Outlook on the future design of timber-concrete-composite structures in the Eurocode

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1. Introduction

In recent years the advantages of timber-concrete-composite structures as increased stiffness and load capacity or improved sound insulation have been a motivation for a wider use of these systems in buildings (see [1]). In order to support this higher demand, clear rules and a common understanding, how to design timber-concrete-composite structure is essential. For this reason the existing knowledge was collected and summarized (see [2]). Within the work of CEN/TC250/SC/PT5.2 this knowledge was transformed into a Technical Specification, which, eventually will be the basis for a new part of Eurocode 5 [3] in the next generation of the Eurocodes.

In this paper the main contents of this Technical Specification as well as the main differences to the common design, according to the available guidelines and technical approvals are presented.

2. Range of application

Timber-concrete-composite structures show a large variability regarding the materials, the cross-sections, the connections that are used or even the design approaches that are followed. A main reason is that the development of the structure but also of the design procedures takes place independently in the different countries. However not all these have been based on sufficient research studies and therefore an appropriate scientific background it is not available. For these reasons not all the conditions, found in practice, are covered and many limitations apply has described below:

- Timber: The Technical Specification is linked to [3], consequently all wood based materials, which are allowed by [3], can be used. However limitations apply in specific conditions such as for example in the models provided to determine the mechanical properties of notch connections where a minimum strength class of C24 and GL24 for solid timber and glued laminated timber respectively is required.
- Concrete: The concrete is limited to following strength classes
  - Normal concrete: ≥ C12/15; ≤ C60/80
  - Light weight aggregate concrete: ≥ LC12/15; ≤ LC60/80
- For the notched connections design models mentioned before, the lower limit of the concrete strength class is set to C20/25 since no sufficient information for lower concrete classes is available.
- Reinforcement: For the reinforcement reference to [4] is made, so all types of reinforcement defined in [4] can be used within TCC-structures designed according to the technical specification-
- Service class: The use of timber is only encouraged in Service class 1 or 2. Despite of this the Technical Specification does not limit the application of TCC in Service class 3, except in the area of the joint between timber and concrete. This joint has to be detailed in such a way that the service conditions correspond to either to Service class 1 or 2, the intention is to prevent water penetration into the joint which is difficult to remove, leading to a reduction of both the durability and performance.
- Dimensions: The thickness of the concrete slab should be between 50mm and 300mm, while the intermediate layers should not exceed the thickness of 50mm.

3. Short term behaviour

3.1 Loads/actions

During the life time of a composite timber-concrete structure two types of loads will act:

- "Common" external loads as dead load, live load, snow loads, wind loads, etc.
- Eigenstresses caused by inelastic strains such as different temperature elongation, shrinkage and swelling of the timber and shrinkage of the concrete

In many situations the temperature variations effects and the swelling/shrinkage of timber are not relevant. The Technical Specification allows a simplification in such conditions typified as quasi constant climatic conditions. In practice these conditions are defined by:

- The initial moisture content is in the range of the final moisture content as already required in [3]
- The annual difference between the values of the maximum and minimum moisture content is lower or equal to 6%
- The maximum difference of the air temperature is lower or equal to 20°C.

This means that typical Service class 1 conditions, as well as, some conditions of service class 2 can be assigned to the quasi constant conditions. In these situations shrinkage/swelling of timber and the effect of temperature variations may be neglected. Despite the shrinkage of concrete has to be considered in the ultimate as well as in the serviceability limit state.

When the three conditions listed above are not met the surrounding conditions are assigned to variable conditions. In these cases the effects of the moisture variation and of the temperature variation have to be considered in the ULS, as well as, in the SLS. These additional inelastic strains are given by

- Temperature: The annual temperature variations as well as the variation between initial state and final state are defined in [5] and are already one common action in bridge design. In this case the temperature variations may be assumed constant over the cross section
- Shrinkage of concrete: The shrinkage of concrete may be obtained in Eurocode 2 [4].
- Shrinkage and swelling of timber: Similar to the temperature two different moisture variations have to be considered
  - Difference between the initial and the final moisture content
  - Annual moisture variations: The procedure of the determination of the annuals moisture content of a sheltered system with direct access of the air is given in annex A of the Technical Specification as described below.

The determination of the annual moisture variations can be done according to the following procedure, if more precise values are unknown:

- Identification of the climate zone according to the Köppen-Geiger-diagram (see Figure 1)

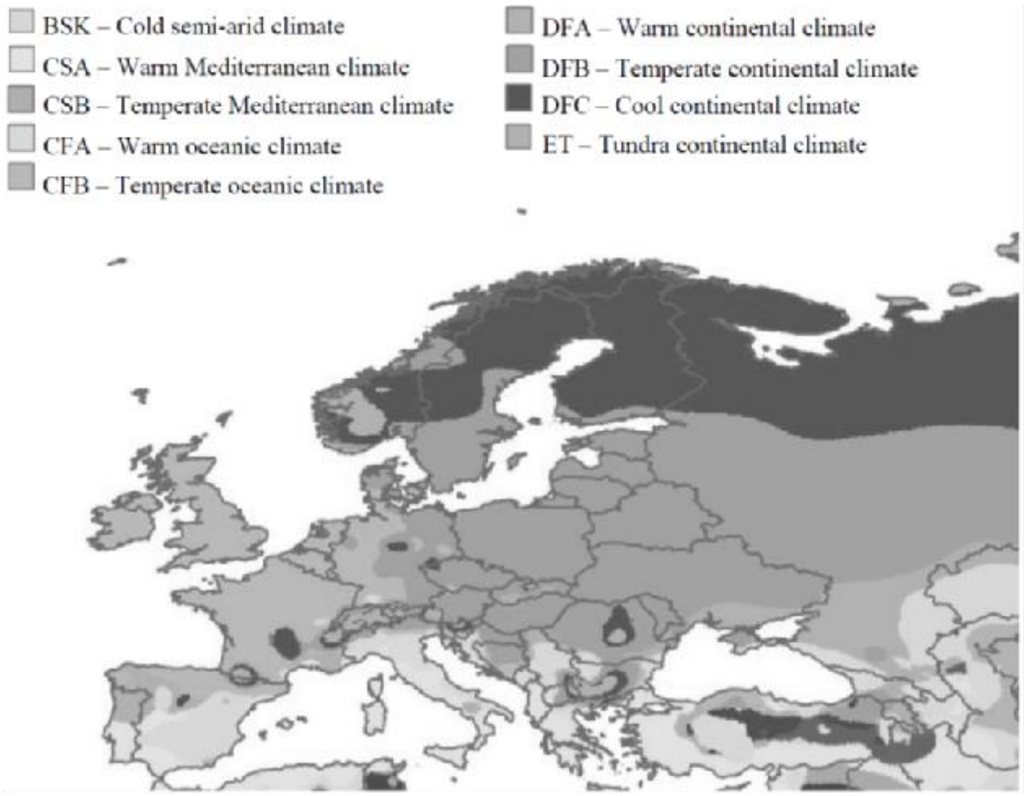

Figure 1: Köppen-Geiger-diagram (see [6])

– Determination of the moisture content by interpolating the values given in [6] (see Tab 1)

Table 1: Variation of the moisture content in % averaged over the cross section Δmc = max mc - min mc (extract from [6])

	Minimum of the width, or twice the depth of the timber cross-section (mm)		
Bez.	38	125	>300
Temperate oceanic Stuttgart, Zurich, Paris, London			
CFB	15%	9%	3%
Temperate continental Southern region Warsaw, Berlin, Munich, Prague			
DFB.2	15.5%	9%	3.5%

The values are under discussion

– Determination of the inelastic strain by

$$\varepsilon = \alpha_{||} \cdot \Delta u \quad (1)$$

ε effective strain caused by moisture variation

Δu annual moisture variation determined according to Table 1

$\alpha_{||}$ Coefficient of swelling/shrinkage = 0,01%/%Δu according to [7]

It is important to mention that in the current technical approvals, available for various connection devices, the consideration of the variation of the moisture content is already required. The main difference between the Technical Specification and these technical approvals is in the way the moisture variation may be taken into account: i) by the global modification of the Modulus of Elasticity in technical approvals, ii) by an explicit load in the Technical Specification.

3.2 Modification factors

As for timber structures the influence of the moisture content and the time on the strength is taken into account with through modification factors of the strength the k_{mod} given in Eurocode 5 [3].

Concerning the connection, the concept of the connection between two timber based products with different k_{mod}-values is applied on TCC-structures, assuming that the k_{mod}-value of the concrete is the modification of the strength of concrete given in Eurocode 2 α_{cc}.

$$k'_{mod} = \sqrt{k_{mod} \cdot \alpha_{CC}} \qquad (2)$$

3.3 Determination of the forces

3.3.1 Methods

The performance of the connection, especially the stiffness, influences the distribution of the forces within the cross section. Therefore the deformability of the connection has to be considered in the evaluation of the forces. Several analysis methods are available:

- Differential equation (see among others [8] and [9])
- γ-method according to [3], Annex B with the extension according to the Technical Specification, Annex B in order to consider the inelastic strains (see [10,11,6,12,13])
- Shear analogy method according to [14] (see among others [15] and [16])
- Modelling as strut&tie model (see among others [17])
- FE-modelling

The decision on the method to be used, depends on the system and the target of the modelling of the system. Often FE is used within the scientific framework whereas the γ-method or the strut & tie model are often used to model the two layered timber-concrete-composite systems. The technical Specification does not introduce any limitation on this issue.

3.3.2 Behaviour of the material in the short term

For the evaluation of the internal forces the mean values of the Modulus of Elasticity of the timber cross section as well as the concrete cross section in compression may be assumed. Cracking of concrete has to be considered in the analysis. This can be done by the introduction of a non-load bearing layer, representing the cracked area. The height of this layer is determined iteratively until no tensile stresses exceed the design strength in tension of the concrete (normally =0 N/mm²). The elastic behaviour in compression is also assumed when proofing the system.
The timber cross section is verified according to [3] by considering the interaction between the normal force and the bending moment in the timber cross section, as regular verification in timber members.

3.3.3 Compatibility

The verification of the TCC-cross section is done by the verification of the concrete cross section and the timber cross section separately. Additionally, the Technical Specification requires the check of the compatibility of the cross section.

For the verification of the concrete cross section, it is assumed, that the reinforcement yields and the strain caused by the stresses reaches -3.5‰. Therefore the strain at the reinforcement is greater or equal 2 ‰. This value is in the range of the maximum strain of the timber cross section, when reaching the strength of the timber (see Figure 2).

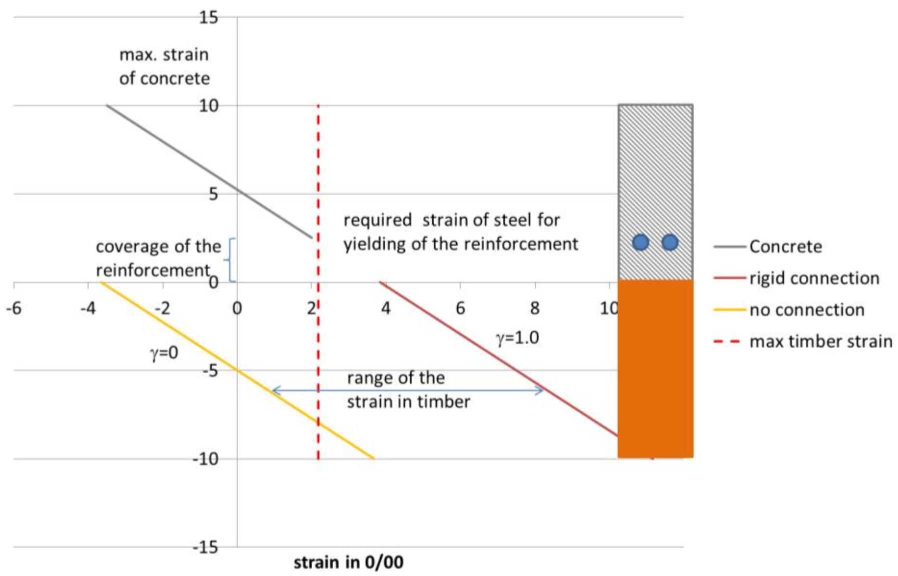

Figure 2: Strain distribution in the cross section

Since the timber cross section is installed underneath the concrete cross section in the tension zone, the strain in the timber cross section are larger than that in the reinforcement. Therefore, when considering the reinforcement as load bearing structure it has to be checked, which strain can be reached. In most of the cases, the reinforcement remains in an elastic strains due to this compatibility of the strains and the curvature of the concrete and the timber cross section.

3.3.4 Effective width

Often timber-concrete-composite structures are realised by connecting timber beams with a concrete slab. Due to the shear deformation of the concrete slab, the concrete slab cannot be activated fully over the complete width. In order to cover this influence an effective width is introduced. This effective width can be divided into the share caused by the normal force (= shell) and caused by bending. Since the reinforcement cannot be activated, only the share of the load distribution caused by normal force is considered. Comparing the different proposals available in existing standards, namely Eurocode 4 [2] and Eurocode 2 [4]. The proposal given in Eurocode 4 [2] shows a better correspondence to the shear lag, so the Technical Specification proposes to use the effective width according to it.

4. Connection devices

4.1 General

The connection mechanical properties can be obtained from: laboratory tests, models given in the Technical Specification or from the connections technical approvals.
In order to avoid a separation between concrete and timber, a minimum uplift force of the concrete of 10% of the shear force, shall be considered in the design of the connection.

4.2 Dowel type fasteners installed 90° to the joint between timber and concrete

4.2.1 Load capacity

Since the theory according to [18] is derived by solving the equilibrium of forces at the joint, this theory can also be applied to connections in timber-concrete-composite structures. The input values on the timber side are the one given in [3]. The embedment strength in the concrete is assumed to be strength under local compression. Therefore the embedment strength is proposed to be 3x of its compressive strength. With this embedment strength the load capacity of the connection can be determined according to the equations given in [3].

4.2.2 Stiffness

In the model it is assumed that the deformation in the concrete is negligible when compared to the deformation in the timber cross section. A similar assumption is made for steel to timber-connection, for this reason the stiffness of a TCC connection with dowel type fasteners installed in 90° is the same of a steel-to-timber connection as given in [3].

4.3 Notched connections

4.3.1 Range of application

Since the existing studies on the load and deformation behaviour of notched connection do not cover the whole possible range of parameters, the application of the equations in Technical Specification are limited to following parameters (see Figure 3):

- Concrete strength greater or equal C20/25 with a maximum diameter of the aggregate size of 16mm
- Timber strength class greater or equal C24 and GL24 resp. or LVL according to EN 14374
- Geometry of the notch
  - Depth of the notch $h_N \geq 20mm$ for normal loads (e.g. in buildings) and $h_N \geq 30mm$ for normal loads (e.g. bridges, warehouses)
  - Length of the timber in front of the notch $l_v \geq 12.5 \cdot t_v$ for robustness reasons, however only $l_v \leq 8\, t_v$, may be considered in the design as load bearing length
  - Length of the notch in the timber $l_N \geq 150mm$ and min. $12.5\, t_v$
  - Diameter of the screw Ø ≥ 6mm in order to prevent uplift between timber and concrete
  - Inclination of the load transferring contact area

$$80° \leq \alpha \leq \min\begin{cases} 115° \\ 90° + \Theta \end{cases} \tag{3}$$

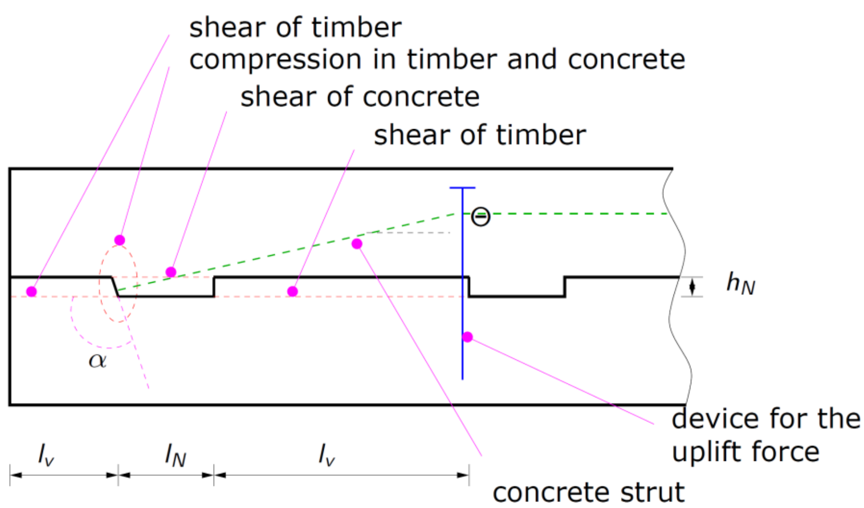

Figure 3: Notched connection

4.3.2 Stiffness

The stiffness of the notched connection is mainly determined by experimental and numerical models, since the load distribution in the anisothropic timber can hardly be described analytically (see [19] and [20]). The model proposed is based on the experimental data available for this connection type (equation 4).

$$K_{ser} = \begin{cases} 1000 \dfrac{kN}{mm \cdot m_{Breite}} & \text{für } t = 20mm \\ 1500 \dfrac{kN}{mm \cdot m_{Breite}} & \text{für } t \geq 30mm \end{cases} \tag{4}$$

The notched connection behaves more or less linear elastic until the failure. Therefore the stiffness in the ULS is the same as in the SLS and does not need to be reduced by a factor of 2/3 as for other TCC connectors.

4.3.3 Load capacity

The load capacity can be determined by evaluating the single failure modes (see [19], [21] and Figure 3). These failure modes can be determined by following equations:

$$F_{R,d} = \begin{cases} f_{v,c,d} \cdot b_N \cdot l_N & \text{shear of concrete} \\ f_{c,d} \cdot b_N \cdot h_N & \text{crushing of concrete} \\ f_{v,h,d} \cdot k_{cr} \cdot b_N \cdot \min(l_V; l_S) & \text{shear of timber} \\ f_{c,0,d} \cdot b_N \cdot h_N & \text{crushing of timber} \end{cases} \quad (5)$$

$F_{R,d}$	Design value of the load capacity
$F_{v,Ed}$	Design shear force between the timber and the concrete
$f_{v,c,d}$	„shear strength" of the concrete $= \frac{\nu \cdot f_{c,d}}{cot\Theta + tan\Theta}$
ν	Reduction factor taking into account the effects of crack on the shear transfer $= 0{,}6 \cdot \left(1 - \frac{f_{ck}}{250\,[N/mm^2]}\right)$
k_{cr}	Crack-Factor according to [3]
b_N	Width of the notch
l_N	Length of the concrete notch
$f_{c,d}$	Design value of the compressive strength of the concrete
$f_{c,0,d}$	Design value of the compressive strength of the timber member parallel to the grain
h_N	Depth of the notch
$f_{v,d}$	Design value of the shear strength of the timber
l_V	Length of the timber in front of the notch
l_S	Length of the timber between the concrete notches
Θ	Inclination of the compressive strut

In addition to these failure modes an uplifting force should be considered in the evaluation. It depends on the assumed inclination Θ of the concrete strut.

$$F_{\perp,d} = \max \begin{cases} 0{,}1 F_{E,d} \\ F_{R,d} \cdot tan\Theta \end{cases} \quad (6)$$

In the Technical Specification this inclination is defined as the inclination of the compressive strut between the contact area of timber and concrete and the anchorage of the device transferring this uplift force. The lower boundary of the inclination is limited by the following equations

$$\Theta \geq \max \begin{pmatrix} \arctan\left(\frac{h_N}{2 \cdot (l_N + l_S)}\right) & \textit{intaraction between the notches} \\ arctan\left(\frac{h_N}{l_N}\right) & \textit{required width of the concrete strut} \end{pmatrix} \quad (7)$$

The first boundary ensures that the interaction between the notches can be neglected, since the elements transferring the uplift force are installed in between the notches. The second boundary ensures that the thickness of the concrete strut is not influenced by the non-load transferring edge of the notch. So the position of the device transferring the uplift force can be chosen "freely" within these boundaries. With increasing distance of this device from the edge of the notch, the forces in the device can be reduced. However the load capacity of the notch is also reduced. So it is up to the designer to develop the appropriate solution to balance between the decreasing load capacity of the concrete and the decreasing uplift force with increasing distance.

5. Long term behaviour

5.1 Critical point in time or "stiffness attracts forces"

In structures, where the long term behaviour influences the load and deformation behaviour often the points in time $t = 0$ and $t = \infty$ (~ t=50 years) are verified. The point in time t=0 is identified as that point in time where the design load (or relevant parts of this load) is applied on the structure.

If the courses of the development of the creep strains of timber and concrete are related to their final values and then compared to each other, it can be shown that the concrete creeps stronger within the first 3 to 7 years compared to the timber. This results to a time dependent change in the ratio between the stiffness of the concrete cross section and the stiffness of the timber cross section. Since stiffness attracts forces, the stresses mainly caused by bending increase in the timber cross section, whereas the stresses in the concrete are reduced. After that period concrete hardly creeps, so the timber cross section can transfer some stresses back to the concrete cross section when creeping. Therefore not only the points in time *t=0* and $t = \infty$ but also the period of *t=3 to 7 years* have to be considered in the design process. This additional point in time leads to an extra effort in the design process. On the other hand often the SLS, namely the limitation of the long term deflection, governs the cross section dimension. Within the development of the Technical Specification studies were performed discussing the increase of the stresses at *t=3-7 years* related to the stresses at $t = 0$ and $t = \infty$. As result this additional point in time does not need to be verified, if the stresses in the timber cross section, obtained for the quasi-permanent combination are increased 25% in the verification. In this way the designer can decide whether this additional point in time has to be verified or may be neglected.

5.2 Creep of timber and concrete

Generally spoken the long term behaviour is often classified in the case "pure creep deformation" and "pure relaxation". In the case of "pure creep strain" the stresses are constant over time, where as in the cases with relaxation the total strain is constant over time. However, composite structures are somewhere in between. The long term behaviour cannot be classified as pure creep phenomenon since the creep deformation of the single components lead to stress redistributions within the composite cross section, resulting in non-constant stresses over the time. On the other hand, all components are deformable so the strain is not constant over time, which is necessary to classify the long term behaviour as a relaxation phenomenon.
This time dependent behaviour is not unique in timber-concrete-composite structures. [21] derived the solution for the effective creep coefficients for concrete structures when an additional layer of concrete is place on top of an existing concrete deck. [22] (see [23]) extended this procedure for timber-concrete-composite structures especially with regard to the deformability of the connection assuming a smeared stiffness along the joint between timber and concrete. In [11] the different temporal development of the creep strain was implemented into the existing procedure. So the effective creep coefficient can be analytically determined, taking into account all relevant parameters as cross section dimension, material properties and different temporal development of the creep strains. However these equations are not feasible for the daily design process. In the Technical Specification the approach indicated in steel-concrete-composite structures is adopted, so the material creep coefficients from the material standards [3] and [4] are modified by a factor ψ (seeTable 2).

Table 2: ψ -factors for the evaluation of the effective creep coefficient for slab systems ($b_t=b_c$; $1/5 < A_{c,eff}/A_t<1$) and beam systems ($b_t << b_c$; $1 < A_{c,eff}/A_t < 5$) in dependence on the composite coefficient γ_1 according to [3] Annex B

Concrete φ = 3,5; k_{def} = 0,6		Concrete φ = 2,5; k_{def} = 0,6		Timber in all cases	
$\psi_c(t$ = 3 - 7$a)$	= 2,5 - $\gamma_1^{1,1}$	$\psi_c(t$ = 3 - 7$a)$	= 1,9 - 0,6 $\gamma_1^{1,1}$	$\psi_t(t$ = 3 - 7$a)$	= 0,5
$\psi_c(t = \infty)$	= 2,6 - 0,8 γ_1^{2}	$\psi_c(t = \infty)$	= 2,0 - 0,5 $\gamma_1^{1,9}$	$\psi_t(t = \infty)$	= 1,0
Concrete φ = 3,5; k_{def} = 0,8		Concrete φ = 2,5; k_{def} = 0,8		Connection in all cases	
$\psi_c(t$ = 3 - 7$a)$	= 2,2 - 0,8 $\gamma_1^{1,2}$	$\psi_c(t$ = 3 - 7$a)$	= 1,7 - 0,5 $\gamma_1^{1,1}$	$\psi_{conn}(t$ = 3 - 7$a)$	= 0,65
$\psi_c(t = \infty)$	= 2,3 - 0,5 $\gamma_1^{2,6}$	$\psi_c(t = \infty)$	= 1,8 - 0,3 $\gamma_1^{2,5}$	$\psi_{conn}(t = \infty)$	= 1,0

For the point in time t=0 all *ψ-values are equal 0*
γ_1 according to [3] Annex B

In order to evaluate the effects of the creep deformation the Modulus of Elasticity and the stiffness of the connection are modified by

$$E_t(t) = \frac{E_0}{1+\psi_t \cdot k_{def}};\; E_c(t) = \frac{E_0(t_{load})}{1+\psi_c \cdot \varphi};\; K = \frac{K_0}{1+\psi_{conn} \cdot k'_{def}} \tag{8}$$

The effective creep coefficient of the connection can be determined by

$$k'_{def} = 2 \cdot k_{def} \tag{9}$$

The increase of the creep coefficient of the connection by the factor 2 is caused by the local high stresses in the range of the anchorage of the connection device.

Since creep and shrinkage interact and the effects of shrinkage are partly reduced by the creep deformation (~ relaxation), the shrinkage value of concrete at *t=3 to 7 years* may be reduced to 60% of its end shrinkage value and for the point in time $t = \infty$ it may be reduced to 90% of its end value.

In propped constructions the shrinkage begins after the end of the post-treatment of the concrete, even if the system is still propped. The reason behind is the fact that the shrinkage leads to eigenstresses which cannot be reduced by creeping during the short period of propping. So the building process does not have an effect on the magnitude of the shrinkage value applied in the structural analysis.

6. Summary and Outlook

The load and deformation behaviour of timber-concrete-composite structures has been widely studied. As one output of these studies technical approvals of connections have been developed. These technical approvals also cover the design procedure, in many cases.

Nevertheless it is planned to introduce clauses about the design of timber-concrete-composite structures within the next generation of the Eurocodes. In preparation for this a Technical Specification has been developed and will be continuously improved, which could be the basis of a new part of [3].

This Technical Specification differs from the typical technical approvals in following issues:

- Loads: In the Technical Specification the loads and actions caused by different elongation of the composite members due to shrinkage, swelling or temperature changes are determined explicitly and are not covered in a global modification factor. So the design can be adjusted to the existing surrounding and boundary conditions. The required partial safety factors of these loads and modifications factors assigned to these loads are given in the Technical Specification
- Design method: One of the most popular design method is the so called γ-method given in [3] Annex B. Unfortunately this method cannot consider inelastic strains, so the Technical Specification Annex B provides an extension in order to cover the effect of different temperature elongation, shrinkage and swelling within this method, by transferring these inelastic strains in a fictitious load.
- Connections: In the Technical Specification design equations and provisions for dowel type fasteners, glued in rods installed perpendicular to the joint and notches are

given. Additionally, the required parameters for the application of the Technical Specification are defined, namely load capacity, stiffness in the ULS and SLS. So technical approvals of other systems can be "docked" to the provisions given in this Technical Specification.

– Long term behaviour: Concerning the long term behaviour it might be necessary to verify an additional point in time, if some limits are exceeded. The effect of stress redistributions on the effective creep coefficients will be covered by the modification of the material creep coefficient.

The Technical Specification is finalized as final draft of the Project team in April 2018. It can be the basis for a new part of [3] dealing with the design of timber-concrete-composite structures. Nevertheless the document is quite dynamic since different opinions and experiences from whole Europe will be considered, which sometimes contradicts each other. For these reasons is expected that there will be some changes in the document from the current status until the consideration in the new generation of the Eurocodes.

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