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Design of composite skin panel for roof in accordance with the requirements EN 1995-1-1 and ДБН В.1.2.-2:2006

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In the article design of composite (timber and plywood) skin panel has been considered according European and national standards.
The panel consists of timber webs and plywood skin. The design parameters are depth of webs and distance between webs within the defined skin parameters.
In design the method of fictitious cross-section is applied. Check of the ultimate limit states is conducted in accordance with EN 1995-1-1 (ДБН В.2.6-161:2017). Loads to the panel are determined according to ДБН В.1.2.-2:2006.
The usage of European and national standards complies with the actual requirements and widen methodological base.

Keywords: roof members; two-sided composite skin panel, design of building constructions, loads on building constructions

Проектування клеєфанерної панелі покриття у відповідності до вимог EN 1995-1-1 і ДБН В.1.2.-2:2006

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В статті розглянуто проектування композитної панелі покриття у відповідності до європейських і національних будівельних норм.
Панелі композитного типу (дерев’яні ребра і обшивка з фанери або OSB) часто застосовуються для покриття легкого типу. Для конструкцій, які мають у своєму складі матеріали з різними фізико-механічними властивостями, при проектуванні застосовується метод зведеного перерізу, що значно спрощує розрахунки. Оскільки обшивки панелі знаходяться в більш напруженому стані ніж ребра, матеріал ребра (деревина) приводиться до матеріалу обшивок за допомогою коефіцієнта приведення. Коефіцієнт приведення знаходиться як відношення модулів пружності матеріалів. Як приклад, в статті наведено конструктивний розрахунок за першим граничним станом.
Остання редакція національних норм щодо розрахунку дерев’яних конструкцій (ДБН В.2.6-161:2017, [9]) повністю базується на європейських будівельних стандартах EN 1995-1-1, [8]. З цих міркувань в статті розглядається проектування конструкції відповідно до оригінального джерела. Натомість визначення характеристик міцності рекомендується виконувати відповідно до національних норм ДБН В.2.6-161:2017, які одночасно виконують функції європейського стандарту EN 338. Навантаження на панель визначаються відповідно до ДБН В.1.2.-2:2006, що є не тільки традиційним в національній методології проектування будівельних конструкцій, але й на думку авторів більше відповідає критеріям надійності.
Застосування в розрахунках одночасно національних і європейських проектних норм відповідає сучасним вимогам і поширює методологічну базу.

Ключові слова: клеєфанерна панель покриття, проектування будівельних конструкцій, навантаження на будівельні конструкції

Introduction. Nowadays new building codes adapted to European ones are used in Ukraine. There is a need to adapt the design methodology for the elements of timber structures.

Two-sided composite (timber and plywood or OSB) skin panel are most common. This type of panels design as a typical glued thin flanged internal I-beam with flanges on the top and bottom faces and subjected to a moment.

Review of the latest research sources and publications. Design methods of composite skin panels for products in USA, Canada, Europe are considered in publications [1,2,3,4]. As the flanges are thin, the stress in each flange due to bending is effectively an axial stress and the design value is taken to be the average value across the flange thickness. In calculating section properties for stressed-skin panels, the designer must take into account the composite nature of the unit. Unless all materials in the panel have similar moduli of elasticity, some method must be employed to make allowance for the differences. Different moduli of elasticity may be reconciled by the use of a «transformed section». The transformed-section approach is common to structural design of composite sections. It consists of «transforming» the actual section into one of equivalent strength and stiffness, but composed of a single material. Sections are generally transformed to the material of the most highly stressed portion of the panel.

The design requirements for the web are that it must be able to support the flexural stresses that arise, that the shear stress in the web must be acceptable, and that the glued joints between the web and the flanges must be able to transfer the horizontal shear stresses at the interface.

Definition of unsolved aspects of the problem. European design recommendations are based on the Eurocode load standards [5, 6]. Loads for building Ukrainian constructions can be used according rules of Ukrainian standard ДБН В.1.2.-2:2006 [7]. Dead loads determined rules of *chapter 5* of ДБН В.1.2.-2:2006 [7]. Snow (variable) loads determined rules of *chapter 8* of ДБН В.1.2.-2:2006 [7].

Problem statement. This article discusses the possibility of using the Ukrainian standards for loads on building constructions and European standards for design timber structure elements.

Basic material and results. Consider designing of composite skin panel accordance with the requirements EN 1995-1-1 [8]. Typical two-sided composite (timber and plywood) skin panel is used as a roof member for different types of roofing materials. Plywood is used for top and bottom skins of panel and timber is used for webs of panel.

For design of composite skin panel can be used rules of Ukrainian standard ДБН В.2.6-161:2017 [9] which is based on rules EN 1995-1-1. In Ukraine should be use characteristic values of strength classes according ДБН В.2.6-161:2017 [9].

The effective flange width concept applies to flanges in compression and in tension, and unless a more detailed calculation is carried out, in accordance with the requirements of 9.1.2(3) [8]. The effective flange width, b_{ef}, as shown in Figure 1, for internal I-shaped sections, will be as follows ([8] *eq. (9.12)*):

$$b_{ef.c} = b_{c.ef} + b_w \text{ and } b_{ef.t} = b_{t.ef} + b_w .$$

Effective flange width of an I-beam section of the panel ([8], *9.1.2*):

- in compression ([8], *Table 9.1*),

$$b_{c.ef} \le min\{ 0,1 \cdot L ; 20 \cdot d_{tf} \};$$

- in tension ([8], *Table 9.1*),

$$b_{t.ef} \le 0,1 \cdot L \text{ (only shear lag).}$$

Designations accepted in the formulas: $b_{c.ef}$ – design width of the flange in compression; $b_{t.ef}$ – design width of the flange in tension; b_w – design width of the web; L – span of panel; d_{tf} – top flange thickness; d_{bf} – bottom flange thickness.

The values $b_{c.ef}$ and $b_{t.ef}$ should not be greater than clear distance between webs b_f .

Panel has a cross-section of two different materials it is convenient to go transformed section or fictitious section.

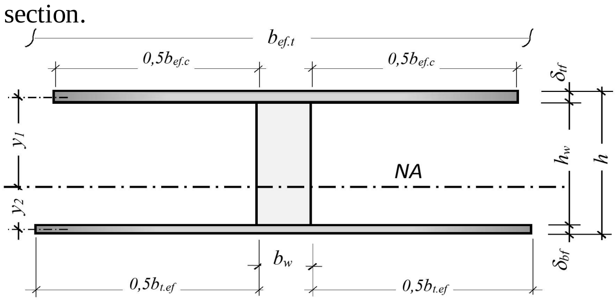

Figure 1 – Transform cross-section as I-beam.

Coefficient for transformed section, as ratio of modulus:

$$n_E = E_{0.mean} / E_{plw.0.mean} . \qquad (1)$$

Transformed web thickness (into plywood):

$$b_{w.tfd} = b_w . n_E .$$

Areas of cross-sections flanges:

- flange in compression,

$$A_{ef.f.c} = b_{ef.c} . d_{tf} ; \qquad (2)$$

- flange in tension,

$$A_{ef.f.t} = b_{ef.t} . d_{bf} . \qquad (3)$$

Area of the webs:

$$A_{ef.w} = b_{w.tfd} . h_w . \qquad (4)$$

Transformed area:

$$A_{ef} = A_{ef.f.c} + A_{ef.f.t} + A_w . \qquad (5)$$

First moment of area of the section about the top face:

$$A_{1st} = A_{ef.f.t} . (h - d_{bt} /2)+ A_{ef.w} . (h_w/2+ +d_{tf})+ A_{ef.f.c} . d_{tf} /2. \qquad (6)$$

Neutral axis (*NA*) depth from the top face:

$$y_t = A_{1st} /A_{ef} . \qquad (7)$$

Second moment of area of the web about the *NA:*

$$I_{ef.w} = \frac{b_{w.tfd} \times h_w^3}{12} + A_{ef.w} \times (y_t - (d_{tf} + \frac{h_w}{2}))^2 \qquad (8)$$

Second moment of area of the top flange about the *NA:*

$$I_{ef.tf} = \frac{b_{ef.c} \cdot \delta_{tf}^3}{12} + A_{ef.f.c} \cdot (y_t - \frac{\delta_{tf}}{2})^2 . \qquad (9)$$

Second moment of area of the bottom flange about the *NA:*

$$I_{ef.bf} = \frac{b_{ef.t} \cdot \delta_{bf}^3}{12} + A_{ef.f.t} \cdot (h - y_t - \frac{\delta_{bf}}{2})^2 \qquad (10)$$

Instantaneous second moment of area the transformed section:

$$I_{ef} = I_{ef.w} + I_{ef.tf} + I_{ef.bf}. \qquad (11)$$

Stress in the flanges due to bending:

- bending stress (compression) in the top flange

$$\sigma_{f.c.max.d} = \frac{M_d}{I_{ef}} \cdot (y_t - \frac{\delta_{tf}}{2}) ; \qquad (12)$$

- bending stress (tension) in the bottom flange

$$\sigma_{f.t.max.d} = \frac{M_d}{I_{ef}} \cdot (h - y_t - \frac{\delta_{bf}}{2}) ; \qquad (13)$$

- bending stress check in the web

$$\sigma_{w.c.d} = \frac{M_d}{I_{ef}} \cdot y_1 \cdot \eta_E , \qquad (14)$$

were $y_1 = max\{(y_t - \delta_{tf}) ; (h - \delta_{bf} - y_t)\}$ – maximum distance from the *NA* to the extreme fibre.

Shear stress at the NA position:

$$\tau_{v.d} = \frac{V_d \cdot S_{tf.NA}}{I_{ef} \cdot b_{w.tfd}} \cdot \eta_E , \qquad (15)$$

were $S_{tf.NA}$ – first moment of area of the section above the NA:

$$S_{tf.NA} = b_{ef.c} \cdot \delta_{tf} \cdot (y_t - \frac{\delta_{tf}}{2}) + b_{w.tfd} \cdot \frac{(y_t - \delta_{tf})^2}{2} . \qquad (16)$$

Shear stress of the glued joint between the web and the flanges:

- first moment of area of top flange above the NA:

$$S_{tf} = b_{ef.c} \cdot \delta_{tf} \cdot (y_t - \frac{\delta_{tf}}{2}) ; \qquad (17)$$

- first moment of area of the bottom flange about NA:

$$S_{bf} = b_{ef.t} \cdot \delta_{bf} \cdot (h - y_t - \frac{\delta_{bf}}{2}) ; \qquad (18)$$

Maximum value of first moment of area about NA,

$S_f = max \{S_{tf} ; S_{bf} \};$

Mean shear stress in the flange across the glue line:

$$\tau_{mean.d} = \frac{V_d \cdot S_f}{I_{ef} \cdot b_{w.tfd}} \cdot \eta_E . \qquad (19)$$

Design example of composite skin panel. Top skin of panel is plywood with thickness $\delta_f = 9$ *mm*, bottom skin of panel is plywood with thickness $\delta_{bf} = 6$ *m*. Strength class of plywood *F20/10 E40/20* with the faces aligned parallel to the direction of span [9]. The timber used for the web is class *C22* [9]. Panel is glued between the flanges and the web. Spanning between two supports *L=4,5 m* apart. Nominal wide of panel – *1,5 m*. Construction sizes for panel 448x149 cm. The structure functions in service class 2 conditions.

The design of panel complies with the rules in [8] at the ULS and [7] (loads).

Dead loads to the panel **-** weight of materials.

Table 1. – Dead load to the 1 m² panel.

Materials	Characteristic load, q^0, $\kappa H/m^2$	Exploitation load, q^e, $\kappa H/m^2$	Safety factor, γ_M	Design load, q, $\kappa H/m^2$
1	2	3	4	5
1. Profile Steel Roofing Sheets	0,15	0,15	1,3	0,195
2. Plywood skin	0,105	0,105	1,1	0,115
3. Timber web	0,078	0,078	1,1	0,086
4. Mineral wool in panel	0,060	0,060	1,2	0,072
5. PE steam insulation	0,005	0,005	1,1	0,006
Total dead load for panel– q_p	0,398	0,398		0,474

Variable loads to the panel – snow load may be set by National Norm [7]

Design snow load and exploitation snow load:

$S_m = \gamma_{fm} S_0 C = 1,04 \cdot 1450 \cdot 1 = 1508 Pa = 1,508\ kN/m^2;$

$S_e = \gamma_{fe} S_0 C = 0,49 \cdot 1450 \cdot 1 = 710,5 Pa = 0,71\ kN/m^2,$

were $S_0 = 1450\ Pa$ – characteristic snow load for Poltava ([7], *Annex A*).

Other values in these formulas calculate according ([7].

Total loads to the panel

Table 2. – Total loads to the 1 m² panel.

Type of load	Characteristic load, $q^0 + S_0$, kN/m^2	Exploitation load, $q^e + S_e$, kN/m^2	Design load, $q_p + S_m$, kN/m^2
1	2	3	4
1. Dead load to the panel	0,398	0,398	0,474
2. Variable loads to the panel (snow)	1,450	0,710	1,508
Total loads for panel– q_{sum}	1,848	1,108	1,982

Loads for 1 meter of panel span

Total design loads for 1 meter of panel span (used table 2):

$q_l = q_{sum} \cdot B,$

were $B=1,5\ m$ – wide of panel.

Exploitation load:

$$q^e_l = 1,108 \cdot 1,5 = 1,662\ kN/m.$$

Design load:

$$q_l = 1,982 \cdot 1,5 = 2,973\ kN/m.$$

Actions. Design bending moment and design shear force:

$M_d = q_l \,.\, l^2/8 = 2,973.4,42^2/8 = 7,26\ kN\cdot m = 726\ kN\cdot cm;$

$V_d = 0,5\ q_l \,.\, l = 0,5.2,973.4,42 = 6,57\ kN,$

were $l = 4,48–0,06 = 4,42\ м$ – design span calculated with support effect.

Modification factor for permanent duration action ([8], *eq .2.6*):

$k_{\mathrm{mod.perm}} = (k_{\mathrm{mod.perm1}} \cdot k_{\mathrm{mod.perm2}})^{0,5} = (0,6 \cdot 0,6)^{0,5} = 0,6$,

were $k_{\mathrm{mod.perm1}} = 0,6$ for solid timber (webs) and *service class 2* ([8], *Table 3.1*);

$k_{\mathrm{mod.perm2}} = 0,6$ for plywood (flanges) and *service class 2* ([8], *Table 3.1*).

Factor for medium-duration action ([8], *eq. 2.6*):

$k_{\mathrm{mod.medium}} = (k_{\mathrm{mod.medium1}}\ k_{\mathrm{mod.medium2}})^{0,5} = (0,8 \cdot 0,8)^{0,5} = 0,8,$

were $k_{\mathrm{mod.medium1}} = 0,8$ for solid timber (webs) and *service class 2* ([8], *Table 3.1*);

$k_{\mathrm{mod.medium2}} = 0,8$ for plywood (flanges) and *service class 2* ([8], *Table 3.1*).

Load sharing factor, $k_{sys} = 1,0$ ([8], 6.6) ($k_{sys} = 1,1$ can be used if required).

Depth factor for solid timber (webs) – take as $k_h = 1$, as the depth is greater than *150 mm* ([8], *eq. (3.1)*).

Geometric properties.

Structural dimensions of the panel are shown in the Fig. 2. Clear distance between webs, $b_f = 430\ mm = 43\ cm$. Top flange thickness, $d_f = 9\ mm = 0,9\ cm$. Bottom flange thickness, $d_{bf} = 6\ mm = 0,6\ cm$. Dimensions of web, $b_w \times h_w = 45x195\ mm$ (*50x200 mm* dimensions of sawn timber before planning according).

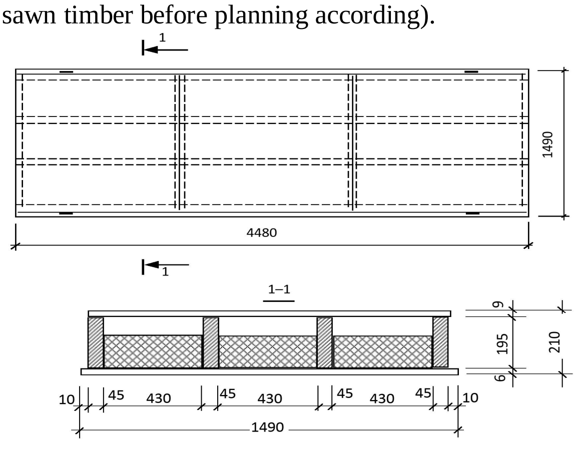

Figure 2 – Dimensions of panel.

Depth parameters of cross-section:

$$h = h_w + \delta_{bf} + \delta_{tf} = 19,5 + 0,6 + 0,9 = 21\ cm,$$

The effective flange width concept applies to flanges in compression and in tension, and unless a more detailed calculation is carried out, in accordance with the requirements of [8], 9.1.2(3). The effective flange width, b_{ef}, as shown in Figure 2, for internal I-shaped sections, will be as follows:

$b_{ef.c} = b_{c.ef} + b_w$ and $b_{ef.t} = b_{t.ef} + b_w$ ([8], *eq (9.12)*).

Effective flange width of an I-beam section of the panel ([8], *9.1.2*):

- in compression ([8], *Table 9.1*),

$b1_{c.ef} \le min\{\ 0,1 \cdot L\ ;\ 20 \cdot d_{tf}\ \} \le 180\ mm = 18\ cm;$

- in tension ([8], *Table 9.1*),

$b1_{t.ef} \le 0,1 \cdot L \le 450\ mm = 45\ cm$ (only shear lag).

The values $b1_{c.ef}$ and $b1_{t.ef}$ should not be greater than value $b_f = 430\ mm$.

So, design sizes of the flanges will be:

- in compression, $b_{c.ef} = b1_{c.ef} = 18\ cm;$
- in tension, $b_{t.ef} = b_f = 43\ cm.$

Effective flange width:

- in compression, $b_{ef.c} = b_{c.ef} + b_w = 18+4,5=22,5\ cm;$
- in tension, $b_{ef.t} = b_{t.ef} + b_w = 43+4,5=47,5\ cm.$

Instantaneous – transformed section properties. As the panel have a cross-section of two different materials it is convenient to go transformed section or fictitious section.

Coefficient for transformed section, as ratio of modulus according (1):

$$n_E = E_{0.mean} / E_{plw.0.mean} = 10 / 4 = 2,5\ ,$$

were $E_{0.mean} = 10\ kN/mm^2$ – mean modulus of elasticity parallel for solid timber, strength classes *C22* ([9], *Table Б.1*);

$E_{plw.0.mean} = 4\ kN/mm^2$– modulus of elasticity parallel for plywood, strength classes *F20/10 A40/20* (([9], *Table Б.5*).

Transformed web thickness (into plywood):

$$b_{w.tfd} = b_w \,.\, n_E = 4,5.2,5 = 11,25\ cm.$$

Areas of cross-sections flanges (2,3):

$$A_{ef.f.c} = 20,25\ cm^2,\ A_{ef.f.t} = 28,5\ cm^2.$$

Area of the webs (4):

$$A_{ef.w} = 219,38\ cm^2.$$

Transformed area (5):

$$A_{ef} = 268,13\ cm^2.$$

First moment of area of the section about the top face (6):

$$A_{1st} = 2935,41\ cm^3.$$

Neutral axis (*NA*) depth from the top face (7):

$$y_t = 10,9\ cm.$$

Second moment of area of the web about the *NA* (8):

$$I_{ef.w} = 6971\ cm^4.$$

Second moment of area of the top flange about the *NA* (9)*:*

$$I_{ef.tf} = 2333 cm^4.$$

Second moment of area of the bottom flange about the *NA* (10)*:*

$$I_{ef.bf} = 2758\ cm^4.$$

Instantaneous second moment of area the transformed section (11):

$$I_{ef} = 11962\ cm^4.$$

Bending stress check in the flanges and web (ULS).

Stress in the flanges due to bending:

- bending stress (compression) in the top flange (12):

$$\sigma_{f.c.max.d} = 0,637\ kN/cm^2 = 6,37\ N/mm^2.$$

- bending stress (tension) in the bottom flange (13):

$$\sigma_{f.t.max.d} = 0,592\ kN/cm^2 = 5,92\ N/mm^2$$

Strength of the top flange for plywood *F20/10 E40/20* and $f_{plw.c.k} = 15\ N/mm^2$ ([9], *Table Б.5*):

$f_{plw.c.d} = f_{plw.c.k} \cdot k_{\mathrm{mod.medium}} \cdot k_{sys} = 15 \cdot 0,8 \cdot 1 = 12\ N/mm^2$.

Strength of the bottom flange for plywood *F20/10 E40/20* and $f_{plw.t.k} = 9\ N/mm^2$ (([9], *Table Б.5*):

$f_{plw.t.d} = f_{plw.t.k} \cdot k_{\text{mod.medium}} \cdot k_{sys} = 9 \cdot 0,8 \cdot 1 = 7,2\ N/mm^2$.

Strength is satisfactory:

$$\sigma_{f.c.max.d} < f_{plw.c.d}\ ,\ \sigma_{f.t.max.d} < f_{plw.t.d}.$$

Bending stress check in the web:

· maximum distance from the *NA* to the extreme fibre,

$$y_1 = max\{(y_t - \delta_{tf}); (h - \delta_{bf} - y_t)\};$$
$$y_1 = max\{(10,9 - 0,9);(21 - 0,6 - 10,9\} = 10\ cm;$$

· bending stress in the web (14),

$$\sigma_{w.c.d} = 1,525\ kN/cm^2 = 15,25\ N/mm^2.$$

Bending strength of the web for solid timber *C22* and $f_{m.0.k} = 22\ N/mm^2$ (([9], *Table Б.1*):

$f_{m.0.d} = f_{c.0.k} \cdot k_{\text{mod.medium}} \cdot k_{sys} \cdot k_h = 22 \cdot 0,8 \cdot 1 \cdot 1 = 17,6\ N/mm^2$.

Strength is satisfactory:

$$\sigma_{w.c.d} < f_{m.0.d}\ .$$

Shear stress of the web (ULS). First moment of area of the section above the NA (16), $S_{tf.NA} = 780,48\ cm^4$.

Shear stress at the NA position (15):

$\tau_{v.d} = 0,095\ kN/cm^2 = 0,95\ N/mm^2$

Shear strength of the web material (solid timber *C22)* with $f_{V.k} = 2\ N/mm^2$ ([9], *Table Б.1*):

$f_{V.d} = f_{V.k} \cdot k_{\text{mod.medium}} \cdot k_{sys} = 2 \cdot 0,8 \cdot 1 =$
$= 1,6\ N/mm^2$.

Design shear strength is greater than the shear stress:

$$\tau_{v.d} < f_{V.d}.$$

Shear stress of the glued joint between the web and the flanges (ULS).

First moment of area of top flange above the NA (17):

$$S_{tf} = 212,58\ cm^4.$$

First moment of area of the bottom flange about NA (18):

$$S_{bf} = 277,94\ cm^4.$$

Maximum value of first moment of area about NA:

$$S_f = max\ \{S_{tf}; S_{bf}\} = max\{212,58; 277,94\} = 277,94\ cm.$$

Mean shear stress in the flange across the glue line (19):

$$\tau_{v.d} = 0,034\ kN/cm^2 = 0,34\ N/mm^2$$

Rolling shear strength of the flange material (plywood *F20/10 E40/20*) with $f_{plw.V.k} = 3,5\ N/mm^2$ (([9], *Table Б.5*):

$f_{plw.V.d} = f_{plw.V.k} \cdot k_{\text{mod.medium}} \cdot k_{sys} = 3,5 \cdot 0,8 \cdot 1 = 2,8\ N/mm^2$.

Rolling shear strength is determined according to ([8], *9.1.2(6)*:

· if $b_w \leq 8 \cdot \delta_{bf}$, then $f_{plw.V.d}$;

· if $b_w > 8 \cdot \delta_{bf}$, then $f_{plw.V.d} \cdot (8 \cdot \delta_{bf} / b_w)$.

Design shear strength is greater than the shear stress:

$$\tau_{v.d} < f_{plw.V.d}$$

5. Conclusion. For design cross-section of composite panel, it can apply a simple method of fictitious cross-section. The presented design methodology can be used to design composite (timber and plywood) skin panel in accordance with the requirements EN 1995-1-1 [8] and ДБН В.1.2.-2:2006 [9],

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