick) stainless steel. This connector is only used when the environment is highly corrosive. 70 teeth in a 100 mm x 100 mm area.
- GS – 16 gauge (1.6 mm thick) galvanised steel. Heavy duty connector. 144 teeth in a 100 mm x 190 mm area.

Engineering Data

Gang-Nail connector properties have been established in accordance with Australian Standard AS1649 'Timber - methods of test for mechanical fasteners and connectors - Basic working loads and characteristic strengths. As well as testing new plate designs, MiTek Australia Ltd. conducts regular tests on their existing connector range and monitors the long term behaviour of joints subjected to constant loading. The CSIRO Division of Forest Products and the NSW Forestry Commission Division of Wood Technology have also done considerable research work on toothed metal plate connectors.

Full scale truss testing programs have been carried out at the Universities of Western Australia and Adelaide, Australian National University and the Cyclone Testing Station at Capricornia Institute of Advanced Education.

Connector properties can be divided into two parts:

- properties dependent on connector plate strength, and
- properties dependent on the characteristics of the timber and the teeth.

Table 1.1 Characteristic Capacity for Steel, Q_s

Connector Plate Strength in N/mm for a pair of plates.				
TYPE OF STRESS	**GQ**	**GE**	**GS**	**G8S**
Longitudinal Tension	263	387	578	535
Lateral Tension	187	226	272	272
Longitudinal Shear	197	297	408	280
Lateral Shear	178	215	323	246

Notes:
1. The longitudinal axis is parallel with the slots.
2. Values in tables do not include the capacity factor.

Table 1.2 Characteristic Capacity for Tooth Q_k, (N/effective tooth)

JOINT GROUP	GQ TOOTH	GE TOOTH	GS TOOTH	G8S TOOTH
J2	501	501	560	590
J3	442	486	560	590
J4	295	295	413	430
JD2	531	678	767	737
JD3	531	678	767	737
JD4	383	383	442	430
JD5	339	339	392	368
JD6	295	295	339	NA

Notes:
1. The longitudinal axis is parallel with the slots.
2. Values in tables do not include the capacity factor .

The characteristic load capacities are modified to determine the design capacities. These modification factors allow for tooth strength variation due to:

- capacity factor
- duration of load
- angle between load direction and plate axis
- angle between load direction and grain
- whether plates are pressed into the timber or rolled in.

Capacity Factor

Values of the capacity factor, f, are listed in Table 2.6 of AS1720.1.

Duration of load

Duration of load factor k_1 is specified in Table 2.7 of AS 1720.1.

Angle between load direction and plate axis.

Where the connector plate is loaded at right angles to its main axis, the design capacities for the tooth should be reduced to 75% for GQ, GS and GE plates, and 60% for G8S plates. For intermediate orientations the reduction factor can be calculated using Hankinson's formula, or the simpler equation:

$$F = 1 - \frac{\theta}{360} \quad \text{For GQ, GS \& GE}$$

$$F = 1 - \frac{\theta}{225} \quad \text{For G8S}$$

Angle between load direction and grain

Where the timber is loaded at right angles to the direction of the grain, the design capacities for the tooth should be reduced to 80% in addition to any modification due to load direction/plate axis. For intermediate orientations, the reduction factor can be calculated using Hankinson's formula:

$$Nt = \frac{Pt \times Qt}{PtSin^2\theta + QtCos^2\theta}$$

Nt = Design Capacity at angle q to grain.
Pt = Design Capacity parallel to grain.
Qt = Design Capacity perpendicular to grain.

Gang-Nail Joints

The basis of joint design is to ensure that there are enough teeth in each member meeting at the joint to resist all member forces and that there is enough plate area to prevent the steel of the connector failing in tension or shear.

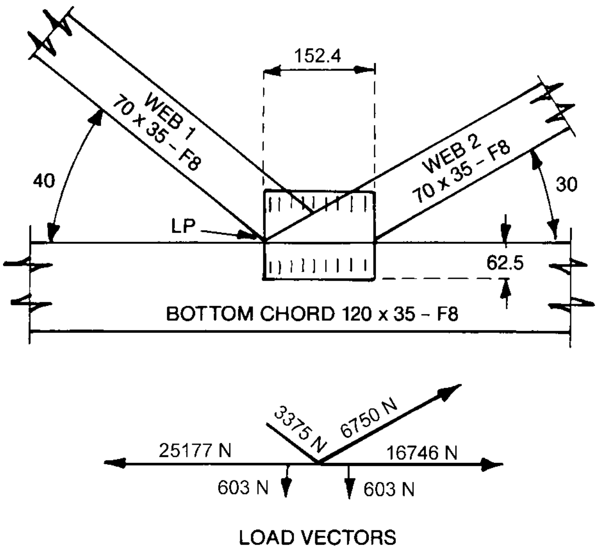
When determining the number of teeth that are required to connect any member to the joint, there are parts of the web or chord where the teeth are considered to be ineffective – a 6 mm wide strip along the edges and a 12 mm long strip across the end. Allowance is also made for the connector to be out-of-position by 6 mm in any direction.

Gang-Nail Joint Design Example

Consider the joint as in Figure 1, assuming the truss is manufactured from Pinus Radiata. Member actions are, as indicated in Figure 1, adjacent to the joint detail. Note that the cutting details employ single cuts, with the 'location point', denoted LP, common to one face of each intersecting member. We shall assume DL and LL is the only load case for this exercise.

Timber is Radiata, i.e. JD4. Consider GQ150125 plate. Note the connector plate axis is at 90^0 to the bottom chord, and the actual connector plate size is 152.4 x 125.

Figure 1 - Sample Joint



Steel Check

By inspection, connector plate shear horizontally above bottom chord will be most critical.

Design Capacity

= ϕ x Shear Length x Q_s

= 0.9 x 152.4 x 178

= 24414 N

Applied Shear

= 25177 - 16746

= 8431 N therefore OK

Figure 2 - Web 2 effective connector plate area.

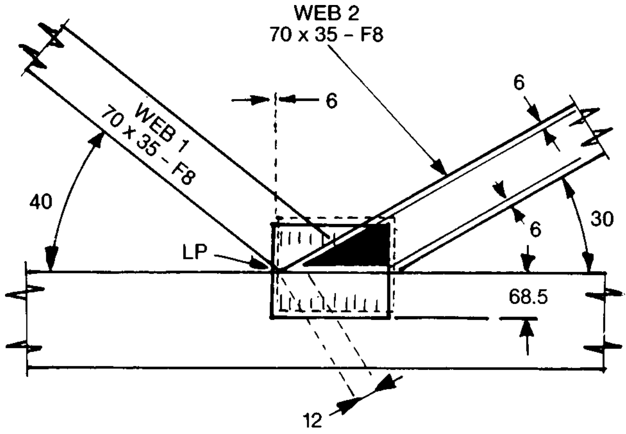


Figure 2 shows the effective connector plate contact area to Web 2. Note the allowance for 6 mm ineffective edge distance and 12 mm ineffective end distance. The connector plate has also been shifted to the limits of connector plate tolerance as discussed previously.

Using a transparent template, the number of effective nails in the shaded area has been counted as 50/side.

$$F = 1 - \frac{60}{360} = 0.83$$

Design Capacity

= 2 x n x ϕ x k_1 x F x Q_k

= 2 x 50 x 0.85 x 0.77 x 0.83 x 383

= 20805 > 6750 N therefore OK.

Figure 3 - Web 1

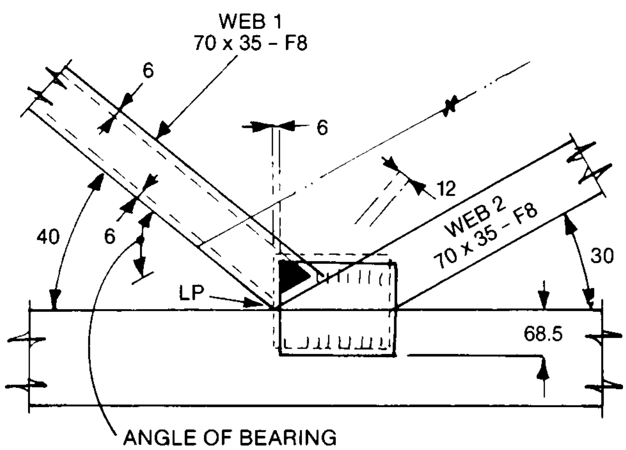


Figure 3 shows the effective plate contact on Web 1. We find 14 teeth to be effective.

$$F = 1 - \frac{50}{360} = 0.86$$

Design Capacity
= 2 x n x f x k_1 x F x Q_k
= 2 x 14 x 0.85 x 0.77 x 0.86 x 383
= 6036 > 3375

Note that the member is in compression, and the angle of bearing as defined in Figure 3 is greater than 60. The member will, therefore, be “self locking”, and nominal plate contact will suffice.

Figure 4 - Bottom Chord

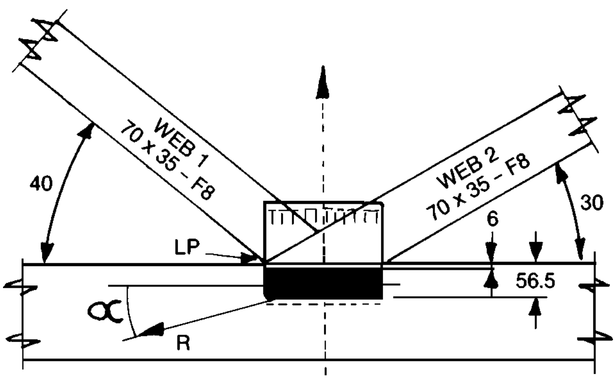


PLATE AXIS

The bottom chord plate area is subject to both axial and shear forces. The resultant force:

$$R = \sqrt{8431^2 + 1206^2}$$

R = 8517 N

$$\alpha = \arctan\left(\frac{1206}{8431}\right)$$

$$\alpha = 8.14^{\circ}$$

Angle between load and plate axis.

$$\varphi = 90 - \alpha = 82^{\circ}$$

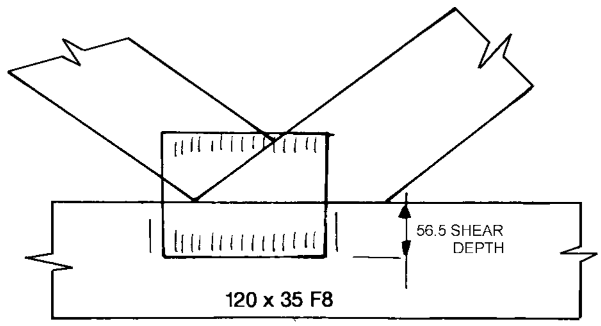
$$F = 1 - \frac{82}{360} = 0.77$$

Using the transparent template, we find 96 teeth to be effective.

Design Capacity
= 2 x n x f x k_1 x F x Q_k
= 2 x 96 x 0.85 x 0.77 x 0.77 x 383
= 37060 > 8517 therefore OK.

Adopt GQ150125 Standard Location.

Figure 5



The bottom chord is subject to shear forces perpendicular to the grain. Generally these are not critical, but should be checked where heavy loads are applied to bottom chords such as on girder trusses or where low density timber is used.

Values for shear at joint details are as given in AS 1720.

Shear at joint = 1206N
Design Capacity
= f k_1 f_{sj} A_s
= 0.8 x 0.77 x 5 x 56.5 x 35 x 2
= 12181N
> 1206 N is therefore OK

Self-locking joints

Some joints are self-locking and do not rely on the connector to resist all the forces in the joint. For example, where the angle between a chord and an intersecting web exceeds 60 degrees and the web is in compression, the web locks against the chord and the connector teeth and are only lightly loaded. All joints need to be designed to resist any tension loads that occur in service or during manufacture, handling and installation.

Splice Joints

Truss bottom chords will nearly always require end-to-end splicing of relatively short pieces of timber to achieve the desired chord length.
When these splices are acting in compression, the forces are transferred by direct end bearing between the pieces and only a nominal connector would be required to hold the pieces in place and resist the stresses of manufacture, handling and installation. However, the normal design procedures apply when the splice is subject to tension loads.

The width of splice plates should be at least 20 mm less than the timber width. This minimises any tendency for the edges of the timber to split around the connector. In trusses made from green timber, keeping the plate away from the edge of the member will also avoid ‘bumps’ occurring in the ceiling or roof line as the connector can locally restrain the timber from its natural tendency to shrink as it dries.