

This is Chapter 4 “Some characteristics of shotcrete” from my PhD thesis

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Enjoy reading.

Best regards,
Alexander Kluckner



Dipl.-Ing. Alexander Kluckner, BSc

**Tunnelling at greater depths:
Study on the ground and system behaviour
when passing a stiff rock block in a weak zone**

DOCTORAL THESIS

to achieve the university degree of
Doktor der technischen Wissenschaften

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Chapter 4

Some characteristics of shotcrete

Concrete (or shotcrete = sprayed concrete) is a cement-based construction material. Its constituents (i.e., cement, aggregates, water, admixtures, and additions) control the properties of the hardening and hardened concrete and are mixed in a way that the requirements are met ([68, p. 395]). The fundamental difference between conventional pre-cast concrete and shotcrete is the moment of initial loading ([129, p. B-5]; exception: loading due to self-weight). In tunnelling, shotcrete usually is loaded externally the moment it is applied to the rock mass.

Sections following summarise some specifics of concrete and shotcrete important when dealing with such material in conventional tunnelling. The improvement of shotcrete—by using, e.g., new cements, additives, and mixing ratios—is a hot topic in research and industry (cf. [208, p. 380]). The need to reduce CO₂ emissions triggers some development. Thus, approaches and related settings used in the past to model shotcrete mathematically may not be suitable anymore for today’s shotcrete boosted with chemicals. The information presented below is not complete. It rather shows basic concepts. To model the shotcrete behaviour accurately, there is no way around to perform tests on the material to be used. Also, the boundary conditions (e.g., lining thickness, ambient temperature) are not to be ignored. Using settings of other shotcrete mixes and other projects, it is difficult (or impossible) to estimate the difference in the modelled behaviour and relating consequences.

Besides the fact that design standards for reinforced concrete do not apply directly to shotcrete,¹ in particular if the shotcrete is young² (cf. [208, p. 369f], [209] in [388, p. 140]), the same recommendations need to be taken seriously: if curing conditions at the site (significantly) deviate from those at standard tests, or from those the design standards and guidelines apply to, investigations and tests need to consider them (cf., e.g., [68, p. 416] referring to the strength development of concrete, and [10, p. C-41] referring to creep tests). Then, and also when comparing or analysing published test data (is the comparability given?), several aspects decide upon the results: e.g., sealed or unsealed conditions, humidity, temperature, sample size and shape, and loading rate (mechanical or thermal load). The aspects are similar to when rock is tested (cf. [199, p. 14ff]). For example, sustained too low or too high temperatures during the curing of concrete can affect its strength and stiffness development negatively ([42, p. 37], [69, 70] in [388, p. 21, 144]). But not only the environmental conditions are decisive. The characteristics and share of each concrete’s constituent are too. For example, the relations between the water-cement ratio and the compressive strength are not unique, depend in particular

¹Shotcrete is not a special concrete. It is still concrete but with particular characteristics and installed differently than cast-in-place concrete. Concrete technological principles are the same for both concrete and shotcrete. However, they need to be adapted. (cf. [129, p. B-31f], [208, p. 305])

²[289, p. 6, 26] uses the term *young* to refer to shotcrete at an age of up to 24 hours.

on the type of cement and aggregates, and require specific tests on samples using mixes intended for the construction site ([68, p. 401]; cf. also [388, p. 142] referring to the calibration of creep models). Now, if the settings of tests and conditions in the laboratory or at the construction site are outside the scope of design standards and guidelines, equations and other suggestions cited therein are probably not applicable anymore. The same applies to model fittings. They usually relate to one particular concrete or setting only.

In analytical or numerical calculations, empirical approximations account, for example, for the development of individual strain components and strengths. Either the formulations are approximations, or the input parameters, or both. Some of the following sections briefly describe which formulations five well-known models utilise to simulate the behaviour of shotcrete. That are:^{3,4}

- Schubert (1988) [358]: He uses the analytical rate of flow method by [107]. It's a one-dimensional consideration;
- Schädlich and Schweiger (2014) [348]: They formulate a three-dimensional constitutive model within the framework of elastoplastic strain hardening/softening plasticity. [349] report on the application of the model;
- Schütz et al. (2011) [368]: They, too, formulate a three-dimensional constitutive model within the framework of elastoplastic strain hardening/softening plasticity. For more details on the model, refer to [369];
- Meschke (1996) [260]: He constitutes a three-dimensional multisurface model based on viscoplasticity considering also hardening and softening mechanisms. Note that the follow-up paper by Meschke et al. (1996; [261]) comprises the calibration of the model;
- Neuner et al. (2017) [276]: Their three-dimensional model bases on the damage plasticity model for concrete proposed by [134]. They've modified it to allow for the simulation of shotcrete behaviour. It also accounts for hardening and softening mechanisms. For corrections, correlations, and applications, refer to [275, 277, 278].

Note that [276, 348, 368] use old data for validation (referring to the year of the publication describing the individual model);⁵ namely from [10, 71, 129, 173, 265, 271, 371, 387]. Probably science lacks of published validation data. Anyway, it must be assumed that the latest shotcrete types differ in the behaviour. A general validity of the fittings is not guaranteed (cf., e.g., [388, p. 131] referring to fittings of creep data, [208, p. 352] referring to the transfer of results from one construction site to another, [10, p. D-9] and [42, p. 34] both referring to the temporal development of strength). Making it more difficult and stated already above, even if the shotcrete to be installed at the construction site is tested, conditions in the laboratory can differ (significantly) from those at the site in magnitude and duration (cf. [352, p. 22] referring to the temporal development of strength).

Recent PhD theses dealing with shotcrete are, for example, [57, 169].

³This list does not intend to be exhaustive regarding concrete or shotcrete models. The list and descriptions in the following sections shall only summarise how other selected researchers have proceeded in the past.

⁴Note that only in this chapter some references from the following list are cited in a different style (author-year citation) to ease the differentiation for the reader.

⁵In a follow-up paper, Neuner et al. (2017; [275]) present results from recently performed tests on shotcrete samples from the Brenner Base Tunnel project. This paper validates the model introduced in [276] anew.

4.1 Hardening of concrete

The hardening concrete comprises aggregates, unhydrated cement and hydrates (summarised as the solid skeleton), and fluid-saturated pores ([155, p. 692])⁶. Hydrates are the products of the physical and/or chemical combination of free water and unhydrated cement. The water is then combined in the solid phase (= hardened cement gel) and is nonevaporable. The term *ageing* relates to this process ([260, p. 3123]). [401, p. 785ff]

The moment the constituents are mixed, a chemical imbalance between water, cement, and hydrates (as soon as some have formed) exists. This difference in chemical potential (also termed *affinity*) causes the chemical reaction to initiate and to proceed ([155, p. 693f]) until a balanced state is reached. This process is termed *cement hydration*⁷. The initial imbalance depends on the initial temperature and the design of the mixture (e.g., cement type, water-cement ratio) ([401, p. 790], [68, p. 62]), and a minimum amount is required to activate the hydraulic reactions in the first place (i.e., minimum activation energy required; [300, p. 912]). Here, the hydration heat amplifies the activation (thermal activation; cf. [401, p. 787])

Hydrates starts to grow at the contact surface between the unhydrated cement and the free water (not chemically bounded yet). With some hydrates having formed, it takes longer for the remaining free water to diffuse (because of thermodynamic imbalance between free water and water bound in hydrates) to the remaining unhydrated cement through the hydrates already formed ([401, p. 787f]; cf. Fig. 5.2 on p. 71). Therefore, the mass of hydrates already formed determines the evolution (i.e., rate) of hydration ([158, p. 2104]). The *hydration degree* is the ratio of current to final hydration extent and is linearly proportional to the mass (or volume) of hydrates formed ([7] in [302, p. 230], [208, p. 368], [314] in [401, p. 790]). It varies between 0 and 1.

4.2 Origin of strength and stiffness growth

Primarily, strength (e.g., compressive and tensile strength) increases because the share of solids in mass per unit volume increases with ongoing hydration. The increase in strength is proportional—at early-age concrete quasi-linear ([263] in [155, p. 693])—to the amount of formed solid hydration products ([370, p. 286]). The increase in the amount of strong, stiff solids, the decrease in the amount of weak, soft fluid-saturated spaces, and the creation of bonds between adjacent solids is also the origin of the increase in stiffness (cf. [370, p. 284]).

However, growth in strength and stiffness, both of which are usually desired, is accompanied by an undesired reduction of the elastic strain limit (i.e., damage threshold; [402, p. 1123]) and ductility⁸ ([348, p. 103]). This is because spaces allowing for movement of fluids in case the shotcrete is loaded continuously are replaced by stiff brittle solids.

Secondarily, the consumption of free water during cement hydration causes the capillary pressure between the liquid and the gaseous phase saturating the pores to increase, which also stiffens the compound ([401, p. 788]).

The phenomenon described here is termed *chemical hardening* ([402, p. 1125f]). It is not to be confused with *work* or *strain hardening* related to elastoplasticity⁹.

⁶It is assumed that the authors refer to an unadjusted concrete. Usually the concrete comprises additives and admixtures to optimise its behaviour to the project specific needs.

⁷Hydration: Attachment of water molecules to solute particles. [16, p. 191]

⁸Ductility describes the material's capability for (large) deformation upon loading without failure. (cf. [149, p. 35], [337, p. 3], [419, p. 148])

⁹Hardening: Stress continues to increase with strain beyond the yield limit of the material. [181, p. 86]

4.3 A note on the behaviour under pressure

All materials exhibit a distinctive behaviour when subjected to a specific mechanical loading (e.g., water pressure acting on a dam) or when exposed to a specific environment (e.g., extension or shortening in case of temperature change). The design of structures requires a correlation between the *action* and *reaction*. Such a correlation is called *constitutive equation*. Fig. 4.1 exemplifies the reaction (i.e., individual strain components) of a hardening¹⁰ concrete specimen exposed to *normal* constant environmental conditions (i.e., constant humidity and temperature) and subjected to a sustained constant compressive load (= action).

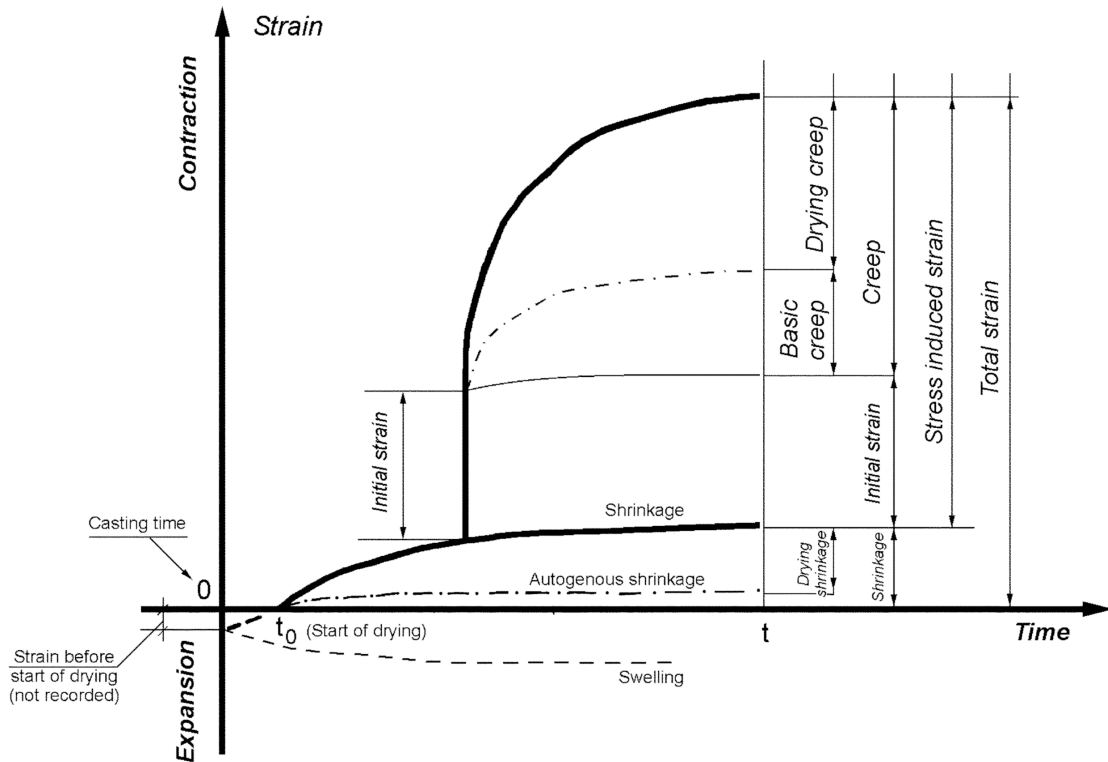


Figure 4.1: Development of strain components of a concrete specimen subjected to a sustained constant compressive load (uniaxial compression test; the moment of load application equals the moment *initial strain* develops) under constant environmental conditions (from [4, Fig. 1.1, p. 2]).

The development of the strain components shown in Fig. 4.1 all depend on the material properties, some of them depend on the environmental conditions, some of them are time-dependent, and others depend on the loading level (mechanical loading or temperature change) only. Some dependencies are described in the following sections. Note that the illustrated development is valid only for a hardening concrete specimen in an uncracked state at the macrolevel (i.e., scale of laboratory test specimen), meaning that the applied load is below approx. 30–40% of the compressive strength (i.e., below the limit of proportionality; cf. Section 4.9 on p. 66) at the moment of load application. Above this threshold, microcracks grow ([118] in [388, p. 16])—resulting in *strain hardening*—, link with each other, and, eventually, form macrocracks. Also, the specimen must not be exposed to any aggressive media (e.g., chemicals), which would alter the specimen's integrity and affect the hardening process. The development of the material properties and, therefore, of the individual strain components might be also different for concrete specimens placed underwater.

¹⁰Note that the reference the graph is taken from refers to hardened concrete. Anyway, the graph shows the onset of autogenous shrinkage which is because of hydration reactions when the concrete sets (cf. [4, p. 3]).

4.4 About strain in shotcreted tunnel linings

Engineers make use of circular linings in tunnels and other curved underground openings because of their high load-bearing capacity as soon as a stress ring (or stress arch if the lining is not closed) forms in the circumferential direction ([260, p. 3158]). The shotcrete lining usually features approximately a biaxial compressive stress state ([260, p. 3158], [129, p. B-5, E-24]). However, often not the compression is problematic but tensile stresses (even if occurring only locally or for a short period). And then tensile strength dominates the lining behaviour ([347, p. 7]).

In the DFOS section (cf. Chapter 8 on p. 169), tensile strain occurred on the first days after the shotcrete application, mostly at the ends of the top-heading lining before the ring was closed. The displacement pattern suggests that the unfavourable orientation of the foliation planes predominantly causes this elongation. At early ages, however, cracking of the lining is mainly because of restrained thermal and chemical shrinkage ([402, p. 1123]). Similarly, [210] (cited in [277, p. 2]) report about cracking because of unfavourable bending moments originating from the deformation of the heterogeneous ground and because of tensile loading induced by shrinkage and thermal gradients.

Because of hydration, temperature rises within the shotcrete (cf. Section 8.9 on p. 196). During this time, when the shotcrete is young, the elastic modulus is low and the creep rates are high. Thus, thermal expansion does not result in significant compressive stresses. However, as soon as the hydration process abates, the shotcrete cools off and shrinks (i.e., thermal shrinkage) ([401, p. 785]). The shotcrete is then older, being less ductile. The bond between the lining and the rock mass, but also between individual shotcrete layers, restrains the deformation to some extent. Autogenous shrinkage, which also takes place from the beginning on, further worsens the situation. Now, if the compressive loading is too low, which often is the case only at the beginning, tensile stresses might get induced locally and, if exceeding the tensile strength, the lining may crack (cf. [388, p. 126]). Thus, the influence of shrinkage and, in particular, of autogenous shrinkage—because it starts already before the shotcrete sets—must not be underestimated. According to [155, p. 696], it can be twice or three times the cracking strain of concrete ($\approx 0.01\%$).¹¹ Hence, the early age deformability in tension is of crucial importance ([368, p. 840]). [388, p. 43]

One must not forget that in-plane strain and stresses affect out-of-plane strain and stresses, and vice versa (cf., e.g., [129, p. B-72] referring to creep strain). The moment the shotcrete has been applied to the rock mass, several phenomena causing strain compete with each other ([401, p. 785]). Along its circumference, the lining is heterogeneous in terms of thickness, application quality, restraints, and environmental conditions and, thus, must deform non-uniformly. Because of anisotropies and imperfections, a strongly heterogeneous state of stress and deformation results in the lining ([129, p. B-36]). According to [388, p. 43], it is the non-uniform nature of volume change which causes cracking rather than the magnitude of shrinkage.

For conventional tunnel drives excavated in sequences (e.g., top and bench/invert heading), like it is for the calibration case (cf. Chapter 9 on p. 201) and for the evaluation case (cf. Chapter 10 on p. 243), tensile stresses might be introduced in the lining when the follow-up heading (e.g., bench/invert heading) undercuts the preceding heading (e.g., top heading) (cf. [129, p. B-65]). [388, p. 117f] kind of disagrees as he states: "Unloading is probably of limited relevance to the lining of a single tunnel constructed on its own with a top heading, bench and invert excavation sequence, since little unloading would be expected to occur." At the DFOS

¹¹According to [121, p. 87], the decrease in tensile strength because of shrinkage cracks is more pronounced for high strength concrete than for normal strength concrete.

section, which is used as the calibration case, a decrease in the compressive strain but also a switch from compressive to tensile strain can be observed when the bench/invert heading passes the section approx. five days after the top-heading excavation (cf., e.g., Fig. 8.13 on p. 192). However, the magnitude of tensile strain is low. The maximum tensile strain occurs within the first 38 hours after the reference measurement (cf. Section 8.8.2 on p. 190). Whether a particular amount of tensile strain at a particular moment is problematic is not to be judged based on the amount only. It rather depends on the change in deformability and strength coming along with ageing of the concrete material. Strength and stiffness increase, but ductility and creep effects decrease ([348, p. 103, 106]). With time, a transition from a ductile to a brittle material response takes place ([337, p. 1, 3], [368, p. 842]) and the strain at peak strength, both for compression and for tension, decreases ([368, p. 836]).

At last, it must be highlighted that in deep tunnels with large deformations the lining rarely fails because of bending and related tensile fractures but because of shearing ([129, p. B-14]). In case of bending and local tensile failure, the lining still makes up a stable multi-hinged arch or ring capable of bearing considerable load ([129, p. B-14]). In general, changes in the lining's stress and strain state are mainly deviatoric rather than hydrostatic ([388, p. 118]). The probability of shear failure may increase in case the rock mass comprises a few highly effective discontinuities crossing the tunnel.

4.5 Restraints

The reaction of the system (here the shotcrete lining) upon loading (mechanical and/or thermal) depends on the system's boundary conditions (here in terms of mechanical constraints) which, in addition, affect the evolution of the system's strain components ([353, p. 23]).

Fig. 4.2 illustrates the consequences of different degrees of restraint. A temperature load acts on a fixed concrete beam that hardens. The temperature increases because of the hydration heat during the hardening process. The beam expands. Because horizontal deformation is prevented on both ends of the beam, stresses get negative (i.e., compressive stresses). Once the hydration has reached its peak, the structure cools off. As the temperature decreases, the beam contracts and introduces tensile stress increments. Note that up to that moment the concrete has hardened already to some degree and features a higher strength and stiffness than in the initial state. Now think on the stiff aggregates in the concrete. They also restrain deformation. If the difference in the deformation of a particular aggregate and of the hardened cement or cement paste surrounding this aggregate is too large (i.e., the degree of restraint is too high), and if the total tensile stress exceeds the current tensile strength of the cement, there the concrete cracks.

At the concrete specimen of which the resulting strains are illustrated in Fig. 4.1, constraints resulted due to the frictional resistance at the contact of the specimen ends with the pressure plates.

At shotcrete tunnel linings, restraints mainly arise because of the bond between the lining and the rock mass to which the shotcrete is applied, the bond between the new lining segment (e.g., at the current excavation round) and the old lining segment (e.g., at the previous excavation round), and if the lining abuts, for example, on the unsupported temporary top-heading invert. Fig. 4.3 schematically illustrates the in-plane situation of a top heading. If the top-heading feet and the unsupported top-heading invert are apart from each other, and if there is, theoretically, no bond between the lining and the surrounding rock mass, the feet can displace downwards freely (cf. Fig. 4.3a). Anyway, this scenario is unrealistic. Rather because of the bond and the

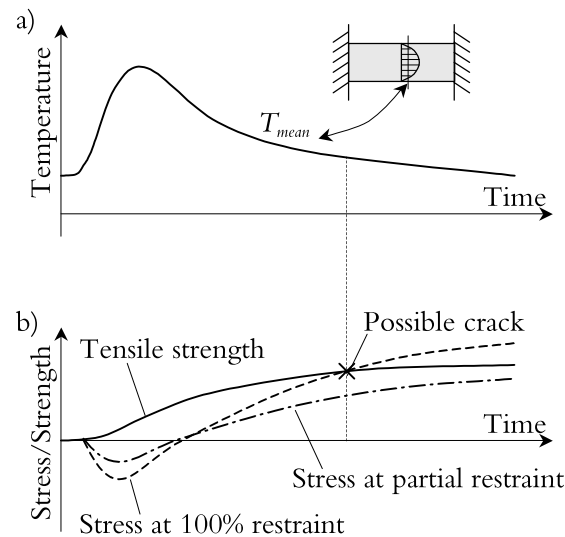


Figure 4.2: Schematic illustration of the restraint problem using the example of a fixed hardening concrete beam subjected to temperature load (from [282, Fig.1, p. 3]): (a) development of the mean temperature with time, (b) development of the stress and the strength with time.

fact that at any one moment the feet touch and get pushed into the rock mass (cf. Fig. 4.3b), deformation is restrained and stresses increase in the lining. Note that whether locally the rock mass displaces downwards relative to the lining or the lining relative to the rock mass depends, for example, on the loading of the system and the stiffness contrast between the lining and rock mass material. Restraints also arise when a lining features two or more layers applied at different times. Then, the shotcrete material of the individual layer differs in strength and stiffness. Other sources for restraints for the shotcrete to deform are, for example, wire meshes, lattice girders, and bolts but also the stiff aggregates in the shotcrete composite.

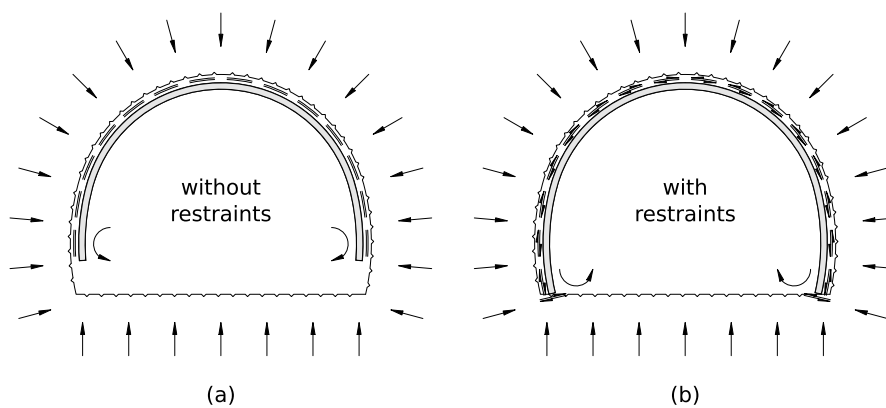


Figure 4.3: Schematic illustration of two in-plane scenarios of a top heading different in the degree of restraints: (a) without restraints (theoretical; unrealistic), (b) with restraints (realistic).

4.6 Strain components

Fig. 4.1 (p. 48) lists most important strain components of a concrete specimen under compressive loading. It misses the plastic strain as the load applied is below the yield strength. The following subsections briefly describe each of the strain components.

4.6.1 Elastic (instantaneous) strain

The moment an external mechanical load is applied to (or is removed from) the shotcrete, it reacts instantaneously and deforms according to its materials' elastic properties. Elastic strain develops. The amount of elastic strain depends on the load increment (or decrement) and on the stiffness properties of the shotcrete at the moment of load application (or removal). The shotcrete is a compound of solids and fluid-saturated pores each featuring a different stiffness. As the concentration of solids and fluid-saturated pores changes as cement hydration proceeds, the overall stiffness of the compound¹² changes as well (cf. Section 4.2 on p. 47). This is often termed *ageing* ([26] in [401, p. 785]; cf. also Section 4.1 on p. 47).

For an uniaxial mechanical load, the elastic strain component results to

$$\Delta\varepsilon_{el,\sigma}(t) = \frac{\Delta\sigma}{E(t)}; \quad (4.1)$$

with

$\Delta\varepsilon_{el,\sigma}(t)$ elastic strain increment (or decrement) due to uniaxial mechanical load [m/m],

$\Delta\sigma$ stress increment (or decrement) (compression or tension) [N/mm²],

$E(t)$ Young's modulus of the concrete at the moment of loading [N/mm²].

The strain increment (or decrement) depends on the age of the concrete—indicated by the term (t)—as the concrete's Young's modulus changes with the curing level.

4.6.2 Thermal elastic (instantaneous) strain

Thermal strain stems from the fact that the motion of the materials' atoms depends on the temperature ([16, p. 267]). At higher temperatures the motion is larger. And because of the repelling forces between atoms, the atoms then need more space and the volume increases. The thermal motion is smaller and atoms need less space if the temperature decreases. Then the volume decreases as well. Thermal strain is elastic as long as the repelling forces do not outweigh the attracting forces. It is similar for elastic strain upon mechanical loading. Note that with increasing pressure (applied externally) the distance between atoms decreases and the repulsion, motion, and temperature increase ([16, p. F6]). [119, p. 1-5, 14-9]

This concept above suggests that lateral elastic deformation upon axial loading (approximated with the Poisson's ratio) results because the motion of atoms aligns with the principal loading axes in a way so that the difference in repulsive and attractive forces gets a minimum (cf. [119, p. 2-5, 12-10], [198, p. 187]).

The concept also suggests that the behaviour of a material strongly depends on the matter's atomic structure. Thus, any empirical concept (e.g., Young's modulus) cannot be more than—and is by definition—an approximation.

Coming back to the shotcrete lining, because of the exothermic character of the hydration process, temperatures in the concrete can increase significantly within a few hours ([401, p. 790]). In massive structures, they can be up to 50 °C ([401, p. 785]). When the rate of hydration slows down, the temperature decreases again ([401, p. 785]) and balances with the temperature of the surrounding environment after some time. The variation in temperature causes the shotcrete to enlarge in case of temperature increase and to shorten in case of temperature decrease.

¹²In isotropic elasticity, the stiffness is described with the Young's modulus, E , and the Poisson's ratio, ν , (cf. Section 4.9 on p. 66), or with the bulk modulus, K , and the shear modulus, G .

The development of concrete temperature with time depends on the hydration process, the rate of external heat supply provided by conduction and volumetric heat sources ([402, p. 1125]), and the thermal capacity and conductivity ([353, p. 9]). Any of these contributors is very material, element, and project specific. For example, if the initial temperature at the moment of shotcrete application is lower, the later the concrete sets and the longer the hydration process lasts (cf. [42, p. 36f]). The external heat supply depends on the rock mass conditions (e.g., rock type, groundwater flow), the activity of heavy machines nearby, and the air supply system. And the thermal characteristics of the lining itself are determined by the linings' dimensions, the used type of aggregates, and the moisture state ([68, p. 62]).

According to [215] (cited in [388, p. 41]), the coefficient of thermal expansion, α_T , reduces from $21 \times 10^{-6} \text{ K}^{-1}$ at 8.4 hours of concrete age to $12 \times 10^{-6} \text{ K}^{-1}$ at 16.4 hours. That following, the coefficient remains constant. [353, p. 15] also reports that the coefficient at very early ages differs from the one of mature concrete. The variation of the coefficient must relate to the continuous change in share of solids, fluids, and gases in mass per unit volume.

A design value of $\alpha_T = 10 \times 10^{-6} \text{ K}^{-1}$ is valid for hardened normal strength and high strength concrete ([121, p. 94]). [353, p. 15] has applied this value to his investigations on restrained hardening concrete. As well as [388, p. 43] did to a rough calculation of the thermal strain of a concrete sample cooling off. Note that [210, p. 1053] and [368, p. 841] have the coefficient assumed being constant with time for their considerations simulating the behaviour of shotcrete.

For a thermal load, the elastic strain component results to ([68, p. 62])

$$\Delta\varepsilon_{el,T}(t) = \Delta T \cdot \alpha_T(t); \quad (4.2)$$

with

$\Delta\varepsilon_{el,T}(t)$ elastic strain increment (or decrement) due to thermal load [m/m],

ΔT change of temperature (heating or cooling) [K],

$\alpha_T(t)$ coefficient of thermal expansion (or contraction) at the moment of loading [K^{-1}].

The strain increment (or decrement) depends on the age of the concrete—indicated by the term (t)—, as the concrete's coefficient of thermal expansion changes with the curing level if not set constant.

Empirical approximation

Often, only the part of the thermal strain is considered that the hydration process causes. The effect of temperature changes in the surrounding (i.e., tunnel and rock mass) is then neglected.

Schubert (1988; [358]) proposes an equation he got by fitting of laboratory data. He himself states that the fitted development may be valid qualitatively but needs to be adjusted to at site conditions. The equation depends just on the concrete age, t . Note that the equation accounts only for thermal strain within the first days after the concrete has started to set (cf. Fig. 2 in the reference). [358, p. 151]

Schütz et al. (2011; [368]) adopted the approach by [358] (cf. previous paragraph) and substituted constant fitting values with six parameters: the coefficient of thermal expansion, α_{th} ; the maximum increase in temperature above the ambient environmental temperature in the tunnel, ΔT_{max} , that occurs after shotcrete installation at the time t_{max} ; the time at which the effect of the hydration on the temperature has ended, t_{zero} ; and A_t and C_t which both depend on t_{max} and t_{zero} . They further assume a constant temperature distribution across the thickness

of the shotcrete body and $\alpha_{th} = \text{constant}$. The implementation has been validated using data from [173]. For calibrated model parameters, refer to Tab. 4 in the reference. [368, p. 841ff]

Meschke (1996; [260]), in contrast, considers the thermal strain by a formulation that is equivalent to Eq. 4.2. Input parameters are the temperature, θ , the reference temperature, θ_0 , and the coefficient of thermal expansion, α^θ . [260, p. 3129]

Neither Schädlich and Schweiger (2014; [348]) nor Neuner et al. (2017; [276]) account for thermal strain separately. However, [276, p. 3], in particular, state that the deformation of unsealed specimens during ordinary shrinkage tests comprises shrinkage strain and thermal strain. The hydration process causes the latter. And at ordinary creep tests, shrinkage strain, thermal strain, and creep strain develop. Although not explicitly stated in any of the two publications, it must be assumed that they concluded to have the thermal strain stemming from the hydration process considered with their formulation of the development of shrinkage strain and with related fittings. Then, however, the question arises whether [358] and [368] have fitted their formulation of the development of shrinkage strain (cf. p. 56) to data that has been corrected prior to fitting by subtracting the thermal strain.

4.6.3 Shrinkage (delayed) strain

The European standard "Eurocode 2: Design of concrete structures" ([20, p. 32]) and the "Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete" by the American Concrete Institute ([5, p. 4f]) distinguish between the two most important shrinkage components, namely

- the autogenous shrinkage and
- the drying shrinkage.

The development of both components depends on time but not on external mechanical loading¹³, and is controlled mainly by the ambient air humidity, the size of the concrete structure, and the concrete's composition ([20, p. 30]; cf. Fig. 4.4). Shrinkage causes a shortening of the structure's dimensions and the volume decreases ([368, p. 840]). Depending on whether shrinkage develops in an unrestrained or restrained setting, either compressive strain¹⁴ (negative; cf. Fig. 4.4) or tensile strain (positive) is introduced.

Compared to cast concrete, shrinkage of tunnel linings made of shotcrete is more pronounced because of "less aggregate content and higher water-cement ratio" ([18] in [347, p. 39]).¹⁵ However, to which amount shrinkage sums up can vary strongly along the lining circumference depending on the moist conditions. From measurements in a tunnel section of a subway in Vienna, [128, p. 84] report the maximum shrinkage being at the tunnel roof, whereas it is only 50% at the side wall and negligible small at the invert.

¹³[260, p. 3129] cites several publications all stating that shrinkage is independent of the acting stresses. However, he also cites [25] who disagrees with this.

¹⁴The term *compressive* is misleading here because no mechanical compressive loading must be present to have the concrete element shrink. The term refers to the value sign rather than on the action causing the strain.

¹⁵The cited statement is dubious since shotcrete usually features a lower water-cement ratio than conventional concrete. [174, Tab. 4–5, p. 7] cites ratios of 0.3 ... 0.65 from standards and guidelines for shotcrete. [225, p. 61] recommends keeping the ratio below 0.5. In contrast, for standard concrete, usual upper thresholds range from 0.45 to 0.75 (cf. [37, Tab. 2, p. 1]). Anyway, [5, p. 6] states that for concrete with a water-cement ratio of less than 0.4 autogenous shrinkage contributes significantly to shrinkage strain. And according to [121, p. 92], drying shrinkage increases but autogenous shrinkage decreases with increasing water-cement ratio. Thus, one must differentiate between the types of shrinkage (cf. next subsections).

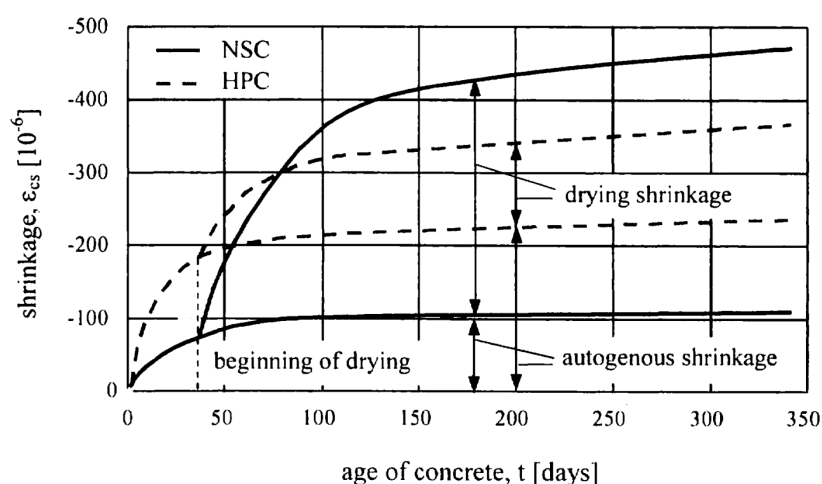


Figure 4.4: Development of autogenous and drying shrinkage with time in normal-strength concrete (NSC) and in high-performance concrete (HPC) (from [120, Fig. 3.1-13, p. 43]). ε_{cs} ... total shrinkage (in metre per metre).

Autogenous shrinkage

Autogenous¹⁶ shrinkage (also termed basic shrinkage; cf. [121, p. 92]) results from two phenomena: hydration shrinkage (also termed chemical shrinkage; cf. [401, p. 785]) and capillary shrinkage (cf. [155, p. 693]). The former is due to volume reduction as the cement hydrates. The reason for the latter is that capillary traction between the liquid and the gaseous phase saturating the porous space increases and compresses the skeleton (i.e., the compound shrinks) ([401, p. 788]) in case free water is *lost*.

Generally, *loss of water* results if free water is consumed by the hydration reaction ([401, p. 788]) or evaporates ([388, p. 37]). By definition, autogenous shrinkage is determined under closed conditions, meaning that any fluid exchange with the surrounding exterior is prevented ([168] in [369, p. 149]). Shrinkage related to water loss due to evaporation is considered by drying shrinkage (cf. next subsection).

According to [279] (cited in [388, p. 37]), capillary shrinkage is possible only as long as the concrete is still plastic and takes place before the cement paste sets ([4, p. 3]). Capillary shrinkage is, therefore, also known as plastic shrinkage (cf. [326, p. 304], [388, p. 37]). In case the concrete specimen is placed underwater, capillary traction does not increase but rather decreases and causes swelling ([202] in [4, p. 3]).

Autogenous shrinkage is most prominent in the curing period and its development (temporal amount, effective period) strongly depends on the temperature and, therefore, also on the size of the concrete structure which controls the heat storage capacity ([353, p. 22]). However, the asymptotic value is independent of temperature variations ([401, p. 791]). According to ([353, p. 22]), a part of the shrinkage already takes places before strength develops and does not affect the stress regime within the structure.

Compared to drying shrinkage, autogenous shrinkage approaches its ultimate value relatively soon (cf. Fig. 4.4). According to [401, p. 791], autogenous shrinkage takes approx. 2 ... 3 months.

Autogenous shrinkage decreases with increasing water-cement ratio and increases with increasing cement paste content ([121, p. 92]).

For further reading on autogenous shrinkage, refer, for example, to [230, 385].

¹⁶Autogenous: Produced independently of external influence or aid. [259]

Drying shrinkage

Drying shrinkage is caused by diffusion of chemically unbounded water ([353, p. 20]), occurs in the hardened cement paste as soon as this *free* water in large voids and capillary pores is absorbed, and takes place over several years ([388, p. 38ff]; cf. Fig. 4.4). Note that for drying shrinkage also the capillary traction is in charge, but the origin of water loss is different.

Diffusion is a natural process where a concentration difference between two adjacent substances (e.g., concrete structure and surrounding air) is levelled out. Hence, the development (amount, effective period) of drying shrinkage depends predominantly on the size of the concrete structure—the thicker the structure, the more chemically unbounded water exists and the more time it takes for water at the structure’s core to diffuse to the structure’s boundary—and on the ambient air humidity determining the (initial) concentration difference. The rate of the drying process decreases disproportionately to the size of the structure ([353, p. 21]).

As drying shrinkage develops in the hardened cement paste, all factors increasing the final quantity of hardened cement paste will yield to increased drying shrinkage (cf. [121, p. 92]). Such a factor can be, for example, an increased cement content ([279] in [388, p. 38]). Using plasticisers or other water-reducing admixtures requires a higher share of cement in the mix. Large stiff aggregates impede shrinkage. Hence, a finer grading curve of the aggregates indirectly boosts drying shrinkage in two ways. First, shotcrete with more content of fine aggregates needs relatively more cement. And second, the shotcrete has less big stiff aggregates, which reduces the restraining effect of such ([313] in [388, p. 39]). The same applies to the water-cement ratio. An increased ratio leads to a reduced proportion of aggregates and to more shrinkage (cf. [121, p. 92]). [388, p. 38f]

By re-wetting the concrete it will swell. This can recover a part of the long-term drying shrinkage. [168, p. 32]

Carbonation shrinkage

Carbonation shrinkage is irreversible and starts at the surfaces of concrete structures due to the reaction of carbonic acid (from carbon dioxide and air) with hydrates of the hardened cement paste ([388, p. 41], [4, p. 3]).

None of the guidelines considered (i.e., [5, 20, 121]) deal with carbonation shrinkage. Thus, its negligible share of total shrinkage is assumed. [284] (cited in [388, p. 41]) reports typical values for the depth of carbonation of 2 to 3 mm after six months. This rate is too low to contribute significantly in the early-age volumetric shrinkage. In particular, because the carbonation process starts in the surface layers.

Empirical approximation

Schubert (1988; [358]) approximates shrinkage with the hyperbolic equation by [2]. Schädlich and Schweiger (2014; [348]) and Schütz et al. (2011; [368]) use the same equation but refer to [3]. The equation considers the ultimate shrinkage strain at infinite time, $\varepsilon_{\infty}^{sh}$, the age of the shotcrete, t , and the constant B (in time units). [348] substitute B with t_{50}^{sh} which refers to the time when 50% of shrinkage has occurred. According to [3, p. 7], the equation covers autogenous, drying, and carbonation shrinkage. [368] use data summarised in [387] to validate their implementation. For calibrated model parameters, refer to Tab. 3 in [368, p. 843]. [348] validate their implementation with data from a shrinkage test on a polymer-modified shotcrete reported in [309, Fig. 7b, p. 7] (cf. [347, p. 39f]). [265] reports in detail about this shotcrete. Tab. 7 in [347, p. 42] lists recommended

values for the model parameters: $\varepsilon_{\infty}^{sh} = [-0.0005; -0.0015]$ (dimensionless), $t_{50}^{sh} = [28; 100]$ (in days).

Both Meschke (1996; [260]) and Neuner et al. (2017; [276]) make use of the semi-empirical shrinkage law by [28]. The equation considers the ultimate shrinkage strain at infinite time and at zero per cent humidity, $\varepsilon_{\infty}^{sh}$, the age of the shotcrete, t , the age of the shotcrete at the start of drying, t_0 , the effective thickness of the shotcrete body, d_{eff} , and the function k_h for the humidity ([260, p. 3129]). [276, p. 14f] validate their implementation with data from [271]. Because not stated otherwise, it must be assumed that the data stems from shrinkage tests on unsealed shotcrete specimens. Thus, shrinkage strain comprises the autogenous and the drying part.

Note one important fact: At shrinkage tests on unsealed specimens, next to autogenous shrinkage and drying shrinkage, also the thermal strain due to the hydration heat contributes to the measured total strain. [276, p. 3]

4.6.4 Creep (delayed) strain

The development of creep strain starts after the initial loading and increases with time under sustained constant load ([4, p. 3]). Creep results in an overall decrease in volume ([388, p. 46]). The cause of creep strain is not fully clarified yet ([4, p. 4], [389, p. 50]) but is controlled predominantly by the properties of the hydrated cement paste ([353, p. 26], [389, p. 50]).

Short-term creep relates to the movement of capillary water within the cement matrix upon load ([332] and [421] in [370, p. 284]; [153, p. 12] terms this *consolidation*). Here, thermodynamic forces, which depend on the stress state, determine the rate of water diffusion (cf. [399] in [370, p. 285]).

Long-term creep relates to changes in the micro-structure of the calcium silicate hydrate (C-S-H) phase ([30] in [370, p. 284]; cf. [353, p. 26]). Because of the external load, microbonds (microprestresses) created during the hydration process break locally and reform. This relaxation or dislocation-type mechanism is termed viscous flow (comprising viscous flow strain and viscous slip). [32] have proposed the theory (cf. Subsection 'Concept of the microprestress force' on p. 60). At a constant load, the potential for viscous flow is limited and the rate of viscous flow decreases continuously. [370, p. 284ff]

According to [141] (cited in [353, p. 26]), also intergranular sliding and microcracking contribute to the viscous deformation.

Note that some creep strains are recoverable (i.e., viscoelastic creep strain) while others are irrecoverable (i.e., viscoplastic creep strain). The former comprises, for example, the delayed elastic strain because of the interaction between the cement matrix and the aggregates. The latter results because of, for example, microcracking, crystal failure, and sliding within the micropores of the cement gel ([370, p. 284]). [4, p. 4f]

As the age of the concrete controls the development of all creep mechanisms (e.g., strength and stiffness increase with age yielding to less delayed elastic and plastic strain; cf. Section 4.6.5 on p. 60 and Section 4.6.6 on p. 62), creep strain relate to the amount of the initial loading and of any loading increment or decrement, and to the moment the loading situation changes. However, [10, p. D-50f] states that the utilisation of the concrete and the loading duration determine the creep rate significantly more than the concrete age.

Considering the boundary conditions, Fig. 4.1 (p. 48) divides creep into two main components: basic creep (non-drying creep) and drying creep. The basic creep is the sum of all creep strain components that develop at a constant moisture state of the concrete (no moisture losses or gains;

no moisture movement through the material) and are, therefore, independent from the size and shape of the structure ([4, p. 3]). Resulting strains are irrecoverable ([31] in [154, p. 27]). Drying creep develops because of changes in the moisture content and the development of a moisture potential within the concrete structure, which causes the capillary water to move (moisture diffusion). Similarly to drying shrinkage (cf. Section 4.6.3 on p. 54), drying creep depends mainly on the structure's size and shape which determine the initial moisture content, the surface area subjected to the surrounding media where the drying process starts, and the volume for which the humidity equilibrium must be found. According to [173] (cited in [388, p. 47]), it decreases with increasing size of the structure. Probably because a bigger structure needs more time for balancing meanwhile the shotcrete hardens.

In case the concrete is subjected to sustained load, both drying shrinkage and drying creep develop. Hence, it is difficult to quantify their share of the total strain ([388, p. 44]). In general, it is assumed that creep develops independently from shrinkage. Shrinkage coefficients are separately determined with shrinkage tests (recording of the change in strain with time because of drying; the sample is not subjected to any load). However, this assumption is probably wrong as creep and shrinkage are both linked to movement of water ([280] in [388, p. 44]).

Creep strain and creep strain rates increase ([388, p. 46])

- the earlier the concrete is loaded resulting in a higher utilisation;
- with increasing magnitude of load¹⁷, resulting in a higher utilisation; creep is proportional to an uniaxial load at low stress levels of $< 0.4f_c$ (cf. limit of proportionality in Section 4.9 on p. 66)¹⁸; above this level, the creep rate increases; progressive creep and, eventually, failure at stress levels of $> 0.8f_c$ ([1]);¹⁹
- with decreasing relative humidity as a lower humidity speeds up the drying process.

With the shotcrete ageing, the creep rate reduces (cf. [348, p. 103]), probably because the amount of solid hydrates already formed increases and this slows down the movement and diffusion of the remaining free water (cf. Section 4.1 on p. 47) and increases the restraining effects.

With regard to the shotcrete mix and the properties of its constituents, the cement content, the aggregates, and the porosity of the cement paste affect creep directly. Creep increases with increasing concrete porosity ([279] in [295, p. 2]) because it allows for more water movement. Substituting cement to some extent with micro-silica reduces the porosity ([388, p. 47]). Creep also increases with increasing cement content (e.g., [173] and [280] in [388, p. 46]), probably as a higher cement content yields to higher hydration temperature which speeds up the drying process. However, a higher cement content—assuming a constant water content—implies a reduced water-cement ratio with which the concrete develops a higher strength as fewer voids remain after the hydration process ([388, p. 20]). A higher strength leads to a lower utilisation and, therefore, to less creep.

Aggregates embedded within the cement paste hamper the development of creep (as for shrinkage too: cf. Section 4.6.3 on p. 54; and for reinforcement: cf. [98] in [295, p. 2]). Bigger and stiffer aggregates decrease the total amount of creep. Unfavourable aggregate shape may

¹⁷Recent tests showed that the relation between creep strain and load level can also be negative (i.e., more strain at lower load levels) (cf. [124] in [153, p. 17]).

¹⁸Note that the limit of proportionality differs for compressive and tensile loading (cf. [153, p. 17]). For differences between creep under compression and creep under tension, refer to, e.g., [99, p. 18f].

¹⁹Related to the level of (constant) loading, creep can be divided into three stages ([153, p. 8]): (1) primary creep with a decreasing rate; (2) secondary creep with a relatively uniform rate; (3) tertiary creep with an (disproportionately) increasing rate. The phases relate to a constant load and temperature state ([333, p. 173]). Because the concrete's strength increases with its age, it can sustain higher loads at later stages (cf. [10, p. D-34f]).

introduce stress concentrations promoting microcracking and, thus, also creep effects ([235] in [153, p. 14]). [388, p. 47]

All other parameters (e.g., water-cement ratio, cement type, amount and type of admixtures and additions) affect the development of creep strain only indirect as they determine the development of the concrete's strength and, hence, the strength-stress ratio (or utilisation). [388, p. 47]

Relaxation

Creep is by definition the increase in strain with time under a sustained stress ([280] in [295, p. 1]). Relaxation, on the other hand, is the decrease in stress with time under constant strain ([280] in [388, p. 44]). If high loads subject the shotcrete prior to relaxation, relaxation then takes place faster and to a higher amount ([129, p. F-60]).

In reality, pure relaxation having a fully constrained deformation state is rare. The same applies to creep. When subjected by a restraint stress, deformation restraints are introduced. The latter partly impedes the development of creep strain. Thus, usually creep and relaxation are active simultaneously and depend on each other. Relaxation reduces the stress level. At lower stresses, less creep develops. [153, p. 21]

Empirical approximation

The rate of flow method ([107]) used by Schubert (1988; [358]) distinguishes between reversible and irreversible creep strain. The reversible part is split into short-term and long-term creep. Each requires an ultimate creep constant $C_{d\infty}$ and a creep rate constant Q . The irreversible part is adjusted with an age-related creep function $C_{(t)}$. Those parameters, the reversible strain at the previous time step, and the stress determine the resulting creep at a particular moment. To account for loadings of $> 0.5f_c$ and the related non-linear dependency of the creep rate on the stresses ([325] in [358, p. 151]), he (but also [128, p. 81]) adds an exponential term to the primary creep function $C_{(t)}$. [358, p. 150f]

Schädlich and Schweiger (2014; [348]) use a viscoelastic approach. It requires, for example, a creep factor, φ^{cr} , the stress, the time at the start of the loading, t_0 , and the time until 50% of creep strains have developed, t_{50}^{cr} . Up to a shotcrete utilisation of 45% in compression, the creep factor is a constant. Above, it is replaced by the function φ_k^{cr} by [110] which considers the utilisation. They validate their implementation with data from [10]. Tab. 7 in [347, p. 42] lists recommended values for the model parameters, e.g.: $t_{50}^{cr} = [1; 5]$ (in days). [348, p. 107f]

To consider creep and relaxation, Schütz et al. (2011; [368]) transferred the uniaxial time-dependent Newton dashpot into the general stress space. The resulting implementation requires, for example, the creep potential, P^{cr} , the applied stress, the initial age of the shotcrete upon loading, t_0 , and the time-dependent viscosity function $\eta(t)$. For the validation, they use data from [206]. For calibrated model parameters, refer to Tab. 2 in the reference. [368, p. 840ff]

Meschke (1996; [260]) uses a Duvaut-Lions type viscoplastic formulation published by [374]. Here, creep strain can develop only in the plastic regime. Next to the stress, the formulation requires, for example, the viscosity parameter η^* . Data from [173] was used for validation ([261, p. 3150ff]). [260, p. 3133f], [276, p. 9f]

Neuner et al. (2017; [276]) applied a modified version of the solidification theory by [29]. The theory was transferred into the effective stress space. It distinguishes between viscoelastic strain and flow strain, among others. Both depend, for example, on the stress and the utilisation of the shotcrete. In the formulation of the viscoelastic strain, the parameter $v(t)$ accounts for the

development of the load bearing volume fraction of the hydrated material. Data from [271] is used for the validation. [276, p. 3, 15ff]

Note one important fact: At creep tests on unsealed specimens, next to basic creep and drying creep, also autogenous shrinkage, drying shrinkage, and thermal strain contribute to the measured total strain. Thus, separate shrinkage tests on unsealed specimens are required to determine creep strain. [276, p. 3]

Concept of the microprestress force

To allow for a description of the microstructural processes determining long-term creep, [32] introduced the microprestress solidification theory. In this theory, the idealised C-S-H layers comprise sheets (cf. Fig. 4.5A and the building blocks in Fig. 4.5B). Note the water molecules in the interlayer region. Between those sheets, microprestress forces act. Resulting bonds are disordered and unstable. The pressure by the water molecules contributes disjoining the sheets but is counterbalanced by the existing bonds. Now, if the stresses apply to the concrete on the macroscale, bonds may break locally. This leads to slip of sheets relative to each other which, on the macroscale, is observed as long-term creep. The sliding of the sheets results in a reduction (relaxation) of the microprestress forces. In [154], the process described here is referred to as dislocation-like sliding in hydrates. [154, p. 10ff]

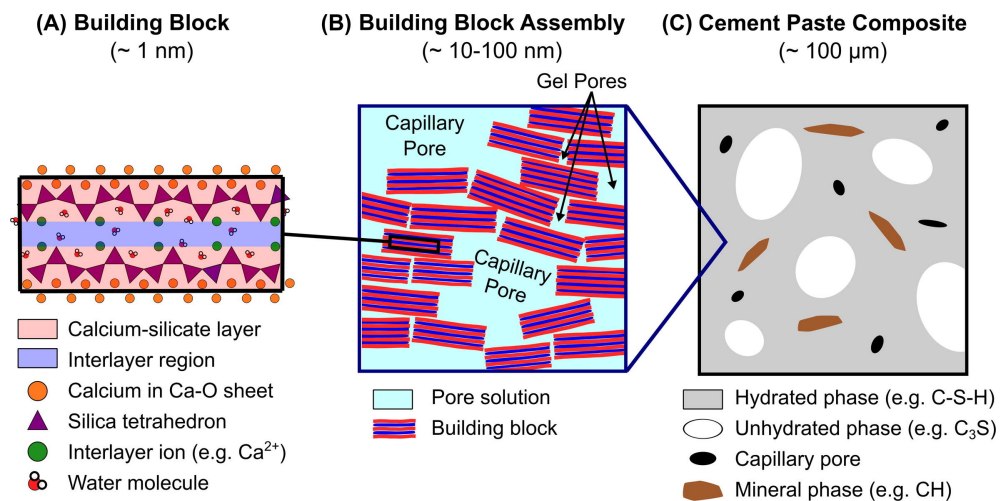


Figure 4.5: A multiscale perspective for cement paste materials (from [296, Fig. 1, p. 15]). (A) Building blocks describe the fundamental atomic interactions of the hydrated phase. (B) At larger scales, the hydrated phase is composed of an assembly of building blocks that form larger structures resulting in a hierarchical pore network. (C) The hydrated phase is intermixed with pores and mineral and unhydrated phases to form a cement paste composite at the microscale.

4.6.5 Plastic (instantaneous) strain

To account for consistency in the description of the individual strain components, the term plastic must not be equated with the term irreversible (or irrecoverable).

Irreversible strain cannot be undone upon unloading (or load reversal). Here, unloading not only relates to a decrease in the level of mechanical stress applied externally to the concrete volume. It also relates to thermal loading and to other changes in the field or boundary conditions (e.g., change in the humidity). Some of this irreversible strain is a consequence of the ageing of the concrete (cf. Section 4.6.6).

It is irreversible because of two effects of which both stem from the change in the share of the constituents of the ageing concrete (more fluids, fewer solids $\rightarrow$ fewer fluids, more solids):

1. As the share of fluids decreases, shear stresses introduced by the unloading (or load reversal) do result in less viscous flow. Instead, solids cope with those shear stresses and transfer them across solid-solid contacts. Note that macroscopically, viscous flow upon unloading can be (falsely) interpreted as elastic strain.
2. As the share of solids increases, solids more and more impede fluid flow and diffusion.

Strain components affected are the elastic, thermal elastic, shrinkage, and creep strain. As long as the concrete has not reached its final state of maturity, some of the elastic and thermal elastic strain remain irrecoverable. As for the shrinkage and creep strain, the concrete still comprises some free water even if it has fully hydrated (i.e., excess water) and more water can be introduced by re-wetting it. This allows for reversing some of the previous shrinkage or creep strain. However, at a hardened concrete, the strain rate is probably that small that strain reversal is non-existent from a practical engineering point of view.

Upon loading, the concrete compacts (or consolidates) and constituents are rearranged ([10, p. D-4, D-28ff]). This is more distinct in case the concrete is young. And, since viscous creep effects take place, also microcracks develop (or rather microbonds are destroyed; cf. Section 4.6.4 on p. 57). All resulting in (mostly) irreversible strain. In particular, the microcracks relating to creep may be also referred to as (visco-)plastic strain.

However, here, plastic strain as a separate component of the strain history of a loaded concrete specimen refers to irreversible deformation of the skeleton²⁰ (cf., e.g., [158, p. 2104], [402, p. 1127]). It also results in microcracks, but those cracks develop instantaneously rather than gradually as of those related to (viscous) creep. They are an immediate consequence of an overloading of the specimen; either in compression or in tension. This overload can be because of mechanical stresses applied externally. But it can be also because of restraints (cf. Section 4.5 on p. 50) combined with tensile strains introduced by shrinkage or by thermal contraction (cf. [156, p. 709], [157, p. 295f]). Note that shrinkage and creep (or relaxation) can reduce a critical compressive stress state.

Plastic strain develops also before the material reaches its peak strength ([348, p. 103]). Consider here the limit of proportionality described in Section 4.9 (p. 66) below which plastic strains are negligible. With the level of loading being above this threshold, the number and size of cracks will continuously increase, eventually leading to failure.²¹ The ageing of the concrete mentioned above indirectly also affects the amount of plastic strain because, with ageing, the strength develops (cf. [156, p. 707]). Thus, during hardening, of which the evolution rate depends on the temperature, any strength or strain thresholds continuously change (cf. Section 4.7). That, the moment of loading and, of course, also the loading magnitude strongly determine whether plastic strain results.

In the framework of numerical plasticity, hardening, softening, and flow rules aim to account for the non-linear stress-strain relationship. They need to consider the time-dependency of the strength and stiffness to allow for a proper modelling of the material behaviour in all of its states of maturity. (cf. [260, p. 3122])

²⁰The skeleton comprises the solid components of the compound (cf. Section 4.1 on p. 47).

²¹The failure mechanisms involved depend on the stress state. It is cracking in pure tension and in mixed tension-compression states. But it is crushing in three-dimensional compression states. (cf. [260, p. 3122f])

Empirical approximation

The approach by Schubert (1988; [358]) is one-dimensional. Thus, it cannot account for plastic strain.

The model by Schädlich and Schweiger (2014; [348]) makes use of a Mohr-Coulomb and Rankine yield surface. On the compression side, they use the hardening and softening rules proposed by [368] (cf. next paragraph). On the tension side, the stress-strain relation is linear up to the tensile strength. Then a linear strain softening relation follows limited by a constant residual strength. [348, p. 103ff]

For the compression yield surface, Schütz et al. (2011; [368]) adopt the approach by [72]. And for the tension yield surface, it is a modified Rankine criterion proposed by [390]. The stress-strain relationship on the compressive side comprises a pre-peak quadratic (or parabolic) strain hardening part and two linear strain softening parts. The latter is limited by a constant residual strength. The tensile stress-strain relation is first parabolic (i.e., pre-peak strain hardening), followed by a linear or exponential strain softening part. Here, too, a constant residual strength limits the softening part. [368, p. 835ff]

Meschke (1996; [260]) applies the Drucker-Prager and the Rankine criterion. On the compression side, the stress-strain relation comprises a quadratic pre-peak part and a constant (perfectly plastic) post-peak strength (i.e., no softening). The post-peak softening part on the tension side is either linear or exponential. [260, p. 3123; Fig. 4, p. 3136]

The model by Neuner et al. (2017; [276]) bases on the one proposed by [134]. And the hardening/softening follows the approach by [258]. [276, p. 3ff]

4.6.6 Irrecoverable strain due to ageing

As the hydration of the concrete (or shotcrete) proceeds, more and more solid phases substitute fluid and gaseous phases per unit volume of the composite (cf. Section 4.1 on p. 47). The concrete ages, continuously loses some of its initial ductility, stiffens (cf. Section 4.6.1 on p. 52), and its porosity decreases. As a result, movement of fluids within the composite, of which some shrinkage and creep effects originate (cf. Section 4.6.3 on p. 54 and Section 4.6.4 on p. 57, respectively), is increasingly impeded.

Now, imagine, even if all strain components listed above would be entirely recoverable (i.e., elastic), because of the ageing of the concrete, inelastic strain would result if the material is loaded at one moment and unloaded at any moment after. Literature terms it *ageing-induced strain* (cf., e.g., [261, p. 3147]).

This phenomenon affects all strain components except for the plastic strain, which is per se inelastic. Ageing-induced irrecoverable strains get small as soon as the concrete approaches its fully hydrated state.

4.7 Peak strain

As in the previous paragraphs written, the concrete's ductility decreases with ageing. At a very early age, when not much hydrate mass has formed yet and the composite still allows for movement of the fluids, the ductility is high. Note that then most parts of the strain are plastic and irrecoverable upon unloading (cf. [10, p. D-36]). With proceeding hydration, the composite stiffens and the share of solids being more brittle increases. Accordingly, the peak strain—strain at peak strength—decreases as the concrete gets older (e.g., [371] in [368, p. 836]). It increases with increasing confining pressure ([64] in [348, p. 105]).

On the compression side, some reported concrete age related values are:

- 1–2 hours: $-0.030 \dots - 0.040$ m/m ([129] in [348, p. 106], [135] in [368, p. 836]);
- 8 hours: -0.005 m/m ([129] in [348, p. 106]);
- 24 hours: -0.002 m/m ([129] in [348, p. 106]).

The values comply with [47] who states that young shotcrete can sustain compressive strain of up to $-0.005 \dots - 0.006$ m/m with little spalling. Experiments suggest that peak strain remains relatively constant after 24 hours (cf. Fig. 8 in [347, p. 13]). Pay attention to the fact that [370, p. 289] estimated the characteristic (or retardation) time of short-term (viscoelastic) creep for a shotcrete material to be approx. 24 hours (cf. Subsection 'Diffusion of absorbed water—short-term creep' on p. 79).

On the tension side, [207] (cited in [368, p. 840]) report a range of peak strain values of $0.0002 \dots 0.0005$ m/m. They also state that peak strain reduces significantly within the first hours after the application of the shotcrete. [63] (cited in [368, p. 840]) reports similarly low values with the tensile peak strain being 3% (at early stages) to 9% (at an age of 28 days) of the compressive peak strain.

4.8 Shotcrete strength

Section 4.2 (p. 47) briefly describes the origin of strength and stiffness increase with time. It relates to the continuous increase in the share of solids and bonds.

After two minutes, shotcrete already features a compressive strength of 0.1 to 0.2 MPa (cf. [289, p. 26]). The increase is first quasi-linear (cf. Section 4.2), but it is usually assumed that the rate then decreases exponentially. Anyway, there may be also phases with a lowered increase followed by a phase with an increased rate (cf. [208, p. 374]). The concrete mixture, the boundary conditions, and triggered chemophysical processes determine its development. Often, the peak strength is assumed to be reached after 28 days. This probably owns to several facts: (1) for standard concrete, the strength classes relate to strengths measured at samples with an age of 28 days (cf. [20, p. 27]); (2) for standard concrete with an age of over 28 days, the average strength is assumed constant (cf. [20, Eq. 3.2, p. 27]); (3) in standard cases, the long-term increase in strength is not of interest. However, having fittings to test data that reach the peak value asymptotically at 28 days may be wrong; at least for the today's shotcrete material. Consider here, for example, test results in [57, p. 23ff]. Fittings for the unreinforced shotcrete he tested, using a cement of class 42.5 and a water-cement ratio of 0.44, suggest a compressive strength of approximately 70 MPa at an age of 112 days, whereas it is approximately 61 MPa at 28 days.

Aggregates feature an inherent (or constant) strength and stiffness. The cement, superplasticisers (water-reducing admixtures), and accelerators are, however, reactive components of the shotcrete and their interaction influences the strength development ([352, p. 23]). According to [42, p. 37], the cement content, the water-cement ratio, and the temperature of the fresh concrete have the largest impact on the shotcrete's performance. Minor variations in one component's share, or the use of another type of component, results in different characteristics (e.g., reactivity) of the (hardening and hardened) shotcrete material (cf., e.g., [42, p. 32ff], [208, p. 351ff], [352, p. 26f]).

However, also the loading history during the hardening process affects the resulting performance. [10, p. D-11ff], for example, found out that if the shotcrete sample is loaded below the damage threshold while it cures, eventually, the sample develops a higher strength compared to

one which has not been loaded at all. He relates this phenomenon to consolidation; pore space gets smaller and this presses excess water out. Note that shotcrete material usually features an increased water content to increase pumpability ([388, p. 20]). The dependency on the loading or strain history makes it impossible to set the laboratory testing program up in a way so that all scenarios are considered. For example, to determine the change in the shotcrete's utilisation correctly in the laboratory requires two theoretically identical samples experiencing the same loading or strain history but tested at different moments (cf. [10, p. D-5]). Anyhow, referring to the shotcrete installed in the tunnel, the exact history is unknown prior to construction (or at all).

Usually, the lining is mainly in a biaxial state of stress (e.g., [203] in [158, p. 2106]). If it is a biaxial compressive state, the strength is up to 25% higher than under an uniaxial compressive state ([368, p. 835]). It is similar for the failure strain (cf. [20, p. 37]). Some models introduce a constant coefficient for the ratio of biaxial to uniaxial compressive strength, f_b/f_c , (e.g., 1.16 in [155, p. 698] and in [261, p. 3146]). This approach is valid only because, according to [63] (cited in [155, p. 698]), the chemical hardening is (approximately) isotropic.²² Strengths develop with (approximately) the same rate (cf. [388, p. 25]). This means that—next to the ratio mentioned before—also following strength ratios remain constant during hardening ([155, p. 698]): ratio of uniaxial compressive yield to uniaxial peak compressive strength, f_{cy}/f_c , (e.g., 0.25 in [155, p. 698], and 0.1 in [277, Tab. 1, p. 6]); ratio of uniaxial tensile to uniaxial compressive strength, f_t/f_c , (e.g., 0.1 in [155, p. 698] and in [348, p. 104]). Those ratios may not be constant for all shotcrete ages or loading situations. For example, according to [21] (cited in [388, p. 155]), the ratio f_y/f_c is increased at an early age because of the shotcrete's high ductility. If the model applied neglects the strength increase in case of a biaxial (or multiaxial) compressive stress state, calculated utilisation ratios are on the conservative side. Anyway, as soon as one principal stress component is tensile, results may be on the unsafe side. Note that, according to [74] (cited in [388, p. 17, 29]), the presence of tensile stress reduces the compressive strength but also peak (and failure) principal compressive and tensile strains. Chapter 8 (p. 169) shows that the strain field in the shotcrete lining of a conventional tunnel drive can be quite non-uniform. All imaginable biaxial stress states seem possible. From pure compression or pure tension to any intermediate state. Making it even more difficult, tension softening affects the material strength in compression, and compression softening affects the strength in tension ([347, p. 7]).

Empirical approximation

When modelling shotcrete strength, its time-dependence should not remain unconsidered. Single-valued yield strengths (analytical considerations) or yield surfaces (numerical simulations) should develop with time (cf. [260, p. 3128], [368, p. 835]). Because compressive strength and tensile strength are not proportional to each other ([121, p. 87]), development formulations shall be independent of each other. Anyhow, often the tensile strength is set proportional to the compressive strength (cf. text above and below).

[365, p. 39f] highlight a problem which arises when following standards. If one considers, for example, the "Austrian Guideline for Sprayed Concrete" ([289, p. 26f, 92f]; cf. also [19, p. 12]) and the "fib Model Code for Concrete Structures" ([121, p. 87]), the former suggests compressive strength values for an age up to 24 hours (i.e., early age strength), whereas the latter provides an equation to calculate the strength up to an age of 28 days. [20, p. 27] cites the same equation,

²²Here, pure chemical hardening (i.e., strength growth) is addressed. It is not to be confused with strain hardening or softening. (cf. [402, p. 1130])

but highlights that for an age of below three days, tests are required to get more precise values. Thus, there is a gap of 48 hours between the age of 24 hours and of 72 hours. According to [365, p. 39f], in conventional tunnelling, the main part of stress redistributions takes place within this period.

In his publication, Schubert (1988; [358]) does not address the strength of shotcrete material.

Schädlich and Schweiger (2014; [348]) use the formulation proposed by [68]. It requires the compressive strength at 28 days, $f_{c,28}$, and the time-dependent coefficient $\beta_{cc}(t)$. The latter depends on the shotcrete age, t , and on the coefficient s . In the original formulation, s is to be selected depending on the type of cement. In their formulation, they substituted s with the function $s_{strength}$ which depends on the compressive strength at 1 day, $f_{c,1}$, and on $f_{c,28}$. A lower boundary is introduced with 0.5% of $f_{c,28}$. And for $t > 28$ days, the compressive strength is assumed constant. Because the formulation by [68] results in very low strength for $t \lesssim 2$ hours, they cite an alternative approach: for the first 24 hours, use the early age strength by [19, 289]; then, the development follows the approach by [292], which depends on t , $f_{c,1}$, and $f_{c,28}$. For their implementation, they consider several ratios constant during the hardening process, e.g., (values in parentheses are recommendations by [347, Tab. 7, p. 42])

- uniaxial compressive yield stress to uniaxial peak compressive strength, f_{cy}/f_c , ([0.10; 0.25]; dimensionless);
- uniaxial residual compressive strength to uniaxial peak compressive strength, $f_{c,r}/f_c$, (0.1; dimensionless);
- uniaxial residual tensile strength to uniaxial peak tensile strength, $f_{t,r}/f_t$, (0; dimensionless);
- uniaxial peak tensile strength at 28 days to uniaxial peak compressive strength at 28 days, $f_{t,28}/f_{c,28}$, ([0.05; 0.10]; dimensionless).

Note that for the tensile strength, they generally make use of a constant ratio of, e.g., $f_t/f_c = 0.1$. The implementation has been validated with data from [129]. [348, p. 104, 106]

Schütz et al. (2011; [368]) also use the approach by [68]. The relation of the coefficient s to the cement type remains untouched. They use the approach to approximate the development of the compressive and tensile strength with time. Values are set constant for $t < 1$ hour and for $t > 28$ days. Similar to [348], they also make use of some strength ratios set constant during hardening. Data from [371] was used for validation. Tab. 1 in the reference lists some calibrated model parameters. [368, p. 839, 842]

Meschke et al. (1996; [261]) make use of a formulation proposed by [288] to describe the development of the strength in the first 24 hours. It depends on the shotcrete age, t , and on the compressive strength on day one, $f_{c,1}$. For the development for $t > 24$ hours, they use an approach by [292]. Note that this approach is not the same as [348] use (cf. text above). Anyway, the formulation here requires t , $f_{c,1}$, and the compressive strength at 28 days, $f_{c,28}$. They relate the development of the tensile strength to the one of the compressive strength following [63, 292]. It is a non-linear relation with two coefficients only. They use data from [129] for validation. [261, p. 3150ff, 3161]

For the approximation of the development of the compressive strength, Neumer et al. (2017; [276]) adopt the approach by [261] formulated for the development of the Young's modulus (cf. Section 4.9). The second-order polynomial for the development of the strength in the first hours (here: $t \leq 6$ hours) is modified by adding a residual coefficient. This ensures that the strength is non-zero for $t = 0$ (here: 1% of $f_{c,28}$). Note that a monotonic growth in strength results only

if $f_{c,1}/f_{c,28} \geq 0.16$. They, too, consider some strength ratios constant during hardening; for example, the uniaxial peak tensile strength to the uniaxial peak compressive strength, $f_t(t)/f_c(t)$. For all constant strength ratios, they use the value resulting at 28 days. Data from [173] was used for validation. [276, p. 6, 13f]

4.9 Shotcrete deformability

Like for the shotcrete strength (cf. Section 4.8 on p. 63), the stiffness increases with time during the hardening process (cf. Section 4.2 on p. 47). In the capillary pores, hydration products grow, increase their volume, and, by that, stiffen the material's rigidity ([370, p. 284]).

The overall stiffness at a particular time is the sum of the inherent stiffness of the individual constituents of the concrete mix (e.g., aggregates, fluids) and viscoelastic effects ([353, p. 9]). The latter relates to recoverable creep strain (cf. p. 57). According to [63, 71] (cited in [388, p. 31]), the rate of increase is higher for the stiffness than for the strength.

As numerically investigated and described in [156] and discussed also by [388, p. 111f, 155], the deformation rate of the ground determines the importance of having the stiffness development modelled properly. If the ground deforms at a very low rate, the stiffness development of the shotcrete at early ages is of less importance. If the deformation rates are higher, it is more important. Consider here that not only the time-dependent (or creep) characteristics of the ground determine the deformation rate, but also the construction sequences of the tunnel drive and the advance rate of the individual headings (i.e., time-independent advance-related deformations). In this regard, it should not go unmentioned that the loading rate at shotcrete linings usually is much lower than at laboratory tests ([10, p. D-13f]). In that case, because load during the hardening can result in a higher shotcrete strength and stiffness (cf. p. 63), the shotcrete stiffness of the tunnel lining is lower than recorded in the laboratory.

Relating to the utilisation, up to the limit of proportionality, concrete behaves approximately like a linear viscoelastic material ([242, p. 384]). Because of the viscoelastic part, the state of strain depends on the time and loading history (cf. [242, p. 394]). Anyway, at unloading, if the concrete would not age, the body would return to its undeformed state; not instantaneously, but with time. Because of the ageing, some strain is irreversible (cf. Section 4.6.6 on p. 62). For concrete and shotcrete, published values for the limit of proportionality range from 30% to 60% of the uniaxial compressive strength: 30% ([370, p. 287f], [118] in [388, p. 110]), 30 ... 40% ([242, p. 384, 396], [388, p. 113]), <40% ([173] in [388, p. 46]), 40% ([153, p. 9]), 50% ([128, p. 81]), 40 ... 60% ([10, p. D-42]). Published data suggests a variability of the threshold depending on the concrete age (cf. Fig. 2.10 in [388, p. 30]). This may explain the range of the threshold cited here.²³ Performing compression tests, the Young's modulus should be determined within a stress range below the limit of proportionality ([388, p. 31]). At a higher utilisation, the modulus of the virgin loading curve, for example, can be much lower and decreases with increasing utilisation (cf. Fig. E-7.4 and Fig. E-7.5 in [129, p. E-36]). Like for the Young's modulus of rock material (cf. Section 3.2.9 on p. 36), the definition of the Young's modulus (e.g., tangent or secant modulus, virgin loading or unloading-reloading loop) decides upon the resulting value. Consider that [315] (cited in [388, p. 34]) suggests a ratio of the unloading-reloading modulus to the modulus of the

²³The initial concrete mix is more a fluid than a solid. With the setting of the cement paste, solid hydrates form (cf. Section 4.2 on p. 47 and Section 4.7 on p. 62). The share of free (not yet combined) water decreases, whereas the share of solids increases. (Very) Early age concrete features a particular ductility allowing for viscous strain (some elastic, some plastic) without failure. This happens on the microscale. Macroscopically, this viscous strain may be falsely ascribed to microcracking. This problem contributes to the difficulty in determining the limit of proportionality.

virgin loading curve of 1.1 to 1.5; [10, Fig. D-19, p. D-30] reports an average value of 1.27.

For unreinforced concrete, the Young's moduli under compressive and tensile loading are similar ([388, p. 37]). However, for the latter, the limit of proportionality seems to be higher: 60% ([74] in [388, p. 22, 35]), up to the fifth percentile of the characteristic axial tensile strength of concrete, $f_{ctk;0.05}$, ([153, p. 9]), 90% ([118] in [368, p. 837]); note that the publications refer to mature concrete. The threshold probably also varies depending on the shotcrete age. According to [52] (cited in [388, p. 36]), under tension, the behaviour of unreinforced and reinforced shotcrete is similar, but the reinforcement makes the composite less brittle once the loading passes the yield limit.

4.9.1 Poisson's ratio

According to experimental results, the Poisson's ratio, ν , does not change significantly with time (i.e., with the degree of hydration) ([214] in [401, p. 787]). [155, p. 695] also report that the ratio may be considered constant at very early ages. However, other investigations show that it significantly changes with the shotcrete age and tends to reach an asymptotic value after a few days (cf. Fig. 2.12 in [388, p. 32]; the graph plots data from [21, 209]). Similarly, [353, p. 15] states for concrete that the ratio at very early ages notably differs from the ratio of mature concrete. The largest differences exist when comparing with the ratio before strength growth starts.

[388, Tab. 5.1, p. 110] declares a constant value of 0.2 a common value for the design of shotcrete linings in soft to blocky ground. [74] (cited in [388, Tab. 5.1, p. 110]) limits the use of this value to a shotcrete state being farther away from failure. It is further assumed that the elastic Poisson's ratios in the compressive and tensile regime are similar ([388, p. 37]).

The approach by Schubert (1988; [358]) is one-dimensional and does not require a Poisson's ratio. Schütz et al. (2011; [368, p. 839]) explicitly state that they set the Poisson's ratio constant. Schädlich and Schweiger (2016; [347, Tab. 7, p. 42]) recommend a value of 0.15 to 0.25. Meschke et al. (1996; [261, p. 3150]) used either 0.25 or 0.3 for their analyses.

4.9.2 Empirical approximation

Schubert (1998; [358]) approximates the development of the Young's modulus, $E(t)$, with a formulation proposed by [67]. It depends only on the shotcrete age, t , and the Young's modulus at 28 days, E_{28} . [358, p. 151]

Schädlich and Schweiger (2014; [348]) use the formulation proposed by [68, p. 51f]. However, instead of using the time-dependent coefficient $\beta_E(t)$ suggested by the standard, they rather use the same coefficient $\beta_{cc}(t)$ as for the formulation of the compressive strength (cf. p. 65). The resulting formulation depends on the shotcrete age, t , the Young's modulus at 28 days, E_{28} , and on the coefficient s . In the original formulation, the latter depends on the type of cement. In their formulation, they substituted s with the function s_{stiff} which depends on the Young's modulus at 1 day, E_1 , and on E_{28} . Note that they consider the Young's modulus constant for $t < 1$ day and for $t > 28$ days. The formulation has been validated using data from [71]. For E_1/E_{28} , they recommend a value between 0.5 and 0.7 ([347, Tab. 7, p. 42]). [348, p. 106]

Schütz et al. (2011; [368]) use also the formulation proposed by [68, p. 52]. But in contrast to [348], they used the relating time-dependent coefficient $\beta_E(t)$. And they stick to the dependency of s on the type of cement. Here, too, the Young's modulus is set constant for $t < 1$ day and for $t > 28$ days. Data from [371] served for the validation. [368, p. 839, 842]

According to Meschke et al. (1996; [261]), the formulation they use to account for the time-dependent Young's modulus bases on the one given in [68]. However, the formulation is rather similar to the one [358] uses who refers to [67]. Anyway, they modified the formulation by introducing a second-order polynomial for the development of the Young's modulus in the first eight hours. They also set the Young's modulus constant for a shotcrete age of over 28 days. The final formulation depends on the concrete age, t , the Young's modulus at day one, E_1 , the Young's modulus at 28 days, E_{28} , and on some constants. If E_1 is unknown, $E_1/E_{28} = 0.6$ is assumed. Data from [122, 173] are used to validate the formulation. Tab. 1 in the reference lists some values for E_1/E_{28} . [261, p. 3148ff, 3159f]

As mentioned already in Section 4.6.4 (p. 57), Neuner et al. (2017; [276]) utilise a modified version of the solidification theory by [29]. The volume function $v(t)$ (cf. p. 59) describes also the development of the Young's modulus with time. It required a modification of $v(t)$, as the development in the first hours did not yield satisfying results. The final implementation requires, for example, the shotcrete age, t , the Young's modulus at day one, E_1 , and some coefficients. The validation was performed using data from [173]. [276, p. 6ff]

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