Materials and Structures
DOI 10.1617/s11527-006-9156-2

ORIGINAL ARTICLE

Numerical analyses for the prediction of the splitting strength of beams loaded perpendicular-to-grain by dowel-type connections

Marco Ballerini · Mirko Rizzi

Received: 11 June 2005 / Accepted: 10 March 2006

Abstract The paper presents a parametric numerical study on the splitting strength of timber beams loaded perpendicular-to-grain by dowel-type connections. The main aims of the numerical investigations are: (1) find out the influence of main connection parameters on the splitting strength of beams; (2) compare the above evaluated influences with the ones proposed by the first author in a recently developed semi-empirical prediction formula. The first part of the paper presents the mentioned new semi-empirical prediction formula which has been developed by means of a survey on experimental data from literature. The formula is presented in its main aspects and later its prediction capability is discussed and compared with the ones of formulae embodied in new European and German design codes for timber structures. The second part of the paper reports the main results of parametric numerical analyses carried out in the framework of Linear Elastic Fracture Mechanics (LEFM) by means of a crack propagation approach. The analyses are performed on beams of different size loaded at mid-span by both single and multiple dowel connections. The main investigated parameters are the connection width (l_r), the connection depth (h_m), and the number of rows of fasteners (n). They are analysed for different beam heights (h) and for different distances of the most distant row of fasteners from beam loaded edge (h_e). The numerical results are compared with available experimental test data and with the relationships embodied in the above-mentioned semi-empirical prediction formula.

M. Ballerini (✉) · M. Rizzi
Faculty of Engineering, University of Trento,
Via Mesiano 77, 38050 Trento, Italy
e-mail: marco.ballerini@ing.unitn.it

Résumé *L'article présente les résultats d'une étude numérique paramétrique qui analyse la résistance à la fissuration de poutres en bois chargées perpendiculairement aux fibres de connexions, avec des connecteurs cylindriques. Le but principal de cette étude numérique est: (1) déterminer l'influence des paramètres principaux des connexions sur la résistance à fissuration des poutres; (2) comparer les résultats obtenus avec les résultats proposés par Ballerini dans une formule récente de prédiction semi-empirique. La première partie de l'article présente la formula citée de prédiction semi-empirique développée sur base d'une analyse des données expérimentales disponibles dans le texte. La formule est illustrée dans ses aspects principaux et par la suite sa capacité prévisible est comparée avec celle des formules adoptées par les récentes normes européennes et allemandes pour les structures en bois. La seconde partie de l'article reprend les principaux* *résultats de l'analyse numérique paramétrique développés dans le domaine de la Mécanique de la Fracture Linéaire Elastique (LEFM) à l'aide d'analyses avec propagation de fissure. Les analyses concernent des poutres de différentes dimensions chargées en ligne de connexions avec un ou plusieurs connecteurs cylindriques. Les paramètres principaux étudiés sont la largeur (l_r), la hauteur (h_m) et le nombre de lignes des connecteurs (n) de la connexion. Les analyses concernaient des poutres de différentes hauteurs (h) et placés à différentes distances par rapport au bord des poutres de la ligne des connecteurs plus éloignée (h_e). Les résultats numériques sont comparés avec les données expérimentales disponibles et les études prévues par la formule citée de prédiction semi-empirique.*

Keywords Timber engineering · Splitting strength · LEFM numerical analyses · Crack propagation

1 Introduction

The design of connections which transfer forces to timber elements in direction perpendicular-to-grain is commonly carried out with reference to the strength of connections and with little care of the splitting strength of timber beams. In spite of this, the formation and the propagation of a crack along the grain of timber elements is a possible failure mechanism which can lead to ultimate loads considerably lower than the ones of connections.

This is particularly true when the distance from the beam loaded edge of the most distant row of fasteners h_e is small compared to the beam height h.

Due to this reason, connections which transfer forces perpendicular-to-grain should be placed either far from the beam loaded edge or properly reinforced.

The prediction of the splitting strength of timber beams is a difficult task since it is influenced by a large number of parameters: the height h and the thickness b of beams and, in case of dowel-type connections, the distance of the furthest row of fasteners from beam loaded edge h_e, the connection height h_m, the connection width l_r, the number of rows n and the number of columns m of fasteners (see Fig. 1).

The first studies on the splitting strength of beams loaded perpendicular-to-grain by dowel-type connections were carried out by Möhler and Lautenschläger [1], Möhler and Siebert [2] and Ehlbeck and Görlacher [3]. In these experimental researches, a total amount of about 140 tests were performed on timber and glulam beams, with different size and different connection fasteners (nails, dowels, ring connectors), mostly in a simply supported beam configuration with the connection at mid-span.

The above experimental results have been the basis of the prediction formula developed by Ehlbeck et al. [4]. The formula is based on both empirical and theoretical considerations. It assumes a non-linear influence of all the following parameters: the beam dimensions b and h (according to both experimental results and Weibull failure theory), the loaded edge distance (h_e or $\alpha = h_e/h$), the joints geometry.

This formula, with some little changes essentially due to simplification purposes, is actually embodied in the new German design standard for timber structures: DIN 1052 [5].

Afterward, on the basis of an energetic approach in the framework of the Linear Elastic Fracture Mechanics (LEFM), Van der Put developed a different theoretical prediction formula [6, 7]. This formula, calibrated on the same experimental data set of the previous one, is

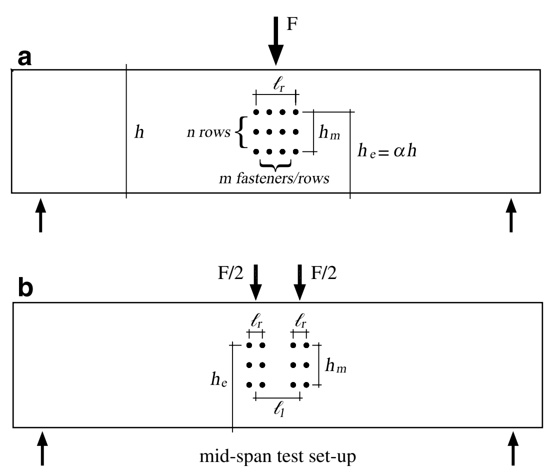

Fig. 1 Parameters of the connection geometry

assumed as basis for design in the new European code for timber structures: EN 1995-1-1 [8]. In contrast to previous formula, it assumes a linear effect of the beam thickness, a different influence of the loaded edge distance α and no effect of connection geometry.

After that, several experimental and numerical studies were carried out to investigate in detail the effect of some parameters. The most relevant experimental studies are those carried out by Yasumura et al. [9, 10], Reske et al. [11], Kasim and Quenneville [12], Ballerini et al. [13, 14].

Concerning the numerical researches, those performed by Borth and Rautenstrauch [15], Ballerini and Bezzi [16] and Acler [17], should be mentioned.

Finally in 2004, Ballerini [18] presented a new semi-empirical prediction formula based on all the experimental data available from literature, and on the main results of theoretical and numerical works. The formula assumes the soundness of the Van der Put LEFM energetic approach but takes also into account the effect of connection geometry.

The paper presents the results of a parametric numerical investigation on the splitting strength of timber beams loaded perpendicular-to-grain by dowel-type connections.

The most important aim of the study is the evaluation of the influence of main connection parameters on beam splitting strength. A secondary goal is the verification of the consistency of effects proposed by Ballerini in his semi-empirical formula. These effects will be checked on the basis of the numerical results and, when possible, also with experimental data.

The first part of the paper presents the Ballerini's prediction formula; the basic assumptions and its derivation are shortly outlined in their main aspects. After that its prediction ability is illustrated and compared with respect to those of formulae embodied in new European and German design codes for timber structures and with test data.

The second part of the paper shows the main results of the parametric study performed with the commercial FE program ANSYS 8.0. The failure loads for different crack lengths, are derived by means of the Wu's fracture criterion on the basis of the stress intensity factors (SIFs) at the crack tip for unit load in modes I and II.

Initially, the results of beams loaded by single-dowel connections are illustrated. Successively, the outcomes of the analyses on beams loaded by 1 row of 2 fasteners with different spacing l_r, and the ones on beams loaded by more rows of 1 dowel with different total connection height h_m, and also different number n of fasteners, are shown. The numerical results are compared with the values of the prediction formula and, when possible, with the experimental data.

2 The prediction formula

The prediction formula presented in this chapter is based on the experimental data available from references [1–3, 9–14]. A total amount of 628 test results on beams loaded by dowel-type connections at mid-span or at one end have been considered. The specimens were simply-supported beams or cantilevered beams.

In the test data set, the beam parameters (b, h) and the ones of the joint geometry (α, h_m, l_r, l_1, n and m—see Fig. 1), range among the following values: b = 40–200 mm, h = 88–1200 mm, α = 0.1–0.75, h_m = 0–90 mm (some data with 200 and 500 mm), l_r = 0–200 mm (some data 320, 420, 700 mm), l_1 = 0–240 mm (some data up to 784 mm), n = 1–6, m = 1–4 (some data 5, 10).

The greater part of the physical studies had been performed on beams loaded by dowel or bolted joints (10–30 mm). Nevertheless, some investigations had used nailed joints (4, 6 mm) and ring connectors (Appel ring, 65 mm in diameter).

Test data have shown that tests characterized by α values up to 0.7 fail by splitting. Cracks occur essentially in line with the most distant row of fasteners from beams loaded edge. With respect to the splitting strength, the following main outcomes can be driven: (1) it is proportional to the beam thickness; (2) it is not influenced linearly by the beam height; (3) it depends greatly by the distance from the loaded edge of the furthest row of fasteners; (4) it is considerably influenced by the other joint parameters (h_m, l_r, n and m); (5) when joints are made with different clusters of fasteners it is also influenced by the distance between clusters l_1; (6) it is not influenced by type and size of fasteners and also by beam slenderness (L/h).

On the basis of the above main results, a prediction formula with the following structure have been looked for:

$$F_{\mathrm{pre}} = F_1(b, h_e, h) \cdot f_w(l_r, l_1, m) \cdot f_r(h_m, n) \tag{1}$$

The first term represents the splitting failure load of beams loaded by single-dowel connections; the second one takes into account the effect of the connection width or of its related parameters; the last one is correlated with the connection height.

2.1 The splitting strength of beams loaded by single-dowel connections

In order to derive the first term of the prediction formula the data of specimens loaded by single-dowel connections were considered. Due to the observed linear influence of the beam thickness on the failure load, the prediction formula of Van der Put [6, 7] was taken into consideration for the term $F_1(b, h, h_e)$. However, as reported in [18], this formula is prone to underestimate the strength of specimens characterized by lower α values and to overestimate the one of specimens with higher α values.

To increase the prediction ability for the splitting strength of beams loaded by single-dowel connections, a guided best fitting of test data has been carried out. As a consequence, the following formula has been obtained:

$$F_1 = 2b \cdot k \cdot \sqrt{\frac{h_e}{1 - \alpha^3}} \tag{2}$$

In Eq. (2), the constant value k can be assumed equal to 14 N/mm$^{1.5}$ on the basis of a calibration on the mean data of the experimental investigations of Ballerini [13, 14].

The prediction ability of Eq. (2) on the whole database of test data with single-dowel connections is shown in Fig. 2.

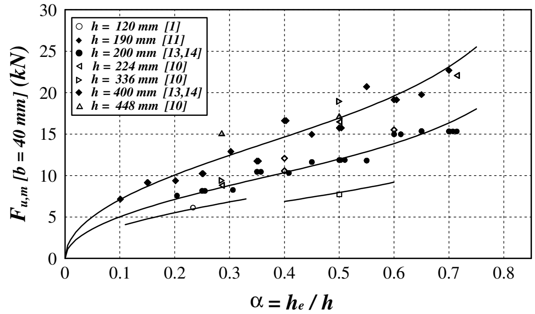

Fig. 2 Prediction capability for beams loaded by single-dowel connections

2.2 The influence of connection width

To obtain the influence of connections width l_r and of the distance between clusters of fasteners l_1, the data of tests with only 1 row of fasteners have been considered.

These data concern tests on simply supported beams with the connection at mid-span and tests with the connection at one end of simply supported or cantilevered beams: in this case l_1 is twice the distance between the centre of the joint and the end of the beam.

In order to find out the effect of different width parameters (l_r, l_1, m) of connections, the average results of test data were divided by the predicted strength of specimens with single-dowel connections (F_1) and plotted versus different non-dimensional parameters. The one which has been found to be better correlated with average test data is $(l_r+l_1)/h$ and its relationship is shown in Fig. 3.

Despite the evident scatter of the average test data, from Fig. 3 it is possible to detect a quite linear effect of the non-dimensional "*total width*" of the connection $(l_r+l_1)/h$ up to a value of about

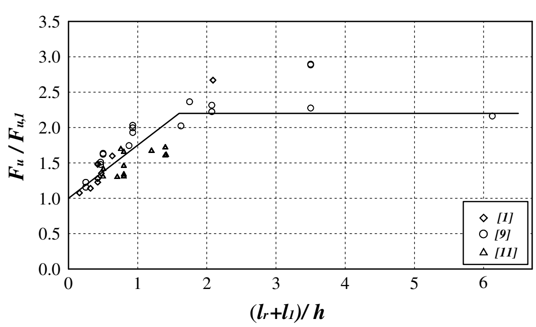

Fig. 3 The effect of connection width

1.6. At this value, the strength is more than twice the one of beams with single-dowel connections. Over this value, very few data characterized by large scatter are available. In any case the effect of connection width parameters over the value of 1.6 seems to be quite negligible.

The effect of connection width parameters can be accounted for by means of the following simple corrective function which is plotted in Fig. 3:

$$f_{\mathrm{w}} = 1 + 0.75 \cdot \frac{l_{\mathrm{r}} + l_1}{h} \leq 2.2 \tag{3}$$

2.3 The influence of connection depth

To investigate the influence of connection depth parameters (h_{m} and n) the test data not already taken into account were considered. As done above for the influence of connection width, the average experimental data of each specimen configuration have been divided by the predicted strength for specimens with 1 row of fasteners: $f_{\mathrm{w}}F_1$. The resulting data set was plotted versus different parameters in order to find out the one with better correlation.

Different non dimensional parameters have been taken into account to assess the effect of connection depth.

In accordance with findings of the experimental tests, they were searched considering the positive effect of the connection depth h_{m} and of the number of rows n, and the negative effect of the beam height h, the loaded edge distance h_{e} or the unloaded edge distance h_1.

Although more complex combinations of n, h_{m}, h, h_{e} and h_1 were considered, the parameter which has shown the best correlation with average test data was found to be n ohm/K. Its correlation with the data set derived from the average strengths is reported in Fig. 4.

From the graph it is evident that the correlation it is not very good. Indeed, a large scatter can be detected especially for values of n ohm/K lesser than 0.25 and very few data are available for values greater than 1. Moreover, the derived data set takes into account the approximations coming from the influence of connection width.

Despite the mentioned limits of derived data set, the correlation between the data set and the

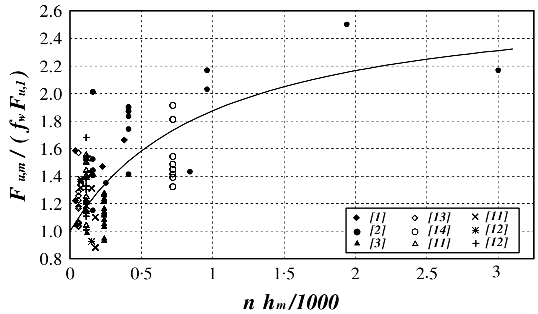

Fig. 4 The effect of connection depth

parameter n ohm/K can be quite well appreciated by means of the following corrective function:

$$f_{\mathrm{r}} = 1 + 1.75 \frac{\chi}{1 + \chi} \quad \text{with} \quad \chi = \frac{n \cdot h_{\mathrm{m}}}{1000} \tag{4}$$

The corrective function f_{r} is plotted in solid line in Fig. 4.

2.4 Global comparison and design formula

In this chapter, the prediction ability of the semi-empirical model is compared with the experimental data and with the design formulae provided by new German and European design codes for timber structures [5, 8]. In addition, a design formula is derived and compared with test data.

The comparison of the semi-empirical prediction formula with experimental data is summarized in Figs. 5 and 6.

Figure 5 shows the prediction ability with respect to the average failure loads; from the

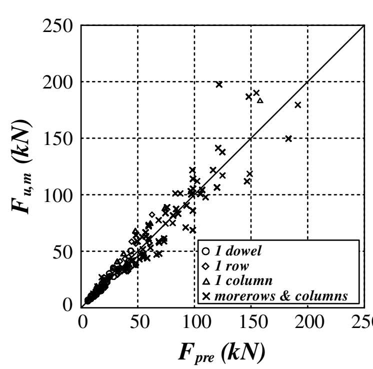

Fig. 5 Predicted versus average splitting failure loads

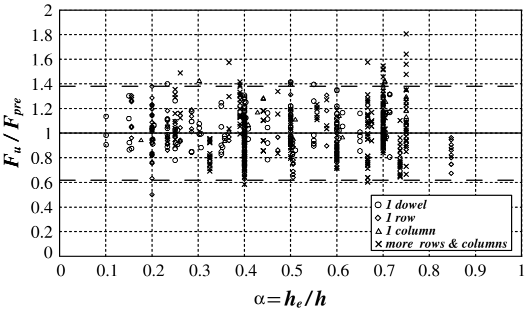

Fig. 6 Ratios F_u/F_{pre} versus α

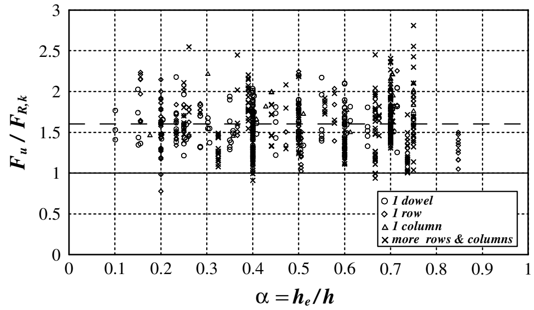

Fig. 7 Ratios $F_u/F_{R,k}$ versus α

graph it is easy to appreciate the good prediction ability on the whole load range.

Figure 6 reports the ratios between the experimental failure loads and the predicted values versus α; from the graph it is possible to notice no residual effect of α and that most data are in between the characteristic values which are respectively 1.38 and 0.62 times the average value.

The comparison with the ability prediction of design formulae embodied in new German and European design codes is summarized in Table 1. For an effective comparison, the design formulae need to be calibrated. As a result of the calibration procedure, a value of 0.89 MPa for the average tension perpendicular-to-grain of timber $f_{t,90,m}$ (used by the formula embodied in DIN 1052), and an average k value of 16.6 N/mm$^{1.5}$ (used by the formula reported in EN 1995-1-1), were found. These values are about 1.8 and 2.4 times the respective characteristic values ($f_{t,90,k}$ = 0.5 MPa; k_k = 7 N/mm$^{1.5}$, [5, 8]).

From Table 1 it can be noted that proposed prediction formula has substantially the same prediction ability of the one embodied in new DIN 1052. On the contrary, the design formula of EC5 is less reliable since it does not take into account the effect of joint geometry.

Table 1 Statistical parameters of ratios F_u/F_{pre} for different prediction models

	DIN 1052: 2004	EN 1995-1-1: 2004	Proposed model
Mean	1.00	1.00	1.03
S.d.	0.20	0.29	0.19
Max	1.60	2.55	1.81
Min	0.47	0.46	0.50

From the semi-empirical prediction formula a simple design formula for the characteristic splitting strength of beams loaded by multiple-dowel connections can be derived:

$$F_{R,k} = 2b \cdot 9 \cdot \sqrt{\frac{h_e}{1-\alpha^3}} \cdot f_w \cdot f_r \tag{5}$$

$$f_w = 1 + 0.75 \cdot \frac{l_r + l_1}{h} \leq 2.0 \tag{6}$$

$$f_r = 1 + 1.75 \frac{n \cdot h_m/1000}{1 + n \cdot h_m/1000} \tag{7}$$

In Eq. (6) the maximum strength increase due to the connection width is limited to 2; it was 2.2 in Eq. (3).

With reference to Eq. (3), the change is due to the physical consideration that connections characterized by large values of the "*total width*" are usually made with clusters of fasteners which act quite independently on the beam. As a consequence, the total splitting strength cannot be larger than twice the splitting strength computed for each cluster of fasteners. Moreover, since there are only very few tests with joints characterized by large values of $(l_r + l_1)/h$, this change has a very limited effect on the prediction ability. The comparison of this design formula with test data is outlined in Fig. 7.

3 Parametric numerical analyses

The numerical analyses were performed by means of the FE program ANSYS 8.0 in the framework of LEFM.

For each specimen the crack propagation loads were derived for different crack lengths by means of the Wu's fracture criterion [19], which states that the load necessary to propagate the crack can be computed as follows:

$$\frac{k_{\mathrm{I}} \cdot F}{K_{\mathrm{IC}}} + \left(\frac{k_{\mathrm{II}} \cdot F}{K_{\mathrm{IIC}}}\right)^2 = 1 \tag{8}$$

In previous formula, k_{I} and k_{II} represent respectively the SIFs in mode I and mode II for unit load; they generally are influenced by the loaded edge distance (h_e or $\alpha = h_e/h$), by the crack length a, and by the joint geometry; the terms K_{IC} and K_{IIC} are respectively the fracture toughness in mode I and mode II.

The timber was modeled as an orthotropic material (E_x = 11,000 MPa, E_y = 890 MPa, G_{xy} = 760 MPa, ν_{xy} = 0.37) in plane stress conditions and cracks were assumed to be in line with the most distant row of fasteners from the beam loaded edge.

Far away from crack tips, rectangular CPS8 and triangular CPS6 elements (continuous plane stress 8 or 6-nodes elements) with size ranging from 0.5 to 10 mm were used. The crack tips were modelled with 16 collapsed CPS8 quarter point elements. At the connection, contact surfaces CONTACT 48 were employed between the beam elements and the dowel ones. Typical mesh details at crack tips and at the dowel/beam contact are shown in Fig. 8.

Since ANSYS 8.0 does not directly provide the SIFs for orthotropic materials, in order to evaluate the SIFs for unit load (k_{I} and k_{II}), the crack opening displacement method (COD) developed by Chen et al. in [20] and recently put forward by Guinea et al. in [21] was used. This method allows the calculation of SIFs by means of the horizontal and vertical displacements of the nodes of the quarter point elements on the crack surface nearest the crack tip.

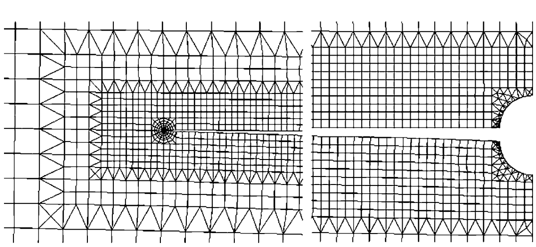

Fig. 8 Mesh details at crack tip (left) and at dowel/beam contact

The splitting failure loads were obtained by means of crack propagation analyses. To this aim, a procedure able to compute the propagation load for each crack configuration and to generate the mesh for next analysis was developed by means of the internal language APDL.

3.1 Analyses on beams with single-dowel connections

The analyses on beams loaded by single-dowel joints were performed only for comparison purpose. Indeed, the splitting strength of 200 and 400 mm high beams had been already investigated in [16] and [17] with the FE program ABAQUS 6.1. In the current research, only 200 mm high beams with values of α ranging from 0.2 to 0.8 were considered.

The SIFs and the derived propagation loads F (both functions of α and a) computed with both FE programs are in good agreement. The related diagrams are not reported here due to lack of space.

The results of these analyses are summarized in Fig. 9 where the experimental, numerical and predicted failure loads are plotted versus the non-dimensional parameter α.

The numerical values were obtained with a calibration of K_{IC} and assuming a $K_{\mathrm{IIC}}/K_{\mathrm{IC}}$ ratio equal to 3.

The calibration was performed on the experimental data with α lower than 0.5; the reason is that such data are more reliable than those with larger α values since usually no plastic deforma-

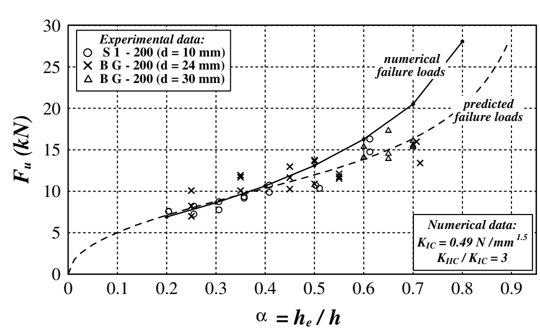

Fig. 9 Experimental, numerical and predicted splitting loads of 200 mm high beams with single-dowel connections

tions can be detected beneath fasteners in those tests. The calibration provided a value of K_{IC} of 0.49 $N/mm^{1.5}$; this value and also the assumed value of K_{IIC}/K_{IC} are in line with toughness data reported in literature [22–24].

Figure 9 shows a good agreement between test data and both numerical/predicted values for α ratios lower than 0.5; for larger α values, it is apparent that the numerical curve overestimates test data.

3.2 Analyses on beams with connections made with 1 row of 2 fasteners

The analyses on specimens with joints made with 1 row of 2 fasteners in line with grain direction, were carried out to obtain the influence of joint width l_r on the splitting strength of beams. In the semi-empirical prediction formula, this influence is taken into account by means of the corrective factor f_w.

The parametric investigation was performed on 2 beam sizes (120 and 240 mm), 3 different α values (0.23, 0.47, 0.7) and 7 values of the connection width l_r (ranging from 0 up to 304 mm). The beam size of 120 mm and the connection width investigated, were selected to compare the numerical results with test data of series C of Möhler and Lautenschläger research [1].

The numerical crack propagation loads versus length a, are summarized in Fig. 10. The length a is the distance from mid-span of the crack tip nearest to the end of the beam; when the crack between the two fasteners is not completely

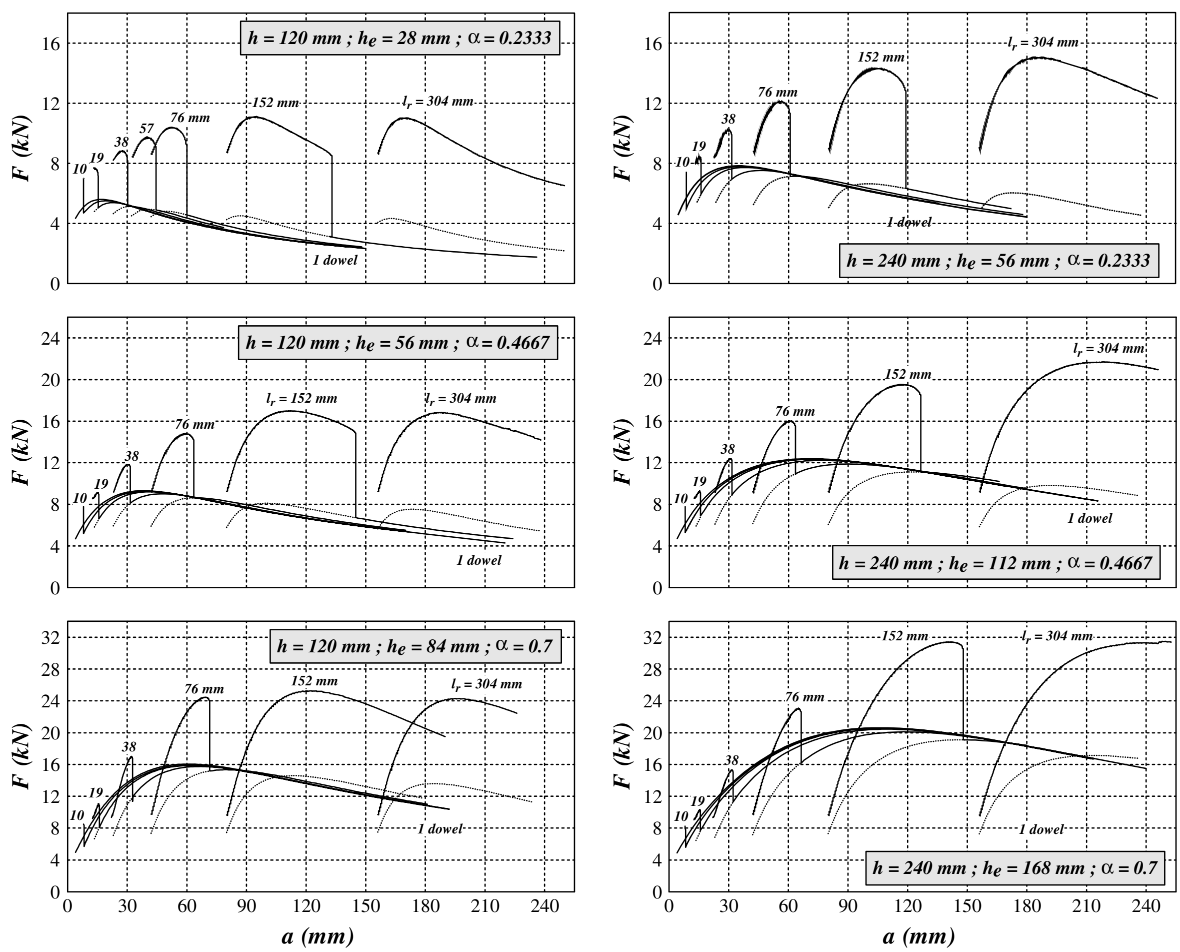

Fig. 10 Failure splitting loads of beams with 2 different heights loaded at mid-span by 2 fasteners with different spacing l_r and at 2 different loaded edge distance h_e versus length a

developed it represents the classical half-crack length.

All curves plotted in graphs of Fig. 10, are the result of the above-mentioned automated crack propagation procedure. Each of them is the result of about 100 numerical analyses.

From graphs of Fig. 10 it is possible to note the great effect of connection width l_r on beam splitting strength; indeed, a great increase in strength is evident up to the complete opening of the crack between the fasteners. After this, the propagation loads severely decrease up to about the values of beams loaded with single-dowel connections.

The maximum splitting loads, which correspond to the beginning of the instable crack propagation, are bigger than the ones of single-dowel connections for large connection widths l_r. On the contrary, the strength increase is very limited or even negligible for small joint widths especially in case of high beams or high α values.

The above considerations are evident in graph of Fig. 11 where the strength increase ($f_w = F_u/F_{u,1}$) is plotted with respect to the ratio l_r/h. From the graph it is possible to note that the maximum strength increase is of about 100% for $\alpha = 0.233$, of about 80% for $\alpha = 0.467$, and of about 55% for $\alpha = 0.7$. The maximum increment is reached when the ratio l_r/h is of about 1.25.

The shape of the increment for l_r/h values lower than 1.25 is greatly affected by α. Particularly, as above mentioned, it is evident the lack of strength increase for ratios l_r/h lower than about 0.2 when α is greater than about 0.45.

On the same graph the data of Möhler and Lautenschläger (series C tests: $h = 120$ mm, $h_e = 28$ mm, $\alpha = 0.233$), are reported. From the graph it is apparent a quite large overestimation of test results even if the general trend is in good agreement. The corrective factor f_w, although it does not take into account the effect of α, is in good agreement both with test results and with the general trend and maximum value of the strength increase.

3.3 Analyses on beams with connections made with 2 or more rows of 1 fastener

These analyses were carried out to evaluate the influence of connection depth parameters (h_m, n) which are condensed by prediction formula in the corrective function f_r.

With respect to this aim, the splitting failure loads of beams loaded at mid-span by connections made with 2, 3, 4 and 5 rows of 1 fasteners, and with an height of 180 mm have been numerically computed. In the analyses 3 different α values and 4 total height h_m were considered. The analyses were carried out assuming the same transversal displacements of all different fasteners.

The results of these analyses are shown in Figs. 12 and 13 respectively for specimens with 2

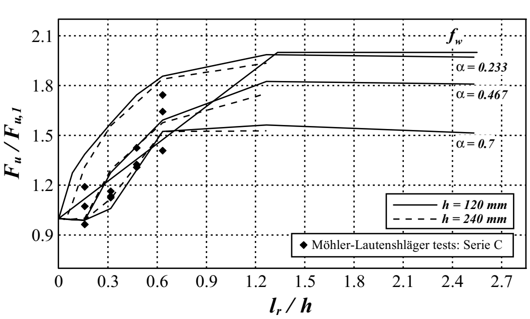

Fig. 11 Experimental, numerical and predicted strength increase

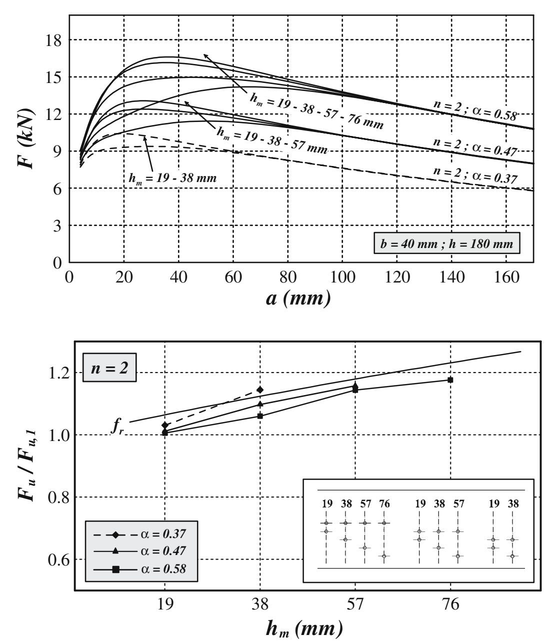

Fig. 12 Numerical failure loads of beams with 2 rows of 1 fastener for different h_m values (above), and strength increase (bottom)

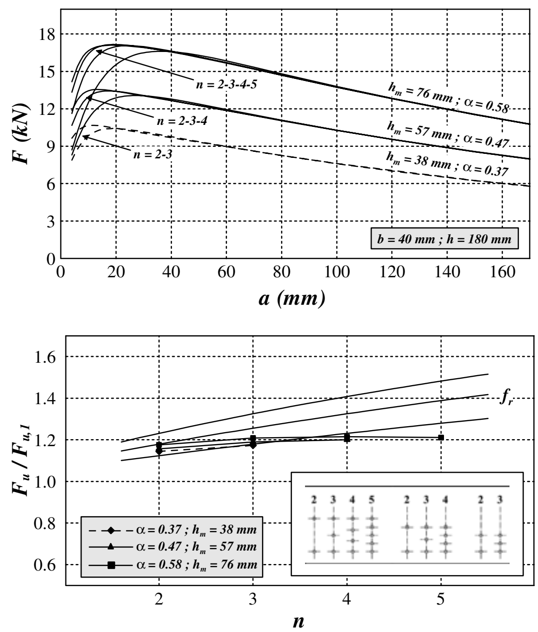

Fig. 13 Numerical failure loads of beams with 2 or more rows of 1 fastener with different spacing and constant h_m (above), strength increase (bottom)

rows of 1 dowel each (different h_m values), and for specimens with 2 or more rows of 1 dowel each (constant h_m values).

In Fig. 12 the propagation loads of beams for different spacing h_m and α values (above), and the strength increases (bottom), are reported. From the first graph, appear an increase of the splitting strength and a decrease of the critical crack length a_{cr} when h_m rises. The second graph shows a maximum strength increase of about 20% in the range of investigated parameter h_m. On the contrary the effect of α is quite negligible. The corrective function f_r, also plotted in the graph, is in a quite good agreement with numerical results.

In Fig. 13 the same results of previous Fig. 12 are reported for specimens characterized by different number of rows n and different spacing (constant h_m values). From the former graph (above), a more limited strength increase (with respect to the previous one) can be detected as n increase and also in this case a reduction of the critical crack length a_{cr} is evident. The latter graph (bottom) confirms the outcomes driven from the first one; indeed the strength increase is very limited as n rises and also α has a quite negligible effect. Lastly, the corrective function f_r gives an overestimation of the effect of n especially for large values of this parameter and for the bigger h_m values.

4 Conclusions

The paper presents the results of a parametric numerical study developed to investigate the effects of main joint parameters on the splitting strength of timber beams loaded perpendicular-to-grain by dowel-type connections. A further target of the investigation is the verification of the effectiveness of the new Ballerini's semi-empirical prediction formula also illustrated in the first part of the paper.

The numerical analyses concerned beams with single-dowel connections, beams with 1 row of 2 dowels at different spacing (different joint width l_r), and beams with 2 and more rows of 1 dowel (different joint depth h_m and rows n).

From the performed analyses, the following conclusions can be taken:

- numerical analyses on beams with single-dowel connections tend to overestimate the strength of specimens with α values greater than 0.5; on the contrary the prediction formula is well in line with test data;
- analyses on beams with 1 row of 2 dowels highlighted the considerable effect of both parameters l_r and α: indeed, the maximum splitting strength increase of beams is of about 100% for l_r/h greater than 1.25 and α not larger than 0.25; the corrective function f_w matches the experimental data better than numerical curves but it does not recognize the effect of α;
- analyses on beams with more rows of 1 dowel showed the limited effect of parameters h_m, n and α; the corrective function f_r is generally quite in good agreement with numerical results with only a limited overestimation of the effect of n.

The results show that numerical analyses are a good approach for the parametric investigation of the effect of joint parameters on the splitting strength of timber beams. This approach can be profitably used to limit the expensive experimental verification to some selected specimens.

Future perspectives in the field suggest further numerical investigations into the number of columns m, the spacing between clusters of fasteners l_1, and larger values of connection width h_m.

References

1. Möhler K, Lautenschläger R (1978) 'Large perpendicular-to-grain connections in glulam beams', ''only available in German'', Forschungsbericht des Lehrstuhls für Ingenieur-holzbau und Baukonstruktionen, Universität Karlsruhe
2. Möhler K, Siebert W (1980) 'Design of perpendicular-to-grain joints for loads suspending for glulam or timber beams', ''only available in German'', Forschungsbericht des Lehrstuhls für Ingenieurholzbau und Baukonstruktionen, Universität Karlsruhe
3. Ehlbeck J, Görlacher R (1983)'Structural behaviour of perpendicular-to-grain connections with steel elements in timber construction', ''only available in German'', Forschungsbericht der Versuchsanstalt für Stahl, Holz und Steine, Abt. Ingenieurholzbau, Universität Karlsruhe
4. Ehlbeck J, Görlacher R, Werner H (1989) 'Determination of perpendicular-to-grain tensile stresses in joints with dowel-type fasteners: a draft proposal for design rules', in Proceedings of CIB-W18 Conference, paper 22-7-2, Berlin, German Democratic Republic, Sept. 1989
5. DIN 1052:2004-08 (2004) 'Design of timber structures—General rules and rules for buildings', ''only available in German'', (DIN, Germany, 2004)
6. Van der Put TACM (1990) 'Tension perpendicular to the grain at notches and joints', in Proceedings of CIB-W18 Conference, paper 23-10-1, Lisbon, Portugal, Sept. 1990
7. Van der Put TACM (1992) 'Energy approach for fracture of joints loaded perpendicular to the grain', in Proceedings of COST 508 Workshop on ''Fracture Mechanics in wooden structures'', Bordeaux, France, April 1992
8. EN 1995-1-1:2004 (2004) 'Eurocode 5 - Design of timber structures. Part 1-1: General rules and rules for buildings', CEN/TC 250/SC5, 2004-11-01
9. Yasumura M, Murota T, Sakai H (1987) 'Ultimate properties of bolted joints in glued-laminated timber', in Proceedings of CIB-W18 Conference, paper 20-7-3, Dublin, Ireland, Sept. 1987
10. Yasumura M (2001) 'Criteria for damage and failure of dowel-type joints subjected to force perpendicular-to-grain' in Proceedings of CIB-W18 Conference, paper 34-7-9, Venice, Italy, Aug. 2001
11. Reske RG, Mohammad M, Quenneville JHP (2000) 'Influence of joint configuration parameters on strength of perpendicular-to-grain bolted timber connections', in Proceedings of 6th World Timber Engineering Conference, WCTE 2000, Whistler, B.C., Canada, Aug. 2000
12. Kasim M, Quenneville JHP (2002) 'Effect of row spacing on the capacity of bolted timber connections loaded perpendicular-to-grain', in Proceedings of CIB-W18 Conference, paper 35-7-6, Kyoto, Japan, Sept. 2002
13. Ballerini M (1999) 'A new set of experimental tests on beam loaded perpendicular-to-grain by dowel-type joints', in Proceedings of CIB-W18 Conference, paper 32-7-2, Graz, Austria, Aug. 1999
14. Ballerini M, Giovanella A (2003) 'Beams transversally loaded by dowel-type joints: influence on splitting strength of beam thickness and dowel size', in Proceedings of CIB-W18 Confe-rence, paper 36-7-7, Estes Park, Colorado, USA, Aug. 2003
15. Borth O, Rautenstrauch K, (2001)'Estimation of the load-carrying capacity of perpendicular-to-grain bolted timber connections by fracture criterion in the framework of LEFM', in Proceedings of the International RILEM Symposium on ''Joints in Timber Structures'', Stuttgart, Germany, Sept. 2001
16. Ballerini M, Bezzi R (2001) 'Numerical LEFM analyses for the evaluation of failure loads of beams loaded perpendicular-to-grain by single-dowel connections', in Proceedings of CIB-W18 Conference, paper 34-7-6, Venice, Italy, Aug. 2001
17. Acler W (2002) 'A numerical study on the splitting strength of beams loaded perpendicular-to-grain by single-dowel connections: influence of geometrical and mechanical parameters', ''only available in Italian'', Degree Thesis, University of Trento, Italy, Oct. 2002
18. Ballerini M (2004) 'A new prediction formula for the splitting strength of beams loaded by dowel-type connections', in Proceedings of CIB-W18 Conference, paper 37-7-5, Edinburgh, Scotland, Aug. 2004
19. Wu EM (1967) 'Application of fracture mechanics to anisotropic plates'. ASME J Appl Mech Series E 34(4):967–974
20. Chen LS, Kuang JH (1993) 'A displacement extrapolation method for two-dimensional mixed-mode crack problems'. Eng Frac Mech 46(5):735–741
21. Guinea GV, Planas J, Elices M (2000) 'KI evaluation by the displacement extrapolation technique'. Eng Frac Mech 66:243–255
22. Hunt DG, Croager WP (1982) 'Mode II fracture toughness of wood measured by a mixed mode test method'. J Mater Sci Lett 1(1):77–79
23. Valentin GH, Boström L, Gustafsson PJ, Ranta-Maunus A, Gowda S (1991) 'Application of fracture mechanics to timber structures, RILEM state-of-the-art-report', VTT – Technical Research Centre of Finland, Research Notes 1262, Espoo, Finland, 1991
24. Tan DM, Stanzl-Tschegg SE, Tschegg EK (1995) 'Models of wood fracture in mode I and mode II'. Holz als Roh-und Werkstoff 53(3):159–164