Primljen / Received: 31.8.2015.
Ispravljen / Corrected: 24.6.2016.
Prihvaćen / Accepted: 30.7.2016.
Dostupno online / Available online: 10.8.2016.

Analysis of timber-concrete composite girders

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Professional paper

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Analysis of timber-concrete composite girders

Timber-concrete composite girders are often used in the renovation of high-rise structures, and they can also be used in bridges. These systems are relatively complex to analyse as two materials presenting different stiffness and rheological properties participate in ensuring an appropriate bearing capacity. While the analysis of instantaneous deformations is clearly defined in Eurocode 5, the analysis of long-term deformations, which are often relevant, is not clearly defined. The objective of this paper is to present the analysis of a timber-concrete composite girder, taking into account instantaneous and long-term deformations in a relatively simple way, suitable for practical engineers.

Ključne riječi:
composite action, timber-concrete composite girder, long-term deformations

Stručni rad

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Proračun spregnutih nosača drvo - beton

Spregnuti nosači drvo-beton često se koriste kod rekonstrukcija objekata visokogradnje, a mogući su i kod mostova. Proračun takvih sustava je relativno kompleksan jer u nosivosti sudjeluju dva materijala različite krutosti i reoloških svojstava. Dok je proračun trenutačnih deformacija jasno definiran Eurokodom 5, proračun dugotrajnih deformacija, koje su često mjerodavne, nije jasno određen. U radu je prikazan proračun spregnutog nosača drvo-beton uzimajući u obzir trenutačne i dugotrajne deformacije na relativno jednostavan i inženjerima u praksi prikladan način. Osim proračuna prema graničnim stanjima dan je i proračun nosivosti spojnih sredstava.

Ključne riječi:
sprezanje, spregnuti nosač drvo-beton, dugotrajne deformacije

Fachbericht

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Berechnung von Holz-Beton-Verbundträgern

Holz-Beton-Verbundträger werden oft bei der Erneuerung von Hochbaukonstruktionen angewandt. Ebenso ist ihre Anwendung bei Brücken möglich. Die Berechnung solcher Systeme ist relativ komplex, da zwei Materiale verschiedener Steifigkeiten und rheologischer Eigenschaften dem Widerstand beitragen. Obwohl Eurocode 5 die Berechnung von Kurzzeitdeformationen klar definiert, ist die Ermittlung von oftmals maßgebenden Langzeitdeformationen nicht deutlich dargestellt. Das Ziel dieser Arbeit ist, die Berechnung von Holz-Beton-Verbundträgern unter Berücksichtigung von Kurz- und Langzeitdeformationen auf einfache Weise und für Ingenieure in der Praxis verständlich darzustellen.

Ključne riječi:
Verbund, Holz-Beton-Verbundträger, Langzeitdeformationen

1. Introduction

The principle of joining together different materials is based on the idea that a material that is highly resistant to tensile stress (e.g. steel, timber, etc.) should be placed in the tensile zone of cross-section, while a material that is highly resistant to compressive stress (concrete being the most frequent one) should be placed in the compressive zone of cross-section. The effective cross-section has a high load bearing capacity and stiffness, and a joint effect of two materials connected in this way is greater than the sum of their individual effects. Lightweight concrete i.e. concrete made with low-weight aggregate (rendered light by addition of expanded polystyrene granules) can also be used in floor structures in order to increase thermal and insulating properties of such structures. Lightweight-aggregate concretes are also considered appropriate because their elastic modulus is similar to that of timber. When using lightweight-aggregate concrete, a special attention must be paid to the selection of connectors (continuous composite actions are more favourable as in case of discrete action the connector can fail at the contact between the connector and concrete much before the relevant edge stress is achieved) [1]. Poorer mechanical properties of such concretes also play a significant role in this respect [1].
The thickness of slabs used does not usually exceed 8 cm, especially in case of traditional concrete, so as to avoid significance increase in self-weight of the structure as a whole. The analysis of timber-concrete composite girders is defined in Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings [2] and in Eurocode 5: Design of timber structures – Part 2: Bridges – National annex [3]. Here it should clearly be emphasized that the analysis made in the mentioned standards is actually the analysis relating only to short-term effects toward the final ultimate state. The issue of calculating long-term deformation of such composite girders is considered in paper [4]. The current situation in this area regarding such influences is presented in this paper through review of relevant literature. The aim of this paper is to present analysis of a timber-concrete composite girder taking at that into account long-term deformations, all this in a relatively simple way that can be useful to practical engineers. Symbols/designations used in the paper do not necessarily correspond to those given in Eurocode 5 (e.g. elastic modulus of concrete is designated as E_c rather than as E_1 as given in Eurocode 5). The authors consider that the designations used in the paper are more favourable for clarity reasons. Eurocode 5 provisions are given for a general composite girder and, as a timber-concrete composite girder is considered in this paper, the mentioned designations are used.

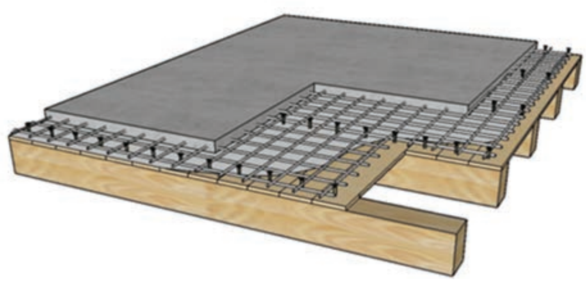


Figure 1. Schematic presentation of the system

2. Analysis of concrete slab and timber beam composite structure with flexible connectors

These systems are dimensioned using the γ-procedure defined in Eurocode 5 [2]. Figure 2 shows a concrete slab and timber beam composite structure with flexible connectors, where s is the distance between connectors. Figure 3 shows the cross section and the corresponding composite-girder geometrical properties that will be explained further on in the paper.

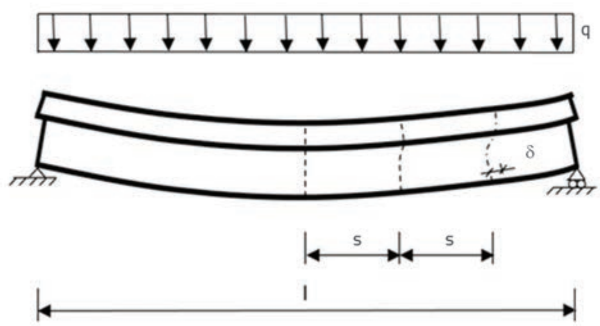


Figure 2. Concrete slab and timber beam composite structure with flexible connectors

2.1. Analysis of short term effects

2.1.1. Verifications for ultimate limit state:

Effective bending stiffness of shear-flexible composite beam is defined as follows:

$$(EI)_{eff} = E_c I_c + E_t I_t + \gamma_c E_c A_c a_c^2 + \gamma_t E_t A_t a_t^2 \quad (1)$$

where:
E_c - secant modulus of elasticity for concrete (E_{cm})
E_t - modulus of elasticity for timber
I_c - moment of inertia for concrete part of cross-section
I_t - moment of inertia for timber part of cross-section
A_c - area of concrete part of cross-section
A_t - area of timber part of cross-section
γ_c - composite action coefficient for concrete
γ_t - composite action coefficient for timber
a_c - eccentricity of centre of the concrete part of cross-section
a_t - eccentricity of centre of the timber part of cross-section

Eccentricity of centre of the timber (a_t) and concrete (a_c) parts of cross-section:

$$a_t = \frac{\gamma_c E_c A_c (h_c + h_t)}{2\sum_{i=1}^{2} \gamma_i E_i A_i} = \frac{\gamma_c E_c A_c (h_c + h_t)}{2(\gamma_c E_c A_c + E_t A_t)} \quad (2)$$

$$a_c = \frac{h_c + h_t}{2} - a_t \quad (3)$$

where:

h_c - height of concrete part of cross section
h_t - height of timber part of cross section

The sliding coefficient for shear-flexible composite T-beam with neutral axis in timber web is determined as follows:

$\gamma_t = 1$

$$\gamma_c = \frac{1}{1 + \frac{\pi^2 E_c A_c s}{K L^2}} \tag{4}$$

where:
s - distance between connectors
K - sliding modulus
L - span.

Longitudinal normal stress in centres of individual parts of the shear-flexible composite T-beam:

$$\sigma_i = \gamma_i \frac{M E_i a_i}{(EI)_{eff}} \tag{5}$$

$$\sigma_{m,i} = \frac{M E_i}{(EI)_{eff}} \left(\frac{h_i}{2} \right) \tag{6}$$

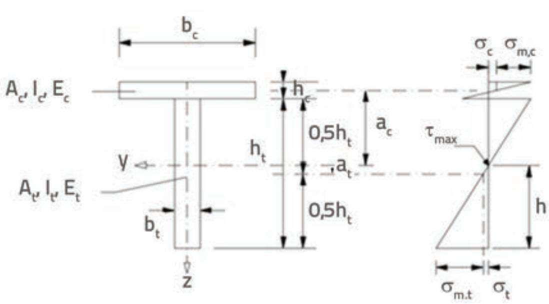


Figure 3. View of cross-section and longitudinal normal stresses in centres of individual parts of shear-flexible composite T-beam, [1]

Moduli of elasticity are:

$$E_c = E_{cm} \tag{7}$$

$$E_t = E_{0,mean} \tag{8}$$

where:
E_{cm} - mean modulus of elasticity in bending, for concrete
$E_{0,mean}$ - mean modulus of elasticity for timber in the direction of fibres

Stresses in the shear-flexible T-section (Figure 4) are due to joint action of a pair of longitudinal forces in centres of individual parts of cross-section (resulting from sliding) and bending (M_{Ed})

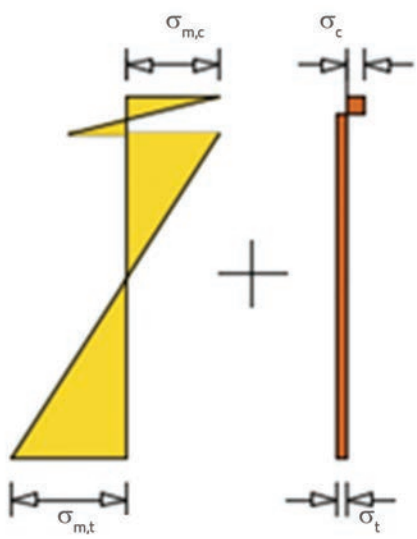


Figure 4 Stress in shear-flexible T-section

Expressions for stress are:

$$\sigma_t = \gamma_t \frac{M_d E_{cm} a_c}{(EI)_{eff}} \tag{9}$$

$$\sigma_c = \gamma_c \frac{M_d E_{cm} a_c}{(EI)_{eff}} \tag{10}$$

$$\sigma_{m,t} = \frac{M_d E_{0,mean}}{(EI)_{eff}} \left(\frac{h_t}{2} \right) \tag{11}$$

$$\sigma_{m,c} = \frac{M_d E_{cm}}{(EI)_{eff}} \left(\frac{h_c}{2} \right) \tag{12}$$

where:
σ_t - stress caused by compressive force in timber part of cross section
σ_c - stress caused by compressive force in concrete part of cross sectio
$\sigma_{m,t}$ - bending stress in timber part of cross-section
$\sigma_{m,c}$ - bending stress in concrete part of cross-section
M_d - design bending moment
$(EI)_{eff}$ - effective bending stiffness.

Assumptions for the analysis:
- connectors are positioned at the design spacing of $s_i = s$, along the length of the element L
- liding modulus, K_i (N/mm) is experimentally defined from shear test diagrams or push-out tests [7], and its values are:

$$K_i = K_{ser} \quad \text{for SLS} \tag{13}$$

$$K_i = K_u = 2/3 K_{ser} \quad \text{for ULS} \tag{14}$$

where:
K_{ser} - initial (useful) sliding modulus
K_u - effective sliding modulus.

If the sliding modulus can not be determined experimentally, as shown in papers [1, 7], then expressions presented in Eurocode 5 [2] can be used, as has been done in Section 3.
In addition, considering the level of shear and yield of connectors, tension can occur in the bottom part of concrete cross-section, i.e. the neutral axis of cross-section can be in the flange of cross-section. This case is not favourable and so attempts are made to prevent this situation by providing a sufficient number of connectors, i.e. to try to reach the situation in which the concrete cross-section is fully in compression. This shows that the limit stress in the bottom part of concrete cross-section must be lower compared to the compressive strength of concrete (as presented in expressions 15 and 16). However, if tension occurs, then the stress must be lower than the tensile strength of concrete f_{ctm}. Stress values at the top, $\sigma_{c,g}$ and bottom edges of concrete slab of shear-flexible composite T-section, $\sigma_{c,d}$ must comply with the following equations:

$$\sigma_{c,g} = \sigma_{m,c} + \sigma_c \leq f_{c,d} \tag{15}$$

$$\sigma_{c,d} = \sigma_{m,c} - \sigma_c \leq f_{c,d} \tag{16}$$

where:
σ_c - stress caused by compressive force in concrete part of cross section
$\sigma_{m,c}$ - bending stress in concrete part of cross-section
$f_{c,d}$ - design compressive strength of concrete.

Bearing capacity verification for timber beam of shear-flexible composite cross-section:

$$\frac{\sigma_{t,d}}{f_{t,o,d}} + \frac{\sigma_{m,d}}{f_{m,d}} \leq 1 \tag{17}$$

where:
$\sigma_{t,d}$ - design tensile stress
$\sigma_{m,d}$ - design bending stress
$f_{t,o,d}$ - design tensile strength parallel to fibres
$f_{m,d}$ - design bending strength.

Verification of shear bearing capacity of the timber part of cross section – fully assumes maximum transverse force, V_d:

$$\tau = \frac{V_d}{(b_{eff} \cdot h_t)} \leq f_{v,d} \tag{18}$$

$$b_{eff} = k_{cr} \cdot b = 0{,}67 \cdot b$$

where:
V_d - design transverse force
b_{eff} - design width of timber element
k_{cr} - cracking factor for shear resistance
b - width of timber cross-section
τ - shear stress
$f_{v,d}$ - design shear strength.

If necessary, the bearing capacity proof (17) can be written at the level of forces as follows:

- Design longitudinal force in timber element:
$$N_{Ed,t} = \sigma_t \cdot A_t \tag{19}$$

- Design resistance to longitudinal force:
$$N_{Rd,t} = f_{t,o,d} \cdot A_t \tag{20}$$

- Design bending moment in timber element:
$$M_{Ed,t} = \sigma_{m,t} \cdot W_t \tag{21}$$

where W_t is the resistance moment for timber cross-section.

- The bearing capacity proof is written as follows:
$$M_{Rd,t} = f_{m,d} \cdot W_t \tag{22}$$

Proof load as follows:

$$\frac{N_{Ed,t}}{N_{Rd,t}} + \frac{M_{Ed,t}}{M_{Rd,t}} \leq 1 \tag{23}$$

2.1.2. Verifications for serviceability limit state: Verification of momentary deflections

The following K_i value has been adopted:

$$K_i = K_{ser}$$

Sliding coefficient:

$$\gamma_c = \frac{1}{1 + \dfrac{\pi^2 E_c A_c s_c}{K_{ser} L^2}} \tag{24}$$

Eccentricities of centres of the timber and concrete parts of cross-section:

$$a_t = \frac{\gamma_c E_c A_c (h_c + h_t)}{2(\gamma_c E_c A_c + E_t A_t)} \tag{25}$$

$$a_c = \frac{h_c + h_t}{2} - a_t \tag{26}$$

Effective flexural stiffness of shear-flexible composite beam:

$$(EI)_{eff} = E_c I_c + E_t I_t + \gamma_c E_c A_c a_c^2 + \gamma_t E_t A_t a_t^2 \tag{27}$$

Total deflection due to permanent load:

$$M_g = \frac{gL^2}{8} \tag{28}$$

$$u_{inst}^{Gk,j}=\frac{5}{48}\cdot\frac{M_g\cdot L^2}{(EI)_{eff}} \quad (29)$$

Total deflection due to variable load:

$$M_q=\frac{qL^2}{8} \quad (30)$$

$$u_{inst}^{Qk,j}=\frac{5}{48}\cdot\frac{M_q\cdot L^2}{(EI)_{eff}} \quad (31)$$

The following conditions must be met for momentary deflection:

$$u_{inst}^{Gk,j}\leq\frac{L}{300} \quad (32)$$

$$u_{inst}^{Qk,j}\leq\frac{L}{300} \quad (33)$$

The following condition must be met for total deflection:

$$u_{inst}^{Gk,j}+u_{inst}^{Qk,j}\leq\frac{L}{200} \quad (34)$$

2.2. Analysis of long-term effects

According to [4], the analysis of such systems with regard to long-term load is much more challenging and complex as mechanical changes in timber, concrete, and steel have to be taken into account due to changes in moisture, temperature, and load over time. Therefore, the following is taken into account during analysis of long-term effects: creep of concrete, creep of timber element, and connector slip (displacement). The analysis presented in this paper is based on paper [6].

2.2.1. Verifications for ultimate limit state:

Total deformation $\varepsilon_{c\sigma}(t,t_0)$ is generally defined as follows:

$$\varepsilon_{c\sigma}(t,t_0)=\frac{\sigma_c(t_0)+\Delta\sigma_c(t,t_0)}{E_{cm,eff}}$$

The effective elastic modulus for the long-term load of concrete is defined as follows:

$$E_{cm,eff}=\frac{E_{cm}}{1+\varphi(t,t_0)} \quad (35)$$

where:

$\varphi(t,t_0)$ - coefficient of creep for concrete.

Final mean value of elastic modulus of concrete in the direction of fibres

$$E_{0.mean,fin}=\frac{E_{0,mean}}{(1+\psi\, k_{def})} \quad (36)$$

where:
Ψ - factor for combined value
k_{def} - deformation factor.

Sliding modulus:

$$K_{ser,fin}=\frac{K_{ser}}{(1+\psi\, k_{def})} \quad (37)$$

Flexural stiffness parameters over the life span of the structure:

- Sliding coefficient for concrete:

$$\gamma_c=\frac{1}{1+\dfrac{\pi^2E_{cm,eff}A_cs}{K_{ser,fin}L^2}} \quad (38)$$

- Eccentricity of centres of the timber (a_t) and concrete (a_c) parts of cross-section:

$$a_t=\frac{\gamma_cE_{cm,eff}A_ch}{\gamma_cE_{cm,eff}A_c+E_{0,mean,fin}A_t} \quad (39)$$

$$a_c=\frac{E_{0,mean,fin}A_th}{\gamma_cE_{cm,eff}A_c+E_{0,mean,fin}A_t} \quad (40)$$

- Effective flexural stiffness:

$$(EI)_{eff}=E_{cm,eff}I_c+E_{o,mean,fin}I_t+\gamma_cE_{cm,eff}A_ca_c^2+\gamma_tE_{o,mean,fin}A_ta_t^2 \quad (41)$$

- Quasi-constant combination:

$$q_{Sd,1}=(\gamma_Gg+\gamma_Q\psi_2q)e=(1,0\cdot g+1,0\cdot 0,3\ q)\cdot e\ [\text{kN/m'}] \quad (42)$$

- Design bending moment:

$$M_{d,q_{Sd,1}}=\frac{q_{Sd,1}L^2}{8} \quad (43)$$

- Design transverse force:

$$V_{d,q_{Sd,1}}=\frac{q_{Sd,1}L}{2} \quad (44)$$

The following conditions must be met for the timber part of cross-section:

- Longitudinal stress in timber caused by longitudinal force, due to load combination $q_{Sd,1}$

$$\sigma_{t(q_{Sd,1})}=\frac{E_{0,mean,fin}a_tM_{d,q_{Sd,1}}}{(EI)_{eff}} \quad (45)$$

- Longitudinal stress in timber by bending moment due to load combination $q_{Sd,1}$

$$\sigma_{m,t_{q_{Sd,1}}}=\frac{1}{2}\cdot\frac{E_{0,mean,fin}h_tM_{d,q_{Sd,1}}}{(EI)_{eff}} \quad (46)$$

Verification of bearing capacity for timber beam having shear-flexible composite cross-section:

$$\frac{\sigma_{t(q_{Sd,1})}}{f_{t,o,d}}+\frac{\sigma_{m,t_{q_{Sd,1}}}}{f_{m,d}}\leq 1 \quad (47)$$

If necessary, proof of bearing capacity (47) can also be written at the level of forces as shown below:

- Design longitudinal force in timber element:
  $$N_{Ed,t} = \sigma_{t(qSd,1)} \cdot A_t \quad (48)$$
- Design resistance to longitudinal force:
  $$N_{Rd,t} = f_{c,o,d} \cdot A_t \quad (49)$$
- Design bending moment in timber element:
  $$M_{Ed,t} = \sigma_{m,t(qSd,1)} \cdot W_t \quad (50)$$
- Design resistance to bending moment:
  $$M_{Rd,t} = f_{m,d} \cdot W_t \quad (51)$$

Proof of bearing capacity is:

$$\frac{N_{Ed,t}}{N_{Rd,t}} + \frac{M_{Ed,t}}{M_{Rd,t}} \leq 1 \quad (52)$$

Shear stress in timber caused by load combination $q_{Sd,1}$

$$\tau_{(q_{Sd,1})} = \frac{V_{d(q_{Sd,1})}}{(b_{eff} \cdot h_t)} \leq f_{v,d} \quad (53)$$

Verification of pressure perpendicular to fibres on the support, for load combination q_{Sd1}

$$\sigma_{d,c,90(q_{Sd,1})} = \frac{V_{d(q_{Sd,1})}}{l_b b_t} \quad (54)$$

The following conditions must be met for the concrete part of cross-section:

- Longitudinal stress in concrete caused by longitudinal force, from load combination $q_{Sd,1}$
  $$\sigma_{c_{(q_{Sd,1})}} = \frac{\gamma_c E_{cm,eff} a_c M_{d,q_{Sd,1}}}{(EI)_{eff}} \quad (55)$$
- Longitudinal stress in concrete by bending moment, from load combination $q_{Sd,1}$
  $$\sigma_{m,c_{(q_{Sd,1})}} = \frac{1}{2} \cdot \frac{E_{cm,eff} h_c M_{d,q_{Sd,1}}}{(EI)_{eff}} \quad (56)$$
- Total stress at top edge of concrete:
  $$\sigma_{c,g} = -\sigma_{c_{(gSd,1)}} - \sigma_{m,c_{(qSd,1)}} < f_{cd} \quad (57)$$
- Total stress at bottom edge of concrete:
  $$\sigma_{t,d} = -\sigma_{c_{(gSd,1)}} + \sigma_{m,c_{(qSd,1)}} < f_{cd} \quad (58)$$

2.2.2. Verifications for serviceability limit state:

The effective elastic modulus for the long-term load of concrete is defined as follows:

$$E_{cm,eff} = \frac{E_{cm}}{1+\varphi(t,t_0)} \quad (59)$$

Final mean value of elastic modulus of concrete in the direction of fibres:

$$E_{0,mean,fin} = \frac{E_{0,mean}}{(1+k_{def})} \quad (60)$$

Sliding modulus:

$$K_{ser,fin} = \frac{K_{ser}}{(1+k_{def})} \quad (61)$$

Sliding coefficient:

$$\gamma_c = \frac{1}{1+\dfrac{\pi^2 E_{cm,eff} A_c s}{K_{ser,fin} L^2}} \quad (62)$$

Eccentricity of centres of the timber (a_t) and concrete (a_c) parts of cross-section:

$$a_t = \frac{\gamma_c E_{cm,eff} A_c (h_c + h_t)}{2(\gamma_c E_{cm,eff} A_c + E_{0,mean,fin} A_t} \quad (63)$$

$$a_c = \frac{h_c + h_t}{2} - a_t \quad (64)$$

Effective flexural stiffness of shear-flexible composite beam:

$$(EI)_{eff} = E_{cm,eff} I_c + E_{o,mean,fin} I_t + \gamma_c E_{cm,eff} A_c a_c^2 + \gamma_t E_{o,mean,fin} A_t a_t^2 \quad (65)$$

Final deflection due to permanent load:

$$M_g = \frac{gL^2}{8} \quad (66)$$

$$u_{fin}^{Gk,j} = \frac{5}{48} \cdot \frac{M_g \cdot L^2}{(EI)_{eff}} \quad (67)$$

Final deflection due to variable load:

$$M_q = \frac{qL^2}{8} \quad (68)$$

$$u_{fin}^{Qk,j} = \frac{5}{48} \cdot \frac{M_q \cdot L^2}{(EI)_{eff}} \quad (69)$$

The following conditions must be met for final deflection:

$$u_{fin}^{Gk,j} \leq \frac{L}{200} \quad (70)$$

$$u_{fin}^{Qk,j} \leq \frac{L}{200} \quad (71)$$

The following conditions must be met for final deflection due to permanent and variable load:

$$u_{fin}^{Gk,j} + u_{fin}^{Qk,j} \leq \frac{L}{200} \quad (72)$$

3. Example of analysis of composite girders

An example of composite construction involving a traditional timber floor and a concrete slab is presented in this section. Generally, there methods are used for the analysis of composite girders: γ method, fixed transverse force method, and elastoplastic method. The γ method is used in this section. In fact, this method is most often used in the analysis of timber-concrete composite systems. The method can be applied if the static system under study is a simply placed beam. In addition, the following assumptions must be met for the use of the method:

1. Timber element must have a solid cross-section
2. The spacing between connectors can either be constant or variable, depending on transverse force
3. The beam is made composite by shear-flexible connection
4. Bending moments generated by forces can be described as sinusoidal or parabolic functions

It should be noted that the results obtained by this method are satisfactory if both materials are situated in the linear-elastic area. A general deficiency of this method lies in the fact that it does not take into account ductility of the connection. Other than this method, the use can be made of the fixed shear force method. The method assumes an elastoplastic loading and load relaxation relationship and thus partly takes into account ductility of the connection. The assumption that all connecting devices yield simultaneously leads to an estimation error, and so this method can be considered relatively conservative. The third calculation method, i.e. the elastoplastic method, is considered to be suitable for the ultimate limit state analysis. The method assumes a perfectly stiff connection, i.e. a perfectly plastic strain to yield relationship. As most connecting devices do not provide a perfectly stiff connection, we obtain a bearing capacity that exceeds a realistic one when calculating structural behaviour under service-life load. The elastoplastic method is favourable for obtaining the final effective stiffness and the bearing capacity of the structure itself, but it overestimates the initial effective stiffness in the elastic area. Based on the above considerations, and as this method has been defined in Eurocode 5, the example will be presented using the γ method.

Usual beam dimensions and floor and soffit layers are given in the example (Figure 5). The condition after renovation (Figure 6) implies removal of wood debris and formwork with lime plaster so as to achieve properties compliant with present-day requirements. Dimension of elements in figures below have been taken from paper [5]. Both permanent and service loads are taken into account. It is assumed that the class of use (moisture) will be 1 and so the factor of change and factor of deformation were assumed to be k_{mod} = 0.9 and K_{def} = 0.6, respectively. The factor for quasi-permanent variable action was defined in accordance with category A: houses, residential buildings.

- class of concrete strength: C25/30 → E_c = 30500 N/mm² ($E_{cm} = E_c$)
- solid wood: class C24 →, $E_{0,mean}$ = 11000 N/mm² ($E_{0,mean} = E_t$), ρ_m = 420 kg/m³
- g (permanent load) = 3,05 kN/m²
- q (service load) = 2,0 kN/m²
- L (span) = 6 m
- e (distance between beams) =0,9 m
- s (distance between connections) = 120 mm

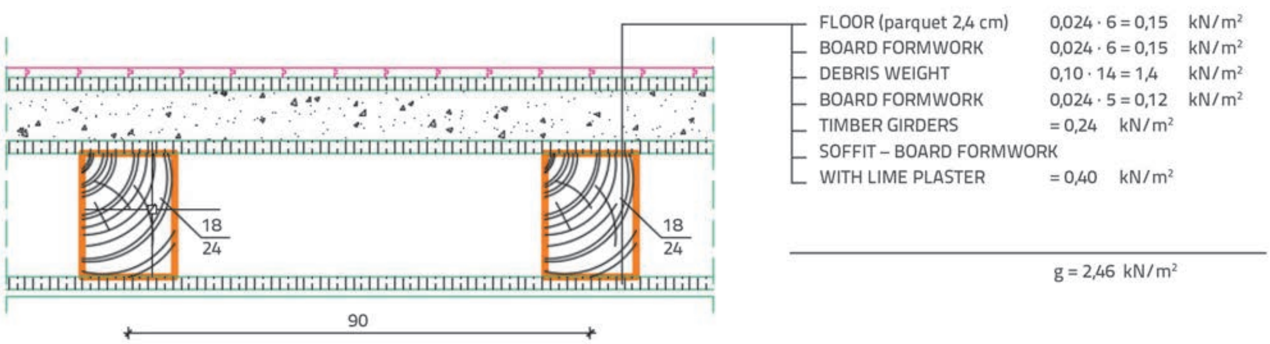
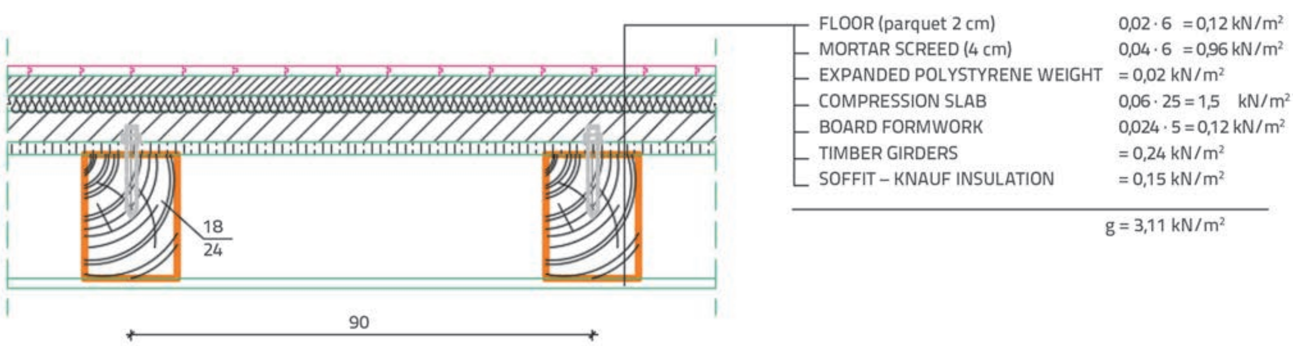


Figure 5. Load of existing floor with layers [5]



Slika 6. Load of composite floor with layers [5]

- d (bolt diameter) = 20 mm
- $\gamma_{M,c}$ (partial safety factor for concrete) = 1,5
- $\gamma_{M,t}$ (partial safety factor for timber) = 1,3.

3.1. Analysis of short-term effects

3.1.1. Verification for ultimate limit state

This evaluation was carried out as follows:

$$I_c = \frac{bh^3}{12} = \frac{90 \cdot 6^3}{12} = 1620\ cm^4 = 16.200.000\ mm^4$$

$$I_t = \frac{bh^3}{12} = \frac{18 \cdot 24^3}{12} = 20.736\ cm^4 = 207.360.000\ mm^4$$

$A_c = b \cdot h = 6 \cdot 90 = 540\ cm^3 = 54000\ mm^2$

$A_t = b \cdot h = 18 \cdot 24 = 432\ cm^3 = 43200\ mm^2$

$$K_{ser} = 2 \cdot \rho_m^{1,5} \frac{d}{23} = 2 \cdot 420^{1,5} \cdot \frac{20}{23} = 14969,5\ \frac{N}{mm}\ (GSU)$$

$$K_u = \frac{2}{3} \cdot K_{ser} = \frac{2}{3} \cdot 14969,5 = 9979,7\ \frac{N}{mm}\ (GSN)$$

Sliding coefficients:

$\gamma_t = 1$

$$\gamma_c = \frac{1}{1 + \frac{\pi^2 E_c A_c s}{KL^2}} = \frac{1}{1 + \frac{\pi^2 \cdot 30500 \cdot 54000 \cdot 120}{9979,7 \cdot 6000^2}} = 0,16$$

Eccentricity of centre of the timber (a_t) and concrete (a_c) arts of cross-section:

$$a_t = \frac{\gamma_c E_c A_c (h_c + h_t)}{2(\gamma_c E_c A_c + E_t A_t)} = \frac{0,16 \cdot 30500 \cdot 54000 \cdot (60 + 240)}{2(0,16 \cdot 30500 \cdot 54000 + 11000 \cdot 43200)} = 53,51\ mm = 5,351\ cm$$

$$a_c = \frac{h_c + h_t}{2} - a_t = \frac{60 + 240}{2} - 53,51 = 96,49\ mm = 9,649\ cm$$

Effective flexural stiffness of the shear-flexible composite beam:

$(EI)_{eff} = E_c I_c + E_t I_t + \gamma_c E_c A_c a_c^2 + \gamma_t E_t A_t a_t^2$
$= 30500 \cdot 16200000 + 11000 \cdot 207360000 + 0,16 \cdot 30500 \cdot 54000 \cdot 96,49^2 + 1 \cdot 11000 \cdot 43200 \cdot 53,51^2$
$= 6,5 \cdot 10^{12}$ Nmm²
$= 6,5 \cdot 10^{10}$ Ncm².

Design load:

$q_{Sd} = (\gamma_G g + \gamma_Q q) \cdot e = (1,35 \cdot 3,11 + 1,5 \cdot 2,0) \cdot 0,9 = 6,48\ kN/m'$

$$M_d = \frac{q_{Sd} L^2}{8} = \frac{6,48 \cdot 6^2}{8} = 29,16\ kNm = 29,16 \cdot 10^6\ Nmm$$

Longitudinal normal stress in centres of individual parts of a shear-flexible composite T beam:

$$\sigma_t = \gamma_t \frac{M_d E_{0,mean} a_t}{(EI)_{eff}} = 1 \cdot \frac{29,16 \cdot 10^6 \cdot 11000 \cdot 53,51}{6,5 \cdot 10^{12}} = 2,64\ N/mm^2$$

$$\sigma_c = \gamma_c \frac{M_d E_{cm} a_c}{(EI)_{eff}} = 0,16 \cdot \frac{29,16 \cdot 10^6 \cdot 30500 \cdot 96,49}{6,5 \cdot 10^{12}} = 2,11\ N/mm^2$$

$$\sigma_{m,t} = \frac{M_d E_{0,mean}}{(EI)_{eff}} \left(\frac{h_t}{2}\right) = \frac{29,16 \cdot 10^6 \cdot 11000}{6,5 \cdot 10^{12}} \cdot \left(\frac{240}{2}\right) = 5,92\ N/mm^2$$

$$\sigma_{m,c} = \frac{M_d E_{cm}}{(EI)_{eff}} \cdot \left(\frac{h_c}{2}\right) = \frac{29,16 \cdot 10^6 \cdot 30500}{6,5 \cdot 10^{12}} \cdot \left(\frac{60}{2}\right) = 4,10\ N/mm^2$$

The stress of the top, $\sigma_{c,g}$ and bottom, $\sigma_{c,d}$ edges of the concrete slab having a shear-flexible composite T section must comply with the following equations:

$f_{ck} = 25$ N/mm²

$$f_{cd} = \frac{f_{ck}}{\gamma_{M,c}} = \frac{25}{1,5} = 16,67\ N/mm^2$$

$\sigma_{c,g} = \sigma_{m,c} + \sigma_c \leq f_{cd}$ $\qquad$ $\sigma_{c,d} = \sigma_{m,c} - \sigma_c \leq f_{cd}$

$\sigma_{c,g} = 4,10 + 2,11 = 6,21\ N/mm^2 \leq 16,67\ N/mm^2$

$\sigma_{c,d} = 4,10 - 2,11 = 1,99\ N/mm^2 \leq 16,67\ N/mm^2$

Verification of bearing capacity for the timber beam having a shear-flexible composite section:

$f_{t,o,k} = 14\ N/mm^2$

$f_{m,k} = 24\ N/mm^2$

$$f_{t,o,d} = k_{mod} \cdot \frac{f_{t,o,k}}{\gamma_{M,t}} = 0,9 \cdot \frac{14}{1,3} = 9,69\ N/mm^2$$

$$f_{m,d} = k_{mod} \cdot \frac{f_{m,k}}{\gamma_{M,t}} = 0,9 \cdot \frac{24}{1,3} = 16,62\ N/mm^2$$

$$\frac{\sigma_{t,d}}{f_{t,o,d}} + \frac{\sigma_{m,t,d}}{f_{m,d}} \leq 1$$

$$\frac{2,64}{9,69} + \frac{5,92}{16,62} \leq 1$$

$0,63 \leq 1$

Verification of shear resistance of timber part of cross-section – fully assumes maximum transverse force, V_d:

$$V_d = \frac{q_{Sd} \cdot L}{2} = \frac{6,48 \cdot 6}{2} = 19,44\ kN = 19440\ N$$

$f_{v,k} = 2,5$ N/mm²

$$f_{v,d} = k_{mod} \cdot \frac{f_{v,k}}{\gamma_{M,t}} = 0,9 \cdot \frac{2,5}{1,3} = 1,73\ N/mm^2$$

$b_{eff} = k_{cr} \cdot b = 0,67 \cdot 180 = 120,6$ mm

$$\tau = \frac{V_d}{(b_{eff} \cdot h_t)} \le f_{v,d}$$

$$\tau = \frac{19440}{(120{,}6 \cdot 240)} \le f_{v,d}$$

τ = 0,67 N/mm^2 ≤ 1,73 N/mm^2

3.1.2. Verifications for serviceability limit state – xerification of momentary deflections

The following value is adopted for the sliding modulus K_i:

$K_i = K_{ser}$ =14969,5 *N/mm*

Sliding coefficients:

$\gamma_t = 1$

$$\gamma_c = \frac{1}{1+\frac{\pi^2 E_c A_c s}{KL^2}} = \frac{1}{1+\frac{\pi^2 \cdot 30500 \cdot 54000 \cdot 120}{14969{,}5 \cdot 6000^2}} = 0{,}22$$

Eccentricity of centres of the timber (a_t) and concrete (a_c) parts of cross-section:

$$a_t = \frac{\gamma_c E_c A_c (h_c + h_t)}{2(\gamma_c E_c A_c + E_t A_t)} = \frac{0{,}22 \cdot 30500 \cdot 54000 \cdot (60+240)}{2(0{,}22 \cdot 30500 \cdot 54000 + 11000 \cdot 43200)} = 64{,}89 \text{ mm} = 6{,}489 \text{ cm}$$

$$a_c = \frac{h_c + h_t}{2} - a_t = \frac{60+240}{2} - 64{,}89 = 85{,}11 \text{ mm} = 8{,}511 \text{ cm}$$

Effective flexural stiffness of shear-flexible composite beam:

$(EI)_{eff} = E_cI_c + E_tI_t + \gamma_c E_c A_c a_c^2 + \gamma_t E_t A_t a_t^2$
= 30500·16200000+11000·207360000+0,22·30500·54000·85,11^2+1·11000·43200·64,89^2
= 7,4 ·10^{12} Nmm^2
= 7,4 ·10^{10} Ncm^2

Total deflection due to permanent load:

g = 3,11 kN/m^2·0,9 m = 2,80 *kN/m'*

$$M_g = \frac{gL^2}{8} = \frac{2{,}80 \cdot 6^2}{8} = 12{,}6 \text{ kNm} = 12{,}6 \cdot 10^6 \text{ Nmm}$$

$$u_{inst}^{Gk,j} = \frac{5}{48} \cdot \frac{M_g \cdot L^2}{(EI)_{eff}} = \frac{5}{48} \cdot \frac{12{,}6 \cdot 10^6 \cdot 6000^2}{7{,}4 \cdot 10^{12}} = 6{,}39 \text{ mm}$$

Total deflection due to variable load:

q = 2,0 kN/m^2·0,9 m = 1,8 *kN/m'*

$$M_q = \frac{qL^2}{8} = \frac{1{,}8 \cdot 6^2}{8} = 8{,}1 kNm = 8{,}1 \cdot 10^6 \text{ Nmm}$$

$$u_{inst}^{Qk,j} = \frac{5}{48} \cdot \frac{M_q \cdot L^2}{(EI)_{eff}} = \frac{5}{48} \cdot \frac{8{,}1 \cdot 10^6 \cdot 6000^2}{7{,}4 \cdot 10^{12}} = 4{,}10 mm$$

The following conditions must be met for momentary deflection:

$$u_{inst}^{Gk,j} \le \frac{L}{300}$$

6,39 mm ≤ 20mm

$$u_{inst}^{Qk,j} \le \frac{L}{300}$$

4,10 mm ≤ 20mm.

The following condition must be met for total deflection:

$$u_{inst}^{Gk,j} + u_{inst}^{Qk,j} \le \frac{L}{200}$$

6,39 + 4,10 ≤ 30

10,49 mm ≤ 30mm

3.2. Analysis of long-term effects

3.2.1. Verification for ultimate limit state

Effective elastic modulus of concrete for long-term load:

$$E_{cm,eff} = \frac{E_{cm}}{1+\varphi(t,t_0)} = \frac{30500}{1+3{,}5} = 6777{,}78 \; N/mm^2$$

The following parameters were used in Figure 7 for calculation of creep coefficient: concrete class C25/30, N curve in diagram (for N class of cement), and start of system load after 10 days (t_0 = 10 days).

$\varphi(t,t_0)$ = 3,5

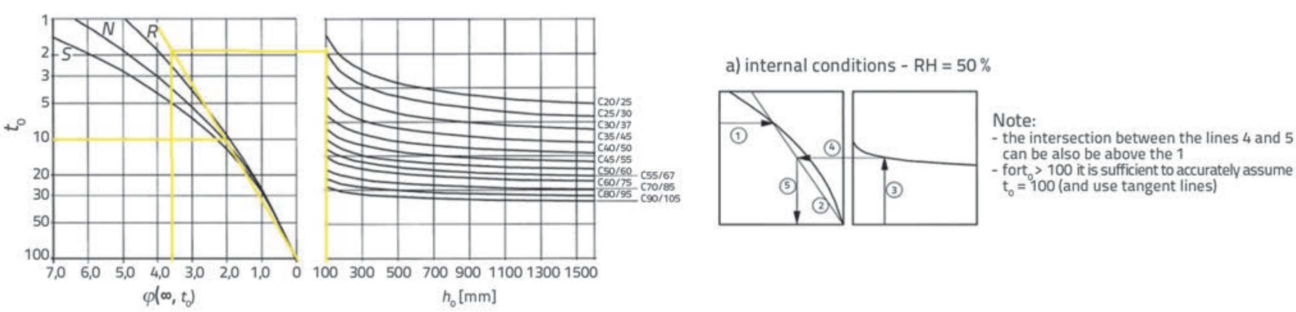


Figure 7. Determination of creep coefficient $\varphi(\infty,t_0)$ for concrete under normal ambient conditions

Final mean elastic modulus for timber:

$$E_{0,mean,fin}=\frac{E_{0,mean}}{(1+\psi\, k_{def})}=\frac{11000}{(1+0,3\cdot 0,6)}=9322,03\ N/mm^2$$

Sliding modulus:

$$K_{ser,fin}=\frac{K_u}{(1+\psi\, k_{def})}=\frac{9979,7}{(1+0,3\cdot 0,6)}=8457\ N/mm$$

Flexural stiffness parameters over service life of structure (KGS):
Sliding coefficient for concrete:

- Sliding coefficients for concrete

$$\gamma_c=\frac{1}{1+\frac{\pi^2 E_{cm,eff} A_c s}{K_{ser,fin} L^2}}=\frac{1}{1+\frac{\pi^2\cdot 6777,78\cdot 54000\cdot 120}{8457\cdot 6000^2}}=0,41$$

- Eccentricity of centres of the timber (a_c) and concrete (a_t) parts of cross-section:

$$a_c=\frac{E_{0,mean,fin}A_t h}{\gamma_c E_{cm,eff} A_c+E_{0,mean,fin} A_t}=\frac{9322,03\cdot 43200\cdot 174}{0,41\cdot 6777,78\cdot 54000+9322,03\cdot 43200}=126,76\ mm$$

$$a_t=\frac{\gamma_c E_{cm,eff} A_c h}{\gamma_c E_{cm,eff} A_c+E_{0,mean,fin} A_t}=\frac{0,41\cdot 6777,78\cdot 54000\cdot 174}{0,41\cdot 6777,78\cdot 54000+9322,03\cdot 43200}=47,24\ \cdot mm$$

$$h=\frac{240}{2}+\frac{60}{2}+24=174\ mm$$

- Effective flexural stiffness:

$$(EI)_{eff}=E_{cm,eff}I_c+E_{o,mean,fin}I_t+\gamma_c E_{cm,eff}A_c a_c^2+\gamma_t E_{o,mean,fin}A_t a_t^2$$
$=6777,78\cdot 16200000+9322,03\cdot 207360000+0,41\cdot 6777,78\cdot 54000\cdot 85,11^2+126,76\cdot 9322,03\cdot 43200\cdot 47,24^2$
$= 5,35\cdot 10^{12}$ Nmm²
$= 5,35\cdot 10^{10}$ Ncm²

- Quasi-permanent combination:

$$q_{Sd,1}=(\gamma_G g+\gamma_Q\psi_2 q)e=(1,0\cdot g+1,0\cdot 0,3\, q)\cdot e$$
$$=(1\cdot 3,11+1,0\cdot 0,3\cdot 0,2)\cdot 0,9=3,34\ [kN/m']$$

- Design bending moment:

$$M_{d,q_{Sd,1}}=\frac{q_{Sd,1}L^2}{8}=\frac{3,34\cdot 6^2}{8}=15,03\,kNm=15,03\cdot 10^6\ Nmm$$

- Design transverse force:

$$V_{d,q_{Sd,1}}=\frac{q_{Sd,1}L}{2}=\frac{3,34\cdot 6}{2}=10,02\ kN=10020\ N$$

The following requirements must be med for the timber part of cross-section:

- Longitudinal stress in timber caused by longitudinal force, from load combination $q_{Sd,1}$

$$\sigma_{t(q_{Sd,1})}=\frac{E_{0,mean,fin}a_t M_{d,q_{Sd,1}}}{(EI)_{eff}}=\frac{9322,03\cdot 47,24\cdot 15,03\cdot 10^6}{5,35\cdot 10^{12}}=1,24\ N/mm^2$$

- Longitudinal stress in timber due to bending moment, from load combination $q_{Sd,1}$

$$\sigma_{m,t_{q_{Sd,1}}}=\frac{1}{2}\cdot\frac{E_{0,mean,fin}h_t M_{d,q_{Sd,1}}}{(EI)_{eff}}=\frac{1}{2}\cdot\frac{9322,03\cdot 240\cdot 15,03\cdot 10^6}{5,35\cdot 10^{12}}=3,14\ N/mm^2$$

- Bearing capacity verification for timber beam of shear-flexible composite cross-section:

$$\frac{\sigma_{t(q_{Sd,1})}}{f_{t,o,d}}+\frac{\sigma_{m,t_{q_{Sd,1}}}}{f_{m,d}}\le 1$$

$$\frac{1,24}{9,69}+\frac{3,14}{16,62}\le 1$$

$0,32 \le 1$

Shear stress in timber caused by load combination $q_{Sd,1}$

$$\tau_{\max(q_{Sd,1})}=\frac{V_{d,q_{Sd,1}}}{(b_{eff}\cdot h_t)}\le f_{v,d}$$

$$\tau_{\max(q_{Sd,1})}=\frac{10020}{(120,6\cdot 240)}\le f_{v,d}$$

$\tau_{\max(qSd,1)} = 0,35$ N/mm² ≤ 1,73 N/mm²

$$f_{v,k}=2,5\ N/mm^2\rightarrow f_{v,d}=k_{mod}\cdot\frac{f_{v,k}}{\gamma_{M,t}}=0,9\cdot\frac{2,5}{1,3}=1,73\ N/mm^2$$

Verification of pressure perpendicular to fibres on the support, for load combination $q_{Sd,1}$

$$\sigma_{c,90(q_{Sd,1})}=\frac{V_{d,q_{Sd,1}}}{l_b b_t}=\frac{10020}{200\cdot 180}=0,28\ N/mm^2$$

The following conditions must be met for the concrete part of cross-section:

- Longitudinal stress in concrete caused by longitudinal force, from load combination $q_{Sd,1}$

$$\sigma_{c_{(q_{Sd,1})}}=\frac{\gamma_c E_{cm,eff}a_c M_{d,q_{Sd,1}}}{(EI)_{eff}}=\frac{0,41\cdot 6777,78\cdot 126,76\cdot 15,03\cdot 10^6}{5,35\cdot 10^{12}}=0,99\ N/mm^2$$

- Longitudinal stress in concrete by bending moment, from load combination $q_{Sd,1}$

$$\sigma_{m,c_{(q_{Sd,1})}}=\frac{1}{2}\cdot\frac{E_{cm,eff}h_c M_{d,q_{Sd,1}}}{(EI)_{eff}}=\frac{1}{2}\cdot\frac{6777,78\cdot 60\cdot 15,03\cdot 10^6}{5,35\cdot 10^{12}}=0,57\ N/mm^2$$

- Total stress at top edge of concrete:

$\sigma_{c,g} = \sigma_{c(qSd,1)} - \sigma_{m,c(qSd,1)} < f_{cd}$

$\sigma_{c,g}$ = -0,99-0,57 = -1,56 M/mm² < 16,67 N/mm²

- Total stress at bottom edge of concrete:

$\sigma_{t,d} = -\sigma_{c(qSd,1)} + \sigma_{m,c(qSd,1)} < f_{cd}$

$\sigma_{t,d}$ = -0,99+0,57 = -0,42 M/mm² < 1,2 N/mm²

$\sigma_{t,d}$ = -0,99+0,57 = -0,42 M/mm² < 2,2 N/mm²

$$f_{ctk;0,05}=1,8\,N/mm^2\rightarrow f_{ctd}=\frac{f_{ctk;0,05}}{\gamma_{M,c}}=\frac{1,8}{1,5}=1,2\ N/mm^2$$

$$f_{ctk;0,95}=3,3\,N/mm^2\rightarrow f_{ctd}=\frac{f_{ctk;0,95}}{\gamma_{M,c}}=\frac{3,3}{1,5}=2,2\ N/mm^2$$

3.2.2. Verification for serviceability limit state:

The effective elastic modulus for the long-term load of concrete is defined as follows:

$$E_{cm,eff} = \frac{E_{cm}}{1+\varphi(t,t_0)} = \frac{30500}{1+3,5} = 6777,78\,N/mm^2$$

Final mean elastic modulus of timber:

$$E_{0,mean,fin} = \frac{E_{0,mean}}{(1+k_{def})} = \frac{11000}{(1+0,6)} = 6875\,N/mm^2$$

Sliding modulus:

$$K_{ser,fin} = \frac{K_{ser}}{(1+k_{def})} = \frac{14969,5}{(1+0,6)} = 9355,94\,N/mm$$

Sliding coefficients:

$\gamma_t = 1$

$$\gamma_c = \frac{1}{1+\dfrac{\pi^2 E_{cm,eff} A_c s}{K_{ser,fin} L^2}} = \frac{1}{1+\dfrac{\pi^2 \cdot 6777,78 \cdot 54000 \cdot 120}{9355,94 \cdot 6000^2}} = 0,44$$

Eccentricity of centres of the timber (a_t) and concrete (a_c) parts of cross-section:

$$a_t = \frac{\gamma_c E_{cm,eff} A_c (h_c + h_t)}{2(\gamma_c E_{cm,eff} A_c + E_{0,mean,fin} A_t)} = \frac{0,44 \cdot 6777,78 \cdot 54000 \cdot (60+240)}{2 \cdot (0,44 \cdot 6777,78 \cdot 54000 + 6875 \cdot 43200)} = 52,74\ mm$$

$$a_c = \frac{h_c + h_t}{2} - a_t = \frac{60+240}{2} - 51,31 = 98,69\,mm$$

Effective flexural stiffness of shear-flexible composite beam::

$$\begin{aligned}(EI)_{eff} &= E_{cm,eff} I_c + E_{o,mean,fin} I_t + \gamma_c E_{cm,eff} A_c a_c^2 + \gamma_t E_{o,mean,fin} A_t a_t^2 \\ &= 6777,78 \cdot 16200000 + 6875 \cdot 207360000 + 0,44 \cdot 6777,78 \\ &\quad \cdot 54000 \cdot 98,69^2 + 1 \cdot 7333,33 \cdot 43200 \cdot 52,74^2 \\ &= 3,99 \cdot 10^{12}\ Nmm^2 \\ &= 3,99 \cdot 10^{10}\ Ncm^2\end{aligned}$$

Final deflection due to permanent load:

g = 3,11 kN/m²·0,9 m = 2,80 kN/m'

$$M_g = \frac{gL^2}{8} = \frac{2,80 \cdot 6^2}{8} = 12,6\,kNm = 12,6 \cdot 10^6\ Nmm$$

$$u_{fin}^{Gk,j} = \frac{5}{48} \cdot \frac{M_g \cdot L^2}{(EI)_{eff}} = \frac{5}{48} \cdot \frac{12,6 \cdot 10^6 \cdot 6000^2}{3,99 \cdot 10^{12}} = 11,84\ mm$$

Final deflection due to variable load:

q = 2,0 kN/m²·0,9 m = 1,8 kN/m'

$$M_q = \frac{qL^2}{8} = \frac{1,8 \cdot 6^2}{8} = 8,1\,kNm = 8,1 \cdot 10^6\ Nmm$$

$$u_{fin}^{Qk,j} = \frac{5}{48} \cdot \frac{M_q \cdot L^2}{(EI)_{eff}} = \frac{5}{48} \cdot \frac{8,1 \cdot 10^6 \cdot 6000^2}{3,99 \cdot 10^{12}} = 7,61\ mm$$

The following conditions must be met for final deflection:

$$u_{fin}^{Gk,j} \le \frac{L}{200}$$

11,84 mm ≤ 30 mm

$$u_{fin}^{Gk,j} \le \frac{L}{200}$$

7,61 mm ≤ 30 mm

The following conditions must be met for final deflection due to permanent and variable load:

$$u_{fin}^{Gk,j} + u_{fin}^{Qk,j} \le \frac{L}{200}$$

11,84 + 7,61 mm ≤ 30 mm

19,45 mm ≤ 30 mm

3.3. Analysis of required reinforcement

The concrete slab must be strengthened with minimum reinforcement so as to ensure proper ductility of cross-section, as well as a lower influence of creep and shrinkage of concrete. This analysis is clearly defined in Eurocode 2 [9].

3.4. Analysis of connectors

The analysis of bearing capacity of composite beam connectors is not defined in Eurocode 5. In this paper, the analysis will be made according to [7, 8].

Parameters for connectors:

- Compressive strength of timber along periphery of hole for load in the direction of fibres:

$$f_{h,o,k} = 0,082 \cdot (1-0,01 \cdot d) \cdot \rho_k = 0,082 \cdot (1-0,01 \cdot 20) \cdot 420 = 27,552\ N/mm^2$$

$$f_{h,o,d} = k_{mod} \cdot \frac{f_{h,o,k}}{1,3} = 0,9 \cdot \frac{27,552}{1,3} = 19,07\ N/mm^2$$

- Typical tensile strength of steel for the construction of connectors S 275:

$f_{u,k}$ = 430 N/mm²

$$f_{u,d} = \frac{f_{u,k}}{1,1} = \frac{430}{1,1} = 390,9\ N/mm^2$$

- Liquid moment for connector:

$$M_y = \frac{f_u}{600} \cdot 180 \cdot d^{2,6} = \frac{390,9}{600} \cdot 180 \cdot 20^{2,6} = 283051,1436\ Nmm$$

- Thickness of intermediate layer (board formwork):

t = 2,4 cm = 24 mm

$$\beta = \frac{f_c}{f_{h,o,d}} = \frac{25}{19,07} = 1,31$$

- Resistance for model representing elastic ideally plastic behaviour of concrete:

$$F_{p,c} = f_{h,o,d} \cdot d \cdot \sqrt{\frac{2\beta}{1+\beta}} \cdot \sqrt{\frac{2M_y}{f_{h,o,d} \cdot d} + \frac{\beta}{1+\beta} \cdot \frac{t^2}{2}} - \frac{\beta}{1+\beta} \cdot t$$

$$= 19{,}07 \cdot 20 \cdot \sqrt{\frac{2 \cdot 1{,}31}{1+1{,}31}} \cdot \sqrt{\frac{2 \cdot 283051{,}1436}{19{,}07 \cdot 20} + \frac{1{,}31}{1+1{,}31} \cdot \frac{24^2}{2}} - \frac{1{,}31}{1+1{,}31} \cdot 24 = 16473{,}75 N$$

- Resistance for model representing linear elastic behaviour of concrete

$$F_{e,c} = \sqrt{4 \cdot M_y \cdot f_{h,o,d} \cdot d} = \sqrt{4 \cdot 283051{,}1436 \cdot 19{,}07 \cdot 20} = 20780{,}35 \ N$$

- Resistance for model representing linear elastic behaviour with concrete crushing

$$F_{cr,c} = d \cdot f_{h,o,d} \cdot \left(-e + \sqrt{e^2 + \frac{4M_y}{d \cdot f_{h,o,d}}} \right) = 20 \cdot 19{,}07 \cdot \left(-5 + \sqrt{5^2 + \frac{4 \cdot 283051{,}1436}{20 \cdot 19{,}07}} \right) = 18960{,}67 \ N$$

- Reference resistance: $F_{p,c}$ = 16473,75 N

Load imposed on connectors – for cross-section with maximum transverse force:

→ for SHORT-TERM DEFORMATIONS

$$F_{t1,d} = \gamma_t \frac{E_t A_t a_t s_{min}}{(EI)_{eff}} V_d = 1 \cdot \frac{11000 \cdot 43200 \cdot 52{,}54 \cdot 120}{6{,}5 \cdot 10^{12}} \cdot 19440 = 8960{,}47 \ N$$

$F_{t1,d} \leq F_{p,c}$

8960,47 N ≤ 16473,75 N

→ for LONG-TERM DEFORMATIONS

$$F_{t2,d} = \gamma_t \frac{E_t A_t a_t s_{min}}{(EI)_{eff}} V_d = 1 \cdot \frac{9565{,}22 \cdot 43200 \cdot 47{,}18 \cdot 120}{5{,}49 \cdot 10^{12}} v10020 = 4269{,}86 \ N$$

$F_{t2,d} \leq F_{p,c}$

4269,86 N ≤ 16473,75 N

4. Conclusion

The analysis of a timber-concrete composite girder is presented in this professional paper. The central part of the paper contains a detailed numerical example or static analysis of a girder, which takes into account not only short-term deformations, but also long-term deformations due to rheological phenomena. The analysis of short-tem deformations was conducted using the γ procedure, which is defined in Eurocode 5 [2, 3]. The analysis of long-term deformations for composite systems is still not fully defined. Deformations depend on the content of moisture in timber, on the shrinkage, swelling and creep of timber, on the shrinkage, creep and temperature changes in concrete, and on the sliding of connectors. The analysis presented in the paper takes into account the influence of moisture in timber, creep of concrete and timber, and the influence of sliding of connectors. These phenomena exert an influence on effective stiffness, and hence on the stress and deflection values. The presented example takes into account the ultimate limit state and serviceability limit state. An example of analysis of bearing capacity of connectors for such girders is given in the final part of the study. Further experimental study of these girders can be made in order to make corrections to design model of resistance, and reliability analysis. Due to complexity of this area, the analysis of reliability of such girders, which should be conducted using methodology presented in [10], is highly demanding, but the authors consider that it is nevertheless indispensable.

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