Experimental study on multiple-bolt steel-to-timber tension joints

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ABSTRACT

An experimental investigation on steel-to-timber joints arranged with multiple bolts was carried out, both in the case of load parallel and perpendicular to the timber grain. The joints were made with timber side members and steel main member; high strength bolts were used for fasteners. The experimental results allowed determining the stiffness in service, the slip at failure and the reference loads (proportional limit, yielding, ultimate). A decreasing ductility of joints was observed at the increase of the number of bolts. With the above characteristic loads the bolt effectiveness of multiple joint loaded parallel to grain was evaluated and compared with that obtained from some analytical predictions found in the literature. It was evidenced that the provisions of Eurocode N.5 provide values for the bolt effectiveness quite closer to experimental results. The results of joints loaded perpendicular to the grain were compared with some analytical predictions; significant differences were noted with all analytical predictions considered.

1. INTRODUCTION

Bolted connections are very often preferred to other types of mechanical fasteners, as for example split rings, because of their quick and easy construction procedure. Even though these connections are simple in construction, they are characterized by a behavior under load rather complex to model in theoretical analysis. In fact, due to the flexural deformability of the bolt and the stress concentration in the hole of joint members, the study of the mechanical behavior of bolted connections needs a three-dimensional analysis. Moreover the behavior strictly depends on some varying parameters as wood specie, geometry of side and center elements, end and edge bolt distances, spacing and number of fasteners, etc. The design of a joint without properly considering the effect of all the above parameters may lead to unsafe structures.

Owing to the complexity of the problem the design values (ultimate resistance, stiffness in service) of bolted timber joints, in many countries, are still based on an empirical fit of experimental results. Some other countries refer to simple analytical two-dimensional models, based on limit equilibrium [1], to determine the ultimate resistance, and on the theory of the beam on elastic foundation [2], to assess the stiffness of the joint in service. These models allow to predict quite correctly the actual values for joints with one single bolt, provided that adequate values for the embedding strength of wood and the elastic stiffness of the wood beneath the bolt (subgrade modulus) are assumed [3, 4].

To extend the validity of these models to the case of joints with multiple fasteners in a row parallel to the loading
direction, suitable bolt-modification factors have to be used. In fact, it has been observed by many researchers (e.g. [5-11]) that the ultimate resistance of a multiple-fastener joint is lower than the sum of the individual bolt capacities because an unequal load share among the fasteners occurs. Theoretically, this different distribution has been found to be dependent of several joint parameters: the number of fasteners, the end and edge distances and the fastener spacing, the dimensions of the joint members, the wood species, the construction tolerances, the load-slip characteristics of single fastener, etc. [12].

Different procedures are available in the literature to evaluate the load distribution among bolts which account for most of the above parameters; some of them assume a linear relationship for the load-slip of the fastener (e.g. [5, 8, 13]) whereas some others assume a nonlinear relationship (e.g. [7, 10, 11]). These procedures, however, do not predict correctly the actual behavior of each bolt since the fabrication tolerances and the variability of single fastener load-slip curve are not accounted for [4, 14].

Actually it is very difficult to take into consideration such parameters because they are both unknown before the joint construction. For this reason design codes of many countries take into account the reduced strength of a multiple-fastener joint by considering very simple empirical relationships for the modification factor.

For multiple fastener joints with the load acting perpendicular to the grain, the ultimate resistance may be determined on the basis of a fictitious shear design over the residual cross section, defined by the farthest row of fasteners and the loaded beam edge [15], or through methods based on the theory of fracture mechanics (e.g. [16, 17, 18]) or also by using empirical relationships (e.g. [19]).

As stated the number of experimental studies on the behavior of multiple fastener steel-to-timber joints available in the literature are rather limited and moreover they concern only joints with steel side members. Provided that in practice are frequently used joints with steel main member, an experimental investigation on three member steel-to-timber joints with multiple bolts have been carried out; moreover high strength bolts (class 8.8) were used.

The purpose of the study was to check if the current analytical predictions for the capacity of multiple-fastener joints are adequate also for the joints considered in the study. Moreover the study was aimed to evidence the actual behavior of such joints with particular concern to the variation of ductility at the increase in the number of bolts.

### 2. Analytical Relationships for Multiple Joint Capacity

It is acknowledged that the ultimate resistance of multiple fastener joints loaded parallel to the grain is lower than the load capacity of a single fastener multiplied by the number of bolts. As stated in the preceding section various parameters (e.g. deformability of joint members, fabrication tolerances, end and edge distances, bolt spacing, etc.) cause an unequal distribution of the load among the fasteners and moreover the timber splits earlier than for single connector, leading thus to a more brittle failure. The reduced resistance of multiple joints is mainly due to these circumstances. On the contrary, for multiple fastener joints loaded perpendicular to the grain the joint capacity is mainly governed by the position and distribution of the fasteners within the depth of the timber member. The joint collapse is normally due to tensile stresses perpendicular to the grain that lead to transversal splitting of the timber member.

#### 2.1 Joints loaded parallel to the grain

Various researchers faced the problem of multiple fasteners joints loaded parallel to the grain (e.g. [5, 7-11, 13, 20]) and proposed analytical relationships for the effective number of fasteners \( n_{\text{ef}} \) to be used for evaluating the resistance of joints. These relationships were obtained from different types of analysis of the joint behavior: some of them empirical and some others theoretical. The resistance of the connection is then obtained with the equation:

\[
F_m = n_{\text{ef}} \cdot F_s
\]

where \( F_m \) and \( F_s \) are the resistance of multiple and single fastener joints, respectively.

The relationship for the effective number of fasteners \( n_{\text{ef}} \) suggested in the Eurocode N. 5 (ENV 1995-1-1 2000) [16] is derived from the research work of Jorissen [11]; it depends on the number of bolts \( n \) per row in the grain direction, the spacing distance \( a \) and the fastener diameter \( d \):

\[
n_{\text{ef}} = n^{0.9} \cdot \frac{\sqrt[3]{a}}{13d} \quad (<n).
\]

The empirical relationship adopted in the Canadian Standards Association (CSA 2001) [15], derived from the research work of Smith [9], is a function of the number of bolts in the load direction, the spacing of fasteners \( a \) and the ratio between the timber element thickness \( t \) and the bolt diameter \( d \), that is:

\[
n_{\text{ef}} = 0.25\beta \left( \frac{t}{d} \right)^{0.5} \left( \frac{a}{d} \right)^{0.2} \cdot n^{0.7}.
\]

The coefficient \( \beta \) takes a unitary value for one single row of bolts in the load direction, 0.8 for two rows of bolts and 0.6 for three rows of bolts.

The National Design Specification for wood construction in the USA (NDS 1997) [21] uses the relationship proposed by Zahn [8], which is derived from the studies of Lantos [12] on the redistribution of the applied load among the bolts of the joint. The method considers the deformability of timber and steel joint members as well as the bolt stiffness; in particular the extensional stiffness of joint members is represented by the product \( EA \) (with \( E \)=modulus of elasticity, \( A \)=cross sectional area of the members to be connected) and a linear load-slip relationship for fasteners is assumed (\( K \)=slope of load-slip curve of single bolt). The expression of the effective number of bolts then is [8]:

\[
n_{\text{ef}} = \frac{m \cdot (1 - m^{2n})}{(1 + \gamma \cdot m^n) (1 + m) - 1 + m^{2n}} \cdot \frac{1 + \gamma m}{1 - m} \quad (4)
\]

where

\[
\gamma = \min \left( \frac{E_s \cdot A_s}{E_m \cdot A_m}, \frac{E_m \cdot A_m}{E_s \cdot A_s} \right), \quad (5)
\]
m = u - \sqrt{u^2 - 1}, \quad (6)

u = 1 + K \frac{a_1}{2} \left( \frac{1}{E_m \cdot A_m} + \frac{1}{E_s \cdot A_s} \right), \quad (7)

and \( E_m, A_m \), \( E_s, A_s \) are the elastic modulus and the sectional area of main and side members, respectively. For the single bolt timber-to-timber joints, Zahn [8] suggests the stiffness presently included in the NDS (1997) [21]

\[ K = 246 \cdot d^{1.5}, \quad (8) \]

while for steel-to-timber joints the value obtained with Equation (8) has to be multiplied by 1.5. The dimension units for the stiffness \( K \) and the bolt diameter \( d \) are \( N/mm \) and \( mm \), respectively.

Another relationship for the effective number of bolts \( n_{ef} \) has been proposed by Van der Put (1977), which is based on the propagation of splitting crack in front of the bolt and hence it is strongly governed by the tensile resistance of wood perpendicular to the grain \( f_{t,90} \).

\[ n_{ef} = 0.5 \cdot \left[ \frac{1 + 8n \cdot f_{t,90} \cdot (d_k - \alpha)^2}{f_{t,0}} - 1 \right] \leq n. \quad (9) \]

The factor \( \alpha \) takes into account the local wedging effect near the hole and assumes values between 1.0 and 2.0 (local effect has a length between 1.0 and 2.0 \( d \)), the parameter \( a_1 \) represents the loaded end distance and \( f_{t,0} \) is the tensile strength of wood parallel to the grain.

A comparison among the effective number of bolts evaluated with Equations (2)-(4) and (9) was carried out considering a three member steel-to-timber joint in tension with wood side members loaded parallel to the grain. Timber side members were 100 mm wide and 50 mm thick, steel members were 100 mm wide and 10 mm thick, while the bolt diameter \( d \) was 16 mm. The bolt end distance \( a_1 \) and the spacing between fasteners \( a_1 \) were taken equal to 7\( d \). The modulus of elasticity of wood was \( E_t = 13000 \text{ MPa} \) and that of steel was \( E_m = 200,000 \text{ MPa} \). The stiffness \( K \) used in equation (7) was assumed according to Equation (8) \( K = 1.5 \cdot 246 \cdot d^{1.5} \). Better values of \( K \) might be obtained from the theory of the beam on elastic foundation (e.g. [4]). The factor \( \alpha \) was assumed unitary and the ratio \( f_{t,90}/f_{t,0} \) was considered equal to 0.025. In Fig. 1 the bolt effectiveness \((n_{ef}/n)\) versus the real number of bolts \( n \) used in the joint is plotted so to evidence the large differences among the considered relationships. This discrepancy is also due to the limited number of useful experimental results available in the literature for comparing and eventually refining theoretical models. Some results obtained with numerical analysis are available too (e.g. [7, 10]), which consider the nonlinear behavior of joint components, but they cannot consider adequately the randomness of fabrication tolerances as well as the different load-slip relationship of each bolt. Actually construction tolerances cause significant variations of the fastener effectiveness as emphasized in [22] where the differences between cases with and without hole spacing tolerances are similar to those shown in Fig. 1.

**2.2 Joints loaded perpendicular to the grain**

In the last decade various research works were devoted to study the capacity of bolted joints loaded perpendicular to the grain (e.g. [17-19, 23, 24]). These studies pointed out that the capacity of multiple bolted joints rarely is based on the embedding resistance of the single fastener (Equation (1)), as for joints loaded parallel to the grain, but it is normally governed by the tensile stresses in the wood perpendicular to the grain. In particular the capacity depends on the ratio between the distance \( b_e \) of the farthest row of fasteners from the loaded beam edge and the beam depth \( h \), on the distribution of bolts in the joint (number of fastener rows and spacing of bolts) and on the tensile strength of wood perpendicular to the grain as well.

Different relationships for the joint capacity are proposed in the literature. In the CSA 2001 [15] a fictitious shear resistance over the residual cross section is proposed

\[ V_a = \frac{2f_t b_t t}{3} \quad \text{for} \quad b_e > 0.5h \]

(10)

where \( V_a \) is the shear force \((\text{max}(V_1, V_2))\) produced in the member of thickness \( t \) by the fasteners \((V_1 + V_2 = F_m)\) (Fig. 2).

The ENV 1995 [16] proposes a relationship which is derived from the studies of Van der Put [17] and is based...
on the theory of fracture mechanics

\[ V_u = 15.5 \cdot \frac{b_c}{h} \left( b_c + \frac{b_c}{h} \right) \quad (11) \]

with obvious meaning of symbols.

Another relationship based on fracture mechanics is proposed by Larsen and Gustaffson [18]. The maximum capacity of the joint is expressed by the equation

\[ F_m = 2 \eta t \sqrt{b_e GG_f} \quad (12) \]

where \( \eta \) is an efficiency factor, suggested by the authors to be assumed equal to 0.66, and \( \sqrt{GG_f} \) is the apparent fracture parameter.

Ehlbeck and Görlacher [19] proposed a relationship based on experimental and theoretical investigations

\[ F_m = \frac{13}{A_{ef}} f_{1,90} \quad (13) \]

where the factor \( \lambda \) considers that only part of the joint load causes tensile stresses, the factor \( k_r \) takes into account that the joint load is distributed over several rows of fasteners, so that only a reduced portion of tensile stresses is acting in the line of the farthest row of fasteners. The effective area \( A_{ef} \) represents a fictitious area where uniform stresses perpendicular to the grain are considered. Ehlbeck and Görlacher suggest for the above parameters the relationships

\[
\begin{align*}
\lambda &= 1 - 3 \left( \frac{b_c}{h} \right)^2 + 2 \left( \frac{b_c}{h} \right)^3 \\
k_r &= \frac{1}{m} \sum_{i=1}^{m} b_i \\
A_{ef} &= \frac{12 \lambda + c h^2}{2} \\
c &= \frac{4}{3} \sqrt{\frac{b_c}{h} \left( 1 - \frac{b_c}{h} \right)^2}
\end{align*}
\]

where \( m \) is the number of rows of fasteners, \( l_i \) is the distance between the outer bolts in the innermost row and \( h_i \) is the distance of the \( i \)-th row of fasteners from the unloaded edge (Fig. 2).

It can be noted that the first three relationships (Equations (10)-(12)) provide the same capacity for joints with different number and distribution of bolts if they have the same distance \( b_c \) of the innermost row of fasteners from the loaded edge. Differently the relationship of Equation (13) considers also the variation in capacity due to the distribution of bolts in the joint.

The experimental study herein presented aims to put in evidence the behavior of multiple joints with steel main member and high strength bolts without appreciable hole spacing tolerances. Both cases of load acting parallel and perpendicular to the grain are considered.

3. EXPERIMENTAL PROGRAM

The experimental tests concern three-member joints assembled with multiple bolts and with timber members loaded both parallel and perpendicular to the grain. All specimens are made keeping constant the wood specie (Eastern-Alps Red Spruce) and the bolt diameter (16 mm). Timber and steel elements were drilled together (16.5 mm diameter) so to reduce considerably the hole-spacing tolerances.

3.1 Specimen details

The specimens were obtained from glued-laminated beams made with boards 33 mm thick bound together with resorcinol glue. In particular, groups of five specimens were assembled with one to six bolts, for wood side members loaded parallel to the grain, and with one to four bolts for wood side members loaded perpendicular to the grain (Fig. 3). The wood members for parallel to grain tests were 150 mm wide and 50 mm thick whereas, for perpendicular to grain tests, they were 132 mm wide (192 mm for four bolt joints) and 55 mm thick (Fig. 4). The steel member was 150 mm wide and 10 mm thick in the former case, while in the latter a T-shaped steel plate (10 mm thick) was used (Fig. 5).

The specimens with wood members loaded parallel to the grain have a single row of bolts in the loading direction.
The joint groups arranged with one, two, three, four and six bolts are identified with capital letters K, L, M, N and S respectively (Fig. 3a). The fastener end distance and the spacing between bolts were fixed equal to 7\(d\) (112 mm), which are the values recommended by the ENV 1995 [16] to avoid premature splitting failure (Fig. 4a).

The specimens with wood members loaded perpendicular to the grain were arranged with one row of bolts perpendicular to the loading direction, for joints with two or three bolts, whereas they were arranged on two rows, for joints with four fasteners. The groups of joints made with one, two, three and four bolts are identified with capital letters O, P, Q and R, respectively (Fig. 3b). The fastener end distance and the spacing between bolts in the loading direction were fixed to 4\(d\) (64 mm) while the spacing between bolts in the direction perpendicular to loading was fixed to 7\(d\) (112 mm) as recommended in ENV 1995 [16] (Figs. 3b,c).

When the timber members were obtained from the beams, six prism samples (55x55x165 mm) and six thin plate samples (30x145x300 mm) were also made for each group so to determine the main physical and mechanical characteristics of the wood.

Prism samples were used for the evaluation of the compressive strength and the modulus of elasticity, both parallel and perpendicular to the grain. Thin plate samples were used for the evaluation of the embedding strength of wood and the elastic stiffness of wood beneath the fastener (subgrade modulus), either parallel or perpendicular to the grain, adopting the embedding test suggested in the EN 383 [25]. The average values of the results obtained using 16 mm dowel diameter are reported in Table 1.

The mechanical characteristics of the steel joint

<table>
<thead>
<tr>
<th>Joint Type</th>
<th>Specific gravity</th>
<th>Moisture content</th>
<th>Compress. strength</th>
<th>Elastic Modulus</th>
<th>Embedding strength</th>
<th>Subgrade modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>parallel to grain</td>
<td>perpendicular to grain</td>
<td>parallel to grain</td>
<td>perpendicular to grain</td>
</tr>
<tr>
<td>Joints loaded parallel to the grain</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>0.445</td>
<td>10.5</td>
<td>54.77</td>
<td>3.47</td>
<td>13781</td>
<td>290</td>
</tr>
<tr>
<td>L</td>
<td>0.432</td>
<td>9.8</td>
<td>50.62</td>
<td>3.23</td>
<td>14484</td>
<td>258</td>
</tr>
<tr>
<td>M</td>
<td>0.438</td>
<td>8.6</td>
<td>51.19</td>
<td>3.38</td>
<td>14709</td>
<td>273</td>
</tr>
<tr>
<td>N</td>
<td>0.456</td>
<td>9.0</td>
<td>56.93</td>
<td>3.73</td>
<td>14314</td>
<td>352</td>
</tr>
<tr>
<td>S</td>
<td>0.428</td>
<td>11.0</td>
<td>53.25</td>
<td>3.52</td>
<td>13382</td>
<td>322</td>
</tr>
<tr>
<td>Joints loaded perpendicular to the grain</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>O</td>
<td>0.435</td>
<td>10.8</td>
<td>51.11</td>
<td>3.07</td>
<td>13926</td>
<td>272</td>
</tr>
<tr>
<td>P</td>
<td>0.439</td>
<td>10.8</td>
<td>51.36</td>
<td>3.24</td>
<td>13760</td>
<td>249</td>
</tr>
<tr>
<td>Q</td>
<td>0.458</td>
<td>8.8</td>
<td>57.04</td>
<td>3.70</td>
<td>13676</td>
<td>306</td>
</tr>
<tr>
<td>R</td>
<td>0.446</td>
<td>9.4</td>
<td>52.46</td>
<td>3.07</td>
<td>13493</td>
<td>270</td>
</tr>
</tbody>
</table>

Fig. 4 - Geometric details of specimens: (a) joints loaded parallel to the grain, (b,c) joints loaded perpendicular to the grain with single or double row of bolts, respectively.

Fig. 5 - Arrangement of inductive transducers: joints with load parallel (a) and perpendicular (b,c) to the grain.
members and of the bolts were determined through tension tests. The main members of the joints are made with steel Fe 510 (yielding stress \( f_{ys} = 432 \) MPa, tensile strength \( f_{ts} = 548 \) MPa) and the bolts belong to class 8.8 (yielding stress \( f_{ybs} = 826 \) MPa, tensile strength \( f_{tbs} = 908 \) MPa). Two steel washers (64 mm in diameter and 6 mm thick) were used under the nuts of the bolts.

The specimens were assembled just before to be tested and the bolts were tighten by hand. The moisture content of wood elements was surveyed before the start of the test. The span distance between supports for tests perpendicular to the grain was 500 mm.

### 3.2 Test procedure

A displacement controlled loading procedure was applied to the specimens by using a hydraulic actuator mounted on a stiff steel frame. One end of each specimen was fixed to the frame and the other end was gripped to the actuator. The displacement of the actuator was increased at a constant rate of 0.36 mm per minute up to the failure of the specimen.

For joints with the wood loaded parallel to the grain (joint types K, L, M, N and S), two LVDT inductive transducers T1÷T2 (sensitivity 0.002 mm) were used to measure the total slip between joint members. The transducers were arranged as schematically illustrated in Fig. 5a. Four LVDT inductive transducers T1÷T2, T3÷T4 were used to measure the steel-timber slip for the case of load acting perpendicular to the grain (joint types O, P, Q and R). Actually the transducers were fixed to the steel main member and to the timber members just over the line of bolts. The arrangements of transducers in joints O, P, Q and R are illustrated in Figs. 5b,c, respectively.

During each test the value of the corresponding load and the readings of the transducers were surveyed, at time intervals of two seconds, and were simultaneously processed so to have the load-slip curve immediately available.

### 4. TEST RESULTS

Forty-five specimens were tested, with five replications for each of the nine joint types considered. Twenty-five specimens concern joints with the load acting parallel to the timber grain and the bolts are set in a row parallel to the load direction; the other twenty specimens concern joints with the timber element loaded perpendicular to the grain and the bolts are set in one or two rows.

#### 4.1 Joints loaded parallel to the grain

The experimental load-slip curves of joints K are plotted in Fig. 6 while those of joints L, M, N and S are plotted in Fig. 7. These curves show a first branch with an increasing slope, which is due to the initial settlement of bolts in the holes. This branch is in general more pronounced in the joints with multiple fasteners due to unavoidable small construction tolerances, which imply a prolonged settlement phase. The second branch is almost linear up to the proportional limit (\( F_{p}, e_{p} \)); its slope \( K_{t} \) represents the stiffness in service of the connection. The third branch, if any, is rather flat due to the plasticization of the wood beneath the bolt and to the eventual formation of one or more plastic hinges to the shank of the bolt. The second and third branches are joined together by an easement curve, which describes the passage from the elastic to the plastic behavior of the joint.

![Fig. 6 - Load-slip curves for single bolt joints.](image)

![Fig. 7 - Load-slip curves for joints loaded parallel to the grain: (a) two bolts, (b) three bolts, (c) four bolts and (d) six bolts.](image)
The “yield load” $F_y$ is the load where the straight lines approximating the second and the third branch intersect. The ultimate bearing capacity $F_u$ is assumed to be the maximum load reached before the failure of the joint and the ultimate slip $s_u$ is the maximum value measured before the collapse (prEN 12512-2001 [26]).

The curves show a very limited plastic branch for specimens with more than one fastener; in particular some specimens collapsed before reaching the flat branch. In these cases the yielding and the ultimate load were assumed coincident ($F_y = F_u$).

The tests on the specimens with single bolt showed that the commencement of splitting cracks (square marks in Fig. 6) occurs for values of the slip included between 1.3 mm and 3.2 mm. Since it was impossible to see the crack at its first appearance an audible crack was surveyed. The rate of propagation of the crack was different for the specimens tested: the higher is the number of bolts, the faster the crack propagation. Moreover in many cases the collapse of the joint was rather sudden after the crack commencement so that only the failure of the specimen is indicated on the curves (Fig. 7). The failure of all specimens tested (circle marks) occurred by the formation of a splitting crack which join all the bolt-holes and develop up to the loaded end of the wood element, as shown in Fig. 8 for specimen N4. In multiple joints the flexural deformation of bolts after tests was not appreciable.

The main parameters characterizing the joint behavior are summarized in Table 2. In particular average values of the proportional limit load $F_p$, the stiffness in service $K_s$, the yield load $F_y$, the ultimate load $F_u$ and the ultimate slip $s_u$, determined from the five replications of each joint type, are reported. The coefficients of variation of the first three parameters ($F_p$, $K_s$, $F_y$) are also noted.

### Table 2 – Results of specimens tested

<table>
<thead>
<tr>
<th>Joint type</th>
<th>Number of bolts</th>
<th>Proportional limit load $F_p$ [kN]</th>
<th>Stiffness in service $K_s$ [kN/mm]</th>
<th>Yield load $F_y$ [kN]</th>
<th>Ultimate load $F_u$ [kN]</th>
<th>Ultimate slip $s_u$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joints loaded parallel to the grain</td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>1</td>
<td>40.42</td>
<td>0.112</td>
<td>58.41</td>
<td>0.077</td>
<td>55.41</td>
</tr>
<tr>
<td>L</td>
<td>2</td>
<td>70.40</td>
<td>0.090</td>
<td>111.10</td>
<td>0.087</td>
<td>101.11</td>
</tr>
<tr>
<td>M</td>
<td>3</td>
<td>106.04</td>
<td>0.101</td>
<td>131.30</td>
<td>0.253</td>
<td>147.85</td>
</tr>
<tr>
<td>N</td>
<td>4</td>
<td>134.02</td>
<td>0.080</td>
<td>131.33</td>
<td>0.175</td>
<td>175.34</td>
</tr>
<tr>
<td>S</td>
<td>6</td>
<td>181.77</td>
<td>0.067</td>
<td>146.61</td>
<td>0.152</td>
<td>245.87</td>
</tr>
<tr>
<td>Joints loaded perpendicular to the grain</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>O</td>
<td>1</td>
<td>19.11</td>
<td>0.118</td>
<td>24.12</td>
<td>0.069</td>
<td>25.84</td>
</tr>
<tr>
<td>P</td>
<td>2</td>
<td>33.00</td>
<td>0.147</td>
<td>55.02</td>
<td>0.101</td>
<td>39.72</td>
</tr>
<tr>
<td>Q</td>
<td>3</td>
<td>42.92</td>
<td>0.117</td>
<td>81.43</td>
<td>0.085</td>
<td>55.89</td>
</tr>
<tr>
<td>R</td>
<td>2x2</td>
<td>64.32</td>
<td>0.109</td>
<td>78.70</td>
<td>0.145</td>
<td>77.06</td>
</tr>
</tbody>
</table>

Fig. 8 - Failure of specimen N4.

Fig. 9 - Load-slip curves for joints loaded perpendicular to the grain: (a) single bolt, (b) two bolts, (c) three bolts and (d) four bolts.
The experimental load-slip curves of joint types O, P, Q and R are plotted in Fig. 9. The curves are fairly different of those of the joints loaded parallel to the grain. In fact, also the curves of joint type O and R (Fig. 9a,b), which show three branches as those of joints loaded parallel to the grain, have the third branch characterized by a series of small drops in load capacity due to the propagation of splitting cracks. Moreover the third branch of the curves of joint type R is very small. On the contrary, the curves of joint types P and Q (Fig. 9b,c) do not have any flat branch; the specimens collapsed suddenly by splitting for a value of the load slightly higher than the proportional limit. The first branch of the curves, which represents the initial settlement of the bolts in the holes, is rather limited for all specimens tested.

An audible commencement of splitting cracks was noted in all specimens with single bolt for a value of the load quite close to the proportional limit (square marks in Fig. 9a). At the first drop of resistance evidenced in the curves, the splitting crack became visible. For joints with two or three bolts (joint types P, Q), the splitting crack became immediately visible after the first audible crack.

The “yield load” $F_y$ was assumed as the load at which the first drop in resistance occurs, due to partial or global splitting of wood members. The collapse of all specimens tested (circle marks in Fig. 9) occurred by the formation of a splitting crack that reaches the edge of the wood members. The flexural deformation of bolts after tests was almost negligible.

The joint type R did not collapse suddenly at the appearance of the splitting crack because load redistribution occurred between the two transversal rows of bolts. In all specimens the splitting crack formed first in the farthest row of bolts from loaded edge, as shown in Fig. 10.

In Table 2 the average values of the proportional limit load $F_p$, the “yield load” $F_y$, the stiffness in service $K_s$, the ultimate load $F_u$ and the ultimate slip $s_u$ are reported. The coefficient of variation of the first three parameters is also reported. The four joint types showed quite similar values of the coefficients of variation, which means that the repeatability of the results seems to be not significantly influenced by the number of bolts.

The ultimate load $F_u$ is equal to the “yield load” $F_y$ for specimens of joint types P and Q. In these cases the ultimate slip $s_u$ is almost always lower than 1.0 mm. The average ultimate slip $s_u$ of joint type R is equal to 2.34 mm while that of joint type O is equal to 3.93 mm.

### 5. COMPARISON OF THE RESULTS WITH ANALYTICAL PREDICTIONS

As stated in section 2, various relationships for the effective number of bolts of a multiple fastener joint subjected to parallel to grain load are available in the literature. A comparison among the predictions of these relationships with the experimental results is summarized in Table 3. In particular in the third, fourth and fifth columns is reported the experimental bolt effectiveness, obtained as the ratio between the capacity of multiple fastener joints and that of the single bolt joint multiplied by the number of bolts ($F_u/nF_u$), on the basis of the proportional limit, the yield load and the ultimate load, respectively. In the other four columns the predictions obtained with Equations (2), (3), (4) and (9) divided by the actual number of bolts ($n_e/n$) are indicated. In these equations the actual geometric parameters of tested joints were used; moreover in Equation (4) the modulus of elasticity of steel was assumed equal to 200 GPa, the modulus of elasticity of wood was that determined by tests (Table 1) and the single joint stiffness

**Table 3 - Comparison between experimental and analytical values of bolt effectiveness for joints loaded parallel to the grain**

<table>
<thead>
<tr>
<th>Joint type</th>
<th>Number of bolts</th>
<th>Proportional limit load</th>
<th>Yield load</th>
<th>Ultimate load</th>
<th>Analytical predictions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$F_p$</td>
<td>$F_y$</td>
<td>$F_u$</td>
<td>$s_u$</td>
<td>Eqn. (2)</td>
</tr>
<tr>
<td>L</td>
<td>2</td>
<td>0.87</td>
<td>0.91</td>
<td>0.91</td>
<td>0.80</td>
</tr>
<tr>
<td>M</td>
<td>3</td>
<td>0.87</td>
<td>0.89</td>
<td>0.85</td>
<td>0.77</td>
</tr>
<tr>
<td>N</td>
<td>4</td>
<td>0.83</td>
<td>0.79</td>
<td>0.76</td>
<td>0.75</td>
</tr>
<tr>
<td>S</td>
<td>6</td>
<td>0.75</td>
<td>0.74</td>
<td>0.70</td>
<td>0.72</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Joints loaded parallel to the grain</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
</tr>
<tr>
<td>M</td>
</tr>
<tr>
<td>N</td>
</tr>
<tr>
<td>S</td>
</tr>
</tbody>
</table>
K was obtained from Equation (8). In Equation (9) the ratio between the tensile strength of wood perpendicular and parallel to the grain \( f_{v0}/f_{v90} \) was assumed equal to 0.025.

As can be noted in Table 3, the joints studied with the load parallel to the timber grain evidenced a reduction in bolt effectiveness even in the case of two bolts. This is mainly due to the different extensional stiffnesses of joint members that cause an unequal distribution of the load between the bolts; in addition unavoidable construction tolerances emphasize this trend.

The analytical predictions obtained with Equation (4) (NDS 1997 [21]) overestimate the experimental results while those obtained with Equation (2) (ENV 1995 [16]), Equation (3) (CSA 2001 [15]) and Equation (9) (proposed by Von der Put [20]) provide a lower effectiveness of bolts than that obtained from tests. The relationship proposed by ENV 1995 [16], for the cases studied, provided predictions quite close to the experimental results. Test results show that not appreciable differences are obtained between the bolt efficiency evaluated at proportional limit load, yield load and ultimate load.

### Table 4 - Comparison between experimental and analytical values of the capacity of joints loaded perpendicular to the grain

<table>
<thead>
<tr>
<th>Joint type</th>
<th>Number of bolts</th>
<th>Experimental Yield load [kN]</th>
<th>Eqn. (10) Yield load [kN]</th>
<th>Eqn. (11) Yield load [kN]</th>
<th>Eqn. (12) Yield load [kN]</th>
<th>Eqn. (13) Yield load [kN]</th>
<th>n ( F_s ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>O</td>
<td>1</td>
<td>25.84</td>
<td>43.85</td>
<td>38.88</td>
<td>19.87</td>
<td>12.53</td>
<td>22.70</td>
</tr>
<tr>
<td>P</td>
<td>2</td>
<td>39.72</td>
<td>43.85</td>
<td>38.88</td>
<td>19.87</td>
<td>28.32</td>
<td>45.41</td>
</tr>
<tr>
<td>Q</td>
<td>3</td>
<td>55.89</td>
<td>43.85</td>
<td>38.88</td>
<td>19.87</td>
<td>47.32</td>
<td>68.11</td>
</tr>
<tr>
<td>R</td>
<td>2x2</td>
<td>77.06</td>
<td>86.36</td>
<td>66.82</td>
<td>27.88</td>
<td>86.75</td>
<td>90.82</td>
</tr>
</tbody>
</table>

The yield load of joints loaded perpendicular to the grain and the predictions proposed by Equations (10), (11), (12) and (13) are reported in Table 4. In these equations the actual geometric parameters of tested joints were used; moreover in Equation (10) the shear strength of wood \( f_v \) was assumed equal to 4.6 Mpa (average value of some shear tests), in Equation (12) the apparent fracture parameter \( G_f \) was assumed equal to 12 N/mm² and in Equation (13) the tensile strength of wood perpendicular to the grain \( f_{v90} \) is assumed equal to 0.55 MPa. The value \( f_{v90} \), which is based on a uniformly stressed volume \( V_o = 0.01 \text{ m}^3 \), was derived applying the relationship [16]

\[
f_{v90} = f_{v90} \left( \frac{V}{V_o} \right)^{0.2}
\]

(14)

to the average tensile stress value obtained from some tests carried out on specimens with a volume \( V = 0.0015 \text{ m}^3 \) \( (f_{v90} = 0.81 \text{ MPa}) \). As a reference, in the last column of Table 4 the value of the capacity of single bolt joint, evaluated using the EYM (European Yield Model [16]), multiplied by the number of bolts is also reported.

The Table evidences significant differences among various predictions and a significant scatter with experimental results. In particular, for the joints studied, the prediction of CSA 2001 [15] (Equation (10)) overestimates the capacity in all cases except joints type Q, the prediction of ENV 1995 [16] (Equation (11)) underestimates the capacity of joints with more than one bolt, the prediction proposed by Larsen and Gustafsson [18] (Equation (12)) gives always very low capacities while the prediction proposed by Ehlbeck and Görlicher [19] (Equation (13)) provides conservative capacities for joint types O, P, Q and overestimates the capacity of joint R.

### 6. CONCLUSIONS

The results presented in this study concern an experimental investigation carried out on three member steel-to-timber joints arranged with multiple high strength bolts. The specimens were made with steel main member and wood side members; some of them were loaded parallel to the grain and the others were loaded perpendicular to the grain. On the basis of the results obtained in the study the following concluding remarks may be drawn.

1. The load-slip curves of the joints studied evidenced a considerable reduction in ductility as the number of bolts increases even though the recommended spacing and end distances of the bolts are adopted. In particular, the curves of multiple-bolt joints \( (n > 1) \) loaded perpendicular to the timber grain do not have any plastic branch.
2. The repeatability of the results is in general fairly good even though an appreciable increase of the scatter was noted, for joints loaded parallel to the timber grain, as the number of bolts grows.
3. The effective number of bolts \( n_e \), for joints loaded parallel to the grain, resulted lower than the number of bolts used \( n \) even for two-bolt joints. Not appreciable differences were noted between the effective number of bolts evaluated on the basis of the proportional limit load or on the basis of the yield load.
4. The analytical prediction of Equation (4) (NDS 1997 [21]) overestimates the values of the effective number of bolts for the joints loaded parallel to the grain. On the contrary, excessively conservative values were obtained with Equation (3) (CSA 2001 [15]) and Equation (6) (Van der Put [20]). Good results were obtained using Equation (2) (ENV 1995 [16]).
5. The capacities estimated by the current predictions for joints loaded perpendicular to the grain are in general considerably different to the values obtained from the experimental tests. In most cases they provide joint capacities excessively lower than the experimental ones but in some cases they overestimate the test results, so that some more theoretical and experimental research is needed to refine the proposed analytical models.
6. Provided that the joints were built with care so to avoid appreciable hole spacing tolerances, analytical predictions
have to consider that in practice lower values for the joint capacity may be obtained.

The conclusions are drawn on the basis of the experimental tests carried out, so that further tests need to be performed with different specimen characteristics to extend them to a general validity.

ACKNOWLEDGEMENTS

The writers wish to thank the glued-laminated factory Stratex s.r.l. of Sutrio-Udine (Italy) for the kind offer of the material used in the tests. The financial support of the Italian Ministry of Education, University and Research (M.I.U.R.) is gratefully acknowledged.

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Paper received: February 4, 2003; Paper accepted: March 19, 2003