

The Stability of Wood Composite Wall Panels with Elastically Deformable Mechanical Links



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Abstract Composite panels with wooden ribs and covers, made of board materials —are the universal enclosing and supporting structures. When such structures are subjected to vertical loads, the main part of the load is absorbed by the vertical load-bearing ribs. Due to the shear resistance of screw connections, part of the load can be absorbed by the sheathing. With relatively large spacing between the fixing points at floor level, an important factor in determining the required cross-sectional dimensions of ribs is to ensure their stability from the wall plane; and considering the joint operation of ribs and sheathing allows the critical compressive force to be significantly increased. The feature of such structures calculating is the necessity to take into account the ties ductility at the seams of the layers. Because of it, the critical force cannot be calculated by the classical Euler formula as for a composite structure with rigid bonds. The presented mathematical model is based on the solution of determinant equation which is obtained from differential equation system of 3-layer composite element considering both mutual shift of layers and shifts of elastic axis from vertical in a given form. The resulting expression allows to determine the value of critical force taking into account major parameters of the composite structure. Considering the sheathing in the framing operation allows increasing the critical force by 70%, which is mainly influenced by the stiffness of the shear bonds. Increasing the thickness of covers does not have the same tangible effect.

Keywords Wood · Wood composite panels · Compression · Stability · Semirigid connections

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1 Introduction

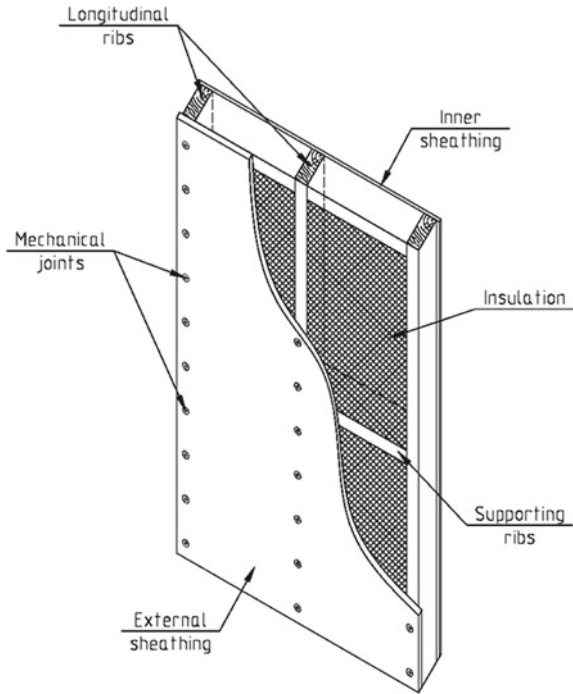
Composite structures, made of various materials, are widely used in the construction of civil building, industrial and agricultural structures [1, 2]. Composite panels with wooden ribs and covers, made of board materials—are the universal enclosing and supporting structures that can be used as covering, floors, wall envelope; and also as the main load-bearing elements in wooden-frame building. Such elements perform the functions of beams, flooring, lining, can provide thermal protection of the building (panels with the insulation inside the structure), and perform the function of horizontal stiffness diaphragms. There are numerous studies devoted to improving the performance of such panels. A new method is proposed in [3] for joining the outer and inner layers of 3-layer panels, which makes it possible to significantly increase the resistance of joints to longitudinal and transverse shear. The bent strength of prefabricated reinforced concrete wall panels with insulation using various connections between the outer bearing reinforced concrete layers is studied in the article [4]. The research for the ultimate compressive load and analysis of the stability of carbon fiber and glass fiber panels with stiffening ribs «Z»-profile and «L»-profile is presented in [5]. The results of studies of shear strength between layers of modular sandwich panels with a sheath made of high density polyethylene depending on the stamping method are presented in article [6]. The development of wood-based materials and structures base has made it not only technically possible but also economically feasible to use ribbed wall panels and volumetric modules in buildings and structures for various purposes [7–9].

The advantages of such panels (Fig. 1) are most evident when it combines load-bearing and enclosing functions, while the main longitudinal ribs take on racks' role, and the covers take on enclosing layers' role.

It is possible to increase the load-carrying capacity of the ribs by effectively engaging the covering panels, for which a rigid adhesive bond at the «rib-sheathing» boundary is traditionally used. At the same time, the use of adhesive bonds significantly complicates the technological process of wall panels' production, which contradicts the main idea of using such structures—simplicity and low production cost. A number of experimental and theoretical studies [10] demonstrated the advisability of taking the sheathing into account when jointing it to the ribs, using semi-rigid mechanical ties. Nails, screws, staples or combined connections based on claw washers are used as mechanical ties. Studies [11] present that taking the participation of the covers due to the screw ties into account gives a significant effect on the stiffness increasing of wall panels during shear when it serve as vertical diaphragms.

However, additional research is needed on the behavior of composite panels, where the ribs are the main load-bearing elements of the frame, and on the influence of the thickness of the covers and the stiffness of the ties on the strength and stability of such panels.

Fig. 1 Ribbed wood composite wall panel



2 Models and Methods

In the engineering design of timber framed ribbed panels it is permissible to consider reduced T- or I-sections where the sheathing acts as the flanges. The normal stresses along the axis of the panel arising in the flanges will be greatest at the ribs and reduce as they move away from the ribs. In the middle of the sheathing the stresses will have a minimum value. Therefore, the reduced width of the sheathing (Fig. 2), rather than the actual width, must be included in the calculation so that the values of the highest longitudinal stresses for the actual and reduced cross-sections are equal [12–14].

According to [15] the possibility of loss of stability in the calculation of centrally-compressed rods is taken into account by the introduction of a bending coefficient, from the condition:

$$\frac{N}{F \cdot \varphi} \leq R_c, \tag{1}$$

where F is the cross-sectional area;

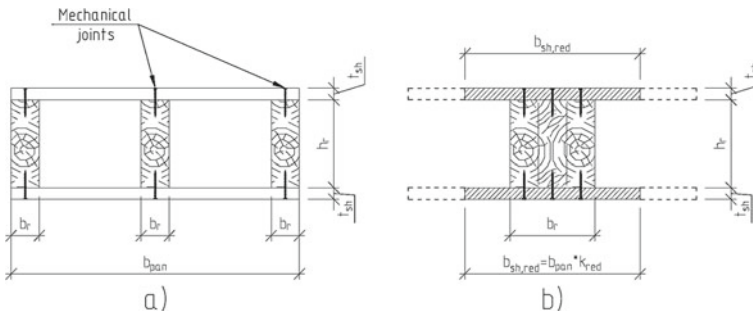


Fig. 2 Schematic of the calculated reduced panel cross-section: **a**—actual cross-section; **b**—design cross-section

φ is the longitudinal bending coefficient determined according to the formula:

$$\varphi = \frac{\sigma_{cr}}{R_c}, \tag{2}$$

where σ_{cr} is the value of compressive stresses corresponding to the loss of stability (critical force N_{cr}).

In order to determine the bending coefficient, it is necessary to determine the fraction of the load that falls on the strut when the critical force N_{cr} is applied. The system of equations for a three-layer composite element [16–21] is presented below:

$$\begin{cases} \frac{T_1''}{\xi_1} = \Delta_{11}T_1 + \Delta_{12}T_2 + \Delta_{10} \\ \frac{T_2''}{\xi_2} = \Delta_{21}T_1 + \Delta_{22}T_2 + \Delta_{20} \end{cases}, \tag{3}$$

where T_i is the force at the joints of the composite bar,

Δ_{ij} , Δ_{iy} are coefficients at the unknowns;

Δ_{i0} are free terms;

ξ_i is the stiffness factor of the shear links in the i -joint, determined by Formula (12).

$$\begin{aligned} \Delta_{11} &= \frac{1}{E_1 \cdot A_1} + \frac{1}{E_2 \cdot A_2} + \frac{c_1^2}{\Sigma EI}; \\ \Delta_{22} &= \frac{1}{E_2 \cdot A_2} + \frac{c_1^2}{\Sigma EI}; \\ \Delta_{12} = \Delta_{21} &= \frac{-1}{E_2 \cdot A_2} + \frac{c_1 c_2}{\Sigma EI}; \end{aligned} \tag{4}$$

$$\begin{aligned} \Delta_{10} &= \frac{-N_1^0}{E_1 \cdot A_1} + \frac{N_2^0}{E_2 \cdot A_2} + \frac{M^0 \cdot c_1}{\Sigma EI}, \\ \Delta_{20} &= \frac{-N_2^0}{E_2 \cdot A_2} + \frac{N_3^0}{E_3 \cdot A_3} + \frac{M^0 \cdot c_2}{\Sigma EI}, \end{aligned} \tag{5}$$

where E_1, A_1, E_3, A_3 —modulus of elasticity and cross-section area of outer and inner sheathing respectively; E_2, A_2 —the same, ribs; c_1, c_2 —distance from centre of gravity of outer/inner sheathing to rib centre respectively; N_{0i} —longitudinal load of i -th element; M^0 —total bending moment from outer load; ΣEI —bending stiffness of a zero shear joint rod.

As we consider the case of central compression and consider the application of external load on the strut with its redistribution onto the sheathing only by the operation of mechanical connections, we assume in further calculations that $M^0 = N_1^0 = N_3^0 = 0$.

The system of Eqs. (3) can be represented by two independent equations for the generalized unknown joint forces \bar{T}_{12} :

$$\bar{T}_1'' = \lambda_1^2 \bar{T}_1 + \bar{R}_1; \quad \bar{T}_2'' = \lambda_2^2 \bar{T}_2 + \bar{R}_2 \tag{6}$$

where $\bar{R}_{1,2}$ are the generalized load terms defined by expressions (7);

$\lambda_{1,2}$ —the characteristic numbers determined by (8).

$$\begin{aligned} \bar{R}_1 &= \sqrt{\xi_1} \Delta_{10} \cos \varphi + \sqrt{\xi_2} \Delta_{20} \sin \varphi; \\ \bar{R}_2 &= -\sqrt{\xi_1} \Delta_{10} \sin \varphi + \sqrt{\xi_2} \Delta_{20} \cos \varphi, \end{aligned} \tag{7}$$

$$\lambda_{1,2} = \sqrt{\frac{1}{2} \left(\xi_1 \Delta_{11} + \xi_2 \Delta_{22} \pm \sqrt{(\xi_1 \Delta_{11} - \xi_2 \Delta_{22})^2 + 4 \Delta_{12}^2 \xi_1 \xi_2} \right)}. \tag{8}$$

The functions of the generalised unknown forces in the joints and $\bar{T}_1 \bar{T}_2$ are of the form:

$$\begin{aligned} \bar{T}_1 &= A_1 sh(\lambda_1 x) + B_1 ch(\lambda_1 x) + \int_0^x R_1(t) sh[\lambda_1 \cdot (x - t)] dt; \\ \bar{T}_2 &= A_2 sh(\lambda_2 x) + B_2 ch(\lambda_2 x) + \int_0^x R_2(t) sh[\lambda_2 \cdot (x - t)] dt. \end{aligned} \tag{9}$$

The expressions bellow are used to move from the functions \bar{T}_1 and \bar{T}_2 to the shear forces $T_1(x)$ and $T_2(x)$:

$$\begin{aligned} T_1(x) &= \sqrt{\xi_1} \cdot \cos \varphi \cdot \bar{T}_1 - \sqrt{\xi_1} \cdot \sin \varphi \cdot \bar{T}_2; \\ T_2(x) &= \sqrt{\xi_2} \cdot \sin \varphi \cdot \bar{T}_1 - \sqrt{\xi_2} \cdot \cos \varphi \cdot \bar{T}_2. \end{aligned} \quad (10)$$

The angle φ is determined by the formula:

$$\varphi = \text{arctg} \left(\frac{\lambda_1^2 - \xi \cdot \Delta_{11}}{\xi \cdot \Delta_{12}} \right). \quad (11)$$

The joint stiffness coefficients ξ_i are determined according to the formula:

$$\xi_i = \frac{n \cdot c_{c,i}}{S_{c,i}}, \quad (12)$$

where $c_{c,i}$ is the stiffness factor of the joints in the i -th joint, determined experimentally;

$S_{c,i}$ —spacing of connections; n —amount of longitudinal ribs.

Substituting expressions (4) and (12) into (12) and integrating the right part of the equations, the following equalities are obtained:

$$\begin{aligned} \bar{T}_1(x) &= A_1 sh(\lambda_1 x) + B_1 sh(\lambda_1 x) \\ &\quad - \left[\frac{(\sqrt{\xi_1} \cdot \Delta_{10} \cdot \cos \varphi + \sqrt{\xi_2} \cdot \Delta_{20} \cdot \sin \varphi) \cdot (ch(\lambda_1 x) - 1)}{\lambda_1^2} \right], \end{aligned} \quad (13)$$

$$\begin{aligned} \bar{T}_2(x) &= A_2 sh(\lambda_2 x) + B_2 sh(\lambda_2 x) \\ &\quad - \left[\frac{(\sqrt{\xi_1} \cdot \Delta_{10} \cdot \cos \varphi + \sqrt{\xi_2} \cdot \Delta_{20} \cdot \sin \varphi) \cdot (ch(\lambda_1 x) - 1)}{\lambda_2^2} \right], \end{aligned} \quad (14)$$

The arbitrary constants A_i and B_i are determined from the boundary conditions (shear forces at the supports without shear hindrances is equal to 0):

$$x = 0 \rightarrow \bar{T}_1(0) = \bar{T}_2(0) = 0; x = l \rightarrow \bar{T}_1(l) = \bar{T}_2(l) = 0.$$

Substituting boundary conditions into expressions (13) and (14), the following expression is obtained: $B_1 = B_2 = 0$,

$$A_1 = \frac{\sqrt{\xi_1} \Delta_{10} \cos \varphi \cdot (1 - chl\lambda_1) + \sqrt{\xi_2} \Delta_{20} \sin \varphi \cdot (1 - chl\lambda_1)}{\lambda_1^2 \cdot shl\lambda_1}; \quad (15)$$

$$A_2 = \frac{\sqrt{\xi_2} \Delta_{20} \cos \varphi \cdot (1 - chl\lambda_2) - \sqrt{\xi_1} \Delta_{10} \sin \varphi \cdot (1 - chl\lambda_1)}{\lambda_2^2 \cdot shl\lambda_2}. \quad (16)$$

Substituting values A_1 and A_2 in (13) and (14) and then the values obtained into expressions (10) makes it possible to determine the shear forces T_i in any cross-section along the height of the post. The normal stresses in the ribs are determined by the formula:

$$\sigma_r = \frac{1}{n_r} \left(\frac{-T_1 + T_2}{A_r} \pm \frac{M_r \cdot 0.5 \cdot t_r}{I_r} \right), \tag{17}$$

where M_p is the moment in the rib resulting from the work of the shear bonds, determined by formula (18) (for a symmetrical three-layer panel, $M_p = 0$); n_t is the number of longitudinal ribs.

$$M_r = - \frac{(T_1 \cdot c_1 + T_2 \cdot c_2) \cdot E_r \cdot I_r}{\Sigma EI} \tag{18}$$

To determine the value of N_{cr} , consider a strut as a composite 3-ply compressed rod, the outer layers of which are sheathing and the inner layers are branches (Fig. 3). The shear bonds in the 2 outside seam are ductility (nails, screws) or rigid (glue) ties. The system of differential equations of a composite compressed rod with account for the longitudinal bend has form:

$$\begin{cases} T_1''/\xi_1 = \Delta_{11}T_1 + \Delta_{12}T_2 + \Delta_{1y}y + \Delta_{10} \\ T_2''/\xi_2 = \Delta_{21}T_1 + \Delta_{22}T_2 + \Delta_{2y}y + \Delta_{20} \\ \Sigma N^0 y'' = \Delta_{y1}T_1 + \Delta_{y2}T_2 + \Delta_{yy}y + \Delta_{y0} \end{cases}, \tag{19}$$

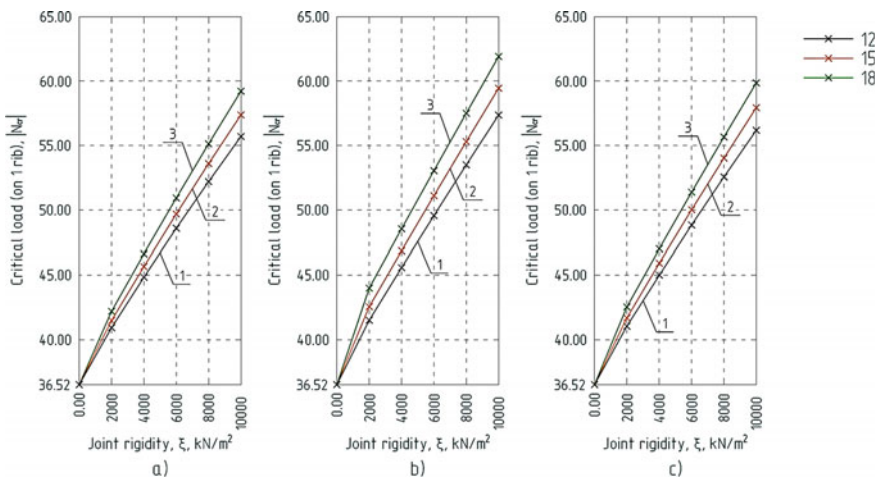


Fig. 3 Dependencies of $[N_{cr}]$ for a single rib on the stiffness of the bracing cladding to ribs: **a**—OSB cladding; **b**—FRC; **c**—particle board; 1— $t_{ob} = 12$ mm; 2— $t_{ob} = 15$ mm; 3— $t_{ob} = 18$ mm

Here:

$$\Delta_{yi} = \Delta_{yi} = \frac{\Sigma N^0}{\Sigma EI} c_i; \quad \Delta_{yy} = \frac{(\Sigma N^0)^2}{\Sigma EI}; \quad \Delta_{y0} = \frac{\Sigma N^0}{\Sigma EI} M^0, \quad (20)$$

where y is the deflection of the element;

ΣN_0 —total longitudinal load on the panel.

Since $M^0 = 0$, the coefficients Δ_{i0} and Δ_{y0} are 0, the system is homogeneous. Its solution is $T_i = 0, y = 0$. However, for some total values of the longitudinal force the homogeneous system has solutions other than zero which correspond to the forms of loss of stability of the compressed composite rod. These values of the total longitudinal force ΣN_0 are critical. At articulated end restraint of the rack the boundary conditions are: $x = 0 \rightarrow T_i = 0, y = 0; x = l \rightarrow T_i = 0, y = 0$. The system of differential equations under these boundary conditions has solution [12]:

$$T_i = \alpha_i \sin \chi x; \quad y = \alpha_y \sin \chi x, \quad (21)$$

where $\chi = k\pi/l$, k is a positive integer that takes into account the form of stability loss;

α_i, α_y are constant coefficients.

The system of equations below is obtained by substituting expressions (21) into (19) and reducing by $\sin \chi x$:

$$\begin{cases} \left(\Delta_{11} + \frac{\chi^2}{\xi_1} \right) \alpha_1 + \Delta_{12} \alpha_2 + \frac{\Sigma N^0}{\Sigma EI} c_1 \alpha_y = 0 \\ \Delta_{21} \alpha_1 + \left(\Delta_{22} + \frac{\chi^2}{\xi_2} \right) \alpha_2 + \frac{\Sigma N^0}{\Sigma EI} c_2 \alpha_y = 0 \\ \frac{\Sigma N^0}{\Sigma EI} c_1 \alpha_1 + \frac{\Sigma N^0}{\Sigma EI} c_2 \alpha_2 + \left[\frac{(\Sigma N^0)^2}{\Sigma EI} + \Sigma N^0 \chi^2 \right] \alpha_y = 0 \end{cases} \quad (22)$$

The solution of this homogeneous system of ordinary equations in which α_i, α_y are independent is different from zero only when its determinant is zero. By reducing the system by $(\Sigma N_0)^2/\Sigma EI$, we obtain the equation:

$$\begin{vmatrix} (\Delta_{11} + \chi^2/\xi_1) & \Delta_{12} & c_1 \\ \Delta_{21} & (\Delta_{22} + \chi^2/\xi_2) & c_2 \\ c_1 & c_2 & [\Sigma EI + \chi^2(\Sigma EI)^2/\Sigma N^0] \end{vmatrix} = 0. \quad (23)$$

Expanding the determinant with respect to ΣN_0 , the formula for the critical force ($N_{cr} = \Sigma N_0$) determination is obtained:

$$N_{cr} = - \frac{\chi^2 \Sigma EI^2 \cdot (\chi^4 + \chi^2 \xi_1 \Delta_{11} + \chi^2 \xi_2 \Delta_{22} + \xi_1 \xi_2 \Delta_{11} \Delta_{22} - \xi_1 \xi_2 \Delta_{12}^2)}{\left(\chi^2 (\chi^2 \Sigma EI - c_1^2 \xi_1 - c_2^2 \xi_2 + \xi_1 \Delta_{11} \Sigma EI + \xi_2 \Delta_{22} \Sigma EI) - \xi_1 \xi_2 \cdot (c_2^2 \Delta_{11} + c_1^2 \Delta_{22} - 2c_1 c_2 \Delta_{12} - \Delta_{11} \Delta_{22} \Sigma EI + \Delta_{12}^2 \Sigma EI) \right)} \tag{24}$$

The minimum values of the critical force are corresponding to $k = 1$, which occurs while the rod bends by one half-wave of the sine wave form of longitudinal bending.

3 Results and Discussion

There is a timber-framed wall panel, measuring 1.0×3.0 m, with three longitudinal ribs (pine wood). The ribs are hinged at the ends. The cross-section of the ribs is 40×100 mm. The covers is connected to the ribs by ductility screw ties. The panel ribs are subjected to longitudinal compressive load transmitted from the upstream panels without eccentricity. There are considered panels with double-sided cladding of 12-, 15- and 18-mm thickness made of OSB/3 oriented chipboard, structural plywood and cement-bonded particleboard. Critical force values and bending coefficients are determined depending on bond stiffness, covers thickness and material. The designed width of the covers according to [22] is $0.9 \text{ bp} = 0.9$ m. The results of calculating the critical force $|N_{cr}|$ are shown in the graphs in Fig. 3. The values are given for one rib. The critical force at $\xi = 0$ is determined for the rib without consideration of the sheathing, by the Euler formula.

The presented method makes it possible to determine the value of critical force for the ribs included in the compressed wood-composite wall panels. After determining the values of the critical force $|N_{cr}|$ according to the Formula (2), it is possible to determine the coefficient of longitudinal bending φ taking into account the participation of the panel covers, and according to the Formula (1) to make a simplified engineering calculation of such structures.

Covers participation together with ribs consideration under compression force can increase the strength and stability of the structure [23, 24]. Depending on the stiffness of the mechanical bonds, the increase in critical force is found to be up to 62% with oriented structural board (OSB) sheathing; up to 69% with plywood (wood-laminated board, glued with the help of urea–formaldehyde composition sheathing) and up to 64% with cement bonded particle board (CBPB). It is evident from the graphs in Fig. 3 that an increase in shear ties stiffness gives a significantly greater increase in $|N_{cr}|$ values than an increase in sheath thickness. Thus, an increase in connection stiffness from 0 to $10,000 \text{ kN/m}^2$ results in increase of $|N_{cr}|$ values by 52–69%, while an increase of cover thickness by 1.5 times (from 12 to 18 mm)— by 6–8%. The use of plywood cladding gives the greatest effect of increasing $|N_{cr}|$, up to 4.5% compared with OSB, up to 2.6% with CBPB.

4 Conclusions

1. The presented mathematical model makes it possible to estimate the stress–strain state of timber-framed wall panels and determine the value of the critical force, taking into account the covers, involved by connection to the ribs using ductility mechanical ties.
2. The two-sided covering of the ribs can significantly increase the critical force value $|N_{cr}|$. The greatest increase in the critical force $|N_{cr}|$ is observed when the mechanical shear ties fixing the sheathing to the ribs are rigid. Increasing the thickness of the sheathing as well as the material used (OSB, plywood, CBPB) has slight effect.
3. In order to increase the strength and stability of timber-wall panels, it is first necessary to reduce the shear resistance of the mechanical ties at the «rib-cover» seams, which can be achieved both by using more shear-resistant ties and by reducing the spacing of the ties.

References

1. Sergeev M, Rimshin V, Lukin M, Zdralovic N (2020) Proceedings conference on multi-span composite beam. IOP Conf Ser Mater Sci Eng (Vladimir). <https://doi.org/10.1088/1757-899X/896/1/012058>
2. Roshchina S, Lukin M, Lisyatnikov M (2020) Compressed-bent reinforced wooden elements with long-term load. Lect Notes Civil Eng 70:81–91. https://doi.org/10.1007/978-3-030-42351-3_7
3. Naik RK, Panda SK, Racherla VA (2020) New method for joining metal and polymer sheets in sandwich panels for highly improved interface strength. J Compos Struct 251:112661
4. Gombedaa MJ, Naito CJ, Quiel SE (2021) Flexural performance of precast concrete insulated wall panels with various configurations of ductile shear ties. J Build Eng 33:101574
5. Elumalai ES, Krishnaveni GR, Sarath Kumar D, Xavier D, Kavitha G, Seralathan S, Hariram V, Micha Premkumar T (2020) Buckling analysis of stiffened composite curved panels. Mater Today Proc 33(Part 7):3604–3611. <https://doi.org/10.1016/j.matpr.2020.05.662>
6. Tahmoorian F, Nematı S, Sharafı P, Samalı B, Khakpour S (2021) Punching behaviour of foam filled modular sandwich panels with high-density polyethylene skins. J Build Eng 33:101634
7. Kavelin AS (2014) Proceedings conference on Sovremennyye problemy promyshlennogo i grazhdanskogo stroitel'stva [Modern problems of industrial and civil construction]. Stroitel'stvo, Rostov, pp 98–100. https://elibrary.ru/download/elibrary_24672247_47513863.pdf (Date of application 29.05.2021)
8. Zhadanov VI, Charikova VV, Xoroshavin EA (2016) Proceedings conference on Konstruktivnye resheniia maloetazhnykh zdaniı na osnove drevesiny, primeniayemye v Finliandii [Structural solutions for low-rise buildings based on wood used in Finland]. Universitetskii kompleks kak regional'nyi tsentr obrazovaniia, nauki i kul'tury, Orenburg, pp 520–525
9. Piatikre-stovskii KP, Travush VI (2016) Paneli dlia sten zhilykh domov i pokrytii razlichnykh zdaniı iz drevesiny [Panels for walls of residential buildings and coverings of various buildings made of wood]. Zhilishchnoe stroitel'stvo, Moscow, No 4, pp 44–47

10. Popov EV, Filippov VV, Melekhov VI, Labudin BV, Tiurikova TV (2016) Vliianie zhestkosti svyazei sdviga pri raschete rebristyykh panelei na dereviannom karkase [Influence of shear bond stiffness in the calculation of ribbed panels on a wooden frame]. *Lesnoi zhurnal, Arkhangel'sk*, No 4, pp 123–134
11. Baszen M (2017) Semi-rigid behavior of joints in wood light-frame structures. *Procedia Eng* 172:88–95. <https://doi.org/10.1016/j.proeng.2017.02.022>
12. Rzhnitsyn AR (1986) *Sostavnye sterzhni i plastinki* [Composite rods and plates]. Stroizdat, Moscow, 314 p. https://pnu.edu.ru/media/filer_public/2013/04/10/3-9_pzhanicin_1986.pdf (Date of application 29.05.2021)
13. Naychuk AY (2013) Estimation of load-bearing capacity and stiffness of timber beams with through-thickness cracks. *Adv Mater Res*. <https://doi.org/10.4028/www.scientific.net/AMR.778.361>
14. Turkovskij SB, Pogorel'tsev AA (2001) Wooden structures with rigid joints in structures with corrosive medium. *Promyshlennoe i Grazhdanskoe Stroitel'stvo* 10–13
15. Roshchina SI, Lukin MV, Lukina AV, Sergeev MS, Lisatnikov MS (2015) Experimental research on pressed-bending reinforced timberwork. *Int J Appl Eng Res* 10:45307–45312
16. Lukin M, Prusov E, Roshchina S, Karelina M, Vatin N (2021) Multi-span composite timber beams with rational steel reinforcements. *Buildings*. <https://doi.org/10.3390/buildings11020046>
17. Griбанov AS, Roshchina SI, Naichuk AY, Melekhov VI (2020) Wooden beams with local wood modification. *IOP Conf Ser Mater Sci Eng*. <https://doi.org/10.1088/1757-899X/896/1/012067>
18. Azinović B, Serrano E, Kramar M, Pazlar T (2018) Experimental investigation of the axial strength of glued-in rods in cross laminated timber. *Mater Struct Constr* 51. <https://doi.org/10.1617/s11527-018-1268-y>
19. Lukina A, Roshchina S, Griбанov A (2021). Method for restoring destructed wooden structures with polymer composites. https://doi.org/10.1007/978-3-030-72404-7_45
20. Aleksiiyevets VI, Aleksiiyevets II, Ivaniuk AM, Roshchina SI (2020) Load-carrying capacity of bolted joints of timber structures under static loading. *IOP Conf Ser Mater Sci Eng*. <https://doi.org/10.1088/1757-899X/896/1/012043>
21. Turkov AV, Makarov AA (2016) Experimental research of cross beams systems of wooden elements for the static and dynamic loads
22. DIN EN 1995–1–1/A2–2014 Eurocode 5: design of timber structures—Part 1–1: general—common rules and rules for buildings; German version EN 1995–1–1:2004/A2:2014
23. Lukina A, Roshchina S, Griбанov A (2021) Method for restoring destructed wooden structures with polymer composites. Presented at the 2021. https://doi.org/10.1007/978-3-030-72404-7_45
24. Lukin M, Sergeev M, Lisatnikov M (2021) Non split wooden beam reinforced with composite reinforcement. Presented at the 2021. https://doi.org/10.1007/978-3-030-72404-7_12