doi: 10.18720/MCE.73.8

Design methods of timber-concrete composite ceiling structure

Методы проектирования деревожелезобетонной композитной потолочной конструкции

M. Al Ali,
V. Bajzecerova,
V. Kvocak,
Technical University in Košice, Košice, Slovak Republic

Канд. техн. наук М. Ал Али,
канд. техн. наук В. Байзечерова
д-р техн. наук В. Квочак
Технический университет г. Кошице, Кошице, Словакия

Key words: timber-concrete; composite structures, Design methods, γ-method

Ключевые слова: деревожелезобетонные элементы; композитные конструкции; методы проектирования; γ-метод

Abstract. Timber-concrete composite structural members are increasingly used in the case of restoration of wooden ceilings. In the other hand, their use also increases in the case of new buildings. Design methods of the composite structures have been evolving since their first use. This expressive evolution of design methods is related to extensive research in this area in last three decades. This paper presents basic information about realisation, experimental and numerical analysis of timber-concrete composite ceiling with nail connections. The design of the mentioned ceiling was realised according to the relevant standards and recommendations in that time. The paper also presents a comparison between the design results of this composite ceiling and deflections measured during experimental short term loading process with the currently widely used analytical calculation model, so called γ-method. This method takes into account the joint compliance of the used nails. In addition, creep behaviour of used materials and concrete shrinkage were implemented in this calculation model. Comparison of the numerical and experimental results shows, that the current method better reflects the real stiffness of the ceiling structure. On the base of mentioned calculation model, the final deformation of the ceiling was also predicted.

Аннотация. При восстановлении деревянных потолочный конструкций все чаще используются древожелезобетонные композитные элементы. С другой стороны, их использование также увеличивается при строительстве новых зданий. Методы проектирования композитных конструкций эволюционировали с момента их первого использования. Эта значительная эволюция методов проектирования связана с обширными исследованиями в этой области за последние три десятилетия. В данной статье представлена основная информация о проведении, экспериментальном и численном анализе деревянного композитного потолка с гвоздевым соединением. Конструкция данного потолка была выполнена в соответствии со стандартами и рекомендациями, действующими на момент проектирования. Также представлено сравнение проектных результатов данного потолка и прогибов, измеренных во время экспериментального краткосрочного процесса загрузки, с широко используемой в настоящее время аналитической моделью расчета, так называемым γ-методом. Этот метод учитывает соответствие используемых гвоздей. Кроме того, в этой расчетной модели были учтены ползучесть используемых материалов и усадка бетона. Сравнение численных и экспериментальных результатов показывает, что текущий метод лучше отражает реальную жесткость потолочной конструкции. На основе указанной расчетной модели также была предсказана окончательная деформация потолка.

Introduction

Composite timber-concrete structures are created by joining several materials with different mechanical and physical properties. First attempts to join timber and concrete were made in the 20ies and 30ies of the last century. The first patent in this field was registered in Germany by Otto Schaub in 1939 [1]. In 1960, bearing structure of timber ceiling was strengthened as a part of the reconstruction of a historic building in Bratislava – Slovakia by joining timber beams with concrete slab using nails [2]. As a result, in 1966, Jozef Poštulka was granted a patent. Later in the 80ies, increased interest in the composite timber-concrete structures has been recorded. The development progressed especially in the

Ал Али М., Байзечерова В., Квочак В. Методы проектирования деревожелезобетонной композитной потолочной конструкции // Инженерно-строительный журнал. 2017. № 5(73). С. 88–95.

field of shear connectors, which resulted in a wide range of joining systems. In the next period, this structural system began to be used not only for renovation of old buildings, but increasingly also in new buildings because of the favourable structural and physical properties.

In the case of the timber part of the ceiling, beams with rectangular cross-sections, eventually solid wood planks are the most commonly used. The use of logs is not excluded [3], especially when used for temporary bridges and bridges on forest roads. It is, of course, possible to couple flat wood-based materials, such as vertical laminated nailed boards [4], glued laminated timber [5], cross laminated timber (CLT) [6, 7], eventually using of more resistant materials, such as laminated veneer lumber (LVL) [8].

The choice of concrete type depends on the size of the load. In usual ceilings and bridge constructions of smaller spans, it is possible to design concrete slabs not only from conventional concrete mixtures (from C12/25 to C50/60), but also from lightweight concrete (LC8/9 up to LC45/50), [6, 9]. For higher load sizes and larger spans, different types of high-grade concrete can be used, [10]. Usually, if the entire concrete layer is in the compressed area, strengthening of the concrete by means of main bearing reinforcement is not necessary. In order to reduce the shrinkage cracks, it is possible to use a structural reinforcement in two directions, eventually, fibre reinforced concrete can be also used [4 and 11]. Shrinkage of the concrete significantly affects the strain of composite timber-concrete elements [12], therefore, the ingredients that reduce the concrete shrinkage can be considered. Another possibility of reducing the undesirable influence of concrete shrinkage can be prefabrication of the concrete slab and subsequent coupling with the timber part using suitable connection means [13] or by gluing [10, 14].

The insuring of composite action between the concrete slab and timber elements can be provided by mechanical or glued connection. In the case of less exposed structures of smaller spans, it is economically advantageous to use dowel-type mechanical shear connectors, such as nails [2, 15]. However, the higher stiffness of coupling can be achieved using a pair of screws in the arrangement at an angle of 45° [16, 17]. The choice of shear connectors, their dimensions, their number and spacing depend on the shear force size in the chink between the wood and the concrete. In the case of joining flat wood-based materials, such as vertically laminated boards, the use of groove joints is appropriate [4, 18, 19]. In this case, the transmission of the shear force in the chink is mainly given by the shear strength of the concrete. The highest stiffness of the coupling can be achieved by gluing [5, 14, 20]. Epoxy adhesives allow bonding not only hardened concrete slab with the timber elements, but also to bond fresh concrete with an epoxy bonding agent. Besides the above mentioned, there are a large number of different shear connectors, such as perforated belts, steel strips with pressed mandrels, grooves in combination with pin joints, various heavy coupling systems for heavy loads for joining the prefabricated concrete slabs [13, 21, 22].

Number of methods were developed for calculation of composite structures. Historically, first used method was the method of idealized cross-section, based on modification factor *n* [2]. In this method, the composite action is considered as ideal rigid. Over the time, many researches and realized constructions proved, that the composite action using conventional dowel-type mechanical shear connectors can not be considered as ideal rigid, particularly in terms of element deflection. In the 60ies, Möhler [23] derived a calculation model of mechanically jointed timber beams with flexible connection. This method has proven to be appropriate and sufficient for the calculation of composite timber-concrete elements [17]. In the next period, the behaviour of these composites under long-term loading has been investigated and several rheological models have been proposed [16, 24, 25]. In addition to analytical and finite element methods, probabilistic methods based on probabilistic deterioration models are also being developed in this area [26].

From the above presented literature review is evident, that the possibilities and methods for design and calculation of these types of composite structures are very extensive. Therefore, within the presented research, the most common and most affordable type of timber-concrete composite ceiling was selected. The main idea of the realised research was to verify and confront the experimentally measured values with the results, obtained using various design methods.

In this paper, numerical and experimental analysis and results of realised timber-concrete composite ceiling with nail connections are presented. The design of the mentioned ceiling was realised according to the relevant standards and recommendations in that time. The paper also presents the comparison between the design results of this composite ceiling and deflections measured during experimental short-term loading process with the currently widely used analytical calculation model, so called γ-method.

Realisation, Methods and Calculation Models

Realisation of the composite ceiling structure

Timber-concrete composite slab was realised within a construction having rectangular shape of dimensions 30x12 m. Two external and one inner walls with spans 2x6 m created the vertical bearing system of the structure. Two-span continuous steel beams, spaced by 3.0 m, were set on the bearing walls. These beams created the main bearing system of the ceiling. At the same time, the steel beams provided a system for transfer of horizontal forces from the roof truss structure. The cross-section of timber beams had a dimensions 120x140 mm. These beams, axially spaced by 805 mm (Figs. 1, 2a), were lying onto the lower flange of the steel beams. The timber beams were covered by 25 mm thick boards, placed perpendicularly to the beams and fixed to them by nails. In the calculation, timber of strength 10 MPa and Young's modulus 10 000 MPa was considered.

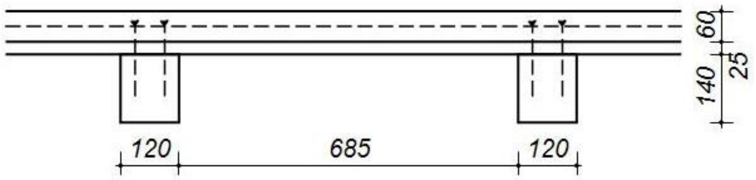

Figure 1. Layout scheme of the ceiling elements

Before hammering nails of dimensions 180/6.3 mm, holes were bored into the timber beams by means of prepared steel pattern plate to prevent the wood cleavage. Borer of diameter 5.5 mm was used to bore holes 80 mm deep, so that 1/3 of the nail length was hammered into the non-bored wood and the nail would protrude from the board by 40 mm.

After hammering the nails, welded net reinforcement was placed and the concrete slab of 60 mm thickness was poured (Fig. 2b). After concrete hardening the reinforced concrete slab started to act as a composite timber-concrete ceiling structure. In the calculation, concrete of strength 11.5 MPa and Young's modulus 27 000 MPa was considered.

a)  **b)** 

Figure 2. a) Layout of steel and timber elements, b) View during realization

Short-term loading test

Deflections of the timber beams were measured in the centre of their span and on the steel beams, in the placing spot of the timber beams. The measurement was carried out using mechanical indicators, as shown in Figure 3a.

Concrete roof tiles were used to create the load for the short-term loading test. The weight of one roof tile is 4.5 kg. The roof tiles were laid in nine rows in the direction of timber beams and in five rows in direction perpendicular to timber beams. By this way, an uniform load was created on an area of 3x2.4 m. The position of the load is illustrated in Figure 3b.

Before starting of the loading test, initial values on the deflection indicators were recorded and considered in the evaluation of final results. In the first stage, the loading test started with five layers of roof tiles uniformly spread onto defined area (3x2.4 m), which represents the load of 1.406 kN/m^2 and 1.132 kN/m uniform load on the monitored beam. The load was gradually increased by adding roof tiles layers as shown in Fig. 3b. In the final loading stage, the load reached a value of 6.48 kN/m^2, which represents 5.216 kN/m uniform load on the monitored beam. The measured deflections are presented in Figure 5.

a)  b) 

Figure 3. a) Set-up of the mechanical indicators, b) Process of loading

Calculation models and methods

Mentioned composite ceiling structure was designed according to relevant, in that time valid recommendation [2], i.e. method of idealized cross-section, based on modification factor *n*. For the verification of obtained results, 3D finite element model was created with consideration of the core and bases of this method. After the implementation of γ-method into new valid standards [27], calculation of the ceiling was carried out using this method to compare the old and new results.

Idealized cross-section method

In this method, the composite cross-section is replaced by timber cross-section with moment of inertia I_i, equated to moment of inertia of the original cross-section, taking into account the rigid connection between timber and concrete. Moment of inertia of the idealized cross-section is calculated according to following equation:

$$I_i = I_t + A_t a_t^2 + n\left(I_c + A_c a_c^2\right), \tag{1}$$

where I_i – moment of inertia of idealized cross-section, I_c – moment of inertia of concrete part, I_t – moment of inertia of timber part, A_c – cross-sectional area of concrete part, A_t – cross-sectional area of timber part, a_t – the distance between the centre of gravity of timber part and idealized cross section, a_c – the distance between the centre of gravity of concrete part and idealized cross section, *n* – modification factor defined as:

$$n = E_c / E_t , \tag{2}$$

where E_c – Young's modulus of elasticity of concrete in bending, E_t – Young's modulus of elasticity of timber.

Equations for calculation of normal and shear stress distribution can be found in [2, 4]. For the design of composite timber-concrete elements, serviceability limit state is often the determining. Deflection, caused by the considered short term loading δ can be calculated as:

$$\delta = \frac{5}{384}\frac{gL^4}{E_t I_i}. \tag{3}$$

Results of mentioned realized structure, calculated according to this method are presented in Table 1. The relationship between the load and the deflection is determined as 0.284*q*, where *q* is the loading of the ceiling in kN/m^2, see Fig. 5. The stiffness of the timber formwork was disregarded in the calculations.

Finite element modelling (FEM)

The model consisted of simply supported timber beams, timber formwork, and concrete slab in accordance with the parameters in Chapter 2.1. Suitable software (FEAT), allowing the creation of surface connection between timber formwork and beams was used. This software also allows the creation of rigid far-connections between the concrete slab and the timber beams. No connections were generated between the timber formwork and the concrete slab. Fig. 4 illustrates a part of the calculation model. Deflections in the mid-span of the beams and stresses in the cross-section, obtained from the FE calculation model, are presented in Figure 5 and Table 1. Results of presented model proved the favourable effect of the timber formwork onto the general stiffness of the composite ceiling.

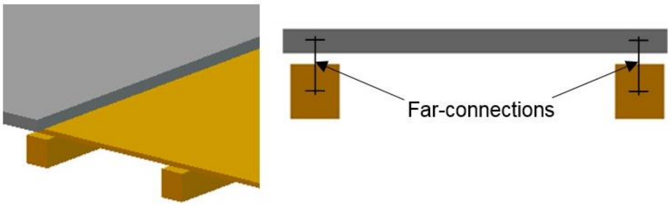

Figure 4. Part of the 3D FE model of the composite ceiling

γ-method

Currently, widely used analytical calculation model, so called γ-method, is based on the linear elastic solution of the simply supported timber-concrete composite beam. This method considers interlayer slip of joined layers, caused by the flexibility of shear connectors. The effective bending stiffness (EI_{ef}) of the simply supported composite beam according to this method, included in [27], can be calculated as:

$$(EI)_{ef} = E_c I_c + E_t I_t + \gamma E_c A_c a_c^2 + E_t A_t a_t^2, \tag{4}$$

where γ-factor is defined as:

$$\gamma = \left[1 + \pi^2 E_c A_c s / \left(K\ L^2\right)\right]^{-1}, \tag{5}$$

where s is the spacing between the connectors, L is the span of the beam, K is the slip modulus of the connectors.

The slip modulus K can be obtained experimentally, but some analytical equations for determination of modulus K are included in [27]. In our case, calculated value of the slip modulus for the ultimate limit state K_u was 6287 N/mm and for the serviceability limit state K_{ser} was 9431 N/mm.

Equations for determination of normal stress distribution are given by [16]. Deflection, caused by the considered short-term loading δ can be calculated as:

$$\delta = \frac{5}{384} \frac{gL^4}{(EI)_{ef}} . \tag{3}$$

Results, obtained using this method are presented in Table 1. The relationship between the load and the deflection is determined as 0.654q, where q is the loading of the ceiling in kN/m^2, see Figure 5. The stiffness of the timber formwork was disregarded in the calculation. The effective width of the concrete slab was considered in the calculation according to [28]. The part of concrete cross-section, where a tensile stress arose, was not included when determining the stiffness of the composite cross-section.

Results

The results of measured and calculated deflections in the mid-span of above mentioned realized ceiling structure are illustrated in Figure 5.

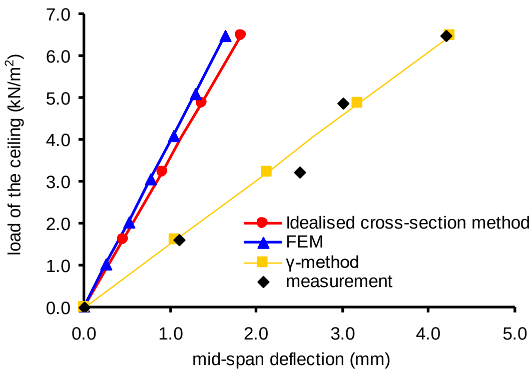

Figure 5. Comparison of measured and calculated values of the deflections

Calculated values of the stresses and deflections in the mid-span according to above mentioned methods and models are listed in Table 1. These results were determined with considering the design loading value of 5.2 kN/m^2.

Table 1. Comparison of the results according to various calculation methods

	$\sigma_{c,top}$ (MPa)	$\sigma_{t,\ bottom}$ (MPa)	F_s (kN)	δ (mm)
Idealised cross-section method	1.88	2.85	2.6	1.83
Finite element method	1.42	2.34	2.73	1.033
γ-method	3.32	4.48	3.17	3.40

$\sigma_{c,top}$ is the normal stress in the top fibres of the concrete part in the mid-span, $\sigma_{t,bottom}$ is the normal stress in the bottom fibres of the timber part in the mid-span, F_s is the maximal shear force in the connectors.

Discussion

As mentioned, this paper deals with numerical analysis using various calculation methods, 3D simulation and experimental measurement of already realized timber-concrete composite structure. Obtained results from these methods [2, 15–17] are compared, confronted with the experimental measurements and discussed in this chapter.

As shown in Figure 5, the deflections were calculated according to three methods, described in Chapter 2.2. From the comparison of the illustrated results is evident, that the measured values are almost identical to the values obtained from the calculation, when using γ-method. Results of this investigation prove, that γ-method is enough accurate and sufficiently appropriate to be used for the calculation of composite timber-concrete structures during short term loading.

Despite the fact that during the short term loading test higher load values were applied, mentioned realised structure was designed in accordance with the regulations in force at that time. Design loading value of 5.2 kN/m^2 was considered for the calculation of the stresses and deflection of the composite structure. This design value of loading is determined by the normative properties of used material to be on the safe side.

As shown in Table 1, values of normal stresses, shear forces and deflections, calculated according to γ-method are higher than the values calculated according to other methods, which do not consider the flexibility of shear connectors. However, presented values of normal stresses in the all cases are far from the limit values. The reached shear forces are close to the limit values. The deflections of the structure from the considered short-term loading meet the acceptable values.

Conclusions

The paper presents results of realized composite timber-concrete structures, obtained by application of experimental test and different calculation methods, described in detail in Chapter 3. The obtained results showed the appropriate sufficiency of γ-method for calculation of these types of composite structures, especially for short term loading. Presented results in the paper proved that the composite action using the dowel-type of shear connectors can not be considered as ideal rigid. In spite of relatively high flexibility, smooth nails are very used due to their availability.

Although the design of realized structure was carried out using methods that do not completely describe the real behaviour of the solved composite structure, the design was conducted with a sufficient reserve, so it is on the safe side. The design author of the mentioned structure is in a permanent contact with the owner of the building. More than 15 years after the realization, any problems with construction were not recorded.

Acknowledgement

This paper is prepared within the research projects VEGA No. 1/0188/16 and No. 1/0538/16, supported by the Scientific Grant Agency of the Ministry of Education of Slovak Republic and by the Slovak Academy of Sciences.

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Mohamad Al Ali,
+421905359228; mohamad.alali@tuke.sk

Viktoria Bajzecerova,
+421556024289; viktoria.bajzecerova@tuke.sk

Vincent Kvocak,
+421556024112; vincent.kvocak@tuke.sk

Мохамад Ал Али,
+421905359228;
эл. почта: mohamad.alali@tuke.sk

Виктория Вайзечерова
+421556024289;
эл. почта: viktoria.bajzecerova@tuke.sk

Винсент Квочак
+421556024112;
эл. почта: vincent.kvocak@tuke.sk

Al Ali M., Bajzecerova V., Kvocak V. Design methods of timber-concrete composite ceiling structure. *Magazine of Civil Engineering*. 2017. No. 5. Pp. 88–95. doi: 10.18720/MCE.73.8.