W M G Burdzik

Applicability of the Eurocode timber connector design formulae to connections in round poles

Synopsis

The Eurocode 5 (1995) timber connection design formulae are discussed briefly and applied to the usual standard bolted or nailed South African round timber pole connections to obtain theoretical strengths. These theoretical strengths are compared with test results for loading parallel and perpendicular to the grain. It is also shown that care must be taken in specifying end distances and how the SABS 0163-1:1994 formulae for groups of connectors loaded perpendicular to the grain can be applied to single bolted connections where end distances are less than specified. Furthermore, it is proposed that the Eurocode 5 formulae be accepted for inclusion in the South African design code, but with adjustments to the embedment stress formulae.

Samevatting

Die Eurocode 5 (1995) houtverbindingsontwerpformules word kortliks bespreek en op die gewone standaard bout- en spykerverbindings wat op ronde houtpale gebruik word, toegepas om sodoende teoretiese sterktes te verkry. Hierdie teoretiese sterktes word vergelyk met toetsresultate vir belasting parallel met en loodreg op die grein. Daar word ook daarop gewys dat sorg gedra moet word wanneer end-afstande gespesifiseer word en daar word geïllustreer hoe die SABS 0163-1:1994-formules vir groepe verbinders wat loodreg op die grein belas word op enkelverbinders toegepas kan word waar die end-afstande minder is as wat gespesifiseer word. Daar word verder voorgestel dat die formules vir insluiting in die Suid-Afrikaanse ontwerpkode aanvaar word, maar met aanpassings van die draspanningformules.

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

A round hollow member is the ideal form for a structural element, where loading may be applied in any direction, be it axial load or bending about any arbitrary axis. We thus find in a tree trunk a fairly ideal structural element. Even though the trunk has material inside it, the stronger material is found around the outside. As long as the outer portion of the trunk is not disturbed by machining, it is very strong.

However, very few structures can be constructed without a need to connect one member to another. Previous generations have used ingenious methods to connect round poles. These are unfortunately time-consuming, require a high level of skill and are thus costly.

Nails and bolts, and especially threaded rods, have simplified connections in structures constructed with round timber poles. However, although these simple connections seem to work for small structures, very little design information is available and very little has been published in South Africa about the safety of these connections. Furthermore, larger and larger structures are being built without the connections being designed properly.

As the South African limit-states design codes have moved away from safe load tables and specify the strength of members and connections in terms of formulae, I felt that design formulae, rather than strength tables, would be more in keeping with the present trends. These design formulae are based on failure mechanisms, which give the designer better insight into the safety of the connection and allow him to strive for the most efficient use of the material.

Limiting the test series

For some years, timber pole structures have been connected by means of 12 mm to 20 mm threaded rods. The poles are placed next to each other, a hole is drilled through all the members and a threaded rod is fed through the hole. Up to five members are sometimes joined in this fashion. The connections of such structures usually cannot be justified when the design is checked in accordance with SABS 0163-1:1994 and SABS 0160:1989, even when one allows for the fact that the structure is seldom subjected to live load. Very few catastrophic failures of such buildings have been reported, so the question must be asked whether the code values are too low and whether the connections have large reserve strength. Twelve millimetre rolled threaded rods are often used in connections and these, together with 7,1 mm drawn nails, were tested to ascertain how accurately the Eurocode 5 (1995) formulae would predict their strength. The 7,1 mm nails were tested to ascertain whether the effect of the smooth surface and the lack of a locking mechanism, such as nuts provide in the case of threaded rods, would influence the strength of the connector.

Eurocode 5 (1995) formulae

Whale (1991), Ehlbeck and Larsen (1993) and others have been involved in quantifying design formulae, test methods and design values for dowel-type connectors (nails, bolts, dowels, screws and staples) for inclusion in Eurocode 5 (1995). The design formulae are based on the yield theory equations of Johansen (1949). The Eurocode 5 (1995) formulae were used in this project as strength predictors for the bolted and nailed connections that were tested. Connections that are constructed with mechanical connectors can fail in one of two ways, ie:

1. Timber failure
2. Connector material failure

Timber failure can be due to crushing of the cells, shear failure parallel to the grain direction or tension perpendicular to the grain. Both shear failure and perpendicular-to-grain tension failures are sudden and give very little warning. Shear failure can be avoided if end distances and spacing between connectors are correctly applied. Crushing of the material seldom leads to sudden failure and it is a far less dangerous form of failure than either shear or tension perpendicular to grain. Connector failure, especially with metal connectors, is generally due to connector material yielding and may be a very ductile failure mode (see Fig 1). This type of failure is very predictable, giving ample warning, and is seldom catastrophic. Ductile failure of a connection can thus be regarded as the ideal failure mechanism.

SAICE Journal/SAISI-joernaal 1997, 39 (2): 1-5

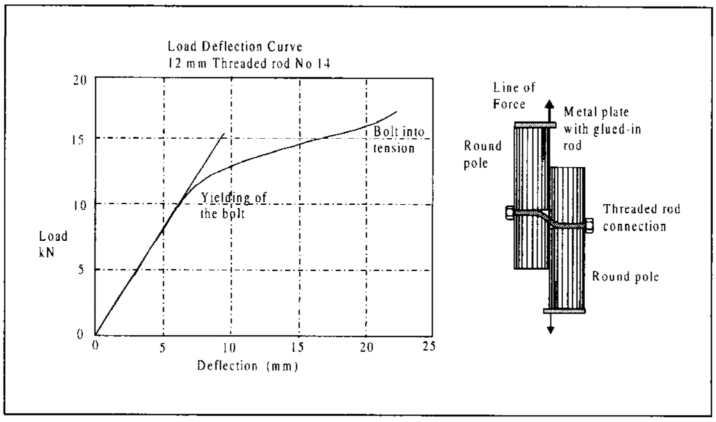
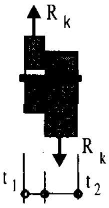


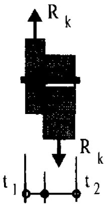
Fig 1: Typical load deflection curve for a 12 mm bolted connection in round timber poles

The characteristic load carrying capacity for doweled connections in single shear is the least of the values that result from the formulae given as follows in Eurocode 5 (1995):

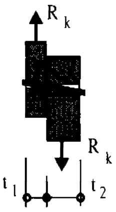
$$R_k = f_{h,1}t_1d \qquad \text{(a)}$$



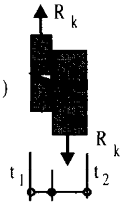
$$R_k = f_{h,1}t_2d\beta \qquad \text{(b)}$$



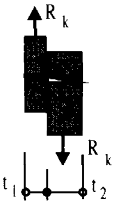
$$R_k = \frac{f_{h,1}t_1d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left(1+\frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right) + \beta^3\left(\frac{t_2}{t_1}\right)^2} - \beta\left(1+\frac{t_2}{t_1}\right)\right] \qquad \text{(c)}$$



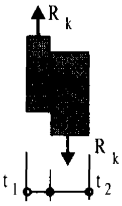
$$R_k = \frac{f_{h,1}t_1d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_y}{f_{h,1}dt_1^2}} - \beta\right] \qquad \text{(d)}$$



$$R_k = \frac{f_{h,1}t_1d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_y}{f_{h,1}dt_1^2}} - \beta\right] \qquad \text{(e)}$$



$$R_k = \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_yf_{h,1}d} \qquad \text{(f)}$$



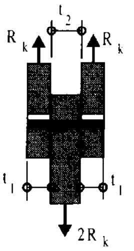
where:

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

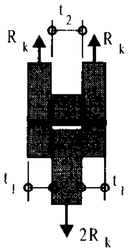
As can be seen from the failure mechanisms, (a), (b) and (c) relate to wood failure and (d), (e) and (f) to connector failure. The last three are therefore the more desirable mechanisms for connector failure where ductility is required, as they have large reserve strength and are able to dissipate a large amount of energy.

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

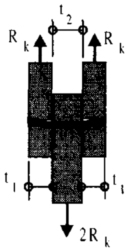
$$R_k = f_{h,1}t_1d \qquad \text{(g)}$$



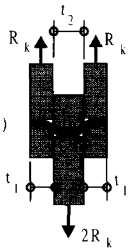
$$R_k = 0{,}5f_{h,1}t_1d \qquad \text{(h)}$$



$$R_k = \frac{f_{h,1}t_1d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_y}{f_{h,1}dt_2^2}} - \beta\right] \qquad \text{(i)}$$



$$R_k = \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_yf_{h,1}d} \qquad \text{(j)}$$



Again, the failure mechanisms (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.

SABS 0163-1:1994 formulae for tension perpendicular to the grain

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 tension perpendicular to grain comes into play. This formula is given in SABS 0163-1:1994 in the form of:

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

where (see Fig 2):

T_r = perpendicular-to-grain resistance, in N
ϕ = capacity reduction factor
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)
γ_{m1} = material factor for duration of load

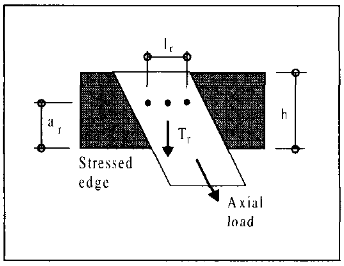


Fig 2: Multiple connectors loaded perpendicular to the grain

γ_{m6} = factor for area stressed by connectors = $\left(\frac{A_{eff}}{A_0}\right)^{0,2}$ (3)
A_{eff} = effective area = $l_{eff}\,d$, in mm²
A_0 = 10^6 mm²
a_r = distance from connector to stressed edge, in mm
b = penetration depth of the connector << 15 d, in mm

$$c = \frac{4}{3}\sqrt{\frac{a_r}{h}\left(1 - \frac{a_r}{h}\right)^3} \qquad (5)$$

d = diameter of the connector, in mm

h = height or depth of timber member, in mm

l_{eff} = effective stressed length, mm = $\sqrt{l_r^2 + (ch)^2}$
(Use $l_{eff}/2$ when distance to edge is less than depth of timber member)

l_r = distance between outer connectors of connector group, in mm

Embedment stress, $f_{h,1}$

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

The following material properties are proposed in Eurocode 5 (1995) in lieu of specific test data:

$$f_{h,1} = 0{,}09\,\rho\, d^{-0{,}36} \text{ MPa for all timber without pre-drilled holes} \qquad (6)$$

and

$$f_{h,1} = 0{,}13\,\rho\, d^{-0{,}36} \text{ MPa for pre-drilled holes} \qquad (7)$$

where:

ρ = density of the timber, in kg/m^3

d = nail diameter, in mm

Bolted joints

The following material properties are proposed in Eurocode 5 (1995) in lieu of specific test data:

For loading parallel to the grain:

$$f_{h,0} = 0{,}082\,(1 - 0{,}01d)\,\rho, \text{ in MPa} \qquad (8)$$

For loading perpendicular to the grain:

$$f_{h,90} = 0{,}036\,(1 - 0{,}01d)\,\rho, \text{ in MPa} \qquad (9)$$

where:

ρ = the density, in kg/m^3

d = bolt nominal diameter (mm), ie not reduced diameter of thread

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

$$f_{h,\alpha} = \frac{f_{h,0}}{2{,}3\sin^2\alpha + \cos^2\alpha} \qquad (10)$$

These equations apply to bolts and dowels with a minimum yield stress of 240 MPa.

Test procedure

Parallel-to-grain loading

Both 7,1 mm nails and 12 mm threaded rods were tested in a single shear connection as illustrated in Fig 1. Creosote-treated, 120 mm diameter round SA Pine poles were used for both bolted and nailed joints. The load line was kept as close as possible to the interface of the two 120 mm poles. As frictional forces can play a large role in the strength of a connection, these were eliminated by hand tightening only in the case of the threaded rods.

In the case of the nailed joints, 7 mm holes had to be pre-drilled into both poles to prevent the poles splitting. As the nails were only 150 mm long, one end was countersunk so that adequate penetration of the point into the second pole could be achieved.

In both the nailed and bolted specimens, the end distance was greater than the seven diameters required by SABS 0163-1:1994. The tests were limited to 12 bolted and 14 nailed connections as the Eurocode 5 (1995) design formulae had predicted connector yielding. Connector yielding is far more predictable than timber failure and thus requires a far smaller number of specimens. The timber that had been treated with creosote had an apparent density that was much greater than one would expect for poles with a diameter of 120 mm. As the density of the poles prior to treatment was unknown, I had to assume the measured density, which led to high values of the embedment stress. The connections behaved in a manner in keeping with the high density and it was thus felt that the creosote treatment could have improved the shear and crushing strength of the timber. An additional 10 specimens were tested later in untreated timber, with end distances equal to the minimum required by the code to ascertain whether actual density would lead to lower strength values.

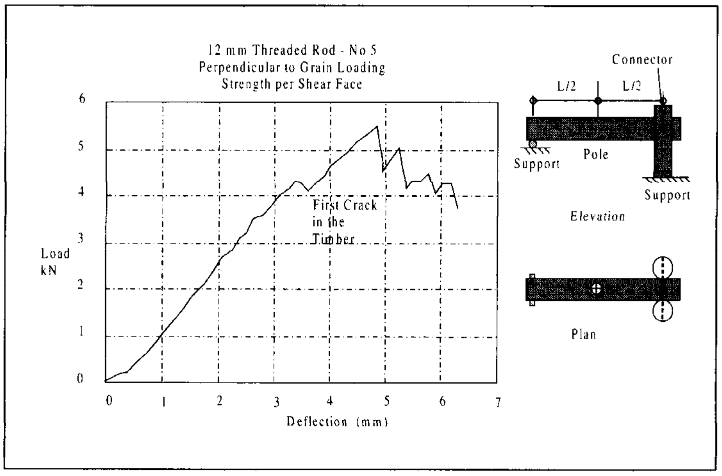


Fig 3: Typical load-deflection curve for 12 mm threaded rod in double shear – short end distance

Perpendicular-to-grain loading

In this case it is very difficult to test single shear connections, and double shear connections, as shown in Fig 3, were thus tested. Untreated *Eucalyptus grandis* poles were used for this series of tests. The tests were restricted to 12 mm threaded rods, as nail lengths were insufficient for double shear tests.

The poles had a mean density of 680 kg/m^3 with a fifth percentile density of 580 kg/m^3. Two end distances were investigated, namely 21 specimens with an end distance equal to six times the bolt diameter, ie 72 mm, and 10 specimens with an end distance of 300 mm. The shorter end distance was used to ascertain whether a reduced end distance would be critical. The shorter end distance is very often used in practice, as it appears to be 'neater'.

Predictions and test results

Parallel-to-grain loading

The round creosote-treated poles were measured and weighed and the density was calculated. The mean density of the sample was 928 kg/m^3, while the lowest density was 860 kg/m^3. If the characteristic density, ie fifth percentile density, of the timber is 860 kg/m^3, the embedment stresses can be calculated. The untreated round poles had a characteristic density of 580 kg/m^3. The rods were tested in tension to ascertain the yield stress and a minimum yield stress of 440 MPa was found.

For 12 mm threaded rods and 7,1 mm nails loaded parallel to the grain, the embedment stress for the 12 mm threaded rod (see Eqn 8) is given by:

$f_{h,0}$ = 0,082 (1 – 0,01 x 12) x 860 (treated poles) = 62,1 MPa

or 0,082 (1 – 0,01 x 12) x 580 (untreated poles) = 41,9 MPa

For 7,1 mm nails (see Eqn 8):

$f_{h,0}$ = 0,082 (1 – 0,01 x 7,1) x 860 = 65,5 MPa

β is the ratio between embedment stress in side 1 and side 2. Therefore β = 1,0, as both are assumed to be of the same density.

The equations for the characteristic strength of the connection loaded parallel to the grain can then be simplified and written as follows, where R_k is the minimum of:

$$f_h t_1 d \qquad (a)$$

$$f_h t_2 d \beta \qquad (b)$$

$$\frac{f_h t_1 d}{2}\,[\sqrt{8} - 2] \qquad (c)$$

$$\frac{f_h t_1 d}{3}\left[\sqrt{4 + \frac{12 M_y}{f_h d t_1^2}} - 1\right] \qquad (d)$$

$$\frac{f_h t_2 d}{3}\left[\sqrt{4 + \frac{12 M_y}{f_h d t_2^2}} - 1\right] \qquad (e)$$

$$\sqrt{2 M_y f_h d} \qquad (f)$$

The 12 mm threaded rod has an internal diameter of 10,1 mm according to SABS 135:1991.

The resultant plastic resistance moment of the connector $M_y = f_y d^3/6 = 440 \times 10{,}1^3/6 = 75\ 555$ N.mm.

The characteristic strength of a bolted connection in single shear is thus the least of:

(a) 89,4 kN
(b) 89,4 kN
(c) 37,0 kN
(d) 30,4 kN
(e) 30,4 kN
(f) 10,6 kN (treated) or 8,7 kN (untreated)

Failure mechanism (f) governs and the characteristic resistance strength of the bolted connection is thus the lesser of ϕ times 10,6 kN, where ϕ is the strength reduction factor as defined in SABS 0162-1:1993, or the characteristic strength is equal to the fifth percentile strength, which one obtains from the test results. The ultimate resistance must then be calculated in accordance with SABS 0163-1:1994. Similar calculations can be done to estimate the strength of the nailed connection. The yield stress of the nails was found to be 450 MPa and the plastic moment of resistance M_y = 26 800 N.mm. The estimated strength of the nailed connection can be calculated from Eqn (f) and is equal to 5,0 kN. The values obtained from the test results are given in Table 1. The load deflection curve for the weakest 12 mm threaded rod is shown in Fig 1 and for the weakest nailed connection in Fig 4.

Table 1: Test results for parallel-to-grain loading

Type of connector	*Theoretical strength in accordance with Eqn (f)* kN	*Highest yield value of test specimens* kN	*Lowest yield value of test specimens* kN	*Mean yield value of test specimens* kN	*Lowest ultimate strength of test specimens* kN
12 mm threaded rods in treated timber	10,6	13,0	10,0	11,7	26,0
12 mm threaded rods in untreated timber	8,7	9,0	7,9	8,2	19,2
7,1 mm nails	5,0	6,3	3,2	5,3	7,1

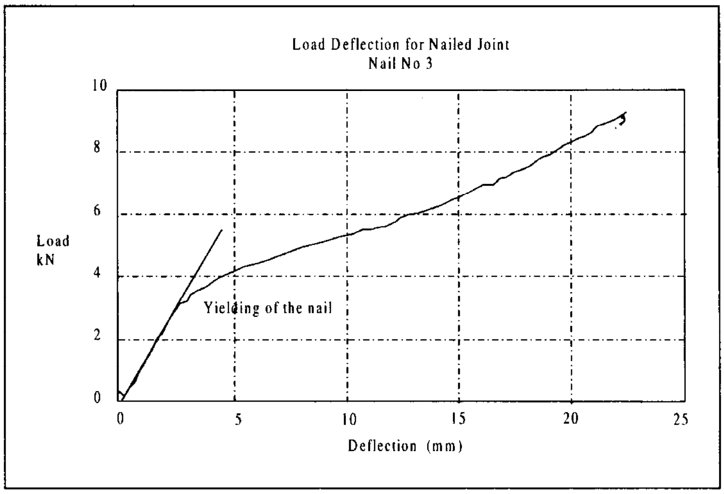


Fig 4: Typical load-deflection curve for 7,1 mm nail connection in round creosote-treated poles

Perpendicular-to-grain loading

End distances that were less than required by SABS 0163-1:1994 were used to ascertain whether reduced distances would make the connection significantly weaker. Reduced end distances are often used to 'neaten' the connection. The predicted embedment or bearing stresses for these specimens were as follows:

For the vertical member loaded parallel to the grain:

$$f_{h0,1} = 0{,}082\ (1 - 0{,}01\ d)\ \rho \quad = \quad 41{,}9\ \text{MPa} \qquad \text{(Eqn 8)}$$

For the horizontal member loaded perpendicular to the grain:

$$f_{h90,2} = 0{,}036\ (1 - 0{,}01\ d)\ \rho \quad = \quad 18{,}4\ \text{MPa} \qquad \text{(Eqn 9)}$$

$$\beta \quad = \quad \frac{f_{h90,2}}{f_{h0,1}} \quad = \quad 0{,}439$$

The plastic resistance moment of the threaded rod is M_y = 75 600 N.mm.

The Eurocode 5 (1995) formulae for double shear predict a single shear face strength of:

(g) = 60,3 kN
(h) = 13,2 kN
(i) = 17,5 kN
(j) = 6,8 kN

Eqn (j) governs and the predicted strength at yield of the bolt is 6,8 kN.

The values obtained from the test results are given in Table 2. The load deflection curve for the weakest 12 mm threaded rod with 72 mm end distance is shown in Fig 3 and for the weakest with 300 mm end distance in Fig 5.

Table 2: Test results for perpendicular-to-grain loading

End distance to connector	*Theoretical strength in accordance with Eqn (j)* kN	*Highest yield value of test specimens* kN	*Lowest yield value of test specimens* kN	*Mean yield value of test specimens* kN	*Lowest ultimate strength of test specimens* kN
72 mm end distance	6,8	8,7	3,8	6,4	4,8
300 mm end distance	6,8	8,7	7,0	7,8	8,0

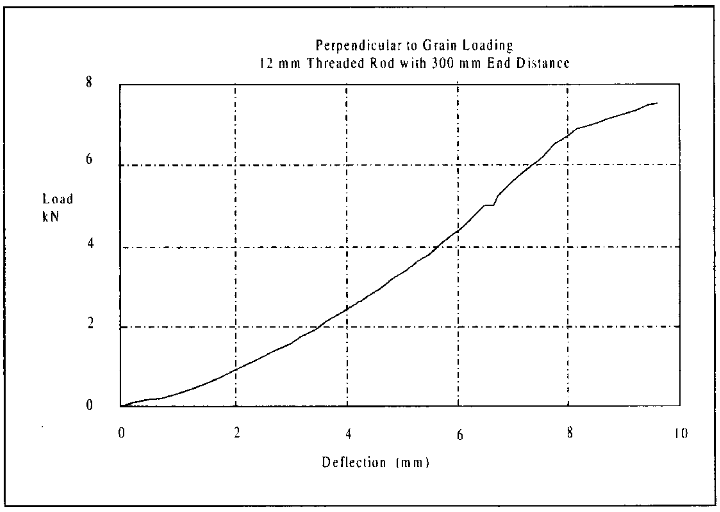


Fig 5: Load-deflection curve for 12 mm threaded rod in double shear – long-end distance

All the connections with the 300 mm end distance failed through bending of the rod as for Eqn (j). The connections with a 72 mm end distance, which showed a low first yield and failure strength, all failed through splitting of the pole. The resistance of a group of connectors loaded perpendicular to the grain in accordance with SABS 0163-1:1994 can be calculated as follows for a single connector:

a_r = 60 mm (Half diameter of pole)

h = 120 mm (Diameter of pole)

$$c = \frac{4}{3}\sqrt{\frac{60}{120}\ (1 - \frac{60}{120})^3} = 0{,}333 \qquad \text{(Eqn 5)}$$

$$\eta = 1 - 3\,(\frac{60}{120})^2 + 2(\frac{60}{120})^3$$

l_{eff} = $\sqrt{1_r^2 + (ch)^2}$ but $l_r = 0$, $\therefore l_{eff} = c\,h$ = 40,0 mm
Halve for short end distance = 20 mm

A_{eff} = 40 x 120 = 4 800 mm², or halved for short end distance = 2 400 mm²

$\gamma_{m6} = (\frac{A_{eff}}{A_0})^2 = 0{,}344$, or for short end distance = 0,299

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 Grade 7 timber, the perpendicular-to-grain tensile strength will not be significantly higher. The higher parallel-to-grain strength of the poles is due to the structure of the tree not having been disturbed by a sawing process. The effect of knots in undisturbed timber is much less than in sawn timber. If one thus assumes that the pole is a Grade 7 member:

$$f_{tp} = 0{,}51 \text{ MPa}$$

The ultimate resistance without the capacity reduction factor is thus:

$$T_r = \phi \times \frac{0{,}51}{0{,}5 \times 0{,}344} \times 4\,800 = \phi \times 14{,}23 \text{ kN (double shear)}$$

$$= \phi \times 7{,}12 \text{ kN (single shear)}$$

or for short edge distance:

$$T_r = \phi \times \frac{0{,}51}{0{,}5 \times 0{,}299} \times 2\,400 = \phi \times 8{,}19 \text{ kN (double shear)}$$

$$= \phi \times 4{,}1 \text{ kN (single shear)}$$

Discussion

The test results show that the Eurocode 5 (1995) formulae predict the strength of bolted connections in round poles fairly well, especially where failure is governed by yielding of the metal connector. From both the parallel-to-grain and the perpendicular-to-grain bolted or nailed connection tests it is apparent that metal failure can be induced. End distances must be large enough to prevent splitting or shear failure. It appears that the seven times connector diameter end distance is large enough to prevent splitting when the poles are loaded parallel to the grain.

Where poles are split at the ends, larger end distances should be considered. The perpendicular-to-grain loading formula given in SABS 0163-1:1994 predicted a characteristic load of 4,1 kN as opposed to a minimum test failure load of 4,8 kN. Larger end distance should also be considered when ductile failure or a large reserve resistance is required. These larger end distances for poles with end splits could be specified on the drawings as general notes.

From the test results it appears that the Eurocode 5 (1995) formulae for embedment stress may lead to values that are on the high side for South African timber. Predicted strength values are higher than the minimum test values. The average density was used to predict the bearing stress for the connectors. As the round poles have their densest material around the outside of the pole, the predicted bearing stress should have been higher than the test values have indicated.

Subsequent tests done at the University of Pretoria by du Plessis (1995) and Engelbrecht (1995) have indicated that the bearing stress is less than that predicted by the Eurocode formulae. Further tests are required to ascertain the relationship between density, connector size and bearing stress, before the Eurocode formulae can be applied to sawn South African timber. Once the relationship between these values is known, serious consideration should be given to inclusion of the Eurocode 5 (1995) formulae in the South African code.

Acknowledgements

I wish to thank the University of Pretoria for the use of their facilities and the Foundation for Research Development, Hans Merensky Holdings (Pty) Ltd, Lignum Lab and the South African Wood Preservers' Association for their financial assistance. I also thank Mr Dave Campbell for his valuable comments.

References

1. CEN. 1995. *ENV 1995-1-1, Eurocode 5, Design of Timber Structures, Part 1.*
2. Du Plessis, B M. 1995. *The bearing strength of connectors, parallel to the grain.* Final year essay, in Afrikaans, Dept of Civil Engineering, Univ of Pretoria.
3. Ehlbeck, J and Larsen, H J. 1993. Eurocode 5 – Design of timber structures: Joints. *Proc,* International Workshop on Wood Connectors, Forest Products Society, USA.
4. Engelbrecht, H J. 1995. *The search for a rational method of determining bearing stress of timber perpendicular to the grain.* Final year essay, in Afrikaans, Dept of Civil Engineering, Univ of Pretoria.
5. Johansen, K W. 1949. *Theory of timber connections.* IABSE, Publ 9.
6. SABS 0160:1989, as amended 1993. *South African Standard Code of Practice. The general procedures and loadings to be adopted in the design of buildings.* South African Bureau of Standards, Pretoria.
7. SABS 0162-1:1993. *South African Standard Code of Practice. The structural use of steel – Part 1: Limit-states design.* South African Bureau of Standards, Pretoria.
8. SABS 0163-1:1994. *South African Standard Code of Practice. The structural use of timber – Part 1: Limit-states design.* South African Bureau of Standards, Pretoria.
9. SABS 0163-2:1994. *South African Standard Code of Practice. The structural use of timber – Part 2: Allowable stress design.* South African Bureau of Standards, Pretoria.
10. SABS 135:1991. *South African Standard Specification. ISO metric bolts, screws and nuts (hexagon and square) (coarse-thread free-fit series).* South African Bureau of Standards, Pretoria.
11 Whale, L R J. 1991. Joints, especially dowel-type joints. *Proc,* 1991 International Timber Engineering Conference, London, United Kingdom.

Discussion on papers

Written discussion on the technical papers in this issue of the *Journal* will be accepted until 31 July 1997. This, together with the authors' replies, will be published in the Fourth Quarter 1997 (December) issue of the *Journal*, or the issue thereafter. For the convenience of overseas contributors only, the closing date for discussion will be extended to 31 August 1997. Discussion must be sent to the Directorate of SAICE.

Such written discussion must be submitted in duplicate, should be in the first person present tense and should be typed in double spacing. It should be as short as possible and should not normally exceed 600 words in length. It should also conform to the requirements laid down in the 'Notes on the preparation of papers' as published on the inside back cover of this issue of the *Journal*.

Whenever reference is made to the above papers this publication should be referred to as the *Journal of the South African Institution of Civil Engineering* and the volume and date given thus: **J SA Inst Civ Eng, Vol 39, No 2, Second Quarter 1997.**