Elevated temperature characteristics of steel reinforcement incorporating threaded mechanical couplers

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Abstract

This paper presents an experimental study into the influence of elevated temperatures on the mechanical properties of hot-rolled steel reinforcement which is spliced using two alternative types of threaded couplers. The investigation includes tests performed under steady-state and transient elevated temperature conditions for reinforcement bars of 16 mm and 20 mm diameter. For comparison purposes, tests carried out under ambient conditions and for non-spliced reinforcement bars are also included in the study. After describing the experimental arrangement and instrumentation, including purpose-adapted digital image correlation techniques, a detailed account of the test results is given. In addition to offering a direct evaluation of the temperature-dependent stiffness and strength properties, the test results provide an assessment of the complete stress-strain response. The strain hardening and ductility properties are also determined as a function of temperature for both spliced and non-spliced specimens. It is shown that the presence of couplers typically influences the ductility characteristics of threaded splices at elevated temperature as a function of the type and geometry of the couplers, whilst the stiffness and strength properties are largely similar to those of their non-spliced counterparts. The performance characteristics obtained from the detailed test measurements are used within the discussions to highlight issues relevant for application in practice.

Keywords: elevated temperatures; steel reinforcement; reinforcement splices; threaded mechanical couplers.

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1. Introduction

In recent years, there has been significant research activity aiming at the development of appropriate fire safety measures for structures [1-4], with a shift from prescriptive idealisations towards more realistic performance-based fire-resistant design approaches [5-7]. More studies are nonetheless required on the influence of temperature on inelastic behaviour, which is critical for a reliable assessment of the performance of structural components and their realistic interactions within the structure under fire conditions.

Structural materials and components typically suffer degradation in stiffness and strength when subjected to elevated temperatures [1,7-9]. Concrete has generally good fire resistance properties, with a much lower thermal conductivity of about 2 W/mK in comparison to steel (approximately 45 W/mK) [1]. Conventional fire design of reinforced concrete (RC) members typically aims at protecting the reinforcement from reaching temperatures above a critical value of around 500ºC for standard grades, by providing an appropriate concrete cover [10]. In contrast, insufficient cover leads to rebar temperatures above the critical temperature with a direct implication on the capacity and ductility [11]. Additionally, spalling of concrete can initiate at temperatures as low as 250-300°C as a function of concrete porosity due to high water vapour pressure [7,12], in the form of a sudden detachment of concrete splinters. As the heating continues, the spalling process is repeated for the remaining concrete, until most of the inherent moisture is lost and no further spalling occurs. As concrete spalling occurs, the reinforcement bars are exposed directly to fire conditions, which may lead to significant loss in member strength [8].

With regards to mechanical couplers, their merits have been long recognized as an alternative to lapped or welded bars due to the reduction in reinforcement congestion and facilitation of a practical assembly process. The performance of mechanical splices is typically defined in terms of the uniaxial ‘in-air’ strength, slip criteria and failure location at ambient temperature, without specific requirements for ductility [13]. Recent studies have shown that the main behavioural characteristics, including ductility, of different splicing forms with couplers depend on the rebar-to-coupler connection, coupler size and arrangement [14]. For example, intrinsic stress concentrations at shear bolts in splices provided with bolt lock couplers (BLC) can result in unreliable failure at the bolts [15]. In splices provided with grouted sleeves (GSC), an insufficient embedment length may generate a bond stress higher than sleeve-grout-rebar bond strength resulting in rebar pull-out from the grout or sleeve [16,17]. Strip-out of the threads or failure of the rebar at the threads in tapered-threaded coupling systems may occur due to reduction in the cross-sectional area of the rebar [18]. Typically, parallel-threaded coupling systems have superior connection characteristics, working
effectively both in tension and compression [19], with splices of relatively compact form exhibiting favourable ductility properties within concrete members [20,21].

Except for GSC types that are provided with a grout layer between the rebar and sleeve, most existing coupling systems seem to rely on the assumption that steel coupling devices would have similar thermal properties as the connected rebars. Limited studies on GSC indicated that the strength and elongation characteristics are directly affected by elevated temperatures, particularly for levels above 400°C at which failure occurred at the rebar within the grouted sleeve [22]. Irrespective of its geometry, the coupler would typically have a reduced concrete cover due to its larger size compared to the rebar, and this has a direct influence on temperature development as well as spalling. Whilst data exists on the temperature-dependent mechanical properties of steel and concrete materials and elements, experimental assessment of the strength and ductility of rebars with mechanical couplers at elevated temperatures is lacking.

This paper presents an experimental investigation into the effect of elevated temperatures on the performance of reinforcement bars which are spliced using two forms of tension-compression couplers, namely parallel threaded couplers (PTC) and cold forged sleeves with parallel thread couplers (PTSC), as well as their corresponding non-spliced counterparts. The tests are part of a wider research project which aims at providing detailed insights into the performance and application of mechanical splices in dissipative RC structures. After providing a brief review of the properties of steel materials at elevated temperatures, the specimen details and testing arrangements are described. The key response characteristics from the ambient, steady-state and transient tests are then discussed and comparative assessments with current codified provisions are presented.

2. Temperature-dependent steel properties

The elevated temperature performance of steel materials depends on various factors such as the chemical composition, steel type, manufacturing process, heating rate, and state of stress [23,24]. Above 500°C, the reduction in strength typically becomes notable [8,10]. The manufacturing process, hot-rolled or cold-worked, has a direct influence on the shape of the stress-strain response and ultimate strains [23], with cold-worked and heat-treated steel products having increased strength, but possible reduction in ductility compared to hot-rolled rebars [24]. The residual strength is also affected by the manufacturing process, with hot-rolled rebars typically exhibiting improved recovery characteristics compared to cold-worked counterparts [1,11,23,25-27].

Codified procedures typically offer prescriptive recommendations for the fire design of RC members in terms of minimum concrete cover [5,6,28] or provide reference to specialised literature.
Reinforcement steel material properties in terms of full constitutive relationships are only given in Eurocode 2 (EC2) [5], while a strength reduction factor for the yield strength is also provided in ACI216-1 [6].

In European guidelines [5], the stress-strain relationships are defined by three parameters: the slope of the linear elastic range \(E_{s,0}\), the proportional limit \(f_{sp,0}\), the maximum stress level \(f_{sy,0}\) and corresponding strain levels (Figure 1a). These are assigned with reduction factors as a function of temperature \(\theta\). The strain at the proportional limit \(f_{sp,0}\) is \(\varepsilon_{sp,0}\), at \(f_{sy,0}\) is \(\varepsilon_{sy,0}\), whilst the deformation at fracture is \(\varepsilon_{st,0}\) and at zero strength is \(\varepsilon_{su,0}\). An elliptic curve is assumed between the proportionality limit and maximum stress level after which a constant strength, without strain hardening, is considered. The linear descending branch proposed beyond a strain of 15% (class B and C reinforcement) is added primarily for numerical reasons for reliable prediction of failure modes in numerical simulations [1].

In terms of reduction in stiffness and strength with temperature, previous test results were reported to be in broad agreement with EC2, assuming that strain hardening is disregarded at all temperature levels [23]. In terms of ductility, the Eurocode approach considers mechanical strains \(\varepsilon_{sy,0}\), \(\varepsilon_{st,0}\) and \(\varepsilon_{su,0}\) as constant values, irrespective of the temperature; these are stipulated as 0.02, 0.15 and 0.2, respectively (for Class B and C reinforcement) and 0.02, 0.05 and 0.1, respectively (for Class A reinforcement). Therefore, the code assumes that the ductility of reinforcement is not affected by the temperature. This assumption was shown to be overly conservative for a set of tests on plain bars with diameters between 6 and 10mm [23] (Figure 1b). In addition to the mechanical strain, under elevated temperatures, the total deformation of reinforcement includes the thermal strain from thermal expansion and the creep strain. The thermal strain, typically recovered after cooling, may be determined using a tri-linear strain-temperature curve as depicted in Figure 1c [5].

Steel materials also exhibit creep when subjected to a combined action of applied stress and elevated temperatures [30-32]. Creep strain becomes significant at approximately 400°C under constant stress state, yet these values may be as low as 300°C as a function of the stress level [7]. Methods for the assessment of creep strains for steel materials exist, and creep can be independently assessed or implicitly accounted for in stress-strain representations used in analysis [33-35]. At constant stress level and under elevated temperatures, the creep strain is a function of the stress and a representative temperature-compensated time parameter \(\Theta\) [33,36]. As illustrated in Figure 1d, the creep response can be divided into three stages referred to as primary, secondary and tertiary creep. The first phase is characterised by a decreasing deformation rate slowing down to a steady strain rate in the second phase. The strain rate accelerates during the tertiary phase because of plastic
instabilities leading to failure [8]. Assessments of the complete creep strain versus temperature-compensated time relationship curve are typically obtained from material specific empirical creep laws [36-38].

This paper presents an experimental investigation into the effect of elevated temperatures on the performance of reinforcement bars incorporating two types of mechanical splices provided with tension-compression couplers as well as the corresponding non-spliced specimens. The tests consider practical temperature levels that are likely to occur within concrete elements, or at the rebar after concrete spalling occurs. The key response characteristics from the tests at ambient temperature, steady-state and transient elevated temperature are discussed. In addition to assessing the adequacy of current codified provisions in determining the conventional stiffness and strength parameters, particular emphasis is given to examining the influence of temperature on post-peak hardening and ductility as well as creep response for both spliced and non-spliced specimens.

3. Experimental response at ambient temperature

Two types of threaded couplers were considered in this study (Figure 2), namely parallel threaded couplers (PTC) and cold forged sleeves with parallel thread couplers (PTSC), used in conjunction with two practical rebar sizes (16 mm and 20 mm). The PTC is a compact coupler with diameter \( d_c \times \text{length} \ L_c = \Phi 25 \times 45 \text{ mm for 16 mm bars and } \Phi 31 \times 54 \text{mm for 20 mm bars, while the PTSC has a diameter } d_c \times \text{length} \ L_c = \Phi 25 \times 206 \text{ mm for 16 mm bars and } \Phi 31 \times 214 \text{ mm for 20 mm bars} [39,40]. These are referred to throughout the paper as compact and slender couplers, respectively. The PTC splice is provided with standard female threads at the coupler and matching male threads at the rebars. The PTSC has matching male-female threads forged from a steel sleeve that is mechanically connected to the unaltered reinforcing bars. Through the fabrication process of these couplers, the cross-sectional area at the threads remains the same.

A total of sixty specimens equally divided between the two diameters were considered, and in three groups, namely non-spliced, compact and slender. The experimental investigation included uniaxial ‘in-air’ assessments as follows: (i) tensile tests at ambient temperature; (ii) steady-state tests at elevated temperature and (iii) transient elevated temperature tests. Specimens incorporating 16 mm rebars were tested in the first phase to obtain qualitative information about the response of mechanical splices at elevated temperatures. These were followed by tests on 20 mm splices with focus on the characteristic behaviour at the coupler region - defined as the length of the coupler plus two rebar diameters each side of the coupler \( (L_c+4d_b) \). The specimen reference uses the format of Ax-yyM-Tz, in which Ax indicates the type of the coupler (0 for non-spliced, C for compact
coupler and S for slender coupler), yyM refers to the rebar size (16 or 20 mm) and Tz denotes the type of the test or temperature (TA: ambient temperature, TS: steady-state elevated temperature, TT: transient elevated temperature). Prior to assessing the response at elevated temperature, it is essential first to determine the properties at ambient temperature as described below.

The ambient temperature tests were carried out in conformity to ISO 15630-1 [41] and ISO 15835-1 [13] that provide guidance on testing procedures for reinforcement steel, and for reinforcement couplers, respectively. An Instron 8802 testing machine with a capacity of 250 kN was used for all tests on bars and splices (Figure 3). The testing machine was provided with high-pressure jaws that restrain the clamped rebar region without slip. Two tests were carried out for each configuration using a one-stage loading procedure with a displacement rate of 3.6 mm/min. Each specimen was approximately 1000 mm in length of which 70 mm was gripped in the jaws of the testing machine. The clear length of the specimens between the two jaws was also carefully measured.

A Digital Image Correlation (DIC) system, consisting of two cameras and a controller was used to record deformations, as this offers a high level of accuracy and practicality compared to conventional mechanical instrumentation at ambient [42, 43] and elevated temperature [38, 44, 45]. High contrast black/white patterns were provided to the front face of the specimens by applying one layer of aerosol white paint and, subsequently, a fine black speckle pattern using the same technique, to ensure smooth detection of surface deformations during testing. The DIC data was further processed to obtain deformation vector fields. From these, uniaxial splice deformations were assessed from assigned virtual gauges corresponding to the coupler region ($L_c + 4\times d_b$) for spliced bars or $4\times d_b$ for non-spliced bars. A frequency of 0.2Hz for recording data was chosen to acquire a sufficiently large pool of data to minimise possible scatter [44].

Figure 4 illustrates the stress-deformation response of the non-spliced members and spliced members at ambient temperature, whilst Table 1 gives the yield strength $f_{sy,20^\circ C}$, ultimate strength $f_{su,20^\circ C}$, ultimate strain $\varepsilon_{su,20^\circ C}$ corresponding to $f_{su,20^\circ C}$, and strain at fracture $\varepsilon_{sf,20^\circ C}$. All curves are largely similar on average with $\varepsilon_{su,20^\circ C}$ in the range 8.5-11.8% and $\varepsilon_{sf,20^\circ C}$ in the range 9.4-12.5%, whilst the yield strain $\varepsilon_{sy,20^\circ C}$ is nearly identical within each set (16 mm and 20mm). The $\sigma$-$\varepsilon$ curves show a consistent decrease in $\varepsilon_{su,20^\circ C}$ between the tested configurations. The non-spliced bars had the highest $\varepsilon_{su,20^\circ C}$, of about 12% (12.4% and 11.2% for 16 mm and 20 mm, respectively), with splices including slender couplers having the highest decrease in ultimate strain, with $\varepsilon_{su,20^\circ C}$ of 8.2% for 16 mm and 8.8% for 20 mm splices. Splices with compact couplers reached $\varepsilon_{su,20^\circ C}$ of 10.5% and 9.4% for 16mm and 20 mm, respectively. The small differences at ultimate are related to typical inherent material variations in coupler length-to-rebar free length ratio. It is worth noting
that both coupling systems perform well under monotonic loading with failure developing mostly outside of the coupler region. Although the level of ultimate deformation decreased for threaded splices compared with the non-spliced specimens, the test results indicate that the minimum requirement for S500 grade Class C reinforcement (i.e. $\varepsilon_{uk}>7.5\%$) in EC2 was met \[46\].

Figures 5a,b illustrate the strain concentrations obtained from optical measurements at yield $\varepsilon_{sy,20^\circ C}$ and prior to fracture $\varepsilon_{sf,20^\circ C}$ at ambient temperature. The intrinsic characteristics of the mechanical splices with compact couplers, that have an enlarged rebar cross-section near the threads due to the manufacturing process, and show minimal stresses recorded at the coupler region. On the other hand, for slender couplers, some strain localisation may occur at the threads as well as at the coupler-to-rebar interface, which can subsequently promote a failure at the coupler region in some cases \[19\].

In addition to the non-spliced bars and the threaded splices described above, ambient temperature tests on material extracted from the couplers to assess its properties, as this may have an influence on the behaviour in some cases as discussed above. Dog-bone type specimens, illustrated in Figure 6a, were manufactured by cutting the couplers, without rebars, in three equal longitudinal slices, which were subsequently welded to high strength 30 mm steel rods. Each specimen cross-section was determined from detailed measurements at the thinnest section, namely at the indentation produced by ribs in the sleeve of the slender couplers and at the root of the thread of the female part of the compact coupler. Three tensile tests were carried out for each configuration and typical results, depicted in Figure 6b, indicate yield strengths around or above the yield strength of the connected rebars. The 0.2% proof strength for the material from the compact coupler was $f_{sy,20^\circ C} = 526$ MPa, whilst for the slender coupler $f_{sy,20^\circ C} = 654$ MPa. The ultimate strengths were 1% and 5%, respectively, higher than the yield. In contrast to the rebar materials that had $\varepsilon_{su,20^\circ C}$ in the range of 12% and showed considerable hardening, the materials from the slender and compact couplers were significantly more brittle developing $\varepsilon_{su,20^\circ C}$ of about 1.5% and 0.5%, respectively.

In general, the larger cross-section at the coupler region is designed to lead to failure within the rebar, which is typically the case under ambient conditions. Consequently, in a splice with finite length, higher straining is developed at the rebar due to an implicit concentration of stresses within the rebar region. Prior to necking, the entire rebar region is continuously strained at different rates across its length. Implicitly, the inelastic strains are distributed over a shorter rebar length, whilst the length corresponding to the coupler is typically elastic, hence producing earlier failure and lower ductility. The reduction in $\varepsilon_{u,20^\circ C}$, between non-spliced-compact-slender splices would become proportionally less significant with the increase in total specimen length. In terms of
capacity at ambient temperature, the yield characteristics are generally not influenced by the presence of a threaded coupler, unless large inherent variations of the material exist. The difference in proof and ultimate strengths between spliced and non-spliced specimens were within ranges of 1% and 3% for the specimens reported in Table 1.

4. Steady state tests

An Instron 3119-408 Environmental Chamber with a temperature range of −150°C to +600°C was used to heat the specimens to the desired temperature with an average heating rate of 12.5°C/min (Figure 3), which is within the realistic heating rates occurring in building fires. A set of six thermocouples were used throughout the length of the specimen. Loading was applied when the temperature difference between two thermocouples at the edges of the couplers stabilised around 5°C or 1% of the target temperature. High contrast black/silver patterns were provided to the front face of the specimens by applying high-temperature aerosol black paint, typically used for vehicle engines, as background, and subsequently, a fine silver speckle pattern. The non-contact DIC system used the cameras to record images through the glass window of the environmental chamber, which were further processed to obtain vector deformation fields. To suppress the red shades produced by the heating, orange filters were provided to the lenses of the DIC cameras.

4.1 Overall constitutive response

The overall experimental stress-strain (σ-ε) curves are illustrated in Figures 7a-f in which the stress was determined from the recorded machine load divided by the nominal rebar area, and the strain was obtained from non-contact DIC measurements. Figures 7a,b show that for non-spliced specimens, at temperatures below 400°C, there is no significant reduction in terms of stiffness and strength, with the yield plateau starting to disappear at 300°C, becoming evident at 400°C. Moreover, the strain hardening diminishes at around 400-500°C with clear softening at θ=600°C. The strain distribution at 600°C in non-spliced bars is also captured in Figure 8a showing DIC images at yield and at fracture for 16 mm rebars, and in Figure 9a,d at 500 °C and 600 °C, respectively, for 20 mm rebars. Prior to rebar fracture, concentration of plasticity occurred at a slower rate after εu,600°C was reached, allowing some ductility to be recovered in comparison with θ<600°C cases.

In contrast, for the slender and compact splices, at θ≥500°C, the necking and fracture typically occurred at the coupler to rebar interfaces (Figure 8c and Figure 9b,c,e,f), and exhibiting a less ductile behaviour in comparison with the non-spliced specimens. Other than that, the reduction in capacity for spliced members in Figures 7c-f with the increase in θ is broadly similar to the non-
spliced specimens in Figures 7a-b. Lower reduction factors in terms of $\varepsilon_{su,\theta}$, are noted for spliced rebars at or above 400°C, in comparison to the non-spliced specimens, due to reductions in initial stiffness $E_{s,0}$ and proportionality limit $f_{sp,0}$, which become strongly visible at 400°C. An analysis of the strain distribution in characteristic regions of the coupler length ($L_c+4d_b$), showed that at $\theta \geq 400^\circ$C, all regions (coupler, coupler-to-rebar interface and rebar) reach strains about 1% at which strain hardening/softening is activated. Up to this temperature level, all components deform in a similar manner, whilst for $\theta \geq 400^\circ$C the response depends on the material properties of the rebar and coupler and strain localisation characteristics.

In terms of $f_{sp,\theta}$ and $f_{sy,0.2\%,\theta}$, the reduction factors for the non-spliced specimens and splices with compact couplers are nearly identical, whilst for the splices with slender couplers these are slightly higher (Table 2b). The material properties of the couplers depicted in Figure 6b point out to a relatively homogenous strength properties throughout the length of threaded compact splices as $f_{sy,20^\circ}$C was within similar ranges with the rebar $f_{sy,20^\circ}$C and, hence, no influence on the response. In contrast, slender couplers had $f_{sy,20^\circ}$C about 20% higher than rebar $f_{sy,20^\circ}$C. This difference in material strengths combined with relatively large geometry, may have influenced the response $\theta \geq 400^\circ$C. Additionally, Figure 8 indicates two distinct strain localisation behaviours for the splices and rebars: two regions with strain localisation for splices and only one region for non-spliced bars. The above comments combined with inherent scatter in material properties may have contributed to the $f_{sp,\theta}$ and $f_{sy,0.2\%,\theta}$ enhancement for the splices with slender couplers in comparison with non-spliced counterparts.

As in the case of non-spliced rebars, a reduction in $f_{sp,\theta}$, occurred at around 300°C and became more significant with further increase in $\theta$. As illustrated by the $\sigma$-$\varepsilon$ in Figure 7c-f and the strain maps in Figure 9b,c,e,f, the 20 mm rebars provided with compact couplers tended to have a more brittle response in comparison to the splices with slender couplers above 500°C. The dilation of the compact couplers combined with the influence of the cold forging manufacturing process of the threaded rebar heads, led to a more rapid failure at the threads. An interesting behaviour was observed for the splice with compact coupler at 600°C, as depicted in Figure 7f, which exhibited significant strain softening. After $\varepsilon_{su,600^\circ}$C was reached, both the rebar and the coupler had a nearly simultaneous necking as captured by the DIC system prior to fracture (see Figure 8b), with failure occurring eventually outside of the coupler.
4.2 Initial stiffness and proportional limit

The reductions in initial stiffness $E_{s,\theta}$ and proportionality limit $f_{sp,\theta}$ are depicted in Figures 10a and 10b respectively, as determined from the $\sigma$-$\varepsilon$ curves in Figures 7. For $E_{s,\theta}$, the results of non-spliced rebars are generally in very good agreement with the EC2 model for Class X reinforcement [5], whilst the curve for Class N reinforcement tends to offer largely conservative estimates of the reduction factors by about 11% on average and a coefficient of variation of 15% compared to the test data. Eurocode recommends Class N for general use, whilst Class X may be used only when there is experimental evidence for the tabulated values [5]. Importantly, similar trends were also obtained for the specimens provided with couplers, in terms of relative reduction of $E_{s,\theta}$ in comparison to $E_{s,20^\circ\text{C}}$, indicating that the Eurocode models can be employed. However, it is worth noting that the presence of the coupler influenced slightly the initial stiffness of the specimen, particularly for the slender couplers where the increase was up to about 10%.

As noted before, the yield plateau clearly demonstrated in Figure 4 at ambient temperature tends to disappear at around 300°C. Figure 10b indicates that the proportionality limit $f_{sp,\theta}$ decreases gradually with $\theta$ in a similar manner to $E_{s,\theta}$. In the case of $f_{sp,\theta}$, there is however a clearer distinction between non-spliced and spliced specimens as a function of the coupler configuration. Whilst the results corresponding to non-spliced specimens are close to the Eurocode curve (Class N), those for splices with couplers are shifted gradually away from curve, particularly at $\theta>300^\circ\text{C}$. The average values between the test-to-model $f_{sp,\theta}$ reduction factors increases from 1.18 to 1.24 and 1.35, as a function of the coupler size, for non-spliced bars, specimens with compact and slender couplers, respectively. The length and cross-section size of the coupler therefore influences the proportionality limit, with larger configurations delaying the development of inelasticity. Nonetheless, the EC2 Class N model offer a largely conservative representation in all cases.

4.3 Yield strength

Figure 11a illustrates the influence of the temperature on the yield strength, considered as the stress corresponding to the initiation of the yield plateau at $\theta \leq 300^\circ\text{C}$, or at the end of the rounded curve region for $\theta>300^\circ\text{C}$. For comparison, the offset 0.2% proof strength and the strength at an effective strain of 2% are shown in Figure 11b for the non-spliced members tested in this paper and from the literature [9,23]. Additionally, the EC2 [5], ACI 216 [6], background document to EC2 [47] $f_{sy,\theta}/f_{sy,20^\circ\text{C}}$ vs $\theta$ reduction curves for hot-rolled reinforcement steel are presented in Figure 11. It is worth noting that according to EC2, $f_{sy\theta}$ corresponds to the ultimate strength $f_{sub}$ as it assumes that strain hardening is negligible at all values of $\theta$ (Figure 1a). Figure 11 illustrates two distinct $f_{sy,\theta}/f_{sy,20^\circ\text{C}}$ vs $\theta$ reduction curves depending on the location of hot-rolled reinforcement in the
member for EC2 Class N reinforcement [5,47]. Curve 1 of Class N (Curve N-1) is associated with ductile failures of members in bending (beams and slabs) and is connected to the 2% strain. On the other hand, curve 3 of Class N (Curve N-3) is applicable to elements with limited strain capacity (strongly reinforced beams, columns) and is connected to the 0.2% strain offset. Additionally, Figure 11 depicts a $f_{sy,\theta}/f_{sy,20^\circ C}$ vs $\theta$ curve for EC2 Class X reinforcement [5] which is derived from a specific test data [47]. ACI 216 also assumes a perfect plastic behaviour without hardening for reinforcement steel at elevated temperatures. As observed from Figure 11a, the test results for both non-spliced and spliced specimens indicate a gradual decrease in the $f_{sy,\theta}$ in relationship to $\theta$ with the test data being largely within the bounds of the code relationships.

For the 20 mm non-spliced specimens (A0-20M), the test results overlap the EC2 Curve N-1, whilst for 16 mm (A0-16M) and 10 mm (D10) [23], these are below the same design curve up to $0\leq400^\circ C$ and align with the curve for $\theta>400^\circ C$. On the other hand, the Class X curve can be regarded as an average test curve. The ACI 216 curve is consistently located below all test results up to 400°C and above the test results beyond this. EC2 Curve N-1 seems to offer unconservative estimates for $0\leq400^\circ C$ for relatively small bar diameters, while EC2 Class X is more conservative. Additionally, EC2 Curve N-3 is a lower bound of all test data for $\theta>100^\circ C$. For splices with compact couplers (AC-20M and AC-16M) developed similar trends as non-spliced specimens, with $f_{sy,\theta}$ located between the EC2 and ACI 216 curves for $0\leq400^\circ C$ and in the vicinity of the design criteria above this temperature. On the other hand, for splices with slender couplers (AS-20M and AS-16M), the results in terms of $f_{sy,\theta}$ overlap with the EC2 reduction factors (Curve N-1).

As for temperatures above 300°C, the yield plateau of hot-rolled steel rebars is absent, a representative measure of yield may be the proof stress at 0.2% strain ($f_{y,0.2\%,\theta}$) or the effective yield strength at a total strain of 2% ($f_{y,2.0\%,\theta}$). The latter value seems to correspond to the critical temperature, as observed from transient tests [9] and is often used for characterisation of yield properties of stainless steel [48]. As observed from Figure 11b that depict $f_{y,0.2\%,\theta}$ and $f_{y,2.0\%,\theta}$ for the non-spliced 20 mm and 16mm bars (A0-20M and A0-16M, respectively) as well as 10 mm, 12 mm and 25 mm bars from the literature (D10 [23], D12 and D25 [9]), $f_{y,2.0\%,\theta}$ is a representative criteria for ductile members [47] as all corresponding data are in the vicinity of the EC2 Curve N-1. Curve N-3 follows the test data at 0.2% strain offset, emphasizing its application for members with limited strain capacity [47]. Class X curve is typically mid-range between $f_{y,0.2\%,\theta}$ and $f_{y,0.2\%,\theta}$ data points.

As shown in Figure 11, the ACI 216 curve intersects the average data trend around 400°C, indicating conservative estimates below this temperature and unconservative estimates above.
Steady state tests on hot-rolled structural steel typically provide results below the Eurocode envelope [48], probably since the model is based on transient data [49-51], indicating that a representative value for $f_{sy,0}$ is either at 1% or 2% strain. Importantly, Figure 11 and Table 2a,b indicate that the observations for non-spliced members in terms of EC2 yield criteria [5,47] are equally apply for splices with compact and slender splices. It is worth noting that due to the more brittle response of some splices for $\theta \geq 500^\circ$ C, these were unable to develop strains in the range of 2%. Hence, for such cases, the maximum strain corresponds to yield and ultimate strength.

4.4 Ultimate strength and strain hardening

Figure 12 illustrates the reduction in ultimate strength $f_{su,0}$ as a function of the steady-state temperature $\theta$ for non-spliced bars (A0, D10 [23]) and rebars provided with compact (AC) and slender couplers (AS), normalised against the ultimate strength at ambient temperature $f_{su,20^\circ}$C. Additionally, the figure presents the EC2 reduction curve for the yield strength $f_{sy,0}$ (Classes N and X) [5]. It is worth noting again that EC2 assumes that the strain hardening is negligible at all $\theta$ and hence the maximum stress level $f_{su,0}$ is treated as the effective yield strength $f_{sy,0}$. Although this is a conservative measure, the $\sigma$-$\varepsilon$ curves for 20 mm non-spliced bars from Figure 7a showed that the strain hardening becomes insignificant only after 500°C. At lower temperature, the actual $f_{su,0}$ is implicitly higher than $f_{sy,0}$.

Based on the results in Figure 12, the temperatures $\theta$ at which strength degradation occurs for non-spliced bars are between 300-400°C. For example, at 400°C, the reduction in $f_{su,0}$ is about 15%, whilst at 600°C it is around 60% in comparison with $f_{su,20^\circ}$C at ambient. The degradation in terms of $f_{su,0}$, for both spliced types, are within the same ranges as those for the non-spliced bars. Generally, for all specimens, beyond $\theta$ around 400°C, $f_{su,0}$ is similar to $f_{su,20^\circ}$C, whilst above 400°C the reduction in ultimate properties is proportional to $\theta$. In terms of strain hardening, rarely mentioned in the literature for reinforcement steels, the parameters $\rho_{y0.2%,\theta}$ and $\rho_{y2.0%,\theta}$ are considered for comparison, with $\rho_{yi,\theta}$ representing the ratio $(f_{su,i,\theta}-f_{sy,i,\theta})/f_{sy,i,\theta}$. For specimens subjected $\theta \leq 300^\circ$C, the proof strength $f_{sy,0.2%,\theta}$ corresponds to $f_{sy,0}$, as below this temperature the yield plateau does not change. Although EC2 does not consider strain hardening, it can be accounted for steel and composite structures in EC3 and EC4 [48,52] for $\theta \leq 400^\circ$C. A constant $\rho_{y,\theta}=0.25$ is considered up to 300°C, followed by a descending branch up to 400°C at which $\rho_{y,\theta}=0$.

The relationship between $\rho_{yi,0}$ and $\theta$ is illustrated in Figure 13a for non-spliced specimens and in Figure 13b for the spliced specimens. It can be observed that for all specimens the test results lie systematically below the envelope with low scatter for $\theta \leq 400^\circ$C. It is also worth noting that $\rho_{y0.2%,\theta}$ increases gradually with $\theta$. At ambient temperature, $\rho_{y0.2%,\theta}$ is within 15.5-19.5% for non-spliced
specimens, which is in good agreement with the codified requirements for S500C reinforcement [46]. An increase in temperature up to 300°C shows proportional enhancement in \( \rho_{y0.2\%} \) by up to 30% in comparison with \( \rho_{y0.2\%,20^\circ C} \). Moreover, as the yield plateau disappears, the ratio between the proof strength and ultimate strength increases with the largest difference being at 400°C. As \( \theta \) rises above 400°C, \( \rho_{y0.2\%,\theta} \) diminishes. On the other hand, \( \rho_{y2.0\%,20^\circ C} \) shows nearly identical values to \( \rho_{y0.2\%,20^\circ C} \) as the yield strength is the same as the proof strength and the strength at 2% strain. At elevated temperatures, there is a systematic reduction in the 2.0% hardening parameter with \( \theta \).

As depicted in Figure 13b, the strain hardening capacity of the specimens with compact couplers seems to follow a similar behaviour to the non-spliced bars, primarily since the size of the coupler with respect to the total length of the specimen is relatively small. Both \( \rho_{y0.2\%} \) and \( \rho_{y2.0\%} \) average trends for compact couplers are largely consistent with \( \rho_{y0.2\%} \) for the non-spliced rebars. The value of \( \rho_{y0.2\%} \) is around 20% at ambient temperature, increasing to 25% at 300°C, and peaking up to nearly 40% at 400°C. At higher \( \theta \), the strain hardening capacity diminishes. These observations are also largely valid for specimens with slender couplers, but the average line is shifted below the corresponding line for compact couplers, with a lower \( \rho_{y0.2\%} \) of about 25% on average. On the other hand, \( \rho_{y2.0\%} \) indicates a linearly decreasing trend, nearly identical to that observed for non-spliced specimens (Figure 13b).

The above observations suggest that strain hardening may be considered for representing the response of reinforcement steel at elevated temperatures with exact values depending on the yield strength definition (i.e. proof strength at 0.2% or effective strength at 2% strain). Similarly, for splices provided with threaded compact and slender couplers, this aspect may be considered as long as the influence from the coupler is considered.

### 4.5 Ductility characteristics

The ductility characteristics of steel reinforcement at elevated temperatures are often disregarded. EC2 assumes that \( \varepsilon_{st0} \) and \( \varepsilon_{su0} \) are constant regardless of \( \theta \) and type of steel, yet it assumes distinct values as a function of the reinforcement grade. For Class B and C, \( \varepsilon_{st0} \) that corresponds to the maximum stress level \( f_{sy} \) is 15%, whilst \( \varepsilon_{su0} \) that corresponds to total loss of strength is 20%. For Class A reinforcement, these values are 5% and 10%, respectively. As mentioned in Section 3, the ultimate strain \( \varepsilon_{su20^\circ C} \), corresponding to the maximum strength \( f_{su20^\circ C} \) was between 8.5-11.8% and the strain at fracture \( \varepsilon_{sf20^\circ C} \) was 9.4-12.5%. The \( \sigma-\varepsilon \) curves in Figure 4 showed a consistent decrease in \( \varepsilon_{su20^\circ C} \) as a function of the coupler size. Although the level of ultimate deformation decreased for spliced specimens compared with the non-spliced rebars, the test results indicated that the minimum criterion for Class C reinforcement (i.e. \( \varepsilon_{uk}>7.5\% \)) in EC2 was met. On the other hand, the \( \sigma-\varepsilon \)
curves at elevated temperature, in Figure 7, indicated a decrease in $\varepsilon_{u,0}$ with $\theta$. As shown in Figure 14a, for the non-spliced members, a rapid decrease occurs until 100°C, whilst up to 500°C $\varepsilon_{u,0}$ is largely constant. Also, the ratio between ultimate strain at 600°C to that at ambient temperature is between 0.25-0.31 as a function of the bar diameter.

For both types of splices, with compact and slender couplers, there is a systematic reduction in $\varepsilon_{su,0}$ compared to non-spliced specimens. At 600°C, $\varepsilon_{su,0}/\varepsilon_{su,20°C}$ is around 0.20 for specimens with slender couplers and 0.10 for those with compact couplers. These ratios involve ultimate strains $\varepsilon_{su,600°C}$ of around 3.5% for non-spliced members and 1.5% for spliced specimens. However, at fracture, non-spliced specimens reached $\varepsilon_{sf,600°C}$ up to 14.4% above $\varepsilon_{sf,20°C}$ at ambient temperature, yet at the expense of a decrease in capacity of around 25%. On the other hand, most specimens with couplers developed much lower fracture strains above 500°C in comparison to non-spliced rebars (Table 2), indicating a more brittle behaviour. Even for non-spliced rebars, at 500°C the failure was largely less ductile than at ambient temperature with no clear sign of necking (Figure 15). For spliced specimens, this is largely attributed to the manufacturing processes. The cold forging of the threaded head for compact couplers, and the machine swaging for slender couplers may have modified the characteristics of the steel bar at and around the threaded region, leading to a reduction ductility. As mentioned before and illustrated in Figure 15b,c, fracture occurred in a rather sudden manner within or in the vicinity of the coupler.

The above observations, in conjunction with the overall characteristics of the stress-strain curves presented in Figures 4 and 7 from this study suggest that the EC2 assumptions regarding the ultimate strain characteristics at elevated temperatures are notably unconservative for the hot-rolled rebars and, more importantly, for the spliced hot-rolled rebars incorporating mechanical couplers.

5. Transient state tests

In addition to the steady-state assessments, transient elevated temperature tests were carried out on non-spliced reinforcement bars as well as rebars provided with compact and slender couplers. In the transient tests, the specimens were initially loaded to a target stress at ambient, which was maintained while the temperature was increased to failure using the same heating rate as in the steady-state cases. Transient testing is considered to be a more realistic representation of actual fire conditions. Specimens incorporating 16 and 20 mm rebars were loaded to about 50%, 70% and 95% of the yield stress, obtained from the ambient tests described in Section 3. The initial applied load was selected using the information acquired from the steady-state tests discussed previously.
The test results from transient state testing are depicted in Table 3, which include the initial stress applied in each test and the temperature at failure. On the other hand, Figure 16 provides a comparison between the results obtained from steady-state tests ($f_{sy,0}$, $f_{sp,0}$, $f_{su,0}$) and the temperatures at which failure occurred in the transient tests for a given stress state. It can be observed that for the non-spliced rebars and specimens provided with slender couplers, the results from the transient testing are closely aligned with the $f_{su,0}$ curve, whilst for the specimens with compact couplers, these are between $f_{sp,0}$ and $f_{su,0}$ curves. This is mainly attributed to the altered material properties at the cold-forged threads and the lower ductile of the coupler material. As expected, the failure temperature $\theta_f$ reduces with the increase in the applied stress $\sigma$. For example, for the non-spliced bars, a $\sigma=0.5 \times f_y$ corresponds to a $\theta_f$ in the range of 600°C, whilst for $\sigma=0.95 \times f_y$, the $\theta_f$ varies between 486-525°C. Overall, the results from the steady-state tests can be used to estimate $\theta_f$ in the transient state.

As noted above, specimens with slender couplers had similar $\theta_f$ as their non-spliced counterparts. The maximum difference was around 5% yet, on average, the $\theta_f$ ratio between the two groups was the same. On the other hand, for specimens with compact couplers the maximum difference in $\theta_f$ compared to non-spliced rebars was 8%, whilst the average $\theta_f$ ratio between non-spliced and compact splices was 0.97. As mentioned before in Section 4, the manufacturing process of the cold forged threaded rebar heads for compact couplers may have influenced the local characteristics of the steel material. Although at low temperatures (e.g. $\theta<300^\circ$C) there is a benefit from increasing the cross-section of the rebar at the coupler-to-rebar interface, the manufacturing process may have a detrimental impact at higher values of $\theta$.

Transient testing can also be readily used for determining creep strains. In contrast to steady state in which creep testing may be difficult and time consuming, transient tests are more effective in the sense that the mechanical strain component can be determined prior to heating. To obtain the creep component, the thermal expansion and additional mechanical strain occurring due to the reduction in elastic modulus due to heating can be extracted. In practice, the thermal and mechanical strains can be determined using codified procedures (e.g. EC2 [5]). To capture this in a more effective manner, the thermal elongation can be determined from a back analysis of the recorded thermal elongation and the mechanical strains can be estimated by considering the reduction in elastic modulus with increase in $\theta$ for each heating interval during the steady-state tests. The reduction factors obtained from tests were herein used to determine the thermal elongation and the reduction in $E_{s,\theta}$ with $\theta$ (Figure 10a).
Figure 17 illustrates the development of the three strain components for the 20 mm non-spliced members after the deduction of mechanical strain applied at ambient temperature. It can be observed that the creep activation temperature, which can be translated into creep activation time, depends on the level of applied stress. For an applied stress in the range of 50% of yield strength $f_{sy}$, creep seems to initiate 300°C, yet creep strains become significant above 450°C. For a stress of 0.7×$f_{sy}$, creep becomes visible around 340°C, whilst for an applied stress around 0.95×$f_{sy}$, creep strains develop above 215°C. These ranges are slightly lower than other results on steel materials reported in the literature [7], indicating that creep strain becomes noticeable at approximately 400°C at a constant stress state, and in the case in which high stress and temperature vary with time, creep strains occur even at 300°C [7].

For spliced specimens with slender couplers, the creep activation temperatures were within similar ranges to those of the non-spliced bars, with a variation of around 10%, indicating that the response was largely by the rebar characteristics rather than the coupler. Although a relatively small number of transient tests were carried out, those on specimens with compact couplers suggested a much lower creep activation temperatures that were about 50% of those shown in Figure 17 for non-spliced rebars. As illustrated by the strain maps from steady-state tests in Figure 9, the compact coupler deformed concomitantly with the rebar, hence contributed significantly to the total deformation. Under transient testing, the manufacturing process of the threads combined with the distinct coupler material properties (as shown in Figure 6b) in comparison to those of the rebar, influenced the ratio between the thermal-mechanical-creep strains in comparison with the non-spliced specimens.

The above described findings indicate that both coupling systems performed well under steady-state monotonic loading with failure developing mostly outside of the coupler region for temperatures below 400 °C, noting that the minimum requirement for S500 Class C reinforcement (i.e. $\varepsilon_{uk}>7.5\%$) in EC2 at ambient conditions was met. In contrast, for the slender and compact splices, at $\theta\geq500^\circ$C, the necking and fracture typically occurred at the coupler to rebar interfaces and exhibiting a less ductile behaviour in comparison with the non-spliced specimens. Other than that, the reduction in capacity for spliced members with the increase in $\theta$ is broadly similar to the non-spliced specimens.

Importantly, similar trends were obtained for the specimens provided with couplers as for the non-spliced specimens, in terms of relative reduction of initial stiffness and proportionality limit, indicating that the Eurocode models can be employed considering that the coupler geometry is accounted for. The comparison between the offset 0.2% proof strength and the strength at an effective strain of 2% with respect to the codified curves indicated that both are representative of
the yield strength for both splices as well as non-spliced elements. The 0.2% proof strength test reduction factors match the codified curve (Curve N-3) for elements with limited strain capacity (strongly reinforced beams, columns), whilst the reduction factors for the strength at an effective strain of 2% follows the code curve (N-1) which is associated with ductile failures of members in bending (beams and slabs).

The degradation ratios in terms of ultimate strengths for both spliced types, were within the same ranges as those for the non-spliced bars, beyond 0 around 400°C, where the ultimate strengths were similar to those at ambient conditions. On the other hand, above 400°C, the reduction in ultimate properties was proportional to temperature. Hardening was observed for all specimens being largely constant up to 300°C, diminishing between 400-500°C and transforming to strain softening at temperatures above 600°C. Key observations suggested that strain hardening may be considered for representing the response of reinforcement steel and threaded splices at elevated temperatures with exact values depending on the assumed yield strength, as long as the influence from the coupler is considered.

The manufacturing process of the rebar at the coupler region may modify the characteristics of the material, leading to modification in the strength and ductility characteristics. Although at low temperatures (e.g. 0<300°C) there is a benefit from increasing the cross-section of the rebar at the coupler-to-rebar interface, the manufacturing process may have a detrimental effect at higher temperatures, particularly for the case of compact couplers. More importantly, EC2 assumptions regarding the ultimate strain characteristics at elevated temperatures seem to be notably unconservative for the hot-rolled rebars and for the spliced hot-rolled rebars incorporating mechanical couplers.

6. Concluding remarks

The paper describes an experimental investigation into the influence of elevated temperatures on the mechanical properties of steel reinforcement with and without threaded couplers. The test specimens included 16 mm and 20 mm hot-rolled deformed bars which were tested at ambient temperature as well as under steady-state and transient elevated temperature conditions. The experimental arrangement and specimen details, and the main results and observations obtained from the tests are presented and discussed. In addition to the overall stress-strain response, particular attention was given to key mechanical parameters such as stiffness and strength, as well as hardening and ductility characteristics. The test results are also used to assess the adequacy of current design guidelines for steel reinforcement and mechanical splices at elevated temperatures.
For both non-spliced rebars and threaded splices, temperatures below 400°C do not affect significantly the stiffness and strength, while the yield plateau becomes less visible above 300°C. The reduction of elastic modulus, proportionality limit and yield strength showed a general decreasing trend as a function of temperature with EC2 reduction factors being broadly in agreement with the test results. Key test observations indicated that the geometry of the coupler influenced the initial stiffness of the splice at ambient temperature, and the proportionality limit at elevated temperatures, with larger configurations delaying the development of inelasticity.

The splices incorporating compact couplers developed similar trends as non-spliced specimens, with test reduction factors for yield strength located between the EC2 and ACI curves for temperatures below 400°C and within the ranges of predicted reduction factors beyond this temperature. For splices with slender couplers, the yield strength reduction factors were in close agreement with EC2. An analysis of a yield criterion considering the 0.2% proof strain and the effective strength at 2% strain indicated that EC2 Class N curves are representative for the rebar steel investigated in this paper. The above findings indicate that current codified models can be employed for assessing of the strength and stiffness properties of both non-spliced and mechanical splices provided with threaded couplers.

A consistent reduction in ultimate strain as a function of the coupler size was obtained in the tests, with both spliced and non-spliced rebars developing more than 7.5% strain at ambient temperature, as required for Class C reinforcement in EC2. Higher temperature reduced the ultimate strain for all specimens up to 500°C. At 600°C, the ductility was largely recovered for non-spliced bars, whilst for specimens with couplers it diminished, indicating that the EC2 assumptions with respect to the ultimate strain at elevated temperatures are notably unconservative. Splices incorporating compact couplers had a more brittle response in comparison to those with slender couplers at temperatures above 500°C, indicating a geometry and manufacturing process-dependent ultimate response.

At temperatures below 500°C, fracture occurred generally outside the coupler region for both types of couplers, whilst at 600°C the necking and fracture typically occurred at the coupler-to-rebar interfaces. The experimental stress-strain curves for spliced and non-spliced rebars showed that strain hardening diminishes between 400-500°C and transforms to strain softening at temperatures above 600°C. This indicates that below 400°C, the actual ultimate strength is higher than the yield strength, while strain hardening parameters can reach values up to 30%, suggesting that the EC2 disregard for strain hardening is unrealistic.

Transient tests carried out at stress levels of 50%, 70% and 95% of the yield strength suggested that the results from the steady-state tests can be used to estimate the failure temperature under transient
response. For non-spliced specimens and those with slender couplers, that reached similar failure temperatures, the stress-failure temperature data was aligned with the steady-state ultimate strength curve. For the splices with compact couplers, failing at marginally lower temperatures, these were between the steady-state proportionality limit and ultimate strength curves. Additionally, an analysis of the creep activation temperatures indicated that the response is governed broadly by the steel rebar characteristics, rather than the coupler. For splices with compact couplers, the initiation of creep occurred at lower temperatures, while for splices with slender couplers these were within 10% of those of their non-spliced counterparts.

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References


[21] Bompa DV, Elghazouli AY. Ductility of Reinforced Concrete Members Incorporating Mechanical Splices. 16th European Conference on Earthquake Engineering (16ECEE), ID 11604


[28] DR AS 3600:2017 - Concrete structures (draft for public consultation), Standards Australia Limited, August 2017
[29] SIA 262 – Concrete structures, 2003 by SIA Zurich
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Nomenclature

**Greek letters**

- $\varepsilon_{cr}$: creep strains
- $\varepsilon_{mech}$: mechanical strains
- $\varepsilon_{sf,\theta}$: strain at fracture
- $\varepsilon_{sp,\theta}$: strain at proportionality limit
- $\varepsilon_{st,\theta}$: strain at fracture according to Eurocode 2
- $\varepsilon_{su,\theta}$: ultimate strain or strain at zero strength according to Eurocode 2
- $\varepsilon_{sy,\theta}$: strain at yield
- $\varepsilon_{th}$: thermal strains
- $\rho_{y,i,\theta}$: strain hardening parameters
- $\sigma$: stress
- $\theta$: temperature
- $\theta_f$: failure temperature
- $\Delta H$: activation energy parameter
- $\Theta$: temperature-compensated-time parameter

**Lowercase latin letters**

- $d_b$: bar diameter
- $d_c$: coupler diameter
- $f_{sp,\theta}$: the proportional limit
- $f_{sy}$: yield strength
- $f_{sy,0.2\%}$: proof stress at 0.2% strain
- $f_{sy,1.0\%}$: effective yield strength at a total strain of 1%
- $f_{sy,2.0\%}$: effective yield strength at a total strain of 2%
- $f_{sy,\theta}$: yield strength or the maximum stress level according to Eurocode 2
- $f_{su,\theta}$: ultimate strength

**Uppercase latin letters**

- $E_{s,\theta}$: elastic modulus
- $L_c$: coupler length
- $R$: gas constant
- $T$: temperature
- $Z$: secondary creep rate
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<th>θ (°C)</th>
<th>$f_{y,0}$ (MPa)</th>
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#### Table 2a Steady state tests

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| AC-20M-TS100  | 100    | 436             | 516                 | 527                 | 532              | 557              | 635            | 6.87         | 7.58         |
| AC-20M-TS300  | 300    | 314             | 496                 | 496                 | 528              | 558              | 622            | 4.73         | 5.43         |
| AC-20M-TS400  | 400    | 273             | 431                 | 497                 | 496              | 554              | 618            | 3.17         | 3.52         |
| AC-20M-TS500  | 500    | 234             | 413                 | 470                 | 498              | -                | 523            | 1.10         | 1.16         |
| AC-20M-TS600  | 600    | 125             | 326                 | 347                 | -                | -                | 336            | 1.08         | 1.08         |
| AC-16M-TS100  | 100    | 499             | 518                 | 523                 | 526              | 553              | 628            | 5.79         | 6.42         |
| AC-16M-TS400  | 400    | 335             | 427                 | 495                 | 516              | 558              | 575            | 3.46         | 3.91         |
| AC-16M-TS600  | 600    | 145             | 203                 | 262                 | 276              | 282              | 282            | 1.96         | 0.00         |
Table 2b Steady state tests: reduction factors with respect to the ambient temperature

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<th>$f_{sy,θ}/f_{sy,20°C}$</th>
<th>$f_{sy,1.0%,θ}/f_{sy,1.0%,20°C}$</th>
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<th>$ε_{um,θ}/ε_{um,20°C}$</th>
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Table 3 Transient state tests

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