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TEST SPECIMEN GEOMETRY FOR LONGITUDINALLY REINFORCED GLULAM BEAMS IN BENDING

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Abstract. Although research into longitudinal reinforcement of timber beams has continued for approximately seven decades, the leading industrialised countries still lack basic normative documents governing the design of such structures. One reason is the absence of unified test methods, which precludes the formation of a comparable experimental database. This paper provides a rationale for the selection of geometric parameters of test specimens of glued laminated timber (glulam) beams reinforced with longitudinal reinforcement for four-point bending tests. A comparative analysis of the requirements of European (EN 408), international (ISO 8375, ISO 13910) and American (ASTM D198, ASTM D4761) standards for bending tests of timber beams has been carried out. A search of the Scopus, Web of Science and Google cholar databases yielded a sample of 31 experimental studies on reinforced beams, the analysis of which revealed a considerable scatter in the geometric parameters of test specimens: span-to-depth ratios ranging from 7.7 to 25.0; depth-to-width ratios from 1.0 to 4.29; and cross-section depths from $90\,\mathrm{mm}$ to $605.5\,\mathrm{mm}$. It has been established that the choice of parameters was independent of the reinforcement ratio and reinforcement method, and shows no trend towards standardisation over time. Limiting values of the geometric parameters have been analytically justified on the basis of four independent criteria: ensuring a flexural failure mode allowing for the increased bending capacity of the reinforced cross-section; minimising the influence of shear deformations on the determined modulus of elasticity; ensuring statistical representativeness in terms of the Weibull size effect and the minimum number of laminations; and ensuring lateral stability of the beam without additional lateral restraints. It is shown that longitudinal reinforcement increases the risk of lateral instability, because the bending capacity increases far more than the critical buckling moment. Recommendations for the geometric parameters of test specimens are formulated: $l/h = 18..21$; $h/b \le 2.4$ for moderate and $h/b \le 2.0$ for high reinforcement ratios; cross-section depth $180..600\,\mathrm{mm}$; and a minimum of four laminations.

Keywords: glued laminated timber; longitudinal reinforcement; test specimen geometry; four-point bending; FRP.

INTRODUCTION

Research into reinforcement of timber beams began in the 1960s [1] with the use of conventional metallic (steel or aluminium) plates and bars; the first attempt to apply glass-fibre reinforced polymers (GFRP) dates back to 1965 and their use became widespread during the 1970s, while from 1992 carbon-fibre reinforced polymers (CFRP) began to be employed for structural strengthening. Over nearly seven decades of experimental and analytical investigation a substantial body of data has been accumulated confirming the effectiveness of the technology. A consolidated review of research on the reinforcement of glued laminated timber structures and on the application of composite strips and fabrics for the strengthening of timber structures is presented in [2, 3]. Despite this, as of 2026, no major building code in economically developed countries contains provisions for the design and analysis of timber structures with longitudinal reinforcement.

The lack of a corresponding normative framework deprives engineers of the ability to exploit the full advantages of longitudinal reinforcement, among which are: an increase in bending capacity of 20..90 % [1, 4]; an increase in stiffness; a transition from a brittle failure mode to a more ductile one [5]; the possibility of reducing the depth of structural members while maintaining equivalent serviceability characteristics; and a more efficient use of lower-grade timber.

Certain advances in the regulatory landscape occurred in the 21st century, when the Italian National Research Council (CNR) published guidelines [6] on the strengthening of existing timber structures with externally bonded FRP systems virtually simultaneously with the publication in the United States of a standard practice [7] that establishes procedures for manufacturers to determine the characteristic values of strength and modulus of elasticity of reinforced glulam beams.

During the preparation of the new generation of Eurocode 5, the working group CEN / TC 250 / SC 5 / WG 7 "Reinforcement" decided against codifying longitudinal reinforcement and focused exclusively on reinforcement resisting stresses perpendicular to the grain (notches, holes, curved beams, connections). The decision of WG 7 [8] was motivated by the fact that longitudinal reinforcement does not represent a typical design situation in European practice; the technology has not been validated by sufficient practical experience; and there are no harmonised product standards or European Technical Assessments (ETA), without which a design standard cannot be drafted.

For the future incorporation of design provisions for timber structures with longitudinal reinforcement into normative documents, it is necessary not only to increase the volume of the experimental database but also to improve its quality, which cannot be achieved without unified test methods. This paper attempts to standardise the geometric parameters of test specimens of reinforced glulam beams for four-point bending tests, in a manner analogous to what has been done in the current standards for unreinforced glulam beams [9, 10, 11].

AIM AND METHODS

The aim of this work is to formulate recommendations for the selection of geometric dimensions of test specimens of glued laminated timber beams reinforced with longitudinal reinforcement, compliance with which will ensure the realisation of a flexural failure mode, minimise the influence of secondary factors on the experimental results, and enable a correct comparison of data from different researchers.

To achieve this aim, a comparative analysis was carried out of the requirements for bending tests of timber beams established by European [9], American [10, 11] and international [12, 13] standards. The key parameters of the loading configurations were systematised: the span-to-depth ratio of the beam, the distances from supports to loading points, the distance between loading points, and the restrictions on the cross-sectional geometry.

To evaluate the current state of the experimental database, a search was conducted of the Scopus, Web of Science and Google Scholar databases covering the past 30 years. The inclusion criteria were: an experimental investigation of four-point bending behaviour of reinforced beams of either sawn timber or glulam, and sufficient completeness of the

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information on specimen geometry and reinforcement parameters. The search yielded a sample of 31 experimental studies, for each of which the cross-sectional dimensions, span length and their ratios, timber type, and reinforcement type, ratio and method were recorded.

The limiting values of the geometric parameters were justified analytically, proceeding from the requirement that the geometry of a test specimen of a reinforced beam must simultaneously satisfy several conditions. First, the proportions of the beam must ensure the realisation of a flexural failure mode rather than premature shear failure. Second, the proportion of shear deformations in the total deflection must be sufficiently small so as not to distort the results of determining the modulus of elasticity in bending. Third, the cross-section depth must be sufficient to ensure statistical representativeness of the mechanical properties of glulam in terms of the Weibull size effect and the minimum number of laminations. Fourth, the depth-to-width ratio must ensure lateral stability of the beam without additional lateral restraints.

RESULTS AND EXPLANATIONS

Normative requirements for bending tests and analysis of the experimental database

The current standards for bending tests of timber beams [9, 10, 11, 12, 13] prescribe a four-point bending configuration and specify the span-to-depth ratio, but do not restrict the absolute dimensions of the cross-section (Table 1). All of the above standards have been developed for unreinforced timber, and none of them contains provisions specific to the testing of beams reinforced with longitudinal reinforcement.

Table 1. Comparison of requirements for bending tests of beams
Табл. 1. Порівняння вимог до випробування балок на згин

Standard	l/h	Distance from support to loading point	Distance between loading points
EN 408 [9]	18 ± 3	$6h \pm 1.5h$	$6h$
ISO 8375 [12]	18 ± 3	$6h \pm 1.5h$	$6h$
ISO 13910 [13]	18 ± 3	$6h \pm 1.5h$	$6h$
ASTM D198 [10]	12..18	$(4..6) \cdot h$ or $(0.4..0.6) \cdot f_m/f_v$	
ASTM D4761 [11]	17..21	$l/3$	

Table 2 summarises the data from 31 studies that satisfy the defined inclusion criteria. The sample does not claim to be exhaustive; however, it is sufficiently representative to identify characteristic trends.

For each study, the cross-sectional dimensions and span, the timber type, and the type, ratio and method of reinforcement are recorded, which allows not only the scatter of parameters to be analysed but also potential correlations between them. The analysis of the data in Table 2 reveals a considerable heterogeneity in the geometric parameters of the test specimens.

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Table 2. Overview of experimental studies on reinforced timber beams in four-point bending
Табл. 2. Приклади досліджень армування дерев'яних балок на чотириточковий згин

Ref.	h, mm	b, mm	l, mm	h/b	l/h	Timber type	A_r', %	A_r, %	Compr. reinf.	Tens. reinf.	Method
1	2	3	4	5	6	7	8	9	10	11	12
[14]	200	100	2700	2.00	13.5	ST	–	0.15..0.42	–	CFRP	EBR
[15]	175	85	1350	2.06	7.7	GLT	0..1.06	0.53..1.06	A400	A400	NSM
[16]	100	100	1800	1.00	18.0	GLT	–	0.52..1.03	–	GFRP	INT
								0.15..0.59		CFRP	
	600	200	10800	3.00				0.14..0.86		GFRP	
								0.09..0.60		CFRP	
[17]	200	200	3600	1.00	18.0	ST	–	0.08..0.12	–	CFRP	EBR/NSM
[18]	223.6	96	3960	2.33	17.7	GLT	–	1.64	–	FRP	EBR
[19]	480	203	7620	2.36	15.9	ST	–	0.88	–	CFRP	EBR
[20]	150	75	1800	2.00	12.0	ST	–	0.62..1.39	–	CFRP	NSM/EBR
[21]	90	50	1350	1.80	15.0	GLT	–	0.44	–	CFRP	EBR
		70		1.29				0.56			
[22]	600	220	10800	2.73	18.0	GLT	0..1.70	0.80..1.70	CFRP	CFRP	INT/EBR
	605.5			2.75	17.8		–	0.80	–	CFRP	
[23]	298.5	81	4877	3.69	16.3	GLT	–	1.10	–	FRP	INT
	307.8			3.80	15.8			3.10			
[24]	305	130	6400	2.35	21.0	GLT	–	1.10	–	GFRP	INT
								3.30			
[25]	200	70	3800	2.86	19.0	ST	–	1.20..3.30	–	GFRP	EBR
	300			4.29	12.7						
[26]	120	60	3000	2.00	25.0	GLT	–	1.00..3.00	–	GFRP	INT
								0.40		CFRP	
[27]	300	100	4000	3.00	13.3	ST	–	0.39..0.79	–	GFRP	NSM
	600	200	10000	3.00	16.7		–	0.26..0.44	–	GFRP	
[28]	210	80	3780	2.63	18.0	GLT	–	0.46	–	CFRP	EBR
								0.93			
[29]	240	100	2200	2.40	9.2	GLT	–	0.50	–	CFRP	EBR
[30]	300	120	2700	2.50	9.0	GLT	–	0.46	–	H-CFRP	INT
		140		2.14				0.40		H-CFRP	
								0.40		L-CFRP	
								0.80		H-CFRP	
[31]	111.5	60	1650	1.86	14.8	GLT	–	1.36	–	S355	EBR
	111.2			1.85				0.91		CFRP	
[32]	160.8	93.5	1800	1.72	11.2	GLT	–	0.87	–	CFRP	INT
	162.1	93.3		1.74	11.1		0.86	0.86	CFRP	CFRP	

Table 2. (continued)
Табл. 2. (продовження**)**

[33]	138	38	2440	3.63	17.7	GLT	–	0.66..0.92	–	CFRP	EBR
[34]	200	115	3600	1.74	18.0	GLT	0..1.00	1.00..2.00	S275	S275	NSM
							0.75..0.83	0.75..1.67	CFRP	CFRP	
[35]	260	90	3740	2.89	14.4	GLT	–	1.28	–	S235	INT
[36]	300	75	5700	4.00	19.0	GLT	0..0.66	0.33..1.32	CFRP	CFRP	NSM
[37]	150	60	2700	2.50	18.0	GLT	–	0.78	–	CFRP	EBR
[38]	190	96	3420	1.98	18.0	GLT	–	1.27	–	FRP	EBR/INT
	215			2.24	15.9			1.12		FRP	
[39]	308	100	4200	3.08	13.6	GLT	–	0.39	–	CFRP	EBR/INT
	310			3.10	13.5			0.90			
	312			3.12	13.4			0.90			
								2.31		AFRP	
[40]	162	82	3000	1.98	18.5	GLT	–	0.37	–	BFRP	EBR
[5]	300	75	5700	4.00	19.0	GLT	0..0.50	0.50..1.79	GFRP	GFRP	NSM/INT
							–	1.00..1.79	–	B400B	
							0..0.66	0.33..0.99	CFRP	CFRP	
[41]	240	100	3750	2.40	15.6	GLT	–	0.50	–	CFRP	EBR
[42]	150	100	2700	1.50	18.0	ST	1.51	0.40	A500C	CFRP	NSM/EBR
[43]	210	100	2700	2.10	12.9	GLT	–	0.75	–	GFRP	ER
								0.75		BFRP	
								0.75..1.91		A400C	

Notes:
- Timber type: ST – sawn timber, GLT – glued laminated timber.
- $A_r^{'}, \%$ – reinforcement ratio in the compression zone; $A_r, \%$ – reinforcement ratio in the tension zone.
- FRP reinforcement types: CFRP – carbon fibre reinforced polymer, GFRP – glass fibre reinforced polymer, BFRP – basalt fibre reinforced polymer, AFRP – aramid fibre reinforced polymer.
- Steel reinforcement: S235, S275, S355 – structural steels according to EN 10025; A400, A500C, A400C – reinforcing bar grades according to DSTU 3760:2019; B400B – reinforcement for concrete according to ISO 6935-2 / EN 10080.
- Reinforcement method: EBR – externally bonded reinforcement, NSM – near-surface mounted, ER – embedded rod, INT – internal reinforcement bonded in the glueline.

Cross-sectional dimensions range from $90\times50\,\text{mm}$ to $605.5\times220\,\text{mm}$, span lengths from $1350\,\text{mm}$ to $10800\,\text{mm}$, span-to-depth ratios from 7.7 to 25, and depth-to-width ratios from 1 to 4.29.

The reinforcement ratio in the tension zone varies from $0.08\,\%$ to $3.30\,\%$, while compression-zone reinforcement has been applied in only 8 out of 31 studies. The vast majority of studies were conducted on glulam beams, whereas sawn timber is represented in 6 studies. Among the reinforcing materials, CFRP predominates (22 studies), followed by GFRP (10 studies), steel elements (5 studies), basalt fibre (2 studies) and aramid fibre (1 study). The reinforcement methods are distributed among externally bonded reinforcement, near-surface mounted reinforcement, internal reinforcement bonded in the glueline and embedded rods, often in combination.

Comparison with the normative requirements (Table 1) reveals that a significant proportion of the specimens fall outside the recommended range $l/h = 18\pm3$, with a noticeable tendency towards lower values of this ratio. Studies with very low l/h values occur both among early and among the most recent works ([15, 29, 30]), indicating no trend towards standardisation over time.

The standard practice [7], which specifically addresses beams reinforced with longitudinal reinforcement, establishes typical reinforcement ratio ranges depending on the material type: 1.0..3.0 % for GFRP, 0.3..0.9 % for CFRP, and 0.2..0.6 % for steel. Comparison with the data in Table 2 shows that some studies fall outside these limits, both below and above.

A check for possible correlations between specimen parameters revealed no systematic dependencies. Studies with high reinforcement ratios are found at both low l/h values ([15, 20, 39]) and high values ([24, 26]), which indicates that the researchers did not adjust the specimen proportions to account for the increased bending capacity of the reinforced cross-section. Similarly, the reinforcement method does not influence the choice of l/h: each method (EBR, NSM, INT, ER) exhibits an internal scatter comparable with the overall range of the sample.

The demonstrated unsystematic selection of the geometric parameters of test specimens can fundamentally distort the understanding of the behaviour of reinforced beams.

Span-to-depth ratio of the test beam

The span-to-depth ratio (l/h) is a key parameter that determines the structural behaviour of the member during testing, since at a low value of l/h failure may be governed by shear stresses, rendering the experimental results unsuitable for evaluating the bending capacity. For unreinforced beams this problem is discussed in detail in section X5.3 of standard [10].

As stated in clause X5.3.1 of [10], for a beam of rectangular cross-section under four-point bending with the loads applied at the third points of the span ($a = l/3$), assuming elastic material behaviour, simultaneous flexural and shear failure occurs at the ratio:

$$\frac{a}{h} = \frac{f_m}{4 f_v} = 0.25 \cdot \frac{f_m}{f_v}, \qquad (1)$$

where f_m and f_v are the bending strength and shear strength of the timber, respectively (mean or characteristic values, depending on the method of determining the material properties).

If the factor $k_{cr} = 0.67$ (clause 6.1.7 of [44]) is applied to account for the reduction of the effective cross-section width due to cracks, the critical ratio becomes:

$$\frac{a}{h} = \frac{0.25}{k_{cr}} \cdot \frac{f_m}{f_v} = 0.373 \cdot \frac{f_m}{f_v}. \qquad (2)$$

This is in good agreement with the recommendations of clause X5.3.3 of [10], where it is stated that specimens tested in bending should have a ratio a/h in the range from $0.4 \cdot f_m / f_v = 4$ to $0.6 \cdot f_m / f_v = 6$ at $f_m / f_v = 10$ (clause X5.3.2 of [10]):

$$0.4 \cdot \frac{f_m}{f_v} \le \frac{a}{h} \le 0.6 \cdot \frac{f_m}{f_v}. \qquad (3)$$

For convenience, this expression can be rewritten as:

$$k_{low} \cdot \frac{f_m}{f_v} \le \frac{a}{h} \le k_{high} \cdot \frac{f_m}{f_v}, \qquad (4)$$

where $k_{low} = 0.4$ is the lower-bound coefficient and $k_{high} = 0.6$ is the upper-bound coefficient of the range.

The above relationships apply to unreinforced beams. Longitudinal reinforcement of the tension zone increases the bending capacity of the beam, while the shear capacity remains essentially unchanged. The validity of this assumption is confirmed by clause 4.10.2 of the standard practice [7], which states that at typical reinforcement ratios (up to 3 % GFRP or 0.9 % CFRP) the maximum shear stresses at the neutral axis of a reinforced beam are practically identical to those of an unreinforced rectangular cross-section. Consequently, as the reinforcement ratio increases, the ratio of the flexural failure load to the shear failure load also increases, and a higher value of a/h is required to ensure a flexural failure mode.

Given that reinforcement of timber beams increases their bending capacity by approximately 20..90 % [1], let us introduce the strengthening factor $k_r = M_{R,r} / M_R =$

$=1.2..1.9$, which is the ratio of the ultimate moments for the reinforced and unreinforced beams. Accounting for the increased capacity, the lower bound of the recommended range increases: $k_{low,r} = k_{low} \cdot k_r$. Substituting the values gives $0.5 \le k_{low,r} \le 0.80$. Thus, at significant reinforcement ratios ($k_r > 1.5$), the lower bound for reinforced beams exceeds the upper bound for unreinforced beams $(k_{low,r} > k_{high} = 0.6)$. At the same time, according to clause X5.3.4 of [10], beams with $a/h > 6$ are intended exclusively for deflection testing, since at such slenderness the effect of horizontal components of support reactions becomes significant and the beam enters a complex stress state.

Therefore, when selecting the proportions of test specimens of beams reinforced with longitudinal reinforcement, according to [10] one should target a value of $a/h = 6$, which corresponds to $l/h = 18$ for third-point loading. This proportion provides a sufficient margin for realising a flexural failure of the reinforced cross-section and minimises the influence of secondary factors on the experimental result.

An additional argument in favour of $l/h \ge 18$ is the effect of shear deformations on the determined modulus of elasticity. The total instantaneous deflection (w_{inst}) comprises the pure bending deflection (w_m) and the additional shear deflection (w_v). According to row 4 of Table X2.1 in [10], for equal concentrated forces applied at the third points of the span:

$$w_m = \frac{23 \cdot P \cdot l^3}{108 \cdot E \cdot b \cdot h^3},\ w_v = \frac{P \cdot l}{5 \cdot G \cdot b \cdot h}. \qquad (5)$$

The ratio of the shear deflection to the bending deflection is then:

$$\frac{w_v}{w_m} = \frac{108}{115} \cdot \frac{E}{G} \cdot \left(\frac{h}{l}\right)^2. \qquad (6)$$

For timber, the ratio of the modulus of elasticity to the shear modulus is typically taken as 16 in accordance with [45]; hence, for a beam with $l/h = 18$:

$$\frac{w_v}{w_m} = \frac{108}{115} \cdot 16 \cdot \left(\frac{1}{18}\right)^2 = 0.046. \qquad (7)$$

Thus, even at $l/h = 18$ the shear deformations amount to approximately $5\ \%$ of the bending deformation, or more than $4\ \%$ of the total deflection. For a beam reinforced with longitudinal reinforcement, stiff reinforcement in the tension zone ($E_r \approx 40..210\,\text{GPa}$) increases the effective bending stiffness $(EI)_{ef}$ of the cross-section, whereas the shear modulus of the timber G remains unchanged because the reinforcement does not contribute to the shear resistance. As a result, the ratio E_{ef}/G for the reinforced cross-section exceeds the E/G ratio of the unreinforced one. For typical reinforcement ratios ($\rho = 0.3..0.9\ \%$ CFRP) $E_{ef} \approx 1.1..1.4 \cdot E_w$, while at high ratios it can reach $E_{ef} \ge 2 \cdot E_w$.

Thus, in a reinforced beam the proportion of shear deformations increases in proportion to the ratio E_{ef}/G and to the square of $(h/l)^2$. For typical reinforcement ratios at $l/h = 18$, the shear deformations amount to $5..7\ \%$ of the bending deformation, and at the value $l/h = 15$, which is permissible under [9, 12, 13], they increase to $7..9\,\%$. The overall cumulative effect of the above factors ($E_{ef} = 2 \cdot E_w$ and $l/h = 15$) on the level of shear-induced deformation can reach more than $13\,\%$ of the bending deformation, or approximately $12\ \%$ of the total deflection.

If during the experiment only the apparent (global) modulus of elasticity was determined without separating the local and global components, this would lead to a systematic underestimation of the measured stiffness and to an erroneous conclusion of a lower-than-expected stiffness gain from reinforcement.

It should be noted that the validation matrix of the standard practice [7] prescribes testing of reinforced glulam beams at $l/h = 21$ for both regulated specimen sizes, which corresponds to the upper bound of the ranges given in [11] ($l/h = 17..21$) and [9, 12, 13] ($l/h = 18 \pm 3$). This further confirms that for beams reinforced with longitudinal reinforcement, l/h values in the range 18..21 are well justified.

Cross-section depth and the minimum number of laminations in the test beams

The bending strength of timber is described by Weibull's weakest-link statistical theory [46]. According to this theory, the strength depends on the volume (depth) of the stressed zone:

$$\frac{f_{m,1}}{f_{m,2}} = \left(\frac{h_2}{h_1}\right)^{\frac{1}{k}}, \tag{8}$$

where k is the shape parameter of the distribution ($k = 5$ for sawn timber, $k = 10$ for glulam, according to [44]). On the basis of this relationship, EN 1995-1-1 [44] establishes the size factor for glulam beams $k_h = \left(\frac{600\,\text{mm}}{h}\right)^{\frac{1}{10}}$, which normalises the bending strength to a reference depth of $600\,\text{mm}$. This means that the test results for beams of different depths can be correctly compared only when the appropriate size factor is applied.

Proceeding from the definition of the size factor, the ideal depth of test specimens would be $h = 600\,\text{mm}$, at which $k_h = 1.0$. However, given $l/h = 18$ as justified in the preceding subsection, the total length of such a specimen including overhang beyond the supports would be $l_{total} = l_{span} + 2 \times 0.5h = 11400\,\text{mm}$. The fabrication and testing of beams of this size is not only costly but also technically unfeasible for a large number of laboratories.

It is therefore necessary to determine the minimum acceptable cross-section depth that will ensure an acceptable value of the size factor and statistical representativeness of the mechanical properties of glulam. The limiting factor may be taken as the minimum number of laminations and their maximum thickness.

Standard [47] permits the manufacture of structural glulam from two laminations; however, the system of strength classes and the design characteristic values are oriented towards cross-sections with four or more laminations. The preceding standard [48] established strength classes exclusively for members with four or more laminations. An analogous approach is adopted in the American standard [49], where the tabulated design values are given for members with four or more laminations, while for two- or three-layer constructions the application of reduction factors is required. This is because with fewer than four laminations the statistical averaging of timber defects is insufficient, leading to increased variability in the mechanical properties and reduced reliability of the results.

The maximum lamination thickness according to [47] must not exceed $45\,\text{mm}$; therefore, the minimum depth of a test beam is $h_{\min} = 4 \times 45 = 180\,\text{mm}$. In this case the size factor $k_h = (600/180)^{0.1} \approx 1.13$, corresponding to an increase of approximately 13 % in the characteristic bending strength compared with the reference depth. This value is acceptable, as it can be accounted for when processing the test results, whereas a further reduction in depth would lead to excessive corrections and would call into question the representativeness of the data.

Although the upper bound of the cross-section depth is formally limited only by the capacity of the laboratory equipment and the cost of specimen fabrication, from a practical standpoint it is advisable to target the range $180\,\text{mm} \le h \le 600\,\text{mm}$, which corresponds to total specimen lengths from approximately $3420\,\text{mm}$ to $11400\,\text{mm}$ at $l_{total}/h = 19$.

Width of the beam cross-section

The depth-to-width ratio of the cross-section determines the susceptibility of the beam to lateral instability during bending. Standards [10, 11] and the ISE/TRADA manual [50] state that at $h/b \ge 3$ additional lateral restraints must be provided to ensure lateral stability.

The limiting value of h/b can be obtained by means of a lateral stability check in accordance with clause 6.3.3 of [44], according to which the critical bending stress for a simply supported beam of rectangular cross-section is:

$$\sigma_{m,crit} = \frac{0.78 \cdot b^2}{h \cdot l_{ef}} \cdot E_{0,05} \,, \qquad (9)$$

where l_{ef} is the effective length for lateral buckling, determined in accordance with Table 6.1 of [44].

Equation (9) is derived from the elastic lateral-torsional buckling solution for a beam of rectangular cross-section under constant bending moment:

$$\sigma_{m,crit} = \frac{M_{crit}}{W_y} \,, \qquad (10)$$

where:

$$M_{crit} = \frac{\pi}{l_{ef}} \cdot \sqrt{E_{0,05} \cdot I_z \cdot G_{0,05} \cdot I_{tor}} \,, \qquad (11)$$

and $G_{0,05} \approx E_{0,05} / 16$ [45].

For unreinforced glulam beams with $l/h = 18$ under four-point bending, the calculation shows that the stability condition is satisfied at $h/b \approx 2.9..3.2$, depending on the timber properties.

For beams reinforced with longitudinal reinforcement, the limiting h/b ratio must be smaller, because longitudinal reinforcement substantially increases the bending capacity $(M_{R,r} = k_r \cdot M_R)$ but has a negligible effect on the critical buckling moment M_{crit}. Although for CFRP strips at typical reinforcement ratios (EBR, INT) with $E_r / E_w = 13$ and $\rho = 0.9$ %, the relative increase in the second moment of area about the weak axis I_z is approximately $11\,\%$, the torsional rigidity I_{tor} remains practically unchanged (increase of $0.004\,\%$). From (Eq. 11) it follows that $M_{crit} \propto \sqrt{I_z \cdot I_{tor}}$; therefore, the overall increase of $M_{crit,r}$ relative to M_{crit} is approximately $5\,\%$. For bars placed in grooves (NSM), the contribution to I_z will be even smaller. At the same time, the bending capacity increases by $20..90\,\%$ [1].

This means that the relative slenderness of the reinforced beam is greater than that of the unreinforced beam:

$$\lambda_{rel,m,r} = \sqrt{\frac{f_{m,r}}{\sigma_{m,crit,r}}} > \lambda_{rel,m} \,, \qquad (12)$$

since the numerator ($f_{m,r} = k_r \cdot f_m$) increases far more than the denominator.

As a consequence, the lateral stability factor k_{crit} (Eq. 6.34 of [44]) decreases, and the beam may require lateral restraints at smaller h/b values than for an unreinforced beam.

Limiting the depth-to-width ratio of the test beam ensures lateral stability without additional lateral restraints, which could distort the test results due to friction and partial restriction of the free deformations of the specimen. The limiting values of h/b are determined from the stability condition using (Eqs. 9..12) at $l/h = 18$ and $E_{0,05} / G_{0,05} = 16$. The boundary between moderate and high reinforcement ratios is defined by the criterion $k_r = M_{R,r} / M_R = 1.5$. The reinforcement ratios at which this level is reached have been determined analytically following [7] for glulam of strength classes GL20h to GL32h; the results are summarised in Table 3.

Table 3. Limiting depth-to-width ratio as a function of the reinforcement ratio
Табл. 3. Граничне відношення висоти до ширини перерізу в залежності від відсотка армування

Reinforcement level	h/b	Reinforcement ratio by material type		
		CFRP	GFRP	Steel
Moderate	≤ 2.4	≤ 1.0 %	≤ 3.0 %	≤ 0.8 %
High	≤ 2.0	> 1.0 %	> 3.0 %	> 0.8 %

The computed values for the moderate reinforcement level are in good agreement with the typical maximum values given in Table 3 of [7], which indicates the validity of selecting $k_r = 1.5$ as the boundary between moderate and high reinforcement levels.

CONCLUSIONS AND RECOMMENDATIONS

Analysis of 31 experimental studies on beams reinforced with longitudinal reinforcement revealed a considerable scatter in the geometric parameters of the test specimens: $90\,\text{mm} \le h \le 605.5\,\text{mm}$, $7.7 \le l/h \le 25$, $1 \le h/b \le 4.29$. The selection of parameters was unsystematic: it was independent of the reinforcement ratio and method, and shows no trend towards standardisation over time. None of the current test standards contains requirements specific to the geometry of specimens of beams reinforced with longitudinal reinforcement, and the standard practice [7] for the test procedure refers entirely to the standard [10], which was developed for unreinforced timber.

Based on an analytical justification that accounts for ensuring a flexural failure mode, minimising the influence of shear deformations, the Weibull size effect, the minimum number of laminations and lateral stability, it is recommended that test specimens of reinforced glulam beams for determining bending strength and stiffness under four-point bending be fabricated with the following geometric characteristics:

- $18 \le l/h \le 21$;
- $h/b \le 2.4$ for moderate reinforcement ratios, and $h/b \le 2.0$ for high reinforcement ratios;
- $180\,\text{mm} \le h \le 600\,\text{mm}$;
- number of laminations ≥ 4;
- overhang of the beam beyond each support $\ge 0.5h$.

The recommended range $l/h = 18..21$ is justified by two independent arguments. First, it follows from the condition of predominance of bending stresses over shear stresses at typical strengthening levels of reinforced beams. Second, as l/h decreases, the proportion of shear deformations in the total deflection increases disproportionately, distorting the stiffness determination results.

Limiting the ratio to $h/b \le 2.4$ ensures lateral stability of the beam at moderate reinforcement levels without additional lateral restraints; at high reinforcement levels the ratio reduces to 2.0 owing to the fact that the bending capacity increases far more than the critical buckling moment. At the same time, when $h/b < 2$ the beam acquires proportions uncharacteristic of glulam structures, leading to an inefficient use of material and not reflecting typical structural solutions.

The minimum cross-section depth $h_{\min} = 180\,\text{mm}$ is determined from the requirement of accommodating at least four laminations of maximum thickness 45 mm [47], which ensures statistical representativeness of the mechanical properties of glulam.

Outlook for further research

The formulated recommendations concern the geometric parameters of test specimens and do not cover a number of related questions that require separate investigation: the effect of the type and configuration of reinforcement on the effective length for lateral buckling; the refinement of the size factor for reinforced beams in which the reinforcement reduces the influence of local timber defects; and the formulation of minimum sample size requirements for obtaining statistically reliable characteristic values for reinforced glulam beams.

ETHICAL DECLARATIONS

The authors have no relevant financial or non-financial interests to report.

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ОБҐРУНТУВАННЯ ВИБОРУ ГЕОМЕТРИЧНИХ ПАРАМЕТРІВ ДОСЛІДНИХ ЗРАЗКІВ КЛЕЄНИХ ДЕРЕВ'ЯНИХ БАЛОК, АРМОВАНИХ ПОЗДОВЖНЬОЮ АРМАТУРОЮ

Орест ЛИСЮК

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Анотація. Незважаючи на те, що дослідження поздовжнього армування дерев'яних балок тривають уже близько семи десятиліть, у провідних країнах світу досі відсутні базові нормативні документи, які б регламентували проєктування таких конструкцій. Однією з причин є відсутність уніфікованих методик випробувань, що унеможливлює формування порівнянної експериментальної бази. У статті обґрунтовано вибір геометричних параметрів дослідних зразків клеєних дерев'яних балок, армованих поздовжньою арматурою, для випробувань на чотириточковий згин. Проведено порівняльний аналіз вимог європейських (EN 408), міжнародних (ISO 8375, ISO 13910) та американських (ASTM D198, ASTM D4761) стандартів випробувань дерев'яних балок на згин. За результатами пошуку у наукометричних базах даних Scopus, Web of Science та Google Scholar сформовано вибірку з 31 експериментального дослідження армованих балок, аналіз якої виявив значний розкид геометричних параметрів зразків: співвідношення прольоту до висоти від 7,7 до 25,0; відношення висоти до ширини від 1,0 до 4,29; висоти перерізу від 90 мм до 605,5 мм. Встановлено, що вибір параметрів не залежав від відсотка та методу армування і не демонструє тенденції до уніфікації з часом. Граничні значення геометричних параметрів обґрунтовано аналітично за чотирма незалежними критеріями: забезпечення згинального характеру руйнування з урахуванням збільшеної несучої здатності армованого перерізу; мінімізація впливу зсувних деформацій на результати визначення модуля пружності; забезпечення статистичної репрезентативності з позицій масштабного фактора Вейбулла та мінімальної кількості ламелей; забезпечення стійкості балки з площини згину без додаткових бічних розкріплень. Показано, що поздовжнє армування підвищує ризик втрати стійкості з площини згину, оскільки несуча здатність на згин зростає значно сильніше за критичний момент втрати стійкості. Сформульовано рекомендації щодо геометричних параметрів дослідних зразків: $l/h = 18..21$; $h/b \le 2,4$ для помірного та $h/b \le 2,0$ для високого рівня армування; висота перерізу $180..600$ мм; кількість ламелей – не менше чотирьох.

Ключові слова: клеєна деревина; поздовжнє армування; геометрія дослідних зразків; чотириточковий згин; FRP

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