W M G Burdzik

Some round timber pole connections

Synopsis

The paper looks at alternative round timber pole connections that make mono-planar structures and trusses a possibility. Three different systems are discussed and where test results are available, these are compared with theoretical values. It is shown how the performance of a connection can be improved by changing the failure mode of the connector and the timber.

Samevatting

Die verhandeling beskou alternatiewe houtpaalverbindings wat enkelvlakkige strukture en kappe moontlik maak. Drie verskillende stelsels word bespreek en waar toetsresultate beskikbaar is, word hulle met teoretiese waardes vergelyk. Daar word gewys hoe die dravermoë van a verbinding verbeter kan word deur die swigtingsmeganisme van die verbinder en die hout te verander.

Walter Burdzik *graduated from the University of Pretoria with BSc Mechanical and Civil Engineering degrees in 1972. After a number of years working for consulting engineers, he returned to the Department of Civil Engineering at the university, where he became involved in timber research. He obtained his PhD in timber engineering in 1989. He serves on all the SABS and Institute for Timber Construction committees that deal with timber and timber-related matters and has been involved in the drafting of the timber design codes.*

Introduction

For many years structures constructed of round timber poles have been connected by means of threaded rods. These have worked reasonably well for small span structures. The poles are generally not in one plane and drilling through the poles and threading the rod through the hole affects the connection. Often, up to five poles are connected in this fashion.

Although many of the structures, especially thatched roofs, have shown large deflections owing to permanent deformation at the joints, the roofing material has been able to disguise any possible distress of the structure and the assumption seems to have been made that the joints are working well. Furthermore, the assumption is made that the threaded rod connections will work for much bigger structures and any structural form as well. The fact that failures of large thatched structures have not been reported could mean that no failures have occurred, the connections have great residual strength, the live load never occurs on the structure or the thatching works as a shell structure.

A very dangerous practice has also arisen in the building industry. Clients assume that contractors know what they are doing. A lack of design methods, especially for connections, makes it very difficult for engineers to design timber pole structures efficiently or to check them for structural integrity. Often the structural analysis would lead one to believe that the structure is unsafe, but the structure shows no distress. Eyes are closed and forms are signed.

As traditional pole structures have the poles placed next to one another at the connections, moments occur in the poles owing to the eccentricity of the connection. Eccentricity in connections is often ignored, especially where the eccentricity is small. This is very seldom the case with pole structures, where eccentricity is large. Often the connection undergoes large displacements and twisting. If connections for mono-planar round pole structures were easy to construct and design methods were available, the task of the engineer would be made very much easier. The engineer would then have the freedom to break from current practice and to design structures that are sound and aesthetically more pleasing. A well-designed sound structure will inherently look better than a structure that is held in place by fate.

It is possible to construct mono-planar structures or trusses where properly designed bolted connections or patented pole connectors are used. Patented connections include Gumbou® 190 connectors and glued-in threaded rods with steel end plates. Bolted or nailed connections with steel plates where the bolt or nail is used to its full potential can lead to very efficient connections. To design these connections, a better understanding of the failure mechanism of the connectors is required. As Eurocode 5 (1995) describes this failure mechanism, it was used as a basis for determining the expected strength of these mono-planar connections.

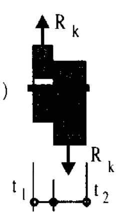
A limited number of Gumbou® connectors were tested in tension parallel to the grain as well as perpendicular to the grain to ascertain the failure mechanisms and to see how well the Eurocode 5 (1995) formulae would predict the strength. The glued-in rod strength values are well known and different configurations were tested to ascertain the best perpendicular-to-grain strength. A theoretical method for bolted joints with steel plates will be given. I am of the opinion that the method described will lead to much stronger, more rigid connections.

Theoretical assumptions

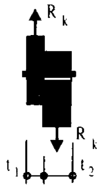
Whale (1991), Ehlbeck (1993) and others have been involved in quantifying the design formulae and test methods of dowel-type connections (nails, bolts, dowels, screws and staples) for inclusion in Eurocode 5 (1995). The design formulae are yield theory equations by Johansen (1949).

The characteristic load carrying capacity, R_k, for doweled connections in single shear is given as follows in Eurocode 5 (1995):

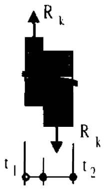
$$R_k = f_{h,1} t_1 d \qquad \text{(a)}$$



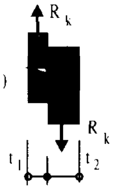
$$R_k = f_{h,1} t_2 d\beta$$

(b) 

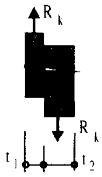
$$R_k = \frac{f_{h,1} t_1 d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left(1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right) + \beta^3\left(\frac{t_2}{t_1}\right)^2} - \beta\left(1 + \frac{t_2}{t_1}\right)\right]$$

(c) 

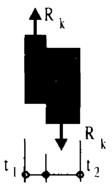
$$R_k = \frac{f_{h,1} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_y}{f_{h,1} d t_1^2}} - \beta\right]$$

(d) 

$$R_k = \frac{f_{h,1} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_y}{f_{h,1} d t_1^2}} - \beta\right]$$

(e) 

$$R_k = \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2 M_y f_{h,1} d}$$

(f) 

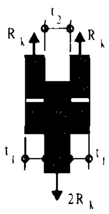
where:

R_k	=	characteristic resistance of the connection, in N
t_1 and t_2	=	thickness of connected material, in mm
$f_{h,1}$	=	characteristic embedment or ultimate bearing stress in t_1, in MPa
$f_{h,2}$	=	characteristic embedment or ultimate bearing stress in t_2, in MPa
β	=	$\frac{f_{h,2}}{f_{h,1}}$
d	=	fastener diameter, in mm
M_y	=	plastic resistance moment of the metal connector, in N.mm
f_y	=	fastener yield stress, in MPa

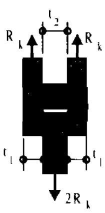
As can be seen from the failure mechanism, equations (a), (b) and (c) relate to wood failure and (d), (e) and (f) to connector failure. The last three are, therefore, the more desirable failure mechanisms for connector failure as they have large reserve strength and are able to dissipate a large amount of energy. As long as shear failure or splitting of the timber can be avoided, the first three failure mechanisms will give high strengths. Although they are not as ductile as failure mechanism (f), they will still show an acceptable degree of ductility.

The characteristic load carrying capacity for doweled connections in double shear is given as follows in Eurocode 5 (1995) under Joint Design:

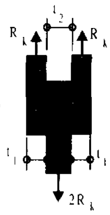
$$R_k = f_{h,1} t_1 d$$

(g) 

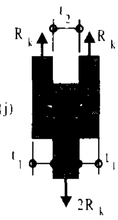
$$R_k = 0{,}5 f_{h,1} t_1 d$$

(h) 

$$R_k = \frac{f_{h,1} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_y}{f_{h,1} d t_2^2}} - \beta\right]$$

(i) 

$$R_k = \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2 M_y f_{h,1} d}$$

(j) 

Once again, the equations (g) and (h) have to do with wood failure and (i) and (j) with connector failure. These formulae are true for single bolted connections where the required end and edge spacing can be achieved. When more than one connector is used to transfer perpendicular-to-grain forces, or where end and edge distances cannot be maintained, an additional factor for perpendicular-to-grain tension comes into play.

The formula for multiple connectors (cf Fig 1) is given in SABS 0163-1:1994 in the form:

$$T_r = \phi \frac{f_{tp}}{\eta \gamma_{m1} \gamma_{m6}} . A_{eff} \qquad (1)$$

where:

T_r	=	perpendicular-to-grain resistance, in N	
ϕ	=	capacity reduction factor	
γ_{m1}	=	material factor for duration of load	
f_{tp}	=	characteristic perpendicular-to-grain stress, in MPa	
η	=	connector depth ratio $= 1 - 3\left(\frac{a_r}{h}\right)^2 + 2\left(\frac{a_r}{h}\right)^3$	(2)
a_r	=	distance from connector to stressed edge, in mm	
h	=	height or depth of timber member (nominal diameter of round pole), in mm	
γ_{m6}	=	factor for area stressed by connectors $\left(\frac{A_{eff}}{A_0}\right)^{0,2}$	(3)
A_0	=	10^6, in mm²	
A_{eff}	=	effective area $= l_{eff} b$, in mm²	(4)
l_{eff}	=	effective stressed length, in mm $= \sqrt{l_r^2 + (ch)^2}$	(5)
		Use $l_{eff}/2$ when end distance is less than depth of timber member	
b	=	penetration depth of the connector, $<15d$.	
l_r	=	distance between outer connectors of connector group, in mm	
c	=	$\frac{4}{3}\sqrt{\frac{a_r}{h}\left(1 - \frac{a_r}{h}\right)^3}$	(6)
d	=	nominal diameter of the fastener, in mm	

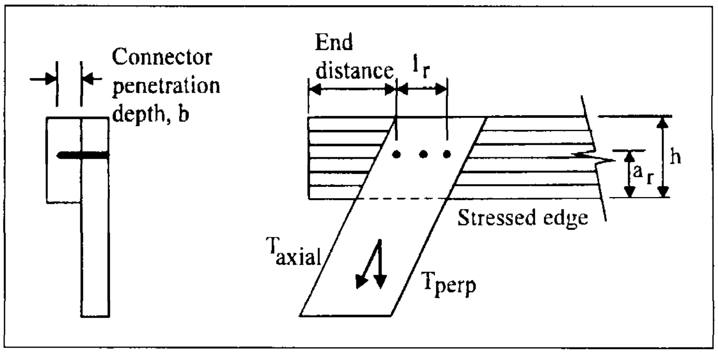


Fig 1: Multiple connector joints loaded perpendicular to the grain

Multiple-fastener joints

If there are more than six bolts or dowels in line, Eurocode 5 (1995) has a provision that stipulates that the load-carrying capacity of the extra fasteners should be reduced by a third, ie for n fasteners the effective number,

n_{ef}, is:

$$n_{ef} = 6 + 2(n-6)/3 \tag{7}$$

No such reduction in load-carrying capacity is required for nailed joints.

Embedment or ultimate bearing stress

The following embedment or ultimate bearing stresses are proposed in Eurocode 5 (1995) in lieu of specific test data:

Nailed joints – Diameter of nail less than or equal to 6 mm

$$f_h = 0{,}09\,\rho\, d^{-0{,}36}\ \text{MPa} \tag{8}$$

for all timber without pre-drilled holes and

$$f_h = 0{,}13\,\rho d^{-0{,}36}\ \text{MPa for pre-drilled holes} \tag{9}$$

ρ = density of the timber, in kg/m^3
d = nominal diameter of the connector, in mm

Bolted joints – Bolts, dowels or nails with a diameter greater than 6 mm

For loading parallel to the grain:

$$F_{h,0} = 0{,}082\,(1 - 0{,}01\,d)\,\rho \tag{10}$$

For loading perpendicular to the grain:

$$F_{h,90} = 0{,}036\,(1 - 0{,}01\,d)\,\rho \tag{11}$$

ρ = density of the timber, in kg/m^3
d = the diameter of the connector

When the load is at an angle a to the grain of the timber, the characteristic embedment or ultimate bearing stress f_h, α should be calculated as follows:

$$f_h, \alpha = \frac{f_{h,0}}{2{,}3\sin^2\alpha + \cos^2\alpha} \tag{12}$$

These equations apply to bolts and dowels with a minimum yield stress greater than or equal to 240 MPa.

Gumbou® connectors

Gumbou® connectors are connectors made specifically for round pole construction. They consist of 400 mm by 100 mm by 2,5 mm thick plates that have been kinked to fit against the poles, as shown in the accompanying photograph. They are connected to the poles by means of eight, 5,5 mm diameter roofing screws, 65 mm long. The screws are driven into the pole at an angle so that it becomes very difficult to remove the connector even when a lateral load is applied. Connectors are bolted together by means of a 12 mm bolt and should always be used in pairs.

Ten of these connections were tested in loading parallel to the grain and ten in loading perpendicular to the grain on untreated Saligna poles. The poles had a mean density of 680 kg/m^3, with a fifth percentile density of 580 kg/m^3. To prevent splitting of the poles, pilot holes were drilled for all the roofing screws.

Theoretical strength of connector

The bearing stress on the steel must be calculated in accordance with SABS 0162:1993 and is given as three times the ultimate stress of 450 MPa. For the nails the following values will apply:

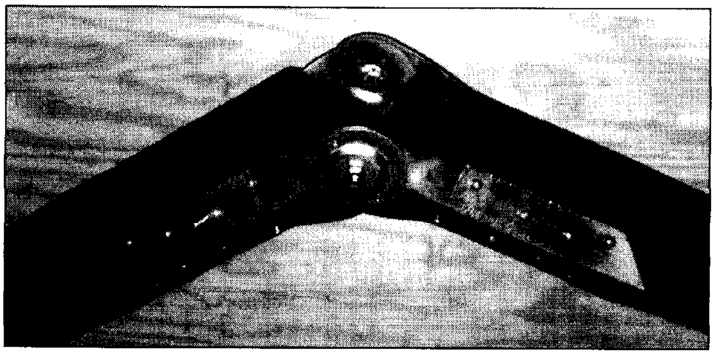
$f_{h,1} = 3 \times 450 = 1\,350$ MPa
$f_{h,2} = 0{,}13 \times 580 \times 5{,}5^{-0{,}36} = 40{,}8$ MPa
$t_1 = 2{,}5$ mm
$t_2 = 60$ mm
$\beta = 40{,}8/1\,350 = 3{,}03 \times 10^{-2}$

$$M_y = f_y d^3/6 = \frac{450 \times 5{,}5^3}{6} = 12\,500\ \text{N.mm}$$

The resistance of a nail loaded parallel to the grain can then be calculated in accordance with the Eurocode 5 (1995) equations for nails in single shear. The resistance is then the minimum of the following:

(a) = 18,5 kN
(b) = 13,5 kN
(c) = 5,7 kN
(d) = 3,0 kN
(e) = 48,2 kN
(f) = 3,3 kN

The failure mechanism (d) will govern. As there are eight nails per side, the ultimate load that can be transferred by the nails will be 24 kN. The load that can be transferred by the 12 mm bolt in single shear can be calculated in accordance with SABS 0162 and is given as 20,3 kN. As the capacity reduction factors, ϕ, for steel and timber are the same for connectors, the shear failure of the bolt will govern. In order to study the nail failure rather than the 12 mm bolt failure, the bolt was removed and a large diameter rod used in its place. The yield forces were thus expected to be equal to the predicted nail yielding load, ie 24 kN.



Gumbou® connectors connecting two poles at an angle (note the angle of the nail)

For loading perpendicular to the grain the position of the nails relative to the position of loading must be taken into account. Relating the forces to the nail group centre, the resultant force on the furthest nail owing to the load and moment owing to the load can be calculated (see Fig 2).

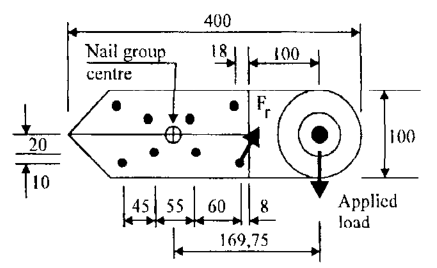


Fig 2: Dimensions of Gumbou® connector and distance to nail-group centre

The polar second moment of area for the nail group can be calculated using:

$I_p = Ar^2 = 34\,136$ A

where A = area of one nail.
The resultant force components on the critical nail are:

F_y owing to vertical force = applied load / 8 = 0,125 x applied load

$$F_y \text{ owing to the moment} = \frac{169{,}75 \times \text{applied load} \times 88{,}75}{34\,136} = 0{,}5115 \times \text{applied load}$$

$$F_x \text{ owing to the moment} = \frac{169{,}75 \times \text{applied load} \times 30}{34\,136} = 0{,}1729 \times \text{applied load}$$

The resultant force in the nail is then equal to:

F_r = 0,6596 x applied load
Applied load = 1,516 x F_r

The maximum applied load that will cause the nail to fail in the mode of mechanism (d) is then for a single-sided connection equal to:

Failure load F = 1,516 x 3,0 = 4,55 kN

or for a double sided Gumbou® connection:

Failure load F = 9,1 kN

The accompanying table gives a summary of the failure loads for the Gumbou® connector for loading parallel and perpendicular to the grain. Ten specimens each were tested for strength parallel to the grain and perpendicular to the grain. Figs 3 and 4 show the load deformation curves for

Test results for Gumbou® connectors, loaded parallel and perpendicular to the grain

Type of loading	*Theoretical yield strength* *kN*	*Highest initial yield strength* *kN*	*Lowest initial strength* *kN*	*Mean initial strength* *kN*	*Lowest ultimate strength* *kN*
Parallel	24,0	37,0	21,0	31,4	31,2
Perpendicular	9,1	9,7	6,5	8,6	8,2

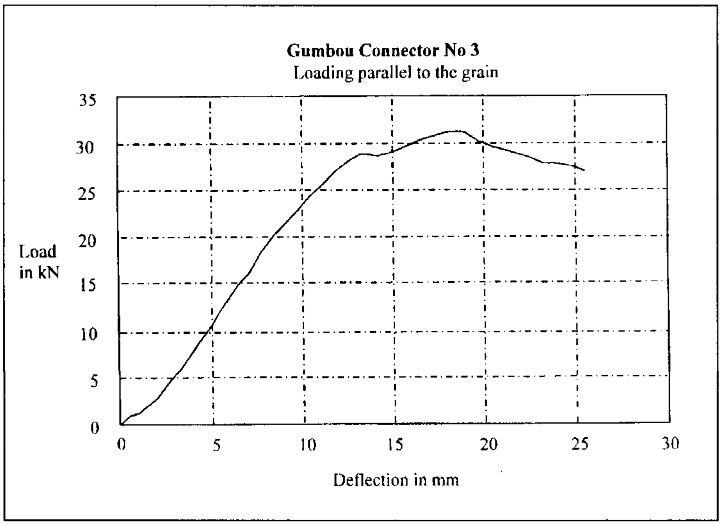


Fig 3: Typical load-deflection curve for a Gumbou® connector loaded parallel to the grain

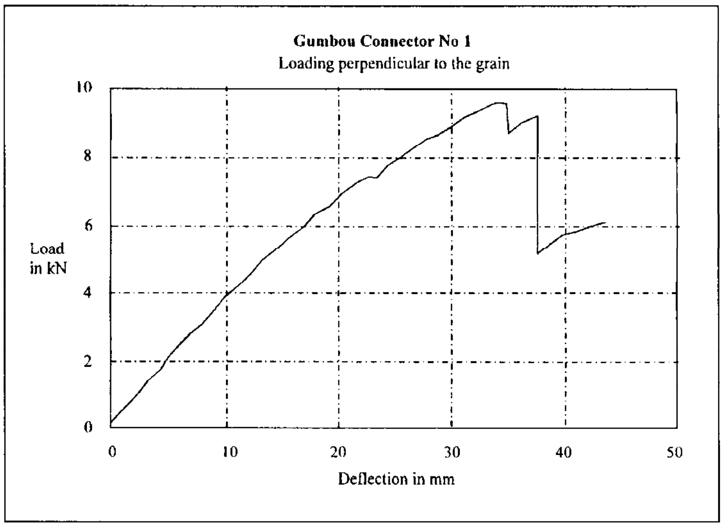


Fig 4: Typical load-deflection curve for Gumbou® connector loaded perpendicular to the grain

the weakest connection for loading parallel and perpendicular to the grain respectively. It is important to note that the loading parallel to the grain has a ductile failure mode and perpendicular to the grain a brittle failure mode. Perpendicular-to-the-grain loading in all cases led to splitting of the pole. In the case of the lowest strength for loading perpendicular to the grain, cracks were formed in the pole when the roofing screws were driven home.

The initial yield strength was defined as the strength at which the load-deflection curve ceased to be linear.

Discussion

The Gumbou® connectors work very well for loading parallel to the grain and the strength can be predicted fairly well by the Eurocode 5 (1995) formulae. The connection also shows great ductility owing to the failure of the connecting bolts rather than the roofing screws. Pre-drilling of the roofing screw holes is necessary to prevent splitting of the poles, as splitting will decrease the strength dramatically.

Loading perpendicular to the grain can also be predicted by the Eurocode 5 (1995) formulae, although splitting of the poles, either through the nailing process or owing to drying cracks or splits, will lead to lower initial yield strengths. In most cases failure is brittle and no residual strength can be expected, ie sudden collapse will occur.

Steel plate and bolted connections

The Eurocode 5 (1995) equations show that for doweled connections, ie nails, bolts, dowels, etc, the highest force will be transferred when the connector is rigid and the timber is crushed. If the end distance is large enough, this failure mode will have a certain amount of ductility. As the dowels are not rigid, the desired failure mechanism can be achieved only by making the dowel rigid relative to the timber.

Example: Load parallel to the grain

A 12 mm threaded rod is used to connect two 120 mm round timber poles, with characteristic density equal to 580 kg/m^3. For loading parallel to the grain, as shown in Fig 5, the threaded rod can be expected to have an initial yield strength of about 8 kN. This is due to the threaded rod yielding as in Eqn (f). If the rod were rigid and rotation of the rod, as shown in the sketch for Eqns (c), (d) and (e), could be prevented, Eqn (a) would govern and the expected strength of that connection could potentially be equal to 60,2 kN. This is far in excess of the single shear strength of the threaded rod, which is only about 20,3 kN. To realise the full potential of the threaded rod, one would require at least four shear planes. In order to achieve this, two plates must be slotted into the round pole as in Fig 6. The plates must be thick enough to carry the bearing load from the bolts and the applied tensile force. According to the *South African Steel Construction Handbook* (1992), the steel plates will need to be at least 3 mm thick to transfer the bearing load. The failure mechanism now changes from single shear Eqn (f) to double shear (j) or (g).

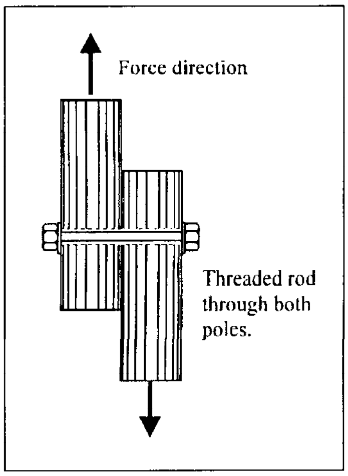


Fig 5: Two poles connected by means of a threaded rod: Transfer of load through bending of the threaded rod

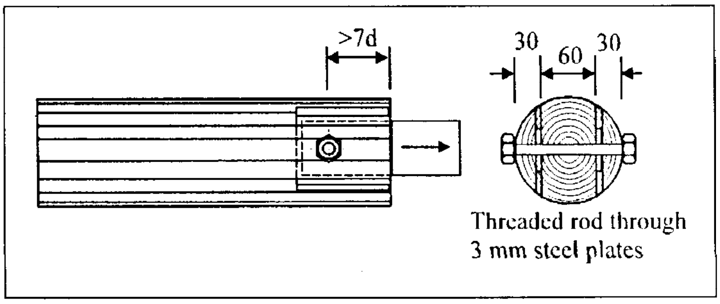


Fig 6: Transfer of load through plates, slotted into the timber pole

The equations for connectors in double shear may now be used with the bearing stress $f_{h,1}$ referring to the outer material, which is timber, and $f_{h,2}$ referring to the inner material, steel. With the timber having a density of 580 kg/m^3, the bearing stress of the timber, $f_{h,1}$, will be equal to 41,8 MPa and the bearing stress for the steel plate, $f_{h,2}$, 1 350 MPa. The yield moment, M_y, of the threaded rod will be equal to 75 600 N.mm.

$$\beta = \frac{f_{h,2}}{f_{h,1}} = 32{,}3$$

The Eurocode 5 (1995) equations for double shear will then lead to the following resistances:

(g)	4 x 15,05	=	60,2 kN
(h)	4 x 24,30	=	97,2 kN
(i)	4 x 107,1	=	428,4 kN
(j)	4 x 12,13	=	48,52 kN

Eqn (j) will still govern, but this is a vast improvement over the expected

8 kN for the same threaded rod in single shear. Not only does this connection not induce moments owing to the eccentricity, but it is also a much stiffer connection than the single shear connection.

Example: Loading perpendicular to the grain

Fig 7 shows a double shear connection, where the outer poles are loaded perpendicular to the grain. If the connector is a small-diameter threaded rod, failure of the connection will be governed by Eqn (j). Assuming that the poles have a fifth percentile density of 580 kg/m³, a maximum load of 6,8 kN per shear face, ie 13,6 kN in total, can be transferred between the central pole and the outer two poles. If the connection is changed to a connection between two poles, one loaded parallel to the grain and the other loaded perpendicular to the grain, plates can be slotted into the poles. A system similar to that used for the parallel-to-grain loading will result (see Fig 8). The load that can be transferred between the plates and the pole, loaded perpendicular to the grain, could be increased to a maximum failure load as given by Eqn (h), which would be 13,2 kN per shear face, or a total of 26,4 kN.

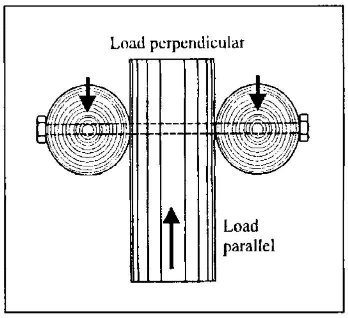


Fig 7: Typical three-pole connection with outer poles loaded perpendicular to the grain

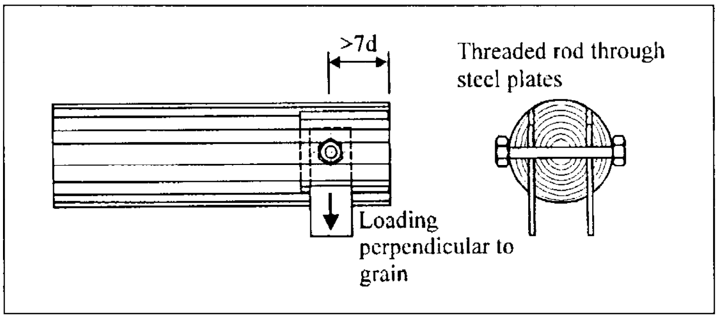


Fig 8: Plates slotted into pole so that a better distribution of load will occur

The Eurocode 5 (1995) equations for double shear would lead to the following:

t_1 = 30 mm
t_2 = 3 mm
$f_{h,1}$ = 0,036 (1 – 0,01 d) ρ = 18,4 MPa
$f_{h,2}$ = 1 350 MPa
$\beta = \dfrac{f_{h,2}}{f_{h,1}} = 73{,}37$

Plastic resistance moment of the threaded rod, M_y = 75 600 N.mm

The resistance force would be the least of:

(g) 4 x 6,62 = 26,5 kN
(h) 4 x 24,3 = 97,2 kN
(i) 4 x 81,14 = 324,6 kN
(j) 4 x 8,12 = 32,5 kN

If the end distance is greater than the thickness of the pole, the pole should not split. If the end distance is less than the thickness of the pole, ie seven times the diameter of the rod as in Fig 8, then the equations given in SABS 0163-1:1994, clause 13.2.2, should be used. The equations refer to groups of connectors loaded perpendicular to the grain.

a_r = 60 mm
h = 120 mm
c = 0,333
η = 0,5
l_{eff} = $c \,.\, h$ = 40 mm
A_{eff} = 4 800 mm²
$\gamma_{m6} = \left(\dfrac{A_{eff}}{A_o}\right) = 0{,}344$

If one looks at the modulus of elasticity of the poles, it is almost the same as that of a Grade 7 timber. Although the bending, compressive and tensile strengths of the poles are all higher than those of the sawn Grade 7 timber, the perpendicular-to-grain tensile strength will not be significantly higher. The higher parallel-to-grain strengths of the poles are due to the fact that a sawing process has not disturbed the structure of the tree. The effect of knots in undisturbed timber is much less than in the case of sawn timber. If one then assumes the timber to be a Grade 7, f_{tp} will have a value of 0,51 MPa.

The ultimate resistance of the connection without the capacity reduction factor is thus:

$$T_r = \phi . \frac{0{,}51}{0{,}5 \times 0{,}334} . 4\,800 \text{ N} = \phi \times 14{,}23 \text{ kN}$$

This is less than the value predicted by the Eurocode 5 (1995) formula, as the Eurocode 5 formula uses crushing of the fibres as a failure criterion and does not investigate tension perpendicular to grain. The strength value of a connection using the slotted-in plates is, however, still better than the value that can be obtained when the typical three-pole configuration is used (see Figs 6, 7 and 8).

Summary

By changing the failure mechanism of the connection it is possible to improve the strength of bolted connections in round timber poles. The most dramatic improvement can be obtained for loading parallel to the grain, where the strength can be improved by a factor of 6. For loading perpendicular to the grain, the improvement in strength is not that dramatic, and designers should consider connectors that transfer load in direct bearing and shear (see Fig 9). This is not always possible as loads are often at an angle to the grain. Designers should then consider an alternative type of connector for connections that must transfer large perpendicular-to-grain load components.

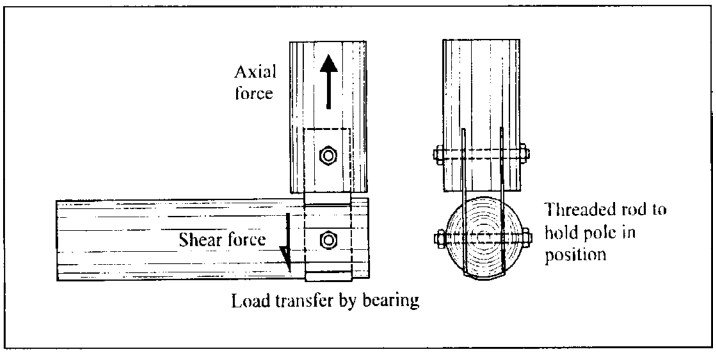


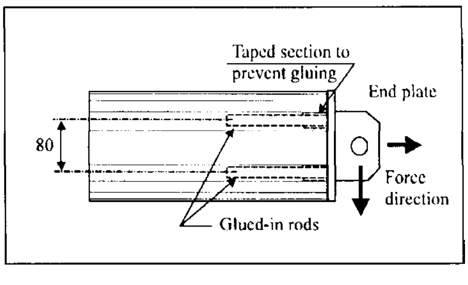
Fig 9: Load transfer in direct bearing instead of through connectors, which causes perpendicular-to-grain tension

Glued-in rod connectors

Glued-in threaded rods can be used very successfully in combination with steel plates as connectors, either in round pole construction or in laminated member construction (see Fig 10). Loads can be transferred either parallel or perpendicular to the grain. Furthermore, properly executed connections avoid eccentric loading of the members and connections. The strength of the connection may be based on the yield strength of the threaded rod, as it is possible to determine the depth of gluing so that the threaded rod will fail rather than the timber. Failure of the rod will also ensure a measure of ductility.

When designing a glued-in rod end connector, it is important to avoid any loading on the rods, which will cause a splitting action in the timber. Loading should be transferred to the bolts in tension and compression rather than in bending on the individual bolts. For loading perpendicular

Fig 10: Possible way to use glued-in rods to effect a connection



to the grain direction, the bolts should be placed in the plane of the induced moment (see Fig 10).

Uneven loading of the bolts or threaded rods, which are theoretically at the same load level, may occur when some nuts are tightened more than others. This is usually not a problem when bolts are used as the bolts have a reasonably long length and are able to yield over that length. Glued-in threaded rods immediately start to transfer load and thus have a very short length over which yielding can occur. When there is a possibility that unequal loading will occur, the ductility of the individual rods may be improved by taping sections of the rod, to prevent gluing in those areas (see Fig 10). The taped section is not fixed rigidly to the timber and will in that way increase the length over which the rod can yield.

Example: Glued-in rod connection

Transfer a factored (limit-states) perpendicular-to-grain load of 20 kN between one 150 mm pole and another. Assume the end plate to be as in Fig 10.

Shear force transferred between end plate and bolts	= 20 kN
Moment due to eccentricity of the connection	= 50 x 20
	= 1 000 kN.mm
Force in bolt due to moment	= 1 000/80
	= 12,5 kN

Design each bolt to take an axial load of 12,5 kN and a shear force of 10 kN. Table 6.25 of the *South African Steel Construction Handbook* (1992) gives strength values for bolts that are subjected to axial as well as shear loads. The table gives the value of the axial load once the ratio between shear force, V_u, and axial load, T_u, is known. For the load combination applied to the 12 mm threaded rod:

$$\frac{V_u}{T_u} = 0{,}8$$

Table 6.25 of the *South African Steel Construction Handbook* (1992) shows that a 12 mm, Grade 4.6 bolt will suffice. A threaded rod with a 12 mm diameter will generally have a higher yield stress than Grade 4.6 bolts and can thus be used.

Summary

The glued-in threaded rods work extremely well for parallel and perpendicular-to-grain loading applications. The magnitude of the force that can be transferred is usually limited only by the number of rods that can be placed into the end of the pole. As long as the end of the member is free of defects, timber failure should not occur. The strength of the timber is based on timber with defects and defect-free timber is very much stronger. Edge distances must be at least 1,5 threaded rod diameters and spacing, centre-to-centre, at least 2,5 threaded rod diameters.

Care must be taken that the rods do not have a splitting action as would be the case with a single large-diameter rod placed in the centre of the round pole and loaded perpendicular to the grain. The bending of the rod will cause large perpendicular-to-grain tensile stresses, which will cause the pole to split. The performance of such a connection will be no better than a similar sized threaded rod used to connect two poles next to each other, as in Fig 5.

Conclusion

Efficient round timber pole connections can be achieved if the full yield strength of the connector or the timber is used. By using the Eurocode 5 (1995) equations, the designer will be able to see which mechanism will allow the greatest force transfer and can thus adjust the connection until this load capacity is reached. Although the Eurocode 5 equations have been tested for European timber, the bearing stress equations were found by du Plessis (1995) and Engelbrecht (1995) to be over-confident for South African timber. I suggest that strengths determined using the Eurocode 5 equations should be reduced by 10 per cent until bearing stress equations have been produced for South African timber.

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