5 Materials

5.1 Concrete

5.1.1 General and range of applicability

The subsequent clauses apply to structural concrete with normal and lightweight aggregates so composed and compacted as to retain no appreciable amount of entrapped air other than intentionally entrained air.

Though the relations in principle also apply for heavyweight concrete, special consideration may be necessary for such concretes.

Concerning compressive strength this Model Code covers concretes up to a characteristic strength of 120 MPa, i.e. normal strength (NSC) and high strength concrete (HSC) are dealt with; see sub-clause 5.1.4.

As a first approximation, the subsequent relations also apply for self-compacting concrete unless additional information is given.

The given relations apply roughly also for green concrete as far as the composition of those concretes deviates from the composition of ordinary structural concrete only by the replacement of a certain amount of cement by fly ash, silica fume, blast furnace slag and natural pozzolans, i.e. chemical reactive substitutes.

The constitutive relations given in these clauses are applicable for the entire range of concrete grades dealt with in this Model Code.

Throughout clause 5.1 the following sign conventions are maintained which may differ from those used in other parts of the Model Code:

– material properties are positive or to be used in absolute terms, e.g. compressive strength, $f_{cm} = |f_{cm}|$.

– tensile stresses and tensile strains (elongations) are positive.

– compressive stresses and compressive strains (contractions) are negative.

– where multiaxial stress states are considered, $\sigma_1 > \sigma_2 > \sigma_3$ is valid for the principal stresses.

The definitions for concrete given within this Model Code comply with ISO 22965-1 “Concrete – Part 1: Methods of specifying and guidance to the specifier” and ISO 22965-2 “Concrete – Part 2: Specification of constituent materials, production of concrete and conformity of concrete”.

Green concrete may be characterised by having a significant improved sustainability compared to ordinary structural concrete. This holds particularly true, if the CO_2 emission associated with a concrete is significantly reduced and/or the energy necessary to produce the concrete and its constituent materials is considerably lower than for ordinary concrete. So far no general accepted limiting values and benchmarks exist.

Green concrete may be produced e.g. by the replacement of cement by chemical reactive or inert fine materials, by a significant reduction of the total binder content and also by the replacement of the aggregates, applying e.g. recycled concrete. Further, environmental harmful substances being possibly contained in concrete making materials, e.g. also in additions and admixtures, have to be excluded.

There is no detailed information available on the constitutive behaviour of green concrete. Hence, an expert has to evaluate the structural behaviour in view of the composition of green concrete.

The information given in the sub-clauses 5.1.4, 5.1.5, 5.1.7 and 5.1.11.2 is valid for monotonically increasing compressive stresses or strains at a constant range of approximately 1 MPa/s < $|\dot{\sigma}_c|$ < 10^7 MPa/s and $30 \cdot 10^{-6}$ s^{-1} < $|\dot{\varepsilon}_c|$ < $3 \cdot 10^2$ s^{-1}, respectively.

For tensile stresses or strains it is valid for 0.03 MPa/s < $\dot{\sigma}_{ct}$ < 10^7 MPa/s and $1 \cdot 10^{-6}$ s^{-1} < $\dot{\varepsilon}_{ct}$ < $3 \cdot 10^2$ s^{-1}, respectively.

5.1.2 Classification by strength

In this Model Code concrete is classified on the basis of its compressive strength. Design is based on a grade of concrete which corresponds to a specific value of its characteristic compressive strength f_{ck} as defined in subclause 5.1.4.

Production control and attestation of conformity of concrete shall be in accordance with ISO 22965-2.

The specification of concrete given to the concrete producer shall include all assumptions made during the design as well as those properties needed to ensure that the needs during transportation and execution on the site are considered.

Concrete grades for normal weight concrete (C) can be selected from the following series:

C12, C20, C30, C40, C50, C60, C70, C80, C90, C100, C110, C120

In some countries also intermediate concrete grades are often used:

C16, C25, C35, C45, C55 and LC35, LC45, LC55, respectively.

Concrete grades for lightweight aggregate concrete (LC) can be selected from the following series:

LC8, LC12, LC16, LC20, LC25, LC30, LC40, LC50, LC60, LC70, LC80

The numbers behind the symbols C and LC denote the specified characteristic strength f_{ck} in MPa.

Unless specified otherwise, the compressive strength of concrete as well as the tensile strength of concrete is understood as the strength value obtained at a concrete age of 28 days.

5.1.3 Classification by density

This classification corresponds to ISO 22965. Concrete is classified in 3 categories of oven-dry density:

– Lightweight aggregate concrete (800 – 2000 kg/m^3)

– Normal weight concrete (> 2000 – 2600 kg/m^3)

– Heavy weight concrete (> 2600 kg/m^3)

With increasing compressive strength concrete generally contains more cement and less water resulting in a higher density of HSC compared to NSC. Also HSC members may contain more reinforcement than NSC members. Nevertheless the related density values may vary within relatively wide limits depending on mix composition and density of aggregate materials (both may vary between countries), reinforcement ratio and air content.

The values given in Table 5.1-1 assume an air content of 2 %. A change of air content by 1 % results in a density change of 1 %. Where a higher accuracy is required than provided by Table 5.1-1 the concrete density may be determined experimentally e.g. according to ISO 1920-5.

For ordinary normal weight concrete, both, normal strength (NSC) and high strength concrete (HSC), the density may be estimated from Table 5.1-1.

Table 5.1-1: Density of NSC and HSC, plain and with different steel reinforcement ratios [kg/m^3]

Reinforcement ratio	C30 (w/c ≈ 0.65)	C80 (w/c ≈ 0.35)	C120 (w/c ≈ 0.25)
0.0 %	2350	2450	2500
1.0 %	2400	2500	2550
2.0 %	2450	2550	2600

The values given in Table 5.1-2 are valid for plain and reinforced lightweight aggregate concrete with usual percentages of reinforcement. These values may be used for design purposes in calculating self-weight or imposed permanent loading.

Where a higher accuracy is required than provided by Table 5.1-2 the concrete density may be determined experimentally e.g. according to ISO 1920-5.

The classification of lightweight aggregate concrete according to its oven-dry density is given in Table 5.1-2.

Table 5.1-2: Density classes and corresponding design densities of lightweight aggregate concrete

Density classes		D1.0	D1.2	D1.4	D1.6	D1.8	D2.0
Oven-dry density ρ [kg/m^3]		801 - 1000	1001 - 1200	1201 - 1400	1401 - 1600	1601 - 1800	1801 - 2000
Density [kg/m^3]	Plain concrete	1050	1250	1450	1650	1850	2050
	Reinforced concrete	1150	1350	1550	1750	1950	2150

5.1.4 Compressive strength

The reference compressive strength of the concrete according to this Model Code is measured on cylinders 150/300 mm in accordance with ISO 1920-3; for classification see sub-clause 5.1.2.

For special requirements or in national codes test specimens other than cylinders 150/300 mm and stored in other environments may be used to specify the concrete compressive strength. In such cases conversion factors should either be determined experimentally or, when given in national codes, used accordingly for a given category of testing equipment.

In the case when concrete cubes 150 mm are used, the characteristic strength values given in Table 5.1-3 shall be obtained for the various concrete grades of normal weight concrete. Table 5.1-4 gives the corresponding characteristic strength values for lightweight aggregate concrete.

Table 5.1-3: Characteristic strength values of normal weight concrete [MPa]

Concrete grade	C12	C20	C30	C40	C50	C60	C70	C80	C90	C100	C110	C120
f_{ck}	12	20	30	40	50	60	70	80	90	100	110	120
$f_{ck,cube}$	15	25	37	50	60	75	85	95	105	115	130	140

Table 5.1-4: Characteristic strength values of lightweight aggregate concrete [MPa]

Concrete grade	LC8	LC12	LC16	LC20	LC25	LC30	LC40	LC50	LC60	LC70	LC80
f_{lck}	8	12	16	20	25	30	40	50	60	70	80
$f_{lck,cube}$	9	13	18	22	28	33	44	55	66	77	88

If there is no test procedure agreed or given in national guidelines, tests may be performed according to RILEM CPC 7, 1975.

Although the uniaxial tensile testing is the most appropriate method to determine the tensile strength of concrete, it is used almost exclusively in research because of the experimental difficulties in performing such experiments. Therefore, in many instances the splitting tensile strength or flexural tensile strength are determined; refer to sub-clause 5.1.5.1 below.

When testing tensile strength special attention should be paid to possible effects of moisture gradients.

In analysis and design of concrete structures the characteristic compressive strength f_{ck} [MPa] is applied. This value may be derived from strength test by the criterion that 5 % of all possible strength measurements for the specified concrete may be expected to fall below the value f_{ck}.

For some verifications in design or for an estimate of other concrete properties it is necessary to refer to a mean value of compressive strength f_{cm} (or f_{lcm} for lightweight aggregate concrete) associated with a specific characteristic compressive strength f_{ck} (or f_{lck} for lightweight aggregate concrete). In this case f_{cm} and f_{lcm} may be estimated from Eq. (5.1-1) and (5.1-2), respectively:

$$f_{cm} = f_{ck} + \Delta f \tag{5.1-1}$$

$$f_{lcm} = f_{lck} + \Delta f \tag{5.1-2}$$

where:

Δf = 8 MPa

5.1.5 Tensile strength and fracture properties

5.1.5.1 Tensile strength

The tensile strength of the concrete and the term “tensile strength”, unless stated otherwise in this code, refer to the uniaxial tensile strength f_{ct} determined in related experiments.

In the absence of the experimental data the mean value of tensile strength f_{ctm} in [MPa] may be estimated for normal weight concrete from the characteristic compressive strength f_{ck}:

$$f_{ctm} = 0.3 \cdot (f_{ck})^{2/3} \quad \text{concrete grades} \leq \text{C50} \qquad (5.1\text{-}3a)$$

$$f_{ctm} = 2.12 \cdot ln\left(1 + 0.1 \cdot (f_{ck} + \Delta f)\right) \quad \text{concrete grades} > \text{C50} \qquad (5.1\text{-}3b)$$

where:

f_{ck} is the characteristic compressive strength according to Table 5.1-3 in [MPa]

Δf = 8 MPa

The lower and upper bound values of the characteristic tensile strength $f_{ctk,max}$ and $f_{ctk,min}$ may be estimated using eqs. 5.1-4 and 5.1-5, respectively:

$$f_{ctk,min} = 0.7 \cdot f_{ctm} \qquad (5.1\text{-}4)$$

$$f_{ctk,max} = 1.3 \cdot f_{ctm} \qquad (5.1\text{-}5)$$

To estimate a mean value of the tensile strength f_{lctm} for lightweight aggregate concrete, f_{ctm} according to Eq. (5.1-3) shall be multiplied by a reduction factor η_l according to Eq. (5.1-6):

$$\eta_l = (0.4 + 0.6 \cdot \rho/2200) \qquad (5.1\text{-}6)$$

where:

ρ is the oven-dry density of the lightweight aggregate concrete in [kg/m³].

The lower and upper bound values of the characteristic tensile strength $f_{lctk,max}$ and $f_{lctk,min}$ may be estimated for lightweight aggregate concrete using eqs. 5.1-4 and 5.1-5, respectively, replacing f_{ctm} by f_{lctm}.

If the tensile strength is measured as splitting tensile strength $f_{ct,sp}$ or as flexural tensile strength $f_{ct,fl}$ a conversion factor A should be determined by means of uniaxial tension tests.

Eq. (5.1-3) was derived by evaluating available data from axial tension and compression tests. The data from splitting and flexural tests were not considered in order to avoid evident uncertainties resulting from indirect testing (refer to *fib* Bulletin 42).

Table 5.1-5 gives tensile strength values for normal weight concrete estimated from the characteristic compressive strength f_{ck} according to eqs. 5.1-3 to 5.1-5.

Table 5.1-5: Tensile strength for different concrete grades [MPa]

Concrete grade	C12	C20	C30	C40	C50	C60	C70	C80	C90	C100	C110	C120
f_{ctm}	1.6	2.2	2.9	3.5	4.1	4.4	4.6	4.8	5.0	5.2	5.4	5.6
$f_{ctk,min}$	1.1	1.5	2.0	2.5	2.9	3.1	3.2	3.4	3.5	3.7	3.8	3.9
$f_{ctk,max}$	2.0	2.9	3.8	4.6	5.3	5.7	6.0	6.3	6.6	6.8	7.0	7.2

If such conversion factors are not available the mean axial strength f_{ctm} may be estimated from the mean splitting strength $f_{ct,sp}$ acc. to Eq. (5.1-7):

$$f_{ctm} = A_{sp} \cdot f_{ct,sp} \qquad (5.1\text{-}7)$$

In existing national and international codes and standards values of the conversion factor A_{sp} may be found which vary from 0.67 to 0.95. However, comprehensive new research results show that this factor is beyond 1. As a compromise solution A_{sp} = 1.0 has been chosen.

Eq. (5.1-8) was deduced from fracture mechanics considerations. In CEB-FIP MC 1990 a coefficient α_{fl} = 0.06 was proposed for normal strength concrete. Since the ratio of flexural strength to axial tensile strength of concrete $f_{ct,fl}/f_{ctm}$ decreases for a given beam depth as the concrete becomes more brittle, α_{fl} depends on the brittleness of the concrete and decreases as brittleness increases. This means that for high strength concrete and for lightweight aggregate concrete lower values of the coefficient α_{fl} can be expected.

where:

$f_{ct,sp}$ is the mean value of splitting tensile strength determinedaccording to ISO 4108

A_{sp} = 1.0

The same conversion factor A_{sp} = 1.0 may be used for lightweight aggregate concrete.

In order to estimate the mean axial tensile strength f_{ctm} from the mean flexural tensile strength $f_{ct,fl}$ Eq. (5.1-8) can be used:

$$f_{ctm} = A_{fl} \cdot f_{ct,fl} \tag{5.1-8}$$

where:

$f_{ct,fl}$ is the mean flexural tensile strength

$$A_{fl} = \frac{\alpha_{fl} \cdot h_b^{0.7}}{1 + \alpha_{fl} \cdot h_b^{0.7}}$$

h_b is beam depth [mm]

α_{fl} = 0.06

5.1.5.2 Fracture energy

The fracture mode of concrete subjected to tension allows the application of fracture mechanics concepts, i.e. energy considerations. In those concepts the fracture energy of concrete G_F is often used as a materials characteristic to describe the resistance of concrete subjected to tensile stresses.

G_F should best be determined from uniaxial tension tests. Most frequently, however, indirect tests, first of all three-point bend tests on notched beams are used, which are easier to perform.

For normal weight concrete the fracture energy depends primarily on the water-cement ratio, the maximum aggregate size and the age of concrete. Curing conditions also have a significant effect on experimentally determined G_F values. Further, G_F is affected by the size of a structural member and in particular by the depth of the ligament above a crack or a notch. The fracture energy of high strength normal weight concrete is also influenced by the above-mentioned parameters, however not to the same extent as in the case of

The fracture energy of concrete G_F [N/m], defined as the energy required to propagate a tensile crack of unit area, should be determined by related tests.

In the absence of experimental data G_F in [N/m] for ordinary normal weight concrete may be estimated from Eq. (5.1-9):

$$G_F = 73 \cdot f_{cm}^{0.18} \tag{5.1-9}$$

where:

f_{cm} is the mean compressive strength according to Eq. (5.1-1) in [MPa] normal strength concrete. The aggregate type and content seem to affect the fracture energy of concrete much stronger than the size of aggregates. This phenomenon is caused by the transition from the interfacial fracture to the trans-aggregate fracture. For high strength concrete the effect of curing conditions on G_F is somewhat less pronounced than for normal strength concrete, but it is still significant (refer to *fib* Bulletin 42).

The knowledge of fracture mechanisms of lightweight aggregate concrete (LWAC) is still insufficient, and the dependence of fracture energy of LWAC on different parameters (density, types of aggregates, strength etc.) must be addressed to future research. LWAC is notch sensitive (most important to this sensitivity are eigen-stresses because of moisture gradients). The maximum crack opening depends on the kind of matrix and the kind of aggregates, respectively. Thus, tests to determine fracture energy and softening behaviour are mandatory if related information on LWAC should be used for analysis and design.

As an approximation Eq. (5.1-10) may be used for estimating the fracture energy of lightweight aggregate concrete:

$$G_{F,l} = G_{FoA} + 16 \cdot f_{lctm} \tag{5.1-10}$$

where:

$G_{F,l}$ is obtained in [N/m]

G_{FoA} = 24 N/m for lightweight aggregate concrete with normal weight sand

= 0 for lightweight aggregate concrete with lightweight sand

5.1.6 Strength under multiaxial states of stress

This failure criterion is one among several acceptable formulations. It has been chosen since it is not too difficult to use and agrees well with test data. For further details and the range of applicability of Eq. (5.1-11) refer to ‘Concrete under multiaxial states of stress - constitutive equations for practical design’, CEB Bulletin 156, Lausanne, 1983 and to Ottosen, N., ‘A Failure Criterion for Concrete’, Journal Engineering Mechanics Division, ASCE, Vol. 103, EM4, August 1977.

The invariants of the stress tensor (I_1) and the stress deviators (J_2 and J_3) used in eqs. 5.1-11 to 5.1-13 may be calculated as follows:

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3$$

$$J_2 = \frac{1}{6}\left[\left(\sigma_1 - \sigma_2\right)^2 + \left(\sigma_2 - \sigma_3\right)^2 + \left(\sigma_3 - \sigma_1\right)^2\right]$$

$$J_3 = \left(\sigma_1 - \sigma_m\right) \cdot \left(\sigma_2 - \sigma_m\right) \cdot \left(\sigma_3 - \sigma_m\right)$$

$$\sigma_m = \left(\sigma_1 + \sigma_2 + \sigma_3\right)/3$$

The mean value of strength under multiaxial states of stress may be estimated from the failure criterion given by Eq. (5.1-11).

For normal weight and self-compacting concrete Eq. (5.1-11a) is valid:

$$\alpha \frac{J_2}{f_{cm}^2} + \lambda \frac{\sqrt{J_2}}{f_{cm}} + \beta \frac{I_1}{f_{cm}} - 1 = 0 \tag{5.1-11a}$$

and for lightweight aggregate concrete f_{cm} has to be replaced by f_{lcm}:

$$\alpha \frac{J_2}{f_{lcm}^2} + \lambda \frac{\sqrt{J_2}}{f_{lcm}} + \beta \frac{I_1}{f_{lcm}} - 1 = 0 \tag{5.1-11b}$$

where

$$\lambda = c_1 \cdot \cos\left[\frac{1}{3} \cdot \arccos\left(c_2 \cdot \cos 3\theta\right)\right] \tag{5.1-12}$$

$$\cos 3\theta = \frac{3\sqrt{3}}{2} \cdot \frac{J_3}{J_2^{3/2}} \tag{5.1-13}$$

The stress coordinates σ_o and τ_o (octahedron stresses) may be calculated as follows:

$$\sigma_o = I_1/3 \qquad \tau_o = \sqrt{\frac{2}{3} \cdot J_2}$$

Note that f_c and f_{c2c} are defined as positive values; all other compressive stresses and strengths are negative values.

No standardized test method is available for determining the multiaxial strength. During the test the load has to be applied by special test devices, which follow the deformation of the specimen and prevent parts of the load being introduced through friction into the lateral load application system. Detailed information is available in: *Gerstle et al.: Behavior of concrete under multiaxial stress.* Journal of the Engineering Mechanics Division, *Proceedings of the ASCE, Vol. 106, No. EM6, Dec. 1980, pp. 1383-1403.*

In the absence of experimental data the biaxial compressive strength and the tri-axial compressive strength at one point on the compressive meridian may be estimated from the uniaxial compressive strength. Note that no consolidated experience exist for a stress level above σ_{com} = -240 MPa for normal weight concrete and σ_{lcom} = -60 MPa for lightweight aggregate concrete. No sufficient experimental data exist for self compacting concrete.

$$f_{c2c} = \left(1.2 - \frac{f_c}{1000}\right) \cdot f_c$$

where $f_c = f_{cm}$ for $f_{c2c} = f_{c2cm}$

$f_c = f_{ck}$ for $f_{c2c} = f_{c2ck}$

$f_c = f_{lcm}$ for $f_{c2c} = f_{lc2cm}$

$f_c = f_{lck}$ for $f_{c2c} = f_{lc2ck}$

The parameters J_2, J_3 and I_1 in eqs. 5.1-11 to 5.1-13 represent the invariants of the stress deviator and stress tensor, respectively, characterizing the state of stress considered.

The coefficients α, β, c_1 and c_2 are material parameters which depend on the uniaxial compressive strength f_{cm} (or f_{lcm} for lightweight aggregate concrete), the uniaxial tensile strength f_{ctm} (or f_{lctm}), the biaxial compressive strength f_{c2cm} (or f_{lc2cm}) and the tri-axial compressive strength at one point on the compressive meridian ($\sigma_1 = \sigma_2 > \sigma_3$) described by σ_{com} and τ_{com} (or σ_{lcom} and τ_{lcom}). To determine these coefficients the parameters given in Eq.(5.1-14) have to be calculated.

$$k = \frac{f_{ctm}}{f_{cm}} \quad f_{2c} = \frac{f_{c2cm}}{f_{cm}} \quad x = \frac{\sigma_{com}}{f_{cm}} \quad y = \frac{\tau_{com}}{f_{cm}} \quad h = -\frac{\sqrt{2} \cdot x + y}{\frac{y}{\sqrt{2}} - \frac{1}{3}} \tag{5.1-14}$$

$$\alpha = \frac{h \cdot \beta - \sqrt{2}}{y} \qquad \beta = \frac{\sqrt{2} - \frac{3 \cdot y}{k \cdot f_{2c}}}{h - \frac{9 \cdot y}{f_{2c} - k}} \tag{5.1-15}$$

$$\lambda_c = \lambda(\theta = 60°) = \left(1 - \frac{h}{3 \cdot y}\right) \cdot \sqrt{3} \cdot \beta + \sqrt{3} + \frac{\sqrt{2}}{\sqrt{3} \cdot y} \tag{5.1-16}$$

$$\lambda_t = \lambda(\theta = 0°) = \left(2 \cdot \sqrt{3} - \frac{f_{2c} \cdot h}{\sqrt{3} \cdot y}\right) \cdot \beta + \frac{\sqrt{3}}{f_{2c}} + \frac{\sqrt{2} \cdot f_{2c}}{\sqrt{3} \cdot y} \tag{5.1-17}$$

$$\tau_{com} = \left[185 - 180 \cdot \frac{f_{cm}}{100} + 260 \cdot \left(\frac{f_{cm}}{100}\right)^2 - 84 \cdot \left(\frac{f_{cm}}{100}\right)^3 \right]$$

$$\tau_{lcom} = \left[250 \cdot \frac{f_{lcm}}{100} - 460 \cdot \left(\frac{f_{lcm}}{100}\right)^2 + 310 \cdot \left(\frac{f_{lcm}}{100}\right)^3 \right]$$

$$\tau_{cok} = \left(0.8 + \frac{f_{ck}}{1000}\right) \cdot \tau_{com} \text{ and } \tau_{lcok} = \left(0.8 + \frac{f_{lck}}{1000}\right) \cdot \tau_{lcom}, \text{ respectively}$$

for $\sigma_{com} = \sigma_{cok} = -240$ MPa and $\sigma_{lcom} = \sigma_{lcok} = -60$ MPa, respectively

with f_{cm}, f_{ck}, f_{lcm} and f_{lck} in MPa.

The coefficients for normal weight concrete given in Figure 5.1-1 are the results of these equations.

$$c_1 = [2 \cdot \cos\theta - 1] \cdot \lambda_t + 4 \cdot [1 - \cos\theta] \cdot \lambda_c \quad \text{for} \quad \frac{\lambda_c}{\lambda_t} \le \frac{1}{2} \tag{5.1-18a}$$

$$c_1 = \frac{\lambda_c}{\cos\left(\frac{\pi}{3} - \frac{1}{3} \cdot \arccos(c_2)\right)} \quad \text{for} \quad \frac{\lambda_c}{\lambda_t} \ge \frac{1}{2} \tag{5.1-18b}$$

$$c_2 = 1 \quad \text{for} \quad \frac{\lambda_c}{\lambda_t} \le \frac{1}{2} \tag{5.1-19a}$$

$$c_2 = \cos\left\{3 \cdot \arctan\left[\frac{\left(2 \cdot \frac{\lambda_c}{\lambda_t} - 1\right)}{\sqrt{3}}\right]\right\} \quad \text{for} \quad \frac{\lambda_c}{\lambda_t} \ge \frac{1}{2} \tag{5.1-19b}$$

To estimate a characteristic multiaxial strength, in eqs. 5.1-11 and 5.1-14 the mean values of uniaxial compressive and tensile strength, biaxial and triaxial compressive strength shall be substituted by the characteristic values of these strengths.

The strength of concrete under biaxial states of stress ($\sigma_3 = 0$) may be estimated from the same criteria as given in eqs. 5.1-11 to 5.1-19.

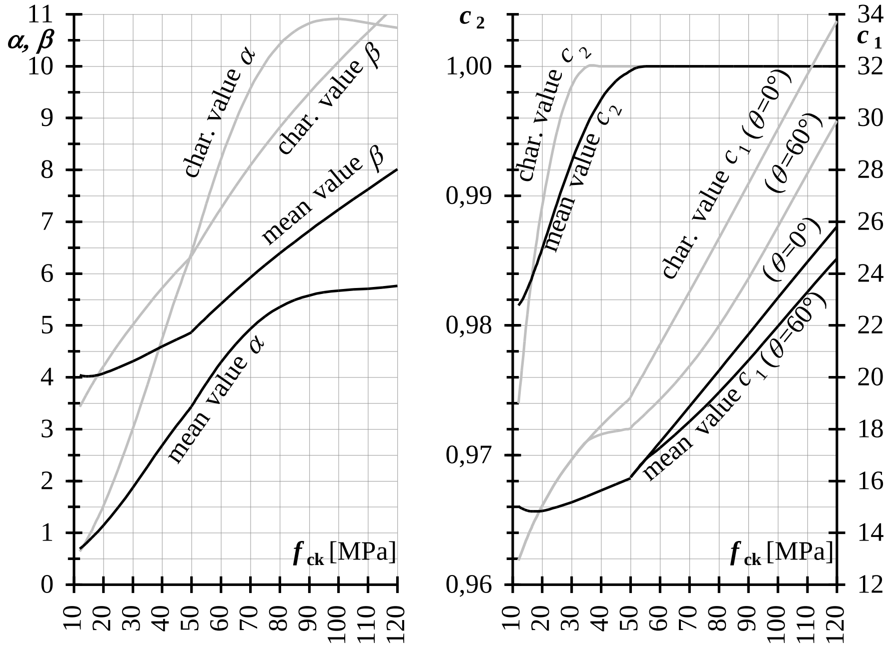

Figure 5.1-1: Coefficients for Eq.(5.1-11), normal weight concrete

5.1.7 Modulus of elasticity and Poisson's ratio

5.1.7.1 Range of application

The information given in this clause is valid for monotonically increasing compressive stresses or strains at a rate of $\dot{\sigma} \approx 0.6 \pm 0.4$ MPa/s or $\dot{\varepsilon} \approx 15 \cdot 10^{-6}$ s^{-1}, respectively. For tensile stresses or strains it is valid for $\dot{\sigma} \approx 0.06$ MPa/s or $\dot{\varepsilon} \approx 1.5 \cdot 10^{-6}$ s^{-1}, respectively.

5.1.7.2 Modulus of elasticity

The modulus of elasticity E_{ci} as obtained from eqs. 5.1-20 and 5.1-21 is defined as the tangent modulus of elasticity at the origin of the stress-strain diagram. It is approximately equal to the slope of the secant of the unloading branch for rapid unloading and does not include initial plastic deformations. It has to be used for the description of the stress-strain diagrams for uniaxial

Values for the modulus of elasticity for normal weight concrete with natural sand and gravel can be estimated from the specified characteristic strength using Eq. (5.1-20):

compression and uniaxial tension according to sub-clauses 5.1.8.1 and 5.1.8.2, respectively, as well as for an estimate of creep acc. to Eq. (5.1-60), sub-clause 5.1.9.4.3. The reduced modulus of elasticity E_c according to Eq. (5.1-23) includes some irreversible strains.

The elastic deformations of concrete largely depend on its composition (especially type of aggregates). The values given in this Model Code (see Table 5.1-7) should be regarded as indicative for general applications. However, the modulus of elasticity should be specifically assessed or experimentally determined if the structure is likely to be sensitive to deviations from these general values. In this context, reference is made to RILEM CPC 8 (1975); a similar test procedure is under development (ISO/DIS 1920-10).

Compared to the use of quartzite aggregates the modulus of elasticity can be increased by 20 % or decreased by 30 % only by changing the type of aggregate. Eq. (5.1-20) and Table 5.1-6 give the qualitative changes α_E in the modulus of elasticity for different types of aggregate.

Table 5.1-6: Effect of type of aggregates on modulus of elasticity

Types of aggregate	α_E	$E_{c0} \cdot \alpha_E$ [MPa]
Basalt, dense limestone aggregates	1.2	25800
Quartzite aggregates	1.0	21500
Limestone aggregates	0.9	19400
Sandstone aggregates	0.7	15100

The modulus of elasticity E_{ci} does not include the initial plastic strain due to its definition. While the limit for the stress σ_c reached in the SLS is set to $\sigma_c = -0.4 \cdot f_{cm}$ this stress level gives an upper limit for the reduction factor α_i (Figure 5.1-2, Eq. (5.1-23)). This factor $\alpha_i = E_c/E_{ci}$ is increasing with increasing concrete strength. For concrete grades higher than C80 the difference between first loading up to $\sigma_c = -0.4 \cdot f_{cm}$ and the unloading branch is smaller

$$E_{ci} = E_{c0} \cdot \alpha_E \cdot \left(\frac{f_{ck} + \Delta f}{10} \right)^{1/3} \tag{5.1-20}$$

where

E_{ci} is the modulus of elasticity in [MPa] at concrete age of 28 days

f_{ck} is the characteristic strength in [MPa] according to sub-clause 5.1.4

Δf = 8 MPa

E_{c0} = $21.5 \cdot 10^3$ MPa

α_E is 1.0 for quartzite aggregates. For different types of aggregate qualitative values for α_E can be found in Table 5.1-6.

Where the actual compressive strength of concrete at an age of 28 days f_{cm} is known, E_{ci} may be estimated from Eq. (5.1-21):

$$E_{ci} = E_{c0} \cdot \alpha_E \cdot \left(\frac{f_{cm}}{10} \right)^{1/3} \tag{5.1-21}$$

The modulus of elasticity for lightweight aggregate concrete E_{lci} can be estimated from Eq. (5.1-22):

$$E_{lci} = \eta_E \cdot E_{ci} \tag{5.1-22}$$

where:

$$\eta_E = \left(\frac{\rho}{2200} \right)^2$$

ρ is the oven-dry density of the lightweight aggregateconcrete in [kg/m³]

E_{ci} is the modulus of elasticity in [MPa] according to Eq. (5.1-20) or Eq. (5.1-21); here α_E = 1.0 for all types of light-weight aggregates

Where only an elastic analysis of a concrete structure is carried out, a reduced modulus of elasticity E_c according to Eq. (5.1-23) should be used in order to account for initial plastic strain, causing some irreversible deformations.

$$E_c = \alpha_i \cdot E_{ci} \tag{5.1-23}$$

than 3 % and may be neglected.

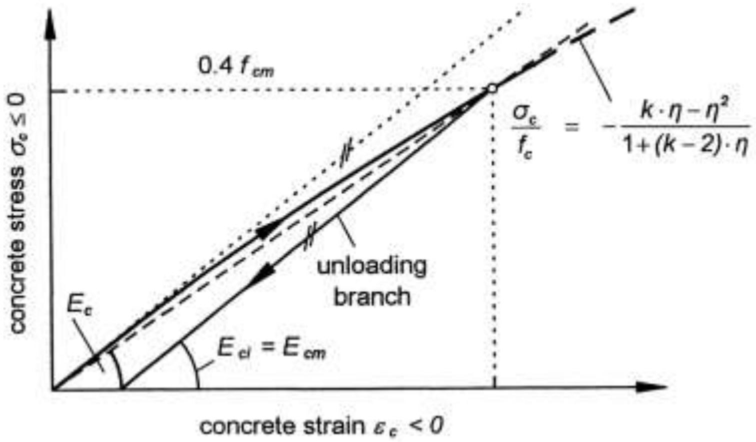

Figure 5.1-2: Definition of different moduli of elasticity (according to fib Bulletin 42)

Note that E_{ci} is considered as the mean value of the tangent modulus of elasticity; hence $E_{ci} = E_{cm}$.

E_c for normal weight concrete and E_{lc} for lightweight aggregate concrete are defined as the secant value of the modulus of elasticity.

where:

$$\alpha_i = 0.8 + 0.2 \cdot \frac{f_{cm}}{88} \leq 1.0 \tag{5.1-24}$$

Values of the tangent modulus E_{ci} and the reduced modulus E_c for different concrete grades are given in Table 5.1-7.

Table 5.1-7: Tangent modulus and reduced modulus of elasticity

Concrete grade	C12	C20	C30	C40	C50	C60	C70
E_{ci} [GPa]	27.1	30.3	33.6	36.3	38.6	40.7	42.6
E_c [GPa]	22.9	26.2	29.7	33.0	36.0	38.9	41.7
α_i	0.845	0.864	0.886	0.909	0.932	0.955	0.977

Concrete grade	C80	C90	C100	C110	C120
E_{ci} [GPa]	44.4	46.0	47.5	48.9	50.3
E_c [GPa]	44.4	46.0	47.5	48.9	50.3
α_i	1.0	1.0	1.0	1.0	1.0

The modulus of elasticity for lightweight aggregate concrete E_{lc} can be estimated by multiplying E_c with the reduction factor η_E given in Eq. (5.1-22):

$$E_{lc} = \eta_E \cdot E_c \tag{5.1-25}$$

5.1.7.3 Poisson's ratio

For a range of stresses -0.6 · $f_{ck} < \sigma_c <$ 0.8 · f_{ctk} the Poisson's ratio of concrete ν_c ranges between 0.14 and 0.26. Regarding the significance of ν_c for the design of members, especially the influence of crack formation at the ULS, the estimation of ν_c = 0.20 meets the required accuracy.

The value of ν_c = 0.20 is also applicable for lightweight aggregate concrete.

5.1.8 Stress-strain relations for short-term loading

5.1.8.1 Compression

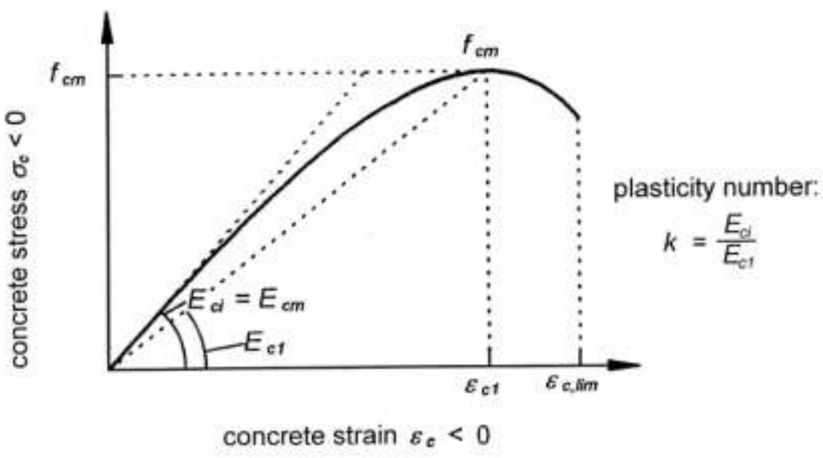

Figure 5.1-3: Schematic representation of the stress-strain relation for uniaxial compression (according to fib Bulletin 42)

The descending part of the stress-strain curve is strongly depending on the specimen or member geometry, the boundary conditions and the possibilities for load redistribution in the structure. In tests a strong influence of the rigidity of the used testing device can be observed. During the softening process micro-cracking occurs in a fracture zone of a limited length and width. One single fracture zone is supposed to be decisive for the failure of a certain member. The stress in the fracture zone drops down with a shear displacement in local shear bands of $w_c \approx 0.5$ mm. The ultimate strain $\varepsilon_{c,lim}$ is caused by the displacement w_c related to a certain length which is given in Figure 4-4 in *fib* Bulletin 42. The descending portion of the stress-strain relation is size dependent and therefore not only a material property (see Figure 4-5 in *fib* Bulletin 42).

The stress-strain diagrams for concrete generally comply with the schematic representation shown in Figure 5.1-3. The descending branch of the stress-strain relations should be considered as the envelope to all possible stress-strain relations of concrete which tends to soften as a consequence of concrete micro-cracking.

The stress-strain diagram may be best determined by corresponding tests.

The relation between σ_c and ε_c for short-term uniaxial compression shown in Fig. 5.1-3 is described by Eq. (5.1-26):

$$\frac{\sigma_c}{f_{cm}} = -\left(\frac{k \cdot \eta - \eta^2}{1+(k-2)\cdot\eta}\right) \quad for \ |\varepsilon_c| < |\varepsilon_{c,lim}| \tag{5.1-26}$$

where

$\eta = \varepsilon_c / \varepsilon_{c1}$

$k = E_{ci} / E_{c1}$

ε_{c1} is the strain at maximum compressive stress (Table 5.1-8)

E_{c1} is the secant modulus from the origin to the peak compressive stress (given in Table 5.1-8)

k is the plasticity number according to Table 5.1-8

Table 5.1-8: Modules E_{ci}, E_{c1}, strains ε_{c1}, $\varepsilon_{c,lim}$ and plasticity number k for normal weight concrete

Concrete grade	C12	C20	C30	C40	C50	C60	C70
E_{ci} [GPa]	27.1	30.3	33.6	36.3	38.6	40.7	42.6
E_{c1} [GPa]	11.1	13.3	16.5	20.0	23.2	26.2	28.9
ε_{c1} [‰]	-1.8	-2.1	-2.3	-2.4	-2.5	-2.6	-2.7
$\varepsilon_{c,lim}$ [‰]	-3.5	-3.5	-3.5	-3.5	-3.4	-3.3	-3.2
k	2.44	2.28	2.04	1.82	1.66	1.55	1.47

Concrete grade	C80	C90	C100	C110	C120
E_{ci} [GPa]	44.4	46.0	47.5	48.9	50.3
E_{c1} [GPa]	31.4	33.8	36	39.3	42.7
ε_{c1} [‰]	-2.8	-2.9	-3.0	-3.0	-3.0
$\varepsilon_{c,lim}$ [‰]	-3.1	-3.0	-3.0	-3.0	-3.0
k	1.41	1.36	1.32	1.24	1.18

For the calculation of ε_{lc1} for lightweight aggregate concrete a factor κ_{lc} is

If only the modulus of elasticity is available from experiments this value may be used for estimating the stress-strain diagram. However, an accurate stress-strain diagram can only be found if the plasticity number *k* was investigated.

Tensile failure of concrete is always a discrete phenomenon. Thus, to describe the tensile behaviour a stress-strain diagram should be used for the uncracked concrete, and a stress-crack opening diagram as shown in Figure 5.1-4 should be used for the cracked section.

introduced taking into account different types of sand:

$$\varepsilon_{lc1} = -\kappa_{lc} \cdot \frac{f_{lck} + 8}{E_{lc}} \tag{5.1-27}$$

where:

f_{lck} is the characteristic strength value for lightweight aggregate concrete in [MPa] according to Table 5.1-4

E_{lc} is the modulus of elasticity in [MPa] for lightweight aggregate concrete according to Eq. (5.1-25)

κ_{lc} 1.1 for lightweight aggregate concrete with light sand

1.3 for lightweight aggregate concrete with natural sand

The stress-strain relation for unloading of the uncracked concrete may described by Eq. (5.1-28)

$$\Delta\sigma_c = E_{ci} \cdot \Delta\varepsilon_c \tag{5.1-28}$$

where:

$\Delta\sigma_c$ is the stress reduction

$\Delta\varepsilon_c$ is the strain reduction

5.1.8.2 Tension

For uncracked normal weight concrete subjected to tension a bilinear stress-strain relation as given in eqs. 5.1-29 and 5.1-30 may be used (Figure 5.1-4):

$$\sigma_{ct} = E_{ci} \cdot \varepsilon_{ct} \quad \mathit{for} \quad \sigma_{ct} \le 0.9 \cdot f_{ctm} \tag{5.1-29}$$

$$\sigma_{ct} = f_{ctm} \cdot \left(1 - 0.1 \cdot \frac{0.00015 - \varepsilon_{ct}}{0.00015 - 0.9 \cdot f_{ctm}/E_{ci}}\right) \quad \mathit{for} \quad 0.9 \cdot f_{ctm} < \sigma_{ct} \le f_{ctm} \tag{5.1-30}$$

where:

E_{ci} is the tangent modulus of elasticity in [MPa] according to Eq. (5.1-20)

ε_{ct} is the tensile strain

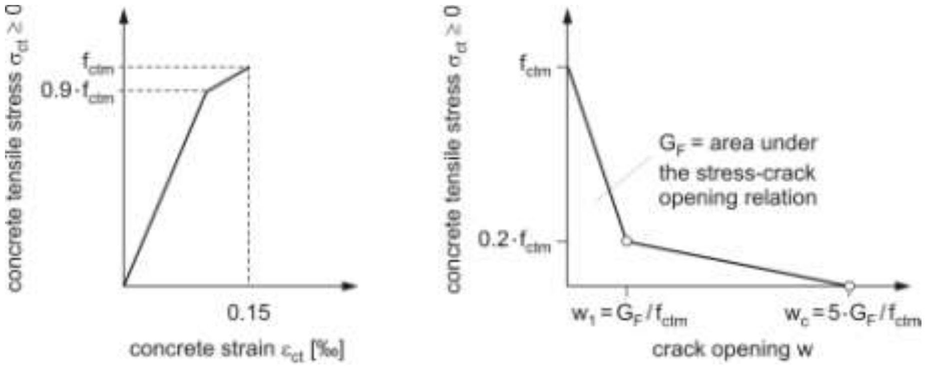

Figure 5.1-4: Schematic representation of the stress-strain and stress-crack opening relation for uniaxial tension (according to fib Bulletin 42)

At tensile stresses of about 90 % of the tensile strength f_{ct} micro-cracking starts to reduce the stiffness in a small failure zone (eqs. 5.1-29 and 5.1.-30). The micro-cracks grow and form a discrete crack at stresses close to the tensile strength. All stresses and deformations in the fracture process zone can be related to a fictitious crack opening *w* (according to *fib* Bulletin 42).

Neglecting the small energy consumed by a complete loading cycle in the stress-strain relation, the maximum strain $\varepsilon_{ct,max}$ can be estimated as $\varepsilon_{ct,max} \approx f_{ctm}/E_{ci}$. For the analysis of the fracture zone a strain $\varepsilon_{ct,max}$ = 0.15 ‰ can be estimated. Due to the localisation of micro-cracking in the fracture zone and the large uncracked areas outside the damage zone this strain is only valid inside the fracture zone.

Regarding the fracture energy in general reference is made to sub-clause 5.1.5.2. To describe to stress-strain relation for uniaxial tension for light-weight aggregate concrete reference is made to Faust, T.: Leichtbeton im konstruktiven Ingenieurbau, Verlag Ernst&Sohn, Berlin 2002, ISBN-10 3433016135

In the case of coinciding plastic potentials *g* and yield functions *f* the flow rule Eq. (5.1-34) is of the associated type, otherwise it is of the non-associated type. Non-associated flow rules should be used in concrete plastic-

σ_{ct} is the tensile stress in [MPa]

f_{ctm} is the tensile strength in [MPa] from Eq. (5.1-3)

For a cracked section a bilinear approach for the stress-crack opening relation according to Fig. 5.1-4 can be estimated by the following eqs. 5.1-31 and 5.1.-32:

$$\sigma_{ct} = f_{ctm} \cdot \left(1.0 - 0.8 \cdot \frac{w}{w_1}\right) \quad for \quad w \leq w_1 \tag{5.1-31}$$

$$\sigma_{ct} = f_{ctm} \cdot \left(0.25 - 0.05 \cdot \frac{w}{w_1}\right) \quad for \quad w_1 < w \leq w_c \tag{5.1-32}$$

where

w is the crack opening in [mm]

w_1 = G_F/f_{ctm} in [mm] when $\sigma_{ct} = 0.20 \cdot f_{ctm}$

w_c = $5 \cdot G_F/f_{ctm}$ in [mm] when $\sigma_{ct} = 0$

G_F is the fracture energy in [N/mm] from Eq. (5.1-9)

f_{ctm} is the tensile strength in [MPa] from Eq. (5.1-3)

5.1.8.3 Multiaxial states of stress

Constitutive relations of the elasto-plastic format, the damage format and combinations may be used to describe triaxial nonlinear concrete behaviour on the macroscopic level in the short time range. Concrete is assumed as isoity models to describe the inelastic volume change under compression, which is characteristic for frictional materials.

Basically, yield functions *f* and plastic potentials *g* can be chosen based on multi-axial failure criteria for concrete. These criteria should depend not only on shear stresses, but also on the first invariant I_1 of the stress tensor to consider the influence of the hydrostatic pressure on the ductility of the material. Thus, formulations as the

– Rankine criterion, where tensile failure occurs when the maximum principal stress reaches the uniaxial tensile strength f_{ct}; refer to Rankine, W.J.M.: 'A Manual of Applied Mechanics', London, 1868

– Drucker-Prager criterion, which is the modification of von Mises criterion including the influence of hydrostatic pressure on yielding; refer to Drucker, D.C.; Prager, W.: 'Soil mechanics and plastic analysis of limit design', Quarterly of Applied Mechanics, Vol. 10, 1952

– Mohr-Coulomb criterion, where the maximum shear stress is the decisive measure of yielding, and the critical shear stress value depends on hydrostatic pressure; refer to Mohr, O.: 'Abhandlungen aus dem Gebiete der technischen Mechanik', Ernst & Sohn, Berlin, 1906

and modifications or combinations of them can be used in concrete plasticity models. For further criteria and detailed information refer to:

– Chen, W.F.; Saleeb, A.F.: 'Constitutive Equations for Engineering Materials', John Wiley & Sons, 1994

– Jirásek, M.; Bažant, Z.P.: 'Inelastic Analysis of Structures', John Wiley & Sons, 2002.

Examples for elaborated plasticity models are given in

– Willam, K.; Warnke, E.P.: 'Constitutive model for the triaxial behaviour of concrete', IABSE Report Vol. 19, Seminar on Concrete Structures Subjected to Triaxial Stresses, Bergamo, 1974

tropic material in the initial unloaded state with an elasticity matrix $\mathbf{E}_0$, which is constant. Here the validity is restricted to small deformations.

The stress-strain behaviour of a general stress-based elasto-plastic format is given by Eq. (5.1-33):

$$\boldsymbol{\sigma} = \mathbf{E}_0 \cdot \left(\boldsymbol{\varepsilon} - \boldsymbol{\varepsilon}_p\right) \qquad (5.1\text{-}33)$$

with the triaxial stress $\boldsymbol{\sigma}$, strain $\boldsymbol{\varepsilon}$ and plastic strain $\boldsymbol{\varepsilon}_p$. Occurrence of permanent plastic strain increments is determined by the flow rule:

$$\dot{\boldsymbol{\varepsilon}}_p = \lambda \frac{\partial g}{\partial \boldsymbol{\sigma}} \qquad (5.1\text{-}34)$$

with the plastic potential *g* and the plastic multiplier λ. The plastic potential *g* is a function of stress $\boldsymbol{\sigma}$ and state variables $\boldsymbol{\alpha}$ representing the load history. The multiplier λ is determined by the Kuhn-Tucker conditions:

$$\lambda \geq 0, \qquad f\lambda = 0, \qquad f \leq 0 \qquad (5.1\text{-}35)$$

with a yield function *f*. The yield function *f* is also a function of stress $\boldsymbol{\sigma}$ and state variables $\boldsymbol{\alpha}$ and implies a limit condition for the material strength. The Kuhn-Tucker conditions distinguish unloading from loading and imply $\dot{\varepsilon}_p = 0$ associated with $f < 0$ or $\dot{\varepsilon}_p \neq 0$ in combination with Eq. (5.1-36):

$$\dot{f} = \frac{\partial f}{\partial \boldsymbol{\sigma}} \cdot \dot{\boldsymbol{\sigma}} + \frac{\partial f}{\partial \boldsymbol{\alpha}} \cdot \dot{\boldsymbol{\alpha}} = 0 \qquad (5.1\text{-}36)$$

This consistency condition and an evolution law for the internal state variables

$$\dot{\boldsymbol{\alpha}} = \lambda\, \mathbf{h}\left(\boldsymbol{\sigma}, \boldsymbol{\alpha}\right) \qquad (5.1\text{-}37)$$

result in an incremental constitutive law in case of loading:

$$\dot{\boldsymbol{\sigma}} = \left[\mathbf{E}_0 - \frac{\mathbf{E}_0 \cdot \dfrac{\partial g}{\partial \boldsymbol{\sigma}} \cdot \dfrac{\partial f}{\partial \boldsymbol{\sigma}}^T \cdot \mathbf{E}_0}{\dfrac{\partial f}{\partial \boldsymbol{\sigma}}^T \cdot \mathbf{E}_0 \cdot \dfrac{\partial g}{\partial \boldsymbol{\sigma}} - \dfrac{\partial f}{\partial \boldsymbol{\alpha}}^T \cdot \mathbf{h}} \right] \cdot \dot{\boldsymbol{\varepsilon}} \qquad (5.1\text{-}38)$$

– Oñate, E.; Oller, S.; Oliver, S.; Lubliner, J.: 'A constitutive model of concrete based on the incremental theory of plasticity', Engineering Computations, Vol. 5, 1988

– Etse, G.; Willam, K.: 'Fracture energy formulation for inelastic behaviour of plain concrete', Journal of Engineering Mechanics, Vol. 120, 1994

– Grassl, P.; Lundgren; K. Gylltoft, K.: 'Concrete in compression: a plasticity theory with a novel hardening law', International Journal of Solids and Structures, Vol. 39, 2002.

For a theoretical framework of damage models refer e.g. to:

– Carol, I.; Rizzi, E.; Willam, K.: 'A unified theory of elastic degradation and damage based on a loading surface', International Journal of Solids and Structures, Vol. 31, 1994.

The state variables $\boldsymbol{\beta}$ can be of scalar, vector and second or higher order tensor type. The use of scalar internal variables enables the description of isotropic damage, whereas tensor-valued state variables are needed for anisotropic damage formulations. Detailed information can be found e.g. in

– Lemaitre, J.: 'A Course on Damage Mechanics', Springer, 1992

– Krajcinovic, D.: 'Damage Mechanics', North-Holland, Elsevier, 1996

– Skrzypek, J.; Ganczarski, A.: 'Modelling of Material Damage and Failure of Structures', Springer, 1999.

The damage limit functions F can generally be chosen based on multiaxial limit criteria for concrete, which are defined in the stress space and can

The elastic law $\dot{\boldsymbol{\sigma}} = \mathbf{E}_0 \cdot \dot{\boldsymbol{\varepsilon}}$ applies in case of unloading. The functions g, f and $\mathbf{h}$ are material functions, which have to be determined on the basis of experimental data. The elasto-plastic format may be extended by multiple yield surfaces and plastic potentials.

The stress-strain behaviour of a general strain-based damage format is given by Eq. (5.1-39):

$$\boldsymbol{\sigma} = \mathbf{E} \cdot \boldsymbol{\varepsilon} \tag{5.1-39}$$

with the triaxial elasticity matrix $\mathbf{E}$, which is variable according to the damage format. Degradation of the elasticity or occurrence of damage is determined by:

$$\dot{\mathbf{E}} = -\lambda' \, \mathbf{G} \tag{5.1-40}$$

with a generalized damage direction $\mathbf{G}$ and a damage multiplier λ'. The generalized damage direction $\mathbf{G}$ depends on strain $\boldsymbol{\varepsilon}$ and state variables $\boldsymbol{\beta}$ representing the load history. The multiplier λ' is determined by the Kuhn-Tucker conditions:

$$\lambda' \geq 0, \qquad F\lambda' = 0, \qquad F \leq 0 \tag{5.1-41}$$

with a damage limit function F. The damage limit function F is also a function of strain $\boldsymbol{\varepsilon}$ and state variables $\boldsymbol{\beta}$ and again should imply a limit condition for the material strength. The Kuhn-Tucker conditions distinguish unloading from loading and imply $\dot{\mathbf{E}} = \mathbf{0}$ associated with $F < 0$ and $\dot{\mathbf{E}} \neq \mathbf{0}$ in combination with Eq. (5.1-42):

$$\dot{F} = \frac{\partial F}{\partial \boldsymbol{\varepsilon}} \cdot \dot{\boldsymbol{\varepsilon}} + \frac{\partial F}{\partial \boldsymbol{\beta}} \cdot \dot{\boldsymbol{\beta}} = 0 . \tag{5.1-42}$$

This consistency condition and an evolution law for the internal state variables

$$\dot{\boldsymbol{\beta}} = \lambda' \, \mathbf{h}'(\boldsymbol{\varepsilon}, \boldsymbol{\beta}) \tag{5.1-43}$$

be transferred into the strain space. Relevant examples are given in

– Ottosen, N.S.: 'A failure criterion for concrete', Journal of Engineering Mechanics, ASCE, Vol. 103, 1977
– Hsieh, S.S.; Ting, E.; Chen, W.F.: 'A plasticity fracture model for concrete', International Journal of Solids and Structures, Vol. 18, 1982
– Willam, K.; Warnke, E.P.: 'Constitutive model for the triaxial behaviour of concrete', IABSE Report Vol. 19, Seminar on Concrete Structures Subjected to Triaxial Stresses, Bergamo, 1974.

Fore more information refer to Chen, W.F.; Saleeb, A.F.: 'Constitutive Equations for Engineering Materials', John Wiley & Sons, 1994.

Strain-based isotropic damage formulations with scalar internal variables which consider tensile as well as compressive damage can be found e.g. in

– Mazars, J.: 'Application de la mécanique de l'endommangement au comportement non linéaire at à la rupture du béton de structure', Technical report, LMT, Université Paris, 1984
– Tao, X.; Phillips, D.V.: 'A simplified isotropic damage model for concrete under bi-axial stress states', Cement & Concrete Composites Vol. 27, 2005.

An orthotropic damage approach based on the second-order integrity tensor as internal variable is described e.g. in

– Carol, I.; Rizzi, E.; Willam, K.: 'On the formulation of anisotropic elastic degradation. I. Theory based on a pseudo-logarithmic damage tensor rate. II. Generalized pseudo-Rankine model for tensile damage', International Journal of Solids and Structures, Vol. 38, 2001.

result in an incremental constitutive law in case of loading:

$$\dot{\boldsymbol{\sigma}} = \left[\mathbf{E} + \frac{1}{\dfrac{\partial F^T}{\partial \boldsymbol{\beta}} \cdot \mathbf{h}'} \mathbf{G} \cdot \boldsymbol{\varepsilon} \cdot \frac{\partial F^T}{\partial \boldsymbol{\varepsilon}} \right] \cdot \dot{\boldsymbol{\varepsilon}} \qquad (5.1\text{-}44)$$

The linear elastic law $\dot{\boldsymbol{\sigma}} = \mathbf{E} \cdot \dot{\boldsymbol{\varepsilon}}$ with $\dot{\mathbf{E}} = \mathbf{0}$ applies in case of unloading. The functions $\mathbf{G}$, F and $\mathbf{h}'$ are material functions, which have to be determined on the basis of experimental data.

The scalar isotropic damage is given as a special case by means of Eq. (5.1-45):

$$\mathbf{E} = (1-D)\,\mathbf{E}_0 \,, \qquad \dot{\mathbf{E}} = -\dot{D}\,\mathbf{E}_0 \,, \qquad \mathbf{G} = \mathbf{E}_0 \qquad (5.1\text{-}45)$$

In Eq. (5.1-45) the restriction $0 \le D \le 1$ and the relation $\lambda' = \dot{D}$ holds. A scalar internal state variable is appropriate. The damage limit function F and the evolution function $\mathbf{h}'$ become scalar functions of strain invariants and of a scalar β. The value β as an equivalent strain measure with a restriction $\beta \ge 0$. Furthermore, in case of loading simple relations like

$$D(\beta) = \begin{cases} 0 & \beta \le \beta_0 \\ 1 - \mathrm{e}^{-\left(\frac{\beta - \beta_0}{\beta_d}\right)^g} & \beta > \beta_0 \end{cases} \qquad (5.1\text{-}46)$$

are appropriate where the material parameters β_0, β_d and g may be derived from uniaxial material behaviour.

For an anisotropic formulation with a higher order tensorial damage variable refer e.g. to

– Govindjee, S.; Kay, G.J.; Simo, J.C.: ‘Anisotropic modelling and numerical simulation of brittle damage in concrete’, International Journal for Numerical Methods in Engineering, Vol. 38, 1995.

Another approach for the material description of concrete is given with the microplane theory, see e.g.

– Bažant, Z.P.: ‘Microplane model for progressive fracture of concrete and rock’, Journal of Engineering Mechanics, Vol. 111, 1985.

– Ožbolt, J.; Li, Y.-J.; Kožar, I.: ’Microplane model for concrete with relaxed kinematic constraint’, International Journal of Solids and Structures, Vol. 38, 2001.

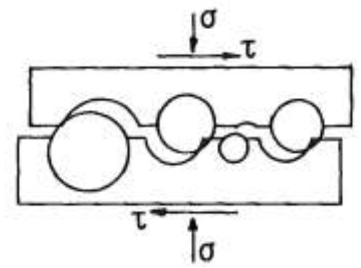

Figure 5.1-5: Principle of shear friction in concrete crack, with unbroken aggregates.

The description of anisotropic damage needs tensor notations, e.g. in case of orthotropic damage according Eq. (5.1-47):

$$G_{ijpq} = \Lambda_0 \left[\dot{d}_{ij}\, d_{pq} + d_{ij}\, \dot{d}_{pq} \right] + G_0 \left[\dot{d}_{ip}\, d_{jq} + d_{ip}\, \dot{d}_{jq} + \dot{d}_{iq}\, d_{jp} + d_{iq}\, \dot{d}_{jp} \right] \tag{5.1-47}$$

with the initial Lamé constant Λ_0, the initial shear modulus G_0 and a second-order symmetric damage tensor $\mathbf{d}$ with components d_{ij}, whose principal values and directions describe damage in three orthogonal directions.

5.1.8.4 Shear friction behaviour in cracks

If in an open crack the crack faces are subjected to shear displacements with opposite signs, resisting shear stresses and normal (wedging) stresses develop as a result of the roughness of the crack faces.

The shear stress τ [MPa] and the normal stress σ [MPa] may be calculated from the subsequent general (mean) relations:

$$\tau = C_f \{-0.04\, f_c + [1.8 w^{-0.8} + (0.292 w^{-0.7} - 0.25)\, f_c]\delta\} \tag{5.1-48}$$

$$\sigma = C_f \{-0.06\, f_c + [1.35 w^{-0.63} + (0.242 w^{-0.55} - 0.19)\, f_c]\delta\} \tag{5.1-49}$$

where

δ	is the shear displacement in [mm]
w	is the crack width in [mm]
f_c	is the concrete cylinder strength in [MPa]

C_f is an aggregate effectivity factor, which is 1.0 if the aggregate does not fracture upon cracking of the concrete. For concrete with weak aggregates, or high strength concrete (with strong cement paste), in which most of the particles are broken, for C_f a value of about 0.35 applies. More accurate values for C_f can be found by carrying out a push-off test.

The crack opening path (development of shear displacement in relation to

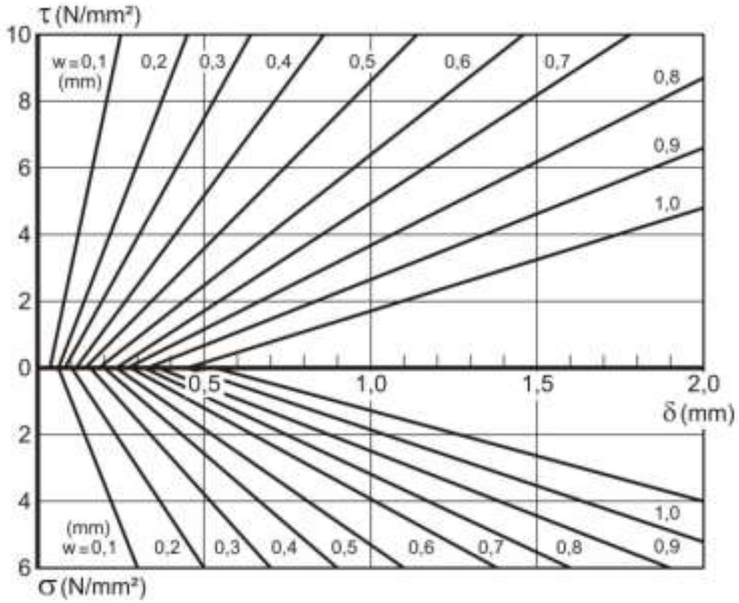

Figure 5.1-6: Relations (eqs. 5.1-48/49) for f_c = 30 MPa, according to fib Bulletin 42.

The compressive strength of concrete at an age *t* depends on the type and strength class of cement, type and amount of admixtures and additions, the water/cement ratio and environmental conditions, such as temperature and humidity conditions.

The tensile strength of concrete primarily depends on those parameters which also influence the compressive strength of concrete. However, tensile and compressive strength are not proportional to each other, and particularly for higher strength grades an increase of the compressive strength leads only to a small increase of the tensile strength.

The development of tensile strength with time is strongly influenced by curing and drying conditions (internal stresses, surface cracking) as well as by the dimensions of the structural members. As a first approximation it may

crack opening) can be constructed from diagrams like shown in Figure 5.1-6. If the relation between normal stress σ and crack opening *w* is given (analogy to spring stiffness), the corresponding values δ and τ can be read.

5.1.9 Time effects

5.1.9.1 Development of strength with time

For a mean temperature of 20 °C and curing in accordance with ISO 1920-3 the related compressive strength of concrete at various ages $f_{cm}(t)$ may be estimated from eqs. 5.1-50 and 5.1-51:

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm} \tag{5.1-50}$$

with

$$\beta_{cc}(t) = exp\left\{ s \cdot \left[1 - \left(\frac{28}{t} \right)^{0.5} \right] \right\} \tag{5.1-51}$$

where

be assumed that for a duration of moist curing $t_s \leq 7$ days and a concrete age $t > 28$ days the development of tensile strength is similar to that of compressive strength, i.e. Eq. (5.1-3) is independent of the concrete age for $t \geq 28$ days. For a concrete age $t < 28$ days residual stresses may cause a temporary decrease of the tensile strength.

For high strength concrete the decrease of the tensile strength due to shrinkage cracks seems to be more pronounced than for normal strength concrete.

In case where the development of tensile strength with time is important it is recommended to carry out experiments taking into account exposure conditions and dimensions of the structural member.

Concretes with a high content of fly ash, natural pozzolans or fine granulated blast furnace slag (e.g. green concrete) show a reduced compressive strength in the young concrete age and a considerable further strength gain at higher ages.

This effect may be more pronounced than considered in Eq. (5.1-51) for a low strength, normal hardening cement.

In the young concrete age the compressive strength of lightweight aggregate concrete mainly depends on the strength of the cement paste. With ongoing hydration the strength of the cement paste approaches the strength of the lightweight aggregates. Therefore hardly any strength gain may be observed after a certain concrete age. This concrete age depends on the strength of the lightweight aggregates. A range between one and four weeks of concrete age is realistic in most cases.

$f_{cm}(t)$ is the mean compressive strength in [MPa] at an age t in [days]

f_{cm} is the mean compressive strength in [MPa] at an age of 28 days

$\beta_{cc}(t)$ is a function to describe the development with time

t is the concrete age in [days] adjusted acc. to Eq. (5.1-85) (taking into account temperature during curing)

s coefficient which depends on the strength class of cement as given in Table 5.1-9

Table 5.1-9: Coefficient s to be used in Eq. (5.1-51) for different strength classes of cement and hardening characteristics (N = normal, R = rapid)

Strength class of cement	32.5 N	32.5 R 42.5 N	42.5 R 52.5 N 52.5 R
s	0.38	0.25	0.20

For lightweight aggregate concrete the compressive strength in [MPa] at various ages may be estimated from:

$$f_{lcm}(t) = \beta_{lcc}(t) \cdot f_{lcm} \tag{5.1-52}$$

where

$\beta_{lcc}(t)$ is the function to describe development with time; $\beta_{lcc}(t) = \beta_{cc}(t)$ where s has to be replaced by s_{lc}

s_{lc} 0.05 for lightweight aggregates of high strength

0.25 for lightweight aggregates of low strength

f_{lcm} is the mean compressive strength in [MPa] at an age of 28 days

5.1.9.2 Strength under sustained loads

5.1.9.2.1 Sustained compressive strength

Due to the counteracting effects of the parameters influencing the strength under sustained loads, $f_{cm,sus}(t,t_0)$ passes through a minimum. The duration of loading for which this minimum occurs depends on the age at loading and is

When subjected to sustained high compressive stresses the compressive strength of concrete decreases with time under load due to the formation of micro-cracks. This strength reduction is counteracted by a strength increase referred to as the critical period $(t\text{-}t_0)_{crit}$. For an age at loading of 28 days, a concrete made of normal cement, type N, $(t\text{-}t_0)_{crit}$ = 2.8 days, $f_{c,sus,min} = 0.78 f_{cm}$. It is generally referred to as sustained load strength of concrete.

Research has shown a slight increase of the sustained load strength with increasing compressive strength of the concrete. However, due to the limited number of tests on high strength concrete the sustained load strength of normal strength concrete should be applied also for high strength concrete.

Resulting from the reduced strength gain of the lightweight aggregate concrete as soon as the strength of the cement paste approaches the strength of the aggregates the critical period is extended and the strength under sustained loads equals 70 to 75 % of the short time strength at the age of 28 days.

due to continued hydration. The combined effect of sustained stresses and of continued hydration is given by eqs. 5.1-53 and 5.1-54:

$$f_{cm,sus}\left(t,t_0\right) = f_{cm} \cdot \beta_{cc}\left(t\right) \cdot \beta_{c,sus}\left(t,t_0\right) \qquad (5.1\text{-}53)$$

with

$$\beta_{c,sus}\left(t,t_0\right) = 0.96 - 0.12\left\{ln\left[72\left(t-t_0\right)\right]\right\}^{1/4} \qquad (5.1\text{-}54)$$

where

$f_{cm,sus}(t,t_0)$ is the mean compressive strength of concrete in [MPa] at time t when subjected to a high sustained compressive stress at an age at loading $t_0 < t$

$\beta_{cc}(t)$ is the time development according to Eq. (5.1-51)

$\beta_{c,sus}(t,t_0)$ is a coefficient which depends on the time under high sustained loads $t\text{-}t_0$ in [days]. The coefficient describes the decrease of strength with time under load and is defined for $(t\text{-}t_0) > 0.015$ days (= 20 min)

t_0 is the age of the concrete at loading in [days]

$t\text{-}t_0$ is the time under high sustained loads in [days]

5.1.9.2.2 Sustained tensile strength

Eq. (5.1-55) has been taken from *fib* Bulletin 42, 2008

Tensile strength under sustained loading $f_{ctk,sus}$ in [MPa] can be estimated from:

$$f_{ctk,sus} = \alpha \cdot f_{ctk} \qquad (5.1\text{-}55)$$

where

f_{ctk} is the short term strength in [MPa]

α = 0.60 for normal strength concrete and 0.75 for high strength concrete

5.1.9.3 Development of modulus of elasticity with time

For lightweight aggregate concrete Eq. (5.1-57) has to be applied with caution. For structures being sensible to deformations, related tests have to be carried out.

The modulus of elasticity of concrete at an age t ≠ 28 days may be estimated from Eq. (5.1-56):

The hydration of the cement in self-compacting concrete is basically controlled by the same mechanisms as that of vibrated concrete. No particular difference in the time-development of properties is thus expected.

Concretes with a high content of fly ash, natural pozzolans or fine granulated blast furnace slag (e.g. green concrete) show a reduced modulus of elasticity in the young concrete age and a further gain of stiffness at higher ages. This effect may be more pronounced than considered in Eq. (5.1-51) for low strength, normal hardening cement.

$$E_{ci}(t) = \beta_E(t) E_{ci} \tag{5.1-56}$$

with

$$\beta_E(t) = \left[\beta_{cc}(t)\right]^{0.5} \tag{5.1-57}$$

where

$E_{ci}(t)$ modulus of elasticity in [MPa] at an age t in [days]

E_{ci} modulus of elasticity in [MPa] at an age of 28 days from Eq. (5.1-20)

$\beta_E(t)$ coefficient which depends on the age of concrete, t in [days]

$\beta_{cc}(t)$ coefficient according to Eq. (5.1-51)

The coefficient s, to be inserted in $\beta_{cc}(t)$ depends on the type of cement (strength class) and the compressive strength of the concrete and may be taken from the following Table 5.1-10.

Table 5.1-10: Coefficient s to be used in Eq. (5.1-49) for different types of cement

f_{cm} [MPa]	strength class of cement	s
≤ 60	32.5 N	0.38
	32.5 R, 42.5 N	0.25
	42.5 R, 52.5 N, 52.5 R	0.20
> 60	all classes	0.20

5.1.9.4 Creep and shrinkage

5.1.9.4.1 Definitions

The distinction between creep and shrinkage is conventional. Normally the delayed strains of loaded or unloaded concrete should be considered as

The total strain at time t, $\varepsilon_c(t)$, of a concrete member uniaxially loaded at time t_0 with a constant stress $\sigma_c(t_0)$ may be expressed as follows:

$$\varepsilon_c(t) = \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t) + \varepsilon_{cs}(t) + \varepsilon_{cT}(t) \tag{5.1-58}$$

$$\varepsilon_c(t) = \varepsilon_{c\sigma}(t) + \varepsilon_{cn}(t) \tag{5.1-59}$$

where:

$\varepsilon_{ci}(t_0)$ is the initial strain at loading two aspects of the same physical phenomena.

Also, separation of initial strain and creep strain is a matter of convention. In structural analysis, the total load dependent strain as given by the creep function (refer to sub-clause 5.1.9.4.3) is of importance. The initial and creep strain components are defined consistently, so that their sum results in the correct load dependent strain.

For the prediction of the creep function the initial strain $\varepsilon_{ci}(t_0)$ is based on the tangent modulus of elasticity as defined in eqs. 5.1-20 and 5.1-56, i.e.

$$\varepsilon_{ci}(t_0) = \sigma_c(t_0)/E_{ci}(t_0)$$

The initial plastic strain occurring at first loading (see Figure 5.1-2) is considered to be part of the creep strain.

The model does not predict local rheological properties within the cross-section of a concrete member such as variations due to internal stresses, moisture states or the effects of local cracking.

The prediction model is not applicable to

- concrete subjected to extreme temperatures, high (e.g. nuclear reactors) or low (e.g. LNG-tanks)
- very dry climatic conditions (average relative humidity RH < 40 %)

The effect of temperature variations during hardening can be taken into account in accordance with Eq. (5.1-85). The effect of 0 °C < T < 80 °C is dealt with in sub-clause 5.1.10.

Here, concrete is considered as an aging linear visco-elastic material. In reality, creep is a non-linear phenomenon. The non-linearity with respect to creep inducing stress may be observed in creep experiments at a constant stress, particularly if the stress exceeds $0.4 \cdot f_{cm}(t_0)$, as well as in experiments with a variable stress history even below stresses of $0.4 \cdot f_{cm}(t_0)$.

$\varepsilon_{cc}(t)$ is the creep strain at time $t > t_0$

$\varepsilon_{cs}(t)$ is the shrinkage strain

$\varepsilon_{cT}(t)$ is the thermal strain

$\varepsilon_{c\sigma}(t)$ stress dependent strain: $\varepsilon_{c\sigma}(t) = \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t)$

$\varepsilon_{cn}(t)$ stress independent strain: $\varepsilon_{cn}(t) = \varepsilon_{cs}(t) + \varepsilon_{cT}(t)$

5.1.9.4.2 Range of applicability

The model for creep and shrinkage given below predicts the time-dependent mean cross-section behaviour of a concrete member moist cured at normal temperatures not longer than 14 days.

Unless special provisions are given the model is valid for ordinary structural concrete (15 MPa ≤ f_{cm} ≤ 130 MPa) subjected to a compressive stress $|\sigma_c| \le 0.4 f_{cm}(t_0)$ at an age at loading t_0 and exposed to mean relative humidity in the range of 40 to 100 % at mean temperatures from 5 °C to 30 °C. The age at loading should be at least 1 day.

It is accepted that the scope of the model also extends to concrete in tension, though the relations given in the following are directed towards the prediction of creep of concrete subjected to compressive stresses.

5.1.9.4.3 Creep

(a) Assumptions and related basic equations

Within the range of service stresses $|\sigma_c| \le 0.4 \cdot f_{cm}(t_0)$, creep is assumed to be linearly related to stress.

In this clause a so-called product formulation for the prediction of creep has been used, i.e. creep after a given duration of loading can be predicted from the product of a notional creep coefficient which depends on the age of concrete at loading and a function describing the development of creep with time. As an alternative, creep may also be described by a summation formulation as the sum of delayed elastic and of viscous strains. Advantages and disadvantages of both approaches are given in: ‘Evaluation of the time dependent behaviour of concrete’, CEB Bulletin 199, Lausanne, 1990.

For a constant stress applied at time t_0 this leads to:

$$\varepsilon_{cc}\left(t,t_0\right)=\frac{\sigma_c\left(t_0\right)}{E_{ci}}\varphi\left(t,t_0\right) \tag{5.1-60}$$

where

$\varphi(t,t_0)$ is the creep coefficient

E_{ci} is the modulus of elasticity at the age of 28 days according to eqs. 5.1-20 or 5.1-21 in [MPa]

The stress dependent strain $\varepsilon_{c\sigma}(t,t_0)$ at time t may be expressed as:

$$\varepsilon_{c\sigma}\left(t,t_0\right)=\sigma_c\left(t_0\right)\left[\frac{1}{E_{ci}\left(t_0\right)}+\frac{\varphi\left(t,t_0\right)}{E_{ci}}\right]=\sigma_c\left(t_0\right)J\left(t,t_0\right) \tag{5.1-61}$$

where

$J(t,t_0)$ is the creep function or creep compliance, representing the total stress dependent strain per unit stress

$E_{ci}(t_0)$ is the modulus of elasticity at the time of loading t_0 according to Eq. (5.1-56); hence $1/E_{ci}(t_0)$ represents the initial strain per unit stress at loading

The application of the principle of superposition is consistent with respect to the assumption of linearity. However, due to the actual non-linear behaviour of concrete some prediction errors are inevitable when linear superposition is applied to creep of concrete und variable stress, particularly for unloading or decreasing strains, respectively. For linear creep prediction models, the error depends on the type of model which is underlying the creep prediction (refer to CEB Bulletin 177).

The structural effects of time-dependent behaviour of concrete are dealt with in detail in CEB Bulletin 215.

For practical applications concrete may be considered as an aging linear viscoelastic material, and for variable stresses and strains, the principle of superposition is assumed to be valid. On the basis of these assumptions and definitions given above, the constitutive equation for concrete may be written as:

$$\varepsilon_c\left(t\right)=\sigma_c\left(t_0\right)J\left(t,t_0\right)+\int_{t_0}^{t}J\left(t,\tau\right)\frac{\partial\sigma_c\left(\tau\right)}{\partial\tau}d\tau+\varepsilon_{cn}\left(t\right) \tag{5.1-62}$$

The relations to calculate the creep coefficient are empirical. They were calibrated on the basis of laboratory tests (creep in compression) on structural concretes.

In this prediction model only those parameters are taken into account

(b) Creep coefficient

The creep coefficient may be calculated from:

$$\varphi\left(t,t_0\right)=\varphi_0\beta_c\left(t,t_0\right) \tag{5.1-63}$$

where:

which are normally known to the designer, i.e. characteristic compressive strength, dimensions of the member, mean relative humidity to which the member is exposed, age at loading, duration of loading and type of cement. It should be pointed out, however, that creep of concrete does not depend on its compressive strength or age at loading per se, but rather on its composition and degree of hydration; creep of concrete decreases with decreasing water/cement ratio, decreasing cement paste content, increasing stiffness of the aggregates and increasing degree of hydration.

For structures being sensible to creep deformations tests are recommended according to:

– RILEM TC 107-CSP: Creep and shrinkage prediction models: Principles of their formation. Recommendation for "Measurement of time-dependent strains of concrete". In: Materials and Structures, Vol. 31, October 1998, pp 507-512
– ISO 1920-9: Testing of Concrete – Part 9: Determination of creep of concrete cylinders in compression.

Due to the inherent scatter of creep and shrinkage deformations, the errors of the model and the general uncertainty caused by randomness of material properties and environment, a deformation prediction may result in a considerable prediction error. After short durations of loading or drying the prediction error is higher than after long durations of loading and drying.

Based on a computerized data bank of laboratory test results a mean coefficient of variation for the predicted creep function V_c = 20 % has been estimated. Assuming a normal distribution this corresponds to a 10 and 5 percent cut-off, respectively, on the lower an the upper side of the mean value of

$$\varphi_{0.10} = 0.74\varphi; \quad \varphi_{0.05} = 0.66\varphi$$

$$\varphi_{0.90} = 1.26\varphi; \quad \varphi_{0.95} = 1.34\varphi$$

The prediction error should be taken into account in a probabilistic approach where appropriate.

φ_0 is the notional creep coefficient, Eq. (5.1-64)

$\beta_c(t,t_0)$ is the coefficient to describe the development of creep with time after loading, Eq. (5.1-69)

t is the age of concrete in [days] at the moment considered

t_0 is the age of concrete at loading in [days], adjusted according to Eq. (5.1-73) and (5.1-85)

The notional creep coefficient may be estimated from:

$$\varphi_0 = \varphi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) \tag{5.1-64}$$

with

$$\varphi_{RH} = \left[1 + \frac{1 - RH/100}{0.1 \cdot \sqrt[3]{h}} \cdot \alpha_1\right] \cdot \alpha_2 \tag{5.1-65}$$

$$\beta(f_{cm}) = \frac{16.8}{\sqrt{f_{cm}}} \tag{5.1-66}$$

$$\beta(t_0) = \frac{1}{0.1 + (t_0)^{0.2}} \tag{5.1-67}$$

where:

f_{cm} is the mean compressive strength at the age of 28 days in [MPa] according to Eq. (5.1-1)

H is the relative humidity of the ambient environment in [%]

h = $2A_c/u$ = notional size of member in [mm], where A_c is the cross-section in [mm²] and u is the perimeter of the member in contact with the atmosphere in [mm]

$$\alpha_1 = \left[\frac{35}{f_{cm}}\right]^{0.7}, \quad \alpha_2 = \left[\frac{35}{f_{cm}}\right]^{0.2} \tag{5.1-68}$$

It is not known whether creep approaches a finite value. Nevertheless, the hyperbolic time function given in Eq. (5.1-69) approaches an asymptotic value for $t \to \infty$. Evaluations on the basis of test results indicate that this equation is a reasonably good approximation for the time development of creep up to 70 years of loading under the conditions indicated in Tables 5.1-11 and 5.1-12. From experimental observations of creep up to 30 years one may conclude that the increase of creep from 70 years up to 150 years of duration of loading will not exceed 5 % of the creep after 70 years.

In cases where a lower level of accuracy is sufficient, the values given in Table 5.1-11 can be accepted as representative values for the creep coefficient after 70 years of loading of a normal weight ordinary structural concrete with a characteristic compressive strength between 20 and 50 MPa. These 70 year values may be taken as final creep coefficients.

Table 5.1-11: Creep coefficient $\varphi(70y,t_0)$ of an ordinary structural concrete after 70 years of loading

Age at loading t_0 [days]	Dry atmospheric conditions (RH = 50 %, indoors)			Humid atmospheric conditions (RH = 80 %, outdoors)		
	Notional size $2A_c/u$ [mm]					
	50	150	600	50	150	600
1	5.8	4.8	3.9	3.8	3.4	3.0
7	4.1	3.3	2.7	2.7	2.4	2.1
28	3.1	2.6	2.1	2.0	1.8	1.6
90	2.5	2.1	1.7	1.6	1.5	1.3
365	1.9	1.6	1.3	1.2	1.1	1.0

For normal weight high strength concrete ($60 \le f_{cm} \le 130$ MPa) the creep coefficient after 70 years of loading may be calculated by multiplying the corresponding values in Table 5.1-12 with $(63/f_{cm})^{0.9}$.

The development of creep with time is described by:

$$\beta_c(t,t_0) = \left[\frac{(t-t_0)}{\beta_H + (t-t_0)}\right]^{0.3} \tag{5.1-69}$$

with

$$\beta_H = 1.5 \cdot h \cdot \left[1 + (1.2 \cdot RH/100)^{18}\right] + 250\,\alpha_3 \le 1500\,\alpha_3 \tag{5.1-70}$$

where:

RH is the relative humidity of the ambient environment in [%]

h = $2A_c/u$ = notional size of member in [mm], where A_c is the cross-section in [mm²] and u is the perimeter of the member in contact with the atmosphere in [mm]

$$\alpha_3 = \left[\frac{35}{f_{cm}}\right]^{0.5} \tag{5.1-71}$$

f_{cm} is the mean compressive strength at the age of 28 days in [MPa] according to Eq. (5.1-1)

Table 5.1-12: Creep coefficient $\varphi(70y,t_0)$ of an normal weight high strength concrete after 70 years of loading

Age at loading t_0 [days]	Dry atmospheric conditions (RH = 50 %, indoors)			Humid atmospheric conditions (RH = 80 %, outdoors)		
	Notional size $2A_c/u$ [mm]					
	50	150	600	50	150	600
1	2.5	2.1	1.8	1.8	1.6	1.5
7	2.0	1.7	1.5	1.5	1.3	1.2
28	1.7	1.4	1.2	1.2	1.1	1.0
90	1.4	1.2	1.0	1.0	0.9	0.8
365	1.1	0.9	0.8	0.8	0.7	0.6

The values given in Table 5.1-12 are calculated for the concrete grade C55 and may only be used in combination with the factor $(63/f_{cm})^{0.9}$ for concrete produced with rapid hardening high strength cement (strength classes 42.5 R, 52.5 N, 52.5 R).

The data given in Tables 5.1-11 and 5.1-12 apply for a mean temperature of the concrete between 10 °C and 20 °C. Seasonal variations of temperature between -20 °C and +40 °C can be accepted. The same holds true for variations in relative humidity around the mean values given in the same table.

Creep of powder type SCC is affected by its high paste content. In general the creep deformation is approximately 10-20 % higher than that of conventional concrete of equal strength. However, the deformations are within the scatter band for ordinary structural concrete, which is defined to be ±30 %. If the structure reacts sensitive to variations in the creep behaviour tests are highly recommended.

The higher creep tendency of lightweight aggregate concrete due to the reduced stiffness of the aggregates is compensated by the lower creep capability of the stiffer cement paste matrix. The creep coefficient, i.e. the ratio of creep and elastic strains, may be estimated by applying a reduction factor η_E.

For lightweight aggregate concrete the related creep coefficient φ_l may be calculated according to Eq. (5.1-72):

$$\varphi_l = \eta_E \cdot \varphi_0 \cdot \beta_c(t,t_0) \qquad (5.1\text{-}72)$$

where

η_E $= (\rho/2200)^2$, with oven-dry density ρ in [kg/m³]

φ_0 is the notional creep coefficient according to Eq. (5.1-64)

$\beta_c(t,t_0)$ describes the development with time according to Eq. (5.1-69)

For concrete grades LC12/13 and LC16/18 the creep coefficient φ_l has to be additionally multiplied with the factor 1.3.

(c) Effect of type of cement and curing temperature

Different types of cement result in different degrees of hydration. Creep of concrete depends on the degree of hydration reached at a given age rather than on the age of concrete. Therefore, the effect of type of cement is taken into account by modifying the age at loading such that for a given modified age the degree of hydration is approximately independent of the type of cement. The value for t_0 according to Eq.(5.1-73) has to be used in Eq.(5.1-67). The duration of loading used in Eq.(5.1-69) is the actual time under load.

Green concretes may for example be produced by replacing a large amount of the cement by the residual product fly ash. Mainly resulting from the reduced cement content a lower creep capability could be observed in corresponding creep experiments.

However, when considering slowly hardening cement in Eq. (5.1-73) describing the delayed hydration of fly ash concretes the creep coefficient is increased due to the lower modified age at loading. The model may therefore overestimate the actual creep deformations of green concretes including fly ash.

The effect of type of cement on the creep coefficient of concrete may be taken into account by modifying the age at loading t_0 according to Eq.(5.1-73)

$$t_0 = t_{0,T} \cdot \left[\frac{9}{2 + t_{0,T}^{1.2}} + 1 \right]^{\alpha} \geq 0.5\, days \qquad (5.1\text{-}73)$$

where

$t_{0,T}$ is the age of concrete at loading in [days] adjusted according to Eq. (5.1-85)

α is a coefficient which depends on the type of cement:

α = -1 for strength class 32.5 N;

α = 0 for strength classes 32.5 R, 42.5 N;

α = 1 for strength classes 42.5 R, 52.5 N, 52.5 R

(d) Effect of high stresses

The nonlinear behaviour of concrete under high stresses mainly results from micro-cracking. Eq. (5.1-74) represents a simplification in so far as it does not take into account the observation that non-linearity decreases with increasing duration of loading.

It should be noted that delayed elastic strains upon total unloading may be assumed as linear functions of stress up to stress levels of $|\sigma_c| = 0.6 f_{cm}(t_0)$ though some experiments indicate some over-proportionality.

For stress levels in the range of $0.4 f_{cm}(t_0) < |\sigma_c| \leq 0.6 f_{cm}(t_0)$ the nonlinearity of creep may be taken into account using Eqs. 5.1-74a/b:

$$\varphi_{0,k} = \varphi_0\, exp\left[1.5\left(k_\sigma - 0.4\right)\right] \qquad \text{for } 0.4 < k_\sigma \leq 0.6 \qquad (5.1\text{-}74a)$$

$$\varphi_{0,k} = \varphi_0 \qquad \text{for } k_\sigma \leq 0.4 \qquad (5.1\text{-}74b)$$

where

$\varphi_{0,k}$ is the nonlinear notional creep coefficient, which replaces φ_0 in Eq.

Due to microstructural mechanisms, becoming dominant for high strength concrete, the total shrinkage has to be separated into autogenous shrinkage and drying shrinkage.

For curing periods of concrete members t_s < 14 days at normal ambient temperatures, the duration of moist curing does not significantly affect the total shrinkage. Hence, this parameter as well as the effect of curing temperature is not taken into account.

In Eqs. (5.1-77) and (5.1-82) the actual duration of drying (t-t_s) has to be used. It is not affected by possible adjustments of t_0 or t_s according to Eqs. 5.1-73 and 5.1-85.

Similar to creep, total shrinkage does not depend on concrete compressive strength per se. Drying shrinkage decreases with decreasing water/cement ratio and decreasing cement content whereas autogenous shrinkage increases with decreasing water/cement ratio and decreases with decreasing cement content.

If the composition of concrete deviates considerably from ordinary structural concrete (e.g. green concrete) it is recommended to run tests. This holds also true for ordinary concrete in case the concrete structures react sensible to shrinkage deformations. Tests should be performed according to:

– RILEM TC 107-CSP: Creep and shrinkage prediction models: Principles of their formation. Recommendation for “Measurement of time-dependent strains of concrete”. In: Materials and Structures, Vol. 31, October 1998, pp. 507-512

– ISO 1920-8: Testing of Concrete – Part 9: Determination of drying shrinkage for samples prepared in the field or in the laboratory.

A mean coefficient of variation of predicted shrinkage has been estimated on the basis of a computerized data bank, resulting in V_s = 35 %. The corresponding 10 and 5 percent cut-off values are

(5.1-63)

$k_\sigma \quad = |\sigma_c| / f_{cm}(t_0)$ which is the stress-strength ratio

5.1.9.4.4 Shrinkage

The total shrinkage or swelling strains $\varepsilon_{cs}(t,t_s)$ may be calculated from Eq. (5.1-75):

$$\varepsilon_{cs}(t,t_s) = \varepsilon_{cas}(t) + \varepsilon_{cds}(t,t_s) \qquad (5.1\text{-}75)$$

where shrinkage is subdivided into the autogenous shrinkage $\varepsilon_{cas}(t)$:

$$\varepsilon_{cas}(t) = \varepsilon_{cas0}(f_{cm}) \cdot \beta_{as}(t) \qquad (5.1\text{-}76)$$

and the drying shrinkage $\varepsilon_{cds}(t,t_s)$:

$$\varepsilon_{cds}(t,t_s) = \varepsilon_{cds0}(f_{cm}) \cdot \beta_{RH}(RH) \cdot \beta_{ds}(t-t_s) \qquad (5.1\text{-}77)$$

where

t is the concrete age in [days]

t_s is the concrete age at the beginning of drying in [days]

$(t$-$t_s)$ is the duration of drying in [days]

The autogenous shrinkage component $\varepsilon_{cas}(t)$ may be estimated by means of the notional autogenous shrinkage coefficient $\varepsilon_{cas0}(f_{cm})$ and the time function $\beta_{as}(t)$:

$$\varepsilon_{cas0}(f_{cm}) = -\alpha_{as}\left(\frac{f_{cm}/10}{6+f_{cm}/10}\right)^{2.5} \cdot 10^{-6} \qquad (5.1\text{-}78)$$

$$\beta_{as}(t) = 1 - exp\left(-0.2 \cdot \sqrt{t}\right) \qquad (5.1\text{-}79)$$

where

f_{cm} is the mean compressive strength at the age of 28 days in [MPa] according to Eq. (5.1-1)

α_{as} is a coefficient, dependent on the type of cement (see Table 5.1-13)

$$\varepsilon_{cs0.10} = 0.55\varepsilon_{cs}\,; \quad \varepsilon_{cs0.05} = 0.42\varepsilon_{cs}$$

$$\varepsilon_{cs0.90} = 1.45\varepsilon_{cs}\,; \quad \varepsilon_{cs0.95} = 1.58\varepsilon_{cs}$$

In cases where a lower level of accuracy is sufficient, the values given in Table 5.1-14 and 5.1-15 can be accepted as representative values for total shrinkage after 70 years of drying of a normal strength normal weight ordinary structural concrete with a characteristic strength between 20 and 50 MPa produced with a cement of types 32.5 R or 42.5 N. Usually these values may be taken as final shrinkage values.

Though shrinkage reaches a final value, little information exists on the shrinkage strains of large members after long durations of drying. Therefore, the values calculated using Eq. (5.1-82) for $2A_c/u$ = 600 mm, and the values given in Table 5.1-14 for shrinkage of members with a notional size of $2A_c/u$ = 600 mm, respectively, are uncertain and may overestimate the actual shrinkage strains after 70 years of drying.

Table 5.1-14: Total shrinkage values $\varepsilon_{cs,70y}\cdot 10^3$ for structural concrete after a duration of drying of 70 years

Dry atmospheric conditions (RH = 50 %, indoors)			Humid atmospheric conditions (RH = 80 %, outdoors)		
Notional size $2A_c/u$ [mm]					
50	150	600	50	150	600
-0,57	-0,56	-0,47	-0,32	-0,31	-0,26

Table 5.1-13: Coefficients α_i used in eqs. 5.1-78 and 5.1-80

strength class of cement	α_{as}	α_{ds1}	α_{ds2}
32.5 N	800	3	0.013
32.5 R, 42.5 N	700	4	0.012
42.5 R, 52.5 N, 52.5 R	600	6	0.012

The drying shrinkage $\varepsilon_{cds}(t,t_s)$ is calculated by means of the notional drying shrinkage coefficient $\varepsilon_{cds0}(f_{cm})$, the coefficient $\beta_{RH}(RH)$, taking into account the effect of the ambient relative humidity and the function $\beta_{ds}(t\text{-}t_s)$ describing the time-development:

$$\varepsilon_{cds0}\left(f_{cm}\right) = \left[\left(220 + 110\cdot\alpha_{ds1}\right)\cdot exp\left(-\alpha_{ds2}\cdot f_{cm}\right)\right]\cdot 10^{-6} \tag{5.1-80}$$

$$\beta_{RH} = \begin{cases} -1.55\cdot\left[1-\left(\dfrac{RH}{100}\right)^3\right] & for\ 40 \le RH < 99\,\%\cdot\beta_{s1} \\ 0.25 & for\ RH \ge 99\,\%\cdot\beta_{s1} \end{cases} \tag{5.1-81}$$

$$\beta_{ds}\left(t-t_s\right) = \left(\frac{\left(t-t_s\right)}{0.035\cdot h^2 + \left(t-t_s\right)}\right)^{0.5} \tag{5.1-82}$$

$$\beta_{s1} = \left(\frac{35}{f_{cm}}\right)^{0.1} \le 1.0 \tag{5.1-83}$$

where

α_{ds1}, α_{ds2}	are coefficients, dependent on the type of cement (see Table 5.1-13)
f_{cm}	is the mean compressive strength at the age of 28 days in [MPa] according to Eq. (5.1-1)
RH	is the relative humidity of the ambient atmosphere in [%]
h	$= 2A_c/u$ is the notional size of member in [mm], with A_c as the cross-section in [mm²] and u as the perimeter of the member

For normal weight high strength concrete ($60 \leq f_{cm} \leq 130$ MPa) the shrinkage after 70 years may be calculated by multiplying the corresponding values in Table 5.1-15 with $(63/f_{cm})^{0.2}$.

Table 5.1-15: Total shrinkage values $\varepsilon_{cs,70y} \cdot 10^3$ for normal weight high strength concrete after a duration of drying of 70 years

Dry atmospheric conditions (RH = 50 %, indoors)			Humid atmospheric conditions (RH = 80 %, outdoors)		
Notional size $2A_c/u$ [mm]					
50	150	600	50	150	600
-0.67	-0.67	-0.59	-0.43	-0.42	-0.38

The values in Table 5.1-15 are calculated for the concrete grade C55 and may only be used in combination with the factor $(63/f_{cm})^{0.2}$ for concrete produced with rapid hardening high strength cement (strength classes 42.5 R, 52.5 N, 52.5 R).

The shrinkage of powder type SCC is affected by its high paste content. The ultimate shrinkage deformation is approximately 20 % higher than that of conventional concrete of equal strength. The deformations are within the scatter band, which is defined to be ±30 %. If the structure reacts sensitive to variations in the shrinkage behaviour tests are highly recommended.

Higher fly ash contents in concrete (e.g. green concrete) tend to decrease the total shrinkage deformations which may result from the reduced cement content. As the given model considers slowly hardening cements which would correctly describe the delayed hydration of fly ash concretes but not the reduced cement content, shrinkage experiments are recommended when shrinkage deformations are decisive in the design of green concrete struc-

in contact with the atmosphere in [mm]

t is the concrete age in [days]

t_s is the concrete age at the beginning of drying in [days]

$(t-t_s)$ is the duration of drying in [days] tures.

In contrast to normal weight concrete the shrinkage behaviour of lightweight aggregate concrete is characterised by swelling deformations in the young concrete age. This results from water stored in the porous aggregates which is only slowly released into the cement paste matrix. This shrinkage characteristic of LWAC is not taken into account in Eq. (5.1-84). The observed swelling deformations are turning into shrinkage deformations only after a longer duration of drying. The final value of drying shrinkage is depending on the moisture content of the aggregates.

For structures being sensible to shrinkage deformations tests are recommended.

The shrinkage of lightweight aggregate concrete $\varepsilon_{lcs}(t,t_s)$ may be roughly estimated by Eq. (5.1-84)

$$\varepsilon_{lcs}(t,t_s) = \eta \cdot \varepsilon_{cs}(t,t_s) \tag{5.1-84}$$

where

$\varepsilon_{cs}(t,t_s)$ is calculated according to Eq. (5.1-75)

η = 1.5 for LC8, LC12, LC16

= 1.2 for LC20 and higher

5.1.10 Temperature effects

5.1.10.1 Range of application

The information given in the preceding clauses is valid for a mean temperature taking into account seasonal variations, between approx. -20 °C and +40 °C. In the following clause the effect of substantial deviations from a mean concrete temperature of 20 °C for the range of approximately 0 °C to +80 °C is dealt with.

5.1.10.2 Maturity

Eq. (5.1-85), originally developed for normal strength concrete, is based on an activation energy for cement hydration of 33 kJ/mol. Research has shown, that the activation energy does not only depend on the type and strength class of cement, but also on the water-cement ratio, additions and admixtures. Nevertheless, there is no data basis available which would enable a modification of Eq. (5.1-85) regarding the use of additions and admixtures in common normal strength and high strength concretes.

The effect of elevated or reduced temperatures on the maturity of concrete may be taken into account by adjusting the concrete age according to Eq. (5.1-85):

$$t_T = \sum_{i=1}^{n} \Delta t_i \, exp\left[13.65 - \frac{4000}{273 + T(\Delta t_i)}\right] \tag{5.1-85}$$

where

t_T is the temperature adjusted concrete age which replaces t in the corresponding equations in [days]

Δt_i is the number of days where a temperature T prevails

$T(\Delta t_i)$ is the temperature in [°C] during the time period Δt_i

5.1.10.3 Thermal expansion

The coefficient of thermal expansion depends on the type of aggregates and on the moisture state of the concrete. Thus it may vary between approx. $6 \cdot 10^{-6}$ K^{-1} and $15 \cdot 10^{-6}$ K^{-1}. For design a value of $10 \cdot 10^{-6}$ K^{-1} may be taken for normal strength and high strength concrete, as well as self-compacting concrete.

Dependent on the stiffness and the coefficient of thermal expansion of the aggregates the coefficient of thermal expansion of lightweight aggregate concrete ranges between $5 \cdot 10^{-6}$ K^{-1} and $11 \cdot 10^{-6}$ K^{-1}. For design a mean value of $8 \cdot 10^{-6}$ K^{-1} may be assumed.

The coefficient of thermal expansion of concrete containing high amounts of fly ash (e.g. green concrete) may be assumed to be $\alpha_T = 10 \cdot 10^{-6}$ K^{-1}.

In case the concrete structure reacts sensible to thermal strains, tests should be performed according to:

RILEM TC 129-MHT: Test methods for mechanical properties of concrete at high temperatures. Recommendations Part 6: Thermal strain. In: Materials and Structures, Supplement March 1997, pp. 17-21

Thermal expansion of concrete may be calculated from Eq. (5.1-86):

$$\varepsilon_{cT} = \alpha_T \Delta T \qquad (5.1\text{-}86)$$

where

ε_{cT} is the thermal strain

Δt is the change of temperature in [K]

α_T is the coefficient of thermal expansion in [K^{-1}]

For the purpose of structural analysis the coefficient of thermal expansion may be taken as $\alpha_T = 10 \cdot 10^{-6}$ K^{-1} for normal weight concrete and $\alpha_T = 8 \cdot 10^{-6}$ K^{-1} for lightweight aggregate concrete.

5.1.10.4 Compressive strength

Eq. (5.1-87) is valid for sealed and unsealed concrete tested in the hot state shortly after completion of the heating. Considering all experimental data a large scatter of the compressive strength values can be observed. If a higher accuracy is required tests must be performed, e.g. according to:

RILEM TC 129-MHT: Test methods for mechanical properties of concrete at high temperatures. Recommendations: Compressive strength for service and accident conditions. In: Materials and Structures, Vol. 28, 1995, pp. 410-414

Sustained moderately elevated temperatures may slightly increase the compressive strength compared to strength development at normal ambient environment if drying of member is possible.

So far no information is available for self-compacting concrete and green concrete.

The effect of temperature in the range of 0 °C ≤ T ≤ 80 °C on the compressive strength of normal strength and high strength normal weight and lightweight aggregate concrete, $f_{cm}(T)$ and $f_{lcm}(T)$, respectively, may be calculated from Eq. (5.1-87a/b):

$$f_{cm}(T) = f_{cm}(1.06 - 0.003 \cdot T) \qquad (5.1\text{-}87a)$$

$$f_{lcm}(T) = f_{lcm}(1.04 - 0.002 \cdot T) \qquad (5.1\text{-}87b)$$

where

$f_{cm}(T)$, $f_{lcm}(T)$ compressive strength in [MPa] at the temperature T in [°C]

f_{cm}, f_{lcm} compressive strength in [MPa] at the temperature 20 °C from eqs. 5.1-1 and 5.1-2

T is the temperature in [°C]

5.1.10.5 Tensile strength and fracture properties

In the range of of 0 °C ≤ T ≤ 80 °C the uniaxial tensile strength f_{ct} of normal strength concrete is significantly affected by temperature according to the following equation:

$$f_{ctm}(T) = f_{ctm}(1.16 - 0.008 \cdot T) \quad (5.1\text{-}88)$$

where

$f_{ctm}(T)$ is the uniaxial tensile strength in [MPa] at the temperature T in [°C]

f_{ctm} is the uniaxial tensile strength in [MPa] at the temperature 20 °C from Eq. (5.1-3)

T is the temperature in [°C]

No information is available on high strength concrete, self-compacting concrete, lightweight aggregate concrete and green concrete.

If the tensile strength is a major input parameter in the design of a structure the values calculated by Eq. (5.1-88) may be reduced or increased by 20%.

Tests may be performed according to:

RILEM TC 129-MHT: Test methods for mechanical properties of concrete at high temperatures. Recommendations Part 4: Tensile strength for service and accident conditions. In: Materials and Structures, Vol. 33, May 2000, pp. 219-223

In the range of 0 °C ≤ T ≤ 80 °C the dependency of the splitting tensile strength $f_{ct,sp}$ on temperature of normal strength normal weight concrete at the time of testing is described by the following equation:

$$f_{ct,sp}(T) = f_{ct,sp}(1.06 - 0.003 \cdot T) \quad (5.1\text{-}89)$$

where

$f_{ct,sp}(T)$ is the tensile splitting strength in [MPa] at the temperature T in [°C]

$f_{ct,sp}$ is the tensile splitting strength in [MPa] at the temperature 20 °C

T is the temperature in [°C]

No information is available on high strength concrete, self-compacting concrete, lightweight aggregate concrete and green concrete.

Eq. (5.1-90) may be used to estimate the effect of elevated or reduced temperatures on flexural strength $f_{ct,fl}$ of normal strength normal weight concrete:

$$f_{ct,fl}(T) = f_{ct,fl}(1.1 - 0.005 \cdot T) \quad (5.1\text{-}90)$$

where

$f_{ct,fl}(T)$ is the flexural strength in [MPa] at the temperature T in [°C]

$f_{ct,fl}$ is the flexural strength in [MPa] at the temperature 20 °C

If moisture gradients may occur, the flexural tensile strength may be lower up to 20 %.

No information is available on high strength concrete, self-compacting concrete, lightweight aggregate concrete and green concrete.

The eqs. 5.1-91a and 5.1-91b might describe the related effect somewhat more pronounced than observed in some cases. Available experimental data show a considerable broad scatter band.

No information is available on high strength concrete, self-compacting concrete, lightweight aggregate concrete and green concrete.

T is the temperature in [°C]

Fracture energy G_F is strongly affected by temperature and moisture content at the time of testing. The effect of temperature on G_F of normal strength normal weight concrete may be estimated from eqs. 5.1-91a/b:

dry concrete: $$G_F(T) = G_F(1.06 - 0.003 \cdot T) \quad (5.1\text{-}91a)$$

mass concrete: $$G_F(T) = G_F(1.12 - 0.006 \cdot T) \quad (5.1\text{-}91b)$$

where

$G_F(T)$ is the fracture energy in [N/m] at a temperature T in [°C]

G_F is the fracture energy in [N/m] at a temperature of 20 °C from Eq. (5.1-9)

T is the temperature in [°C]

5.1.10.6 Modulus of elasticity

Eq. (5.1-92) is valid for sealed and unsealed concrete.

No information is available on self-compacting concrete.

If the structure reacts sensible to concrete stiffness, tests are recommended according to:

RILEM TC 129-MHT: Test methods for mechanical properties of concrete at high temperatures. Recommendations: Modulus of elasticity for service and accident conditions. In: Materials and Structures, Vol. 37, March 2004, pp. 139-144

The effect of elevated or reduced temperatures at the time of testing on the modulus of elasticity of normal strength and high strength normal weight concrete and lightweight aggregate concrete at an age of 28 days may be estimated from Eq. (5.1-92):

$$E_{ci}(T) = E_{ci}(1.06 - 0.003 \cdot T) \quad (5.1\text{-}92a)$$

$$E_{lci}(T) = E_{lci}(1.04 - 0.002 \cdot T) \quad (5.1\text{-}92b)$$

where

$E_{ci}(T)$, $E_{lci}(T)$ modulus of elasticity in [MPa] at the temperature T in [°C]

E_{ci}, E_{lci} modulus of elasticity in [MPa] at the temperature 20 °C from Eq. (5.1-20) and (5.1-22) in [MPa]

T is the temperature in [°C]

5.1.10.7 Creep and Shrinkage

5.1.10.7.1 Creep

The relations to predict the effect of temperature up to 80 °C on creep

The effect of temperature prior to loading may be taken into account using given in this clause are only rough estimates. For a more accurate prediction considerably more sophisticated models are required which take into account the moisture state of the concrete at the time of loading and distinguish between basic creep and drying creep in more detail. Neglecting these parameters the relations given in this clause are generally more accurate for thick concrete members with little change in moisture content than for thin members where significant changes in moisture content occur, particularly at elevated temperatures.

There is no information available on self-compacting concrete, lightweight aggregate concrete and green concrete.

If the structure reacts sensible to concrete creep, tests are recommended according to:

RILEM TC 129-MHT: Test methods for mechanical properties of concrete at high temperatures. Recommendations Part 8: Steady-state creep and creep recovery for service and accident conditions. In: Materials and Structures, Vol. 33, January-February 2000, pp. 6-13.

If the structure reacts sensible to concrete creep, tests are recommended according to:

RILEM TC 129-MHT: Test methods for mechanical properties of concrete at high temperatures. Recommendations Part 7: Transient creep for service and accident conditions. In: Materials and Structures, Vol. 31, June

Eq. (5.1-85).

Eqs. (5.1-93) to (5.1-96) describe the effect of a constant temperature differing from 20 °C while a normal weight concrete is under load.

The effect of temperature on the time development of creep is taken into account using $\beta_{H,T}$ from Eq. (5.1-93):

$$\beta_{H,T} = \beta_H \cdot \beta_T \tag{5.1-93}$$

with

$$\beta_T = exp\left[1500/\left(273+T\right)-5.12\right] \tag{5.1-94}$$

where

$\beta_{H,T}$ is a temperature dependent coefficient replacing β_H in Eq. (5.1-69)

β_H is a coefficient according to Eq. (5.1-70)

T is the temperature in [°C]

The effect of temperature on the creep coefficient is taken into account using Eqs. (5.1-95) and (5.1-96):

$$\varphi_{RH,T} = \varphi_T + \left(\varphi_{RH} - 1\right)\varphi_T^{1.2} \tag{5.1-95}$$

with

$$\varphi_T = exp\left[0.015\left(T-20\right)\right] \tag{5.1-96}$$

where

$\varphi_{RH,T}$ is a temperature dependent coefficient which replaces φ_{RH} in Eq. (5.1-64)

φ_{RH} is a coefficient according to Eq. (5.1-65)

T is the temperature in [°C]

For an increase of temperature while the structural member is under load, creep may be estimated from Eq. (5.1-97):

$$\varphi\left(t,t_0,T\right) = \varphi_0 \beta_c\left(t,t_0\right) + \Delta\varphi_{T,trans} \tag{5.1-97}$$

with

1998, pp. 290-295.

Eq. (5.1-99) is a simplification as some experiments indicate not only an acceleration of shrinkage but also an increased autogenous shrinkage deformation if the concrete is subjected to ongoing elevated curing temperatures. This effect decreases with increasing concrete strength.

If shrinkage is a major input parameter tests may be performed according to:

RILEM TC 129-MHT: Test methods for mechanical properties of concrete at high temperatures. Recommendations Part 7: Shrinkage for service and accident conditions. In: Materials and Structures, Vol. 33, May 2000, pp. 224-228.

$$\Delta\varphi_{T,trans} = 0.0004(T-20)^2 \quad (5.1\text{-}98)$$

where

φ_0 is the notional creep coefficient according to Eq. (5.1-64) and temperature adjusted according to Eq. (5.1-95)

$\beta_c(t,t_0)$ is a coefficient to describe the development of creep with time after loading according to Eq. (5.1-69) and temperature adjusted according to Eq. (5.1-93)

$\Delta\varphi_{T,trans}$ is the transient thermal creep coefficient which occurs at the time of the temperature increase

T is the temperature in [°C]

5.1.10.7.2 Shrinkage

Temperatures between 0 °C and 80 °C mainly influence the time-development of autogenous shrinkage. Therefore, as given in Eq. (5.1-99), the autogenous shrinkage at concrete age t is calculated using the effective concrete age t_T according to Eq. (5.1-85):

$$\varepsilon_{cas}(t) = \varepsilon_{cas0}(f_{cm}) \cdot \beta_{as}(t_T) \quad (5.1\text{-}99)$$

$\varepsilon_{cas0}(f_{cm})$ is the notional shrinkage coefficient according to Eq. (5.1-78) and $\beta_{as}(t_T)$ the time function according to Eq. (5.1-79).

The effect of a constant temperature differing from 20 °C while the concrete is drying is described by means of Eqs. (5.1-100) to (5.1-105).

The effect of temperature on the time-development of drying shrinkage is taken into account using $\alpha_{sT}(T)$ from Eq. (5.1-100):

$$\alpha_{sT}(T) = 0.035 \cdot h^2 \exp[-0.06(T-20)] \quad (5.1\text{-}100)$$

where

$\alpha_{sT}(T)$ is a temperature dependent coefficient replacing the product $0.035h^2$ in Eq. (5.1-82)

T is the temperature in [°C]

The effect of temperature on the notional shrinkage coefficient is taken into account using Eqs. (5.1-101) to (5.1-105):

$$\beta_{RH,T} = \beta_{RH} \cdot \beta_{sT} \tag{5.1-101}$$

$\beta_{RH,T}$ is a temperature dependent coefficient which replaces β_{RH} in Eq. (5.1-77). It is calculated using Eq. (5.1-102):

$$\beta_{sT} = 1 + \left(\frac{4}{103 - RH}\right) \cdot \left(\frac{T-20}{40}\right) \tag{5.1-102}$$

$$\beta_{RH} = \begin{cases} -1.55 \cdot \left[\ 1 - \left(\dfrac{RH}{100}\right)^3\ \right] & \textit{for } 40 \le RH < RH_T \\ 0.25 & \textit{for } RH \ge RH_T \end{cases} \tag{5.1-103}$$

$$RH_T = 99 \cdot \beta_{s1} + \beta_{s1,T} \le 100\ \% \tag{5.1-104}$$

$$\beta_{s1} = \left(\frac{35}{f_{cm}}\right)^{0.1} \le 1.0 \quad \text{(see Eq. (5.1-83))}$$

$$\beta_{s1,T} = \left(\frac{T-20}{25}\right)^3 \tag{5.1-105}$$

where

RH is the relative humidity of the ambient environment in [%]

T is the temperature in [°C]

f_{cm} is the mean compressive strength at the age of 28 days in [MPa] according to Eq. (5.1-1).

The effect of elevated temperatures on shrinkage is influenced considerably by the moisture content of the concrete prior to heating and the moisture loss after an increase of temperature.

Whether a concrete specimen is shrinking or swelling under certain ambient climate conditions is determined by its internal relative humidity and the temperature dependent water sorption capacity. The transition between shrinkage and swelling (RH_T) is therefore dependent on the concrete compressive strength and the concrete temperature.

5.1.10.8 Effect of high temperatures

The material properties are significantly affected by the influence of high temperatures. Changes to the microstructure of concrete during a high temperature exposure result in corresponding changes in mechanical and physical properties which are mainly caused by the thermo-hydraulic and thermo-

With regard to the material properties of concrete at high temperatures reference is made to *fib* Bulletin 38 “Fire design of concrete structures - materials, structures and modelling” and 46 “Fire design of concrete structures - structural behaviour and assessment” as well as to chapter 3 “Material properties” in EN 1992-1-2:2004 “Eurocode 2: Design of concrete structures -

Part 1-2: General rules - structural fire design".

mechanical behaviour of the material.

From a structural fire design point of view reference is made to clause 7.5.1 of this Model Code.

5.1.10.9 Low temperature (cryogenic temperature)

Approximate moisture content of structural concrete [% by mass]:

– general indoor: 2 %

– general outdoor: 4 %

– exposed to rain: 6 %

Eq. (5.1-106) is also valid for lightweight aggregate concrete.

No information is available for self-compacting concrete. However, it is supposed that the behaviour of self-compacting concrete deviates not significantly from ordinary structural concrete; i.e. data on self-compacting concrete are supposed to meet the scatter band of ordinary concrete.

The compressive strength increases at low temperature as a function of temperature and moisture content of the concrete. The strength gain can be estimated from Eq. (5.1-106):

$$\Delta f_{cm} = 12m\left[1-\left(\frac{T+170}{170}\right)^2\right] \qquad (5.1\text{-}106)$$

with

Δf_{cm} gain in compressive strength in [MPa]

m is the moisture content in [% by mass]

T is the temperature in [°C]

Eq. (5.1-106) is valid for T between $0° \geq T \geq -170$ °C, for a single temperature drop.

Tensile strength and modulus of elasticity increase at low temperature. They can be estimated by means of eqs. 5.1-3 and 5.1-20 inserting the respective compressive strength.

5.1.11 Properties related to non-static loading

5.1.11.1 Fatigue

5.1.11.1.1 Fatigue strength

Fatigue tests exhibit a large scatter in the number of cycles to failure. Therefore, often probabilistic procedures are applied in evaluating fatigue behaviour of concrete. For further details refer to 'Fatigue of concrete structures', State-of-the-Art Report, CEB Bulletin 188, Lausanne, 1988 and ´Constitutive modelling of high strength / high performance concrete´, State-of-the-Art Report, *fib* Bulletin 42, Lausanne, 2008.

For constant stress amplitude the number N cycles causing fatigue failure of plain concrete may be estimated from equations 5.1-107 to 5.1-112. They are valid for pure compression, compression-tension and pure tension, respectively.

(I) Pure compression

The relations in eqs. 5.1-107 to 5.1-109 are valid for concrete tested under sealed conditions. They are also valid for large concrete sections of low permeability. Thin concrete sections which are allowed to dry may exhibit higher fatigue strengths.

However, for concrete with lightweight aggregates, which is allowed to dry, the increase in strength is not verified.

On the other hand, permeable concrete immersed in water may have a lower fatigue strength than expressed by these relations. If pores are filled with water, even lower fatigue strength may be obtained due to water pressure. Although, if the fatigue strength is related to the static strength the various concrete types exhibit similar response.

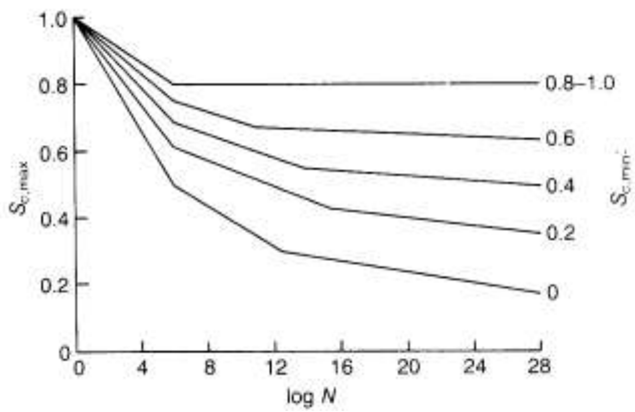

Figure 5.1-7: S-N relations according to eqs. 5.1-107 to 5.1-109

The fatigue reference compressive strength $f_{ck,fat}$ has been introduced to take into account the increasing fatigue sensitivity of concrete with increasing compressive strength.

Due to the higher paste content of self compacting concrete and different pore structure the fatigue strength is lower than that of ordinary concrete. If no tests are performed on the concrete to be used, the fatigue reference compressive strength according to Eq. (5.1-110) should be reduced by 20 %.

For $S_{c,min}$ > 0.8, the S-N relations for $S_{c,min}$ = 0.8 are valid. For $0 \leq S_{c,min} \leq 0.8$, eqs. 5.1-107 to 5.1-109 apply:

$$\log N_1 = \left(12 + 16 S_{c,\min} + 8 S_{c,\min}^2\right)\left(1 - S_{c,\max}\right) \tag{5.1-107}$$

$$\log N_2 = 0.2 \log N_1 \left(\log N_1 - 1\right) \tag{5.1-108}$$

$$\log N_3 = \log N_2 \left(0.3 - 0.375 S_{c,\min}\right) / \Delta S_c \tag{5.1-109}$$

(a) if $\log N_1 \leq 6$, then $\log N = \log N_1$

(b) if $\log N_1 > 6$ and $\Delta S_c \geq 0.3 - 0.375 S_{c,\min}$, then $\log N = \log N_2$

(c) if $\log N_1 > 6$ and $\Delta S_c < 0.3 - 0.375 S_{c,\min}$, then $\log N = \log N_3$

with

$$S_{c,\max} = \left|\sigma_{c,\max}\right| / f_{ck,fat}$$

$$S_{c,\min} = \left|\sigma_{c,\min}\right| / f_{ck,fat}$$

$$\Delta S_c = \left|S_{c,\max}\right| - \left|S_{c,\min}\right|$$

The fatigue reference compressive strength $f_{ck,fat}$ may be estimated from equation 5.1-110:

$$f_{ck,fat} = \beta_{cc}(t)\,\beta_{c,sus}(t,t_0)\,f_{ck}\left(1 - f_{ck}/250\right) \tag{5.1-110}$$

If Eq. (5.1-111) is applied it may be assumed that the concrete always fails in compression.

For concrete in tension, the crack propagation can be different for various types of concrete due to the difference in the internal structure. For normal concrete the crack propagates in the cement paste and in the interface around the aggregates. However, for high strength concrete and concrete with light-weight aggregates the crack propagates in the cement paste and through the aggregates due to the relatively higher strength of the cement paste. Thus, concrete types where the strength of the aggregates is of importance, the fatigue life of the aggregates also should be considered. However, test results have shown that the fatigue life seems relatively equal for the various concrete types, see *fib* Bulletin 42.

The fatigue lives given by these equations correspond to a probability of failure p = 5 % in a log-normal distribution for any given maximum stress. If limited data are available for an estimate of fatigue lives the evaluation of the 5 % defective of fatigue life should be done at a confidence level of 75 %.

Eqs. 5.1-107 to 5.1-112 are applicable for stress levels $S_{c,max}$ and $S_{ct,max}$ < 0.9 and for frequencies f > 0.1 cycle/min. For higher stress levels and lower frequencies, i.e. low cycle fatigue, lower values of log N than predicted by eqs. 5.1-107 to 5.1-112 may be expected. For further details refer to CEB Bulletin 188.

A value of $\beta_{c,sus}(t,t_0)$ = 0.85 has been chosen to take account of actual frequencies of loading which are in most practical cases significantly lower than those applied in experiments.

(II) Compression-tension with $\sigma_{ct,\max} \leq 0.026\left|\sigma_{c,\max}\right|$

$$\log N = 9\left(1 - S_{c,\max}\right) \quad (5.1\text{-}111)$$

(III) Pure tension and tension-compression with $\sigma_{ct,\max} > 0.026\left|\sigma_{c,\max}\right|$

$$\log N = 12\left(1 - S_{ct,\max}\right) \quad (5.1\text{-}112)$$

with

$S_{ct,\max} = \sigma_{ct,\max} / f_{ctk,\min}$

N	is the number of cycles to failure
$S_{c,max}$	is the maximum compressive stress level
$S_{c,min}$	is the minimum compressive stress level
$S_{ct,max}$	is the maximum tensile stress level
ΔS_c	is the stress level range
$\sigma_{c,max}$	is the maximum compressive stress in [MPa]
$\sigma_{c,min}$	is the minimum compressive stress in [MPa]
$\sigma_{ct,max}$	is the maximum tensile stress in [MPa]
f_{ck}	is the characteristic compressive strength from Table 5.1-3
$f_{ck,fat}$	is the fatigue reference compressive strength from Eq. (5.1-110)
$f_{ctk,min}$	is the minimum characteristic tensile strength
$\beta_{cc}(t)$	is a coefficient which depends on the age of concrete at the beginning of fatigue loading, to be taken from sub-clause 5.1.9.1, Eq. (5.1-51)
$\beta_{c,sus}(t,t_0)$	is a coefficient which takes into account the effect of highmean stresses during loading. For fatigue loading it may be taken as 0.85

(IV) Spectrum of load-levels

The value of the Palmgren-Miner sum indicating failure is varying in various codes from 0.2 to 1.0. Consequently, the Palmgren-Miner rule is only a very rough approximation of the actual concrete behaviour. It may over- or underestimate the actual fatigue strength of concrete subjected to varying repeated loads. Rest periods in the loading may increase the fatigue life.

Different parts in concrete area are exposed to changing maximum and minimum stress levels. The different parts have to be treated using, e.g. Palmgren-Miner rule. Numerical simulations with for example the finite element method allows for treating this effectively.

In Eq. (5.1-114) it is assumed that creep due to repeated loading is equal to creep under a constant stress ($|\sigma_{c,max}|$) + $|\sigma_{c,min}|$)/2 acting during a time ($t - t_0$) = (1/1440)·(n/f) = duration of repeated loading [days], where

n is the number of cycles applied at a frequency f

f is the frequency of repeated loading [min^{-1}]

Therefore, Eq. (5.1-114) gives only a rough estimate of the creep strains due to repeated loads. It does not take into account variations of E_c due to repeated loads as well as of tertiary creep which develops prior to fatigue failure. For further details refer to CEB Bulletin 188.

To estimate the fatigue life for a spectrum of load levels the Palmgren-Miner summation may be applied. Fatigue failure occurs if $D = 1$.

$$D = \sum_i \frac{n_{Si}}{n_{Ri}} \tag{5.1-113}$$

where

D is the fatigue damage

n_{Si} is the number of acting stress cycles at a given stress level and stress range

n_{Ri} is the number of cycles causing failure at the same stress level and stress range according to eqs. 5.1-107 to 5.1-112.

5.1.11.1.2 Fatigue strains

For maximum compressive stresses $|\sigma_{c,max}| < 0.6 f_{ck}$ and a mean stress ($|\sigma_{c,max}|$) + $|\sigma_{c,min}|$)/2 < $0.5 f_{ck}$ the strain at maximum stress due to repeated loads of a given frequency f may be estimated from Eq. (5.1-114):

$$\varepsilon_{cf}\left(n\right) = \frac{\sigma_{c,\max}}{E_{ci}\left(t_0\right)} + \frac{\sigma_{c,\max} + \sigma_{c,\min}}{2E_{ci}} \varphi\left(t, t_0\right) \tag{5.1-114}$$

where

ε_{cf} is the strain at maximum stress due to repeated loads

$\sigma_{c,max}$ is the maximum compressive stress in [MPa]

$\sigma_{c,min}$ is the minimum compressive stress in [MPa]

E_{ci} is the modulus of elasticity of concrete in [MPa] at a concrete age of 28 days according to Eq. (5.1-20)

$E_{ci}(t_0)$ is the modulus of elasticity of concrete in [MPa] at a concrete age t_0 according to Eq. (5.1-56)

$\varphi(t,t_0)$ is the creep coefficient according to Eq. (5.1-63)

t_0 is the age of concrete at the beginning of repeated loading in [days]

t is the age of concrete at the moment considered in [days]

The given constitutive relations are valid also for lightweight aggregate concrete.

No information is available for self-compacting concrete. However, it is supposed that the behaviour of self-compacting concrete deviates not significantly from ordinary structural concrete; i.e. data on self-compacting concrete are supposed to meet the scatter band of ordinary concrete.

5.1.11.2 Stress and strain rate effects – impact

5.1.11.2.1 Range of applicability

The information given below as well as in sub-clauses 5.1.4, 5.1.5 and 5.1.7 is valid for monotonically increasing compressive stresses or strains at a constant range of approximately 1 MPa/s < $\left|\dot{\sigma}_c\right|$ < 10^7 MPa/s and $30 \cdot 10^{-6}$ s^{-1} < $\left|\dot{\varepsilon}_c\right|$ < $3 \cdot 10^2$ s^{-1}, respectively. In the correspondent equations all strain and stress values have to be used as absolute values.

For tensile stresses or strains the information is valid for 0.03 MPa/s < $\dot{\sigma}_{ct}$ < 10^7 MPa/s and $1 \cdot 10^{-6}$ s^{-1} < $\dot{\varepsilon}_{ct}$ < $3 \cdot 10^2$ s^{-1}, respectively.

5.1.11.2.2 Compressive strength

For a given strain and stress rate, respectively, the compressive strength under high rates of loading may be estimated from eqs. 5.1-115 and 5.1-116:

$$f_{c,imp,k} / f_{cm} = \left(\dot{\varepsilon}_c / \dot{\varepsilon}_{c0}\right)^{0.014} \qquad \text{for } \dot{\varepsilon}_c \leq 30\, s^{-1} \qquad (5.1\text{-}115a)$$

$$f_{c,imp,k} / f_{cm} = 0.012\left(\dot{\varepsilon}_c / \dot{\varepsilon}_{c0}\right)^{1/3} \qquad \text{for } \dot{\varepsilon}_c > 30\, s^{-1} \qquad (5.1\text{-}115b)$$

with $\dot{\varepsilon}_{c0}$ = $30 \cdot 10^{-6}$ s^{-1}

and

$$f_{c,imp,k} / f_{cm} = \left(\dot{\sigma}_c / \dot{\sigma}_{c0}\right)^{0.014} \qquad \text{for } \dot{\sigma}_c \leq 10^6 \text{ MPa s}^{-1} \qquad (5.1\text{-}116a)$$

$$f_{c,imp,k} / f_{cm} = 0.012\left(\dot{\sigma}_c / \dot{\sigma}_{c0}\right)^{1/3} \qquad \text{for } \dot{\sigma}_c > 10^6 \text{ MPa s}^{-1} \qquad (5.1\text{-}116b)$$

with $\dot{\sigma}_{c0}$ = 1 MPa s^{-1}

5.1.11.2.3 Tensile strength and fracture properties

(a) Tensile strength

For a given strain and stress rate, respectively, the tensile strength under high rates of loading may be estimated from eqs. 5.1-117 and 5.1-118:

$$f_{ct,imp,k} / f_{ctm} = \left(\dot{\varepsilon}_{ct} / \dot{\varepsilon}_{ct0}\right)^{0.018} \quad \text{for } \dot{\varepsilon}_{ct} \leq 10\, s^{-1} \tag{5.1-117a}$$

$$f_{ct,imp,k} / f_{ctm} = 0.0062\left(\dot{\varepsilon}_{ct} / \dot{\varepsilon}_{ct0}\right)^{1/3} \quad \text{for } \dot{\varepsilon}_{ct} > 10\, s^{-1} \tag{5.1-117b}$$

with $\dot{\varepsilon}_{ct0} = 1\cdot10^{-6}$

and

$$f_{ct,imp,k} / f_{ctm} = \left(\dot{\sigma}_{ct} / \dot{\sigma}_{ct0}\right)^{0.018} \quad \text{for } \dot{\sigma}_{ct} \leq 0.3\cdot10^{6} \text{ MPa s}^{-1} \tag{5.1-118a}$$

$$f_{ct,imp,k} / f_{ctm} = 0.0062\left(\dot{\sigma}_{ct} / \dot{\sigma}_{ct0}\right)^{1/3} \quad \text{for } \dot{\sigma}_{ct} > 0.3\cdot10^{6} \text{ MPa s}^{-1} \tag{5.1-118b}$$

with $\dot{\sigma}_{ct0} = 0.03$ MPa s^{-1}

(b) Fracture energy

The information available regarding the effect of stress or strain rate on the fracture energy is too incomplete to be included in this Model Code.

5.1.11.2.4 Modulus of elasticity

The effect of stress and strain rate on the modulus of elasticity may be estimated from Eq. (5.1-119):

$$E_{c,imp} / E_{ci} = \left(\dot{\sigma}_{c} / \dot{\sigma}_{c0}\right)^{0.025} \tag{5.1-119a}$$

$$E_{c,imp} / E_{ci} = \left(\dot{\varepsilon}_{c} / \dot{\varepsilon}_{c0}\right)^{0.026} \tag{5.1-119b}$$

with $\dot{\sigma}_{c0} = 1$ MPa s^{-1} and $\dot{\varepsilon}_{c0} = 30\cdot10^{-6}$ s^{-1} for compression

with $\dot{\sigma}_{ct0} = 0.03$ MPa s^{-1} and $\dot{\varepsilon}_{ct0} = 1\cdot10^{-6}$ s^{-1} for tension

5.1.11.2.5 Stress-strain diagrams

No information is available for the strain-softening region.

There is little information regarding the effect of high stress or strain rates on the shape of the stress-strain diagrams.

As an approximation, for monotonically increasing compressive stresses or strains up to the peak stress, Eq. (5.1-26) may be used together with Eqs. (5.1-115) and (5.1-116) for the peak stress $f_{c,imp}$, Eq. (5.1-119) for the modulus of elasticity $E_{c,imp}$, and Eq. (5.1-120) for the strain at maximum stress $\varepsilon_{c1,imp}$.

The effects of high stress and strain rates on the strains at maximum stress in tension and compression may be estimated from Eq.(5.1-120):

$$\varepsilon_{c1,imp} / \varepsilon_{c1} = \left(\dot{\sigma}_c / \dot{\sigma}_{c0}\right)^{0.02} = \left(\dot{\varepsilon}_c / \dot{\varepsilon}_{c0}\right)^{0.02} \qquad (5.1\text{-}120)$$

with $\dot{\sigma}_{c0}$ = 1 MPa s^{-1} and $\dot{\varepsilon}_{c0}$ = $30 \cdot 10^{-6}$ s^{-1} for compression

with $\dot{\sigma}_{ct0}$ = 0.03 MPa s^{-1} and $\dot{\varepsilon}_{ct0}$ = $1 \cdot 10^{-6}$ s^{-1} for tension

where

$\varepsilon_{c1,imp}$ is the impact strain at maximum load

ε_{c1} is the strain a maximum load for static loading from sub-clauses 5.1.8.1 and 5.1.8.2 for compression and tension, respectively.

5.1.12 Transport of liquids and gases in hardened concrete

The subsequent relations are valid for normal and high strength normal weight concrete according to sub-clause 5.1.2 unless otherwise noted.

Liquids, gases or ions may be transported in hardened concrete by the transport mechanisms permeation, diffusion, capillary suction and by mixed modes of transport mechanisms.

Transport characteristics are difficult to predict since the may vary by several orders of magnitude depending on concrete composition (e.g. water/cement ratio), type of materials (e.g. cement, puzzolanic additives), age, curing and moisture content of the concrete (e.g. storing conditions).

The relations presented in this chapter may be assumed as reasonable approximations. However all relations correlated with compressive strength have to be handled with carefully, as the compressive strength represents first a substitute value for the microstructure and second a mean value over the whole concrete cross-section whereas the transport characteristics in the concrete cover are authoritative concerning concrete durability. Therefore, when a more accurate prediction of transport characteristics is required, they should be determined experimentally.

For further details concerning the transport properties of normal weight concrete refer to RILEM TC 116 PCD, State-of-the-Art Report: Performance Criteria for Concrete Durability (1995) or to RILEM TC 146 TCF, State-of-the-art Report: Penetration and Permeability of Concrete: Barriers to organic and contaminating liquids (1997) as well as to *fib* Bulletins 51 and 53, "structural concrete", Textbook (2010)

Self-compacting concrete (SCC) with a comparable strength exhibits a denser microstructure than normal weight concrete, so that the models presented in this chapter should be on the safe side for SCC. Nevertheless, for further details concerning self-compacting concrete refer to RILEM TC 205 DSC, State-of-the-art Report: Durability of Self-Compacting Concrete (2007).

Regarding lightweight concrete, it appears that its transport coefficients are slightly lower compared to normal strength concrete of the same grade mainly due to the usually higher quality of the inner contact zone. However this difference becomes negligible for higher strength grades. Further details concerning lightweight aggregate concrete can be found in e.g. Faust, T., "Leichtbeton im konstruktiven Ingenieurbau", Verlag Ernst & Sohn, Berlin, 2002.

5.1.12.1 Permeation

Permeation is the flow of liquids, e.g. water, or of gases, e.g. air, caused by a pressure head.

5.1.12.1.1 Water permeability

In normal strength concrete the flow of water does not only occur in the capillary pores of the hydrated cement paste but also through internal microcracks as well as along the porous interfaces between the matrix and coarse aggregates. These effects increase the permeability of concrete which therefore equals or exceeds the permeability of the hydrated cement paste matrix.

The flow of water in the hydrated cement paste depends on the presence of interconnected capillary pores which are mainly determined by the water/cement ratio of the mix and the degree of hydration of the cement. Despite a low water/cement ratio, insufficient curing, which may result in a low degree of hydration especially in the near surface region, may lead to a high permeability.

The appropriate use of silica fume or fly ash (e.g. according to EN 206-1 "Concrete – Part 1: Specification, performance, production and conformity"),

The transport of water is generally described by Darcy's law, see Eq. (5.1-121):

$$V = K_w \frac{A}{l} \Delta h_w t \qquad (5.1\text{-}121)$$

where

V is the volume of water in [m³] flowing during time t

Δh_w is the hydraulic head in [m]

A is the penetrated area in [m²]

t is the time in [s]

l is the thickness in [m]

as it is often the case in high strength concrete, leads to a densification of the matrix and the porous interface because of the preceding puzzolanic reactions and the filler effect of those additives. Depending on age and composition of the concrete this effect can be even more pronounced than it is expressed by Eq. (5.1-122).

The experimental determination of the coefficient of water permeability is not standardised so far. However the penetration of water into concrete can be measured according to EN 12390-8 “Testing hardened concrete - Depth of penetration of water under pressure” and converted into a coefficient of water permeability, but it has to be considered as an approximate value only.

K_w is the coefficient of water permeability for water flow in [m/s]

For mature concrete the coefficient of water permeability may be estimated roughly from the mean compressive strength of concrete f_{cm} according to Eq. (5.1-122):

$$K_w = K_{w0} \cdot \frac{1}{f_{cm}^{\ 6}} \tag{5.1-122}$$

where

K_w is the coefficient of water permeability in [m/s]

K_{w0} = $4 \cdot 10^{-3}$ [m/s]

f_{cm} is the mean compressive strength in [MPa]

5.1.12.1.2 Gas permeability

For a stratified laminar flow the volume of gas flowing through a porous material is given by Eq. (5.1-123):

$$V = K_g \frac{A}{l} \frac{p_1 - p_2}{\eta} p_m \frac{1}{p} t \tag{5.1-123}$$

where

V is the volume of gas in [m³] flowing during time t

K_g is the coefficient of gas permeability in [m²]

A is the penetrated area in [m²]

l is the thickness in [m] of the penetrated section

$p_1 - p_2$ is the pressure difference in [N/m²]

p_m is the mean pressure = $(p_1 + p_2)/2$ in [N/m²]

η is the viscosity of gas in [Ns/m²]

p is the local pressure, at which V is observed in [Ns/m²]

t is the time in [s]

Similar to the flow of water, gases may pass through the pore system and micro-cracks of concrete under the influence of an external pressure. The coefficient of permeability K_g [m²] in Eq. (5.1-123) represents a constant material parameter. Therefore, the viscosity η of the gas flowing, as well as the pressure level p, have to be considered in the calculation of the volume of gas V.

If only one type of gas is considered η is normally taken as unity. Then K_g represents the specific permeability for the gas considered, and is given in [m/s].

If also the influence of the pressure level p_m is neglected, the volume of gas flowing can be calculated from

$$V = \bar{K}_g \frac{A}{l} \frac{p_1 - p_2}{p} t \tag{5.1-124}$$

where

$\bar{K}_g$ is the coefficient of gas permeability [m²/s]

As it is the case for water permeability lower water/cement ratio may lead to a lower coefficient of gas permeability with higher compressive strength. The use of additives (e.g. according to EN 206-1) may even result in a further

As a rough estimate, K_g for air, oxygen and nitrogen may be determined from the mean compressive strength of concrete f_{cm} from Eq. (5.1-125):

densification especially at very high strength grades.

Aside from the pore structure of the concrete, the moisture content exerts an essential influence on its gas permeability. Eq. (5.1-125) is valid for a relative pore humidity of the concrete of less than about 65 %. With increasing relative humidity of the concrete, K_g may be reduced by a factor up to 10^{-3}. In contrast for concrete specimens that have been oven-dried before testing, K_g should be assumed one magnitude higher.

Considering all experimental data a large scatter of the gas permeability values can be observed. Therefore, when a more accurate prediction is required, the gas permeability should be determined experimentally. This may be done according to the RILEM Technical recommendation: “Measurement of the gas permeability by RILEM – CEMBUREAU method”, Materials and Structures, Vol. 32, pp. 176-178, 1999.

In most cases transient diffusion phenomena occur, i.e. the amount of substance diffusing varies with location x and time t. From Fick’s first law of diffusion the balance for a volume element penetrated is derived as the second law of diffusion, which describes the change in concentration for an element with time according to Eq. (5.1-126) which is valid for one-dimensional flow:

$$\frac{\partial c}{\partial t} = D\frac{\partial^2 c}{\partial x^2} \tag{5.1-126}$$

In cases where the diffusing substance becomes immobile, such as in the case of diffusion of chloride ions, Eq. (5.1-126) has to be expanded:

$$\frac{\partial c}{\partial t} = D\frac{\partial^2 c}{\partial x^2} + s \tag{5.1-127}$$

where s = sink, i.e. amount of transported substance which becomes immobile.

Frequently, the diffusion of ions is described by Eq. (5.1-129):

$$\frac{\partial c_{free}}{\partial t} = D_{eff}\,\frac{\partial^2 c_{free}}{\partial x^2} \tag{5.1-129}$$

$$K_g = K_{g0} \cdot \frac{1}{f_{cm}^{\;4.5}} \tag{5.1-125}$$

where

K_g	is the coefficient of gas permeability in [m²]
K_{g0}	$= 2 \cdot 10^{-10}$ [m²]
f_{cm}	is the mean compressive strength in [MPa]

5.1.12.2 Diffusion

Gases, liquids and dissolved substances are transported due to a constant concentration gradient according to Fick’s first law of diffusion according to Eq. (5.1-128):

$$Q = D\frac{c_1 - c_2}{l}At \tag{5.1-128}$$

where

Q	is the amount of substance transported in [g]
$c_1 - c_2$	is the difference in concentration in [g/m³]
l	is the thickness of the penetrated section in [m]
A	is the penetrated area in [m²]
t	is the time in [s]
D	is the diffusion coefficient in [m²/s]

where c_{free} = concentration of free ions, D_{eff} = effective diffusion coefficient. If some of the ions become immobile, this is taken into account by an adjustment of the diffusion coefficient. Therefore, D_{eff} in Eq. (5.1-129) is not a constant but varies with time of exposure.

The transport of water vapour in the pore system of concrete involves different transport mechanisms and driving forces, therefore $D \neq$ const. In most cases diffusion theory is applied to describe moisture migration. As driving force the local moisture concentration c [g/m³] may be considered.

The diffusion coefficient D at local moisture concentration c may be determined experimentally according to EN 12086 “Determination of water vapour transmission properties”. This test method has been widely used for concrete specimens, but it has to be kept in mind that it is intended originally for thermal insulating products.

A more convenient approach to describe the water vapour diffusion is achieved by the definition of a relative pore humidity $0 < H < 1$ which is correlated with the moisture concentration c by sorption isotherms.

For transient phenomena, such as drying of a concrete cross-section, the balance equation 5.1-126 is transformed to Eq. (5.1-130):

$$\frac{\partial H}{\partial t} = \frac{\partial}{\partial x}\left(D(H) \frac{\partial H}{\partial x} \right) \qquad (5.1\text{-}130)$$

Eq. (5.1-132) is taken from Bazant, Z.P., Najjar, L.J., “Drying of concrete as a non-linear diffusion problem”, Cement and Concrete Research, Vol. 1, pp. 461-473, 1971.

Eq. (5.1-132) is valid for normal strength concrete only. No test data covering high strength concrete are available.

5.1.12.2.1 Diffusion of water

The transport of water in the vapour phase can be described by Fick’s first law of diffusion introducing a gradient of the relative pore humidity as the driving force. The diffusion coefficient D is a non-linear function of the local relative pore humidity H. The volume of water flowing is given by Eq. (5.1-131):

$$V = D(H) \frac{dH}{dx} A t \qquad (5.1\text{-}131)$$

where

V	is the volume of transported water in [m³]
$D(H)$	is the diffusion coefficient in [m²/s] at relative pore humidity H
dH/dx	is the gradient in relative pore humidity in [m^{-1}]
A	is the penetrated area in [m²]
t	is the time in [s]

For isothermal conditions the diffusion coefficient can be expressed as a function of the relative pore humidity $0 < H < 1$ according to Eq. (5.1-132):

$$D(H) = D_1 \left[\alpha + \frac{1-\alpha}{1 + \left[(1-H)/(1-H_c) \right]^n} \right] \qquad (5.1\text{-}132)$$

where

D_1	is the maximum of $D(H)$ for $H = 1$ in [m²/s]
D_0	is the minimum of $D(H)$ for $H = 0$ in [m²/s]
α	$= D_0/D_1$

H_c is the relative pore humidity at $D(H) = 0.5D_1$

n is an exponent

H is the relative pore humidity

The following approximate values may be assumed

α = 0.05

H_c = 0.80

n = 15

D_1 may be estimated from Eq. (5.1-131):

$$D_1 = \frac{D_{1,o}}{f_{cm} - 8} \tag{5.1-133}$$

where

$D_{1,o}$ = $1 \cdot 10^{-8}$ [m²/s]

f_{cm} is the mean compressive strength in [MPa]

5.1.12.2.2 Diffusion of gases

The diffusion of gases such as air, oxygen (O_2) or carbon dioxide (CO_2) is primarily controlled by the moisture content of the concrete. For intermediate moisture contents the diffusion coefficient for carbon dioxide or oxygen is in the range of $10^{-7} < D < 10^{-10}$ m²/s.

There exist so far no international standards to determine the diffusion coefficients of gases like oxygen or carbon dioxide.

The diffusion coefficient for oxygen D_{O2} through non-carbonated concrete may be determined following Eq. (5.1-135):

$$log\left(D_{O2}/D_{O2,O}\right) = -0.02\, f_{cm} \tag{5.1-135}$$

where

D_{O2} is the diffusion coefficient of O_2 in [m²/s]

$D_{O2,O}$ = $10^{-6.5}$ [m²/s]

f_{cm} is the mean compressive strength in [MPa]

Eqs. 5.1-135 and 5.1-136 are valid for normal strength concrete stored in a constant environment of approximately 20 °C, 65 % relative humidity. For concrete exposed to a natural environment, particularly to rain, the diffusion coefficients are substantially lower than estimated from Eq. (5.1-135) or Eq. (5.1-136).

Based on eqs. 5.1-127, 5.1-129 and 5.1-136 the progress of carbonation of a concrete under controlled conditions may be estimated from Eq. (5.1-134):

$$d_c^2 = 2D_{CO2}\frac{C_a}{C_c}t \tag{5.1-134}$$

where

The diffusion coefficient for carbon dioxide D_{CO2} through carbonated con-

d_c is the depth of carbonation at time t in [m]

D_{CO2} is the diffusion coefficient of CO_2 through carbonated concrete in [m²/s] (from Eq. (5.1-136))

C_a is the concentration of CO_2 in the air in [g/m³]

C_c is the amount of CO_2 required for complete carbonation of a unit volume of concrete in [g/m³]

For normal weight concrete made of Portland cement and exposed to a standard environment, C_a/C_c may be taken as $8 \cdot 10^{-6}$.

It should be kept in mind, however, that in particular the relative humidity of the surrounding atmosphere as well as the properties and the composition (e.g. the use of blast furnace slag cements) of a particular concrete have a strong influence on D_{CO2} so that Eq. (5.1-134) cannot give a reliable estimate of the progress of carbonation of a structure in service.

A more sophisticated model concerning the progress of carbonation is presented in sub-clause 5.1.13.2.

crete may be estimated from Eq. (5.1-136):

$$\log\left(D_{CO2}/D_{CO2,O}\right) = -0.05 f_{cm} \tag{5.1-136}$$

where

D_{CO2} is the diffusion coefficient of CO_2 in [m²/s]

$D_{CO2,O}$ = $10^{-6.1}$ [m²/s]

f_{cm} is the mean compressive strength in [MPa]

5.1.12.2.3 Diffusion of chloride ions

The diffusion coefficients of dissolved substances increase with increasing moisture content of the concrete.

The prediction of the transport of chloride ions into concrete is very complex because chlorides penetrating into concrete may be transported not only by diffusion but also by capillary suction of a salt solution. In addition, the external chloride concentration is variable, and parts of the chloride ions intruded become immobile due to chemical reaction or time dependant physical adsorption. The amount of bounded chlorides depends on the type of cement used and must be in equilibrium with the concentration of chlorides dissolved in the pore water. Only the dissolved chlorides take part in the diffusion process. In carbonated concrete all chlorides are dissolved in the pore water.

Eq. (5.1-137) is valid for normal and high strength Portland cement concrete without additives and a mean compressive strength f_{cm} lower than 95 MPa.

Eq. (5.1-138) may be used for normal and high strength concrete with reasonable amounts of silica fume, fly ash (e.g. according to EN 206-1) or blast

For chloride ions the effective diffusion coefficients in mature concrete as defined in Eq. (5.1-129) may be estimated from the compressive strength of concrete f_{cm} according to Eq. (5.1-137):

$$D_{Cl^-} = D_{Cl^-0} \cdot \frac{1}{f_{cm}^{\ 1.5}} \tag{5.1-137}$$

where

D_{Cl^-} is the effective coefficient of diffusion in [m²/s]

D_{Cl^-0} = $5 \cdot 10^{-9}$ [m²/s]

f_{cm} is the mean compressive strength in [MPa]

The appropriate use of additives or Portland blast furnace slag-cements may even lead to lower coefficients of diffusion which can be expressed by Eq. (5.1-138):

furnace slag cements up to a compressive strength f_{cm} of 130 MPa.

There exists so far no overall international standard for the determination of diffusion coefficients. However reasonable results can be achieved with an electrically accelerated method according to Tang, L., “Electrically accelerated methods for determining chloride diffusivity in concrete”, Magazine of Concrete Research, Vol. 48, pp. 173-179, 1996, which is standardised e.g. in Finland in NT Build 492.

Further information and a sophisticated model concerning the penetration of chlorides into concrete can be found in chapter 5.1.13.3.

Similar to water permeability, capillary suction is strongly influenced by the moisture content of the concrete. As the pore humidity of the concrete increases, the rate of water absorption and thus M_w decrease.

For a uniform pore humidity and no substantial microstructural variations within a concrete section exposed to capillary suction, the exponent n in Eq. (5.1-139) may be taken as n = 0.5. If the moisture distribution is non-uniform, n < 0.5.

Eq. (5.1-140) is valid for a uniform pore humidity of the concrete of approximately 65 % and for moderately oven-dried concrete. The coefficient of water absorption depends not only on the moisture state of the concrete, but also on microstructural parameters which are linked with concrete composition and type of materials used (e.g. water/cement ratio, silica fume, fly ash, etc.). Considering all experimental data, a large scatter of the capillary suction values has to be kept in mind, so that predictions solely based on a concrete strength are rather uncertain.

Therefore, when a more accurate prediction is required, the coefficient of water absorption may be determined experimentally according to EN ISO 15148 “Determination of water absorption coefficient by partial immersion” or alternatively according to RILEM Technical recommendation: “Determi-

$$D_{Cl^-,add} = D_{Cl^-0,add} \cdot \frac{1}{f_{cm}^{\,2.5}} \qquad (5.1\text{-}138)$$

where

$D_{Cl^-,add}$ is the effective coefficient of diffusion in [m²/s]

$D_{Cl^-0,add}$ = $5 \cdot 10^{-8}$ [m²/s]

f_{cm} is the mean compressive strength in [MPa]

5.1.12.3 Capillary suction

Liquids, particularly water, may be transported into concrete by capillary suction or absorption. Water absorption may be expressed by Eq. (5.1-139):

$$w = w_1 \left(t/t_1\right)^n = M_w t^n \qquad (5.1\text{-}139)$$

where

w is the water absorbed per unit area at time t in [m³/m²]

w_1 is the water absorbed at a given time t_1

t is the duration of water absorption in [s]

n = 0.5

M_w = w_1 / t_1^n is the coefficient of water absorption in [m/s$^{0.5}$]

For a rough estimate the coefficient of water absorption for a given concrete strength may be determined from Eq. (5.1-140):

$$M_w = M_{w0} \cdot \frac{1}{f_{cm}^{\,2.5}} \qquad (5.1\text{-}140)$$

where

M_{w0} = 0.2 [m/s$^{0.5}$]

f_{cm} is the mean compressive strength in [MPa] nation of the capillary absorption of water of hardened concrete”, Materials and Structures, Vol. 32, pp. 178-179, 1999.

5.1.13 Properties related to durability

5.1.13.1 General

When considering concrete properties related to durability deterioration models describing the time dependent degradation of concrete are essential.

Indirect degradation of concrete may be caused by

– carbonation-induced corrosion of reinforcing steel,

– chloride-induced corrosion of reinforcing steel.

Direct degradation of concrete may be caused by

– freeze/thaw attack (internal damage, scaling),

– acid action (dissolving action),

reactivity of aggregate (internal damage).

Several models for indirect and direct deterioration are considered in the following clauses.

The durability of structural concrete components in service is determined by the transport of aqueous and gaseous substances in the pore system of concrete and their interaction with the hydrated paste matrix, aggregate or steel reinforcement. The substances may cause degradation and loss of serviceability by their direct action on the concrete microstructure or, indirectly, enable other reactions leading to deterioration.

Some degradation models have found a relatively broad international acceptance. Such models usually contain parameters which need to be quantified for material and environmental effects on the deterioration process and transfer parameters which consider uncertainties resulting from experimental setups. However, operational standards are not available for the quantification of most parameters. Information must therefore be found by measurements with equivalent material or on existing structures and in the literature, for instance in *fib* Bulletin 34 “Model Code for Service Life Design”.

5.1.13.2 Carbonation progress

The propagation of the carbonation front from the concrete surface may be described by Eq. (5.1-141):

$$x_c(t) = \sqrt{2 \cdot k_e \cdot k_c \cdot R_{NAC,0}{}^{-1} \cdot C_S} \cdot \sqrt{t} \cdot W(t) \qquad (5.1\text{-}141)$$

with

$x_c(t)$ carbonation depth at the time t in [mm]

t time in [years]

k_e environmental function [-]

k_c execution transfer parameter [-]

C_S CO_2-concentration in the air in [kg/m³]

$W(t)$ weather function [-]

The exposure of concrete structures to atmospheric CO_2 results in the carbonation of the hydration products accompanied by a reduction in pH of the pore solution which can induce corrosion of the steel reinforcement. The penetration of the carbonation front depends on the concentration of CO_2 in the atmosphere and the amount of hydration products able to react with CO_2. If gas diffusion is assumed, the carbonation depth is proportional to the square root of time (see also sub-clause 5.1.12.2.2).

Eq. (5.1-141) has been developed in the European research project DuraCrete and slightly revised in the research project DARTS:

– The European Union – Brite EuRam III. Modelling of Degradation. DuraCrete, Probabilistic Performance based Durability Design of Concrete Structures, 1998

– DARTS, Durable and Reliable Tunnel Structures. Deterioration Modelling. DARTS R2.1 – May 2004

The inverse effective carbonation resistance $R_{ACC,0}^{-1}$ varies in dependence of water/cement ratio from $3 \cdot 10^{-11}$ to $15 \cdot 10^{-11}$ (m²/s)/(kgCO$_2$/m³) for CEM I, from 0 to $17 \cdot 10^{-11}$ (m²/s)/(kgCO$_2$/m³) for CEM I with fly ash and between $8 \cdot 10^{-11}$ and $80 \cdot 10^{-11}$ (m²/s)/(kgCO$_2$/m³) for CEM III.

Further details may be found in *fib* Bulletin 34 "Model Code for Service Life Design".

The penetration of chlorides (e.g. deicing salt) changes the chemical composition of the pore solution of concrete adjacent to the steel reinforcement causing corrosion to set in. If chloride penetration is diffusion-controlled an error function may be used to describe the penetration profiles.

In the European joint research projects DuraCrete and DARTS a model for the prediction of time and depth dependent chloride content has been developed and validated (see Eq. (5.1-143)).

The chloride migration coefficient $D_{RCM,0}$ varies in dependence of the water/cement ratio from $8 \cdot 10^{-12}$ to $25 \cdot 10^{-12}$ m²/s for CEM I, from $4 \cdot 10^{-12}$ to $15 \cdot 10^{-12}$ m²/s for CEM I with fly ash and between $1 \cdot 10^{-12}$ and $5 \cdot 10^{-12}$ m²/s for CEM III.

$R_{NAC,0}^{-1}$ inverse effective carbonation resistance of concrete in [(mm²/years)/(kg/m³)]

$$R_{NAC,0}^{-1} = k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t \tag{5.1-142}$$

with

$R_{ACC,0}^{-1}$ inverse effective carbonation resistance of dry concrete, determined at a certain time t_0 using the accelerated carbonation test ACC in [(mm²/years)/(kg/m³)]

$R_{NAC,0}^{-1}$ inverse effective carbonation resistance of dry concrete (65 % RH) determined at a certain time t_0 using the normal carbonation test NAC in [(mm²/years)/(kg/m³)]

k_t regression parameter for the test effect of the ACC test [-]

ε_t error term for inaccuracies which occur conditionally when using the ACC test method in [(mm²/years)/(kg/m³)]

5.1.13.3 Ingress of chlorides

The change of the chloride content of concrete exposed to chloride ingress is given by Eq. (5.1-143):

$$C(x,t) = \left(C_0 + \left(C_{s,\Delta x} - C_0 \right) \cdot \left[1 - erf\left(\frac{x - \Delta x}{2 \cdot \sqrt{D_{app,C} \cdot t}} \right) \right] \right). \tag{5.1-143}$$

with

x depth in [m]

C0 initial chloride content of concrete in [wt.-%/c]

CS,Δx chloride content at a depth of Δx in [wt.-%/c]

Δx depth of the convection zone in [m]

$D_{app,C}$ apparent chloride diffusion coefficient in concrete in [m²/s]

$$D_{app,C}(t) = k_e \cdot D_{RCM,0} \cdot k_t \cdot A(t) \tag{5.1-144}$$

with

$D_{RCM,0}$ chloride migration coefficient in [m²/s]

k_e environmental variable [-]

k_t test method variable [-]

$A(t)$ aging function [-]

$$A(t) = \left(\frac{t_0}{t}\right)^a \qquad (5.1\text{-}145)$$

with

t concrete age in [s]

t_0 reference concrete age in [s]

a age exponent in [-]

The exponent a varies in dependence of cement type from 0.30 to 0.65.

Further details may be found in *fib* Bulletin 34 “Model Code for Service Life Design”.

5.1.13.4 Freeze-thaw and freeze-thaw deicing agent degradation

(a) Mechanisms

The degree of internal damage caused by freeze-thaw attack depends on

– material properties determined by concrete composition including porosity, pores size distribution and strength

– the actual service environment, i.e. the conditions at the concrete surface and their variation with time covering relative humidity, surface contact with water and temperature.

– the degree of saturation which varies with time and location in the concrete due to moisture transport by capillary suction, water vapour diffusion together with capillary condensation and water vapour sorption.

When combined with deicing salt, freeze-thaw attack is also affected by material factors such as aggregate type and reactivity. Besides moisture content, environmental factors such as the minimum freezing temperature, the rate of freezing and the cation types in the deicing agent are important.

At present, no validated model exits for the calculation of the resistance of a given concrete in a structural component to the action of frost or frost combined with deicing agents. Current design aims at avoiding damage by the specification of concrete composition for a particular service environment and standard testing methods for resistance to freeze-thaw and freeze-thaw deicing salt action.

The exposure of concrete structural components to subzero temperatures in service can result in internal cracking and thus to a loss of strength due to moisture transport and the expansion of water on freezing.

The deterioration of concrete caused by freeze-thaw attack with deicing agents is related to complex processes associated with physical and chemical changes in the pore solution, binder paste matrix and aggregate. It results in scaling, i.e. external damage.

A service life model to describe the internal damage caused by freeze-thaw-attack was developed by Fagerlund. The model is based on the observation that a critical water saturation degree S_{CR} exists, above which the material is damaged by frost. Below S_{CR} no severe damage occurs.

(b) Models

Models to be included here, though being rather crude, are still under discussion

5.1.13.5 Alkali-aggregate reaction

The chemical reaction between alkalis in the pore solution of concrete and reactive aggregate results in the formation of an expansive alkali silica gel. This leads to deformation and cracking when the internal pressure exceeds the tensile strength of the aggregate and/or the binder paste matrix. Ultimately, degradation and loss of serviceability of the concrete structure occur.

At present no suitable predictive analytical or numerical method exits for durability modelling of concrete behaviour with respect to the alkali-aggregate reaction. Contemporary concrete design aims at the avoidance of AAR (also termed ASR = alkali-silica reaction) which is usually achieved by limiting the alkali content of the cement or the use of non-reactive aggregate. The third method, to guarantee a sufficient low water content, is difficult to achieve in practice.

For further details see: CONTECVET: A Validated Users Manual for Assessing the Residual Service Life of Concrete Structures - Manual for Assessing Structures Affected by ASR, EC Innovation, Programme IN309021, 2001

(a) Damage monitoring

The following methods may be used to predict the future expansion of structures affected by AAR:

a) Monitoring the expansion of cores taken from the structure

b) Monitoring movement of the structure

c) Use of known expansion behaviour of similar concrete under similar exposure conditions

The observed expansion behaviour has to be extrapolated after correcting the data for the effect of restraint.

(b) Models

Models to be included here, though being rather crude, are still under discussion.

5.1.13.6 Degradation by acids

On contact of an aggressive medium with the concrete surface, acid attack proceeds immediately without an initiation period. A corroded surface layer of low mechanical strength forms due to the dissolution of the binder matrix and, if dissolvable, the aggregate particles. The depth of corrosion increases as time passes. The attacking medium may be classified as follows:

a) mineral acids

b) buffer solutions including organic acids, carbonic acid or ammonium salts

The degree of degradation of concrete caused by acid attack is defined by a corrosion depth *d* with respect to the original surface. It comprises the depth of material removed by abrasion and/or crystallization pressure and the depth of corroded material remaining on the concrete surface.

If the loss of surface material is negligible and the strength of the acid is assumed to be constant, the corrosion depth *d* [m] may be estimated from:

$$d = k_c \sqrt{ct} \qquad (5.1\text{-}146)$$

The service life of a structural component is defined by the time needed for the corrosion to reach a given depth.

So far no prediction formula for the constant k_c may be given. This constant should be determined by appropriate experiments.

where

c concentration of acid in [mol/L], see eqs. 5.1-147 or 5.1-148

t contact time in [s]

k_c is a constant

The effect of concrete composition on the corrosion process is given by the constant k_c which includes the effect of cement content and type, additions, w/c ratio and aggregate solubility. For mineral acids c [mol/L] is given by the proton concentration of the acid as calculated from its pH by means of Eq. (5.1-147):

$$c = 10^{-pH} \tag{5.1-147}$$

In case of buffering media it is necessary to know the pH and the total content c_{tot} of acid and acid anions (e.g. acetate and acetic acid), dissolved CO_2 or ammonium:

$$c = \frac{10^{-pH} c_{tot}}{(10^{-pH} + K_s)} \tag{5.1-148}$$

where

K_s dissociation constant in [mol/L]

c_{tot} total content of acid and conjugate base, dissolved CO_2 or ammonium in [mol/L]

So far no prediction formula for the constant k_p may be given. This constant should be determined by appropriate experiments.

If the corroded surface concrete is continuously removed during attack, corrosion proceeds according to:

$$d = k_p t^p \tag{5.1-149}$$

where

k_p is a constant

p is a constant with $0.5 < p \leq 1.0$

5.1.13.7 Leaching progress

The leaching of environmentally relevant substances such as Cr, V, Zn from concrete structural components commences on first contact of the concrete surface with water. The leaching rate is determined by the solubility and

The cumulative leaching of a substance from a given concrete surface area in constant contact with water is given empirically by:

dissolution kinetics of the environmentally relevant substances in the pore solution of concrete and the diffusion of the species through the pore solution to the concrete surface. Availability describes the total amount of a particular substance per cubic metre concrete which can be leached.

Especially on first contact, environmentally relevant substances on the concrete surface enter the water by the wash-off mechanism. The leaching rate depends on the supply of water to the surface and dry periods. Leaching scenarios include the following:

a) constant contact, e.g. ground water on foundations

b) intermittent contact, e.g. seepage water on foundations, rain on facades

c) flowing water, e.g. shotcrete tunnel liners

The size of a structural component limits the total amount of leachable substances. For small sizes, depletion progressively lowers the leaching rate.

The leaching potential of the substance in question may be assessed for a particular concrete composition in terms of the cumulative leaching E_{56} [mol/m^2] obtained after 56 days in a tank test, according to NEN 7345 (standard of the Netherlands).

$$E = k_1(1 - e^{-k_2 t}) + k_3\sqrt{t} + k_4 t \qquad (5.1\text{-}150)$$

where

E cumulative leaching in [mol]

t total contact time in [s]

k_1, k_2, k_3, k_4 constants

The constants k_i are essentially materials constants determined by concrete composition (essentially content of cement and additions, w/c ratio) and the availability of the substances in the concrete.

If wash-off and depletion effects are negligible and the dissolution kinetics of the substances in the pore solution is fast, leaching is controlled by diffusion; so Eq. (5.1-150) simplifies to Eq. (5.1-151):

$$E = k_3\sqrt{t} \qquad (5.1\text{-}151)$$

where

$$k_3 = 2Ac_{mo,0}\sqrt{\frac{D_{eff}}{\pi}} \text{ in [mol/s}^{0.5}] \qquad (5.1\text{-}152)$$

A area of concrete surface in [m^2]

$c_{mo,0}$ initial availability of substance in concrete in [mol/m^3] according to availability test NEN 7341

D_{eff} effective diffusion coefficient of a substance in concrete in [m^2/s]

The effective diffusion coefficient is a materials parameter depending on concrete composition and age. If diffusion-controlled leaching is assumed, D_{eff} can be calculated from the availability test and tank test results using eqs. 5.1-151 and 5.1-152 according to NEN 7345.