

Timber-concrete composite structures

STEP lecture E13
A. Ceccotti
University of Florence

Objectives

To describe composite timber-concrete load-bearing structures and to discuss the parameters affecting their design.

Summary

This lecture deals with the composite timber-concrete technique with special reference to its performance in buildings. In particular this lecture considers structural elements where the cross-section is built from both materials linked by mechanical fasteners. Therefore the most important features of this technique and its hygro-thermo-mechanical behaviour are illustrated from a designer's point of view. A simple design example is included.

Introduction

Why is it that in some countries timber-concrete composite structures are popular? Because the coupling of a concrete layer on the compression side and of a timber section on the tension side, allows the best properties of these two materials to be utilised. In fact concrete is used only in compression, where it gives its best performance in terms of strength and stiffness, and timber is used in tension, so that concrete in tension, which is only dead weight, is eliminated. Therefore it is possible to have a structurally efficient section, rigid and light at the same time. In this way the load carrying capacity of a traditional timber floor can be doubled and its out-of-plane rigidity improved three or four times. Figure 1 presents the self weight for a given service load-span combination for different types of floors.

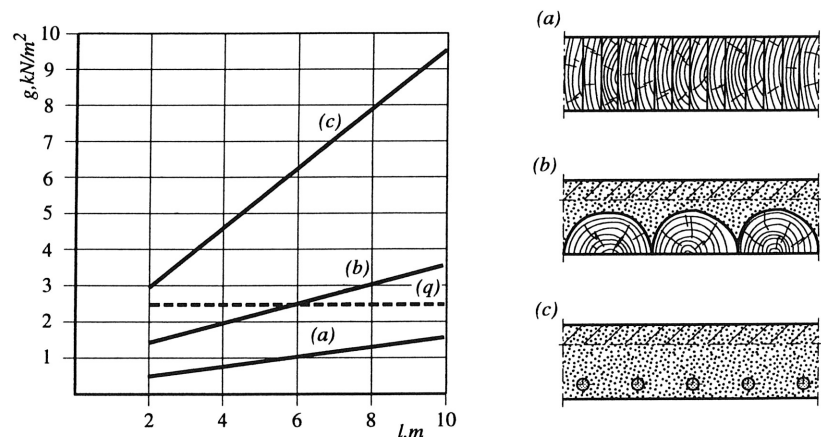


Figure 1 Floor self weight g versus span l for a service load q of 2.5 kN/m^2 , in the case of (a) an all-timber, (b) timber-concrete and (c) all-concrete section - from Natterer (1993).

Moreover, the spring effect, so annoying to the user when jumping or just walking on the floor, is reduced; and vibrational damping is, in general, closer to 2% than to 1%, which means that serviceability verifications for vibration are more easily satisfied.

Further, the in-plane rigidity becomes so large that it may be considered infinite.

In other words a floor, for example, is so rigid that it is able to keep its shape and consequently the shape of the entire building. This is very important for example for the survival of the structure under an earthquake. In addition this permits seismic calculation procedures based on this assumption. This is not the case for a masonry structure with simple timber floors without any in-plane rigidity as in old European constructions, see STEP lecture D10 (Figure 9).

Of course it is necessary that both the timber beams and the concrete slab are well connected to the masonry wall (see Figure 2).

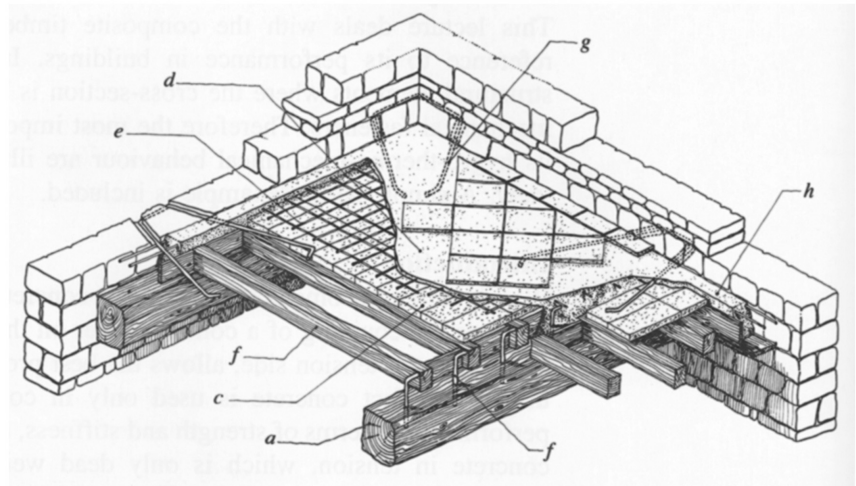


Figure 2 Example of earthquake-resistant design with an existing timber floor in a middle-European masonry building. (a) main beam; (b) secondary beam; (c) brick tiles; (d) concrete slab; (e) steel mesh; (f) steel fasteners epoxy glued in to timber; (g) steel stirrups connecting concrete layer with masonry; (h) all-around reinforced concrete girder.

Sound insulation is also improved. On one hand, for air-transmitted noises, it is improved with respect to an all-timber floor due to the increased mass of the floor, and on the other hand, for impact noises, sound insulation is improved with respect to an all-concrete floor due to the higher damping.

Regarding fire the upper concrete slab constitutes an efficient barrier against fire propagation that increases the fire resistance in comparison with an all-timber floor. In addition on the bottom timber joists are more fire-resistant than corresponding joists made of steel or pre-fabricated prestressed concrete.

Finally, the cost of a composite timber-concrete floor is competitive when compared with an all-concrete floor. Actually it is not only the cost/square metre of the product per se, as may be found in a common building prices list, that has to be taken into account, because there are other factors that contribute to saving money on the rest of the structure and on the building site too (e.g. more rapid execution, less concrete formwork and less stabilisation needs because timber elements can partly provide these features themselves, reduced foundations because of less structural weight, etc.).

For all the above reasons composite timber concrete structures are popular in some countries, both in the case of renovation works of old timber structures (i.e. timber

floors in old masonry structures) and in the case of new floors in new masonry houses too. This technique is also used for bridges (mostly in America and in the Pacific region).

This technique has also been used, although less frequently, for composite walls where the concrete layer gives the racking strength and the timber ribs contribute to the flexural stiffness out-of-plane and provide the necessary stiffening against buckling.

In principle, however, it could appear very difficult to obtain a good marriage between these two materials, because their mechanical properties and their hygro-thermal behaviour are so different. But on the other hand no collapses or difficult serviceability problems have been reported in twenty years of use (RILEM, 1992). In the following pages the reasons why this marriage can work and the main design criteria to be followed to ensure good results will be given.

Types of connection

In Figure 3 a table of the most commonly used connection systems is shown.

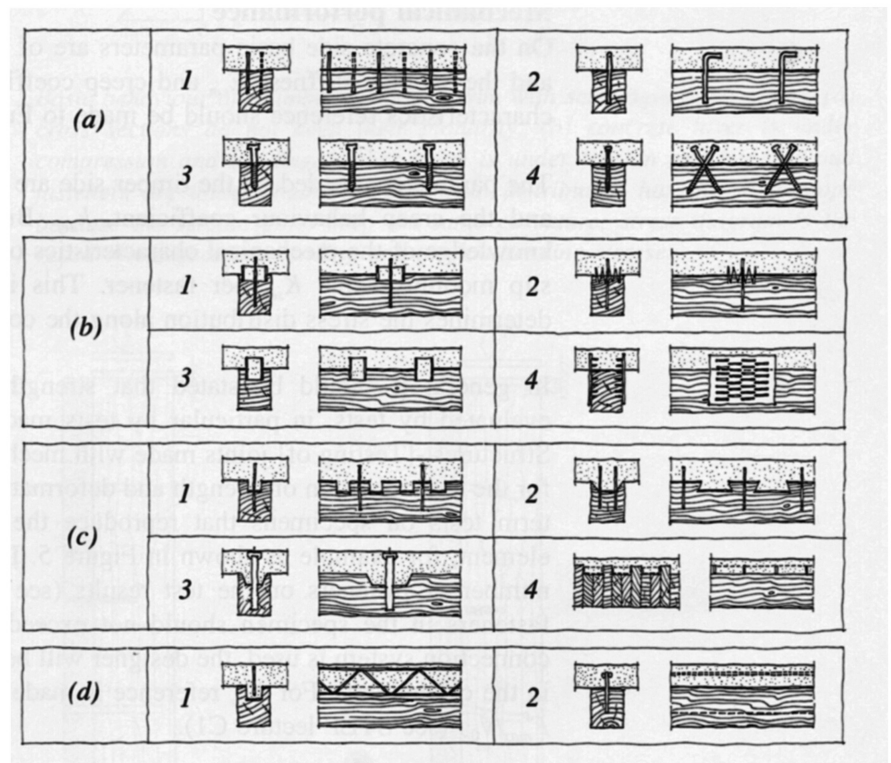


Figure 3 Examples of different timber-concrete connection systems. (a1) nails; (a2) reinforced concrete steel bars, glued; (a3/4) screws; (b1/2) connectors, split rings and toothed plates, respectively; (b3) steel tubes; (b4) steel punched metal plates; (c1) round holes in timber and fasteners preventing uplifting; (c2) square indentation and fasteners; (c3) cup indentation and prestressed steel bars; (c4) nailed timber planks deck and steel shear plates slotted through the deeper planks; (d1) steel lattice glued to timber; (d2) steel plate glued to timber.

The stiffness of the connection system may be assumed as a sort of classification index. For example, elements connected by nails, screws or dowel shaped fasteners

(A) are less rigid than elements connected by surface connectors (B) and even less rigid than elements when some notches have been cut into the wood itself (C). The stiffest connections are those where a bond between concrete and wood is obtained (D). For cases A,B and C the behaviour is like that of composite structures with a semi-rigid behaviour where the main point is that cross sections do not keep their planarity, see Figure 4. Only in sections with a D system of connection will plane sections remain plane.

Roughly speaking it is possible to say that the values of the effective bending stiffness El_{ef} may be about 50%, with A types, and up to 100%, with D types, of the bending stiffness of the correspondent sections rigidly connected.

Design calculations in D case may be made easily, since there is no slip, by just "transforming" the concrete section into an equivalent timber section having the same centre of gravity but with an increased width E_c/E_t times the real width, or by just using the method described in STEP lecture B10 for glued composite sections.

In the other cases, when the semi-rigid behaviour is obtained, the method, used for mechanically jointed beams and columns (see STEP lecture B11) may be used.

Mechanical performance

On the concrete side basic parameters are of course the characteristic strength f_{ck} and the average stiffness E_{cm} and creep coefficient ϕ of concrete layer. For these characteristics reference should be made to Eurocode 2.

The parameters needed on the timber side are the strength $f_{m,k4}$ and stiffness $E_{0,mean}$ and the creep behaviour coefficient, k_{def} . But the most important point is the knowledge of the mechanical characteristics of the joints, mainly stiffness, i.e. the slip modulus value K_{ser} per fastener. This is because the rigidity of the joint determines the stress distribution along the composite structural element.

In general it should be stated that strength and stiffness properties must be evaluated by tests, in particular by tests made according to EN 26891 "Timber Structures -[Testing of] joints made with mechanical fasteners - General principles for the determination of strength and deformation characteristics", that means short term tests on specimens that reproduce the real arrangement in the structural element, for example as shown in Figure 5. In order to avoid an influence of the number of fasteners on the test results (see STEP lecture C15) the number of fasteners in the specimen should not exceed two. In this way, no matter what connection system is used, the designer will be able to use the correct slip modulus in the calculations. For K_{ser} reference is made to the initial slip modulus $K_{ser} = 0,4 F_{max}/v_{0,4}$ (see STEP lecture C1).

It is useful here to remind the reader that, when performing global analysis i.e. calculating internal actions and the consequent stress distribution at an ultimate limit state, mean values of material stiffnesses and slip modulus of connections have to be used. Actually this is because in EC5 only the mean values of slip moduli are given and the characteristic values are not available. Therefore only the mean values of the modulus of elasticity may be used: in fact if using at the same time the characteristic value of the modulus of elasticity and the mean value of the modulus of slip the calculated values of the resultant stresses would be on the unsafe side.

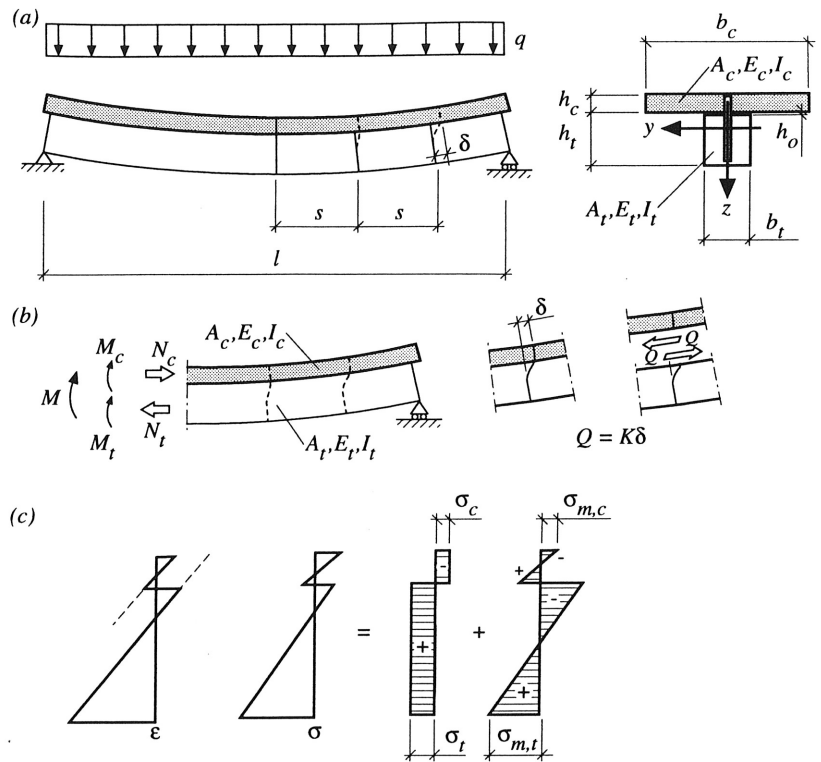


Figure 4 Basic behaviour of a timber-concrete beam with semi-rigid connections. (a) cross sections do not keep their planarity; (b) concrete layer is under compression and bending, timber beam is under tension and bending and fasteners are under shear; (c) the strain distribution has the same slope because the section parts keep the same curvature; stress diagram is the result of compression-bending and tension-bending stresses.

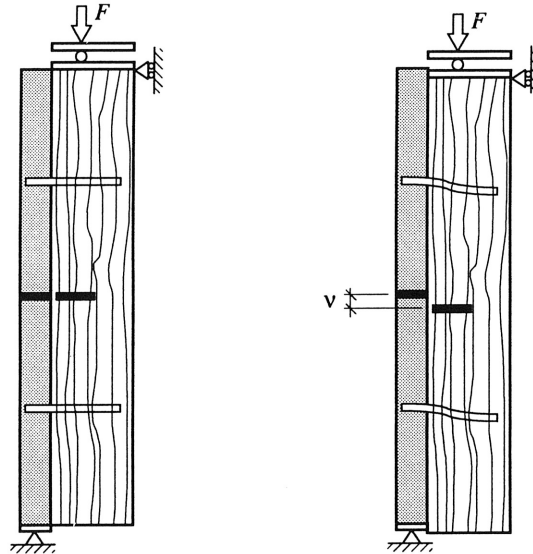


Figure 5 A possible test arrangement for determining load-slip behaviour of a connection system.

Another reason for using mean values is that Eurocode 2 only specifies a nominal value for the modulus of elasticity of the concrete, which is assumed to be an average value. Of course when performing the cross-section verification i.e. calculating design strengths, reference has to be made to 5-percentile characteristic resistances.

An elastic analysis is allowed also at the ultimate states. That means that not only timber is considered linearly elastic but also joints and concrete that have, in fact, a very well pronounced plastic behaviour. This is allowed, and is on the safe side, by considering a "nominal" secant modulus of elasticity for concrete (see Eurocodes 2 and 4) and an "equivalent" secant slip modulus for joints. For global analysis i.e. for the calculation of the global internal actions, normal force and moment, on the concrete layer and the timber beam, the concrete part is considered uncracked. This means that the entire second moment of area " I_c " must be considered. On the other hand, for the cross-section verification the concrete will be considered to have no tensile strength. For the concrete compression side the verification is made assuming as upper limit of resistant strength the limit value of the plasticization stress in compression (according to Eurocode 2). On the tension side, if it is the case of a cracked section, reinforcing bars shall be provided. For the slip modulus of joints reference is made to the K_u value where $K_u = 0,6 F_{max}/v_{0,6}$ according to EN 26891. In general K_u is not so different from $2/3 K_{ser}$.

For long term behaviour, reference has to be made to creep performances of concrete, timber and joints utilising their relevant creep coefficient as conventional reduction factors. For the concrete to timber connection just the coefficient for timber joints will be used, considering that for the most part joint deformation will occur in the timber. Of course, it is evident that the method of reducing the elastic moduli is only a conventional system for taking into account time deformation but the real value of elastic moduli has no reduction in time. This method tends to over-estimate the real deformation at the final stage as it can be calculated, for example, utilising a step by step method. Therefore it is on the safe side from a designer's point of view.

Constraint forces would arise when the two parts are subjected to dimensional variations: e.g. when concrete shrinks. In this particular case the length reduction of the concrete slab will favour fasteners because it tends to reduce their deformation. On the other hand this would increase the deflection of the beam, although this can be counteracted by giving the beam a camber. But, finally, most of the shrinkage happens when the entire structure is still propped and the usual cracks in the concrete layer will reduce the importance of the phenomenon dramatically.

Most interesting is the case of temperature variations in the concrete (the concrete layer is more affected by temperature variations than by environmental humidity variations) and moisture variations in the timber. The level of stress that can be calculated by a simple elastic calculation utilising the conventionally reduced elastic moduli, will only have a significant influence for very stiff connections and for longer elements.

Recommendations for design and construction

Do not use wet timber. If it is unavoidable, use timber without pith or be sure that fissures will not affect fastener lines. Leave the propping in place for more than the time allowed for all-concrete elements.

Use corrosion-protected fasteners: zinc-coated steel or stainless steel.

Reinforce the concrete especially if thick concrete sections are being designed, in order to avoid loss of stiffness due to large cracks on the concrete tension side.

When casting try to protect the timber from moisture, i.e. using plastic layers or using concrete with additives in order to reduce the water/cement ratio (that also allows smaller concrete shrinkage). This is not crucial for timber but for appearance underneath. Pay attention to timbers that do not allow the concrete to harden (e.g. in the case of larch, due to sugar extractive).

With increasing spans prefer softer connections in order to minimise eventual constraint actions, and, when possible, the author prefers a structure where the concrete layer is mainly important for reducing deflections rather than for reducing the stress values in the timber.

Design example - data

A composite timber-concrete simply supported floor has a span $l = 4,00 \text{ m}$, beams spacing $0,50 \text{ m}$ and semi-rigid connections like type A2 in Figure 3. Dimensions are shown in Figure 6.

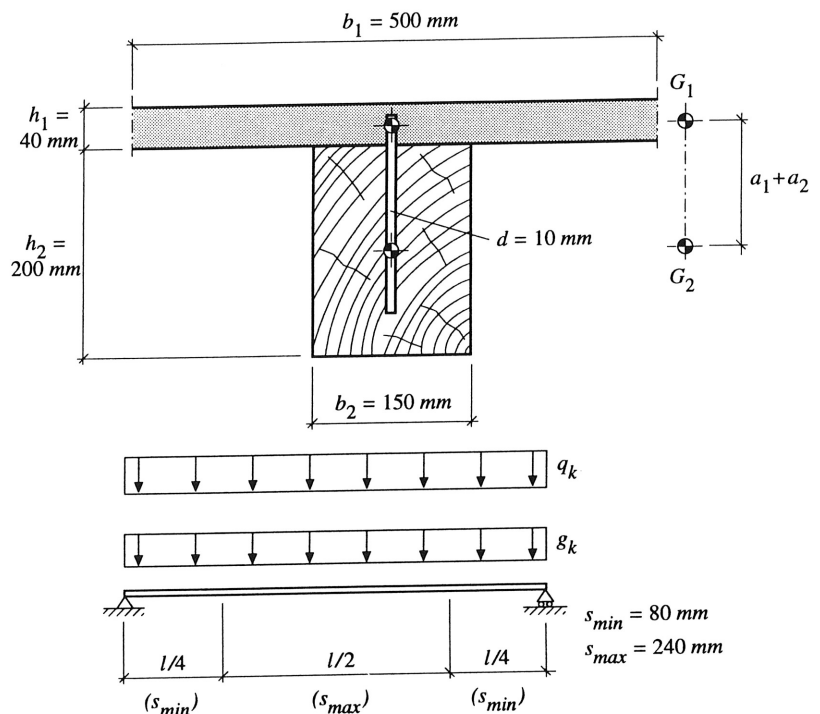


Figure 6 Example of a composite beam with semirigid connections under bending actions: reference values (s , fasteners spacing).

Action side

Characteristic values of permanent and variable loads, per beam:

permanent load $g_k = 0,70 \text{ kN/m}$
variable load $q_k = 4,15 \text{ kN/m}$ (medium term)

I design load combination (only permanent)

$$M_{d,I} = 1,35 g_k l^2 / 8 = 1,9 \text{ kNm}$$

$$V_{d,I} = 1,35 g_k l / 2 = 1,9 \text{ kN}$$

II design load combination (permanent + medium term)

$$M_{d,II} = (1,35 g_k + 1,5 q_k) l^2 / 8 = 14,3 \text{ kNm}$$

$$V_{d,II} = (1,35 g_k + 1,5 q_k) l / 2 = 14,3 \text{ kN}$$

Note: because the ratio $1,35 g_k / (1,35 g_k + 1,5 q_k) = 0,13$ is much less than the ratio $k_{mod,perm} / k_{mod,med.term}$ ($0,6/0,8 = 0,75$) it is evident that only the II load combination is decisive. For this reason, in the following, reference will be made only to the medium term load combination.

Material side

Top slab: concrete strength class C25/30, according to ENV 206 "Concrete performance, production, placing and compliance criteria". Effective flange width b_1 is calculated according to EC4 as $2l/8$ but not greater than the beam spacing, 500 mm.

Beam: solid timber, strength class C22, according to prEN 338 "Structural timber. Strength classes".

Fasteners: steel bars for reinforced concrete B500B, according to EN 10080, $d = 10 \text{ mm}$, $s_{ef} = 0,75 s_{min} + 0,25 s_{max} = 120 \text{ mm}$.

Service Class 1.

Material properties

The characteristic strength values as well as the modulus of elasticity are taken from EC2 and prEN 338, respectively. As already said, for the modulus of elasticity and the slip modulus of the connection, the mean value is used in the design although an ultimate limit state is considered.

Concrete	$f_{ck,cube} = 30 \text{ N/mm}^2$	$f_{ctm} = 2,6 \text{ N/mm}^2$	$E_{cm} = 30000 \text{ N/mm}^2$
	$\phi_{\infty,10} = 2,25$ (permanent load)		
	$\phi_{1,10} = 1,35$ (medium term load)		

Timber	$f_{m,k} = 22 \text{ N/mm}^2$	$f_{t,0,k} = 13 \text{ N/mm}^2$	$f_{v,k} = 2,4 \text{ N/mm}^2$
	$\rho_{0,k} = 340 \text{ kg/m}^3$	$f_{h,0,k} = 25,1 \text{ N/mm}^2$	$E_{0,mean} = 10000 \text{ N/mm}^2$

EC5: part 1-1: 3.1.7	Service class 1:	$k_{mod} = 0,8$ (medium term load combination)
EC5: part 1-1: 4.1		$k_{def} = 0,6$ (for permanent load)
		$k_{def} = 0,25$ (for medium term load)

Fasteners	$f_{u,k} = 500 \text{ N/mm}^2$	$M_{y,k} = 0,8 \cdot 500 \cdot 10^3 / 6 = 66700 \text{ Nmm}$
	$K_{ser} = 0,125 d E_{0,mean}$	$= 12500 \text{ N/mm}$
	$K_u = 2 K_{ser} / 3$	$= 8330 \text{ N/mm}$

according to test results valid when deformation in the concrete may be considered negligible, i.e. the case when $f_{ck,cube} \geq 30 \text{ N/mm}^2$ and the height of the fastener penetrating the concrete is bigger than $3d$.

Design strength values are calculated according to EC2, EC4 and EC5 respectively.

Concrete

$$f_{cd} = \frac{0,83 \cdot 0,85 f_{ck,cube}}{1,5} = 14,1 \text{ N/mm}^2$$

$$f_{ctmd} = \frac{0,85 f_{ctm}}{1,5} = 1,47 \text{ N/mm}^2$$

Timber

$$f_{m,d} = \frac{0,8 \cdot 22}{1,3} = 13,5 \text{ N/mm}^2$$

$$f_{t,0,d} = \frac{0,8 \cdot 13}{1,3} = 8,00 \text{ N/mm}^2$$

$$f_{v,d} = \frac{0,8 \cdot 2,4}{1,3} = 1,47 \text{ N/mm}^2$$

$$f_{h,0,d} = \frac{0,8 \cdot 25,1}{1,3} = 15,4 \text{ N/mm}^2$$

Connections

concrete side (localised compression)

$$R_d = P_{Rd} = 0,23 d^2 \sqrt{f_{ck} \frac{E_{cm}}{\gamma_v}} = 14,6 \text{ kN}$$

where

d is the dowel diameter,

f_{ck} is the characteristic cylinder strength of the concrete ($f_{ck} = 1,5 f_{cd}$),

γ_v is the partial factor of the material ($\gamma_v = 1,25$),

E_{cm} is the nominal value of the modulus of elasticity of the concrete

$$E_{cm} = 30000 \text{ N/mm}^2$$

shear failure of the fastener

$$R_d = P_{Rd} = 0,8 f_u \frac{\pi d^2}{4 \gamma_v} = 25,1 \text{ kN}$$

where f_u is the specified tensile strength of the steel of the dowel ($f_u = 500 \text{ N/mm}^2$).

timber side

$$R_d = 1,5 \sqrt{2 M_{y,d} f_{h,2,d} d} = 6,49 \text{ kN}$$

Computation

Ultimate state

Initial stage

The effective bending stiffness of the cross-section is calculated according to EC5, Annex B. The writer suggests a small change in the EC5 formula for the a_2 calculation: i.e. change the expression $(h_1 + h_2)/2$ into $(a_1 + a_2)$. In this way it is possible to take into account also cross-sections that have a gap between part 1 (concrete) and part 2 (timber).

Values of cross-section:

$$A_1 = 20000 \text{ mm}^2$$

$$A_2 = 30000 \text{ mm}^2$$

$$I_1 = 2,66 \cdot 10^6 \text{ mm}^4$$

$$I_2 = 100 \cdot 10^6 \text{ mm}^4$$

$$\gamma_1 = 0,16$$

$$\gamma_2 = 1,00$$

$$a_1 = 91,2 \text{ mm}$$

$$a_2 = 28,8 \text{ mm}$$

$$EI_{ef} = 2,12 \cdot 10^{12} \text{ Nmm}^2$$

Design normal stresses:

Concrete side

$$\sigma_{c,1,d} = 2,93 \text{ N/mm}^2$$

$$\begin{aligned}
\sigma_{m,1,d} &= 4,06 \text{ N/mm}^2 \\
\sigma_{c,d} &= 2,93 + 4,06 = 6,99 < 14,1 \text{ N/mm}^2 \\
\sigma_{t,d} &= 4,06 - 2,93 = 1,13 < 1,47 \text{ N/mm}^2 \quad \text{concrete side,}
\end{aligned}$$

the last formula states that concrete is not cracked; therefore calculation stops here (see "Mechanical performances");

timber side,

$$\begin{aligned}
\sigma_{t,2,d} &= 1,95 \text{ N/mm}^2 \\
\sigma_{m,2,d} &= 6,77 \text{ N/mm}^2
\end{aligned}$$

$$\frac{\sigma_{t,2,d}}{f_{t,0,d}} + \frac{\sigma_{m,2,d}}{f_{m,d}} = 0,71 \leq 1$$

Design shear stress in the beam:

on the safe side and for the sake of simplicity the shear force V_d is considered to be totally carried by the timber beam,

$$\tau_{2,max} = 0,72 \text{ N/mm}^2 < 1,47 \text{ N/mm}^2$$

Design fastener load:

$$F_{1,d} = \frac{\gamma_1 E_1 A_1 a_1 s_{\min} V_d}{EI_{ef}}$$

$$F_{1,d} = \frac{0,16 \cdot 30000 \cdot 20000 \cdot 91 \cdot 80 \cdot 14300}{2,12 \cdot 10^{12}} = 4,68 \text{ kN} < 6,49 \text{ kN}$$

Final stage

With time due to the higher creep of concrete with respect to timber, action forces tend to migrate from concrete to timber. In other words stresses decrease in the concrete and increase in the timber. At final stage an effective modulus of elasticity can be used which is calculated as an average value of k_{def} coefficients weighted according to the loads (i.e. in this case 15% for permanent and 85% for medium-term load).

The new fictitious moduli of elasticity (and consequently slip modulus) to be taken into account are therefore the following:

$$E_{1,ef} = 30000 \left(\frac{0,15}{1 + 2,25} + \frac{0,85}{1 + 1,35} \right) = 12200 \text{ N/mm}^2$$

$$E_{2,ef} = 10000 \left(\frac{0,15}{1 + 0,60} + \frac{0,85}{1 + 0,25} \right) = 7740 \text{ N/mm}^2$$

$$K_u = 2 K_{ser} / 3 = 6450 \text{ N/mm}$$

Values of cross-section

$$\begin{aligned}
A_1 &= 20000 \text{ mm}^2 & A_2 &= 30000 \text{ mm}^2 \\
I_1 &= 2,66 \cdot 10^6 \text{ mm}^4 & I_2 &= 100 \cdot 10^6 \text{ mm}^4 \\
\gamma_1 &= 0,26 & \gamma_2 &= 1,00 \\
a_1 &= 94 \text{ mm} & a_2 &= 26 \text{ mm} \\
EI_{ef} &= 1,53 \cdot 10^{12} \text{ Nmm}^2
\end{aligned}$$

Design normal stresses

Concrete side

$$\sigma_{c,1,d} = 2,83 \text{ N/mm}^2$$

$$\begin{aligned}
\sigma_{m,1,d} &= 2,29 \text{ N/mm}^2 \\
\sigma_{c,d} &= 2,83 + 2,29 = 5,12 < 14,1 \text{ N/mm}^2 \\
\sigma_{t,d} &= 2,83 - 2,29 = 0,54 \text{ N/mm}^2 \text{ (Compression)}
\end{aligned}$$

Timber side

$$\begin{aligned}
\sigma_{t,2,d} &= 1,88 \text{ N/mm}^2 \\
\sigma_{m,2,d} &= 7,25 \text{ N/mm}^2 \\
\frac{\sigma_{t,2,d}}{f_{t,0,d}} + \frac{\sigma_{m,2,d}}{f_{m,d}} &= 0,77 \leq 1
\end{aligned}$$

Serviceability limit state - Deflections

Initial stage

Values of cross-section

$$\begin{aligned}
A_1 &= 20000 \text{ mm}^2 & A_2 &= 30000 \text{ mm}^2 & E_{cm} &= 30000 \text{ N/mm}^2 \\
E_{0,mean} &= 10000 \text{ N/mm}^2 & K_{ser} &= 12500 \text{ N/mm} \\
\gamma_1 &= 0,22 \\
a_2 &= 36,7 \text{ mm} \\
a_1 &= 83,3 \text{ mm} \\
(EI)_{ef} &= 2,4 \cdot 10^{12} \text{ Nmm}^2
\end{aligned}$$

$$\begin{aligned}
u_{1,inst} &= 1,0 \text{ mm} \\
u_{2,inst} &= 5,8 \text{ mm} = l / 690 < l / 300
\end{aligned}$$

Final stage

permanent load

$$\begin{aligned}
E_{cm} &= 30000/(1+2,25) = 9230 \text{ N/mm}^2 \\
E_{0,mean} &= 10000/(1 + 0,60) = 6250 \text{ N/mm}^2 \\
K_{ser} &= 12500/(1 + 0,60) = 7810 \text{ N/mm} \\
\gamma_1 &= 0,36 \\
a_2 &= 31,7 \text{ mm} & a_1 &= 88,3 \text{ mm} \\
(EI)_{ef} &= 1,36 \cdot 10^{12} \text{ Nmm}^2
\end{aligned}$$

$$u_{1,fin} = 1,7 \text{ mm}$$

medium term load

$$\begin{aligned}
E_{cm} &= 30000/(1 + 1,35) = 12760 \text{ N/mm}^2 \\
E_{0,mean} &= 10000/(1+0,25) = 8000 \text{ N/mm}^2 \\
K_{ser} &= 12500/(1 +0,25) = 10000 \text{ N/mm} \\
\gamma_1 &= 0,35 \\
a_2 &= 32,3 \text{ mm} & a_1 &= 87,7 \text{ mm} \\
(EI)_{ef} &= 1,76 \cdot 10^{12} \text{ Nmm}^2
\end{aligned}$$

$$u_{2,fin} = 7,8 \text{ mm}$$

$$u_{1,fin} + u_{2,fin} = 9,5 \text{ mm} = l / 420 < l / 200$$

Although in the final stage the effective moduli of elasticity decrease considerably, the stiffness of the composite cross-section decreases to a much lesser extent.

Concluding summary

Timber-concrete composite load-bearing structures are very useful for the production of stiff (in- and out-of-plane) and resistant floors (and walls). Following few design rules it is possible to realise suitable structural elements which are easy to calculate and with a suitable long term performance.

References

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