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RELIABILITY OF GLUED TIMBER ROOFS BELONGING TO CC3 CLASS CONSEQUENCES OF FAILURE

NIEZAWODNOŚĆ DACHÓW Z DREWNA KLEJONEGO O KLASIE KONSEKWENCJI ZNISZCZENIA CC3

Abstract

The arenas with great grandstands are the public places where the consequences of failure are very high. For this reason, according to EN 1990, they belong to CC3 class consequences of failure. The reliability class RC3 is associated with the consequences class CC3 and is defined by the reliability index β = 4.3 with probability of failure $p_f = 8{,}54*10^{-6}$. The distributions of horizontal and shear resistance within glued timber body – bolts will be described depending on material characteristics of glued timber body and bolts components. The paper will present methods of timber structural design belonging to CC3 consequences failure class.

Keywords: *structural reliability, timber structures, timber joints*

Streszczenie

Konstrukcje budowlane takie jak hale sportowe, widowiskowe, w których konsekwencje awarii są bardzo wysokie zaliczane są wg normy PN-EN 1990 „ Podstawy projektowania konstrukcji" do klasy konsekwencji zniszczenia CC3. Klasie CC3 odpowiada klasa niezawodności RC3, która jest definiowana przez wskaźnik niezawodności β = 4,3. W klasie CC3 definiuje się dopuszczalne prawdopodobieństwo awarii na poziomie $p_f = 8{,}54*10^{-6}$. W pracy przedstawione zostaną metody projektowania elementów konstrukcji przykryć oraz ich połączeń tak aby obiekty te mogły być zakwalifikowane do klasy konsekwencji zniszczenia CC3.

Słowa kluczowe: *bezpieczeństwo konstrukcji, drewno klejone, połączenia drewniane*

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1. Introduction

Safety assessments of timber members require taking into consideration many different reasons. The main reason of glued timber structure failure is connection damage. The shear connection will be discussed in this paper. Shear connections have to transfer forces between structural members – glued timber body and bolts with adequate degree of safety. The load-carrying mechanism of bolted shear connections is complex and analytical methods for predicting the shear resistance are not applicable. Instead, the resistance of the connections may be determined using empirical formulas. The distributions of horizontal and shear resistance within glued timber body – bolts will be described depending on material characteristics of glued timber body and bolt components. The characteristic resistance of timber shear connection is obtained as a minimum of two variables: bolts resistance and steel body resistance. Probability function of these minima will be defined and described in this paper. Laboratory tests provide the only practicable basis for specifying safety margins for ultimate strength connections.

2. Probabilistic distributions of basic strength properties

According to EN-1995-1-1, the design value X_d of a glued timber basic strength property shall be calculated as:

$$X_d = k_{\mathrm{mod}} \frac{X_k}{\gamma_M} \tag{1}$$

where:

- X_k – characteristic value of basic property,
- k_{mod} – modification factor, defined in EN 1995,
- γ_M – partial safety factor for connections defined in EN 1990.

The distribution parameters can be determined up on the information given in Table 1.

Table 1

Distributions of basic properties

Property	Distribution	COV
Bending strength $f_{m,k}$	lognormal	0.19
Bending modulus of elasticity $E_{0,\mathrm{mean}}$	lognormal	0.08
Glued timber density γ_κ	normal	0.07

3. Damage model assumptions

The Eurocode for glued timber structures EC5, EC0 [1, 2], refers to this strength modification with a load duration factor k_{mod}. Traditionally, the load duration factor is determined empirically by experience on glued timber structures, but there are probabilistic methods connected with damage accumulation models to estimate factor k_{mod} as well [3]. The load duration factor k_{mod} is defined in EC5 as a factor which takes into account the effects of load duration and ambient climate on the strength parameters of structural glued timber members. The mechanism, leading to the reduction in strength of a glued timber member under sustained load, is a creep rupture. This could arise from propagation of voids in the microstructure of the timber at a stress level lower than the short-term strength. A number of models of creep rupture, involving a damage state variable (similar to that used in the analysis of metal fatigue), have been proposed to assess damage accumulation in wood structural members subject to loading histories, typically modeled as a renewal pulse process. The basic model has the following form [4]:

$$d\alpha/dt = F\,[\sigma(t)] \tag{2}$$

where:

- t – time,
- α – the damage state variable which ranges from 0 (no damage) to 1 (failure), the function F(.) has two constants that must be determined from test data,
- $\sigma(t)$ – the ratio of the applied stress to the failure stress under short-term loading.

4. Calibration of modification factor k_{mod} for Tatra mountain zone

The occurrence of snow packages at times $X1$, $X2$ is modelled by the Poisson process. The duration between snow packages is exponentially distributed with expected value $1/\lambda$, where: λ – expected number of snow packages per year, $\lambda = 1.95$ snow packages per year.

- The magnitude of the maximum snow load P_m in one snow package (snow pulse) is assumed to be Gumbel distributed (expected value $\mu_P = 0.78$ kN/m^2, standard deviation $\sigma_P = 0.39$ kN/m^2).
- The duration of a snow package T is modelled by $X_T P_m$, proportional to the maximum snow load of snow package X_T – exponentially distributed with expected value $\mu_{XT} = 145$ days/(kN/m^2).
- The time variation of snow packages is assumed to be rectangular.
- The annual maximum snow load on timber roof is determined by:

$$Q_{S,\max} = C\,P_{SG,\max} \tag{3}$$

$P_{SG,\max}$ is annual maximum snow load on the ground and is Gumbel distributed (expected value $\mu_S = 1.86$ kN/m^2 and standard deviation $\sigma_S = 0.43$ kN/m^2). C is shape factor and is assumed Gumbel distributed. Characteristic value of maximum snow load $P_{SG,\mathrm{char}}$ on the ground, was obtained for a $T = 50$ year reference period [6].

In the code format EC5, load duration effect is represented by a modification factor k_{mod}. The following design equation can be found in the codes [7].

$$\frac{z f_k k_{\mathrm{mod}}}{\gamma_m} - \left((1-\kappa)\gamma_G G_k + \kappa\gamma_Q Q_k\right) = 0 \tag{4}$$

Where z is the design variable, f_k and Q_k are the characteristic values of short-term strength and variable load, γ_m, γ_Q are the partial safety factors and κ, the coefficient, $0 \le \kappa \le 1$. G_k, γ_G are characteristic value and partial safety factors of permanent loads.

In the following probabilistic approach [4, 5] all uncertainties related to strength, model and loads are included in a way consistent with the background for the partial safety factors in Polish structural codes. The limit state function for the long-term load carrying capacity can be formulated as:

$$g = 1 - \alpha\left(f_o, G, Q, A, B, T_L, \kappa\right) \tag{5}$$

where:

α	–	is the damage function obtained from (1),
f_o	–	the short term strength,
A, B, C	–	the parameters in damage accumulation model,
G	–	the permanent load,
$Q = Q(t)$	–	the variable load as a function of time.

Taking into account the decrease with time of the strength, the following alternative limit state equation is used:

$$g = z(1-\alpha) f_o - \left((1-\kappa)G + \kappa Q\right) \tag{6}$$

Then k_{mod} factor can be estimated as follows:

$$k_{\mathrm{mod}} = \frac{\gamma_m^S(\beta)}{\gamma_m^L(\beta)} \tag{7}$$

Where β is the reliability index (EC 0) β = 4.8 for Polish standards. The partial safety factor for the long-term strength $\gamma^L_m(\beta)$ is calibrated to a target reliability index β. The partial safety factor for the short -term strength $\gamma^S_m(\beta)$ was obtained using equation (6) with $k_{\mathrm{mod}} = 1$. Table 2 presents k_{mod} factors for different expected duration of snow packages.

Table 2

k_{mod} modification factors for Tatra zone

μ_{Xt} = 145 days/(kN/m^2)	1	0,5	0,25	0,15	0,10	0,05
k_{mod}	0.72	0,74	0,76	0,78	0,80	0,84

5. Modelling of double shear glued timber connections

5.1. Characteristic value of double glued timber connection

The load capacity of two shear glued timber members connections in EN-1995-1-1 models based on the so-called Johansen model. The rigid – plastic model for connecting glued timber elements is assumed. The elastic model of steel is applied for steel fasteners. A few possibilities of fastener damage are being considered. This should be connected with glued timber presser $F_{JH,1}$ $F_{JH,2}$ and with steel fastener bending capacity $F_{JH,3}$ and $F_{JH,4}$. These capacities with line effect $F_{ax,R}$ are described as follows:

$$F_{JH,1} = f_{h,1} t_1 d \tag{8}$$

$$F_{JH,2} = 0.5 f_{h,2} t_2 d \tag{9}$$

$$F_{JH,3} = 1.05 \frac{f_{h,1} t_1 d}{2+\beta}\left[-\beta + \sqrt{2\beta(1+\beta) + \frac{4(\beta(2+\beta)M_{y,R}}{f_{h,1} d t_1^2}}\right] + \frac{F_{ax,R}}{4} \tag{10}$$

$$F_{JH,4} = 1.15 \sqrt{\frac{2\beta}{1+\beta}} \sqrt{2 M_{y,R} f_{h,1} d} + \frac{F_{ax,R}}{4} \tag{11}$$

$$\beta = \frac{f_{h,2}}{f_{h,1}}, \quad M_{y,R} = 0.3 f_u d^{2.6}, \quad f_{h.i} = 0.082(1 - 0.01d)\rho_i, \quad i = 1, 2$$

The values $F_{JH,1}$, $F_{JH,2}$, $F_{JH,3}$, $F_{JH,4}$ are calculated upon 5% fractile basic variables as: glued timber strength $f_{h,i}$, density ρ_i, fastener steel strength f_u. The characteristic value of double glued timber member's connection $F_{v,Rk}$ is obtained as minimum of partial capacities according to EN 1995-1-1:

$$F_{v,Rk} = \min\left(F_{JH,1,k}, F_{JH,2,k}, F_{JH,3,k}, F_{JH,4,k}\right) \tag{12}$$

5.2. Design value of double glued timber connection

Following EN 1995-1-1 design value of glued timber connection $F_{v,Rd}$ was obtained as:

$$F_{v,Rd} = k_{\mathrm{mod}} \frac{F_{v,Rk}}{\gamma_M} \tag{13}$$

Generally, using EN 1990 assumptions, design value of glued timber connection $F_{v,Rd}$ can be described as:

$$F_{v,Rd} = F^{-1}(F_v;\ \Omega, f, f_{conn}, X, T, p_f) \tag{14}$$

where:

- $F^{-1}(.)$ – probability distribution fractile on level $p_f = \Phi^{-1}(\beta)$, β – reliability index for target reliability class according to EN 1990,
- $\Phi(.)$ – Laplace function,
- F_v – random glued timber capacity,
- Ω – kind connection,
- f, f_{conn} – random parameters of strengths.

For CC3 class consequences of failure design value of shear resistance of connection $F_{v,Rd}$ for $T = 50$ year period is defined as follows:

$$F_{v,Rd} = k_{\mathrm{mod}} F^{-1}[F_v;\ \Omega, f_0, f_{conn}, X, p_f = \Phi^{-1}(4.3)] \tag{15}$$

6. Probabilistic model

The variables of glued timber strength f_h and steel fastener strength f_u are random. These random variables are characterized by adequate probability distribution functions $P(.)$ and the first two probabilistic moments $Ex(.)$, $Ex^2(.)$. It is assumed that probability density functions of glued strength and steel fastener strength are log-normal with coefficients of variation $\nu_{fh} = 0.25$ and $\nu_{fh} = 0.07$. The expected values depend on glued timber classes and fastener steel grades. There is a new random variable F_{JH}

– joint capacity defined as a minimum of the partial joint strengths $F_{JH,i}$:

$$F_{JH} = \min_{i=1\ldots4}\left(F_{JH,i}\right) \tag{16}$$

The form and parameters of probability functions of glued timber joint capacity F_{JH} are unknown. The statistical moments $E(.)$, $Var(.)$ of random variable F_{JH} were established upon Monte Carlo methods specifying histogram $P^{MC}(F_{JH})$. The form of probability function of F_{JH} is conveniently defined by minimum cross-entropy H condition [8].

$$H(P^{MC}, P^*;\ f_i) = \sum_i P^*(f_i) \ln \frac{P^*(f_i)}{P^{MC}(f_i)} \tag{17}$$

The characteristic values $F_{JH,k}$, design value for CC3 classes of consequence of failure $F_{JH,d}$ partial safety factor are obtained by using the following formulas:

$$F_{JH,k} = P^{*-1}[\Theta(-1.64)], \quad F_{JH,d} = P^{*-1}[\Theta(-4.3)], \quad \gamma_M = F_{JH,k} / F_{JH,d} \tag{18}$$

Example

Three examples of glued timer GL 24 h member double shear connections are considered. The steel fastener thicknesses are T1, 2T1, T1 with class 4.8 and diameter *D*. Three cases of thicknesses T1 = 50 mm, 100 mm, 150 mm and two cases of diameters *D* = 10 mm, 16 mm are taken into account. The subscript "1" is connected with joint capacity described by using formula (8), subscript "2" – formula (10) and subscript "3" (11). Total capacity of glued joint is without subscript. The example presents the cases with: positive skewnes – *G* – Gumbell (Frechet) probability distributed, neutral skewness – *N* – (Normal) distributed and negative skewness (Weibull) *W*. The Table 3 shows characteristics of random vectors generated using the Monte Carlo method – *E*(.) – expected value, *COV*(.) – Coefficient of Variation for tree options (*A*), (*B*), (*C*). Table 4 shows final results of joint capacity estimation, columns 6, 7 represent the characteristic and design values calculated using probabilistic methods and columns 8, 9 – using Eurocode procedures.

Table 3

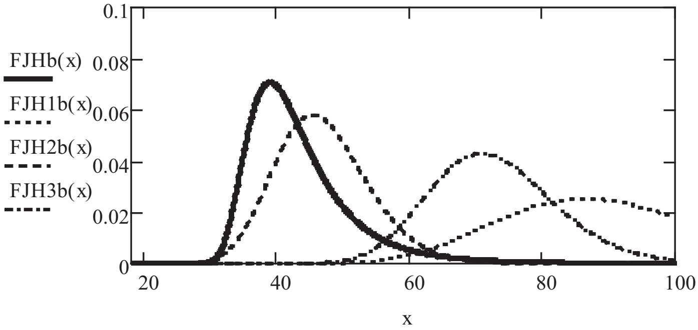
Probabilistic partial joint capacity parameters

	D1 [mm]	T1 [mm]	*E*(1) [kN]	*COV*(1)	*E*(2) [kN]	*COV*(2)	*E*(3) [kN]	*COV*(3)
(*A*)	16	100	52.11	0.19	40.93	0.13	72.62	0.12
(*B*)	16	150	89.2	0.18	46.43	0.15	72.12	0.13
(*C*)	10	50	16.61	0.20	19.68	0.11	33.43	0.11

Table 4

The glued timber joint capacity parameters

1	2	3	4	5	6	7	8	9
Option	Expected value [kN]	Coefficient of variation	Skewness	Prob. distr.	$F_{JH,k}$ [kN] Prob.	$F_{JH,d}$ [kN] Prob.	$F_{v,Rk}$ [kN] EN	$F_{v,Rd}$ [kN] EN
(A)	35.91	0.14	0.07	*N*	41.63	34.27	44.48	37.37
(B)	39.65	0.16	0.68	*G*	42.23	40.29	45.89	44.57
(C)	13.97	0.18	–0.19	*W*	14.36	11.47	12.34	15.89



Ill. 1. Minimization of random variables of timber glued joint capacity

7. Conclusions

The probabilistic models for the glued timber joints with dowels in double shear have been formulated in such a way that they can be easily applied in structural reliability analysis. It has been noticed that a significant effect was found in the time variation of snow impulses-packages on the accumulated damage. Therefore the observed snow packages are quite different and the triangular and rectangular time variations are included in the present probabilistic calibration of load duration factors. The probability functions of capacity of timber joints in double shear connections were estimated with the entropy criterion. The target reliability index equal 1.64 and 3.4 was used to obtain characteristic and design capacity of double shear joints. More research is needed on the variance parameters, as found in practice, of the glued timber characteristic joints and on the assumptions of the shape of capacity probability distribution of glued timber joints.

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