Structures and Buildings
Volume 164 Issue SB2
Long-term load–deformation behaviour of timber–concrete joints
Van de Kuilen and Dias
ice | proceedings

Proceedings of the Institution of Civil Engineers
Structures and Buildings 164 April 2011 Issue SB2
Pages 143–154 doi: 10.1680/stbu.10.00021
Paper 1000021
Received 03/03/2010 Accepted 11/11/2010
Published online 11/03/2011
Keywords: codes of practice & standards/concrete structures/timber structures

ice
Institution of Civil Engineers
publishing

Long-term load–deformation behaviour of timber–concrete joints

1 **Jan Willem G. Van de Kuilen** PhD
Professor, Delft University of Technology, Delft, the Netherlands; Munich University of Technology, Munich, Germany

2 **Alfredo M. P. G. Dias** PhD
Assistant Professor, University of Coimbra, Coimbra, Portugal


1


2

This paper discusses the long-term mechanical behaviour of timber-to-concrete joints made with dowel-type fasteners. Despite the influence that the long-term behaviour of joints has on the mechanical behaviour of a timber–concrete structure and consequently on its design, there is still a lack of research in this area. This paper presents experimental research, carried out at the University of Coimbra and Delft University of Technology, on seven joint configurations using different types of fasteners and different materials. For each joint configuration, either four or ten tests were performed resulting in a total of forty tests. A comprehensive description of the test specimens and test setup is given. The experimental creep–time curves were fitted to a creep–time model and used to predict joint creep values over longer timeframes (10 and 50 years). The values obtained were compared with values available in the literature for timber-to-concrete joints with other types of fasteners and timber-to-timber joints with dowel-type fasteners. The approach for timber-to-timber joints suggested by Eurocode 5 was used to determine creep values for timber-to-concrete joints. The results obtained were compared with test results to assess the accuracy of predicting creep values of timber-to-concrete joints with dowel-type fasteners. It was concluded that creep values measured in long-term experimental tests are usually higher than those obtained from the model indicated in Eurocode 5, particularly for environmental conditions corresponding the service class 2.

1. Introduction

Timber–concrete composite structures combine the good mechanical performance of timber in tension and concrete in compression. However, if the structure is to perform well, an efficient joint system is required: the fastener must be able to transfer forces between both materials with limited slip between them. Timber-to-concrete joints are therefore a key issue in these systems, as noted previously by several researchers (e.g. Balogh *et al.*, 2008; Van der Linden, 1999). In a structural analysis, both ultimate limit states and serviceability limit states must be verified. Due to the different long-term behaviour of timber, concrete and joints, the stress distribution at long term is different from that at short term, and either short- or long-term behaviour might govern the design. Consequently, both the short- and the long-term behaviour of the system components have a direct influence on the system's structural behaviour and therefore on its design.

The design of these types of structures is usually performed using the model presented in Eurocode 5 (CEN, 2003c). The model is able to predict, with adequate accuracy, stresses and deformations on a structure, despite the assumed simplifications (Jorge, 2005; Van der Linden, 1999). The model analysis is based on an effective bending stiffness EI_{eff} function from the joint stiffness, usually considered through the joint slip modulus K_s determined in accordance with EN 26891 (CEN, 1991). The effective bending stiffness is then used to compute the stresses and deformations in the concrete and in the timber members as well as the loads acting on the joints.

Long-term analysis is usually carried out using the effective modulus method. The basis of the method is also the linear elastic simplified approach proposed in Eurocode 5 (CEN, 2003c). In this case, the effective values of the elastic properties of concrete (modulus of elasticity), timber (modulus of elasticity) and joints (slip modulus) are used instead of the short-term values of the elastic properties (Ceccotti, 1995). However, agreement on the accuracy of this model is not consensual. Van der Linden

(1999) found good agreement between the model and test results for timber–concrete composite beams tested in long-term tests using four different joint configurations. Other recent studies, however, showed that in some situations the methodology underestimates the long-term effects on timber–concrete structures (Ceccotti *et al.*, 2007; Jorge, 2005). In order to address this issue, more complex models and approaches have been proposed to predict the long-term behaviour of timber–concrete structures; examples include the algorithm based on the finite differences method developed by Kuhlman and Schaenzlin (2001) and a numerical procedure that takes the joint into account through a uniaxial finite element proposed by Fragiacomo and Ceccotti (2006).

For any of the aforementioned methodologies, the long-term behaviour of the joints is a key parameter in the analysis, which might be defined by joint deformation–time or creep–time curves. However, since such information is usually unavailable, the long-term behaviour of joints is often taken into account through the relative creep value of the joints, defined in terms of the initial and final deformations (Equation 2).

Despite its importance, few studies have assessed joint creep values. Furthermore, the results that are available were obtained for specific joint configurations such as the Tecnaria fastener (Fragiacomo *et al.*, 2007), notches associated with steel fasteners (Kuhlman and Michelfelder, 2004) and inclined screws (Jorge, 2005). The validity of these results for other types of fastener has not been demonstrated. Values for joint creep can be derived from the creep data available for timber–concrete beams using finite-element models (Van der Linden, 1999), but such an approach is affected by various types of uncertainties and errors that result not only from model inaccuracies but also from the assumptions made in terms of individual material properties (namely the creep values considered for timber and concrete). Eurocode 5 (CEN, 2003c) also gives indications for determining creep values in timber-to-timber joints. This methodology might be applicable to timber-to-concrete joints, but this has not yet been verified experimentally.

The objective of this research was to obtain creep values for timber-to-concrete joints made with dowel-type fasteners. The data are presented here based on experimental results available for periods of 2–3 years and extrapolated for 10 and 50 years using numerical models available in the literature. The results obtained are analysed and compared with published results for other joint types tested in similar conditions. The creep values obtained are also compared with the values obtained using the Eurocode 5 (CEN, 2003c) approach in order to assess its accuracy for estimating the creep behaviour of timber-to-concrete joints. Since the creep values given in Eurocode 5 (CEN, 2003c) are not directly comparable with the values measured in the long-term tests, the paper presents the method used to modify the experimental results in order to make them suitable for comparison with the code.

2. Derivation of creep data and creep factor for timber–concrete joints

As already noted, only a limited number of studies have researched the effect of joints' long-term behaviour on the long-term behaviour of a composite structure. There are several reasons for this, including

(*a*) the high priority given to addressing short-term strength issues
(*b*) the time necessary to obtain results (often longer than project durations)
(*c*) the high costs involved in performing long-term tests
(*d*) the usually high proportion of transitory load in floor applications.

Some data on long-term shear tests are, however, available. Kuhlman and Michelfelder (2004) tested timber–concrete joints made with notches and notches associated with steel fasteners (head and self-tapering screws), testing one specimen for each joint configuration. A load of approximately 30% of average short-term load-carrying capacity of the joint obtained in short-term shear tests was applied over a period of approximately 8 months.

Jorge (2005) tested timber-to-concrete joints made with inclined screws using different lightweight concretes with and without an interlayer. Five joint configurations were tested in a total of eighteen tests. The test duration varied between 140 days for one joint configuration and 606 days for the other four configurations. The tests were performed in controlled climatic conditions (air relative humidity (RH) 65 ± 5%, temperature 20 ± 2°C) with a long-term load approximately equal to 30% of the average short-term load-carrying capacity determined in short-term shear tests.

More recently, 12 long-term tests were performed on a commercial fastener developed specifically for timber–concrete systems (Fragiacomo *et al.*, 2007). Two configurations were tested – one with normal-strength concrete and the other with lightweight concrete. The load applied to the specimens corresponded to 30% of the average load-carrying capacity obtained in short-term shear tests. The tests were performed in constant air RH of 70% for the first 103 days. After this period, climatic cycles were applied for 333 days.

As a consequence of the limited amount of available data, some authors derived creep values for timber-to-concrete joints indirectly from data derived from long-term tests on timber–concrete composite beams. Van der Linden (1999) used this methodology to derive creep values for three timber-to-concrete joint configurations: inclined screws, nailplates and a combination of notches and dowels. The creep values were obtained from the deformations of timber–concrete beams using finite-element modelling and assuming the creep of concrete was defined by a lognormal function whose parameters were obtained from literature. The tests were performed in uncontrolled climatic conditions, leading to timber moisture contents that varied with season from around 12 to 15%, with a mean value of 13%.

Due to the limited amount of tests available on timber-to-concrete joints, analysis of creep in timber-to-timber joints might be useful. From the various studies available on this topic, the research undertaken at Delft University of Technology by Van de Kuilen (1999, 2008) is of particular interest for this work. The environmental test conditions were exactly the same as used in part of the tests described here for dowel-type fasteners (specimens 10mmB and INT (see Table 1)). The experiments of Van de Kuilen (1999) are also relevant due to their timeframe, which was already 13 years in 1999. These long-term tests were performed with three joint configurations (nails, toothed plates and split ring joints).

In Eurocode 5 (CEN, 2003c), the long-term slip of timber joints is calculated using the joint deformation factor $k_{\mathrm{def,calc}}$, calculated based on the deformation factors from the two joined materials. The same approach can be used for timber-to-concrete joints, resulting in Equation 1, where k_{def} is the deformation factor of the wood-based material and φ_{c} is the creep value of concrete. The deformation factor has a similar definition to the conventional creep value (Equation 2), where δ_{inst} and δ_{∞} are the initial and final joint slip, respectively. For this reason, the deformation factor will hereafter be referred to as the creep value, without distinction from the conventional relative creep value. This factor varies significantly (between 0·6 and 4·0) with the type of wood product and service class. In this study, the value of 0·6 was used for situations corresponding to service class 1 and 0·8 was used for situations corresponding to service class 2.

1. $$k_{\mathrm{def,calc}} = 2(k_{\mathrm{def}}\varphi_{\mathrm{c}})^{1/2}$$

2. $$\varphi = \frac{\delta_{\infty} - \delta_{\mathrm{inst}}}{\delta_{\mathrm{inst}}}$$

Nevertheless, the creep value given in Eurocode 5 cannot be directly compared with the creep values determined from slip values measured in long-term shear tests (Van de Kuilen, 1999).

Assuming that the determination procedure given in Eurocode 5 for timber-to-timber joints is also valid for timber-to-concrete joints, the joint final slip is calculated using the joint creep value and its initial slip. The value of the initial slip is determined from the slip modulus K_{s} determined either from experimental tests performed in accordance with EN 26891 (CEN, 1991) or from the expressions given in Eurocode 5 part 1 (CEN, 2003c) in combination with the indications from Eurocode 5 part 2 (CEN, 2004), resulting, for dowel-type fasteners, in Equation 3. The joint final slip, calculated according to Eurocode 5, is then given by Equation 4.

3. $$K_{\mathrm{s}} = 2\frac{\rho_{\mathrm{m}}^{1\cdot5}d}{23}$$

4. $$\delta_{\infty,\mathrm{calc}} = \delta_{\mathrm{inst}}(1 + k_{\mathrm{def,calc}}) = \frac{\boldsymbol{F}}{K_{\mathrm{s}}}(1 + k_{\mathrm{def,calc}})$$

where ρ_{m} is the mean timber density (kg/m^3), d is the diameter of the fastener (mm) and $\boldsymbol{F}$ is the load applied to the joint (N).

In the long-term experimental tests, the creep slip is also determined considering the initial slip. However, in these cases the initial slip is generally taken as the slip after 1 min, 10 min, 1 h or even 12 h. Often, the value at 10 min is taken for the initial stiffness ($K_{10\mathrm{min}}$), but no effect on the definition of 'initial stiffness' has previously been reported. Prior loading is not an option such as it is for timber members since the initial slip would be removed from the actual creep test. This slip would not, or only partly, be recovered after unloading. The final slip of the joint, calculated from the long-term creep value determined from the experimental tests is then given by:

Specimen	Fastener	Concrete class	No. of tests	LTL/STL*	LTL: kN	Environment
8mm	8 mm smooth dowel	C25/30	4	0·33	4·5	Controlled
10mmA	10 mm smooth dowel	C25/30	4	0·31	6·9	Controlled
HSC	10 mm smooth dowel	C50/60	4	0·30	7·1	Controlled
MP	10 mm smooth dowel	C25/30	4	0·29	7·4	Controlled
C	10 mm smooth dowel	C25/30	4	0·31	8·1	Controlled
10mmB	10 mm profiled dowel	C30/37	10	0·30	20·3	Uncontrolled
INT	10 mm profiled dowel with interlayer	C30/37	10	0·30	18·9	Uncontrolled

* LTL, load applied in long-term tests; STL, load applied in short-term tests

Table 1. Properties of the tested configurations

$$\delta_{\infty,\text{meas}} = \delta_{\text{inst}}(1 + k_{\text{def,meas}}) = \frac{\boldsymbol{F}}{K_{10\text{min}}}(1 + k_{\text{def,meas}}) \tag{5}$$

It then becomes clear that the reported creep values obtained from the experimental tests cannot be used without modification since they lead to different long-term deformations due to the use of a different initial stiffness. Assuming that the final deformation calculated with the joint stiffness from a design code has to be the same as the final deformation measured in the creep test (Equation 6), it is possible to derive a modification method.

$$\delta_{\infty,\text{calc}} = \delta_{\infty,\text{meas}} \tag{6}$$

This equation results in a relation between the creep value determined from Eurocode 5 and that derived from long-term experimental tests:

$$k_{\text{def,mod}} = \frac{K_{\text{s}}}{K_{10\text{min}}}(1 + k_{\text{def,meas}}) - 1 \tag{7}$$

When comparisons are made between creep values obtained in joints and creep values obtained in structural timber-based elements, the same analysis needs to be done.

3. Experiment

Seven joint configurations were tested using different types of fasteners and materials (Table 1). The tests were performed in controlled and uncontrolled climatic conditions and the load level applied to the joints corresponded to around 30% of the short-term load-carrying capacity of each joint configuration. The tests took place for relatively long periods (between 2 and 3 years), but these timeframes are short when compared with the lifetime of the structures (often 50 years or more). To overcome this issue, the creep values obtained were fitted to a creep model that was used afterwards to predict the 10 and 50 year creep of the joints.

The purpose of the various joint configurations was to consider the situations most likely to be applied in practice. Three types of dowel were used – 10 and 8 mm smooth dowels of unknown steel grade with a mean ultimate strength of 476 MPa, and 10 mm profiled dowels obtained from concrete reinforcement bars of steel quality S500 according to EN 10080 (CEN, 2005). In addition, both normal-strength concrete (C25/30 and C30/37) and high-strength concrete (C50/60) were used. The wood-based materials used were

(*a*) glued laminated timber (spruce) obtained by gluing C18 solid timber in accordance with EN 338 (CEN, 2003a) with a mean density $\rho_{\text{m}} = 454\ \text{kg/m}^3$
(*b*) solid non-graded maritime pine, $\rho_{\text{m}} = 605\ \text{kg/m}^3$
(*c*) solid non-graded chestnut, $\rho_{\text{m}} = 566\ \text{kg/m}^3$.

One of the test configurations had a 20 mm thick interlayer between the timber and concrete. The interlayer was formed from timber planks with the fibre direction perpendicular to the fibre direction from the main timber member. The properties of the test configurations are given in Table 1; further information about the test configurations can be found in Dias *et al.* (2006, 2007, 2010).

The main objectives of the long-term tests were the assessment of the creep properties of the joints using conditions representative of joints in floors and to obtain results in the shortest period of time. In order to reach the first objective, the load had to be as close as possible to the service load applied to the floor, leading to similar creep behaviour. However, creep phenomena increase with increasing applied load. All the test specimens were thus subjected to a long-term load that made it possible to obtain data in a relatively short period but was low enough to assume linear mechanical creep behaviour (creep caused solely by mechanical load), as occurs in actual structures. That load level was considered to be 30% of load-carrying capacity, as previously determined in short-term shear tests (Dias *et al.*, 2007). The decision was made to have only one load level in order to avoid introducing another variable that would complicate analysis of the results due to the number of tests performed, particularly for the test series 8mm, 10mmA, HSC, MP and C.

Since a great number of tests had to be done, specimens were loaded in columns in such a way that the same load was applied to all the test samples in the same column (Figures 1 and 2). Each column comprised ten test specimens for test configurations 10mmB and INT, and four test specimens for the other configurations. This arrangement and positioning of the specimens meant that the long-term load on each test specimen would be as close as possible to the 30% short-term strength intended.

Test configurations 10mmB and INT were tested at Delft University of Technology in an uncontrolled, indoor, non-heated environment. The temperature and RH at this location have been monitored over a long period of time, and they correspond approximately to service class 2 in accordance with Eurocode 5 (CEN, 2003c). The RH and temperature measured in the laboratory during the test period are shown in Figure 3. Tests with configurations 8mm, 10mmA, HSC, MP and C were performed at the University of Coimbra in controlled environmental conditions; temperature (20 ± 2°C) and RH (65 ± 5%) were monitored and controlled using temperature/moisture controlling equipment. These conditions do not correspond exactly to any of the service classes indicated in Eurocode 5 (CEN, 2003c), being more favourable than service class 1 due to the absence of climatic cycles. The test conditions were, however, closest to service class 1 conditions and therefore the controlled conditions of this work were assumed as corresponding approximately to service class 1.

The duration of the long-term tests varied between 655 days for

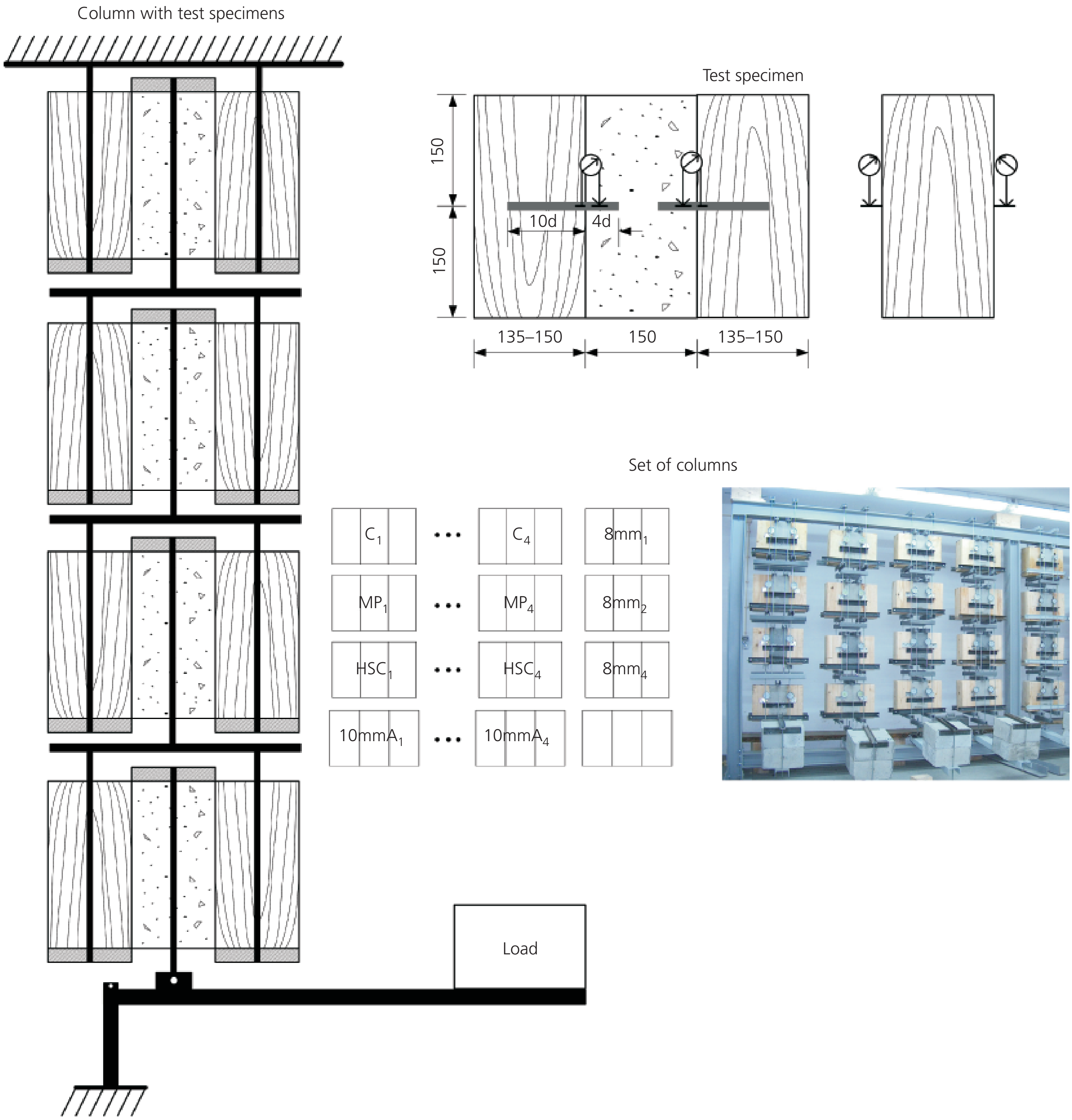

Figure 1. Test setup in controlled climatic conditions (temperature 20±2°C, RH 65±5%) for specimens 8mm, 10mmA, HSC, MP and C (dimensions in millimetres)

INT and around 1160 days for 10mmA, MP, HSC and C test configurations (Table 2).

For the specimens in controlled climatic conditions (Figure 1), the slip was measured at mid-height of the test specimen in both shear planes (resulting in four slip measurements), in exactly the same locations as the short-term tests. These measurements were performed using analogue displacement gauges fixed to the timber member measuring against a steel plate fixed to the concrete member. In the other configurations

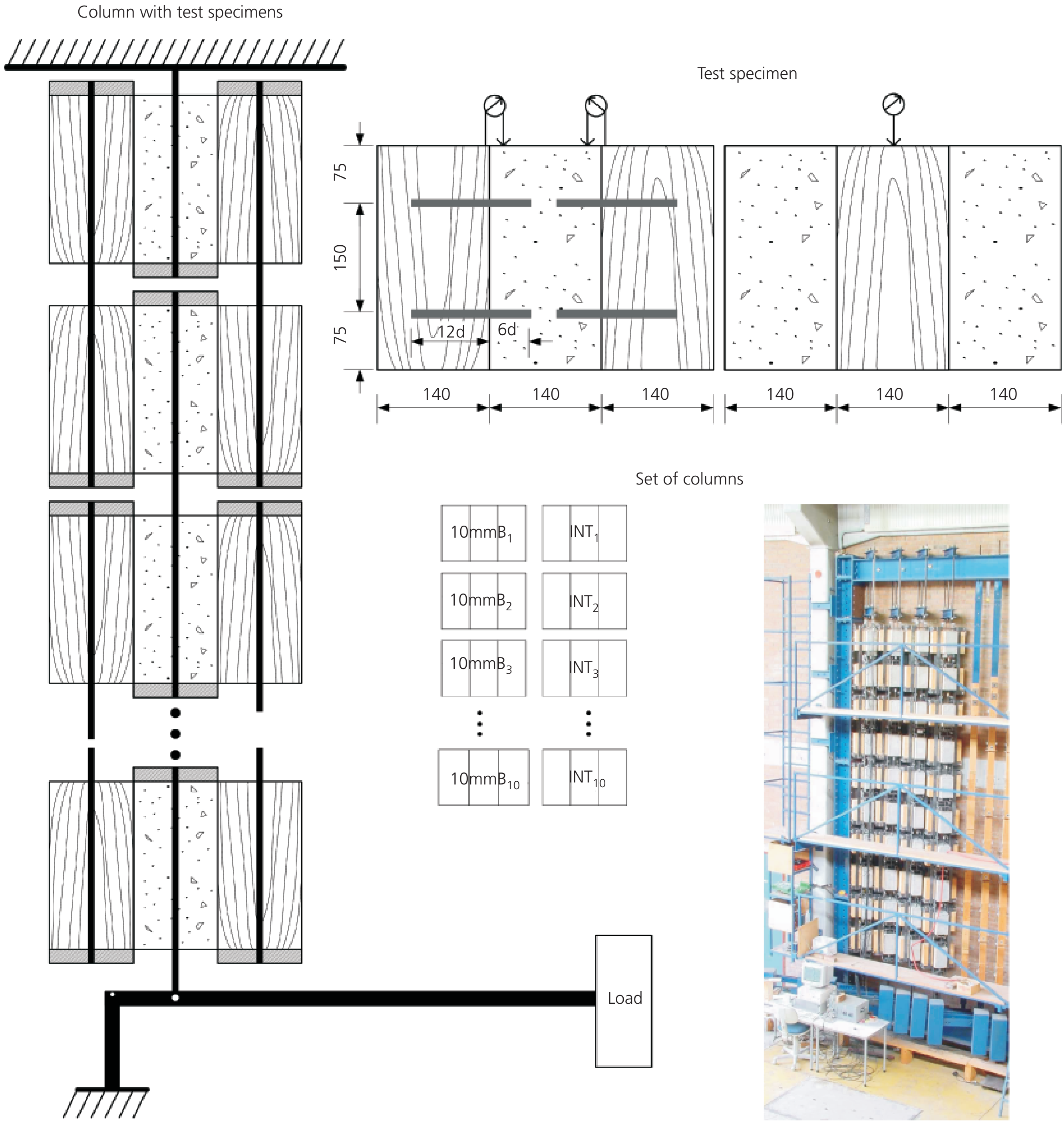

Figure 2. Test setup in uncontrolled climatic conditions (specimens 10mmB and INT) (dimensions in millimetres)

(10mmB and INT, Figure 2), the slip was measured at the top of the test specimen on the symmetry plane, resulting in two slip measurements. These measurements were performed using removable digital displacement gauges placed on top of a steel plate fixed to the concrete member measuring against the timber member.

4. Creep modelling

Due to the limited duration of the creep tests, it was important for the derivation of creep factors for long-term deformations that reliable extrapolations could be achieved by the use of appropriate models. There are no analytical models developed specifically to analyse the long-term behaviour of timber-to-concrete

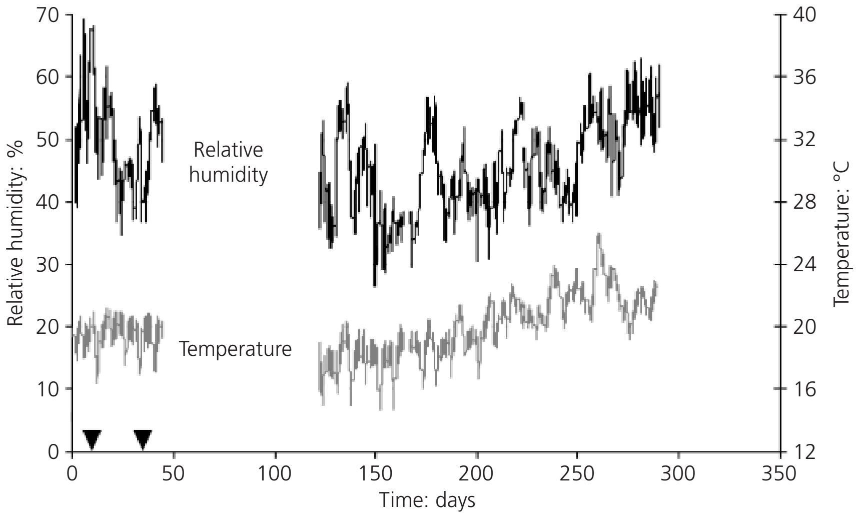

Figure 3. Measured climatic conditions for specimens 10mmB and INT

Specimen	Measurement period: days	Age: days*	Load—LTL: kN	φ_{meas}	C_1	C_2: per day	φ†	
							10 years	50 years
8mm	1000	414	4·5	0·80	0·142	1·216	0·99	1·22
10mmA	1160	239	6·9	0·57	0·080	0·789	0·66	0·79
HSC	1160	235	7·1	1·58	0·237	2·246	1·86	2·25
MP	1160	237	7·4	0·80	0·150	1·197	0·96	1·20
C	1160	232	8·1	0·83	0·121	1·167	0·97	1·17
10mmB	682	428	20·3	1·11	0·178	2·954	2·67	2·95
INT	655	449	18·9	1·08	0·242	3·600	3·21	3·60

* Age of specimens at beginning of long-term test loading
† Creep value calculated using the creep model from Equation 9 with the parameters given in Table 2

Table 2. Measuring periods and creep data of the long-term test results

joints, and models developed for timber-to-timber joints are usually used as an alternative. One such model was proposed by Van de Kuilen (1999), based on the theory of deformation kinetics presented by Van der Put (1989).

The model assumes that, generally, the long-term behaviour of joints can be represented by a non-linear Maxwell element than has an elastic strain rate $\mathrm{d}\varepsilon_{\mathrm{el,i}}/\mathrm{d}t$ representing the elastic behaviour of the material and a visco-elastic strain rate $\mathrm{d}\varepsilon_{\mathrm{v,i}}/\mathrm{d}t$. The sum of the two represents the slip $\mathrm{d}\varepsilon_{\mathrm{i}}/\mathrm{d}t$:

$$8.\quad \frac{\mathrm{d}\varepsilon_{\mathrm{i}}}{\mathrm{d}t} = \frac{\mathrm{d}\varepsilon_{\mathrm{el,i}}}{\mathrm{d}t} + \frac{\mathrm{d}\varepsilon_{\mathrm{v,i}}}{\mathrm{d}t}$$

Assuming the following hypotheses, it is possible to arrive at approximate solutions suitable for use in creep modelling.

(*a*) The behaviour of timber can be represented by a single non-linear Maxwell element with a parallel spring, and the parallel system can be reduced to a three-element system consisting of two parallel Maxwell elements, one of which has a dashpot.
(*b*) The elements are subjected to constant load.
(*c*) The creep time is long enough so that parameters C_1 and C_2 of Equation 9 are not affected by the time the steep part of the creep curve initiates.

Using these assumptions it is possible to express the creep value in accordance with the conventional formulation, resulting in (Van de Kuilen, 1999; Van der Put, 1989):

$$\varphi = \frac{\delta_{\infty} - \delta_{\text{inst}}}{\delta_{\text{inst}}} = C_1 \ln(1 + C_2 T) \quad (9)$$

in which T represents time. Figure 4 shows that the parameter C_2 is closely related to the initial part of the time–slip behaviour, while C_1 mainly influences the behaviour after a certain period of time. Therefore, it is possible to conclude that the model may be used to predict the creep value of joints for long periods (Van de Kuilen, 1999). Due to the complexity of the phenomena involved in the long-term behaviour of timber joints, C_1 and C_2 are not calculated from the joint properties but are obtained by adjusting the model to the experimental results.

5. Test results and discussion

The creep–time curves obtained from the long-term tests are presented in Figures 5–11, along with the analytical model

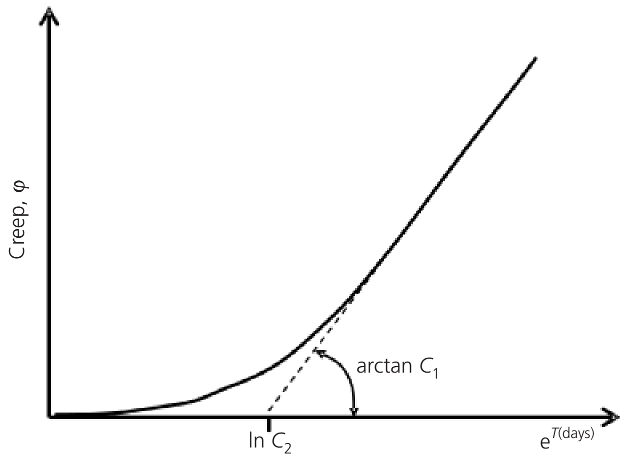

Figure 4. Parameters C_1 and C_2 and the relationship between creep and time

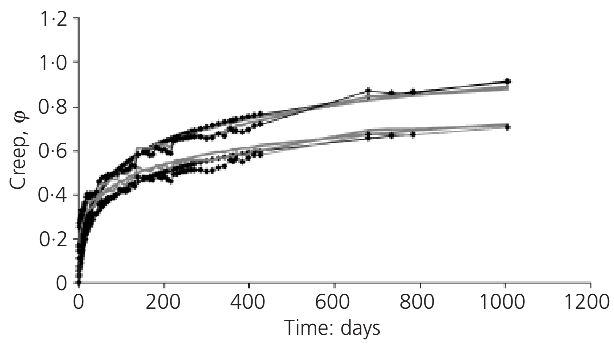

Figure 5. Creep–time curves for the 8mm test configuration joints

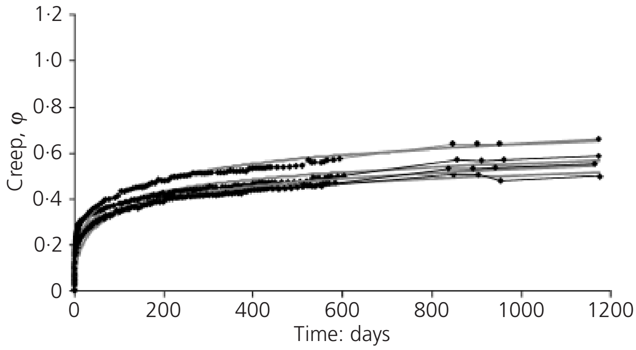

Figure 6. Creep–time curves for the 10mmA test configuration joints

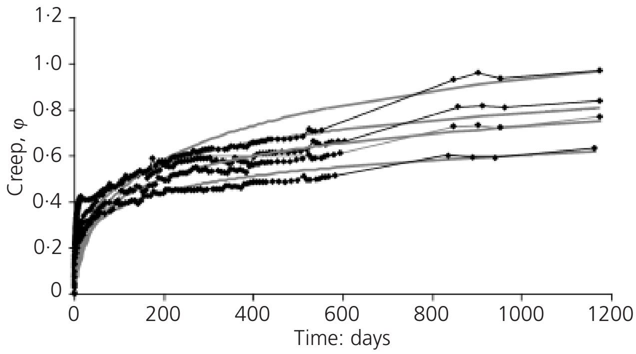

Figure 7. Creep–time curves for the HSC test configuration joints

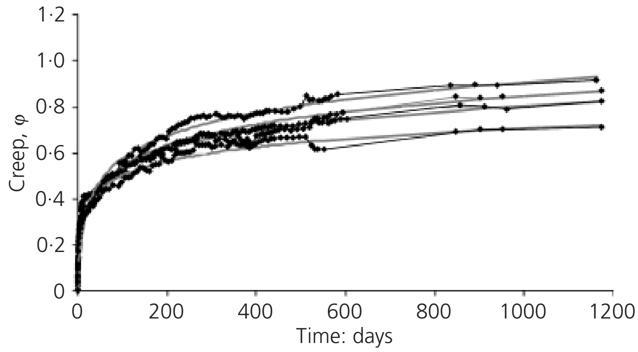

Figure 8. Creep–time curves for the MP test configuration joints

curves (Equation 9) fitted to the experimental results using the least-square minimum method (on a linear scale). For test configurations 8mm, 10mmA, HSC, MP and C, all four test results are presented; the mean curve of the ten test results is presented for 10mmB and INT.

The test duration and load applied to each test configuration are given in Table 2, along with creep values obtained directly from the test results without modification and parameters C_1 and C_2 (Equation 9) obtained by fitting the model results to the test data.

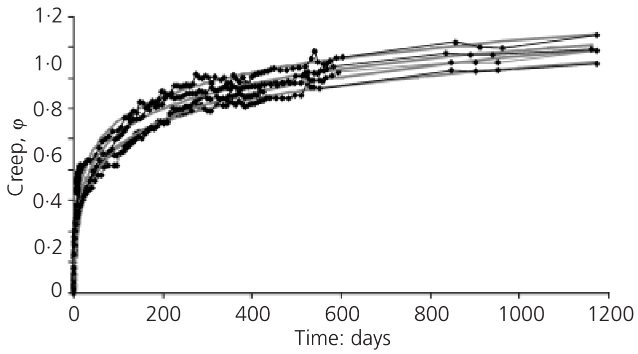

Figure 9. Creep–time curves for the C test configuration joints

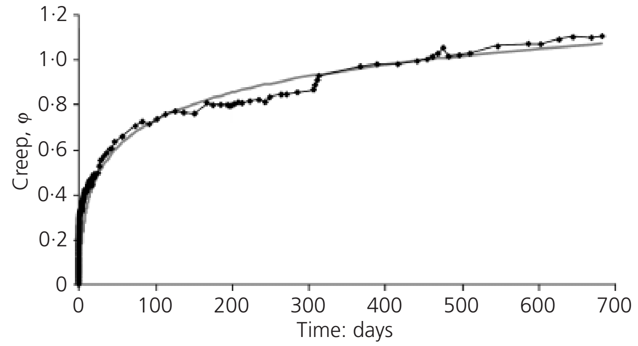

Figure 10. Creep–time curves for the 10mmB test configuration joints

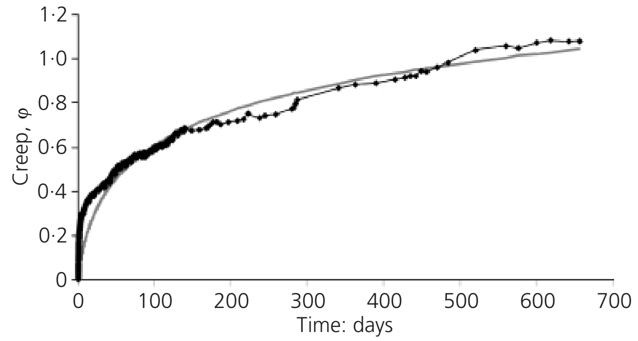

Figure 11. Creep–time curves for the INT test configuration joints

This model was then used to predict the creep values for 10 and 50 years, which are also given in Table 2. The values given are the mean values for each test configuration.

After load durations ranging from ≈2 years for the tests in uncontrolled climatic conditions to ≈3 years for the tests in controlled conditions, the creep values obtained directly from the long-term tests varied between 0·57 for 10mmA and 1·58 for HSC. No significant differences were found between creep values obtained for the tests performed in controlled and uncontrolled climatic conditions, probably due to the shorter measuring periods of the tests in the latter. On the other hand, the predicted creep values for 10 and 50 years are significantly higher for the test configurations tested in uncontrolled climatic conditions. This also depends on the first major moisture change after test initiation, which changes the creep from the first stage (up to a duration of ln C_2 with a smaller increase in creep (see Figure 4)) to the second stage (after a duration of ln C_2 when the steep part of the curve is governed by C_2 (Figure 4). For the tests performed under controlled climatic conditions, the highest values were again obtained for the HSC joint configuration and the lowest for 10mmA, whose creep was approximately one-third of that determined for the HSC test configuration. In terms of properties, the only difference between these test configurations is the type of concrete used (C25/30 in 10mmA and C50/60 in HSC), which – in accordance with Eurocode 2 (CEN, 2003b) – should result in lower creep values for the C50/60 concrete and higher for the C25/30 concrete. For this reason, lower creep values were expected for HSC than for 10mmA, in contradiction to the creep values determined from the test results. The reason for the difference in the creep values is probably not due to differences in the concretes used.

The cause of the differences might be due to the estimation of the initial deformations to which the creep values are quite sensitive (Van de Kuilen and Dias, 2007). Indeed, the initial deformations measured were 1·142 mm for 10mmA and 0·485 mm for HSC. These deformations do not match the elastic deformations measured in the short-term tests, which were quite similar for both configurations. The high initial deformations for the 10mmA test series led to lower creep values at long term and the low initial deformations for the HSC test series led to higher creep values at long term. Furthermore, due to the methodology used to determine the initial deformations, the creep values obtained from experimental long-term tests cannot be directly compared with the values given in Eurocode 5 (CEN, 2003c) as discussed earlier. For that reason, the creep values determined directly from the long-term tests using Equation 2 were modified using Equation 7. For the short-term initial stiffness, two possibilities were considered: values calculated in accordance with the Eurocode 5 models (Equation 3) ($k_{\mathrm{def,mod,1}}$) and values obtained in short-term tests for joints with the same configurations (Dias *et al.*, 2010) ($k_{\mathrm{def,mod,2}}$). In ideal conditions, the initial stiffness should be determined in short-term shear tests. Nevertheless, that it is not always practical and in such situations using Eurocode 5 values might be an acceptable option because, for dowel-type fasteners, the differences between test results and Eurocode 5 values are usually lower than 20% (Dias *et al.*, 2003). The modified creep values are presented in Table 3 for three periods – the measuring period, 10 years and 50 years – conforming to the durations of load classes in Eurocode 5.

Table 3 lists the creep values ($k_{\mathrm{def,calc}}$) calculated in accordance with the indications from Eurocode 5 (CEN, 2003c) ($k_{\mathrm{def,calc}}$) for the various timber-to-concrete joint configurations. The creep

Specimen	$k_{def,calc}$*	Measuring period		10 years		50 years	
		$k_{def,mod,1}$	$k_{def,mod,2}$	$k_{def,mod,1}$	$k_{def,mod,2}$	$k_{def,mod,1}$	$k_{def,mod,2}$
8mm	2·16	2·08	1·75	2·29	1·93	2·55	2·15
10mmA	2·16	4·19	3·49	4·42	3·69	4·76	3·97
HSC	1·29	2·75	2·13	3·05	2·37	3·46	2·68
MP	2·16	2·66	2·40	2·89	2·60	3·24	2·92
C	2·16	1·43	1·81	1·54	1·95	1·69	2·14
10mmB	2·27	1·68	3·63	2·92	6·32	3·15	6·81
INT	2·27	—	3·59	—	7·26	—	7·93

* Creep value calculated assuming service class 1 (in accordance with Eurocode 5) for the test configurations in the controlled climatic environment and service class 2 for the test configurations in the uncontrolled climatic environment

Table 3. Creep values measured in the tests, modified, and creep forecasts for 10 and 50 years. $k_{def,mod,1}$ and $k_{def,mod,2}$ are creep values obtained from the experimental measurements modified using Equation 7 using the initial stiffness given in Eurocode 5 and the initial stiffness obtained in the short-term tests respectively

values of the individual materials were determined following the indications given in Eurocode 5 for timber and Eurocode 2 (CEN, 2003b) for concrete.

Figure 12 shows the predictions for joint creep after 50 years, with and without modification. The results presented in Table 3 and Figure 12 clearly demonstrate that the values obtained directly from tests will underestimate long-term deformations due to differences in the estimation of the initial deformation. That difference is particularly significant for test configuration 10mmA for which the quite low creep values without modification do not lead to low modified creep values. This fact indicates that the low creep values determined for this test configuration directly from the long-term test results might be, at least partially, due to an overestimation of the initial deformation.

The 50-year modified creep values using the initial stiffness obtained from the short-term shear tests ranged from 2·14 to 3·97

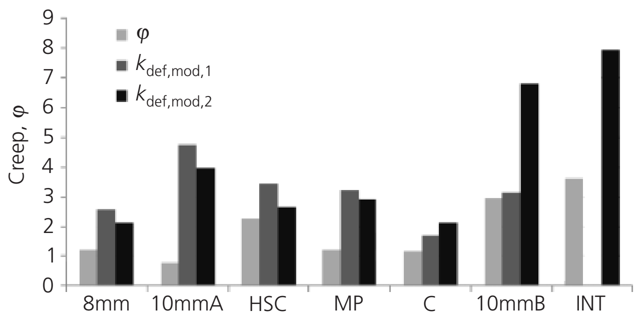

Figure 12. Predictions for joint creep after 50 years, with and without modification

(mean of 2·77) for the tests in controlled climatic conditions, and from 6·81 for a dowel without interlayer to 7·93 with interlayer in uncontrolled climatic conditions. If the modification is performed using the initial stiffness calculated from the model proposed in Eurocode 5 (CEN, 2003c), the variation is higher, reflecting the increased error that results from the use of numerical models; the creep values thus obtained varied between 1·69 and 4·76 for the joints tested in controlled climatic conditions.

These results show that significantly higher creep values are expected for joints exposed to service class 2 conditions rather than service class 1 conditions. The mean values obtained for controlled climatic conditions were 0·92, 1·09 and 1·32 for the measuring period, 10 years and 50 years, respectively. For the uncontrolled climatic conditions and for the same periods, the values were 1·10, 2·94 and 3·28, respectively. If the comparison is made with the modified values ($k_{def,mod,2}$), the trend is similar with mean values of 2·32, 2·50 and 2·77 for controlled climatic conditions and 3·61, 6·79 and 7·37 for uncontrolled climatic conditions. Indeed, the creep values predicted for 10 and 50 years for controlled climatic conditions (class 1) are approximately half those obtained in the uncontrolled climatic conditions.

The creep predictions obtained with the indications given in Eurocode 5 (CEN, 2003c) were 2·16 and 2·27 for service class 1 and 2, respectively, when normal-strength concretes were used. The code values for controlled climatic/service (service class 1) conditions are higher than that obtained for a 50-year loading period without modification (1·32). Nevertheless, the predicted value is lower than the value modified to consider the initial stiffness used in design. The modified values were 2·79 and 3·06 for the initial stiffness obtained in short-term shear tests and from

Eurocode 5, respectively. Furthermore, for the tests under uncontrolled climatic conditions the creep values obtained were also always higher than the Eurocode 5 prediction (7·37 and 3·15 modified using the initial stiffness obtained in short-term shear tests and from Eurocode 5, respectively).

The test results also show that the predicted creep values for 10 and 50 years for the joints with an interlayer are higher than those for joints without. However, the difference is not significant, being approximately 15%.

The creep values given in Table 4 were obtained from a number of studies available in the literature with timber-to-concrete joints and timber-to-timber joints with dowel-type fasteners. These values are compared with the creep values obtained in this work. In order to have comparable values, the creep models (Equation 9 and Table 2) were used to determine predictions for the same loading periods available in each case. The creep values from other research where the joints were tested in service class 1 or equivalent were compared with the mean value obtained for the test configurations in controlled climatic conditions. The creep values from other studies where the joints were tested in service class 2 were compared with the values obtained for test configuration 10mmB. It is important to point out that the grade of the timber used was not always the same; however, in all the test data available solid timber and glued laminated timber showed similar creep behaviours, independently of the timber grade.

The results presented here for controlled climatic tests are similar to those obtained by Jorge (2005) for inclined screws under exactly the same testing conditions. They are, however, higher than the value obtained by Fragiacomo *et al.* (2007) for a commercial fastener, with a difference of 0·18 (≈30%).

For the joints tested in service class 2 conditions, the differences in creep values are much higher. The test results in this work are much lower than those obtained by Van der Linden (1999) based on data available from bending tests on composite systems. The same is true for tests carried out on timber-to-timber nailed joints tested in the same conditions as the 10mmB tests (Van de Kuilen, 1999), with values almost double those reported here. The test results presented here are approximately one-third of those of Kuhlman and Schaenzlin (2001) for tested notches and notches combined with steel fasteners.

These comparisons show that creep values obtained from slip measurements in long-term tests can vary significantly depending on the specific conditions used in each situation, particularly for tests performed in uncontrolled climatic conditions. Nevertheless, the results obtained in the tests presented here are in line with the values available in literature.

6. Conclusions

This paper describes long-term shear tests performed to determine the creep–time behaviour of timber-to-concrete joints. Analysis of the results indicated that creep values are rather sensitive to environmental test conditions and particularly to initial deformations. Therefore, a procedure was developed in order to allow the modification of creep values to consider initial deformations other than those considered in long-term tests, usually based on the slip measured 10 min after application of load.

Due to the significant differences between creep values derived using initial deformations measured in long-term tests and short-term tests, it can be concluded that the creep values used in design codes need to be modified. Indeed, the creep values should consider the initial deformations as used in the short-term design of the structure and not be related to the initial deformation based on measurements in long-term tests. The results also show that, whenever possible, initial deformations should be based on short-term shear tests.

A comparison of the creep values obtained in this work with published values showed that the variation in terms of joint creep values could be significant. Nevertheless, the results obtained in these tests are in line with those obtained in similar research

	Joint	Service class	Test duration: days	φ_{meas}	φ_{calc}†
Fragiacomo *et al.*, 2007	Commercial	1	103	0·55	0·73
Kuhlman and Michelfelder, 2004	Notch with screws	2	240	0·49	1·45
Jorge, 2005	Inclined screws	1	606	0·92	0·98
Van der Linden, 1999*	Inclined screws	1–2	1200	5·50	1·45
Van der Linden, 1999*	Dowel with notch	1–2	1200	3·90	1·45
Van de Kuilen, 1999	Nails (timber-to-timber)	2	4500	3·20	1·69

* Values obtained from bending tests through numerical simulations
† Creep value calculated using the adjusted parameters from Equation 9 given in Table 2

Table 4. Creep values obtained in other research

programmes on other types of timber-to-concrete joints and similar timber-to-timber joints.

The results obtained for creep values in environmental conditions equivalent to service class 2 were significantly higher than (approximately double) those in service class 1 equivalent. On the other hand, the modified creep values (using the same initial stiffness) for dowel-type fasteners were always higher than those obtained using the model indicated in Eurocode 5 (CEN, 2003c). For service class 1, the differences between code and experimental values are moderate but the differences for the joint configurations tested in uncontrolled climatic conditions were quite significant.

Taking these and other published results into account, it is considered that Eurocode 5 (CEN, 2003c) predictions might be acceptable for timber-to-concrete joints exposed to service class 1 conditions. Nevertheless, consideration of lower creep values for higher strength concretes leads to an underestimation of joint creep. The values obtained with Eurocode 5 for service class 2 lead to non-conservative solutions in terms of long-term deformations.

REFERENCES

Balogh J, Fragiacomo M, Gutkowski RM and Fast RS (2008) Influence of repeated and sustained loading on the performance of layered wood–concrete composite beams. *ASCE Journal of Structural Engineering* **134(3)**: 430–439.

Ceccotti A (1995) Timber–concrete composite structures. In *Timber Engineering – Step 2* (Blass H (ed.)). Centrum Hout, Almere-Buiten, pp. E13/1–E13/12.

Ceccotti A, Fragiacomo M and Giordano S (2007) Long-term and collapse tests on a timber–concrete composite beam with glued-in connection. *Materials and Structures* **40(1)**: 15–25.

CEN (European Committee for Standardization) (1991) EN 26891: Timber structures. Joints made with mechanical fasteners. General principles for the determination of strength and deformation characteristics. CEN, Brussels.

CEN (2003a) EN 338: Structural timber – strength classes. CEN, Brussels.

CEN (2003b) Eurocode 2: Design of timber structures. Part 1-1: General – common rules and rules for buildings. CEN, Brussels.

CEN (2003c) Eurocode 5: Design of timber structures. Part 1-1: General – common rules and rules for buildings. CEN, Brussels.

CEN (2004) Eurocode 5: Design of timber structures. Part 2: Bridges. CEN, Brussels.

CEN (2005) EN 10080: Steel for the reinforcement of concrete. Weldable reinforcing steel. General. CEN, Brussels.

Dias AMPG, Van de Kuilen JWG and Cruz HMP (2003) Mechanical properties of timber–concrete joints made with steel dowels. *Proceedings of CIB-W18 Meeting 36, Denver, CO*. CIB-W18, Karlsruhe, paper 36-7-4.

Dias AMPG, Van de Kuilen JWG, Cruz HMP and Lopes SMR (2006) Densified veneer wood for notched joints in timber concrete composite structures. *Proceedings of 9th Conference on Timber Engineering – WCTE 2006, Portland, OR*.

Dias AMPG, Lopes SMR, Van de Kuilen JWG and Cruz HMP (2007) Load-carrying capacity of timber–concrete joints with dowel-type fasteners. *ASCE Journal of Structural Engineering* **133(5)**: 720–727.

Dias AMPG, Cruz HMP, Lopes SMR and Van de Kuilen JWG (2010) Stiffness of dowel-type fasteners timber–concrete joints. *Proceedings of the Institution of Civil Engineers, Structures and Buildings* **163(4)**: 257–266.

Fragiacomo M and Ceccotti A (2006) Long-term behavior of timber–concrete composite beams. I: Finite element modeling and validation. *ASCE Journal of Structural Engineering* **132(1)**: 13–22.

Fragiacomo M, Amadio C and Macorini L (2007) Short- and long-term performance of the 'Tecnaria' stud connector for timber–concrete composite beams. *Materials and Structures* **40(10)**: 1013–1026.

Jorge LFC (2005) *Timber–concrete Composite Structures using LWAC*. PhD thesis, University of Coimbra, Coimbra (in Portuguese).

Kuhlman U and Michelfelder B (2004) Grooves as shear-connectors in timber–concrete composite structures. *Proceedings of 8th Conference on Timber Engineering – WCTE 2004, Lathi*, pp. 301–306.

Kuhlman U and Schaenzlin J (2001) Composite of vertically laminated decks and concrete. *Proceedings of IABSE Conference on Innovative Wooden Structures and Bridges, Lathi*. IABSE, Zurich, pp. 507–512.

Van de Kuilen JWG (1999) *Duration of Load Effects in Timber Joints*. PhD thesis, Delft University of Technology, Delft.

Van de Kuilen JWG (2008) Creep of timber joints. *Heron* **53(3)**: 133–156.

Van de Kuilen JWG and Dias AMPG (2007) Creep of timber and timber–concrete joints. *Proceedings of CIB-W18 Meeting 40, Bled*. C1B-W18, Karlsruhe, paper 40-7-7.

Van der Linden M (1999) *Timber Concrete Composite Floors*. PhD thesis, Delft University of Technology, Delft.

Van der Put TACM (1989) *Deformation and Damage Processes in Wood*. PhD thesis, Delft University of Technology, Delft.