Strengthening of beams or slabs using temperature induction

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Abstract
Several aged reinforced concrete (RC) structures need to be strengthened. A typical method is to subsequently add rebar into slots filled with bonding mortar. However, this kind of strengthening only acts for live loads, not for dead loads, if no prestressing or prior lifting is applied. To overcome this deficiency, a novel method is presented. It activates the postinstalled reinforcement for all types of loading by temporarily inducing temperature from the outside. Doing so, statically indeterminate structures are locally relieved from stresses by constraint moments from systematically induced vertical temperature gradients during strengthening. To set the size of the gradients, the nonlinear material behavior, especially the significant reductions in constraint forces due to softening, is taken into account. If tempering is stopped after strengthening, stresses from dead loads redistribute over the newly formed total cross section. The method is theoretically derived, discussed, and verified in the lab on a two-span RC girder. It turns out appropriate to enhance the bending resistance of beams or slabs.

KEYWORDS
beam, constraint, gradient, nonlinear calculation, reinforced concrete, reinforcement, softening, strengthening, temperature induction

1 INTRODUCTION

The maintenance of existing structures gains increasing importance. A large share of the building stock worldwide needs to be strengthened or renewed. Especially, bridges are in a critical state, mainly due to the strong increase in traffic loads, deicing salts, or tightening in the demands of standards.1–3

Strengthening offers a sustainable alternative to new constructions. It saves resources, reduces emissions, and minimizes disturbing interventions in the environment. Cross section strengthening is done with reinforcing bars embedded into mortar or shotcrete4,5 and sheets or plates bonded to the surface.6,7 Typical materials are steel, fiber-reinforced polymers, or textile-reinforced concrete (RC) solutions that have been widely established over the last 20 years.1,8,9

But, all these techniques are just effective against live loads. They do not contribute to the bearing resistance against dead loads, if no temporary lifting of the structure or prestressing is applied. Consequently, the initial deformations of dead loads remain imprinted. Creep

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redistribution is not an issue as aged concrete tends to creep less.\textsuperscript{10} Moreover, extensive cracking that would provoke redistribution is usually excluded.

Mechanical lifting by hydraulic jacks frequently goes along with pronounced interventions and costs. The same applies to subsequent prestressing that requires specific anchorages and deviations of the tendons. Thus, both methods are seldom used for standard beams or slabs in buildings or small bridges. Especially, these structures exhibit a large share of dead load compared to the overall loads (70\%–90\% are typical\textsuperscript{11,12}). Consequently, standard solutions of strengthening become ineffective.

Figure 1 conceptually illustrates the mechanical situation of a conventional strengthening with postinstalled rebar $\Delta A_s$ placed into slots on cross-sectional level. The dead load $g$ causes initial tensile strains $\varepsilon_{s,0}$ in the initial reinforcement $A_s$. After strengthening, live loads $q$ might act, too. They increase the strains in the initial ($\varepsilon_{s,0} + \varepsilon_{s,q}$ in $A_s$) and postinstalled reinforcement ($\varepsilon_{s,q}$ in $\Delta A_s$). $A_s$ is pronouncedly stressed while $\Delta A_s$ only receives shares from $q$. Both strains significantly differ.

As a solution, a new method is developed that allows temporary stress relief by temperature induction at the location of strengthening. During strengthening, inverted temperature strains—artificially induced from the outside—superimpose load-induced ones. Once the measure is completed, temperatures are withdrawn and strains in $A_s$ and $\Delta A_s$ harmonize on a uniform level. That way, $\Delta A_s$ gets prestrained and the initial reinforcement $A_s$ is relieved from $g$. The concept works on structural level in case of statically indeterminate structures where imposed deformations activate continuity stresses. Figure 2 illustrates the method on a two-span girder. A temperature gradient $\Delta T_z$ over a length $L$ causes curvature over the cross section’s height that yields linear constraint moments $M$ (left, center). $\Delta T_z$ is steered in such a way that $M(x = L)$ at the location of strengthening (the internal support) corresponds to the inverted bending moment from dead load $g$ (left top). Thus, both actions neutralize and the cross section gets unstressed (left bottom). After strengthening, temperatures are withdrawn (right, bottom) and the initial moment from dead load returns. On cross-sectional level, $A_s$ and $\Delta A_s$ now feature the same reduced strains (section, right).

In Löschmann,\textsuperscript{13} the method has been applied to a first noncracked prototype. Now, it is extended to also cover cracked conditions by including nonlinear calculations that account for softening. This considerably increases the method’s scope of application.

The beneficial use of temperature-induced constraint forces is not yet common practice in civil engineering. But, simulations of temperature fields\textsuperscript{14–16} to determine the mechanical effects on the design or the recalculation\textsuperscript{3,17–19} have been extensively investigated. An active use or control of temperatures is so far limited to few applications, for example, to accelerate the strength development of concrete\textsuperscript{20} or to prevent freezing of bridge roadways.\textsuperscript{21} Michels\textsuperscript{22} and Strieder\textsuperscript{23} have used thermal effects in structural strengthening. Strips made of iron-based shape memory alloys are prestressed after installation by releasing previously induced plastic deformations through heating. However, relaxation and fatigue behavior\textsuperscript{22} and the effect of varying temperatures on the stress conditions\textsuperscript{23} have not been sufficiently studied yet. By contrast, in mechanical and electrical engineering, the use and control of temperature is common practice.\textsuperscript{24–26}

The paper is structured as follows: First, the fundamentals of temperature effects on structures are briefly examined. Then, active steering of constraint moments using temperature induction is presented. The focus is set on nonlinear materials such as cracked RC, which require a realistic estimation of stiffness distribution to set the temperature gradient.\textsuperscript{27,28} Finally, the method is experimentally verified on a two-span RC girder strengthened at its internal support.

\section{Steering of Moments by Temperature Induction}

\subsection{Theoretical background}

In addition to external loads, structures are exposed to internal deformations, in particular from temperature variations. If the imposed deformations are restrained, stresses occur.\textsuperscript{29} They depend on the static system\textsuperscript{30,31} and the temperature field.\textsuperscript{14} Figure 3 shows a nonlinear field $\Delta T(y,z)$ of a rectangular cross section with area $A$ and moment of inertia $I_y$. The constant portion $\Delta T_m$ of the field is the change of the mean temperature. $\Delta T_z$ quantifies the temperature difference of the linear
portion over the height $h$. Temperature differences $\Delta T_y$ over the width are usually small enough to be neglected. $\Delta T_m$ (Equation 1) and $\Delta T_z$ (Equation 2) can be determined by removing the nonlinear portion $\Delta T_e$ from the entire field $\Delta T(z)$.

$$\Delta T_m = \frac{1}{A} \int T(y,z) \, dA,$$  \hspace{1cm} (1)

$$\Delta T_z = \frac{h}{I_y} \int T(y,z) \cdot z \, dA.$$  \hspace{1cm} (2)

Since plane cross sections tend to remain plane according to Bernoulli’s hypothesis, deformations from the nonlinear temperature portion are directly restrained by adjacent cross section fibers. The resulting eigenstresses are self-equilibrating over the cross section and therefore do not cause overall deformations and section forces. However, they increase the local stress level.\(^{14,29}\)

The effect of constant and linear temperature portions depends on the static system. In statically determinate systems, deformations occur without stresses. Temperature strains are proportional to the temperature change by the coefficient of thermal expansion $\alpha_T$. $\Delta T_m$ causes constant elongation or contraction $\varepsilon_{\Delta T}$ (Equation 3), whereas $\Delta T_z$ induces curvature $\kappa_{\Delta T}$ (Equation 4).

$$\varepsilon_{\Delta T} = \alpha_T \Delta T_m, \hspace{1cm} (3)$$

$$\kappa_{\Delta T} = \frac{\alpha_T \Delta T_z}{h}. \hspace{1cm} (4)$$

In statically indeterminate systems, thermal deformations are restrained so that constraint forces arise.\(^{17}\) According to the theory of elasticity, the axial force $N$ results from the difference of the total strains $\varepsilon$ and the temperature strains $\varepsilon_{\Delta T}$ multiplied by the axial stiffness $EA$ (Equation 5). Considering Bernoulli’s assumptions of plane sections, the bending moment $M$ can be expressed by the difference of the entire curvature $\kappa$ and the temperature-induced curvature $\kappa_{\Delta T}$ multiplied with the bending stiffness $EI$ (Equation 6).

$$N = \int \sigma \, dA = \varepsilon \, EA - \frac{\alpha_T \Delta T_m \, EA}{\varepsilon_{\Delta T}}, \hspace{1cm} (5)$$

$$M = \int \sigma \cdot z \, dA = \kappa \, EI - \frac{\alpha_T \Delta T_z \, EI}{\kappa_{\Delta T}}. \hspace{1cm} (6)$$
The calculation of sectional forces from restrained deformation can be done analogue to direct loads by using force, displacement, or finite-element methods. In the force method used in the paper, the thermal deformations ($\epsilon_{\Delta T}$, $\kappa_{\Delta T}$) are directly included in the principle of virtual forces.

### 2.2 Steering method

Steering of sectional forces is performed by inducing temperatures over a predefined length $L_i$ on a statically indeterminate structure (cp. Figure 4, top). The paper focuses on nonlinear material behavior but also includes linear ones. The former implies softening that affects the induced sectional forces. The study restricts on steering of bending moments so that temperature-induced constraint moments $M_i$ are introduced. Analogously, the method can be transferred to axial and shear forces.

$M_i$ depends on

- the vertical temperature difference ($\Delta T_z$),
- the length ($L_i$) and the position of temperature induction,
- the static system,
- the bending stiffness ($EI$) and
- the material’s coefficient of thermal expansion ($\alpha_T$).

#### 2.2.1 Linear-elastic material behavior

In case of linear elasticity, analytical formulas can be derived to calculate $M_i$ (cp. Section 2.1). Below, this is done for beams clamped at both ends. The procedure can be transferred to multispan beams, which correspond to clamped beams with a reduced degree of clamping (Figure 4). The rotational springs $c_\phi$ at the supports depend on the bending stiffness and the length of the adjacent spans. $c_\phi = 0$ refers to free rotations, and $c_\phi \to \infty$ refers to full clamping. For $c_\phi \to \infty$ and a gradient symmetrically positioned at mid-span, $M_i$ is constant (Figure 4, blue lines) and corresponds to $\kappa_{\Delta T}$ multiplied by $EI$ and the relation of $L_i$ to the span length $L$ (Equation 7). Lower degrees of clamping reduce the moment.

$$M_i = \kappa_{\Delta T} \cdot \frac{L_i}{L} EL = \frac{\alpha_T \Delta T_z \cdot a}{h} \cdot \frac{L_i}{L} EL. \quad (7)$$

If $\Delta T_i$ is asymmetrically positioned, $M_i(x)$ becomes linear (red lines). It depends on $L_i$ and its location defined by the distance $a$ between induction and the support (Equation 8). The maximum of $M_i$ is obtained when $\Delta T_i$ is acting over two thirds of $L$.

$$M_i(x) = \frac{\alpha_T \Delta T_z \cdot a}{h} \cdot \frac{L_i}{L^3} \cdot (6\alpha (2a + L_i - L) + L (\alpha^2 - 3L_i + 4L)) EI. \quad (8)$$

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**Figure 4** Bending moments from vertical temperature gradients in single-span and multispan girders
In multispan beams, $M_i(x)$ continues with decreasing amplitudes in the following spans and approaches zero at the last support. Depending on the degree of clamping, $M_i(x)$ is alternating or one-sided. By superimposing constraint moments from several gradients, effects in other spans can be completely eliminated so that ideal triangular and trapezoidal moment shapes occur.\

2.2.2 | Nonlinear material behavior\

Structures with softening materials like concrete require realistic evaluations of stiffness distributions, as constraint forces significantly decrease by softening.\(^{17,29}\) Moments are still linear but reduced in their extents.

In the paper, moment–curvature relations ($M$-$\kappa_m$) that incorporate tension stiffening integrate the nonlinearity of cracked RC on cross-sectional level. Doing so, the stress–strain relation of the reinforcement is enriched by concrete contributions and smeared to an average strain $\varepsilon_{sm}$ (Figure 5, top). The standard formulations of Model Code\(^{27,33,34}\) are used here that distinguish the enriched stress–strain relation into four linear branches of a noncracked, cracked, stabilized cracked, and (plastic) yielding situation (Figure 5, top). Moreover, a simplified approach that does not differentiate between the two cracking situations and exhibits a straight path until yielding (dashed line) is applied. This simplification is suitable for repeated loading and unloading.\(^{35,36}\) For the concrete, a linear constitutive law is assumed for compression as well as tension until the tensile strength is reached. The postcracking tensile strength is set to zero in the crack.

The average curvature $\kappa_m$ of a cross section results from the strain plane defined by $\varepsilon_{sm}$, the effective depth $d$, and the average height of the compression zone $x_m$\(^{10}\)

$$\kappa_m = \frac{\varepsilon_{sm}}{d-x_m} = \frac{M}{E_{\text{eff}}} \quad (9)$$

where $E_{\text{eff}}$ denotes the effective bending stiffness that decreases with rising moment.

On the structural level, the beams or slabs are discretized into 20–30 elements per span with higher element densities at the internal support. The numerical calculation is performed in an iterative way (Figure 5, bottom) using linear predictions and incremental loading (up to five load steps) until a sufficient accuracy is achieved. For more details, the reader is referred to Heek.\(^{27}\)

2.3 | Practical realization\

2.3.1 | Temperature induction\

The realization of induction includes the choice and setting of a tempering system, a direct thermal transfer between the system and the concrete surface to minimize thermal losses, the fixing to the RC structure, as well as a control unit to steer the flow rate of the thermal input. Moreover, the whole induction unit should be thermally isolated. Clauß\(^{37}\) develops different methods and discusses them in detail. For the sake of brevity, only short descriptions are given here while the reader is referred to the reference for detailed information.

Linear vertical profiles are obtained by tempering the top and the bottom of a member. Relative to the
member’s initial temperature (≈ 20°C), heating yields higher and cooling lower temperature. Suitable heating systems are tempered water (20°C–100°C), heating mats (<200°C), or infrared heaters (<400°C). Here, heating mats are employed as they are easy to handle and ensure direct surface contact with negligible thermal losses in conduction. Cooling can be achieved by cooling liquids like tempered water (0°C–20°C) or with Peltier elements (> – 10°C). In this study, chillers combined with water circuits are used and coupled via thermal pads to the concrete. The latter allow free relative deformations as well as a sound thermal transfer. Figure 6 gives an overview of the alternative methods.

2.3.2 | Induced temperature field

The induced temperature fields are three-dimensional. This holds true even for the stationary state that is treated in the following. The fields exhibit neither an ideal induction length \( L_i \) nor pure linear vertical temperature profiles as assumed in the beam calculations (Figure 7). Thus, \( M_i \) results from integrating individual temperatures over the cross section height \( h \) and the overall span length \( L \). Variations over the width are usually small enough to be neglected (cf. Section 2.1).

For an application of the method according to Section 2.2, it is recommended to transfer the actual temperature field into a simplified one that activates the same extent of \( M_i \). Doing so, an effective temperature gradient \( \Delta T_{\text{eff}} \) is introduced that constantly acts over \( L_i \) and vanishes in front and behind (Figure 7, bottom). Moreover, the variation of \( T \) over the cross section’s height is linearized to \( \Delta T_z \). The effective value then reads:

\[
\Delta T_{\text{eff}} = \frac{\int_0^L \Delta T_z(x) \, dx}{L_i}.
\]  

(10)

In practice, temperature fields should either be measured in grids and interpolated in between (measuring campaign) or estimated upon single measuring points that are enriched by calculated stationary temperature fields (hybrid approach). Here, a measuring campaign is used employing fiber-optic sensing (FOS). FOS allows a quasi-
continuous measurement—every millimeter—along the fiber. Fibers are placed along the longitudinal direction to gain a fine discretization of temperature data in the $x$ direction (“structural level”). Figure 7 shows the concept with two fibers FOST$_{T1}$ and FOST$_{T2}$. On the sectional level, two single temperatures are recorded for each

**FIGURE 7** Nonlinear temperature field from bilateral induction with discrete measuring points and linearized vertical courses (top) and interpolation on the structural level (bottom). FOS, Fiber-optic sensing

**FIGURE 8** Static system, sketch, and picture of the test setup with drawings of two cross sections, all lengths in (mm)
sectional “slice” and are linearly combined to approximately determine $\Delta T_z$.

3 | EXPERIMENTS

3.1 | Test objective

The method of temperature induction is experimentally verified on the example of the two-span RC girder (Figure 2). After strengthening, the same reduced stress level under dead load should prevail in the initial ($A_s$) and the postinstalled reinforcement ($\Delta A_s$), respectively. Moreover, load–deformation and cracking behavior until ultimate loading should correspond to that of a beam that is built with $A_s + \Delta A_s$ right from the beginning.

3.2 | Test setup

The two-span girder has a total length of 5.20 m and span lengths of 2.50 m (Figure 8). The supports consist of steel half-cylinders that freely rotate in half shells. The two outer sliding bearings have greased layers of polytetrafluoroethylene. The girder has a rectangular cross section ($b/h = 0.25/0.16$ [m]) and is made of concrete C20/25 with a maximum grain size of 16 mm. The material parameters are determined from small-scale tests according to DIN EN 12390 and summarized in Table 1 in their average values (number of samples: 3) together with the geometry, reinforcement, and loads. The compressive strength was determined on cubes (150 $\times$ 150 $\times$ 150 [mm]) while the tensile strength is derived from the splitting tensile strength tested on cylinders ($d = 300$, $h = 150$ mm). Young’s modulus was determined on cylinders, too. The coefficient of thermal expansion ($\alpha_T$) is assumed to $10 \times 10^{-6}$ 1/K according to EN 1991-1-5. $A_s$ at the internal support amounts to 2Ø8 mm (1.01 cm$^2$). $\Delta A_s$ is the same and placed in a slot at center of 1.20 m length next to the two tempering units surrounded by polystyrene. The bottom of the section is reinforced by 2Ø10 mm (1.57 cm$^2$) that continuously run over the total length. Generally, bending dominates and a ductile bending failure is expected. Thus, no stirrups are provided.

The self-weight of the girder, the two heating installations, as well as four point loads $F = 3.3$ kN from steel platens act during strengthening and ensure an initially cracked state. The temperature difference is induced in both spans on $L_i = 1.00$ m by silicone heating mats (top, electrical power: 3 kW) and circulating cooled water in steel tanks (bottom, 9 kW).

The beam is equipped with 14 strain gauges ($S_1$–$S_{14}$) and three fiber optic sensors ($FOS_1$–$FOS_3$) glued on $A_s$ and $\Delta A_s$ for point-wise and quasi-continuous strain measurements, respectively, in the relevant zones (Figure 9, left). Load cells (AST, KAF 100) at the supports record the reaction forces in the final load test. Displacement sensors (Messotron, WT 100) measure the deflections at mid-span.

For temperature evaluations, the beam contains six thermocouples ($T_1$, $T_3$–$T_6$, and $T_8$) for the vertical gradient in the center of one tempering zone, two thermocouples ($T_2$ and $T_7$) near one lateral surface, and one fiber optic sensor ($FOS_T$) along the beam axis. The fiber-optic measuring system ODIsi 6108 (Luna) generally measures frequency shifts of the introduced light, which result from both strain and temperature changes. For temperature

| TABLE 1 Parameters of the test setup and the specimen |
|-----------------------------------------------|
| Parameter | Symbol | Value | Unit       |
| Geometry  | $L$    | 2.50  | m          |
| Induction length | $L_i$ | $2 \times 1.00$ | m |
| Height | $h$   | 0.16  | cm         |
| Width | $b$   | 0.25  | cm         |
| Concrete | Compressive strength | $f_{cm}$ | 31.3 | N/mm$^2$ |
|          | Tensile strength | $f_{ctm}$ | 2.3 | N/mm$^2$ |
|          | Young’s modulus | $E_{cm}$ | 30,633 | kN/m$^2$ |
| Reinforcing steel | Yield strength | $f_{ym}$ | 543 | N/mm$^2$ |
|          | Young’s modulus | $E_{sm}$ | 201,007 | kN/m$^2$ |
| Reinforcement | Initial reinforcement | $A_s$ | 1.01 | cm$^2$ |
|          | Supplementary reinforcement | $\Delta A_s$ | 1.01 | cm$^2$ |
| Loads | Self-weight | $g$ | 0.96 | kN/m |
|          | Point load | $F$ | 3.3 | kN |
measurements, the fiber is decoupled from the component by guiding it in a capillary made of the polymer polyether ether ketone (Figure 9b). Figure 9c shows the equipped formwork (Section C-C). For the evaluation of ΔT_{eff} according to Equation 10, ΔT_z is estimated from the thermocouples T_3–T_6 and proportionally transferred to each measuring point along FOST.

3.3 Test procedure and precalculation of temperatures

The testing follows the procedure illustrated in Figure 2 with a final examination of the achieved bearing capacities. It comprises four steps, namely:

1. Initial state: Dead loads from the beam, steel platens, and the tempering equipment act. The beam contains only A_s and an empty slot for ΔA_s.
2. Induction and strengthening: Induction starts and when the intended stationary temperature state of ΔT_{eff} is reached, strengthening (ΔA_s) is applied, the mortar is added and hardens (1 day). For practical application, it is recommended to use shrinkage-free mortar to avoid redistribution and losses in strain in the postinstalled reinforcement.
3. Acclimatization: Tempering is stopped, and all artificial temperatures vanish.
4. Load test: Tempering equipment and steel platens are dismantled and the beam is loaded until failure. Loading P is applied by a hydraulic test cylinder which is path-controlled. A steel crossbeam distributes P in 2P/2 that act at mid-span.

The first three steps summarize the strengthening. Figure 10 shows selected pictures of the procedure in the lab: Filling of the slot with mortar (a,b) and load test with loading equipment (c).

Following the procedure, required temperature differences and associated bending moments are precalculated according to the theory derived in Section 2. Doing so, a distinction is made between linear-elastic material behavior (cf. Section 2.2.1) and a nonlinear one using M-κ relations (cf. Section 2.2.2). The results for the four states are listed in Table 2.

As intended, cracking already occurs under dead loads. The moment at the internal support—estimated to −3.1 kNm by the linear theory—exceeds the cracking moment (M_cr = −2.4 kNm) and thus decreases by 16% to −2.6 kNm if softening is taken into account.

The impact of softening on the temperature demand is even more pronounced.
\[ \Delta T_z = \frac{M_i(L)}{L} \cdot \frac{h}{\alpha_T} \cdot \frac{4L^3}{3L(2a + L_i)} \cdot \frac{1}{EI}. \]  

(11)

\( \Delta T_z \) amounts as given above (Equation 11) in the linear-elastic case with the parameter definition of Section 2.2 and \( M_i(L) \) being the constraint moment at the location \( L \), so at the internal support, \( \Delta T_z \) yields 34°C. This moderate temperature difference can be achieved through balanced heating and cooling. The concept of balancing means that the temperature at top is increased half the difference of \( \Delta T_z \) while at bottom, it is lowered to the same extent. For example, with an initial temperature of 25°C, the top surface needs to be heated to \( 25 + \frac{34}{2} = 42°C \) while the bottom surface needs to be cooled down to \( 25 - \frac{34}{2} = 8°C \).

Under cracked conditions, \( \Delta T_z \) rises to 69°C (+103%). Now, either a cooling system that enables temperatures below the freezing point (0°C) needs to be introduced for balanced heating and cooling (25 - \( \frac{69}{2} = -9.5°C \)) or a change of the mean temperature has to be accepted. This gives rise to longitudinal dilatations due to the arising constant portion \( \Delta T_m \).

Two aspects should be noted: First, due to inevitable losses, the temperature of the cooling liquid and the heating mat both need to exceed the advised surface temperatures to reach \( \Delta T_z \) safely. Second, in longitudinal direction, the temperature field fades out beyond the tempered area (Figure 7). It lets the effective temperature difference rise to \( \Delta T_{eff} \).

The theoretical bending resistance of the strengthened cross section is estimated from the yielding force in the reinforcement (Table 1) factored by the lever arm of inner forces \( z \) and yields \( M_y \approx -f_{ym}(A_s + \Delta A_s) \cdot \frac{z}{54.3(1.01 + 1.01)0.125} = -13.7 \text{ kNm} \).

### 3.4 Test results

#### 3.4.1 Strengthening process

Figure 11 shows the three steps of strengthening over time from the initial state (left) over the induction period (0 till about 32 h) to the acclimatization after tempering is stopped (> 35 h, right). The figure displays temperatures (at top) and steel strains in \( A_s \) and \( \Delta A_s \) (at bottom).

Starting from laboratory conditions of about 20°C, a stationary temperature field is reached after about 8 h of induction. The heating in the mat (dotted red line) achieves 80°C—so does almost the upper concrete surface (dashed red line, where the volatility is caused by the control unit)—while the concrete surface at bottom remains at about 20°C (dashed blue line). Although water cooling of 5°C is provided (dotted blue line), the heating impact from the top dominates cooling. It increases the mean temperature of the cross section by \( \Delta T_m \) and comes along with longitudinal dilatation. This happens without constraints due to the sliding bearings.
The gradient over \( h \) is linearly extrapolated from the inner thermocouples \( T_3 \) and \( T_6 \). The difference between them, denoted \( \Delta T_{3-6} \), amounts to 33°C (cp. \( \Delta T_{3-6} \) in Figure 11), so \( \Delta T_z \) results in \( 33 \cdot 160/120 = 44°C \). Figure 12 (left) shows this linearization and the resulting value of \( \Delta T_z \) between the outer surfaces as well as the constant portion \( \Delta T_m = 23°C \). Finally, all temperatures return to 20°C during acclimatization (> 48 h).

The initial strain \( \varepsilon_{s,0} \) in \( A_s \) amounts to 0.75‰. It is derived from the peak value of the sensor FOS2 close to the support and is slightly below the expected value in the cracked state. It amounts to \( |M|/(z \cdot A_s \cdot E_s) = 2.4/(0.125 \cdot 1.01 \cdot 201,007) = 0.9% \) considering a round out of bending moments from \(-2.6 \) to \(-2.4\) kNm over the actual bearing length of 0.15 m but neglecting tension stiffening effects. The single strain gauge \( S_3 \) records an even lower value of 0.50‰. During induction, \( \varepsilon_s \) decreases almost proportionally to the rising temperature gradient having a minimum of 0.09‰ and a slight increase caused by delayed microcracking and slight creep redistributions to around 0.20‰ when strengthening. Consequently, a full decrease is not reached, but a small residual strain remains in \( A_s \). This value remains as a difference between the strains in \( A_s \) and \( \Delta A_s \) after acclimatization. Thus, the strains in \( A_s \) and \( \Delta A_s \) in red or green and light red or green, respectively, run parallel with a distance of about 0.20‰. The actual strains read 0.64‰ (\( A_s \)) and 0.44‰ (\( \Delta A_s \)). The bottom reinforcement is located close to the neutral axis of the cross section and thus exhibits low strains (blue lines).

**FIGURE 11** Temperature (top) and strain developments (bottom) during the initial and induction state and the acclimatization. FOS, Fiber-optic sensing

**FIGURE 12** Vertical temperature profile interpolated from discrete measurements by thermocouples (left) and longitudinal path from a fiber-optic sensor with derived vertical temperature differences (right). FOS, Fiber-optic sensing
The effective temperature difference $\Delta T_{\text{eff}}$ according to Section 2.3.2 is evaluated using the longitudinal profile measured by FOST. Doing so, $\Delta T_z$ is proportionally transferred to each measuring point taking the ratio of estimated $\Delta T_z$ (44°C) and measured values $T = 56°C$ (Figure 12, left) as the reference of proportionality. Any inaccuracies due to nonlinear gradients are small and roughly balance out. $\Delta T_z$ is linearized along the beam to simplify the calculation of $\Delta T_{\text{eff}}$. The linear part runs over 395 mm and enters the induction zone at 140 mm. Thus, the constant part ($\Delta T_z = 44°C$) has a length of 1000 – 2 · 140 = 720 mm and $\Delta T_{\text{eff}}$ is calculated according to Equation 10 to 49°C.

3.4.2 | Load test

Figure 13 (left) presents the load–deflection response of the beam during the load test. It shows a typical ductile behavior with a pronounced yielding plateau at $P \approx 65$ kN. First cracks occur at $P \approx 15$ kN in the mortar around $A_{s}$. The ratio of reinforcement at sagging (1.57 cm²) and the internal support ($A_{s} + \Delta A_{s} = 2.02$ cm²) of 0.78 almost comply with the ratio of the corresponding bending moments at those positions (0.83). Thus, only moderate redistributions occur in the course of ongoing yielding. Finally, the test is stopped after pronounced cracks have formed at mid-span as well as at the internal support.

The strains in $A_{s}$ and $\Delta A_{s}$, measured by the two fibers FOS$_2$ and FOS$_3$, respectively, run almost parallel over the full loading process. Moreover, they more and more align. Figure 13 (right) illustrates this by the strain developments over $w$. Both strains start from their initial values around 0.5‰ induced by the strengthening (cf. Figure 11) and rise up to about 3‰ even beyond the yield strain at 2.7‰.

4 | DISCUSSION

4.1 | Strengthening process

During strengthening, almost stationary and linear vertical temperature profiles are generated if heating from the top and cooling from the bottom are combined and steered from control units and the areas of induction are thermally insulated. The energy consumed during operation of the temperature induction systems (35 h, cp. Figure 11) amounts to approx. 320 kWh or associated costs of about 100 €. Simple linear extrapolation of the energy costs to a real structure yields 1000 €. This complies to an induction length of 10 m and a rectangular cross section of 1.60 × 2.50 m and might serve as a rough estimate.

Three essential aspects should be noted for temperature induction. First and as expected, the actually induced temperatures within the concrete fall below those set in the control units caused by losses. This is due to the location of the sensors at a certain distance to the surface. Temperature control works much better if the sensors are placed within the concrete that means embedded into boreholes. As a rule, distances as well as thermal transports between the sensor of the control unit and the concrete should be as small as possible. Second, constant temperature portions $\Delta T_m$ almost inevitably occur, as balanced heating and cooling is hardly possible when high temperature differences shall be induced. This gives rise to longitudinal dilation. Third, the temperature fields fade out beyond the tempering zone. This enlarges the effective temperature difference. Here, $\Delta T_z$ is increased by 5°C to $\Delta T_{\text{eff}}$, which corresponds to a gain of 11%.

The induced $\Delta T_{\text{eff}}$ of 49°C is lower than the calculated temperature demand of 69°C for the cracked state. Tempering was steered by the strains in $A_{s}$ tolerating a small residual of 0.09‰. The same applies to further strain increase by delayed microcracking and creep, as they
hardly occur at elder structures. Besides the residual strain, the coefficient of thermal expansion taken from the standard could contribute to the deviation between testing and recalculation. The coefficient depends on the concrete mixture and should be accurately determined by testing for further investigation.

Figure 14 demonstrates the pronounced impact of softening on strains. For this purpose, strains in $A_s$ are plotted over $\Delta T_{eff}$ considering linear-elastic (left) or nonlinear (right) material behavior in theory (dotted blue, $\varepsilon_{s,calc}$) and experimental data ($\varepsilon_s$) obtained from FOS2 at the internal support (red). With initiation of induction, $\Delta T_{eff}$ starts from 0°C. The experimental strains decrease almost linearly. After about 30°C, inclination is reduced due to stiffness losses caused by cracking. The linear calculation (left) overestimates the initial strains and linearly decreases to cross the $x$-axis at 34°C (cf. Table 2). The nonlinear calculation (right) runs almost parallel to the experimental strains and underlines the necessity to consider softening in the design of a tempering system. $\varepsilon_{s,calc}$ slightly exceeds the measured strains as round outs of the bending moments are not included.

Table 3 summarizes the strains in $A_s$ and $\Delta A_s$ as expected by calculation as well as actually measured by FOS2 in the test. Expected strains amount to $\varepsilon_{s,calc} = 0.75/2 = 0.38\%$ (half of the initial strains $\varepsilon_{s,0}$), if the reinforcement is doubled and strains evenly distributed to the two reinforcement shares $A_s$ and $\Delta A_s$ (cp. Figure 11). Testing has yielded $0.50\%$ and $0.36\%$, respectively. The distribution is thus slightly imbalanced with a small plus on the side of the initial reinforcement. It should be noted that the strain portions from delayed microcracking and creep have been subtracted for the abovementioned reasons. These reductions have yielded $0.14\%$ for $A_s$ and $0.09\%$ for $\Delta A_s$.

4.2 Load test

The load–deformation behavior of the strengthened beam is analyzed on cross-sectional level using the moment–curvature relation. Figure 15 shows different relations for the cross section at the internal support. The diagram displays the experimental data (dashed red line) and three theoretical idealizations (solid lines) for a cross section

- with $A_s$ and no strengthening (Reference 1, blue), which characterizes a lower bound;
- with $A_s + \Delta A_s$ right from the beginning, without further strengthening (Reference 2, green line). It defines the upper bound and the aim for strengthening with temperature induction; and
- with $A_s$, employing standard strengthening with $\Delta A_s$ when dead loads act, so starting from $M = -2.6$ kNm (red line).

| Reinforcement | Initial state $\varepsilon_{s,0} (\%)$ | After acclimatization $\varepsilon_{s,calc} (\%)$ | $\varepsilon_s (\%)$ |
|---------------|----------------------------------------|-----------------------------------------------|------------------|
| $A_s$         | 0.75                                   | 0.38                                          | 0.50             |
| $\Delta A_s$  | —                                      | 0.38                                          | 0.36             |

FIGURE 14 Comparison of the experimental strain course to those from linear-elastic (left) and nonlinear calculations (right) as a function of $\Delta T_{eff}$

FIGURE 15 Theoretical and experimental moment–curvature relations of cross sections with different reinforcements, partly strengthened with or without temperature induction
The relations are determined with the parameters from Table 1 and the procedure outlined in Section 2.2.2 using the simplified (straight) approach as the test beam was already preloaded and cracked before strengthening. For comparison, branches assuming detailed cracking conditions are added in dotted lines.

For the experimental relation, \( M \) is derived from the reaction forces and the cylinder load measured by load cells and \( \kappa \) from strain gauges at the internal support.

As expected, all strengthened alternatives with \( A_s + \Delta A_s \) reach the yield limit at \( M = -13.7 \text{kNm} \). In contrast, Reference 1 with half of the reinforcement (blue line) yields at about \(-13.7/2 = -6.9 \text{kNm} \). The strengthened alternatives differ in their paths in the cracked state. The standard version of strengthening (red line) initially exhibits the steepest response and follows Reference 1. It kinks after strengthening at \( M = 2.6 \text{kN} \) and rises with higher stiffness until yielding of \( A_s (-11 \text{kNm}) \). Here, it kinks again and finally reaches the top plateau with an additional yielding of \( \Delta A_s \). Pronounced cracking, redistributions, and deflections arise. Reference 2 (green line) shows the steepest behavior with two distinct kinks at the initiation of cracking and with joint yielding of \( A_s + \Delta A_s \). Its behavior is almost met by the strengthening with temperature induction given by the experimental data (dashed red). The latter only marginally deviates with slightly softer branches in the cracked state as well as round outs at the onset of cracking and yielding, where \( A_s \) slightly anticipates \( \Delta A_s \) regarding the strains. Thus, yielding partly occurs prior to Reference 2 and deflections increase to a minor extent.

5 | CONCLUSIONS

The paper presents a strengthening method for RC structures that activates postinstalled reinforcements already for the dead load. For this purpose, constraint moments are systematically induced by temperature to locally neutralize bending moments from the dead load during strengthening. The method applies to statically indeterminate beams or slabs and can be used, for example, for bridges and buildings. It is experimentally verified on a two-span RC girder. Strengthening with respect to the bending resistance is provided.

The following conclusions are drawn:

- Systematic temperature induction enables to modify bending moments in statically indeterminate structures. By changing the cross section during induction, the existing stress levels in the initial and the new part of a section are imprinted. This allows postinstalled reinforcement to be prestressed and activated for the dead load without severe cracking.
- The prior activation of the reinforcement positively affects serviceability and durability as deflections and crack widths decrease. Strengthening becomes significantly more effective compared to standard solutions, especially as dead load usually dominates live loads.
- Suitable tempering systems are silicone heating mats and chillers combined with water circuits. Stationary and linear vertical temperature profiles are obtained by induction from the top and the bottom of a cross section and a surrounding thermal insulation.
- The induced constraint moments strongly depend on the bending stiffness of the structure. They can be predicted by nonlinear calculations based on moment-curvature relations that account for softening and tension stiffening effects.
- Neutralization of the negative bending moment at the internal support by induced constraint moments usually causes increasing sagging moments. This correlation must be carefully taken into account in practical application.

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DATA AVAILABILITY STATEMENT

The data that support the findings of this study are available from the corresponding author upon reasonable request.

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