WCTE 2014 Quebec City Canada August 10-14
World Conference on Timber Engineering

CONNECTION FOR ROUND WOOD TIMBER MEMBERS USING MULTIPLE GLUED-IN RODS

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ABSTRACT:

This paper presents a research work undertaken, whose main purpose was the development and validation of an innovative connection system for round wood members. The connection proposed is based on axially glued-in rods. A preliminary research was undertaken in order to assess the properties of the glued-in rods using different gluing lengths, steel grades and timber densities. Based on these results three connection configurations were defined aiming the optimization of the connection load carrying capacity. These configurations were afterwards tested in laboratory to assess their mechanical performance. The results obtained showed that the proposed connection leads to high performance, either in terms of load carrying capacity or stiffness if appropriate connection configurations are used.

KEYWORDS: Connections, Round wood, Glued-in Rods, Experimental testing

1 INTRODUCTION

Round wood is a suitable material for various structural applications, namely truss systems for roofs and bridges. In Portugal a large amount of this material is available from the forest thinning. Various studies have been undertaken in order to create the conditions required so that small diameter round wood members can be used as structural components [1-4]. These small sections, need to be connected through suitable connecting systems. The issue of effectively connecting round wood elements has been the target of several studies. However, the available connections are expensive, hard to implement, have aesthetic problems and must be developed for each particular case, preventing their mass production [5-8].The main goal of this study was the development and validation of innovative connection systems for this type of timber member.

An extensive review of connections for small diameters round members was presented by Lukindo et al., [9]. From that review, the connections that presented more potential, for this specific application, were the dowel nut and the central plate, since they are suitable to transfer compression and tension stresses and are also easily connected to nodes.

The connections with central plates require a slot across the pole's diameter to allow the insertion of the plate. Different strategies can be used to increase the load carrying capacity, such as for example, placing bands or wires around the connectors and to ridge the steel plate. Nevertheless, these connections have aesthetic problems, particularly, when external steel devices are added to increase the load carrying capacity [7-11].

The dowel nut connections are made with a through bolt, placed in a longitudinal hole through the pole centre, and a dowel nut, placed transversely in the section. This type of connection was studied in various research projects [5, 12]. These connections, however, require a high level of precision in order to assure the correct position of the longitudinal hole and the transversal hole, and this lack of easy manufacture is a significant disadvantage.

Having these aspects into consideration it becomes clear that the development of a connecting system that combines the advantages of the two connection types described

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before, but without its disadvantages, namely aesthetic problems and complexity of execution, would represent an enormous step forward for round wood structures made of small diameter members. Aiming this end, an innovative possibility was considered based on the use of glued-in-rods.

The glued-in rods connections were studied mainly in glulam connections in applications rather different from the one addressed in this study. This connection has a superior aesthetic when compared with the previous connections, since the visible metallic parts are much smaller.

From the research undertaken three connection configurations were proposed. These configurations were optimized in order to increase the load carrying capacity. A total of 84 tests were performed to determine the mechanical properties of the connection. The results are presented and compared with the one available for other connection types. Additionally, a numerical model is proposed to predict the load carrying capacity of this type of connection.

2 MATERIALS AND METHODS

The development of the connection comprised three independent phases: preliminary tests, definition of the configurations and laboratory testing of the final connection configurations.

Since the connection was based on glued-in rods, before the final configurations were defined, it was necessary to determine the mechanical properties of the single glued-in components, namely its load carrying capacity. This was done through a batch of preliminary tests that include the analysis of the following parameters: steel grade, density of timber and gluing length.

The timber used was Portuguese Maritime pine (*Pinus pinaster,* Ait) obtained from small diameter trees, whose densities and diameters are given in Table 1. The steel rods used in these preliminary tests had a nominal diameter of 10mm, and two steel grades: 4.8 and 8.8.

Table 1: *Round wood preliminary tests*

d (mm)	**$\rho_{12\%}$ (kg/m^3)**	**d_{rw} (mm)**
Minimum	496	186
Maximum	697	242

The glue used was Icosit K101 TW® [13] which was considered to have physical and mechanical properties suitable for the intended application.

The load was applied to the test specimens with a rate of 0.035mm/s, which was expected to lead to the specimen failure in approximately 100seconds. The results for the 4.8 and 8.8 steel grade rods are given in Table 2 and Table 3 respectively.

Table 2: *Preliminary test results for 4.8 steel rods*

Glued depth	**F_{rd} (kN)**		**Nr of tests**
	Average	**St. dev**	
50 mm	23.82	3.33	22
75 mm	30.54	1.31	38
100 mm	31.37	1.16	36
125 mm	31.67	0.67	20

For the specimens with a glued depth of 50mm (5d) the failure occurred mainly in the timber, but in five specimens the failure occurred in the glue, as it is presented in Figure 1. For the specimens with a glued length of 75mm (7.5d) the failure occurred mainly (20 specimens) due to tension failure of the steel, but in 10 specimens the failure occurred in the timber while in the remaining 8 specimens the failure occurred in the glue. When the glued length was increased to ten times the rod diameter (100mm) the failures occurred always on the steel fastener. Based on these results it was concluded that a glued length of 10 times the diameter was enough in order to maximize the strength of the 4.8 steel grade rods.

a) b)

Figure 1: *Failure of the glued-in rod in the preliminary tests, a) failure on timber, b) failure on glue [14]*

A smaller number of specimens were prepared and tested using 10mm, 8.8 steel grade rods. In these tests larger gluing depths were used due to the higher strength of the rod, namely: 100mm (10d), 125mm (12.5d) and 150mm (15d). The results obtained are given in Table 3.

Table 3: Preliminary test results for 8.8 steel rods

Glued depth	F_{rd} (kN)		Nr of tests
	Average	**St. dev**	
100 mm	46.31	1.48	5
125 mm	46.34	0.72	5
150 mm	46.33	0.28	5

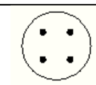
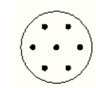
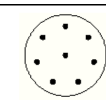
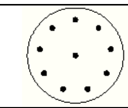
In the 10d gluing depth the failure occurred in the steel rod once and in the timber in the remaining four tests. In the other two gluing lengths the failure occurred always in the steel rods. These clearly indicate that the 10d penetration depth does not assure the maximum strength when 8.8 steel grade rods are used. However, the load carrying capacities obtained for the three glued lengths are quite similar, indicating that the optimum length is close to the 10 times the diameter. In spite of that a glued depth larger than the 10d is recommended once the failure in steel is ductile whereas the failure on timber or glue is brittle.

Based on the results from these preliminary tests it was possible to optimize the rod distribution in the cross section. This optimization was the base to define the connection configurations that were tested. In this analysis the following objectives were defined:

- maximize the load carrying capacity of the connections;
- fulfil the edge (2.5d) and fastener (5d) spacing requirements appropriate for this application,
- obtain ductile failure modes,
- minimize the global cost of the connection.

In Table 4 are presented the configuration of the fasteners disposition for the three nominal diameters, for the various round wood diameters which are expected to be found, more often, in practice. Additionally, the maximum load carrying capacity in proportion to the characteristic tension strength of the round wood member was determined. Once the objective is to obtain failures governed by the steel strength, timber properties were estimated, based on the minimum boundary, namely for round wood tension stress and log diameter. A timber grade corresponding to C18 in accordance with the strength grades defined in EN 338 [15] was used together with the minimum nominal diameter for that configuration. For the same reasons, the maximum load carrying capacity of the connections was determined assuming the upper boundaries, namely ultimate tension stress in all the steel fasteners.

Table 4: Configuration of the connection and the expected load carrying capacities in proportion to the characteristic tensile strength of the round wood member

d (mm)	Round wood diameter (mm)			
	100-120	120-140	140-160	160<
8				
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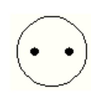
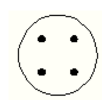
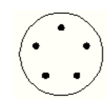
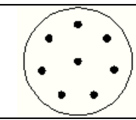
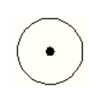
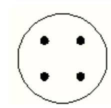
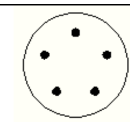
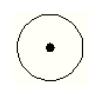
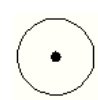
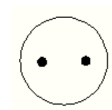
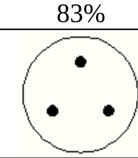
10				
	59%	82%	75%	92%
12				
	42%	59%	87%	83%
14				
	58%	40%	59%	68%

Based on this analysis, and on the objectives defined before it was decided to select the following configuration for the connection tests:

- 8 mm with four steel rods
- 10mm with four steel rods
- 12mm with four steel rods

In Figure 2 is presented a scheme of the test configuration used in the tests.

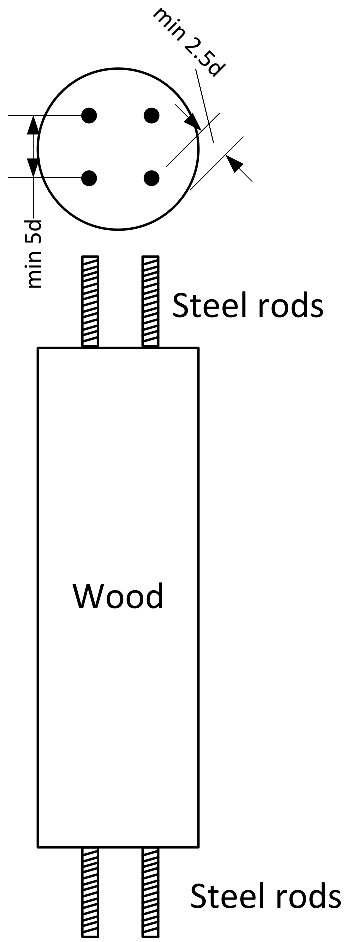


Figure 2: *Test configuration*

The three configuration selected have a good compromise between the load carrying capacity expected and the complexity/cost of the connections. The number of tests performed, for each one of the test series, is presented in Table 5.

Table 5: *Tested configurations*

d (mm)	Nr. tests		$\rho_{12\%}$ (kg/m^3)	ω (%)
		Average	554.6	13.2
8	19	Maximum	674.6	14.1
		Minimum	465.1	12.1
		Average	573.0	13.3
10	50	Maximum	682.8	14.0
		Minimum	417.0	12.0
		Average	574.3	13.2
12	20	Maximum	677.2	13.9
		Minimum	506.0	12.2

For these three configurations, laboratory tests were performed following the indications given in EN 26891 [16], to determine the connection stiffness and load carrying capacity. In order to obtain hinged conditions, on the connection ends, a special test device was designed to be accomplished to the universal testing machine, and it is presented in Figure 3. Similar connection arrangement is also an advantage in practice once it allows a much more uniform load distribution.

Figure 3: *Hinged system used in the connection test*

The deformations were measured as the differential displacement between the hinged system and the round wood member at the section were the end of the steel rods is located.

Figure 4: *Test set up*

After the test, a full disk was cut from the middle of the timber element to determine the density ($\rho_{12\%}$ - adjusted to 12% moisture content) and moisture content (ω) at the time of testing, the results obtained are also given in Table 5.

3 RESULTS

The failure of the glued-in rod connections, as expected, occurred in the steel rod, with significant plastic deformations occurring before the connection failure. In Figure 5 is presented a typical load slip curve of the connection.

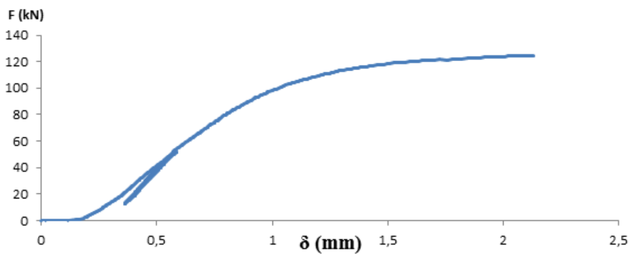


Figure 5: *Typical load slip curve of the connection*

The failure modes observed in the tests were caused by the yielding and failure of the steel rods. Nevertheless, in one configuration the failure occurred on timber, probably due to a deep timber crack in the connection area that initiated the failure mechanism for that connection (Figure 6). The failure in the steel rods does necessary result in significant plastic deformations which are quite clear from the load-slip graph presented in Figure 5. As it was expected, the failure took place with the failure of, at least, two rods. In many of the connections three of the rods failed at the moment of the connection failure. These results clearly indicate that, with this configuration, both the type of failure and the failure load are closely connected to the rod mechanical properties. Additionally, it is also clear that the strength of the steel rods is used to a large extend due to their elastic-plastic mechanical behaviour of the single rods.

Figure 6: *Failure modes observed in the tests*

A short summary of the results obtained for the failure load (F) and stiffness (K_s), is given in **Error! Reference source not found.**.

Table 6: *Resume of the test results obtained in the experimental tests*

d (mm)	Nr. tests		F_{exp} (kN)	K_s (kN/mm)
8	19	Average	74.1	109.2
		Maximum	75.6	217.3
		Minimum	71.5	48.7
10	50	Average	126.5	128.2
		Maximum	133.3	277.3
		Minimum	111.9	55.6
12	20	Average	155.3	171.6
		Maximum	158.9	301.9
		Minimum	151.0	103.2

The test results showed a relatively low variation of the load carrying capacity, as it should be expected when the failure is governed by steel failure. This fact, directly connected with the failure modes observed, almost always in the steel, is highly favourable for the load carrying capacity design values.

Neither in the literature nor in the code [17], could a suitable model be found, to predict the connection load carrying capacity. Due to the failures observed in the experimental tests, as a first approach, to estimate the load carrying capacity, it can be assumed that all the four steel rods had a stress applied equal to their yielding load, when the connection failure took place. Assuming the nominal rod diameter and the steel grade used (4.8), the expected failure loads were computed and are presented in Table 7

Table 7: *Estimated and observed failure loads*

d (mm)	$F_{rw,k}$* (kN)	$F_{est,y}$ (kN)	$F_{est,u}$ (kN)	F_{exp} (kN)	Δ_y (%)	Δ_u (%)
8	86.4	64.3	80.4	74.1	13%	-9%
10	124.4	100.5	125.7	126.5	21%	1%
12	169.3	144.8	181.0	155.3	7%	-17%

* Strength grade C18 was assumed in the calculations and the minimum wood member suitable for the connection configuration

The results presented in Table 7 clearly show that the estimation of the connection failure load based on the steel yielding strength underestimates the load carrying capacity of the connection. On the other hand, if the estimation of the connection load carrying capacity is based on the steel tension strength the model tends to overestimate the connection load carrying capacity. Nevertheless the last estimate is closer to the experimental determined load carrying capacity. These analyses comes in line with what was concluded from the observed failure modes, that the rods reach their yielding stress before the connection failure, but not all the rods reach their ultimate load carrying capacity.

These results and analyses show that for design purposes the yielding stress shall be used for the estimation of the connection load carrying capacity. In spite of showing larger differences to the actual load carrying capacities than the estimative based on the ultimate load carrying capacity, it is on the safe side.

In terms of the elastic stiffness the connection developed showed an excellent performance. Indeed, the values measured in the test for the elastic stiffness were high, leading to connection deformations in service that are in most of the situations lower than 1 mm. Such low deformation level can be disregarded in most of the structural applications of small diameter round wood material.

4 CONCLUSIONS

In the research work presented and discussed in this paper a new connection configuration was developed and tested for truss applications made using small diameter round wood members. This connection configuration is based on the glued-in rods and was able to eliminate or at least to significantly minor the disadvantages identified with the connection configurations available for these applications. Additionally to the excellent mechanical performance, the developed connection is easier to assembly than the other connections and with less visible metallic parts, which results in a more visually appealing solution.

The optimization of the connection configuration led to solutions on which the failure was governed by the steel. These allow the combination of high load carrying capacities, with high stiffness in service (low deformations) and high ductility. These three mechanical properties are key factors to a high and safe performance of a structural component.

The failure loads obtained in the tests were at the same level from the load carrying capacity of the timber members in tension. It is then easy to design this type of connections so that the failure is close to the maximum load carrying capacity of the timber members (at least for most current mid/low strength classes) but with the failure occurring in the ductile material which is the steel. The connection load carrying capacity varied between 74,1 kN for 8mm rods applied in a round wood member with a minimum diameter of 100mm and a maximum of 155,3 kN for 12mm rods applied in round wood members with a minimum diameter of 140mm.

Due to the type of failure that was observed, in the steel rods, very low coefficients of variation were obtained for this mechanical property. This is relevant once the characteristic values necessary for the design will significant benefit from this low variation.

This connection type did also show a very high stiffness. The deformations expected in service are rather low, and in most of the practical applications can be disregarded.

The high ductility that results from the failure modes that occur, also assures a very high ductility in practice with the yielding stress being reached only on the steel components.

ACKNOWLEDGEMENT

This study has been supported by Fundação para a Ciência e Tecnologia, with the scholarship SFRH/BD/44909/2008, and by QREN, with funds from FEDER, under the Programa Operacional de Factores de Competitividade, with the research project PTDC/ECM/099833/2008.

NOTATION

Δ_y is the difference between the connection estimated load carrying capacity, $F_{est,y}$, and the one observed in the experimental tests

Δ_u is the difference between the connection estimated load carrying capacity, $F_{est,u}$, and the one observed in the experimental tests

d is the nominal diameter of the steel rod

d_{rw} is the average diameter of the round wood member

$\rho_{12\%}$ is the timber density for 12% moisture content

ω is the moisture on timber members

F_{exp} is the load carrying capacity of the connection obtained experimentaly

$F_{est,y}$ is the estimated load carrying capacity of the connections obaitend using the steel yielding stress

$F_{est,u}$ is the estimated load carrying capacity of the connections obatiend using the steel ultimate stress

F_{rd} is the failure load obtained experimentally for a single glued-in rod

$F_{rw,k}$ is the characteristic value of the nominal round wood tension failure load

K_s is the slip modulus of the connection

δ is the connection slip

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